

**ISTANBUL TECHNICAL UNIVERSITY★GRADUATE SCHOOL OF**  
**SCIENCE ENGINEERING AND TECHNOLOGY**

**STABILIZATION OF EXPANSIVE SOIL USING COPOLYMER,  
HOMOPOLYMER POLYPROPYLENE, FLY ASH AND LIME**

**M.Sc. THESIS**

**Samad KAZEMIKHOSROWSHAHI**

**Department of Civil Engineering**

**Soil Mechanics and Geotechnical Engineering Programme**

**MAY , 2014**



**ISTANBUL TECHNICAL UNIVERSITY★GRADUATE SCHOOL OF**  
**SCIENCE ENGINEERING AND TECHNOLOGY**

**STABILIZATION OF EXPANSIVE SOIL USING COPOLYMER,  
HOMOPOLYMER POLYPROPYLENE, FLY ASH AND LIME**

**M.Sc. THESIS**

**Samad KAZEMIKHOSROWSHAHI**

**(501101311)**

**Department of Civil Engineering**

**Soil Mechanics and Geotechnical Engineering Programme**

**Thesis Advisor: Assoc.Prof. Aykut ŞENOL**

**MAY , 2014**



**İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ**

**ŞİŞEN ZEMİNİN KOPOLİMER, HOMOPOLİMER POLİPROPİLEN,  
UÇUCU KÜL VE KİREÇ KULLANILARAK STABİLİZASYONU**

**YÜKSEK LİSANS TEZİ**

**Samad KAZEMIKHOSROWSHAHI  
(501101311)**

**İnşaat Mühendisliği Anabilim Dalı**

**Zemin Mekaniği ve Geoteknik Mühendisliği Programı**

**Tez Danışmanı: Doç.Dr. Aykut ŞENOL**

**MAYIS 2014**



**Samad KAZEMIKHOSROWSHAHI**, a **M.Sc.** student of **ITU Graduate School of Science Engineering and Technology** student ID **501101311** successfully defended the **thesis** entitled “**STABILIZATION OF EXPANSIVE SOIL USING COPOLYMER, HOMOPOLYMER POLYPROPYLENE, FLY ASH AND LIME**” which he prepared after fulfilling the requirements specified in the associated legislations, before the jury whose signatures are below.

**Thesis Advisor :**      **Assoc.Prof.Dr. Aykut ŞENOL**      .....  
Istanbul Technical University

**Jury Members :**      **Assoc.Prof.Dr. Ahmet Oğuz Tan**      .....  
Istanbul Technical University

**Prof.Dr. Kutay Özaydın**      .....  
Yıldız Technical University

**Date of Submission : 22 April 2014**

**Date of Defense :    22 May 2014**





*To my parents,*



## **FOREWORD**

I would like to express my deep gratitude to my master thesis advisor, Assoc. Prof. Aykut ŞENOL for his endless support, kindness, encouragement and for his precious advice through this work. I have learned not only academic knowledge and skills from him, but also I found him as a real gentle-hearted one. I would like to thank Assist Prof. Hasan YILDIRIM for his great support and also our laboratory technician İsmail CANTEKİN. I would also like to thank Ehsan ETMINAN for his help and fresh ideas and for his friendship. Also I want to thank my dear friend Feyzullah GÜLŞEN. Finally I want to thank my parents and my family for their moral and financial support and I would like to dedicate this thesis to my parents. They, more than anyone else, have been the best role models I could have hoped for. Any accomplishments of mine are due to their support.

19.04.2014

Samad KAZEMI KH.  
(Civil Engineer)



## TABLE OF CONTENTS

	<u>Page</u>
<b>FOREWORD .....</b>	<b>ix</b>
<b>TABLE OF CONTENTS .....</b>	<b>xi</b>
<b>ABBREVIATIONS.....</b>	<b>xiii</b>
<b>LIST OF TABLES.....</b>	<b>xv</b>
<b>LIST OF FIGURES.....</b>	<b>xvii</b>
<b>SUMMARY .....</b>	<b>xix</b>
<b>ÖZET .....</b>	<b>xxi</b>
<b>1. INTRODUCTION .....</b>	<b>1</b>
<b>2. EXPANSIVE SOILS .....</b>	<b>3</b>
2.1 Concept .....	3
2.2 Distribution of Expansive Soils .....	4
2.3 Identification and Classification of Expansive Soils .....	5
2.3.1 Site characterization .....	5
2.3.2 Expansive clays mineralogy .....	7
2.3.3 Factors influencing shrink-swell potential of soils .....	10
2.3.4 Active zone.....	13
2.3.5 Classification of expansive soils .....	14
2.3.5.1 Classification using engineering index properties .....	15
2.3.5.2 Classification by use of free swell percent.....	20
2.3.5.3 Classification by use of coefficient of linear extensibility (COLE).....	20
2.4 Swelling Mechanism of Expansive Soils .....	22
2.5 Geotechnical Problems Related to Expansive Soils .....	23
<b>3. SWELL PRESSURE PREDICTION AND STABILIZATION TECHNIQUES     OF EXPANSIVE SOILS.....</b>	<b>31</b>
3.1 Swell Pressure and Potential Identification .....	31
3.2 Prediction Methods to Determine the Swell Pressure .....	31
3.2.1 Swell prediction based on odometer tests .....	31
3.2.1.1 Method A .....	31
3.2.1.2 Method B .....	32
3.2.1.3 Method C .....	33
3.2.2 Empirical procedures for swell prediction .....	33
3.3 Stabilization Techniques of Expansive Soils.....	36
3.3.1 Replacement of expansive soils with non-expansive soils.....	36
3.3.2 Sand cushion .....	37
3.3.3 Cohesive non swelling layer (CNS) .....	37
3.3.4 Moisture control .....	37
3.3.5 Surcharge loading.....	37
3.3.6 Pre-wetting .....	38
3.3.7 Reinforcing the soil using Geosynthetic .....	38
3.3.7.1 Definition and classification .....	39
3.3.8 Chemical stabilization .....	41

3.3.8.1 Cement stabilization .....	41
3.3.8.2 Fly ash stabilization .....	42
3.3.8.3 Lime stabilization .....	43
3.3.9 Thermal treatment .....	44
<b>4. DETERMINATION OF SWELLING POTENTIAL OF STABILIZED EXPANSIVE SOILS.....</b>	<b>47</b>
4.1 Materials Used in Laboratory Tests .....	47
4.1.1 Bentonite .....	47
4.1.2 Copolymer fiber .....	49
4.1.3 Homopolymer polypropylene .....	50
4.1.4 Fly ash .....	51
4.1.5 Lime .....	52
4.2 Laboratory Tests .....	54
4.2.1 Sieve analysis and hydrometer test .....	54
4.2.2 Atterberg limits test .....	55
4.2.3 Standard proctor compaction test .....	56
4.2.3.1 Effects of additive materials content on the optimum water content ...	57
4.2.3.2 Effect of additive material content on max. dry unit weight .....	59
4.2.4 One-dimensional odometer swell test .....	61
4.1.1 Plain bentonite .....	62
4.1.2 Copolymer mixtures with bentonite .....	64
4.1.3 Homopolymer polypropylene mixtures with bentonite .....	66
4.1.4 Comparing the effect of copolymer and homopolymer polypropylene on the swelling behavior of bentonite .....	68
4.1.5 Fly ash mixtures with bentonite .....	68
4.1.6 Lime mixtures with bentonite .....	70
<b>5. CONCLUSIONS AND RECOMMENDATIONS.....</b>	<b>73</b>
<b>REFERENCES.....</b>	<b>75</b>
<b>APPENDICES.....</b>	<b>81</b>
APPENDIX A.1 .....	82
APPENDIX A.2 .....	94
<b>CURRICULUM VITAE.....</b>	<b>97</b>

## ABBREVIATIONS

<b>AASHTO</b>	: American Association of State Highway and Transportation Officials
<b>A</b>	: Activity
<b>B</b>	: Bentonite
<b>CH</b>	: High Plasticity Clay
<b>CP</b>	: Copolymer
<b>FA</b>	: Fly Ash
<b>HPP</b>	: Homopolymer Polypropylene
<b>L</b>	: Lime
<b>LL</b>	: Liquid Limit
<b>PL</b>	: Plastic Limit
<b>PI</b>	: Plasticity Index
<b>PI<sub>w</sub></b>	: Weighted Plasticity Index
<b>SL</b>	: Shrinkage Limit
<b>USCS</b>	: Unified Soil Classification System
<b>w</b>	: Water Content
<b>w<sub>opt</sub></b>	: Optimum Water Content
<b>γ<sub>d</sub></b>	: Dry Unit Weight
<b>γ<sub>d max</sub></b>	: Maximum Dry Unit Weight
<b>σ<sub>s</sub></b>	: Swell Pressure





## LIST OF TABLES

	<u>Page</u>
<b>Table 2.1</b> : Characteristics of typical clay minerals .....	8
<b>Table 2.2</b> : Specific surface area of clay minerals .....	8
<b>Table 2.3</b> : Typical values of activities for clay minerals .....	9
<b>Table 2.4</b> : Soil properties that influence shrink-swell potential .....	10
<b>Table 2.5</b> : Environmental conditions influence shrink-swell potential .....	11
<b>Table 2.6</b> : Soil expansion prediction by liquid limit.....	15
<b>Table 2.7</b> : Soil expansion prediction by plasticity index. ....	16
<b>Table 2.8</b> : Expansive soil classification based on shrinkage limit.....	17
<b>Table 2.9</b> : Classification based on percent finer than no. 200 sieve, liquid limit, and standard penetration resistance for Rocky mountain soils .....	17
<b>Table 2.10</b> : Classification of expansive soils using COLE factor .....	22
<b>Table 3.1</b> : Classification used to PE value.....	34
<b>Table 3.2</b> : Prediction the percent swell, $S_p$ .....	35
<b>Table 3.3</b> : Cement requirement by volume for an effective stabilization of various soils.....	42
<b>Table 4.1</b> : Physical properties of bentonite.....	48
<b>Table 4.2</b> : Chemical properties of bentonite .....	48
<b>Table 4.3</b> : Physical properties of copolymer .....	49
<b>Table 4.4</b> : Physical properties of homopolymer polypropylene .....	50
<b>Table 4.5</b> : Chemical properties of fly ash .....	51
<b>Table 4.6</b> : Physical properties of fly ash. ....	52
<b>Table 4.7</b> : Chemical properties of lime.....	53
<b>Table 4.8</b> : Geotechnical properties of the bentonite. ....	63
<b>Table 4.9</b> : Results of standard proctor compaction test for bentonite-copolymer ...	65
<b>Table 4.10</b> : Results of standard proctor compaction test for bentonite-homopolymer polypropylene .....	67
<b>Table 4.11</b> : Results of standard proctor test for bentonite-fly ash mixtures. ....	69
<b>Table 4.12</b> : Results of standard proctor compaction test for bentonite-lime .....	71



## LIST OF FIGURES

	<u>Page</u>
<b>Figure 2.1</b> : Identified expansive soils regions in the world .....	4
<b>Figure 2.2</b> : Flow chart for site characterization .....	6
<b>Figure 2.3</b> : Electron micrograph of clay minerals .....	7
<b>Figure 2.4</b> : Seasonal moisture content variation in soil profile .....	13
<b>Figure 2.5</b> : Typical plots of moisture content versus depth for exploratory borings in expansive soils .....	14
<b>Figure 2.6</b> : Atterberg limits description, volume change and generalized stress-strain response of expansive soils .....	16
<b>Figure 2.7</b> : Classification chart based on plasticity index and liquid limit.....	18
<b>Figure 2.8</b> : Classification chart for compacted clays based on activity and percent clay .....	18
<b>Figure 2.9</b> : Classification chart based on plasticity index and clay content .....	19
<b>Figure 2.10</b> : Classification chart based on weighted plasticity index.....	19
<b>Figure 2.11</b> : Phase diagram of free swell.....	20
<b>Figure 2.12</b> : Expansion potential as a function of colloids and COLE .....	22
<b>Figure 2.13</b> : Moisture inclusion in clay which causes swelling .....	23
<b>Figure 2.14</b> : Typical damages due to expansive soils.....	24
<b>Figure 2.15</b> : crack patterns and heaving resulting from expansive soils on light buildings .....	25
<b>Figure 2.16</b> : Typical crack pattern on a floor due to center heave .....	26
<b>Figure 2.17</b> : Decreased usefulness of deformation.....	27
<b>Figure 2.18</b> : Road damages due to the expansive soils .....	28
<b>Figure 2.19</b> : Crack types associated with expansive clays .....	29
<b>Figure 3.1</b> : Deformation versus vertical stress, Method A .....	32
<b>Figure 3.2</b> : Deformation versus vertical stress, Single-Point, Method B .....	32
<b>Figure 3.3</b> : Deformation versus vertical stress, Loading-after-Wetting, Method C .....	33
<b>Figure 3.4</b> : Relationship to use in Van der Merwes' empirical heave prediction method.....	34
<b>Figure 3.5</b> : Types of geosynthetics applicable in geotechnical engineering. ....	40
<b>Figure 3.6</b> : Mixing and shaping of fly ash stabilized soil.....	43
<b>Figure 3.7</b> : Lime stabilization in a road construction .....	43
<b>Figure 3.8</b> : Scheme for deep thermal treatment of soil.....	45
<b>Figure 4.1</b> : The bentonite used in the laboratory tests. ....	47
<b>Figure 4.2</b> : Grain-size distribution of Bentonite. ....	48
<b>Figure 4.3</b> : Copolymer fibers.....	49
<b>Figure 4.4</b> : Homopolymer polypropylene fibers .....	50
<b>Figure 4.5</b> : The Fly ash used in the laboratory tests .....	51
<b>Figure 4.6</b> : Grain-size distribution of fly ash.....	52
<b>Figure 4.7</b> : The lime used in the laboratory tests.....	53
<b>Figure 4.8</b> : Grain-size distribution of lime. ....	53
<b>Figure 4.9</b> : Hydrometer analysis of bentonite and lime.....	54

<b>Figure 4.10 :</b> Atterberg limits of bentonite – fly ash mixtures.....	55
<b>Figure 4.11 :</b> Atterberg limits of bentonite – lime mixtures.....	55
<b>Figure 4.12 :</b> Standard proctor compaction curve. ....	56
<b>Figure 4.13 :</b> Effect of copolymer content on the optimum water content of bentonite-copolymer mixtures. ....	57
<b>Figure 4.14 :</b> Effect of homopolymer polypropylene on the optimum water content of bentonite-homopolymer polypropylene mixtures. ....	58
<b>Figure 4.15 :</b> Effect of fly ash content on the optimum water content of bentonite-fly ash mixtures. ....	58
<b>Figure 4.16 :</b> Effect of lime content on the optimum water content of bentonite-lime mixtures. ....	59
<b>Figure 4.17 :</b> Effect of copolymer content on max. dry unit weight of bentonite. ....	59
<b>Figure 4.18 :</b> Effect of homopolymer polypropylene content on max. dry unit weight of bentonite. ....	60
<b>Figure 4.19 :</b> Effect of fly ash content on max. dry unit weight of bentonite. ....	60
<b>Figure 4.20 :</b> Effect of lime content on max. dry unit weight of bentonite. ....	61
<b>Figure 4.21 :</b> Bentonite-copolymer sample prepared for the swell test and One-dimensional swell test setup. ....	62
<b>Figure 4.22 :</b> Atterberg limit tests .....	63
<b>Figure 4.23 :</b> Liquid limit test for plain bentonite. ....	63
<b>Figure 4.24 :</b> Standard proctor compaction test of bentonite-copolymer mixtures...	64
<b>Figure 4.25 :</b> Results of one-dimensional swell test for bentonite-copolymer mixtures. ....	65
<b>Figure 4.26 :</b> Standard proctor compaction test for bentonite-homopolymer polypropylene mixtures. ....	66
<b>Figure 4.27 :</b> Results of one-dimensional swell test for bentonite-homopolymer polypropylene mixtures. ....	67
<b>Figure 4.28 :</b> One-dimensional swell test of bentonite mixtures with copolymer and homopolymer polypropylene. ....	68
<b>Figure 4.29 :</b> Standard proctor compaction test of bentonite-fly ash mixtures. ....	69
<b>Figure 4.30 :</b> One-dimensional swell test of bentonite-fly ash mixtures. ....	70
<b>Figure 4.31 :</b> Standard proctor compaction test of bentonite-lime mixtures.....	70
<b>Figure 4.32 :</b> One-dimensional swell test of bentonite-lime mixtures .....	71
<b>Figure 4.33 :</b> The effects of four different stabilizers on the swell pressure of bentonite.....	71

## **STABILIZATION OF EXPANSIVE SOIL USING COPOLYMER, HOMOPOLYMER POLYPROPYLENE, FLY ASH AND LIME**

### **SUMMARY**

Expansive soils, also called as shrink-swell or black cotton soil, are one of the widespread typical problematic soils in the world. They are defined by their volume changes due to variation in the water content which results in large damages to the structures which are based on or in contact with them. The swelling potential of expansive soils is related to three major factors: geology, engineering properties of the soil, and local environmental conditions. Geology primarily indicate the presence of expansive clay minerals in soil. Engineering properties of soil include water content, plasticity and dry density. The most important local environmental conditions to consider are the activity factor which indicates amount of the clay fraction in the soil, initial water content, and confining pressure.

According to the geotechnical literature, annual costs of damages related to the expansive soils are estimated at several billions of dollars worldwide. The constructions on the expansive soils without considering the shrink-swell potential of subgrade soil layer can be result in serious damages to the buildings, the pavements, the high ways, the railways, the retaining walls, the pipe lines, and also causes some problems such as slope failure. According to the seasonal and the climatic state and drainage condition, wetting and drying of expansive soil cause to swell or to shrink in the soil mass respectively.

The aim of this study is to investigate the effects of four different synthetic fibers and chemical materials on the swelling potential of expansive soil. Sodium bentonite obtained from Ankara region was used as the expansive soil. Four types of additional materials including copolymer (CP), homopolymer polypropylene (HPP), fly ash (FA) and lime (L) were evaluated as the potential stabilizers to decrease the swelling pressure of bentonite. Depending on the type of additional materials, they were blended with bentonite in different percentages to assess optimum state. As a part of research evaluation, a series of laboratory experimental program was performed. The Laboratory testing program was performed in the Istanbul Technical University's Prof. Dr. Hamdi Peynircioğlu Soil Mechanics Laboratory. The index properties of bentonite and also compaction parameters of bentonite and bentonite-additive mixtures were determined. According to the Atterberg limits test, fly ash inclusion cause reduction in liquid limit and plastic limit. The plasticity index also decreases. In the case of lime, both liquid limit and plastic limit decrease which cause a reduction in plasticity index. The soil becomes more workable by reduction of plastic properties and due to formation of cementing material. In the case of standard proctor compaction test, there is not any remarkable changes in the optimum water content of the bentonite mixtures with copolymer, homopolymer polypropylene and lime, but in the case of fly ash, the variation of water content approaches to 8%. According to the C method of ASTM- D4546 standard, Odometer swell test was used

to measure swell pressures. The method is based on the regular procedure of loading after wetting of the samples. In this method, the sample was provided at the optimum water content and the swelling pressure of the sample was measured by adding weight and keeping dial gage in the zero. The constant weight which keeps the dial gage at zero for 24 hours indicates the swelling pressure of the sample. Initially, the swell pressure of plain bentonite was determined and then it was compared with the swell pressures which were obtained from bentonite-additive materials mixtures prepared in difference percentages. The results approve that bentonite-fiber mixtures are effective significantly and decrease the swelling pressure up to 68%. All data were analyzed considering the improved values to determine the optimum state.

## **ŞİŞEN ZEMİNİN KOPOLİMER, HOMOPOLİMER POLİPROPİLEN, UÇUCU KÜL VE KİREÇ KULLANILARAK STABİLİZASYONU**

### **ÖZET**

Büzülen-şişen da adlandırılan şişen zeminler dünyada yaygın olarak bulunan sorunlu bir zemindir. Su muhtevastaki çeşitliliklere bağlı olarak değişen ve üzerinde bulunan veya temas halinde bulunduğu yapılarda büyük hasarlarla sonuçlanan hacime göre belirlenirler. Şişen zeminlerin şişme potansiyeli üç temel faktörle bağlantılıdır; jeoloji, zeminin teknik özellikleri ve yerel çevresel şartlar. Jeoloji ilk olarak zeminde şişen kil minerallerinin varlığını belirler. Zeminin teknik özellikleri su muhtevası, plastisite ve kuru yoğunluğu içerir. Göz önünde bulundurulması gereken en önemli yerel çevresel koşullar zeminde kil parçacıklarının varlığına işaret eden hareket faktörü, başlangıçtaki su muhtevası ve hapsolmuş basınçtır. Geoteknik literatürüne göre dünya çapında şişen zeminlere bağlı hasarların yıllık bedeli birkaç milyar dolardır.

Zeminin büzülme-şişme potansiyeli ve zeminaltı toprak tabakası göz önünde bulundurulmadan şişen zeminler üzerinde inşaat yapılması yapılara, kaldırımlara, otoyollara, demiryollarına, istinad duvarlarına, boru hatlarına, şev kaymasına vs ciddi oranda hasar verebilir. Mevsimsel ve iklimsel durum ve drenaj koşullarına bağlı olarak, şişen zeminin ıslanması zemin kütlesinin şişmesine, kuruması ise büzülmesine neden olur.

Şişen zeminlerin şişme potansiyelinin önceden tahmin edilmesi uygun iyileştirme uygulamalarının ve problemlili killerin büzülme-şişme davranışlarının neden olabileceği beklenmedik hasarların önlenmesini sağlayan tasarım kriterlerinin kullanılmasını sağlar. Ayrıca labotatuvar test sistemlerine dayanan birkaç tahmin metodu ve gözleme dayalı bazı yaklaşımlar da şişen killerin davranışını anlayabilmek için geliştirilmiştir. Laboratuvar tabanlı metodlar kesinliğine ve güvenilir sonuçlar sağlamasına rağmen gözleme dayalı metodlara kıyasla daha maliyetlidir ve daha çok zaman alır.

Genellikle verimsiz zemin koşulları fazla gözenek su basıncına veya dayanıklılık olmamasına bağlanır ve deformasyonlar ve çökmelerle bağlantılıdır, fakat şişen zeminlerde emiş gücü ve büzülme-şişme potansiyeli dikkat çekicidir. Bu nedenle zemin iyileştirilmesinin temel amacı bahsedilen bu sorunları düzeltmektir. Zemin geliştirme prosedüründe kullanılan metodda teknik performansın yanında maddi ve çevresel etkenler de göz önünde bulundurulmalıdır. Genellikle zemin geliştirme teknikleri kesme kuvvetini arttırmak, geçirgenliği azaltmak ve zemin kompresibilitesini azaltmaktan oluşur. Şişen zeminlerde büzülme-şişme potansiyeli geliştirilmesi gereken en önemli sorundur.

Şişen zemin stabilizasyon teknikleri çevresel koşullar, aktif bölge derinliği, zemin tabakasının şişme potansiyeli, mevcut ekipman, materyaller ve geliştirme metodunun maliyetine göre değişkenlik gösterir. Zemin geliştirme prosedüründe yalnızca genişleme potansiyelini azaltmak değil, geoteknik bir yapıyı tasarlamak için gerekli olan temel faktörler de göz önünde bulundurulmalıdır.

Bu metodlar üç temel başlıkta kategorize edilebilir;

- I) Nem kontrolü veya kompaksiyon gibi, zemin üzerinde işlem gerektiren teknikler.
- II) Çeşitli güçlendirme metodları gibi, zeminin mekanik özelliklerini değiştirmek için dış malzemelerin kullanılmasına dayanan metodlar.
- III) Zeminin yapısını değiştiren kimyasal katkı maddeleri kullanılarak yapılan geliştirme teknikleri.

Şişen zeminlerin bazı stabilizasyon teknikleri zemin değiştirilmesi, kum yastığı, bağlayıcı şişmeyen tabaka, nem kontrolü, ek yükleme, önceden ıslatma, zemin güçlendirmesi, ısıtma işlemi ve çimento stabilizasyonu, uçucu kül stabilizasyonu ve kireç stabilizasyonu içeren bazı kimyasal metodlar olarak sıralanabilir.

Şişen zeminlerin şişme potansiyelini azaltmak amacıyla birçok projede kimyasal maddeler başarıyla kullanılmıştır. Termal enerji santrallerinde kömür yakılmasıyla ortaya çıkan endüstriyel bir yan ürün olan uçucu kül birçok ülkede büyük miktarda üretilen bir atık maddesidir. Zemin ve yol stabilizasyonunda uçucu kül kullanımı dünyada yaygın olarak kullanılan bir uygulamadır ve birçok durumda tatmin edici sonuçlar elde edilmiştir. Şişen zemine uçucu kül eklenmesiyle katyon değişimi işlemi esneklik, hareketlilik ve şişme potansiyeli azalmasıyla sonuçlanır.

Yol altyapısı inşaatlarında kireç kullanıma uygun bir maddedir. Güçsüz zeminlere kireç uygulaması gelişmiş bir yöntemdir ve zeminin güç ve sertlik özelliklerini başarılı bir şekilde arttırmak için yıllardan beri kullanılmaktadır. Ekonomik ve çevresel etkileri en aza indirmesinden dolayı kireç zemin stabilizasyonunda uygun bir alternatiftir. Kireç ve kilin minerallerinin puzolanik tepkimesi toprağın çimentolaşmasıyla ve böylece daha dayanıklı hale gelmesiyle sonuçlanır. Zamanla su muhtevası ve hava sıcaklığı gibi çevresel etkenlere bağlı olarak zeminin dayanıklılığı artar. Genel olarak kireç stabilizasyonu zemindeki dayanıklılığı ve deformasyonun modül şeklini artırır ve şişme potansiyelini azaltır.

Araştırmanın bir parçası olarak büzülme-şişme potansiyelinin gelişimini hesaplamak için gelişigüzel dağıtılmış lifler kullanılarak zemin güçlendirmesi yapılmıştır. Polipropilen dünya çapında laboratuvarlarda zemin güçlendirmesi testinde yaygın olarak kullanılır. Polipropilen lif güçlendirme testi toprağın serbest kompresif dayanıklılığını geliştirmiş ve şişen killerin hacimsel bükülme deformasyonunu ve şişme basıncını azaltmıştır.

Bu çalışmanın amacı dört farklı sentetik lif ve kimsayal maddenin şişen zeminin şişme potansiyeline olan etkilerini araştırmaktır. Şişen zemin olarak Ankara bölgesinden elde edilen sodyum bentonit kullanılmıştır. Bentonitin şişme basıncını azaltmak için olası stabilizatörler olarak kopolimer, homopolimer polipropilen , uçucu kül ve kireç olmak üzere dört ek madde değerlendirilmiştir. Optimum durumu hesaplamak için ek



maddeler türlerine bağı olarak bentonitle farklı oranlarda karıştırılmıştır. Araştırma değerlendirmesinin bir parçası olarak bir takım laboratuvar deney programı uygulanmıştır. Laboratuvar testleri İstanbul Teknik Üniversitesi, Prof. Dr. Hamdi Peynircioğlu Zemin Mekaniği Laboratuvarı'nda uygulanmıştır. Bentonitin içerik özellikleri ve bentonit ve bentonit katkı karışımların kompaksiyon parametreleri belirlenmiştir. Bu çalışmada bentonitin ve uçucu kül ve kireç içeren iki kimyasal katkı stabilizatörün Atterberg limitleri ASTM-D4318 standardına göre incelenmiştir. Elde edilen sonuçlara göre uçucu külün dahil olduğu durumda likit limit ve plastik limitte azalma olmuştur. Ayrıca plastisite indeksi de düşmüştür. Kirecin kullanıldığı durumda likit limit de plastik limit de düşmüştür ve plastisite indeksinin de azalmasına neden olmuştur. Zemin, plastik özelliklerini kaybetmesiyle ve katılaştırıcı maddenin oluşumuyla daha işlenebilir bir hale gelmiştir. Katkı maddelerinin bentonitin kompaksiyon parametreleri üzerindeki etkisi hesaplanmıştır. Kopolimer, homopolimer polipropilen ve kireç içeren bentonit karışımlarının optimum su muhtevasında kayda değer bir değişim gözlenmezken uçucu kül içeren karışımın su muhtevası varyasyonu %8'e yaklaşmıştır.

ASTM-D4546 standardının C metoduna göre şişme basıncını ölçmek için ödometre şişme testi uygulanmıştır. Bu metod örneklerin ıslatıldıktan sonra ölçülmesi prosedürüne dayalıdır. Bu metotta optimum su muhtevasına sahip bir numune kullanılmıştır ve numunenin şişme basıncı ağırlık eklenerek ve gösterge sıfırda tutularak ölçülmüştür. Göstergeyi 24 saat boyunca sıfırda tutan sabit ağırlık numunenin şişme basıncını belirtir. Önce sade bentonitin şişme basıncı belirlenmiş daha sonra farklı oranlarda hazırlanmış bentonit katkı karışımların şişme basınçlarıyla karşılaştırılmıştır. Şişme testi için kullanılan tüm numuneler standart Proktor kompaksiyon aparatıyla optimum su seviyesine getirilmiştir.

Elde edilen sonuçlar bentonit-lif karışımların önemli ölçüde etkili olduğunu ve şişme basıncını %68'e kadar azalttığını doğrulamaktadır. Tüm veriler optimum hali belirlemek için geliştirilmiş değerler göz önüne alınarak analiz edilmiştir.



## **1. INTRODUCTION**

Safety and the strength of any structure besides structure design engineering, basically depends on the soil and subgrade which is built on. Nowadays, increase of urban societies and extension of roadway and railway networks, makes the optimized utilization of engineering methods necessary for a safe and economical construction. Lack of proper soil and suitable subgrade condition for construction is one of the major limitative items to define road paths or projects locating. The simplest solution is to remove poor soil and replace it with proper engineering materials but it is not possible in all cases. Also the soil replacement brings excessive costs for the engineering projects. The in-situ soil treatment is conventional approach and progress in the geotechnical engineering and the diverse methods of soil treatments help to solve these problems and make it possible to construct optimized structures. Generally poor soil conditions are attributable to excess pore water pressure or lack of strength and associated with the deformations and the settlements but in the expansive soils the suction and shrink-swell potential are also remarkable. Thus the major aim of soil treatment is to reclaim the mentioned problems. Not only expected engineering performance but also cost and environmental aspects of the utilized method should be considered in a soil improvement procedure.

The chemical materials have been used successfully on many projects in order to reduce the swelling potential of expansive soils. Fly ash is a waste material is generating in large quantities in many countries. It is an industrial by-product from burning coal at thermal power plants. The stabilization of soils and pavements with the coal fly ash is a widespread method in the world with acceptable results in many cases. Fly ash is non-plastic fine silt with various formations depended on the coal burned. The annual quantity of fly ash generated worldwide exceeds 600 million tons while in Turkey it is approximately 13 million tons produced in 11 power plants. Considering excess amount of produced fly ash and its low cost, it can be utilized as an alternative material in the geotechnical engineering and the construction of geoenvironmental infrastructures (Şenol et al., 2003).

Lime is an appropriate material for road infrastructures constructions. Lime treatment of poor soils is an advanced method and already is being used for many years to enhance soil strength and stiffness properties successfully. Minimizing both the economical and the environmental effect makes lime as a suitable alternative to soil stabilization (Bernardo et al., 2012).

As a part of the research, soil reinforcement was performed by using randomly distributed fibers to evaluate shrink-swell potential improvement. Polypropylene widely used in laboratory testing of soil reinforcement worldwide. Polypropylene fiber reinforcement improved the unconfined compressive strength of soil and reduced both volumetric shrinkage strains and swell pressures of the expansive clays (Puppala and Musenda 2001). With respect to ASTM C-1116 “Standard Specification for fiber reinforced concrete and shotcrete” homopolymer polypropylene is used in concrete applications in order to prevent concrete cracking caused by plastic and settlement shrinkage that occurs prior to initial state.

The purpose of this research is to decrease the swelling potential of an expansive soil using alternative additive materials. A high plasticity bentonite is used as the expansive soil. The fly ash and the lime as the chemical materials, and the copolymer and the homopolymer polypropylene as the synthetic fibers mixed with bentonite to evaluate engineering properties of the treated soil. For classifying and determining engineering properties of mentioned admixtures such as the Atterberg limits, the compaction parameters and the swelling pressure, the related experiments were performed. Each four alternative materials was mixed with bentonite in different dosages and the rate of improvement was measured by one-dimensional swelling test. All data were compared considering decreased swelling potential in order to get a general rule for effect of mentioned materials usage on the stabilization of expansive soils.

## **2. EXPANSIVE SOILS**

### **2.1 Concept**

Expansive soils, also called as shrink-swell soil, are one of the widespread typical problematic soils in the world. The expansive soils are defined by their volume changes due to variation in water content. Environmental and the seasonal water changes cause large amounts of swell and shrinkage movements in these soils. These large movements cause serious damages in many structures are built on the expansive soils. The annual cost of damage is estimated at several billions of dollars worldwide (Nelson and Miller, 1992). Geotechnical engineers did not recognize damages associated with buildings on expansive soils until the late 1930. The U.S. Bureau of Reclamation made the first recorded observation about soil heaving in 1938 (Chen, 1988). Since then a number of investigations have done to determine the expansive soils properties and innovate the improvement methods.

As a part of developed research in the expansive soils, the potential problems to design of shallow foundations to support lightweight structure on the expansive soils is more critical than the design of foundations for heavy loads (Meehan and Karp, 1994). The traditional design criteria of considering bearing capacity proves the failure in expansive soils (Lucian, 2006).

The swelling potential of expansive soils is related to three major factors: geology, engineering properties of the soil, and local environmental conditions. Geology primarily indicate the presence of expansive clay minerals in soil. Engineering properties of soil include water content, plasticity and dry density. The most important local environmental conditions to consider are the activity factor which indicates amount of the clay fraction in the soil, initial water content, and confining pressure (Sabtan, 2005).

For every construction such as buildings, highways, water channel and pipeline projects, pavement systems, airport slabs and etc., it is essential they will be built on a proper soil and subgrade. Last decades, many researches were done to innovate new

methods for improving the engineering properties of problematic soils. Generally, the soil improvement techniques consist of enhancing shear strength, reduction of permeability and reduction of soil compressibility. In expansive soils, the shrink-swell potential is the main problem which should be improved. The stabilization methods of expansive soils can be categorized in three major divisions consisting:

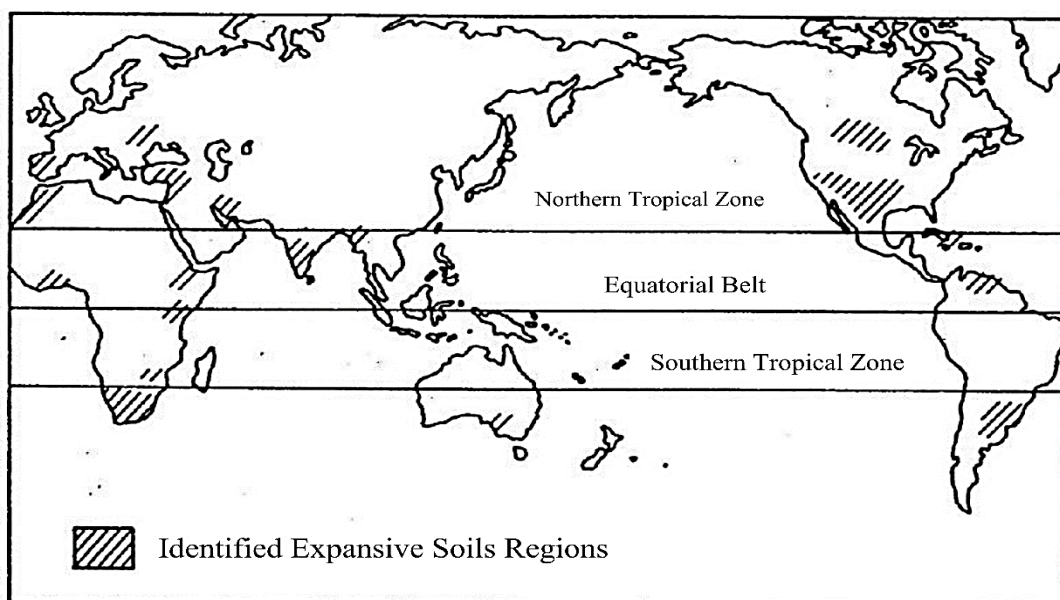
I) The techniques which involve work on the soil such as moisture control or compaction.

II) The methods which based on using external materials to change mechanical properties of soil such as various reinforcement methods.

III) The improvement techniques which change the nature of soil using chemical additives.

## 2.2 Distribution of Expansive Soils

The expansive soils are found extensively in the arid and the semi-arid regions of the world. The presence of expansive soils dramatically affects the construction activities in many parts of south-western the United States, South America, Canada, Africa, Australia, Europe, India, China and the Middle East. By the extension of constructional activities, more expansive soil regions are being discovered every year, especially in underdeveloped countries (Chen, 1975). Figure 2.1 shows the identified expansive soils regions in the world.



**Figure 2.1 :** Identified expansive soils regions in the world (Chen, 1975).

## **2.3 Identification and Classification of Expansive Soils**

### **2.3.1 Site characterization**

During the geotechnical investigation, the subsoil profile and the engineering properties of subgrade materials should be investigated. The common soil site investigation practices for non-expansive soils do not have generally sufficient data to determine shrink-swell potential of the soil. Hence in presence of expansive soils, the effects of environmental conditions such as the active zone depth, the seasonal rainfall and the evaporation which cause moisture changes on swell potential of the soil, should be evaluated. For the characterization of a site for the building or the highway where are constructed on the soils with shrink-swell potential, two major factors must be identified:

- Shrink-swell properties of the soil
- Environmental conditions that cause changes on the water content of the soil

Figure 2.2 illustrates the flow chart that Nelson and Miller have suggested for site characterization of expansive soils consisting of three main steps:

- I) Reconnaissance.
- II) Preliminary investigation.
- III) Detailed investigation.

The reconnaissance survey includes the use of maps, field or regional observations and historical evidences of potential problems in the areas. The data obtained from the first step, are used for the determination of scope of the preliminary investigation.

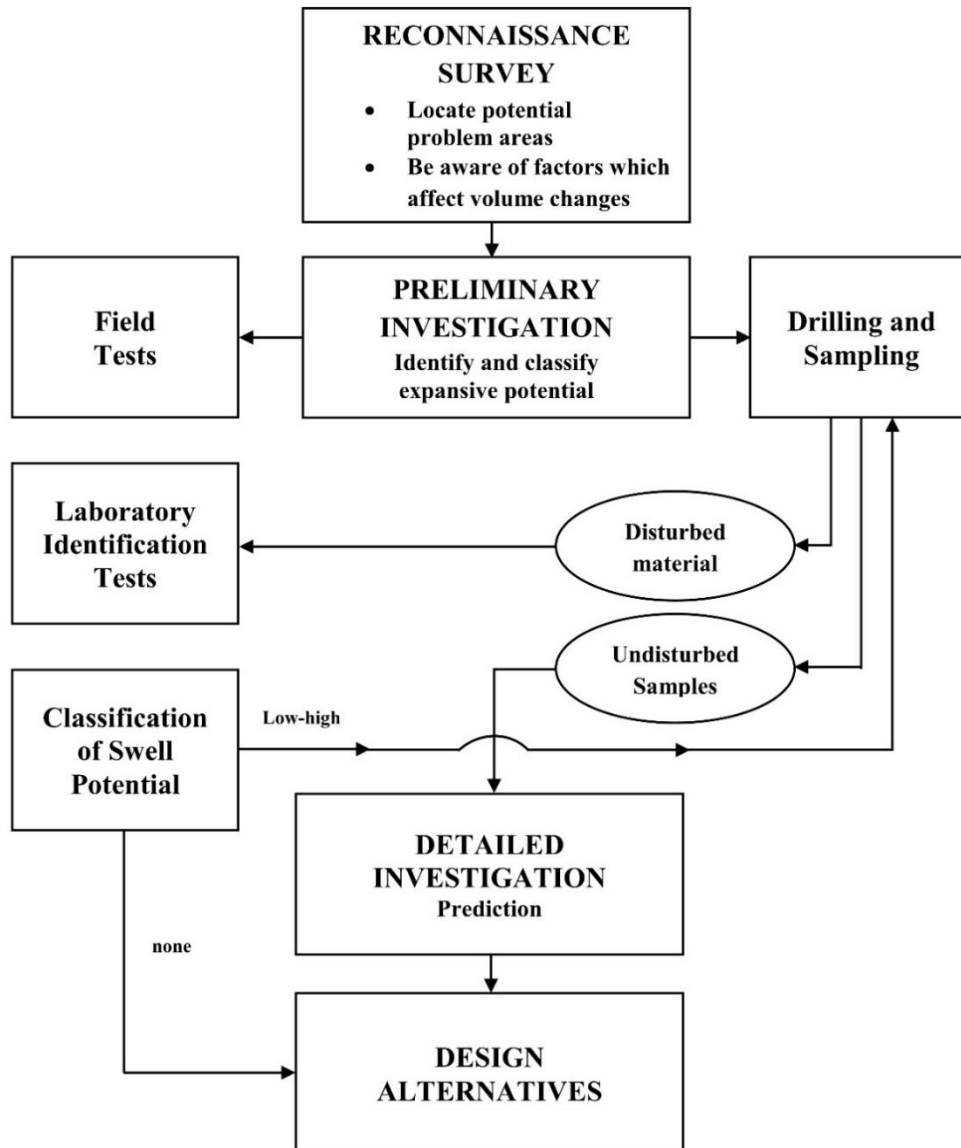
The existence of shrink-swell potential for the soil at the site, is explored in preliminary investigation. It may involves some primary subsurface sampling and related laboratory testing and analysis. For more reliable results, soil which is sampling for tests should be undisturbed as it is possible. The subsurface profile should be defined accurately and soil should be identified and classified. Shrink-swell potential could be investigated according to the various classification methods.

The site characterization program should be design flexible for using the data gathered during the investigation. If, it is necessary to modify the program to achieve more reliable results. Generally, the three main aspects of site characterization include:

- I) The soil environmental factors that influence shrink-swell behavior.

II) The site exploration process, consisting of drilling and sampling methods.

III) The various techniques available in the field to identify expansive soils during preliminary investigation.



**Figure 2.2 :** Flow chart for site characterization (Nelson and Miller, 1992).

The detailed investigation illustrates the soil profile as closely as possible and includes determination of soil properties and measurement of swelling and shrinking potential of soil at the site. The quantitative site evaluation needs relatively undisturbed soil samples for the prediction testing system. Despite the difficulty and high costs of obtaining undisturbed samples, but with skillful and experienced drilling crew and high quality field control, samples with acceptable quality can be obtained by classic sampling procedures.

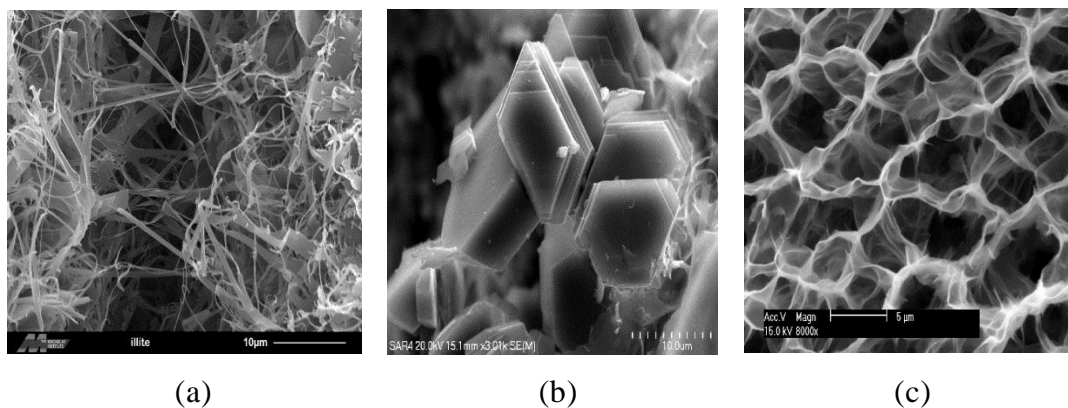


### 2.3.2 Expansive clays mineralogy

The clay mineralogy is a fundamental factor controlling expansive soil behavior. The clay minerals have different swelling potential due to the variation of the electrical field associated with each mineral. The swelling potential of an entire soil mass depends on the quantity and type of clay minerals in the soil, the formation and specific surface area of the clay particles, and the chemical properties of the water which has been absorbed by clay particles. The clay minerals can be categorized generally in three main types consisting of kaolinite, illite and montmorillonite. Figure 2.3 shows an electron micrograph of typical clay minerals. Montmorillonite is a member of the smectite family and is the main constituent to product bentonite. In terms of the chemical structure, montmorillonite could be separated in two types as Na-montmorillonite and Ca-montmorillonite. As it is mentioned in Table 2.3, Na-montmorillonite is more active and capable for expansion than the Ca- montmorillonite. Soil containing montmorillonite is usually white, grey, buff, bright pink or pink but may have colors of yellow, pink, or blue (Day, 1999 ). The engineering properties of three main structural groups of clay minerals can be state as:

- Kaolinite group → Generally nonexpansive.
- Mica-like group → Includes illites and vermiculites, which are capable to expansion, but generally don't cause significant problems.
- Smectite group → Includes montmorillonites, which are highly expansive and are the most problematic clay minerals.

The particle features and the engineering properties of three main clay minerals are described in Table 2.1.



**Figure 2.3 :** Electron micrograph of clay minerals; (a) illite, (b) kaolinite, (c) montmorillonite (<http://www.minersoc.org>).

**Table 2.1 :** Characteristics of typical clay minerals: (a) Skempton, 1953; (b) Mitchell, 1976.

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

The specific surface area of clay minerals is one of the fundamental factors affecting swelling, colloidal, and rheological properties and it is defined as the total surface including particle external surface and the area of interlayer faces. The specific surface area could be estimated either from the adsorption isothermes such as BET method (Brunauer et al., 1938) or color absorption technique (Gungor and Tulun, 1996). Table 2.2 indicates a comparison of specific surface of clay minerals obtained from two different theoretical and experimental methods.

**Table 2.2 :** Specific surface area of clay minerals: Data in  $\text{m}^2/\text{g}$ . I-theoretical data (Moorlock and Highley, 1991); II- experimental data (Gilchrist et al., 1993).

Type of clay mineral	I	II
Na – montmorillonite	–	727
Ca – montmorillonite	800	733
Illite	150	84
Kaolinite	50	219

In 1953, Skempton defined a term called activity that indicates potential of fine-graded soils for expansion. The activity is a combination of Atterberg limits and the clay content in the soil mass.

$$\text{Activity (A)} = \frac{\text{Plasticity index}}{\text{Percent by weight finer than } 2\mu\text{m}} \quad (2.1)$$

According to the Skempton, considering the term of activity, the clays are classified to three classes:

- Inactive, for activities less than 0.75.
- Normal, for activities between 0.75 and 1.25.
- Active, for activities greater than 1.25.

The active clays provide the most potential for expansion. Table 2.3 shows typical values of activities for various clay minerals. As it is shown in Table 2.3, montmorillonite is the only active mineral of clay and can be mentioned as the main factor for expansion potential of clays.

**Table 2.3 :** Typical values of activities for clay minerals (Skempton, 1953).

<b>Mineral</b>	<b>Activity (A)</b>
Kaolinite	0.33 to 0.46
Illite	0.9
Montmorillonite (Ca)	1.5
Montmorillonite (Na)	7.2

There are various methods to identify clay minerals. With respect to the similarity between wavelength of X-Ray and atomic plane spacing of clay crystals, the X-Ray diffraction is the most popular and well suited technique for identification of clay minerals. Differential thermal analysis and electron microscopy are other two popular methods to determine mineral formation of clays. Some other mineralogical methods include chemical analysis, infrared spectroscopy, and color adsorption (Grim, 1968). Also radio frequency electrical dispersion has been used to evaluate mineral structure of clays (Basu and Arulanandan, 1973).

### 2.3.3 Factors influencing shrink-swell potential of soils

The shrink-swell potential of expansive soils is depends on many factors. The soil minerals and the water content are two main items which influence the expansive potential of the soil mass. However many of the factors influencing mechanism of swelling also affected by the physical soil properties such as plasticity or density. The factors influencing the shrink-swell potential of a soil can be categorized in three various groups, the soil characteristics that influence the nature of the internal force field, the environmental factors which influence the changes may occur in the internal force system, and the state of stress. The environmental conditions includes three main factors as the initial moisture content, the moisture variations, and the stress conditions. The soil properties and the environmental factors which affect the expansive shrink-swell potential of the soil are described in Tables 2.4 and 2.5 respectively.

**Table 2.4 :** Soil properties that influence shrink-swell potential  
(Nelson and Miller, 1992).

<b>Factor</b>	<b>Description</b>	<b>References</b>
<b>Clay mineralogy</b>	Clay minerals that typically result in soil volume changes are montmorillonites, vermiculites, and some mixed layer minerals. Illite and Kaolinites are infrequently expansive, but can cause volume changes when particle sizes are extremely fine (less than a few tenths of a micron).	Grim (1968); Mitchell (1973, 1976) ; Snethen et al. (1977)
<b>Soil water chemistry</b>	Swelling is repressed by increased cation concentration and increased cation valence. For example, $Mg^{2+}$ cations in the soil water would cause less swelling than $Na^{+}$ cations.	Mitchell (1976)
<b>Soil suction</b>	Soil suction is an independent effective stress variable, represented by the negative pore pressure in unsaturated soils. Soil suction is related to saturation, gravity, pore size and shape, surface tension, and electrical and chemical characteristics of the soil particles and water.	Snethen (1980); Fredlund and Morgenstern
<b>Dry density</b>	Higher densities usually indicate closer particle spacing, which may mean greater repulsive forces between particles and large swelling potential.	Chen (1973); Komornik and David (1969); Uppal (1965)

**Table 2.4 :** Soil properties that influence shrink-swell potential (continued).

<b>Factor</b>	<b>Description</b>	<b>References</b>
<b>Plasticity</b>	Generally, soils that exhibit plastic behavior over wide range of water content and that have high liquid limits have greater potential for swelling and shrinking. Plasticity is an indicator of swell potential.	
<b>Soil structure and fabric</b>	Flocculated clays tend to be more expansive than dispersed clays. Cemented particles reduce swell. Fabric and structure altered by compaction at higher water content or remolding. Kneading compaction has been shown to create dispersed structures with lower swell potential than soils statically compacted at lower water contents.	Johnson and Snethen (1978); Seed et al. (1962)

**Table 2.5:** Environmental conditions influence shrink-swell potential (Nelson and Miller, 1992)

<b>Factor</b>	<b>Description</b>	<b>References</b>
<b>Initial moisture condition</b>	A desiccated expansive soil will have a higher affinity for water, or higher suction, than the same soil at higher water content, lower suction. Conversely, a wet soil profile will lose water more readily on exposure to drying influences, and shrink more than a relatively dry initial profile. The initial soil suction must be considered in conjunction with the expected range of final suction conditions.	
<b>Moisture variations</b>	Changes in moisture in the active zone near the upper part of the profile primarily define heave. It is in those layers that the widest variation in moisture and volume change will occur.	Johnson (1969)
<b>Climate</b>	Amount and variation of precipitation and evapotranspiration greatly influence the moisture availability and depth of seasonal moisture fluctuation. Greatest seasonal heave occurs in semiarid climates that have pronounced, short wet periods.	Holland and Lawrence (1980)

**Table 2.5 :** Environmental conditions influence shrink-swell potential (continued)

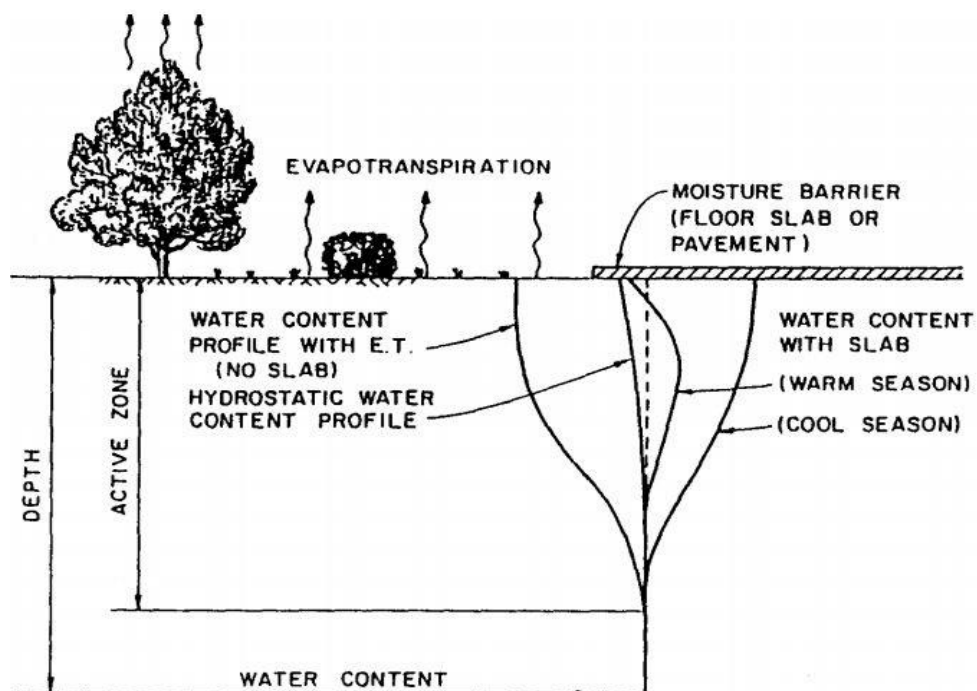
	<b>Factor</b>	<b>Description</b>	<b>References</b>
<b>Moisture variation</b>	<b>Groundwater</b>	Shallow water tables provide a source of moisture and fluctuating water tables contribute to moisture.	
	<b>Drainage and manmade water sources</b>	Surface drainage features, such as ponding ground a poorly graded house foundation, provide sources of water at the surface; leaky plumbing can give the soil access to water at greater depth.	Krazynski (1980); Donaldson (1965)
	<b>Vegetation</b>	Trees, shrubs, and grasses deplete moisture from the soil through transpiration, and cause the soil to differentially is wetted in areas of varying vegetation.	Buckley (1974)
	<b>Permeability</b>	Soils with higher permeability, particularly due to fissures and cracks in the field soil mass, allow faster migration of water and promote faster rates of swell.	Wise and Hudson (1971) De Bruijn (1965)
	<b>Temperature</b>	Increasing temperature cause moisture to diffuse to cooler areas beneath pavements and buildings.	Johnson and Stroman (1976) ; Hamilton (1969)
<b>Stress condition</b>	<b>Stress history</b>	An overconsolidated soil is more expansive than the same soil at the same void ratio, but normally consolidated. Swell pressure can increase on aging of compacted clays, but amount of swell under light loading has been shown to be unaffected by aging. Repeated wetting and drying tend to reduce swell in laboratory samples, but after a certain number of wetting-drying cycles, swell is unaffected.	Mitchell (1976); Kassiff and Baker (1971)
	<b>In situ conditions</b>	The initial stress state in a soil must be estimated in order to evaluate the probable consequences of loading the soil mass and/or altering the moisture environment therein. The initial effective stresses can be roughly determined through sampling and testing in a laboratory, or by making in situ measurements and observations.	

**Table 2.5 :** Environmental conditions influence shrink-swell potential (continued)

Factor	Description	References
Stress condition	<b>Loading</b> Magnitude of surcharge load determines the amount of volume change that will occur for a given moisture content and density. An externally applied load acts to balance interparticle repulsive forces and reduce swell.	Holtz (1959)
	<b>Soil profile</b> The thickness and location of potentially expansive layers in the profile considerably influence potential movement. Greatest movement will occur in profiles that have expansive clays extending from the surface to depth below the active zone. Less movement will occur if expansive soil is overlain by non-expansive material or overlies bedrock at shallow depth.	Holland and Lawrence (1980)

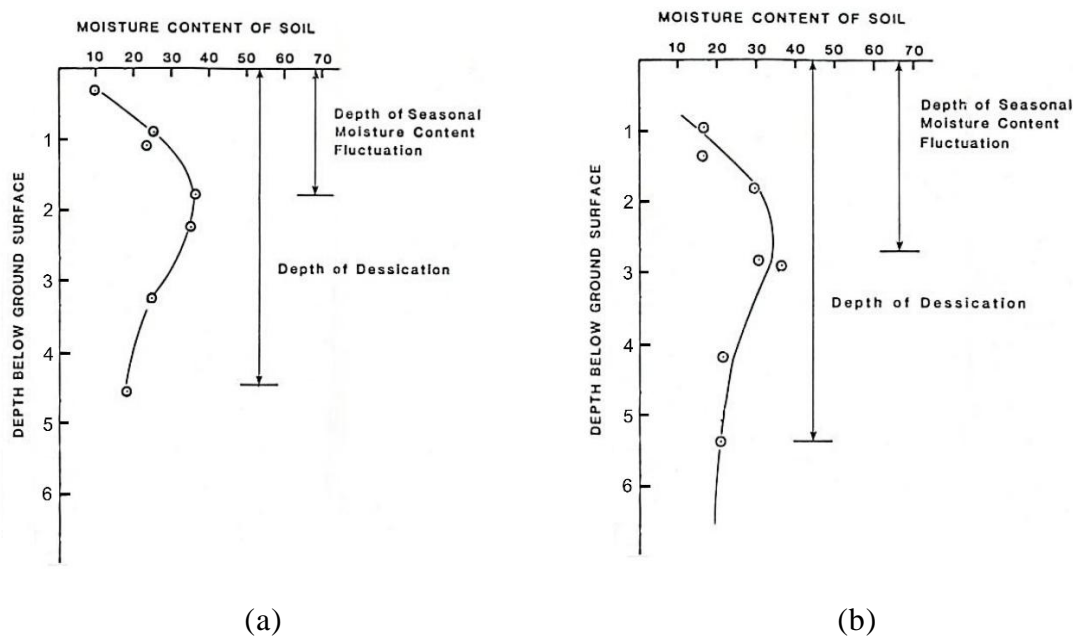
#### 2.3.4 Active zone

The moisture content is one of the major factors which controls the shrink-swell procedure of expansive soils. The depth of active zone also affects the frost heaving pattern in the capable soils and gives an accurate overview of the expansion potential in the soil profile. Figure 2.4 illustrates the seasonal water content variation in the soil profile.



**Figure 2.4 :** Seasonal moisture content variation in soil profile (Nelson and Miller, 1992)

The active zone or the zone of seasonal fluctuation is defined as the region that is close enough to the ground surface in which the soils experience a change in the moisture content due to the precipitation or the evapotranspiration depending on the climatic or the seasonal conditions (Hamilton, 1977, Day, 1999 and Chen, 1988). The depth of the active zone changes depending on soil type, soil structure, topography and climate, but as it is shown in Figure 2.5, it usually varies between 1.00 m and 4.00 m. By increasing the depth of the active zone, the region which is affected by the soil expansion increases and results in a large potential of heave due to the soil expansion.



**Figure 2.5 :** Typical plots of moisture content versus depth for exploratory borings in expansive soils: (a) boring 1; (b) boring 2 (Krazynski, 1976)

### 2.3.5 Classification of expansive soils

For the geotechnical investigation, it is necessary to identify the engineering properties of the soils. The soil classification is a systematic approach to categorize the soils into various groups and subgroups according to their general engineering behavior. Related test systems and observations are needed for detailed description. The soil classification is generally done based on two main methods; the Unified Soil Classification System (USCS) and the American Association of States Highways and Transportation Officials Method (AASHTO). The soils rated CL or CH by USCS, and A6 or A7 by AASHTO, may be considered capable to expansion (Nelson and Miller, 1992). There is not a standard classification procedure to classify expansive soils and



it varies depending on the different locations, environmental conditions, and available testing equipment. More than the type and structure of the soil, the classification of expansive soils is based on determining the degree of expansion. The expansive soil classification schemes indicate the potential expansion hazard of soils and categorizes them into groups by quantitative terms as low, medium, high, and very high, or also by other terms such as noncritical, marginal, and critical.

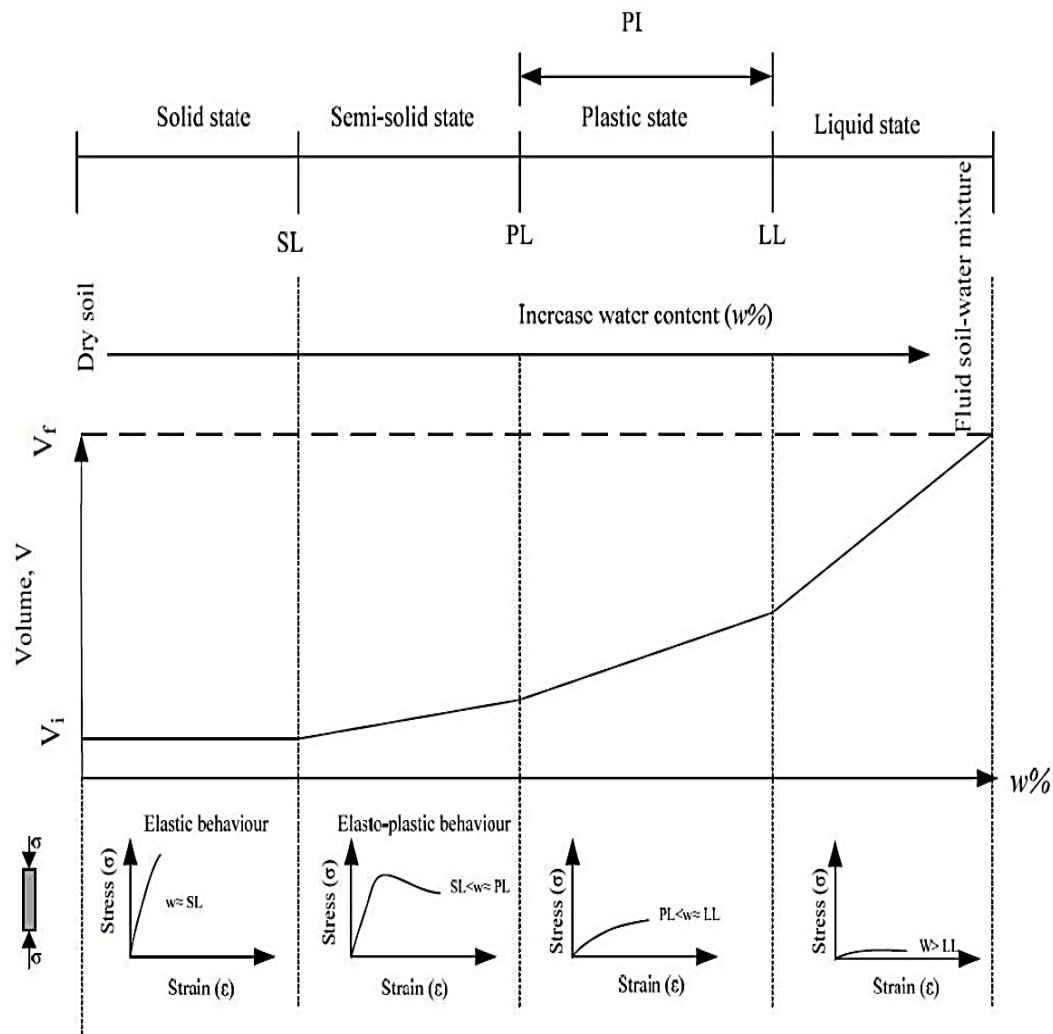
### 2.3.5.1 Classification using engineering index properties

The Atterberg limits test is one of the fundamental tests in every geotechnical laboratories and it is the most popular approach to predict swell behavior of expansive soils. Many researches have been done to determine the swell potential using the index properties of soil. The determination of the swell potential by liquid limit is the simplest approach which is shown in Table 2.6. According to the suggested values in Table 2.7, the soil expansion also can be estimated by the plasticity index. In 1955, Altmeyer suggested the use of shrinkage limit or the linear shrinkage for eliminating the use of clay content shown in Table 2.8. In that criteria, also the probable swell percent indicated and the soil is rated as noncritical, marginal, and critical.

With respect to Atterberg (1911) and Casagrande (1932), the index values such as the liquid limit (LL), the plastic limit (PL), and the shrinkage limit (SL) give a useful description of consistency of soil and help to understand the correlation between the limits and the engineering properties like the compressibility, the shear strength and the permeability. Figure 2.6 describes the relation among Atterberg limits, the volume changes, and the stress-strain response of expansive soils. The volume of the soil increase largely beyond the liquid limit state. Also by increasing the water content, the shear strength of the soil decreases.

**Table 2.6 :** Soil expansion prediction by liquid limit (Sridharan, 2000).

Degree of Expansion	LL (%)	
	Dakshanamurthy and Raman (1973)	Chen (1975)
Low	20 – 35	< 30
Medium	35 – 50	30 – 40
High	50 – 70	40 – 60
Very High	> 70	> 60



**Figure 2.6 :** Atterberg limits description, volume change and generalized stress-strain response of expansive soils (Lucian, 2006).

**Table 2.7 :** Soil expansion prediction by plasticity index.

Degree of Expansion	PI (%)			
	Holts and Gibbs (1956)	Seed et al. (1962)	Raman (1967)	Chen (1975)
<b>Low</b>	< 20	< 10	< 12	0 – 15
<b>Medium</b>	12 – 34	10 – 20	12 – 23	10 – 35
<b>High</b>	23 – 45	20 – 35	23 – 32	20 – 55
<b>Very High</b>	> 32	> 35	> 40	> 35

As it is indicated in Table 2.7, by increasing the plasticity index the swelling potential of the soil increases.

**Table 2.8 :** Expansive soil classification based on shrinkage limit or linear shrinkage (Altmeyer, 1955).

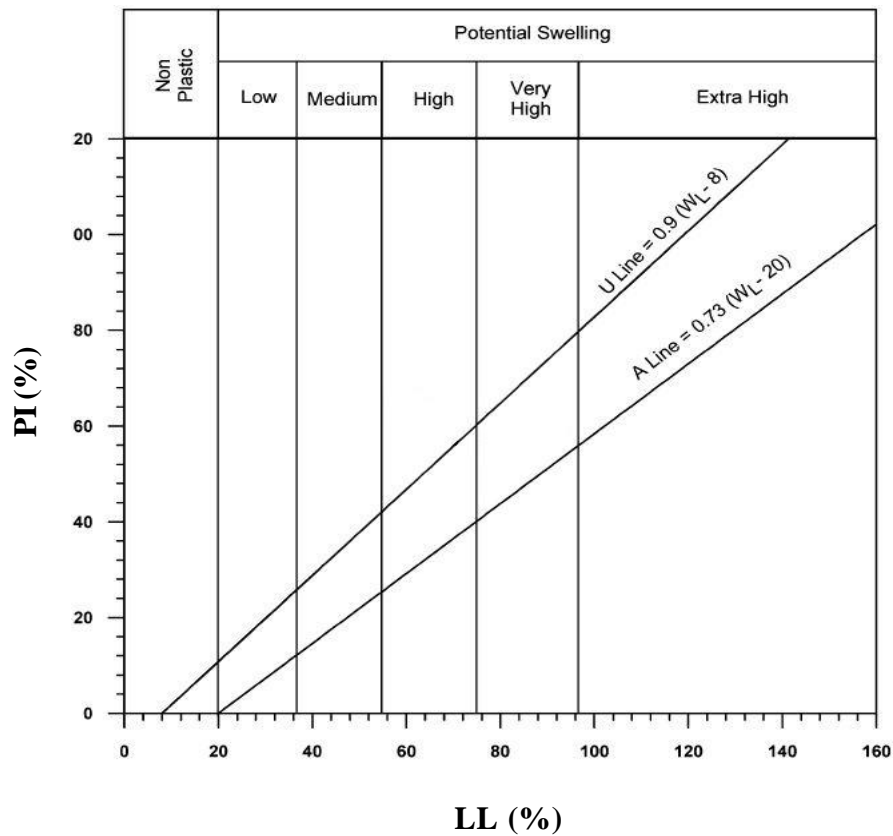
Linear Shrinkage	SL (%)	Probable Swell (%)	Degree of Expansion
< 5	> 12	< 0.5	Noncritical
5 – 8	10 – 12	0.5 – 1.5	Marginal
> 8	< 10	> 1.5	Critical

Seed et al. in 1962 and Chen in 1965 innovated a new classification criteria based on the index properties and the particle size. Chen developed a correlation between the percent of particle finer than No. 200 sieve size, the liquid limit, and the standard penetration blows counts to estimate the expansion potential. Table 2.9 shows the classification criteria suggested by Chen for classifying expansive soils. In 1962, Seed et al classified the expansive soils using the activity and the percent clay. Figure 2.8 indicates classification chart for the compacted clays based on the activity and the percent clay.

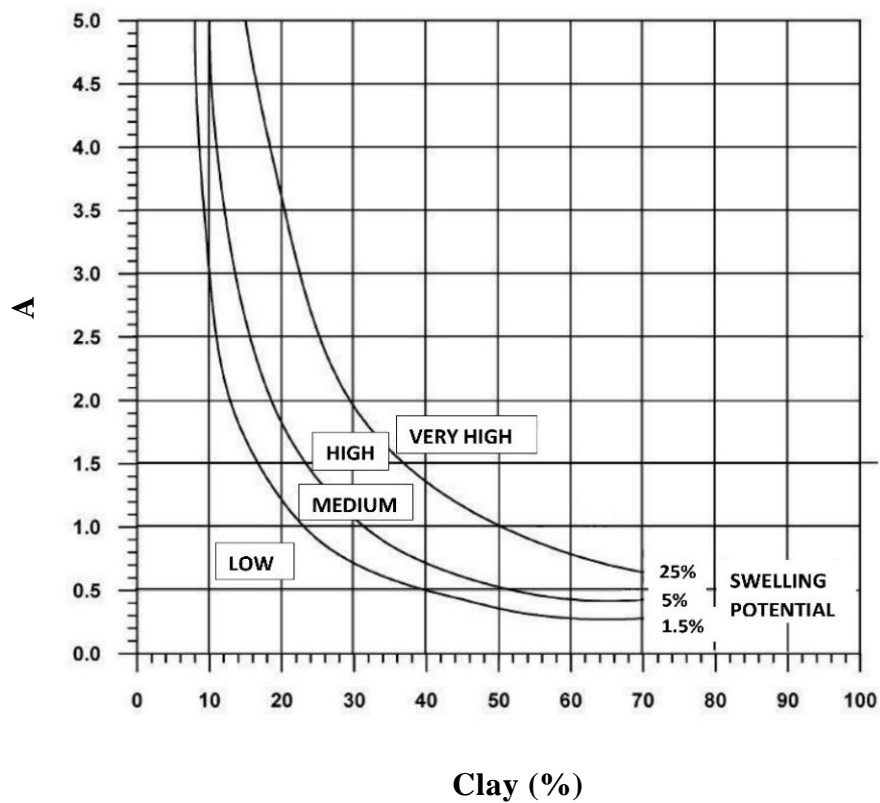
Apart of the indicated values in the Tables 2.7, 2.8 and 2.9, classification of expansive soils based on the index properties also can be done in term of the plasticity index and the liquid limit (Dakshanamanthy and Raman, 1973), the activity and the clay content (Seed et al., 1962), the plasticity index and the clay content (Skempton, 1953), and also the weighted plasticity index and the clay content (Van der Merve, 1964). Figures 2.7, 2.8, 2.9, and 2.10 illustrate the charts of the mentioned classification methods respectively.

**Table 2.9 :** Classification based on percent finer than no. 200 sieve, liquid limit, and standard penetration resistance for Rocky mountain soils (Chen, 1965).

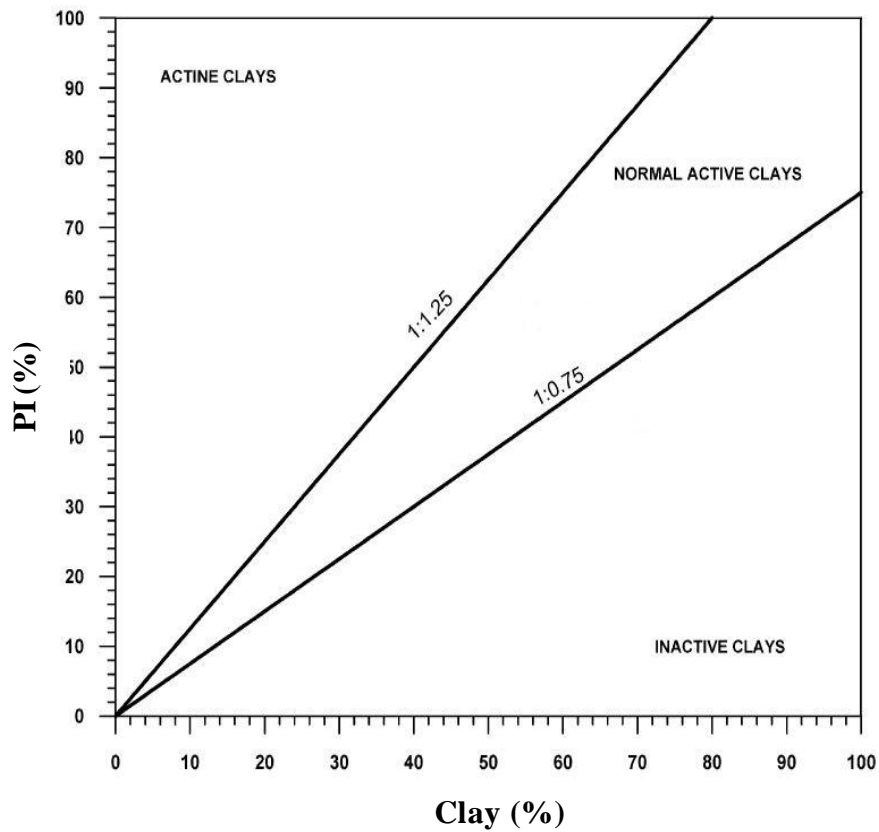
Laboratory and Field Data				
Percentage Passing No. 200 Sieve	Liquid Limit (%)	Standard Penetration Resistance (Blows/0.3m)	Probable Expansion (% Total Volume change)	Degree of Expansion
> 95	> 60	> 30	> 10	Very High
60 – 95	40 – 60	20 – 30	3 – 10	High
30 – 60	30 – 40	10 – 20	1 – 5	Medium
< 30	< 30	< 10	< 1	Low



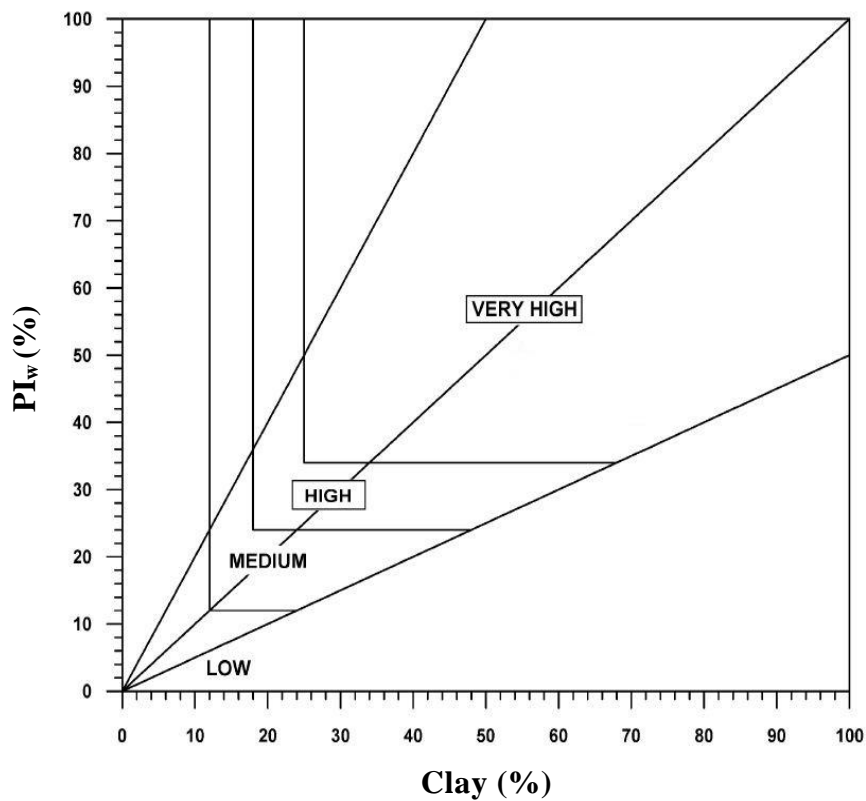
**Figure 2.7 :** Classification chart based on plasticity index and liquid limit (Dakshanamanthy and Raman, 1973).



**Figure 2.8 :** Classification chart for compacted clays based on activity and percent clay (Seed et al., 1962).



**Figure 2.9 :** Classification chart based on plasticity index and clay content (Skempton , 1953).



**Figure 2.10 :** Classification chart based on weighted plasticity index and clay content (Van der Merve, 1964).

### 2.3.5.2 Classification by use of free swell percent

According to Holtz and Gibbs (1956), the free swell test is defined as the ratio of the increase in volume of the soil from a loose dry powder form to the equilibrium sediment when it is poured into water, expressed as the percentage of the original volume (Figure 2.11). The percent of free swell is expressed as:

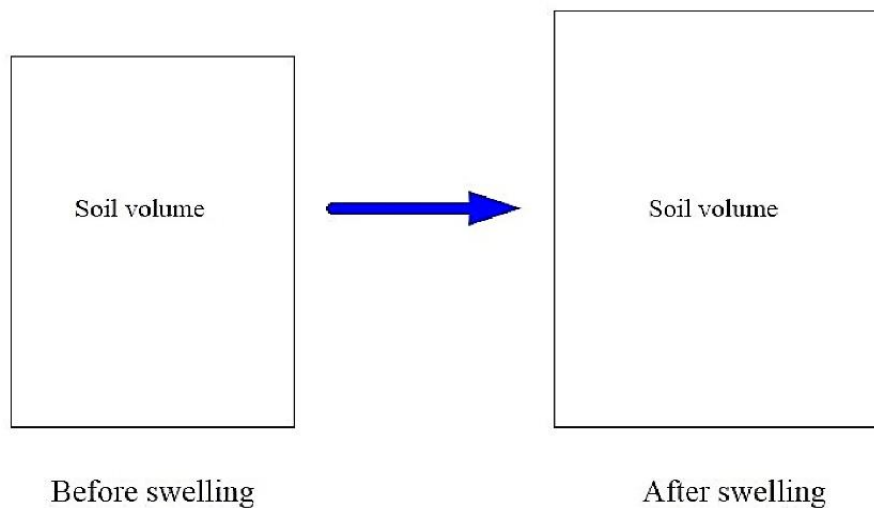
$$\text{Free swell percent} = (\Delta V/V) \times 100 \% \quad (2.2)$$

where;

$\Delta V = V_f - V_i$  = change in initial volume (V) of a specimen

$V_i$  : Initial volume ( $10 \text{ mm}^3$ ) of the specimen

$V_f$ : Final volume of the specimen



**Figure 2.11:** Phase diagram of free swell (Lucian, 2006).

Soils with free swell less than 50% are not likely to be capable to expand remarkably, while soils with free swells in excess of 50 percent could present swell problems. Values of 100% or more are associated with clay which could swell considerably, especially under light loadings.

### 2.3.5.3 Classification by use of coefficient of linear extensibility (COLE)

The coefficient of linear extensibility (COLE) is one of the important engineering properties of soils related to the engineering index properties and the cation – exchange

capacity (CEC) to characterize the shrink-swell potential of soil. The method was developed by McKeen and Hamberg (1981) and Hamberg (1985). It is an extended scheme of the concepts of Pearring (1963) and Holtz (1969), who innovated a mineralogical classification chart based on the correlations between the mineralogy, the clay activity (A) and a new parameter, the cation-exchange activity ( $CEA_c = CEC/\text{clay content}$ ). COLE parameter can be obtained from the laboratory procedure. The undisturbed samples are briefly immersed in a flexible resin and allowed to dry in laboratory. The resin coating has to be impermeable to water but permeable to water vapor (Thomas, 1998). The clods are put to field tension of 33 kPa or 10 kPa tension (1/3- or 1/10-bar tension), weighed in air and water to determine weight and volume using Archimedes principle. After the clods are dried by oven dryness, their weight and volume measured again. When coarse fragment are present, the COLE is calculated as follows:

$$COLE_{ws} = \left[ \frac{1}{C_m \times \left( \frac{\gamma_{d33<2mm}}{\gamma_{d<2mm}} \right) + (1 - C_m)} \right]^{\frac{1}{3}} - 1 \quad (2.3)$$

where,  $COLE_{ws}$  : Coefficient of linear extensibility on a whole-soil base in  $\text{cm cm}^{-1}$

$\gamma_{d33<2mm}$  : Dry density at 33 kPa water retention on a <2 mm base ( $\text{g/cm}^3$ )

$\gamma_{d<2mm}$  : Dry density, oven-dry or air-dry, on a <2 mm base ( $\text{g/cm}^3$ )

$C_m$  : Coarse fragment (moist) conversion factor.

$C_m$  is calculated as follows:

$$C_m = [\text{volume moist } < 2 \text{ mm fabric } (\text{cm}^3)] / [\text{volume moist whole soil } (\text{cm}^3)]$$

$$\text{or } C_m = (100 - \text{vol} > 2 \text{ mm}) / 100$$

where;  $\text{vol. } > 2 \text{ mm}$  = volume percentage of the >2 mm fraction

If no coarse fragments;

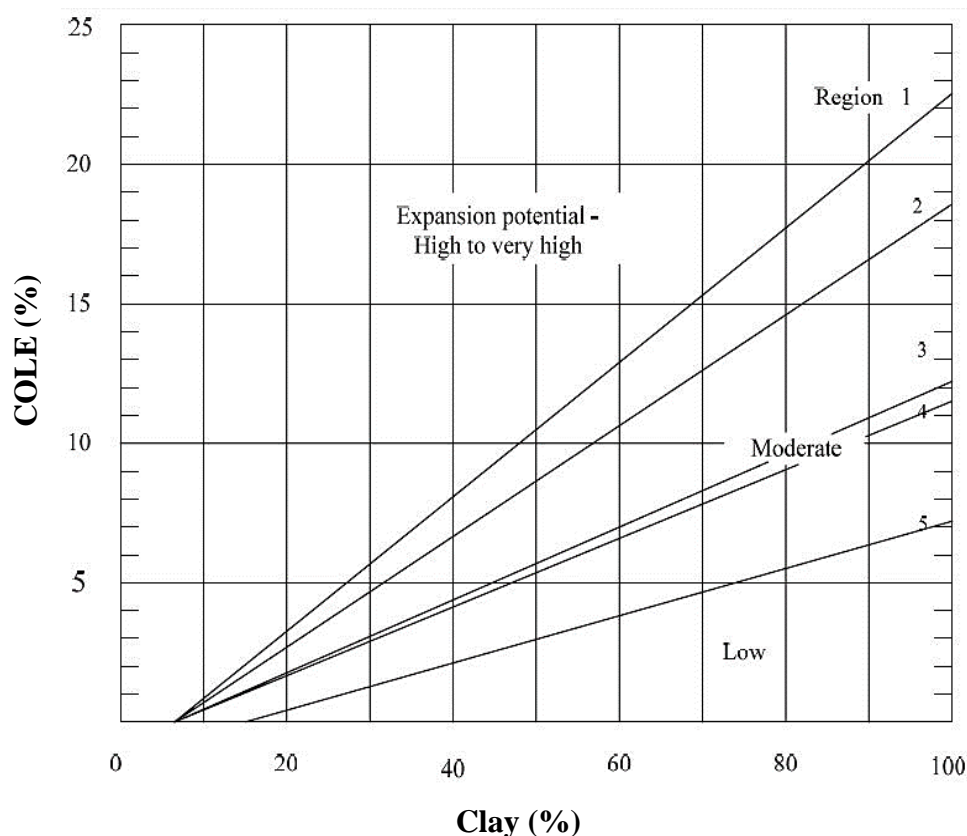
$C_m = 1.00$ , and the previous equation reduces to:

$$COLE_{ws} = \left( \frac{\gamma_{d<2mm}}{\gamma_{d33<2mm}} \right)^{\frac{1}{3}} - 1 \quad (2.4)$$

According to the determined COLE, a range of soil shrink-swell potential can be evaluated based on data in Table 2.10. The quantitative swell potential can also be estimated by correlating the colloids content and the COLE factor using Figure 2.12.

**Table 2.10 :** Classification of expansive soils using COLE factor  
(Thomas et al., 2000).

Soil expansion potential	COLE
Low	< 0.03
Moderate	0.03 – 0.06
High	0.06 – 0.09
Very high	> 0.09



**Figure 2.12:** Expansion potential as a function of colloids and COLE  
(Hardcastle, J. H., 2003).

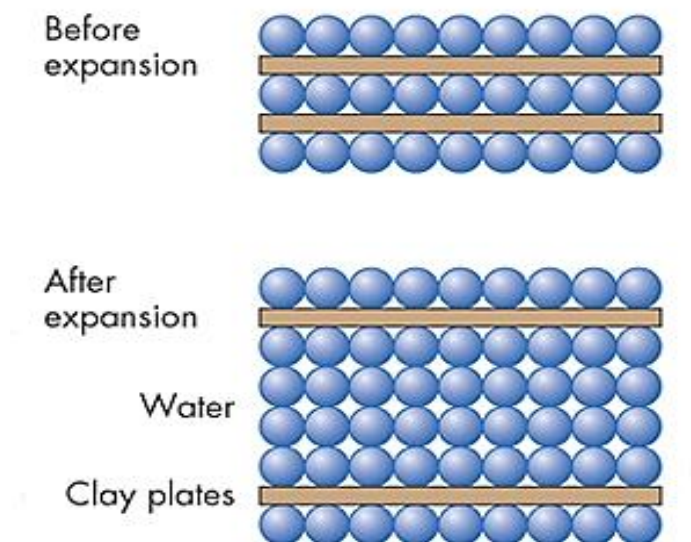
## 2.4 Swelling Mechanism of Expansive Soils

The expansive soils also can be mentioned as clays. The swelling potential of the expansive soils is highly related to their mineralogy and the chemical structure of the soil mass. The montmorillonite is the clay mineral that presents most of the expansive



soil problems. Soils with a high percentage of swelling clay have a very high affinity for water partly because of their small size and partly because of their positive ions (Day, R. W., 1999). Expansive soil usually swells and sticks when it is wetted, and shrink when dry developing wide cracks or a puffy appearance. From the mineralogical standpoint, the magnitude of expansion depends on the kind and amount of clay minerals present, their exchangeable ions, electrolyte content of aqueous phase, and the internal structure.

The swelling behavior is commonly attributed to the intake of water into the montmorillonite, an expanding lattice clay mineral in expansive soils. With respect to Chen, (1988), montmorillonite is made up of a central octahedral sheet, usually occupied by aluminium or magnesium, sandwiched between two sheets of tetrahedral silicon sites to give a 2 to 1 lattice structure. The formation of clay minerals and their potential for absorbing water, permits a large amount of water to be adsorbed in the interlayer in the clay crystalline, resulting in the remarkable swelling of soil (Patrick and Snethen, 1976). Figure 2.13 shows how water pushing out the clay plates which finally causes to the heaving of the soil.



**Figure 2.13 :** Moisture inclusion in clay which causes swelling  
(<http://www.superiorfoundationrepair.com>)

## 2.5 Geotechnical Problems Related to Expansive Soils

The constructions on the expansive soils without considering the shrink-swell potential of subgrade soil layer can be result in serious damages to the buildings, the pavements, the high ways, the railways, the retaining walls, the pipe lines, and also causes some

problems such as slope failure. According to the seasonal and the climatic state and drainage condition, wetting and drying of expansive soil cause to swell or to shrink in the soil mass respectively. The shrink-swell procedure results in considerable deformations in the soil layer which can influence the foundation or superstructure built on the expansive layers. The intensity of the expansive soils induced problems varies depending on the expansion potential of the soil, thickness of soil layer, and designing criteria. Figure 2.14 shows four different typical damages due to the expansive soils.



(a)



(b)



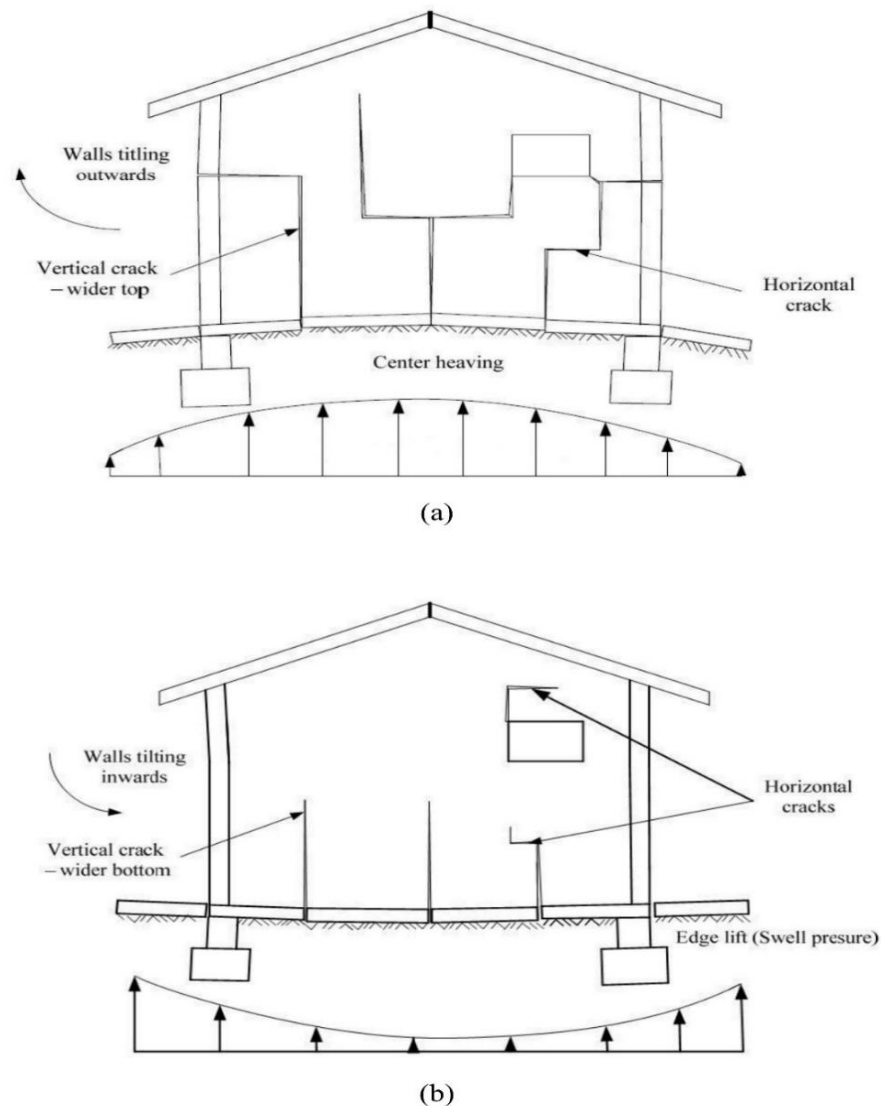
(c)



(d)

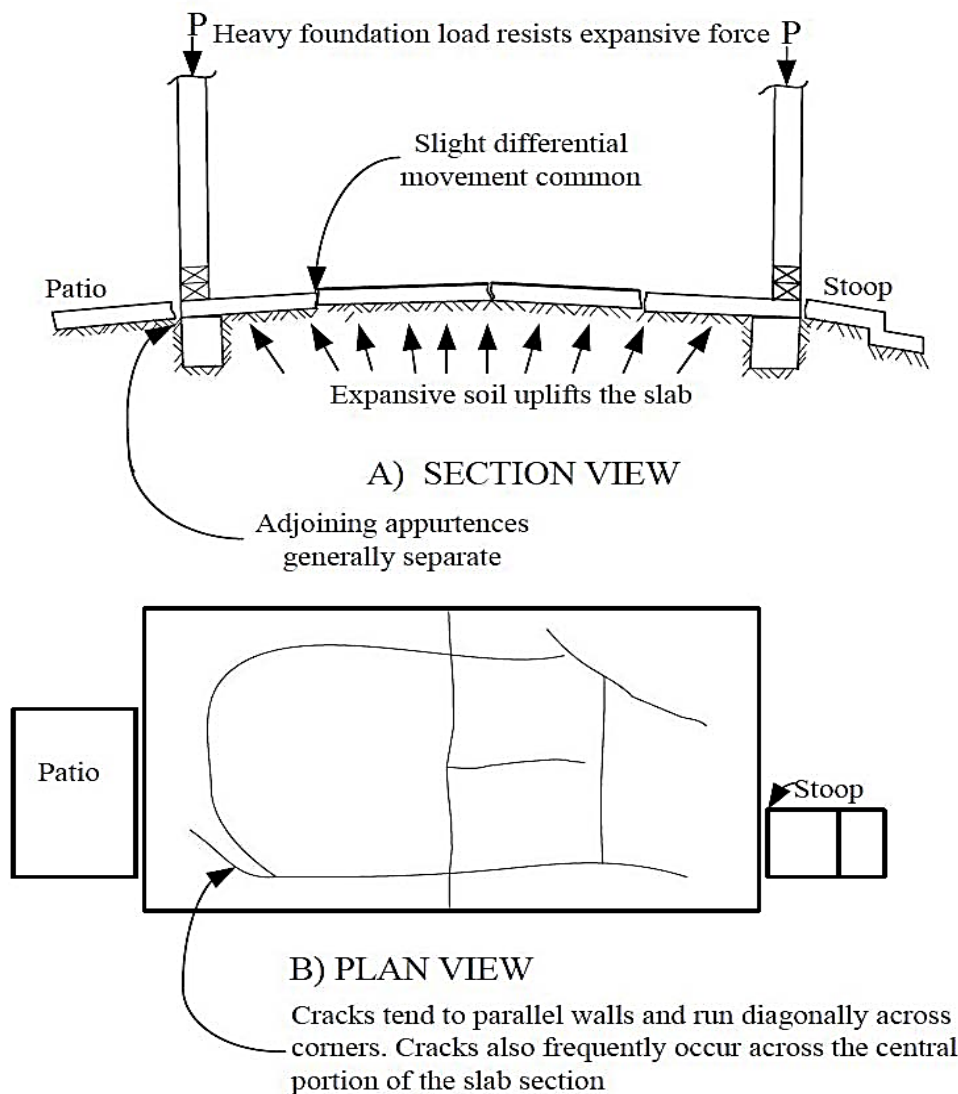
**Figure 2.14 :** Typical damages due to expansive soils; a) Differential vertical movement caused by expansive soil measures 9 cm at a pavement joint failure in the Meadow Creek subdivision in Frisco, Texas (<http://www.capitalgeotechnical.com>) , b) longitude cracks on the road caused by expansive soils (<http://www.capitalgeotechnical.com>), c) buckled foundation damaged from expansive clay (<http://www.cenews.com>), d) severe cracks on the wall resulting from structural damages of expansive soils (<http://www.basementsystem.ca>).

The structures which their foundations located at the shallow depths or the active zone, are most vulnerable to swell-shrink on the expansive soils. Especially light buildings can be affected by differential heave of foundation or slabs and experience some significant problems like heaving and cracking of floor slabs and walls, jammed doors and windows, ruptured pipelines, and etc. Based on the observed cracks, the degree of damage ranges from hairline cracks to the severe cracks, very severe cracks or total collapse. The pattern of the cracks depends on whether it is a dooming heave or a dish shaped lift heave. The dome effect occurs when the movement of the moisture from the perimeter to the center of the house while the dish effect results from the moisture moving from center to the perimeter. Figure 2.15 shows a schematic expectation of heaving and crack patterns due to dooming and dishing behavior of swell soils.



**Figure 2.15 :** crack patterns and heaving resulting from expansive soils on light buildings; a) center heave or dooming b) Edge heave or dishing. (Lucian, 2006)

Apart of mentioned crack patterns, in most cases, cracks due to shrinkage and expansive clay usually run from corner towards adjacent opening and are uniform in width or v-shaped, wider at the top than the foundation wall (Mika and Desch, 1998 and Ransom, 1981). This pattern of cracks arises when the moisture flow is from the perimeter to the center of the house. Figure 2.16 illustrates the typical crack pattern in the concrete slab-on-grade concrete due to center heaving of expansive soils (Day, 1999).



**Figure 2.16 :** Typical crack pattern on a floor due to center heave (Day, R. W., 1999).

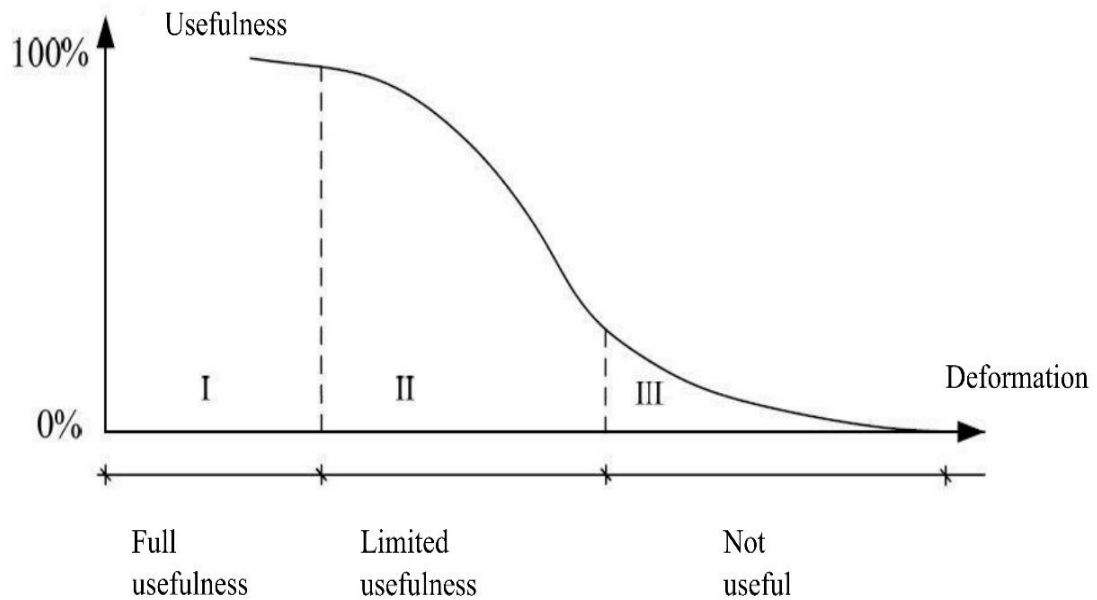
All structures have a defined range of usefulness during the various damages that they experience during their lifetime. The damages cause due to design faults or other reasons such as unsuitable construction materials, poor workmanship or calamities,

poor drainage condition, climatic condition and complex behavior of expansive soils. Figure 2.17 indicates the reduction of usefulness of a structure, due to the deformation. With respect to Hintze (1994), the usefulness of a structure considering the deformations generally can be categorized in three states:

I) Full usefulness

II) Limited usefulness

III) Not useful



**Figure 2.17 :** Decreased usefulness of deformation (Hintze, 1994).

Not all deformations lead structures to the useless state. Despite the deformation at the earlier steps in stage I, the structure is still in full usefulness. In stage II, the deformations raise and the usefulness associated with this deformations drops to limited usefulness state. In phase III, the deformation is almost gradual but the structure is no longer useful.

Expansive soils also can be result in very serious damages on the roads and the pavements. Many roads were abandoned or replaced due to the exercise of frequent maintenance. In general, because of flexibility and tolerance of asphalt to some displacements, asphalt pavements perform better than rigid concrete pavements response to expansive soils. However, asphalt pavements still suffers seriously from cracks and displacements due to the expansive soils. Expansive soils induced road and pavements damages cause necessity to maintenance works which vary from filling up

cracks with asphalt emulsion slurry or emulsified asphalt to patch repair and overlays, costs multi million dollars worldwide (Dafalla and Shamrani, 2011). Figure 2.18 exhibits tow cases of roads suffering cracks and displacements seriously due to the expansive soil subgrade.

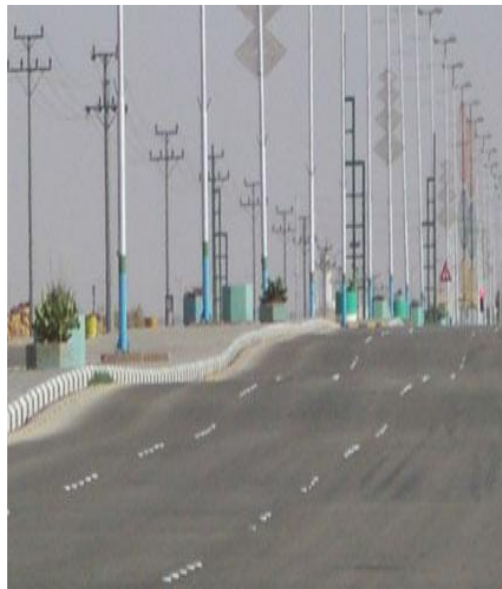
The cracks on the asphalt resulted from expansive soils can be categorized in two phases:

I) swelling phase

II) Shrinkage phase



(a)



(b)

**Figure 2.18 :** Road damages due to the expansive soils; (a) Tayma-Tabuk Road (abandoned), (b) King Abdulaziz Road- AlGhatt (Dafalla and Shamrani, 2011).

Figure 2.19 presents typical features of six types of crack associated with expansive soils. Transverse cracks occur when the subgrade of the road is of medium to high swelling potential and the heave is beyond the tolerance of the asphalt mix. This type occurs when a subsurface flow of moisture takes a transverse direction and a mound/depression feature is formed. Position of cracks is either on the top of the mount or the bottom of the depression. Longitudinal cracks take place either parallel to the edge or along the asphalt joint between lanes. Block cracks occur when the underneath of a large patch is subjected to moisture changes.

Three new forms of damage are introduced. The first type is "yield" cracks and they occur when the asphalt is subjected to frequent up and down movements due to swelling and repeated heavy tire pressure. This is mostly taking the form of multiple



parallel longitudinal cracks. The second type introduced is the "spot ridge" cracks and defined as those having a clearly defined center where damage is excessive. The third is the "green zone" cracks and these are related to the vicinity of landscaping and green areas.



Longitudinal Crack



Transverse Crack



Block Crack



Yield Cracks



Spot Ridge Cracks



Green Zone Cracks

**Figure 2.19 :** Crack types associated with expansive clays  
(Dafalla and Shamrani, 2011).





### **3. SWELL PRESSURE PREDICTION AND STABILIZATION TECHNIQUES OF EXPANSIVE SOILS**

#### **3.1 Swell Pressure and Potential Identification**

The swell pressure is usually defined as the pressure required to recompress the fully swollen sample back to its initial volume.

#### **3.2 Prediction Methods to Determine the Swell Pressure**

Predicting the swell potential of expansive soils leads toward the use of proper treatment applications and designing criteria which prevent unexpected damages due to shrink-swell behavior of problematic clays. Furthermore the several prediction methods which are based on the laboratory testing systems, also some empirical approaches developed to anticipate the behavior of swell clays. In spite of the accuracy and reliable results gained by the laboratory based methods, they cost more and take much time in comparison to empirical methods. In general, considering the importance of project, the type of soil, the local conditions, and the economical aspects of the techniques can help to choose a proper swell prediction method.

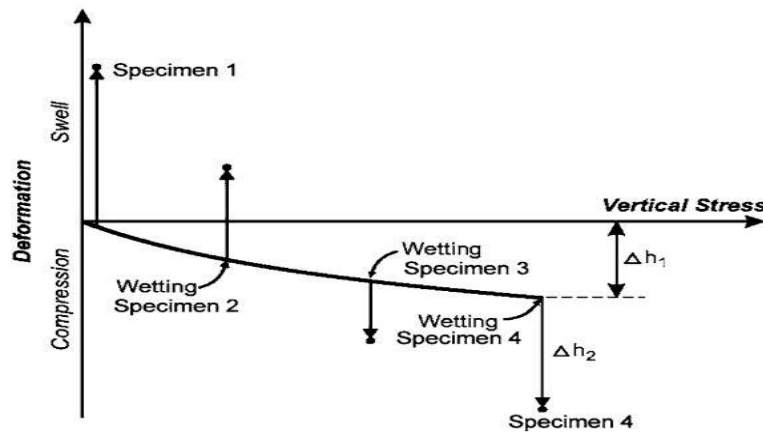
##### **3.2.1 Swell prediction based on odometer tests**

The most common heave prediction tests involve the use of one-dimensional consolidation apparatus or odometer. According to the ASTM-D4546 standard, there are three main method for heave prediction of expansive soils using one – dimensional odometer test.

##### **3.2.1.1 Method A**

This method can be used for measuring one-dimensional wetting-induced swell or collapse (hydro- compression) strains of compacted or natural soils over a range of vertical stresses. Four or more identical specimens are assembled in consolidometer units. Different loads are applied to different specimens and each specimen is has been given access to free water until the process of primary swell or collapse is completed under a constant vertical total stress. The resulting swell or collapse deformations are

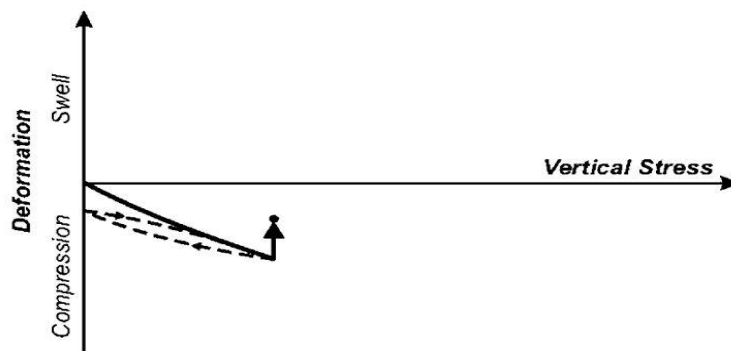
measured. The final water contents and dry densities are also measured. This method can be referred as wetting-after-loading tests on multiple specimens. The data obtained from these tests can be used to estimate one-dimensional ground surface heave or settlement. In addition, the magnitude of “Swell Pressure,” the minimum vertical stress required for preventing swell, and the magnitude of free swell, the swell strain corresponding to a near zero stress of 1 kPa can be interpreted from the test results.



**Figure 3.1 :** Deformation versus vertical stress, Method A (ASTM-D4546).

### 3.2.1.2 Method B

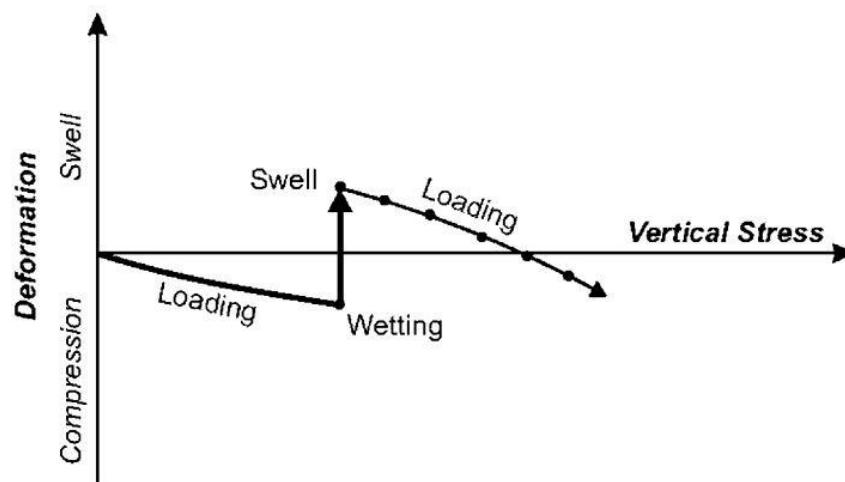
This method can be used for measuring one-dimensional wetting-induced swell or collapse strain of a single “intact” specimen of natural soil, or a single “intact” specimen of compacted soil obtained from an existing fill or embankment. The specimen is loaded to a specific vertical stress, typically the in-situ vertical overburden stress or a particular design pressure, or 1 kPa for measuring the free swell strain, and then inundated to measure the wetting induced strain under that particular stress. This method can be referred to as single point wetting-after- loading test on a single specimen.



**Figure 3.2 :** Deformation versus vertical stress, Single-Point Test Method B (ASTM-D4546).

### 3.2.1.3 Method C

This method is for measuring load-induced strains after wetting-induced swell or collapse deformation has occurred. It can be referred to as loading-after-wetting test. The results would apply to situations where new fill and/or additional structural loads are applied to the ground that has previously gone through wetting-induced heave or settlement. The first part of the test is the same as in Method A or B. After completion of the swell or collapse phase, increments of additional vertical loads are applied to the specimen in the same manner as in a consolidation test, Test Methods ASTM-D2435, and the load-induced deformations are measured.



**Figure 3.3 :** Deformation versus vertical stress,  
Loading-after-Wetting Test Method C (ASTM-D4546).

### 3.2.2 Empirical procedures for swell prediction

Empirical procedures are usually based on the test data which are developed from the particular geographic region. In order to decrease time and cost of laboratory testing methods, many studies have done to evolve empirical relationships for prediction of heave, but the major disadvantage of them is that they are based on a limited data and only can be applied in the regions they were developed. The results of the empirical methods are not accurate enough to be used as a quantitative prediction of heave and the only can be considered as indicator of the expansion potential. (Nelson and Miller, 1992).

In 1964, Van der Merwe developed a simple equation by using the potential expansiveness and the reduction factor to account for decreasing heave with depth.

The expansive soil layer is divided to  $n$  layers and the total heave is estimated by equation 3.1.

$$\rho = \sum_{i=1}^{i=n} F_i \times PE_i \quad (3.1)$$

where;

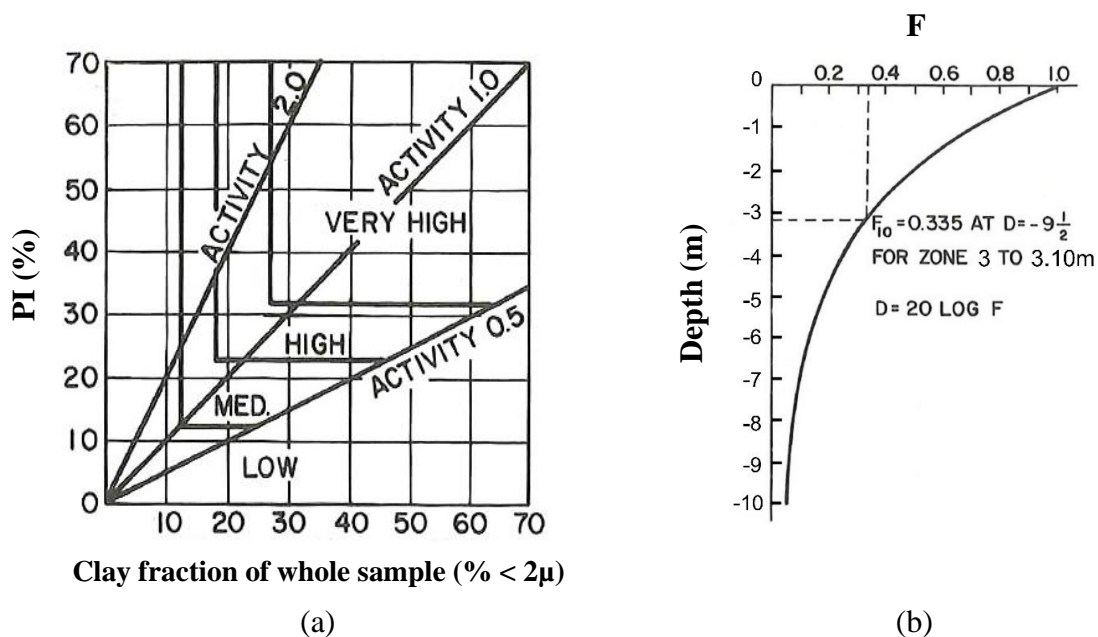
$F_i$  : reduction factor for layer  $i$

$PE_i$  : potential expansiveness for layer  $i$

$F_i$  is obtained from Figure 3.4.b. The value of the potential expansiveness  $PE$  is determined using Table 3.1 which is obtained from soil classification shown in Figure 3.4.a based on plasticity index and clay content.

**Table 3.1** : Classification used to PE value

Expansion potential	PE
Very high	8.30 cm per meter depth
High	4.10 cm per meter depth
Medium	2 cm per meter depth
Low	0 cm per meter depth



**Figure 3.4** : Relationship to use in Van der Merwes' empirical heave prediction method: a) potential expansiveness, b) reduction factor (Van der Merwe, 1964).

The mentioned empirical procedure does not consider initial soil conditions like water content, suction, or density. The method can be used only as an indicator of heave and it is not more reliable for quantitative predictions.

Schneider and Poor (1974) using Texas clays proposed statistical relationships between measured swell for different surcharges and plasticity index and water content. Table 3.2 indicates the presented equations to predict the percent swell,  $S_p$ .

**Table 3.2 :** Prediction the percent swell,  $S_p$  (Schneider and Poor, 1974).

Surcharge (kPa)	Log $S_p$
0	$0.90 (PI/w) - 1.19$
3	$0.65 (PI/w) - 0.93$
15	$0.51 (PI/w) - 0.76$
30	$0.41 (PI/w) - 0.69$
57	$0.22 (PI/w) - 0.62$

In 1980, Weston improved the Van der Merwe's method to take into account the moisture content. He proposed a method of calculating swell based on the liquid limit which can be determined more accurately than plastic index. The percent swell is calculated as indicated in equation 3.2.

$$\text{Swell (\%)} = 0.00041 (W_{LW})^{4.17} (P)^{-0.386} (w_i)^{-2.33} \quad (3.2)$$

where;

$$W_{LW} = (LL) \left( \frac{\% < 0.425 \text{ mm}}{100} \right)$$

P : vertical pressure in  $\text{kN/m}^2$  (kPa), under which swell takes place

$w_i$  : initial moisture content (%)

In 1987, Pidgeon proposed an empirical relation more user-friendly for determining free swell which calculated as follows:

$$\text{Swell (\%)} = FS \left[ 1 - \left( \frac{\log P}{\log P_S} \right) \right] \quad (3.3)$$

where;

FS : free swell (%)

P : pressure induced by the foundation and the overburden (kPa)

P<sub>s</sub> : swelling pressure of the soil (kPa)

Another empirical method was suggested by Vijayvergiya V. N. and Ghazzaly O. I. (1973) using the following relationships:

$$\log s = \frac{1}{12} (0.4 \text{ LL} - w_n + 5.5) \quad (3.4)$$

$$\log s = \frac{1}{19.5} (6.242 \gamma_d + 0.65 \text{ LL} - 130.5) \quad (3.5)$$

where;

S : swell (%)

$\gamma_d$  : the dry weight density in kN/m<sup>3</sup>

LL and  $w_n$  are liquid limit and the natural water content.

### **3.3 Stabilization Techniques of Expansive Soils**

The expansive soil stabilization techniques vary depending on the environmental conditions, the active zone depth, the swell potential of the soil layer, the available equipment, the materials and the cost of improvement method. Not only decreasing expansion potential but also the basic factors for designing a geotechnical structure should be considered during the soil improvement procedure.

#### **3.3.1 Replacement of expansive soils with non-expansive soils**

In this technique, the expansive soil layer is excavated up to certain depth (active zone depth) and is replaced by proper soil which is not expansive and has a good bearing capacity. After replacing the new layer, it needs to be compacted up to a certain ratio depending on the project and loads which will be applied on the soil. Considering the cost of improvement procedure, it can be applied only where the proper alternative soil is easily and cheaply available nearby. Removal and replacement is a shallow ground improvement method and is generally practical only above the ground water table. Earthwork applications are difficult and cost more when the soil layer is wet or submerged (Rakesh. and Jain, 2012).

### **3.3.2 Sand cushion**

Sand cushion method consists of removing entire depth or a part of soil stratum and replacing it with the sand. The replaced sand is compacted to the desired density and thickness. This method minimizes the negative effects of poor soil. (Satyanarayana,1969). The major advantage of the sand cushion method is its ability to adapt itself to volume changes caused by shrink-swell behavior of the expansive soil. However, there are some limitation for the sand cushion method particularly when it is applied in deep strata. The high permeability of sand create a conductivity condition which makes it easy for water to ingress and be accumulated from surface runoff.

### **3.3.3 Cohesive non swelling layer (CNS)**

Replacement problematic expansive soils with cohesive non – swelling material may improve the disadvantages of sand cushion method which are related to high permeability of sands. The method is proposed by Katti et al. (1983) using cohesive non-swelling (CNS) layer to decrease the swelling effects of the expansive soil. The heave potential of the expansive soil underlying a CNS layer is reduced exponentially with increase in thickness of the CNS layer and approaches to a value of no heave around depth of 1m. Also the shear strength of the underlying expansive soil at the interface and below increases with the thickness of CNS layer. The method is recommended for construction of canals in the expansive soil areas.

### **3.3.4 Moisture control**

The source of swelling drives from an increase in water content of the soil mass. If the soil is isolated from any moisture changes, volume change could be reduced or minimized. In this context water membranes are becoming an increasingly promising in method for limiting access of water and minimize moisture changes particularly in the construction of pavements and roads.

### **3.3.5 Surcharge loading**

Loading the expansive soil with pressure greater than the swelling pressure is method by which swelling can be prevented. The use of this method is limited to low to moderate swelling pressures. Because of the nonlinear nature of the pressure-swell

relationship, by increasing the swell pressure the use of a surcharge becomes less efficient. The surcharge method is most effective when swell pressures are low and some heaving can be accepted in the construction project such as in a secondary highway system. For large projects involving high foundation pressures and which anticipated swell pressure low to moderate, this method may also be effective. Many soils exhibit swell pressure too high to be controlled by normal surcharge loads alone. Swell pressure up to about 25 kPa can be controlled by 1.3 m of fill and a concrete foundation. However, some soils may have swell pressure as high as 400 kPa.

### **3.3.6 Pre-wetting**

Pre-wetting is based on the theory that increasing the water content in the expansive soils will cause heave to occur prior to construction and thereby eliminate problems afterward. But there are some serious difficulties that limit the application of the pre-wetting method. Expansive soils typically exhibit low permeability and the time required for adequate wetting can be up to several years. Furthermore, after wetting the soil for long periods of time, serious loss of soil strength can cause reduction in bearing capacity and slope stability. Vertical sand drains drilled in a grid pattern can decrease the wetting time. Despite of the long time and difficulties for application, the method can be used successfully in some cases such as highly fissured and desiccated soils.

### **3.3.7 Reinforcing the soil using Geosynthetic**

Geosynthetics have been used successfully worldwide in several sections of civil engineering. They are well accepted as construction material and their use offers excellent economic alternatives to the conventional solutions of many civil engineering problems.

Geosynthetics are versatile in use, adaptable to many field conditions and can be used in combination of several building materials. They are utilized in a wide range of applications in many areas of civil engineering, specially geotechnical, transportation, water resources, geoenvironmental, coastal, and sediment and erosion control engineering to achieve technical or economic benefits (Shukla and Yin, 2006).



### **3.3.7.1 Definition and classification**

Geosynthetic is a generic name representing a broad range of planar products manufactured from polymeric materials. The term “geosynthetics” includes two parts which the first part “geo”, referring to the utilization for improving the performance of civil engineering works involving the earth, ground or soil. The second part “synthetics” referring to the fact that the materials are almost from man-made products. The most common types of geosynthetics which are used in contact with soil, rock and can be categorized as geotextiles, geogrids, geonets, geomembranes and geocomposites. Geotextile is a planar, permeable, polymeric textile product in form of a flexible sheet. Geogrid is a planar, polymeric product consisting of a mesh or net-like network intersecting tensile-resistant elements, called ribs, integrally connected at the junctions. Geonet is a planar, polymeric product consisting of a regular dense network of integrally connected parallel sets of ribs overlying similar sets at various angles. Geomembrane is a planar, relatively impermeable, synthetic sheet manufactured from materials of low permeability to control fluid migration in a project as a barrier or liner. Geocomposite is a product that is assembled or manufactured in laminate or composite form from two or more materials, of which one at least is geosynthetic, to enhance the performance more effectively than when used separately (Shukla and Yin, 2006).

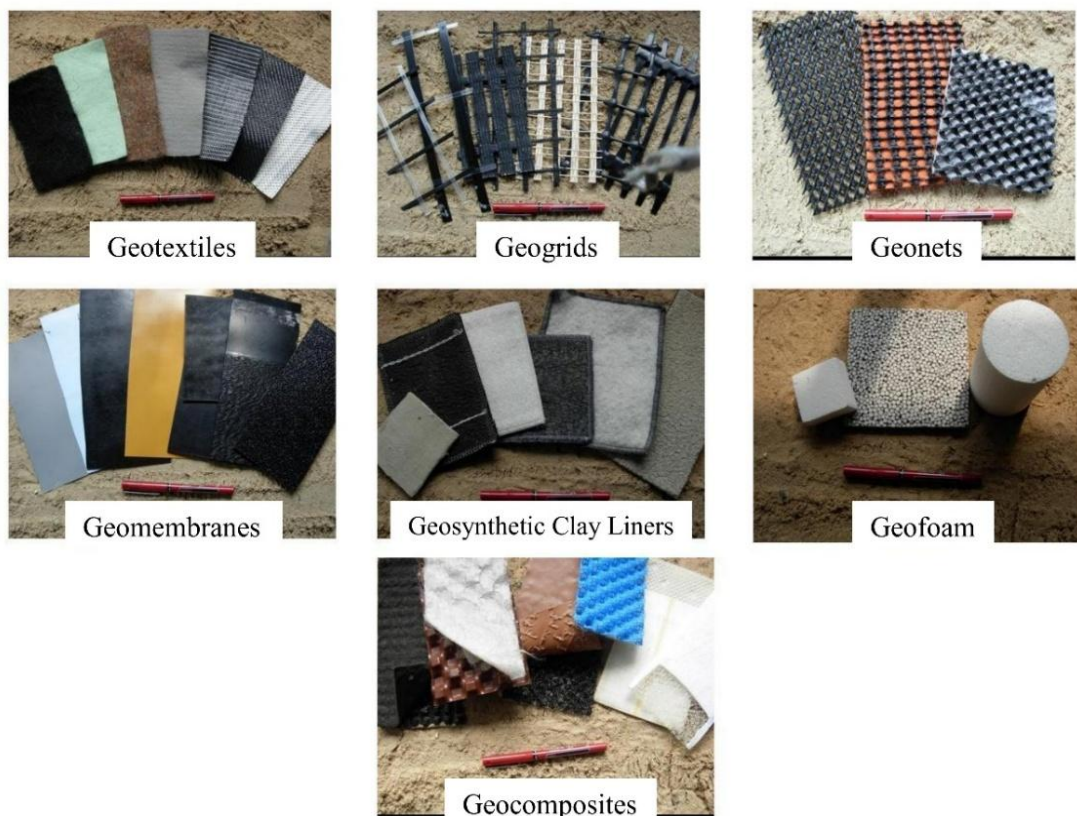
### **3.3.7.2 Functions and applications**

In general, geosynthetic applications are defined by their primary, or principal, function. Geosynthetics have six primary functions:

- Filtration
- Drainage
- Separation
- Reinforcement
- Fluid barrier
- Protection

In a number of applications, in addition to the primary function, geosynthetics usually perform one or more secondary functions. It should be considered that both the primary and secondary functions in the design computations and specifications. More than 150 separate applications of geosynthetics have been identified (Koerner, 2005).

Some examples for the application of the geosynthetics are described follow: Geotextile filters replace graded granular filters in trench drains to prevent soils from migrating into drainage aggregate or pipes. They are also used as filters below riprap and other armor materials in coastal and river bank protection systems. Geotextiles and geocomposites can also be used as drains, by allowing water to drain from or through soils of lower permeability. Geotextiles are often used as separators to prevent fine-grained subgrade soils from being pumped into permeable, granular road bases and to prevent road base materials from penetrating into the underlying soft subgrade. Separators maintain the design thickness and roadway integrity. Geogrid and geotextile reinforcement enables embankments to be constructed over very soft foundations. They are also make it possible to construct slopes at much steeper angles. Polymeric reinforced backfills for retaining walls and abutments was mentioned in the Introduction.



**Figure 3.5 :** Types of geosynthetics applicable in geotechnical engineering.

Geomembranes, thin-film geotextile composites, geosynthetic-clay liners, and field-coated geotextiles are used as fluid barriers to impede the flow of a liquid or gas from one location to another. This geosynthetic function has application in asphalt pavement overlays, encapsulation of swelling soils, and waste containment. As a function of protection, the geosynthetic acts as a stress relief layer. A protective cushion of nonwoven geotextiles is often used to prevent puncture of geomembranes (by reducing point stresses) from stones in the adjacent soil or drainage aggregate during installation and while in service (Guang-Xin et al, 2008). Figure 3.5 shows types of geosynthetics used in geotechnical engineering.

### **3.3.8 Chemical stabilization**

The aim of chemical stabilization on the expansive soils, is to change the nature of the clay by interaction between clay minerals and chemical materials such as fly ash, cement, lime or combination of these in small quantities. According to the literature, chemical stabilization decrease the swelling potential and gives an improved behavior to the expansive soil. In this study fly ash and lime are used as the chemical stabilizer materials.

#### **3.3.8.1 Cement stabilization**

Cement is increasingly used as a stabilizing material for soils, particularly for the construction of highways and earth dams. It can be used to treatment sandy and clayey soils.

The hydration of Portland cement is a complex pozzolanic reaction that produces a variety of different compounds and gels. As in the case of lime, the cement has an effect to decrease the liquid limit and to increase the plasticity index, workability of clayey soils and the potential of volume change (Chen F. H.,1988).

For clayey soils, the cement stabilization is effective when fine fractions (passing No. 200 sieve) are less than about 40 %, the liquid limit is less than 45 to 50, and plasticity index is less than about 25 (Murthry, 2002). The optimum requirements of cement by volume for an effective stabilization of various types of soil are given in Table 3.3. Cement stabilization increases the strength of soils and the strength increases with the curing time. (Fattah et al., 2010)

**Table 3.3 :** Cement requirement by volume for an effective stabilization of various soils (Das, B. M., 2011)

Soil		
AASHTO Classification System	Unified Soil Classification System	Percent cement by volume
A-2 and A-3	GP, SP and SW	6 – 10
A-4 and A-5	CL, ML and MH	8 – 12
A-6 and A-7	CL, CH	10 – 14

Comparing with lime, Portland cement is not as effective as lime in stabilization of highly plastic clays. Some clay soils have such a high potential for water absorption that the cement may not hydrate sufficiently to produce the complete pozzolanic reaction. Generally using of cement is recommended when soils are not reactive (Mitchell and Raad, 1973).

### 3.3.8.2 Fly ash stabilization

Fly ash by itself has little cementitious value but in the presence of moisture it reacts chemically and forms cementitious compounds and attributes to the improvement of strength and compressibility characteristics of soils. It has a long history of use as an engineering material and has been successfully employed in geotechnical applications (Bhuvaneshwari et al., 2005). Fly ash consists of often hollow spheres of silicon, aluminium and iron oxides and unoxidized carbon. There are two major classes of fly ash, class C and class F. The former is produced from burning anthracite or bituminous coal and the latter is produced from burning lignite and sub bituminous coal. Both the classes of fly ash are pozzolans, which are defined as siliceous and aluminous materials (Çokça, 2001). By adding fly ash to an expansive soil, cation exchange process results in reduction of plasticity, activity and swell potential. With respect to the stable exchangeable cations provided by the fly ash, the time-dependent cementation process (pozzolanic reaction) results in formation of cemented compounds characterized by their high shear strength and low volume change (Nalbantoglu and Tuncer 2001; Nalbantoglu and Gucbilmez 2002). Figure 3.6 shows the mixing and shaping process of fly ash stabilized soil.



**Figure 3.6 :** Mixing and shaping of fly ash stabilized soil ([www.fhwa.dot.gov](http://www.fhwa.dot.gov)).

### 3.3.8.3 Lime stabilization

Lime stabilization commonly is used in clayey soils for improvement of the embankments and the road constructions. Adding lime to the fine materials causes to a reduction in the plasticity index of the soil. Also the pozzolanic reaction between lime and clay's minerals results in the cementation of the soil which rises the strength of the soil. Depending on the environmental conditions such as the water content and the temperature, as a function of the time, the strength of soil is increased by passing the time. In general, lime stabilization increases the strength and shape module of deformation in the soil and decreases the swelling potential (Özaydın, 1999). Figure 3.7 shows a lime stabilization used in a road construction.



**Figure 3.7 :** Lime stabilization in a road construction  
(<http://www.hiwaystabilizers.co.nz>)

Some benefits of lime stabilization can be listed as below:

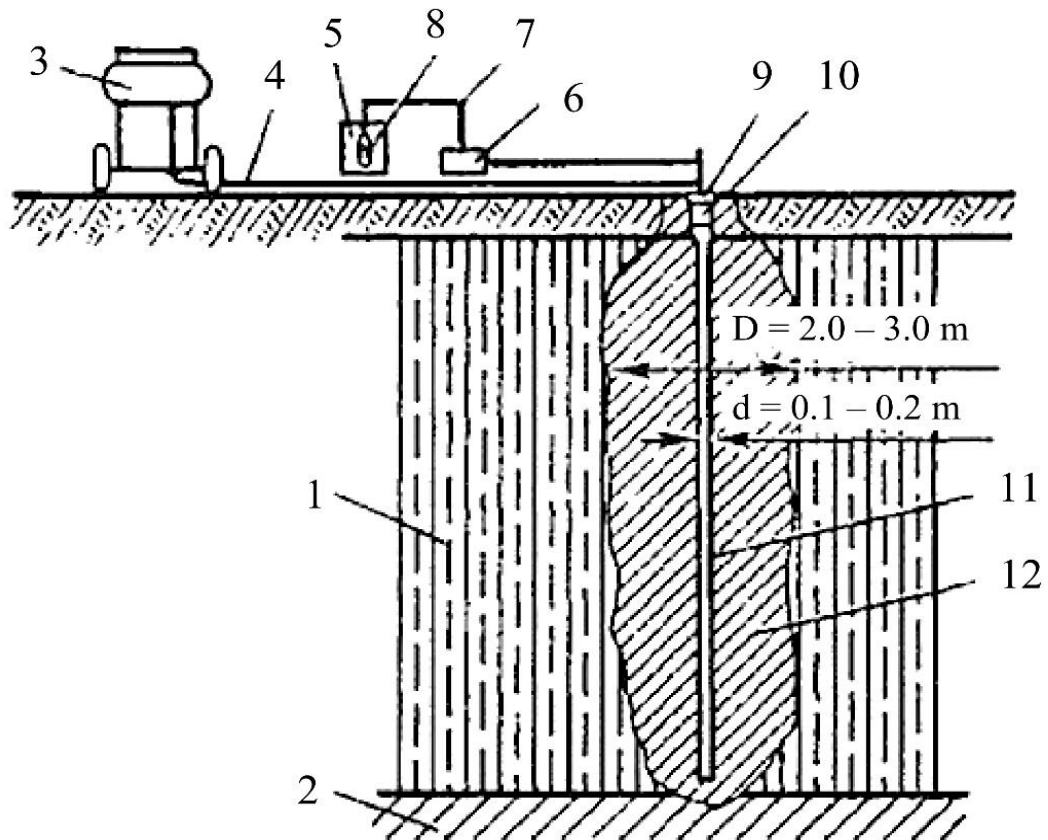
- Reduces the risk of costly subgrade pavement failures.
- Creates a more uniform pavement layer.
- Reduces the soils moisture content and increases its workability.
- Eliminates or reduces the necessity for undercutting and the associated cost and environmental impact that transportation and disposal of unsuitable material involves.
- Reduces construction time.
- Is a proven, cost effective and durable pavement component.
- Reduces transport demands on existing roading infrastructure by reducing aggregate and "undercutting-to-waste" requirements.
- Provides a superior working platform especially during winter construction.

Mixing procedures for lime and cement stabilization is similar to the Portland cement application but in case of the Portland cement, it has a shorter hydration and setting time, so one difference in technique is that the time between cement addition and final mixing should be shorter than that used for lime treatment (Portland Cement Association, 1970).

### **3.3.9 Thermal treatment**

Basically, thermal treatment consists of driving exhausted gases from burning oil at temperatures around 1000°C, into holes in the ground. Depth of treatment can be approached up to 20.00m. Thermal stabilization can be applied in two main open-firing and closed-firing techniques. In open-firing method, two holes are bored in the soil so that they intersect. The combustion nozzle is placed over one and the combustion of gases exit from the other. In the closed-firing method a single hole closed system is used in which the burner temperature is controlled by maintaining an excess air pressure. Figure 3.8 illustrates a scheme for deep thermal stabilization of soil.

The temperature distribution with depth in the soil depends on its porosity, the water content, the excess pore-water pressure and the temperature of the gases injected. The temperature does not exceed 100°C until the free water enclosed in the pores of the soil completely evaporated. In the soils containing large proportion of organic collides, the collide minerals react at low temperature. Thermal stabilization cannot be applied to soils that are saturated, because the latent heat of evaporation of water make it too expensive.



**Figure 3.8 :** Scheme for deep thermal treatment of soil (Litvinov, 1960): 1. collapsible loess soil; 2. non-collapsible loess soil; 3. compressor; 4. cold air pipeline; 5. liquid fuel container; 6. pressurized fuel pump; 7. Fuel pump line; 8. Filters; 9. Nozzle; 10. combustion chamber; 11. borehole; 12. Stabilized zone

Clay soils harden on heating and if heated to a high temperature they remain hard. It is because of the fact that changes occur in the crystalline structure of the clay minerals above 400°C, notably the loss of the (OH) group. A remarkable reduction occurs in the plasticity index of a clay if it is heated to 400°C. Also the moisture absorption capacity of clay appreciably reduced after it has been heated to 600°C. The permeability of a clay soil increases on heating up to 600°C or 700°C, above which, due to the onset fusion, it decreases slightly. By thermal treatment, the swelling potential of clay is reduced as its compressibility (Bell, 1993).





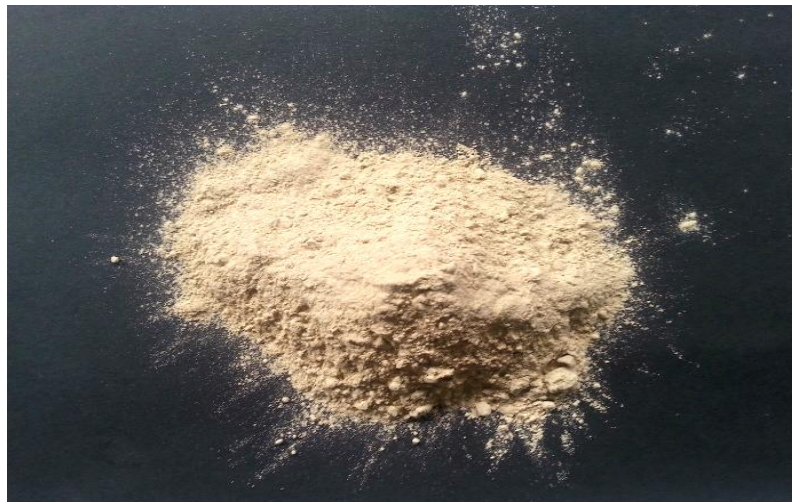
## **4. DETERMINATION OF SWELLING POTENTIAL OF STABILIZED EXPANSIVE SOILS**

### **4.1 Materials Used in Laboratory Tests**

A comprehensive laboratory investigation has been performed to evaluate the effect of the copolymer, the homopolymer polypropylene, the fly ash and the lime to reduce swelling potential of the bentonite. All mentioned additive materials were combined with the bentonite in various percentages and related experiments were performed to determine the success of the improvement.

#### **4.1.1 Bentonite**

Bentonites are the clay rocks altered from glassy igneous materials such as volcanic ash or tuff (Grim and Güven, 1978). In this investigation, a high plasticity sodium bentonite was used as the expansive soil to evaluate the effect of additive materials on the expansion potential of the expansive soils. Figure 4.1 shows the bentonite which was taken from Canbensan Bentonite Company located in 70 km away from Ankara, Turkey. The physical properties of the bentonite which are shown in Table 4.1 indicate that the bentonite predominately contains montmorillonite which is one of the main factors for swelling of the clay soils. Chemical properties of bentonite also is summarized in Table 4.2 while grain-size distribution is exhibited in Figure 4.2.



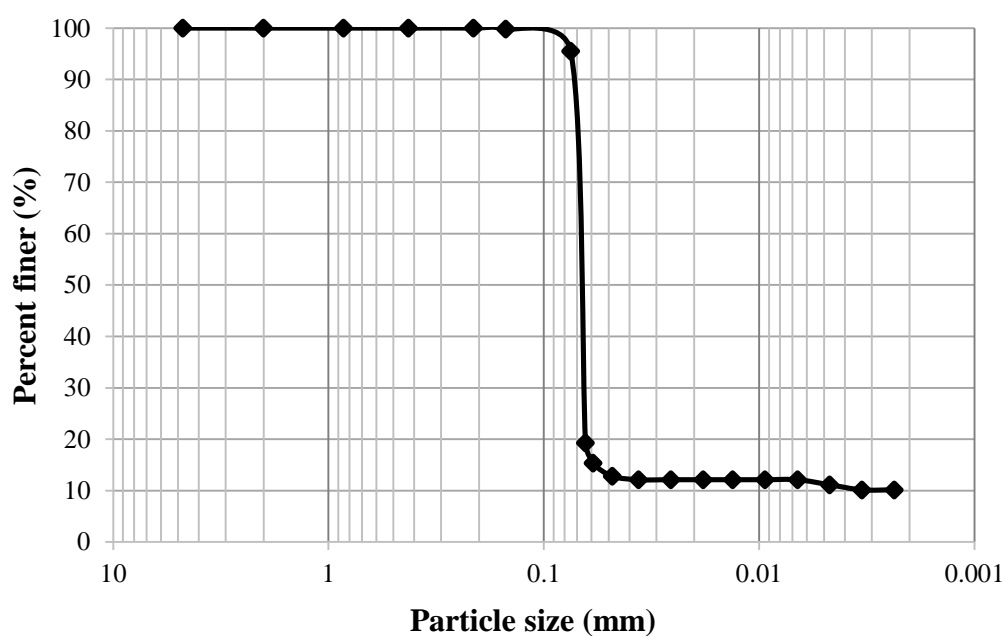
**Figure 4.1 :** The bentonite used in the laboratory tests.

**Table 4.1 : Physical properties of bentonite**

Properties of bentonite	Value
Color	yellow
Methylene Blue Value	340 mg/gram
Montmorillonite Content	> 85%
Amount of Moisture	11% (on dry substance)
API Water Loss	14 ml
Sieve Analysis	90% pass the No. 200 sieve
PH (in 6.5% mud)	10.4
Minimum Application Temperature	1 °C

**Table 4.2 : Chemical properties of bentonite**

Parameters	Value (%)
SiO <sub>2</sub>	59 – 61
Al <sub>2</sub> O <sub>3</sub>	18 – 20
MgO	2.5 – 3.5
K <sub>2</sub> O	0.5 – 1.5
Fe <sub>2</sub> O <sub>3</sub>	4 – 6
CaO	0.5 – 1.5
Na <sub>2</sub> O	2 – 3
TiO <sub>2</sub>	0.5 – 1.5

**Figure 4.2 : Grain-size distribution of Bentonite.**

### 4.1.2 Copolymer fiber

Copolymer fiber is made of 100% virgin materials consisting of a twisted fibrillating network fiber, yielding a high-performance concrete reinforcement system. The extra heavy-duty fiber offers maximum long-term durability, structural enhancements, and effective secondary/temperature crack control by incorporating a truly unique synergistic fiber system of long length design. Table 4.3 demonstrates the physical properties of the copolymer.

**Table 4.3 : Physical properties of copolymer**

Properties of copolymer	Value
Color	Gray
Form	Monofilament Fiber
Acid/Alkali Resistance	Excellent
Specific Gravity	0.91
Absorption	Nil
Tensile Strength	758
Length	54 mm
Compliance	ASTM C-1116

The aim of using copolymer is to inhibit plastic and settlement shrinkage cracking prior to the initial set, and to reduce hardened concrete shrinkage cracking, improve impact strength, and enhance concrete toughness and durability as an alternate secondary/temperature/structural reinforcement. Both twisted monofilament network form and deformed form of copolymer fibers are demonstrated in Figure 4.3.



(a) Copolymer fibers



(b) Deformed copolymer fibers

**Figure 4.3 : Copolymer fibers in : (a) Copolymer fibers, (b) Deformed copolymer fibers.**

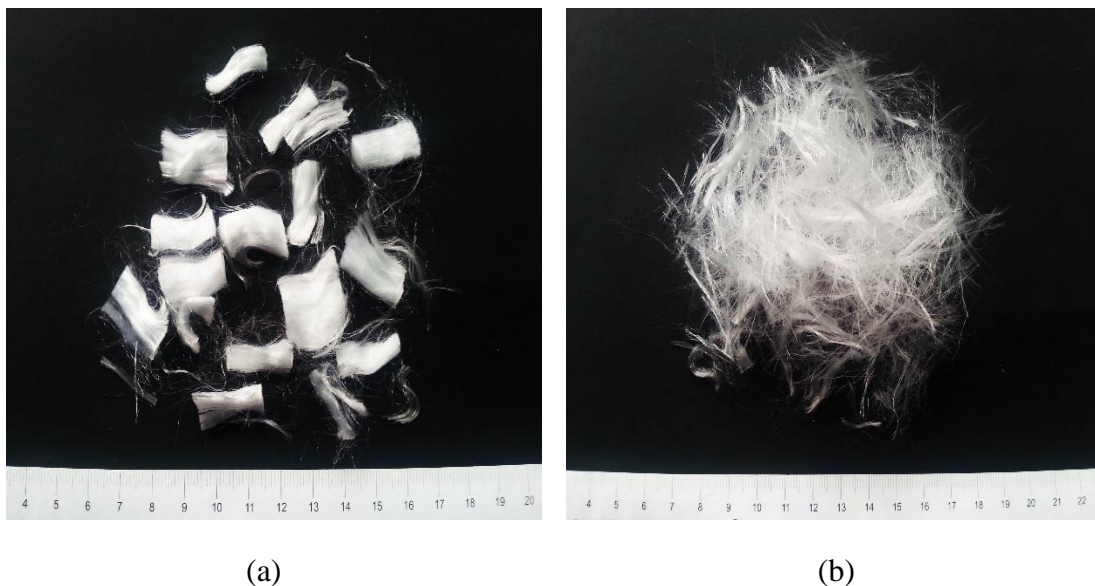
### 4.1.3 Homopolymer polypropylene

Virgin Homopolymer polypropylene fiber is made of 100% virgin homopolymer polypropylene monofilament fibrous reinforcement. This fiber offers long-term durability, and increased impact resistance. Homopolymer polypropylene fiber is used to reduce plastic and settlement shrinkage. Table 4.4 demonstrates the physical properties of homopolymer polypropylene.

**Table 4.4 :** Physical properties of homopolymer polypropylene

Properties of HPP	Value
Color	White
Form	Monofilament Fiber
Acid/Alkali Resistance	Excellent
Specific Gravity	0.91
Absorption	Nil
Tensile Strength	758
Length	15 mm
Compliance	ASTM C-1116

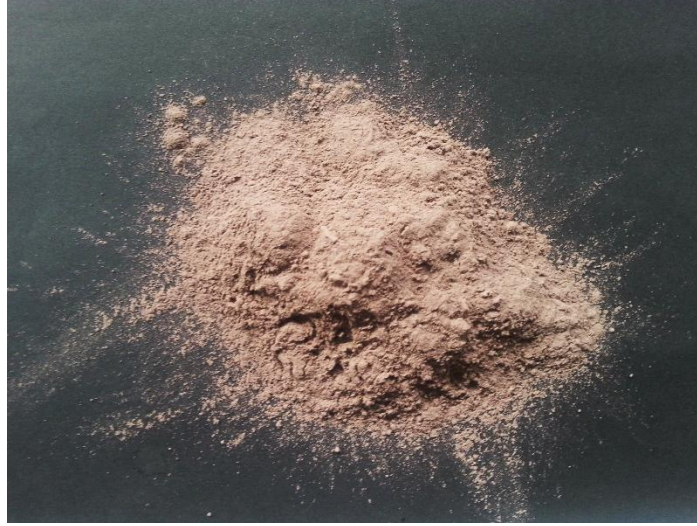
The objective of using homopolymer polypropylene is to inhibit concrete cracking caused by plastic and settlement shrinkage that occurs prior to initial set. Both twisted monofilament network form and deformed form of homopolymer polypropylene fibers are demonstrated in Figure 4.4.



**Figure 4.4 :** HPP fibers in (a) fibrillated form and (b) deformed form.

#### 4.1.4 Fly ash

As the first chemical stabilizer, Fly ash was used in the tests. Figure 4.5 shows the utilized fly ash obtained from Çayırhan power station located on 120 km of Ankara, Turkey. Table 4.5 demonstrates the chemical characteristics of class C fly ash used in the tests, while Table 4.6 shows the physical properties of it.



**Figure 4.5 :** The Fly ash used in the laboratory tests

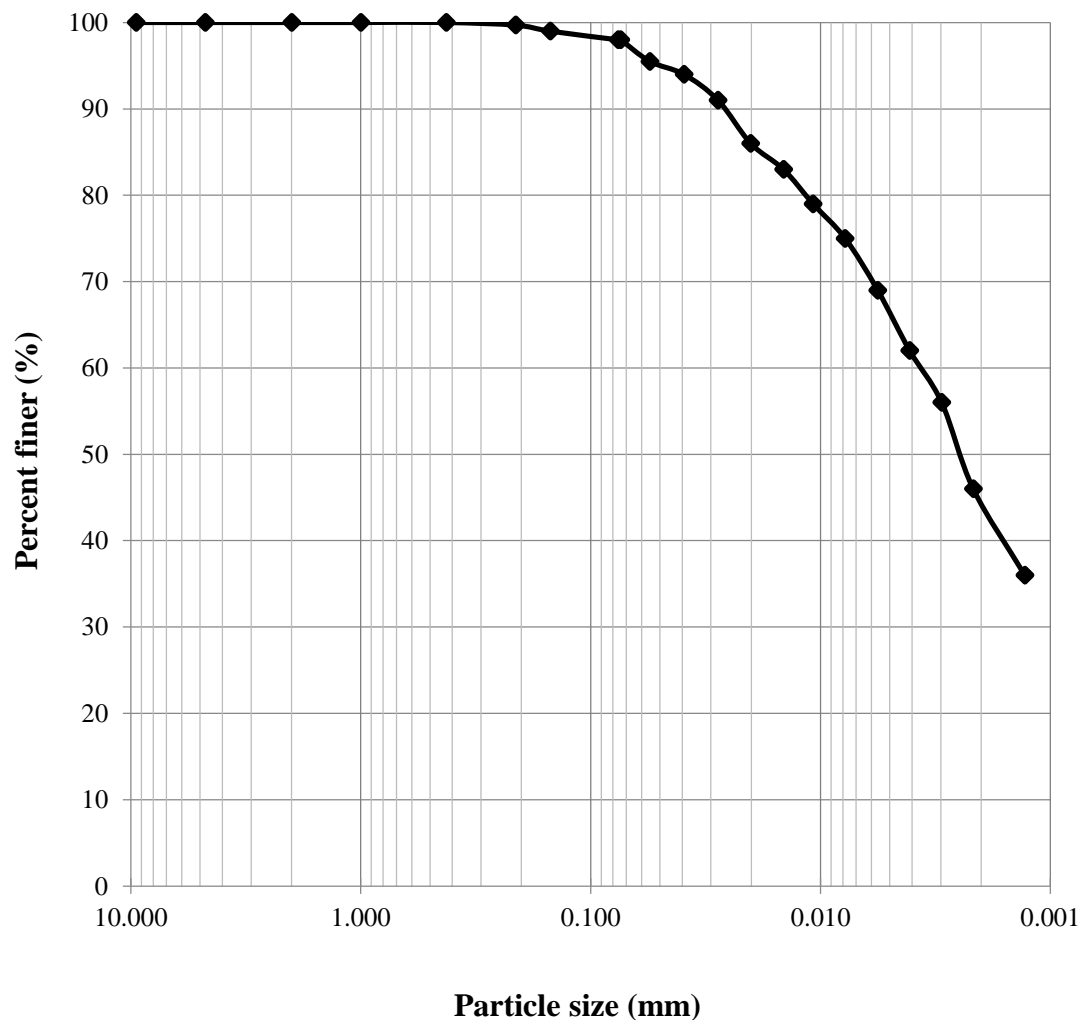
**Table 4.5 :** Chemical properties of fly ash

Parameter	Value (%)
$\text{CaCO}_3 + \text{MgCO}_3$	1.25
$\text{H}_2\text{O}$	0.20
$\text{SiO}_2$	45.00
$\text{Al}_2\text{O}_3$	13.90
$\text{Fe}_2\text{O}_3$	8.26
$\text{CaO}$	15.11
$\text{MgO}$	6.68
$\text{SO}_3$	4.26
$\text{Na}_2\text{O}$	2.13
$\text{K}_2\text{O}$	2.78
$\text{Cl}$	0.06
Loss on ignition	0.22
S.CaO	0.15
TOTAL	100%

**Table 4.6 :** Physical properties of fly ash.

Specific surface (cm/gr)	Specific gravity (gr/cm <sup>3</sup> )	Activity Index (%)
2100	2.34	83

Both sieve analysis and hydraulic analysis tests have been performed on the fly ash and the results are illustrated as the grain-size distribution curve in Figure 4.6.



**Figure 4.6 :** Grain-size distribution of fly ash.

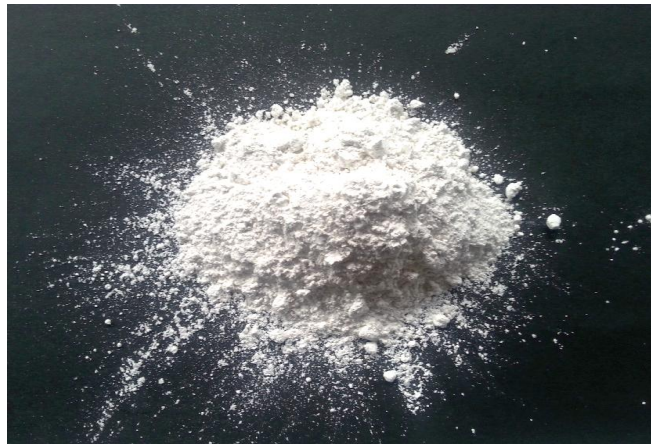
#### 4.1.5 Lime

Figure 4.7 exhibits the lime which was used as the second chemical alternative. The lime is obtained from Kimtas Kirec Company located in Izmir, Turkey. The X-Ray method was used to determine the chemical composition of the lime shown in Table 4.7. Also, the grain-size distribution curve of lime exhibited in Figure 4.8.

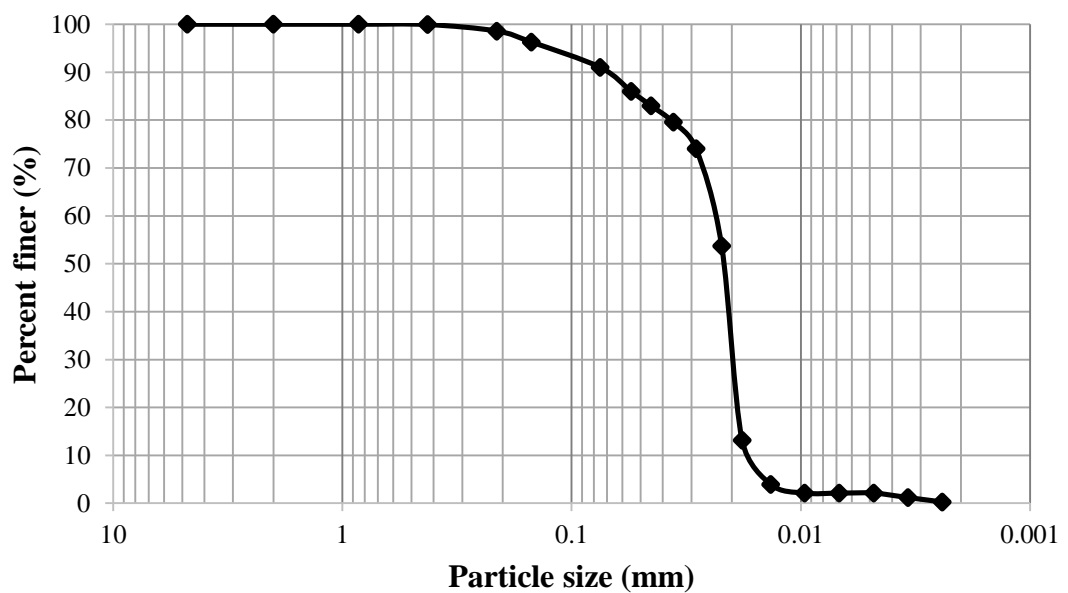


**Table 4.7 :** Chemical properties of lime.

Major parameters	Value (%)	Minor Parameters	Value (%)
SiO <sub>2</sub>	0.36	V	0.0061
Al <sub>2</sub> O <sub>3</sub>	0.14	Zn	0.0013
Fe <sub>2</sub> O <sub>3</sub>	0.13	Rb	0.0005
MnO	0.02	Sr	0.0192
CaO	65.45	Y	0.0003
MgO	0.51	Zr	0.0017
K <sub>2</sub> O	0.02	Mo	0.0003
P <sub>2</sub> O <sub>5</sub>	0.03	Ce	0.0126
LOI (Loss on ignition)	33.35	<b>Total</b>	<b>100</b>



**Figure 4.7 :** The lime used in the laboratory tests



**Figure 4.8 :** Grain-size distribution of lime.

## 4.2 Laboratory Tests

The experimental testing program was performed during 2012-2013 years in Istanbul Technical University's civil engineering faculty in the geotechnical engineering laboratory called Prof. Dr. Hamdi Peynircioğlu laboratory. The maximum dry unit weight and the related optimum water content of samples determined by using standard proctor compaction apparatus. Totally 70 various laboratory tests consisting of 1 pycnometer test, 3 sieve analyses, 3 hydrometer analyses, 11 liquid limit tests, 8 plastic limit tests, 24 standard proctor compaction tests and 20 one dimensional swelling tests were performed. It should be considered that in order to get more accurate and reliable results, whenever that, there was a suspicion about the results of any test, the same test was put into experiment once again. Also for checking the results, some of the tests were repeated randomly.

### 4.2.1 Sieve analysis and hydrometer test

For classification the main and additive materials which are used in the laboratory testing program, the sieve and the hydrometer analysis were performed on the bentonite, fly ash and lime. With respect to the results of the tests and considering USCS, all three materials were categorized in the fine material group. Figure 4.9 shows the hydrometer test of bentonite and lime. Unlike the bentonite, sedimentation of the lime is about completed at the end of the test.

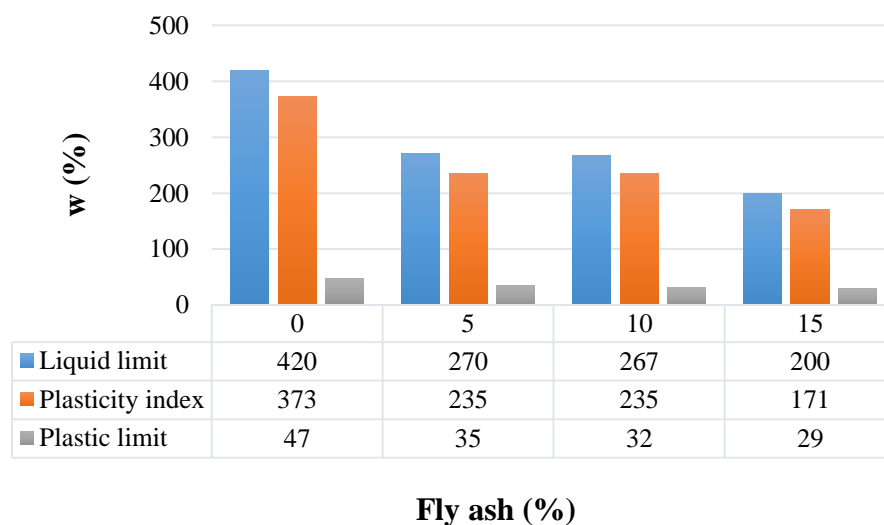


**Figure 4.9 :** Hydrometer analysis of bentonite (right) and lime (left).

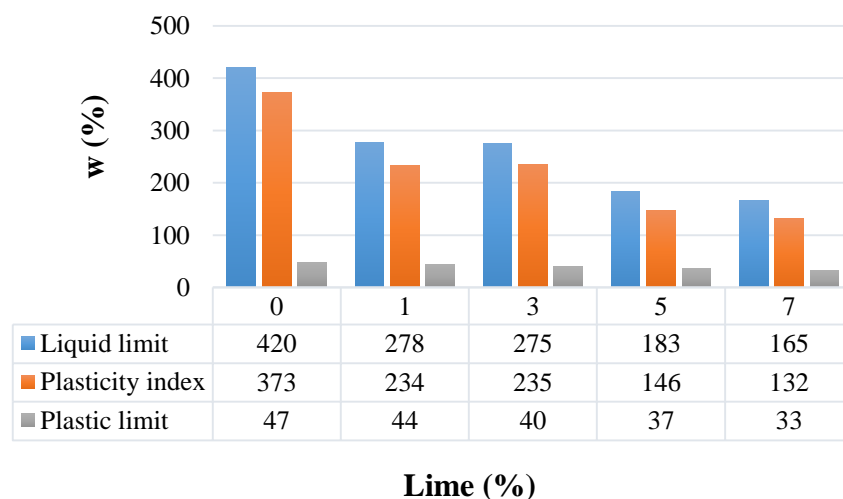


#### 4.2.2 Atterberg limits test

One of the main steps for evaluating the geotechnical properties of the fine materials is to determine the Atterberg limits. In this study the Atterberg limits of bentonite and two chemical additive stabilizers consisting fly ash and lime are explored according to the ASTM-D4318 standard. According to the results, fly ash inclusion cause reduction in liquid limit and plastic limit. The plasticity index also decreases. In the case of lime, both liquid limit and plastic limit decrease which cause a reduction in plasticity index. The soil becomes more workable by reduction of plastic properties and due to formation of cementing material. Figures 4.10 and 4.11 illustrate the effect of chemical stabilizers on the Atterberg limits of the bentonite.



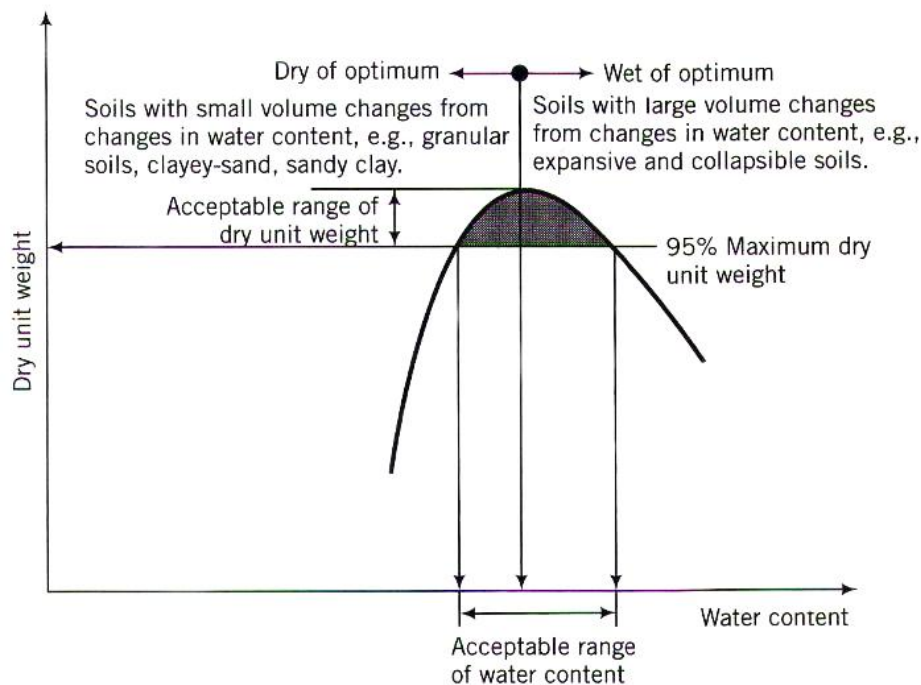
**Figure 4.10 :** Atterberg limits of Bentonite – Fly ash mixtures



**Figure 4.11 :** Atterberg limits of Bentonite – Lime mixtures

### 4.2.3 Standard proctor compaction test

Compaction test indicates two main factors of the soil including the optimum water content and the maximum dry unit weight. The soil type and the applied energy on the soil layer are two main factors which affect on the compaction of the soil. In this investigation, the compaction parameters of samples were provided using the standard proctor compaction test in accordance with ASTM-D698 standard. To determine the maximum heave of the expansive soil, there was a need to provide the soil sample at the densest state, so initially the optimum water content of each bentonite-additive mixture was determined and then the samples were prepared by standard proctor compaction apparatus for measuring the expansion of the soil. Figure 4.12 illustrates the typical curve of the compaction test.



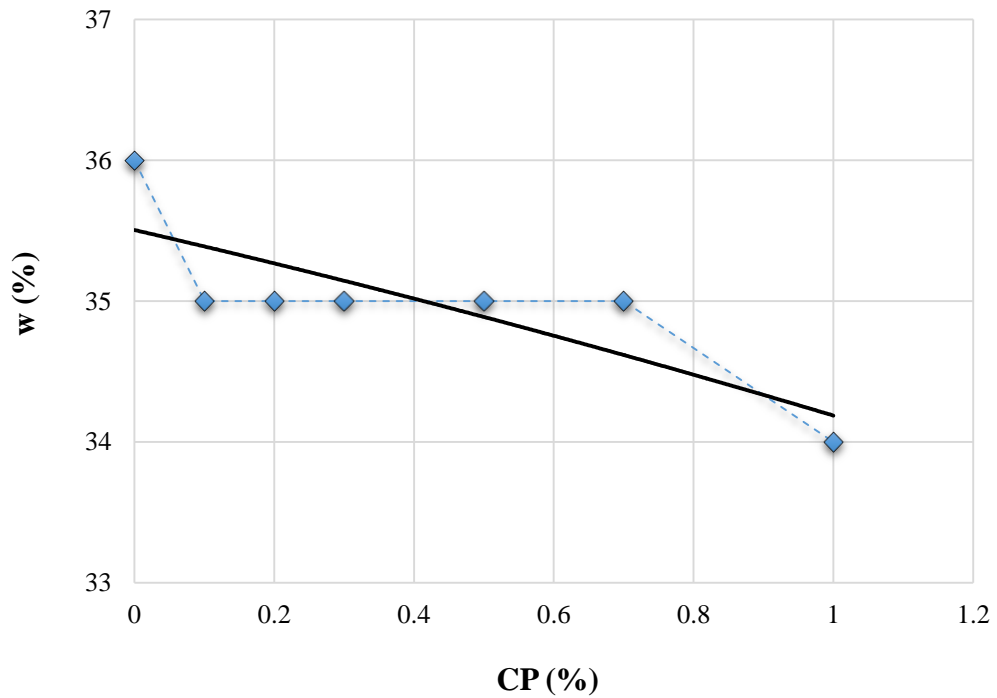
**Figure 4.12 :** Standard proctor compaction curve (Budhu, 2010).

According to the Figure 4.12, the compaction behavior of the soil can be divided in two section which separated by the optimum water content point. With respect to the Budhu, before the optimum point named as “dry of optimum”, there is a small volume changes due to the changes in the water content, but after the optimum point named as “wet of optimum”, there is a large volume changes for any small variation in the water content. The expansive soils stands in the second

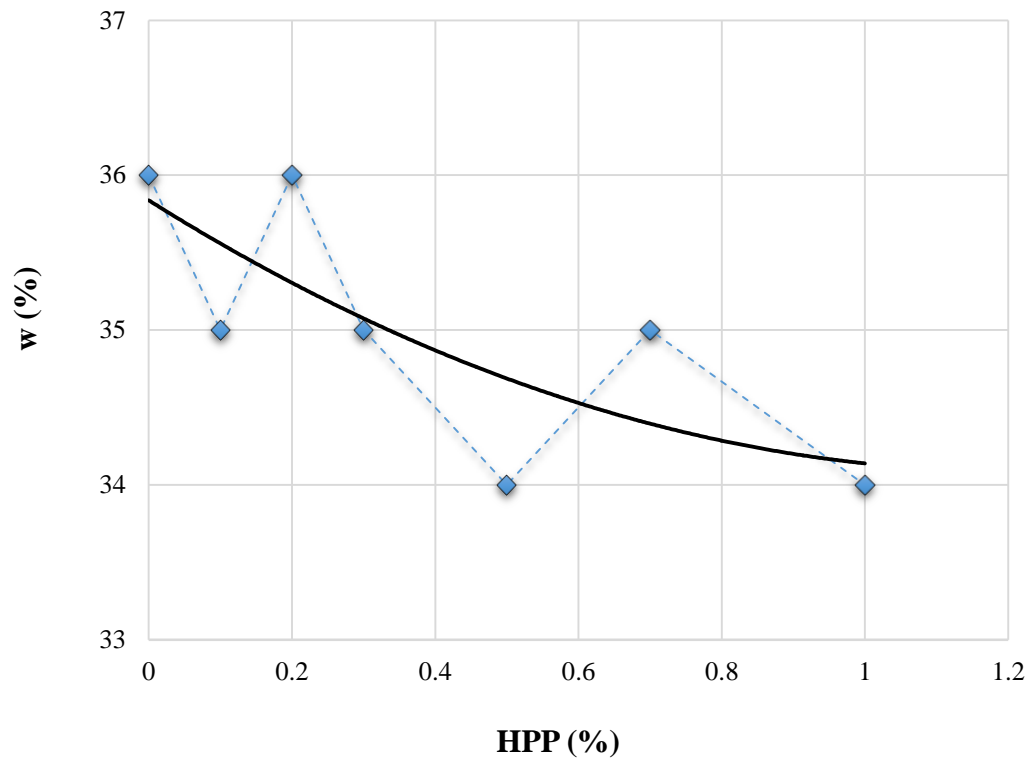
region and are so sensitive to the water content variation. Also the acceptable ranges of water content and dry unit weight are shown. It is difficult to prepare a sample exactly in the desired water content or dry unit weight. In this investigation all samples were provided considering the mentioned acceptable ranges.

#### 4.2.3.1 Effects of additive materials content on the optimum water content

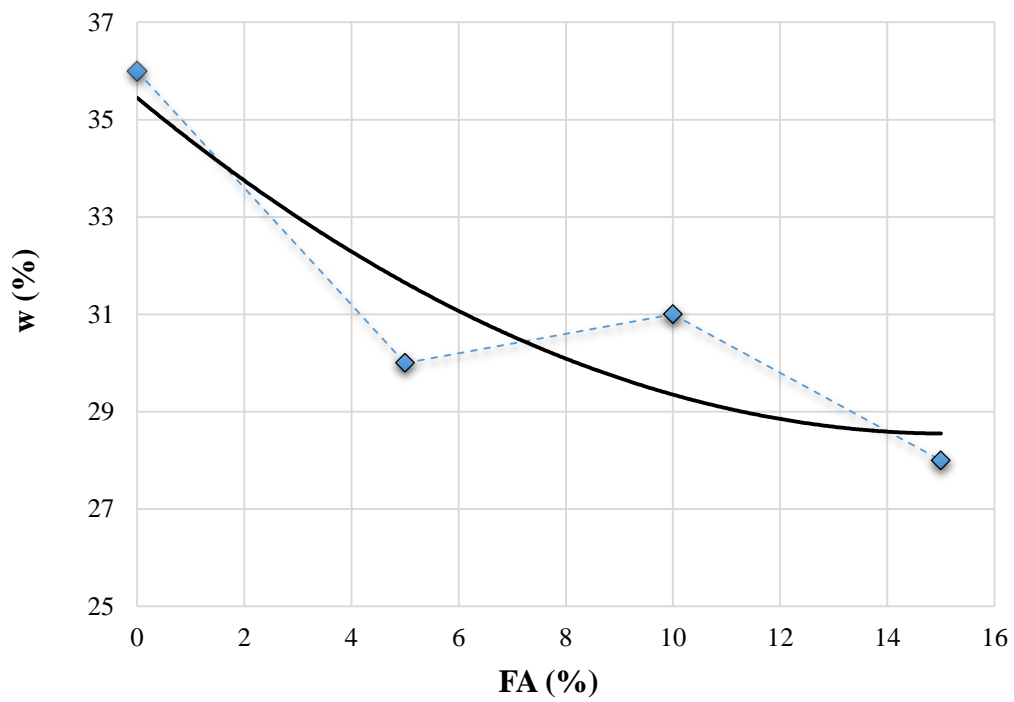
The optimum water content of bentonite and its mixtures with additive materials varies slightly by changing the type and amount of additive materials. There is not a large change in the optimum water content of the bentonite mixtures with fibers involving copolymer and homopolymer polypropylene, and also lime and it does not exceed 3%. The only remarkable variation in the water content occurs in presence of fly ash while it approaches to 8%. According to the results showed in the Figures 4.13-4.16, generally by increasing the copolymer, homopolymer polypropylene and fly ash contents, the optimum water content is decreased while this pattern is a bit different about lime and after a reduction, there is an increase in the optimum water content of bentonite-lime mixture.



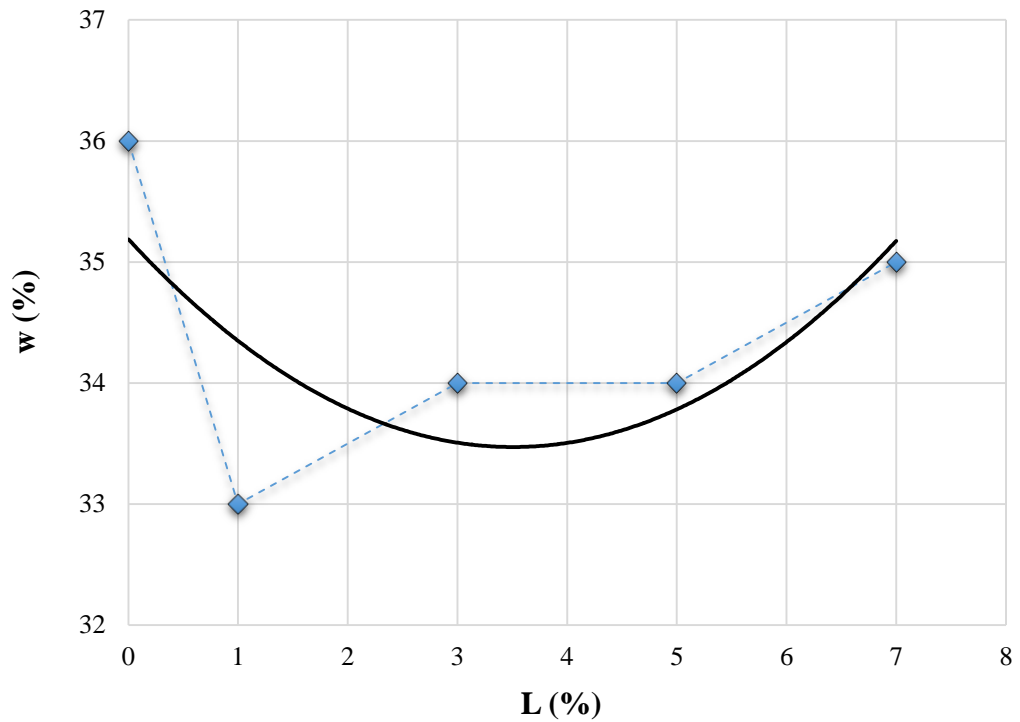
**Figure 4.13 :** Effect of copolymer content on the optimum water content of bentonite-copolymer mixtures.



**Figure 4.14 :** Effect of homopolymer polypropylene content on the optimum water content of bentonite-homopolymer polypropylene mixtures.



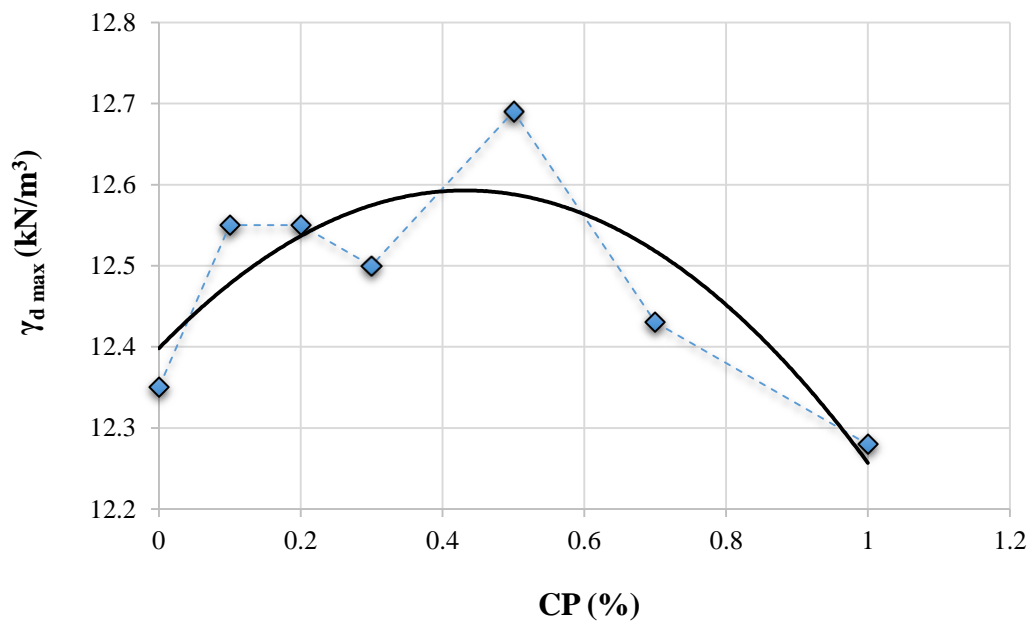
**Figure 4.15 :** Effect of fly ash content on the optimum water content of bentonite-fly ash mixtures.



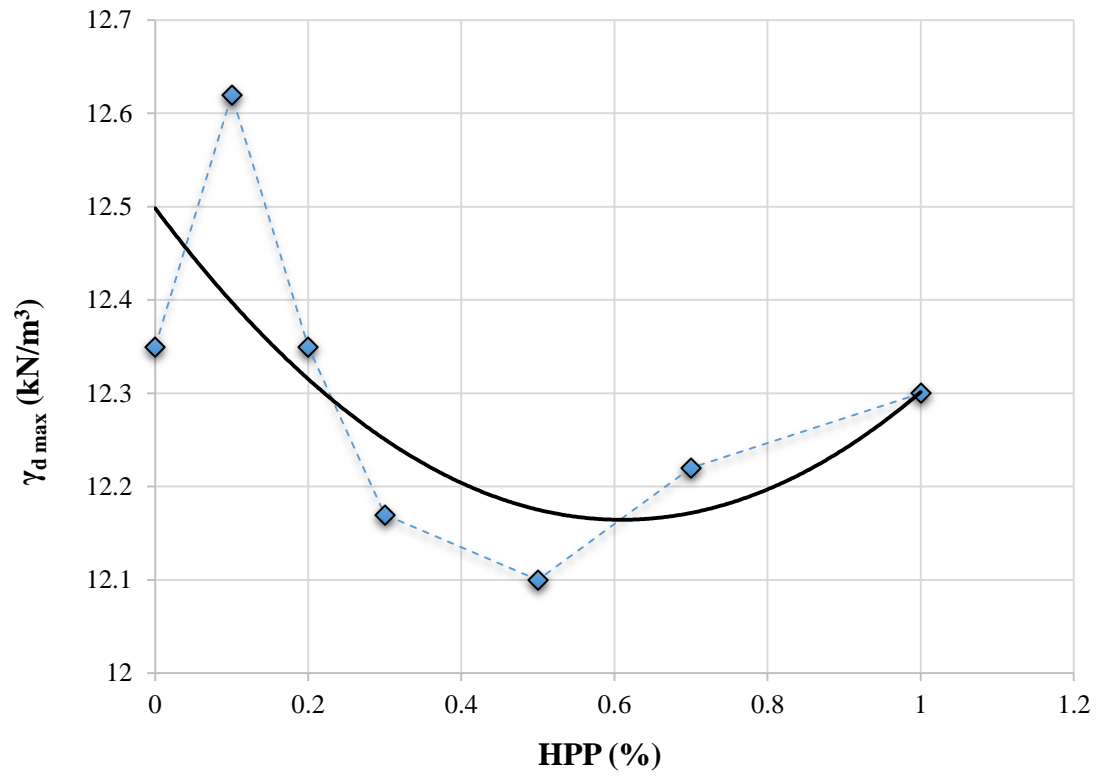
**Figure 4.16 :** Effect of lime content on the optimum water content of bentonite-lime mixtures.

#### 4.2.3.2 Effect of additive material content on max. dry unit weight

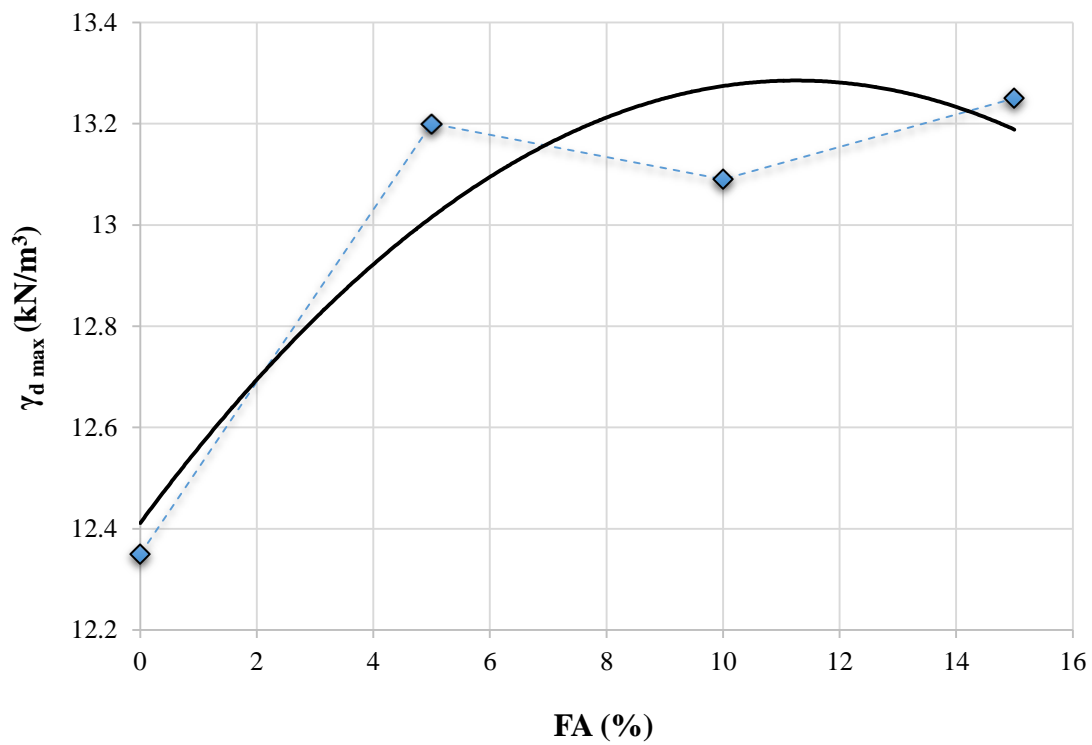
Figures 4.17-4.20 show the effect of additive material content on the maximum dry unit weight of bentonite. According to the results, except fly ash, by increasing the additive material content, the maximum dry unit weight of bentonite decreases.



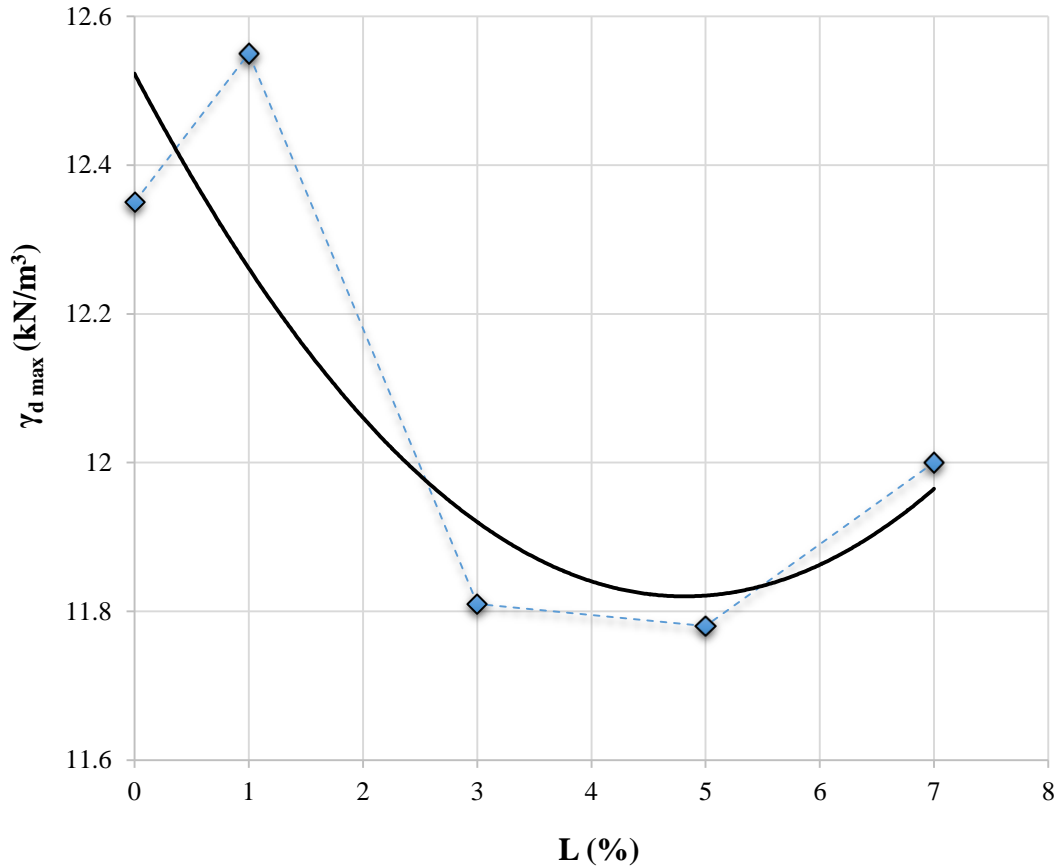
**Figure 4.17 :** Effect of copolymer content on max. dry unit weight of bentonite.



**Figure 4.18 :** Effect of homopolymer polypropylene content on max. dry unit weight of bentonite.



**Figure 4.19 :** Effect of fly ash content on max. dry unit weight of bentonite.



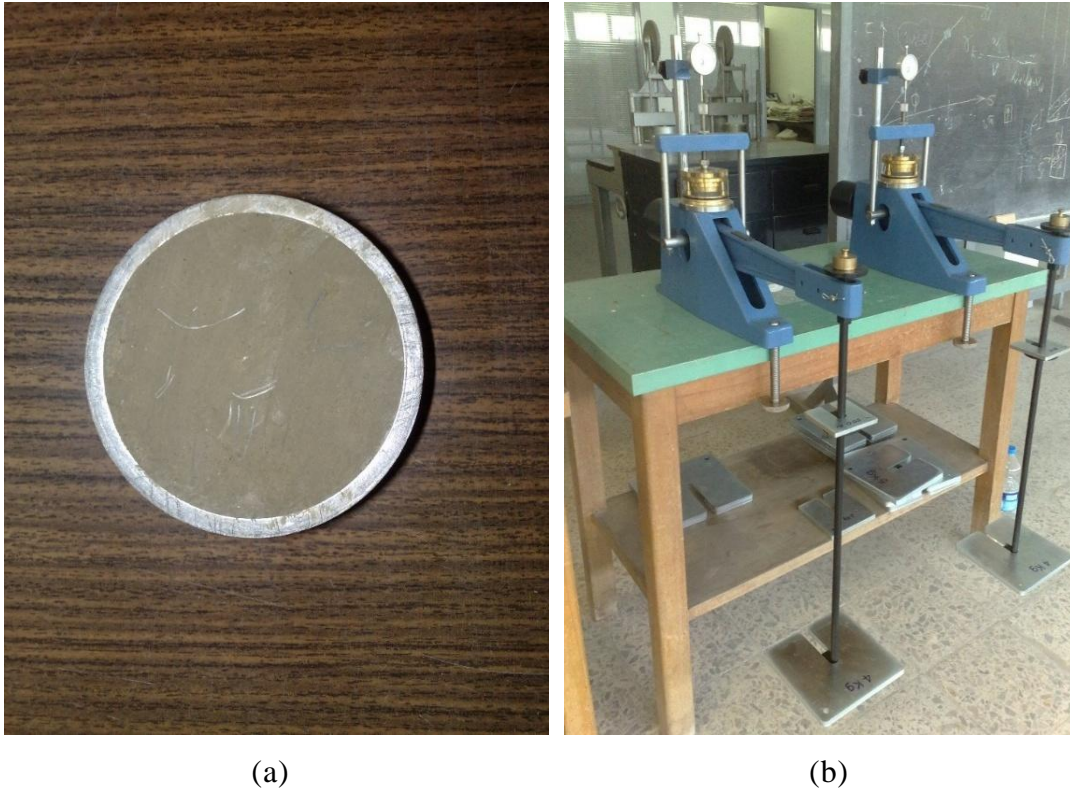
**Figure 4.20 :** effect of lime content on max. dry unit weight of bentonite.

#### 4.2.4 One-dimensional odometer swell test

For evaluating the swelling potential of bentonite and the effect of the additive materials on the swelling potential, one-dimensional swell test was performed using the C method of ASTM D-4546 standard. The method is based on the regular procedure of loading after wetting of the samples. In this method, the sample was provided at the optimum water content and the swelling pressure of the sample was measured by adding weight and keeping dial gage in the zero. The constant weight which keeps the dial gage at zero for 24 hours indicates the swelling pressure of the sample. Initially, the swell pressure of plain bentonite was determined and then it was compared with the swell pressures which were obtained from bentonite-additive materials mixtures prepared in difference percentages. Samples were prepared in the rings with 5.00 cm diameter. To obtain more accurate results and decrease the faults during the tests, for every percent of additive materials, two bentonite-additive mixtures were provided. Figures 4.21.a and 4.21.b show the samples prepared for the swell test and the one-dimensional swell test setup respectively.

#### 4.1 Evaluation of Laboratory Testing Data

All data obtained from the laboratory tests were analyzed to get a certain pattern for the behavior of treated expansive soil, to evaluate the effect of different stabilizers on the expansion potential of the bentonite.



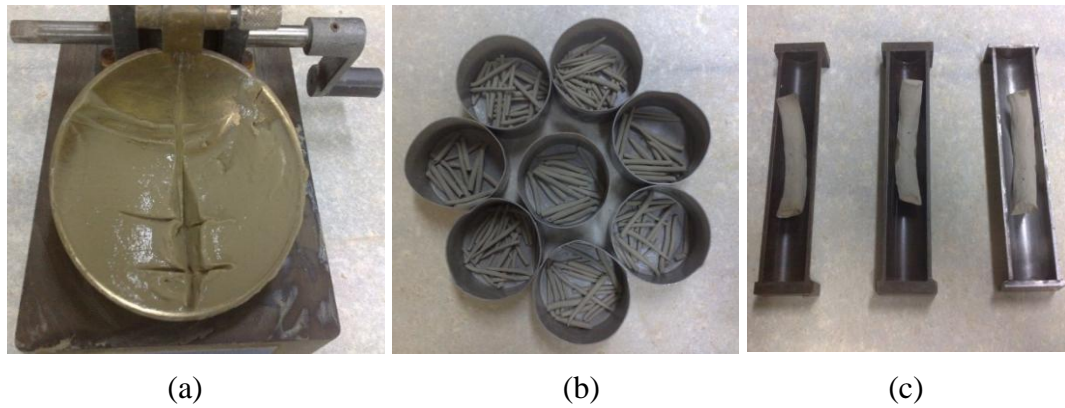
**Figure 4.21 :** (a) A bentonite-copolymer sample prepared for the swell test; (b) One-dimensional swell test setup.

##### 4.1.1 Plain bentonite

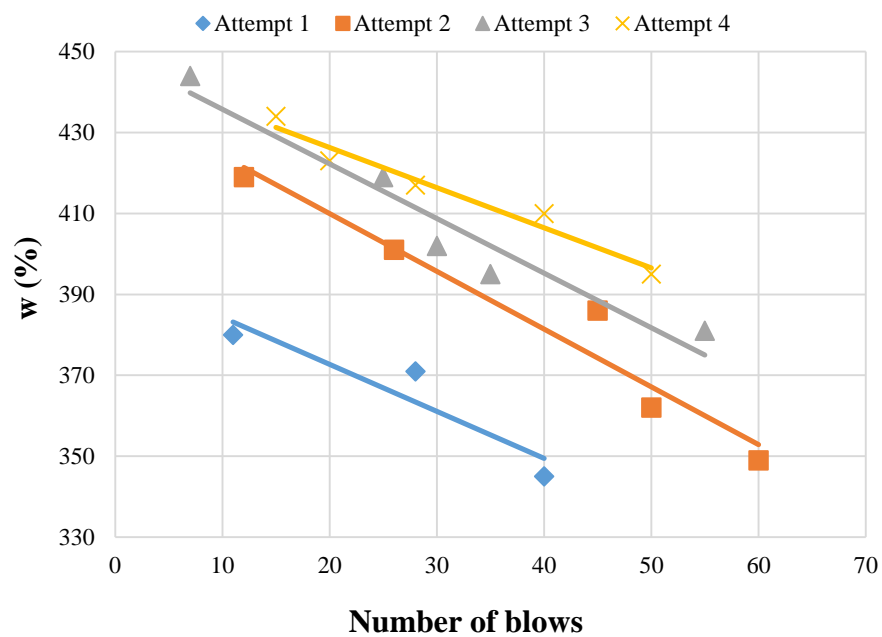
For investigating the effect of additive materials on bentonite, it is necessary to consider the initial properties of bentonite. As a fundamental geotechnical laboratory test for fine materials, the Atterberg limits test which shown in Figure 4.22 has been performed on the bentonite to evaluate the index properties. Also hydrometer and pycnometer tests were done for determining the grain size distribution and the specific gravity respectively.

With respect to sensitivity of bentonite for adsorbing water, several tests were done in order to obtain accurate values. Figure 4.23 demonstrates the results of four liquid limit tests and Table 4.8 indicates the geotechnical properties of bentonite which was used in the laboratory testing program.





**Figure 4.22 :** Atterberg limit tests; a) Liquid limit experiment performed on cassagrande apparatus, b) plastic limit test, c) shrinkage limit test.



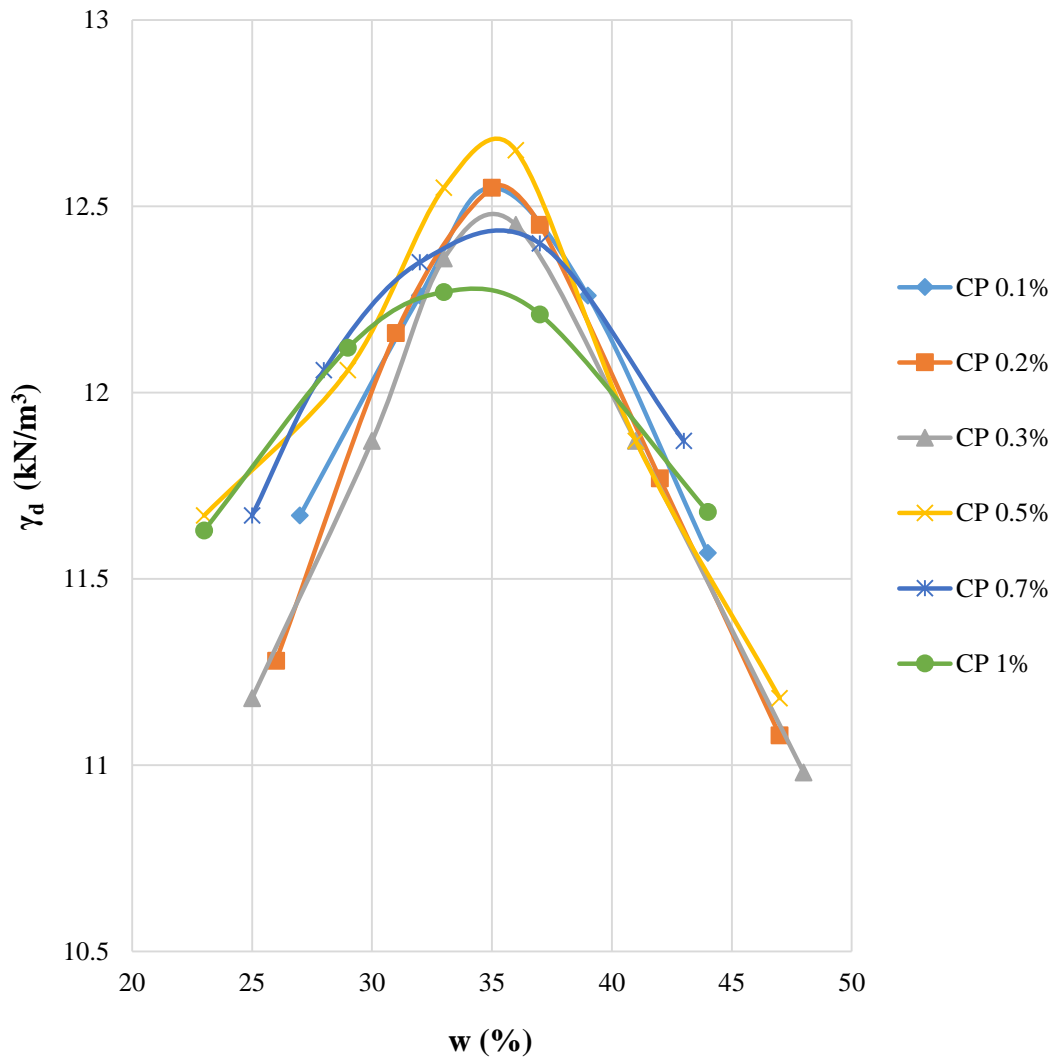
**Figure 4.23 :** Liquid limit test for plain bentonite.

**Table 4.8 :** Geotechnical properties of the bentonite.

Bentonite	
Liquid limit, LL (%)	420
Plastic limit, PL(%)	47
Shrinkage limit, SL (%)	39
Plasticity index, PI (%)	373
Optimum water content, $w_{opt}$ (%)	36
Maximum dry unit weight, $\gamma_{d \max}$ (kN/m <sup>3</sup> )	12.35
Specific gravity, $\gamma_s$ (kN/m <sup>3</sup> )	2.48
Soil type	CH

#### 4.1.2 Copolymer mixtures with bentonite

As a fiber stabilizer, copolymer was mixed with bentonite in six different percentages of 0.1%, 0.2%, 0.3%, 0.5%, 0.7% and 1%. The standard proctor compaction test has been performed on all bentonite-copolymer mixtures in the mentioned dosages to evaluate the optimum water content in every state. Figure 4.24 shows the results of standard proctor compaction test of bentonite-copolymer mixtures.



**Figure 4.24 :** Standard proctor compaction test of bentonite-copolymer mixtures.

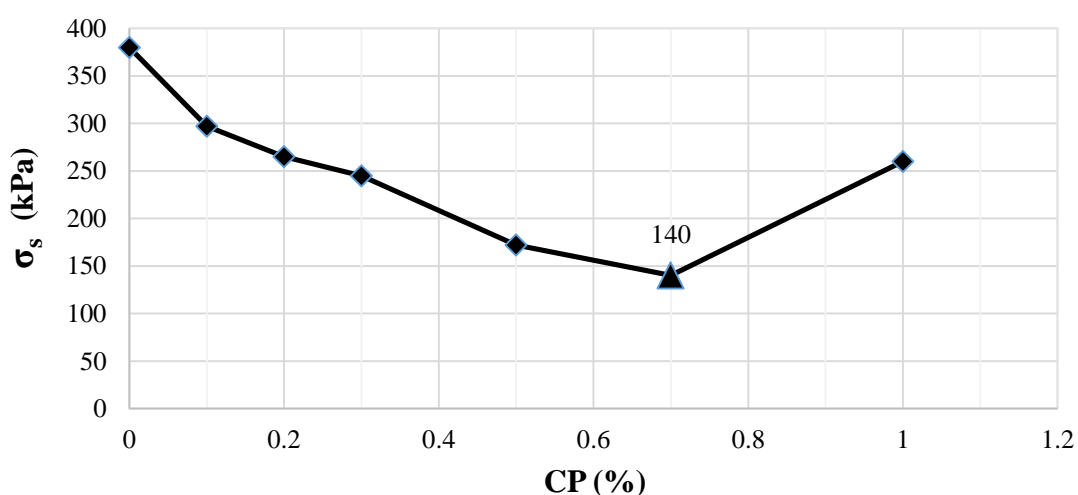
According to the results shown in Table 4.9, except copolymer 1% the optimum water content is same in all bentonite-copolymer mixtures and there is not any remarkable changes in the maximum dry unit weight.

**Table 4.9 :** Results of standard proctor compaction test for bentonite-copolymer mixtures.

Main material	Additive material	Additive material content (%)	Optimum water content (%)	Max. Dry unit weight (kN/m <sup>3</sup> )
Bentonite	–	–	36	12.35
Bentonite	CP	0.1	35	12.55
Bentonite	CP	0.2	35	12.55
Bentonite	CP	0.3	35	12.50
Bentonite	CP	0.5	35	12.69
Bentonite	CP	0.7	35	12.43
Bentonite	CP	1	34	12.28

One dimensional swell test has been performed for determining the effect of various amounts of copolymer on expansion potential of the bentonite. For getting more accurate results and decreasing the faults during the tests, for every percent of copolymer content, two bentonite-copolymer samples were provided in the related optimum water content.

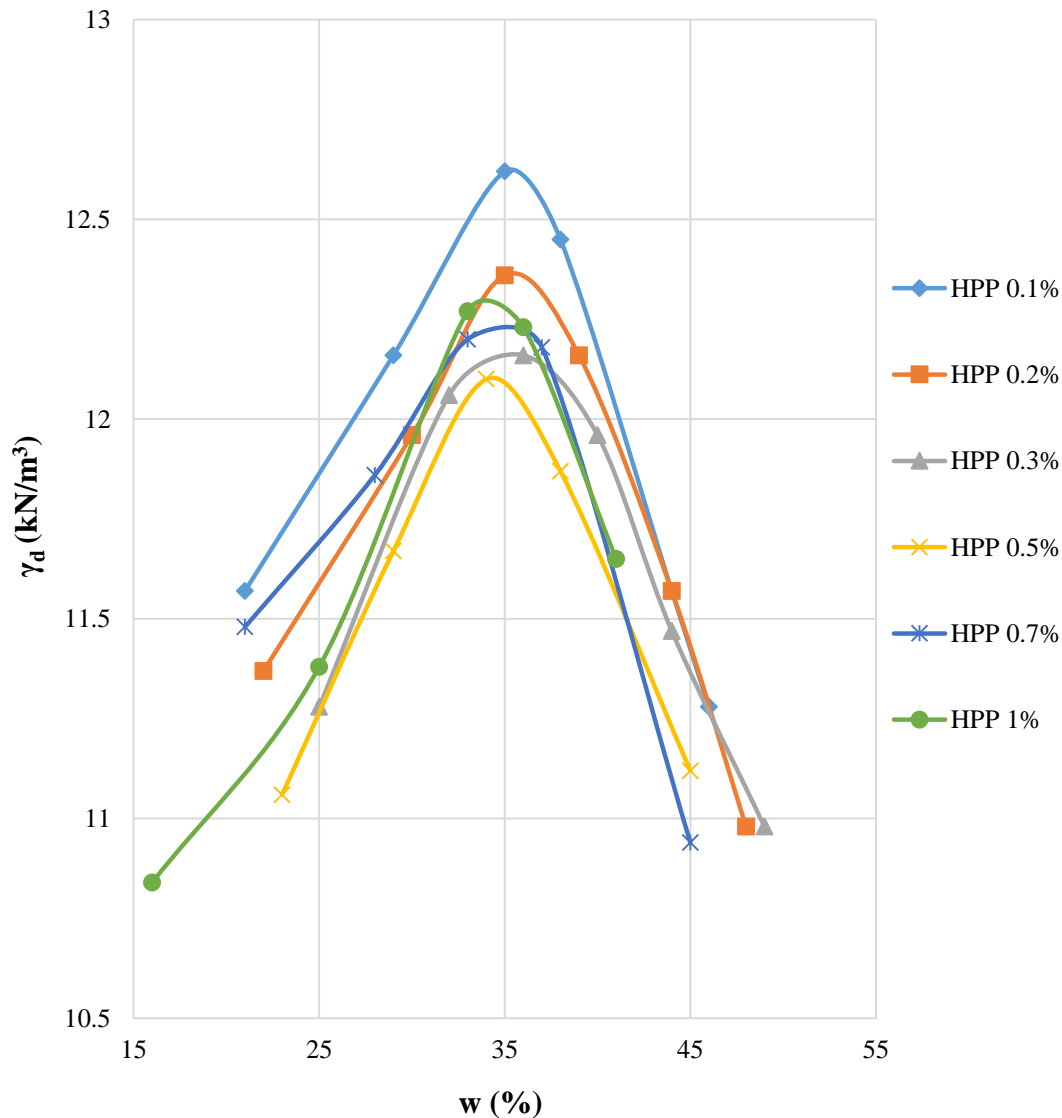
The results of one-dimensional swell test (Figure 4.25), indicates a significant reduction on the swell pressure of bentonite by adding copolymer fiber. To a particular point the more copolymer content the less expansion potential occurs and after that increasing copolymer content is not effective and heaving rises. Comparing with the plain bentonite, at the optimum point the swelling potential was reduced at most 64% while entire soil mass contained 99.93% bentonite and 0.7% copolymer.



**Figure 4.25 :** Results of one-dimensional swell test for bentonite-copolymer mixtures.

#### 4.1.3 Homopolymer polypropylene mixtures with bentonite

As the second fiber stabilizer in this investigation, homopolymer polypropylene (HPP) was used in 6 various percentages of 0.1%, 0.2%, 0.3%, 0.5%, 0.7% and 1% to mix with bentonite to reduce the swelling potential. As a part of evaluation procedure, standard proctor test was done and all mixtures are prepared at the optimum water content to determine the swelling behavior of stabilized samples. The results of standard proctor compaction tests are exhibited in Figure 4.26.



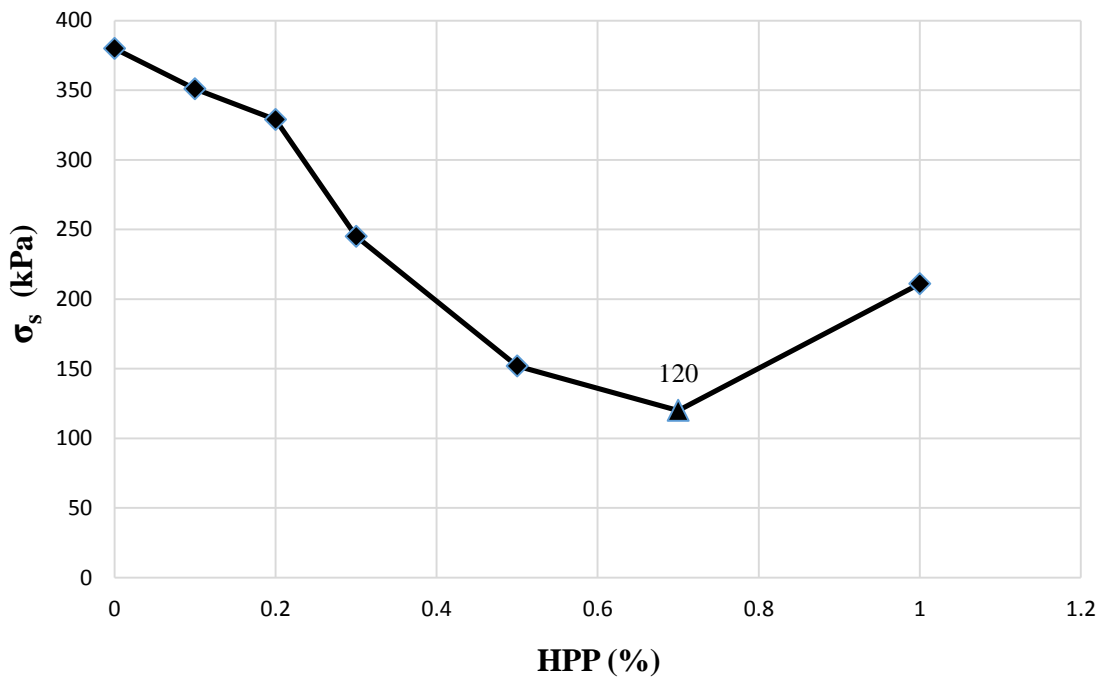
**Figure 4.26 :** Standard proctor compaction test for bentonite-homopolymer polypropylene mixtures.

According to the results of standard proctor compaction test (Table 4.10), there is not any significant change on the optimum water content and the maximum dry unit weight of mixtures among various bentonite-homopolymer polypropylene mixtures.

**Table 4.10 :** Results of standard proctor compaction test for bentonite-homopolymer polypropylene mixtures.

Main material	Additive material	Additive material content (%)	Optimum water content (%)	Max. Dry unit weight (kN/m <sup>3</sup> )
Bentonite	–	–	36	12.35
Bentonite	HPP	0.1	35	12.62
Bentonite	HPP	0.2	36	12.35
Bentonite	HPP	0.3	35	12.17
Bentonite	HPP	0.5	34	12.10
Bentonite	HPP	0.7	35	12.22
Bentonite	HPP	1	34	12.30

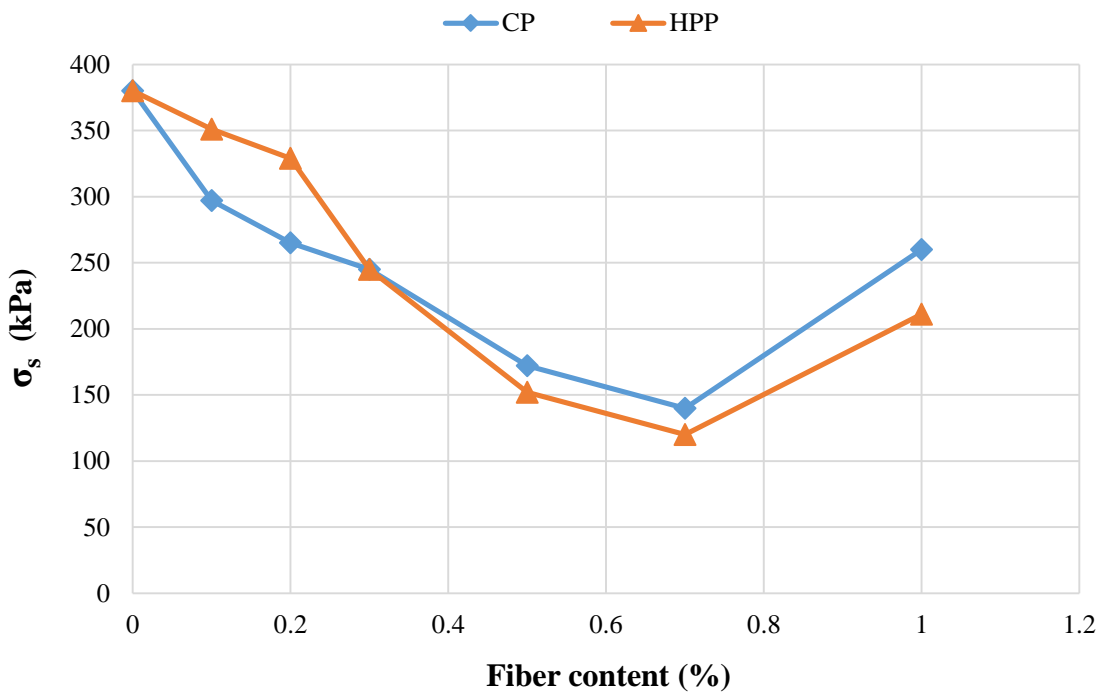
Figure 4.27 clearly describes a remarkable reduction in swelling potential of bentonite by increasing the content of homopolymer polypropylene. The swell pressure from 380 kPa for plain bentonite reduced 68% and approached to 120 kPa while the bentonite-homopolymer polypropylene mixture contained 99.93% bentonite and 0.7% Homopolymer polypropylene. After the optimum point, the swell pressure was increased and adding more homopolymer polypropylene was ineffective.



**Figure 4.27 :** Results of one-dimensional swell test for bentonite-homopolymer polypropylene mixtures.

#### 4.1.4 Comparing the effect of CP and HPP on the swelling behavior of bentonite

As it is exhibited in Figure 4.28, copolymer and homopolymer polypropylene affect the swelling potential of bentonite in a very similar way. Both of them reduce swell stress significantly and the maximum swell improvement occurs on the 0.7% content of both fiber types in the soil mass. also the results of the standard proctor compaction test indicates similar values for optimum water content and the maximum dry unit weight of both fiber stabilizers.

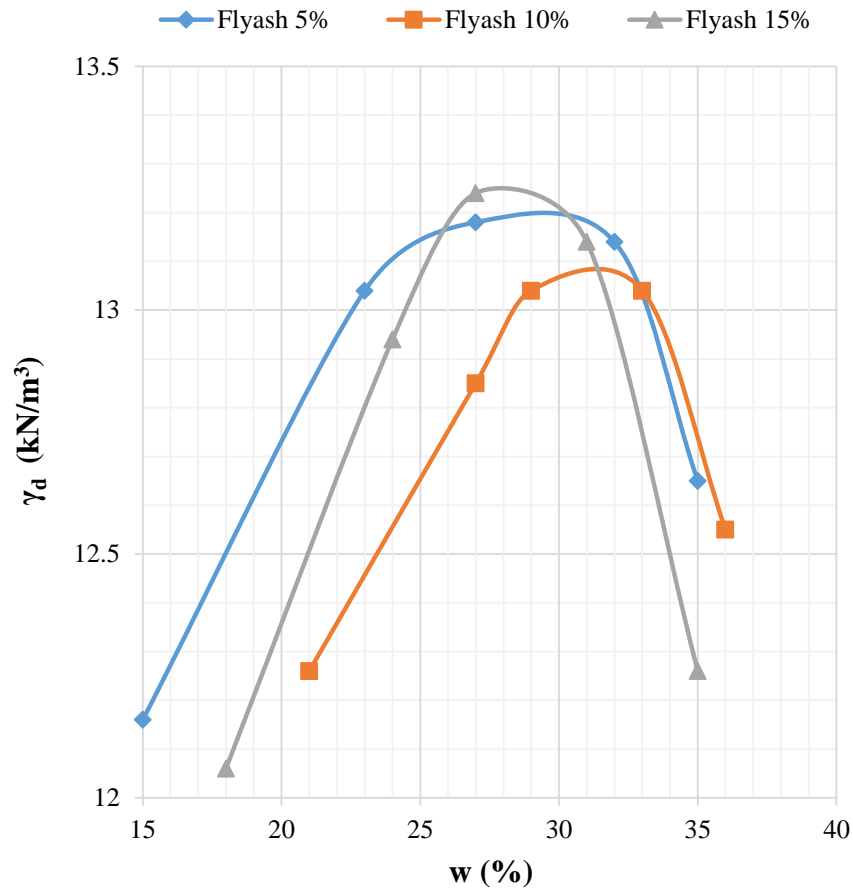


**Figure 4.28 :** One-dimensional swell test of bentonite mixtures with copolymer and homopolymer polypropylene.

#### 4.1.5 Fly ash mixtures with bentonite

Fly ash as an economical material was mixed with bentonite in three different percentages of 5%, 10%, and 15%. Standard proctor compaction test has been performed on all bentonite-FA in the mentioned dosages to evaluate the optimum water content. Figure 4.29 shows the curves of the standard proctor compaction test of bentonite-FA mixtures.

With respect to the results of Table 4.11, by increasing Fly ash content, optimum water content is decreased gradually but there is not any remarkable changes for the maximum dry unit weight.

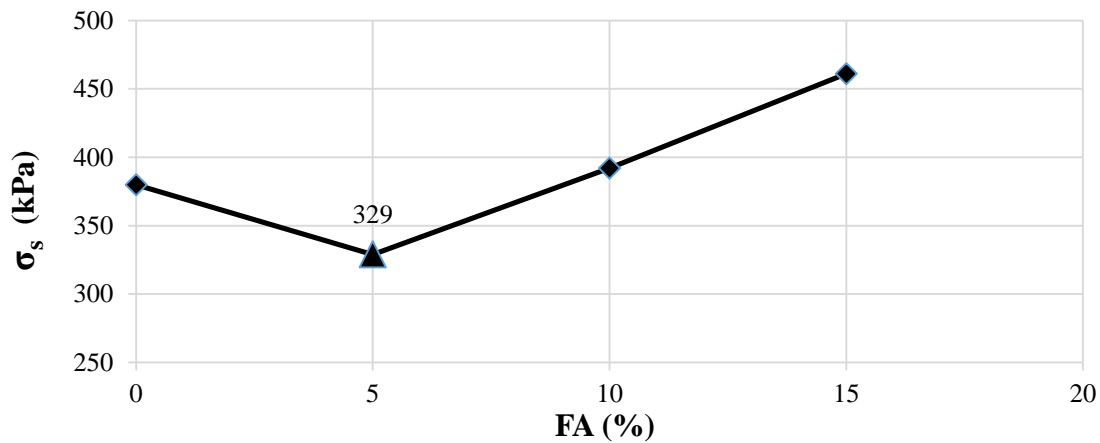


**Figure 4.29 :** Standard proctor compaction test of bentonite-fly ash mixtures.

**Table 4.11 :** Results of standard proctor test for bentonite-fly ash mixtures.

Main material	Additive material	Additive material content (%)	Optimum water content (%)	Max. Dry unit weight (kN/m <sup>3</sup> )
Bentonite	—	—	36	12.35
Bentonite	Fly ash	5	30	13.20
Bentonite	Fly ash	10	31	13.09
Bentonite	Fly ash	15	28	13.25

According to the results of one-dimensional swell test for bentonite-fly ash mixtures (Figure 4.30), there is not a significant reduction on the swell pressure of bentonite by adding Fly ash. The most reduction in swelling potential occurs with 5% of Fly ash in the mixture while the swell stress is decreased about 14%.

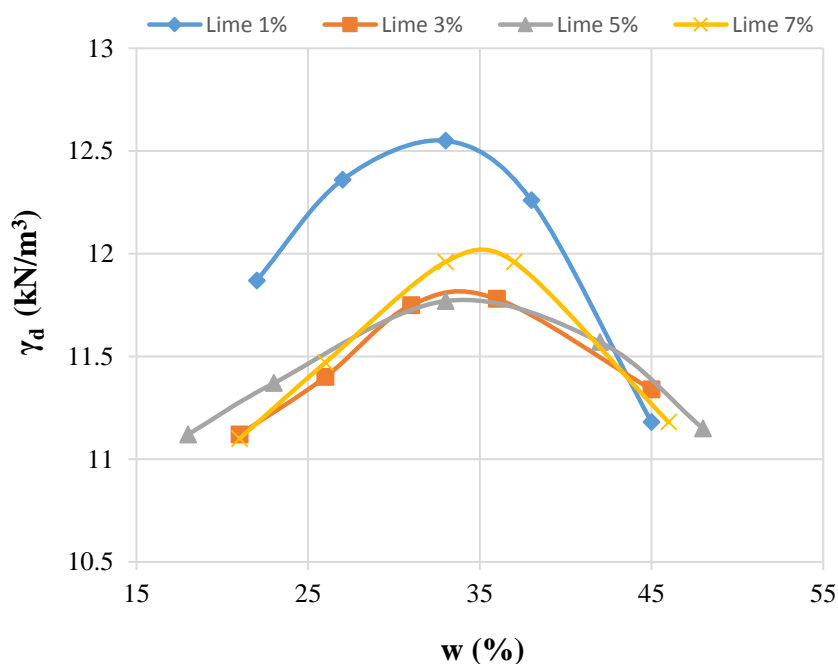


**Figure 4.30 :** One-dimensional swell test of bentonite-fly ash mixtures.

#### 4.1.6 Lime mixtures with bentonite

As the second chemical stabilizer in this study, lime was used in 4 various percentages of 1%, 3%, 5% and 7% to mix with bentonite to reduce the swelling potential. As a part of evaluation procedure standard proctor tests were done and all mixtures are prepared in optimum water content for determining swelling behavior of stabilized samples. The results of standard proctor compaction tests is shown in Figure 4.31.

The results which are shown in Table 4.12 indicate about same optimum water content for various mixtures of lime with bentonite, there is a reduction in maximum dry unit weight for 3 and 5 percent of lime content in the bentonite-lime mixtures.

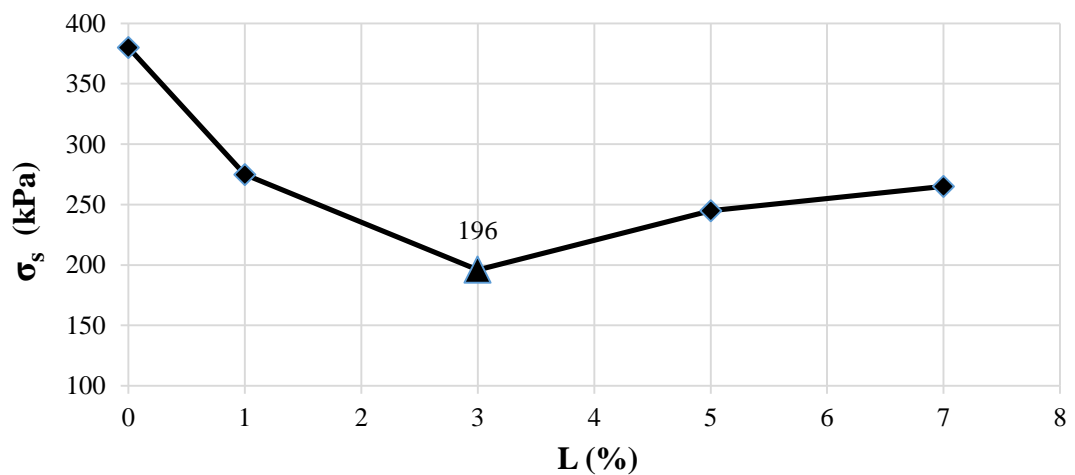


**Figure 4.31 :** Standard proctor compaction test of bentonite-lime mixtures.



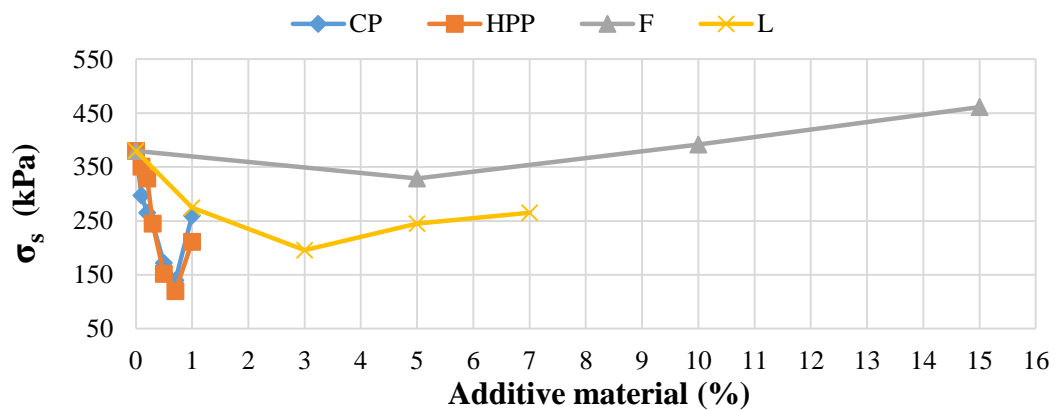
**Table 4.12 :** Results of standard proctor compaction test for bentonite-lime mixtures.

Main material	Additive material	Additive material content (%)	Optimum water content (%)	Max. dry unit weight (kN/m <sup>3</sup> )
Bentonite	–	–	36	12.35
Bentonite	Lime	1	33	12.55
Bentonite	Lime	3	34	11.81
Bentonite	Lime	5	34	11.78
Bentonite	Lime	7	35	12.00



**Figure 4.32 :** One-dimensional swell test of bentonite-lime mixtures

Considering Figure 4.32, the most reduction of swelling potential of bentonite occurs with 3% of lime content in the mixture. Adding lime to the bentonite can reduce its swell stress at most 49%.



**Figure 4.33 :** The effects of four different stabilizers on the swell pressure of bentonite.

Figure 4.33 shows the swell test results of the bentonite which is stabilized with four different additive materials. According to the results, the maximum swell reduction occurred by inclusion of the homopolymer polypropylene while fly ash is not very effective to reduce the swell potential.

## 5. CONCLUSIONS AND RECOMMENDATIONS

The effect of four types of fiber and chemical stabilizers on the swelling potential of the expansive soil has been evaluated. A high plasticity bentonite was used as the expansive soil.

Copolymer was mixed with bentonite in 6 different percentages of 0.1%, 0.2%, 0.3%, 0.5%, 0.7% and 1%. By adding copolymer, the maximum dry unit weight of stabilized soil increased but there was not remarkable changes on the optimum water content. There was a significant reduction on the swell pressure. Comparing with plain bentonite, at the optimum point, the swelling potential reduced at most 64% when the entire soil mass contained 99.93% bentonite and 0.7% copolymer.

As the second fiber stabilizer, same as the copolymer, homopolymer polypropylene also was used in 6 various percentages of 0.1%, 0.2%, 0.3%, 0.5%, 0.7% and 1% to mix with bentonite. There was not any significant change on the optimum water content and the maximum dry unit weight of the different bentonite-homopolymer polypropylene mixtures. The swell pressure from 380 kPa for plain bentonite reduced 68% and approached to 120 kPa while the bentonite-homopolymer polypropylene mixture contained 99.93% bentonite and 0.7% homopolymer polypropylene. After the optimum point, the swell pressure was increased and adding more homopolymer polypropylene was ineffective.

In the case of fly ash stabilization, there was a significant reduction on the liquid limit and plastic limit. Also, the maximum dry unit weight and the optimum water content decreased by increasing the fly ash content. To a particular point, increasing fly ash content caused to a reduction on the expansion potential, then, including more fly ash was ineffective and heaving increased. Comparing with the plain bentonite, at the optimum point, the swelling potential reduced at most 14% while entire soil mass contained 95% bentonite and 5% fly ash.

In the case of lime treatment, liquid limit and plastic limit reduced. Maximum dry density decreased while there was no remarkable changes in the optimum water content. Results approve that lime stabilization was very effective on reduction of

expansion potential of the expansive soils. Considering the swell pressure of the plain bentonite, the most reduction on the swell pressure approached to 49% and occurred with inclusion of 3% lime with 97% bentonite.

Comparison fly ash and lime stabilization results, indicates that both of them reduced the Atterberg limits of the treated soil and made it more workable. Because of chemical interaction between water and the mentioned stabilizers, absorption of water by the stabilized soil mass decreased which resulted in less values of optimum water content.

Copolymer and homopolymer polypropylene affected the swelling potential of bentonite in a very similar way. Both of them reduced swell pressure significantly and the maximum improvement occurred with 0.7% content of both fiber types in the soil mass. Both fly ash and lime decreased the swelling potential but, the lime was more effective and the results approve that it can be used for improving the expansive soils successfully. However, the effect of the utilized stabilizers on the shear strength and bearing capacity of the expansive soils can be investigated in the future studies.

## REFERENCES

- Altmeyer, W. T.** (1955). Discussion of engineering properties of expansive clays. *Proc. Am. Soc. Civil Eng.* 81 (Separate No. 658): 17 – 19.
- ASTM C1116/C1116M–10a**, Standard Specification for Fiber-Reinforced Concrete. *ASTM International, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA 19428-2959, United States.*
- ASTM D698-12e1** Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort. *ASTM International, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA 19428-2959, United States.*
- ASTM D2435** Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading. *ASTM International, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA 19428-2959, United States.*
- ASTM D4318** Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. *American Society for Testing and Materials, USA.*
- ASTM D4546** Standard Test Methods for One-Dimensional Swell or Collapse of Cohesive Soils. *ASTM International, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA 19428-2959, United States.*
- Basu, R., and Arulanandan, K.** (1973). A new approach to the identification of swell potential in soils. *Proc. 3rd Int. Conf. Expansive Soils*, Haifa, Israel 1:1–11.
- Bell, F.G.** (1993). *Engineering treatment of soils*. Published by E & FN Spon, an imprint of Chapman & Hall, 2-6 Boundray Row, London SE1 8HN.
- Bernardo, C., Clara, C., Davide, L. P., and Antonio, B.** (2012). Definition of laboratory optimization protocol for road bitumen improved with recycled tire rubber. *Construction and Building Materials*.
- Bhuvaneshwari, S. and Robinson, R. G. and Gandhi S. R.** (2005). Stabilization of expansive soils using fly ash. *Fly Ash Utilization Programme (FAUP), TIFAC, DST, New Delhi – 110016.*
- Brunauer S., Emmett P. H. and Teller E.** (1938). Adsorption of gases in multi molecular layers. *Journal of the America Chemical Society*, 60: 309-319.
- Budhu, M.** (2010). *Soil Mechanics and Foundation*, 3rd edition. Wiley, USA.
- Chen, F. H.** (1965). The use of piers to prevent the uplifting of lightly loaded structures founded on expansive soil. *Concluding Proc. Eng. Effects of Moisture*

*Changes in Soils, Int. Res. Eng. Conf. Expansive Clay Soils, Supplementing the Symposium in Print*, Texas A&M Press, pp. 152 – 171.

- Chen, F. H.** (1975). *Foundation on expansive soils*. Elsevier Scientific Publishing Co., New York.
- Chen, F.H.** (1988). *Foundation on expansive soils*. Elsevier, Amsterdam.
- Çokça, E.** (2001). Use of Class C Fly Ashes for the Stabilization – of an Expansive Soil. *Journal of Geotechnical and Geoenvironmental Engineering* Vol. 127, July, pp. 568-573.
- Dakshanamanthy, V. and Raman.** (1973). A simple method of identifying an expansive soil. *Soils and. Foundations*, Vol. **13**. No. 1, p. 97-104.
- Day, R. W.** (1999). *Geotechnical and foundation engineering design and construction*, McGraw- Hill Companies, New York.
- Dafalla, M. A., Shamrani, M.A.** (2011). Road Damage Due to Expansive Soils: *Survey of the Phenomenon and Measures for Improvement*.
- Das, B. M.** (2011). *Principles of Foundation Engineering*. 7th edition. Cengage Learning, Stamford CT 06902, USA.
- Fattah, M. Y. and Firas, A. S. and Bestun, J. N.** (2010). A treatment of expansive soil using different additives. *Acta Montanistica Slovaca Ročník 15*, číslo 4, 290-297.
- Gilchrist, F. R., D. S. Gamble, H. Kodama, and U. S. Khan.** (1993). Atrazine interactions with clay minerals: *Kinetics and equilibria of sorption. J. Agric. Food Chem.* 41: 1748-1755.
- Grim, R. E.** (1968). *Clay Mineralogy*, 2nd ed. McGraw-Hill, New York.
- Grim R. E., Güven, N.** (1978). Bentonites geology, mineralogy, properties and uses. *Developments in Sedimentology*, Elsevier, New York.
- Guang-Xin, L., Yun-Min, C., Xiao-Wu, T.** (2008). Geosynthetics in Civil and Environmental Engineering: *Geosynthetics Asia 2008 Proceedings of the 4th Asian Regional Conference on Geosynthetics in Shanghai, China*.
- Gungor N. and Tulun T.** (1996). Evaluation of sodium and calcium bentonites for industrial application. *Journal of Science and Industrial Research*, 55: 268-273.
- Hamberg, D. J.** (1985). A simplified method for predicting heave in expansive soils. *M.S. thesis, Colorado State University, Fort Collins, CO*.
- Hamilton, J. J.** (1977). *Foundations on swelling or shrinking subsoils*. Canadian Building Digest, CBD-184.
- Hintze, S.** (1994). Risk analysis in foundation engineering with application to piling in loose friction soils in urban situation, *Doctoral Thesis, Division of Soil and Rock Mechanics, KTH, Sweden*.
- Holts, W. G., and Gibbs, H. J.** (1956). Engineering properties of expansive clays. *Transact. ACSE* 121:641 – 677.

- Hardcastle, J. H.** (2003). Evaluation of treatment of expansive volcanic soils. *US95, Owyhee County, Idaho: final report*, University of Idaho.
- Katti, R. K., Bhangle, E. S., Moza, K. K.** (1983). Lateral pressure of expansive soil with and without a cohesive non-swelling soil layer- applications to earth pressures of cross drainage structures of canals and key walls of dams (studies of K0 condition). *Central Board of Irrigation and Power. Technical Report 32*, New Delhi, India.
- Koerner, R.** (2005). *Designing with geosynthetics*, 5 Edition. Pearson Prentice Hall. Upper Saddle River, New Jersey, 07458.
- Krazynski, L. M.** (1976). Engineering Properties of Expansive Soils. *Geological Soc. of America, Eng. Geology Div., Symp. Expansive Soils*, Denver, CO.
- Litvinov, I. M.** (1960). Stabilization of settling weak clayey soils. *Highway Research Board Special Report No. 60*, Washington, D.C., pp. 94-112.
- Lucian, C.** (2006). Geotechnical Aspects of Buildings on Expansive Soils in Kibaha, Tanzania: *Preliminary Study*.
- Mattheus F. A. Goosen** (2006). *Expansive soils, recent advances in characterization, and treatment*. Taylor & Francis.
- Mitchell, J. K.** (1976). *Fundamentals of Soil Behavior*. John Wiley, New York.
- Mitchell, J. K. and Raad** (1973). Control of volume changes in expansive earth materials. *Proc. Workshop Expansive Clays and Shales in Highway Design and Construction, Vol. 2, D. R. Lamb and S. J. Hanna, ed. Federal Highway Administration*, Denver, CO, pp. 200 – 219.
- McKeen, R. G. and Hamberg, D. J.** (1981). *Characterization of expansive soils*. Transportation Research Record 790, Transportation Research Board.
- Meehan, R. L. and Karp, L. B.** (1994). California housing damage related to expansive soils, *Journal of Performance of Constructed Facilities*. 8, no. 2, 139-157.
- Mika, S. L. J and Desch, S.C.** (1998). Structural surveying, *Macmillan Education LTD*, London.
- Moorlock, B.S.P. & Highley, D.E.** (1991). An Appraisal of fuller's earth resources in England and Wales. *British Geological Survey Technical Report*, WA/91/75.
- Murthry, V.N.S.** (2002). *Geotechnical Engineering, Principles and Practices of Soil Mechanics and Foundation Engineering*. Marcel Dekker. Inc.
- Nalbantoglu, Z. and Tuncer ER** (2001). Compressibility and hydraulic conductivity of a chemically treated expansive clay. *Can Geotech J* 38(1):154-160.
- Nalbantoglu, Z. and Gucbilmez, E.** (2002). Utilization of an industrial waste in calcareous expansive clay stabilization. *Geotech Test J* 25(1): 78-84.
- Nelson and Miller** (1992). *Expansive Soils Problems and Practice in Foundation and Pavement Engineering*. Wiley, New York, 1992, 259 pp.
- Özaydın, K.** (1999). *Zemin Mekaniği*. Birsen Yayınevi Ltd. Şti. Istanbul.
- Patrick, D. M and Snethen, D. R.** (1976). An occurrence and distribution survey of

expansive materials in the United States by physiographic areas, *Interim Report, Federal Highway Administration Office of Research and development*, Washington, D.C. 20590.

- Pearring, J. R.** (1963). A study of basic mineralogical, physical-chemical, and engineering index properties of laterite soils. *Dissertation. Texas A&M Univ.*, Collage Station, TX.
- Portland Cement Association** (1970). Recommended practice for construction of residential concrete floors on expansive soil, Vol. 2. *Portland Cement Association*, Los Angeles.
- Puppala, A. J., and Musenda, C.** (2001). Effect of fiber reinforcement on strength and volume change in expansive soils. *Transportation Research Record*, 1736, Paper No. 00 – 0716, 134 – 140.
- Rakesh, K. and Jain P. A.** (2012). Prospect of using granular piles for improvement of expansive soils. *In: International Journal of Advanced Engineering Technology*.
- Raman, V.** (1967). Identification of expansive soils from the plasticity index and the shrinkage index data. *Indian Eng. Calcutta* 11 (1):17–22.
- Ransom, W. H.** (1981). *Building failures; Diagnosis and avoidance*, E. and FN Spon LTD, New York.
- Sabtani, A. A.** (2005). Geotechnical properties of expansive clay shale in Tabuk, Saudia Arabia, *Journal of Asian Earth Sciences* 25(2005) 747-757.
- Satyanarayana, B.** (1969). Behavior of expansive soil treated or cushioned with sand. *In: Proceedings of Second International Conference on Expansive Clay Solis*, Texas A&M University, College Station, TX, PP. 308 – 316.
- Seed, H. B., Mitchell, J. K., and Chan, C. K.** (1962a). Studies of swell and swell pressure characteristics of compacted clays. *Highway Res. Board Bull.* 313:12 – 39.
- Seed, H. B., Woodward, R. J., Jr., and Lundgren, R.** (1962b). Prediction of swelling potential for compacted clays. *J. Soil Mech. Found. Div., Am Soc. Civil Eng.* 88(SM3):53 – 87.
- Sridharan, A.** (2000). Classification procedures for expansive soils. *Proc. InsN Civ. Engrs*, 2000,143, Oct., 235-240.
- Shukla, S. K., Yin, J. H.** (2006). Fundamentals of geosynthetic engineering. *Taylor and Francis group*, London, UK.
- Skempton, A. W.** (1953). The colloidal activity of clays. *Proc. 3rd Int. Conf. Soil Mech. Found. Eng.*, Switzerland. V.1:57 – 61.
- Şenol, A., Edil, T. B., Acosta, H., A., Benson. , C. H.** (2003). Soft Subgrades Stabilization by Using Various Fly Ashes, *Resources, Conservation and Recycling*, vol.46, p.p. 365-376.
- Thomas, P. J.** (1998). Quantifying properties and variability of expansive soils in selected map units, PhD Dissertation. *Virginia Polytechnic Institute and State University*, USA.



**Thomas, P. J., Baker, J. C. and Zelazny, L. W.** (2000). An expansive soil index for predicting shrink-swell potential. *Virginia Polytechnic Inst. and St. Univ.*, USA.

**Van der Merwe, D. H.** (1964). The prediction of heave from the plasticity index and the percentage clay fraction of soils. *Civil Eng. South Africa* 6:103 – 107.

**Url-1** <<http://www.minersoc.org>>, data retrieved 19.11.2013.

**Url-2** <<http://www.superiorfoundationrepair.com>>, data retrieved 26.12.2013.

**Url-3** <<http://www.capitalgeotechnical.com>>, data retrieved 14.01.2014.

**Url-4** <<http://www.cenews.com>>, data retrieved 05.02.2014.

**Url-5** <<http://www.basementsystem.ca>>, data retrieved 27.01.2014.

**Url-6** <<http://www.hiwaystabilizers.co.nz>>, data retrieved 26.02.2014.

**Url-7** <<http://www.fhwa.dot.gov>>, data retrieved 08.12.2013.

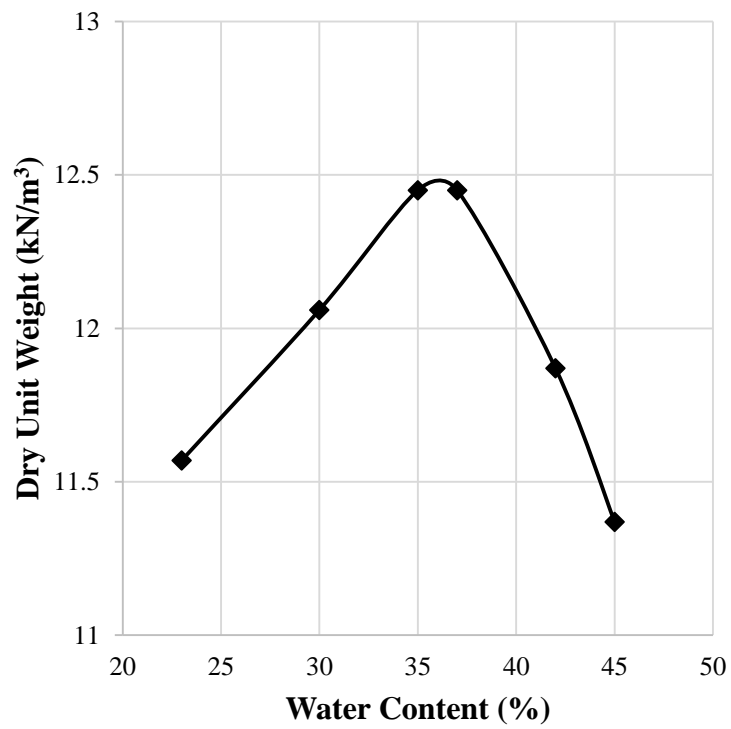


## **APPENDICES**

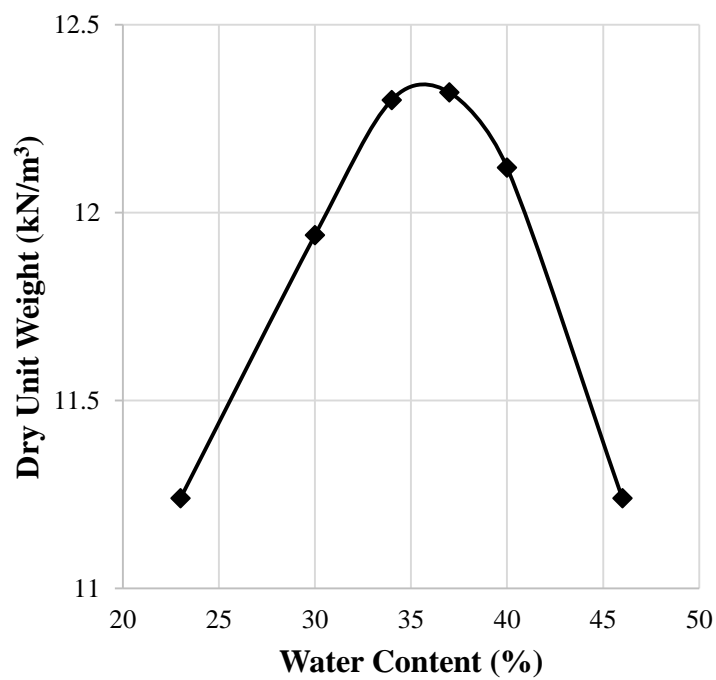
**APPENDIX A.1 :** Results of standard proctor compaction test

**APPENDIX A.2 :** Water content control to prepare sample for swell test

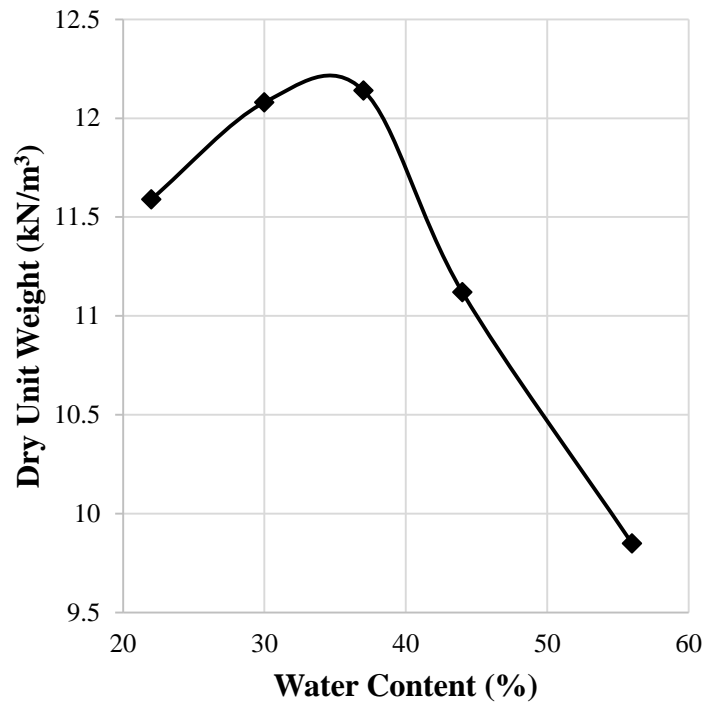
## APPENDIX A.1



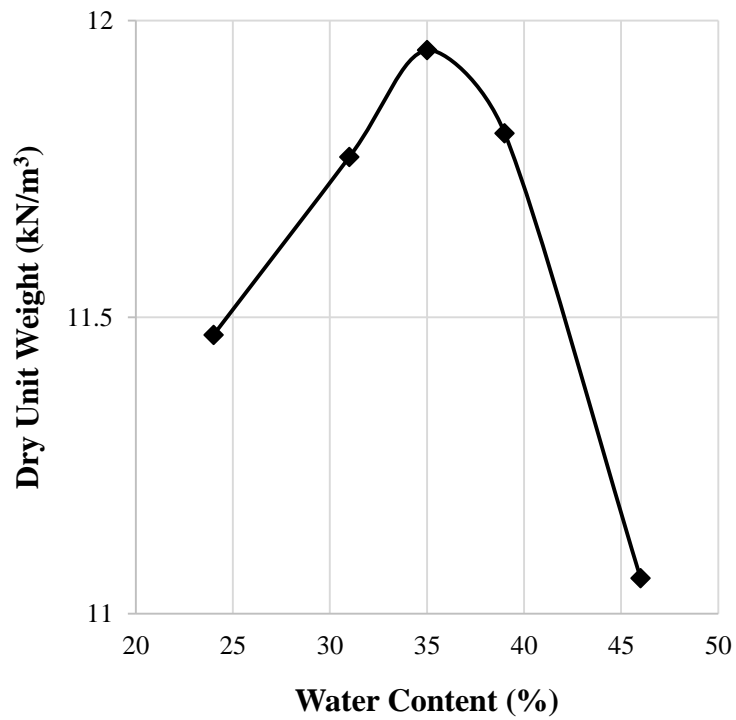
**Figure A.1.1 :** Standard proctor compaction test of bentonite – attempt 1  
(  $w_{opt} = 36\%$  ;  $\gamma_{d\ max} = 12.49\text{ kN/m}^3$  )



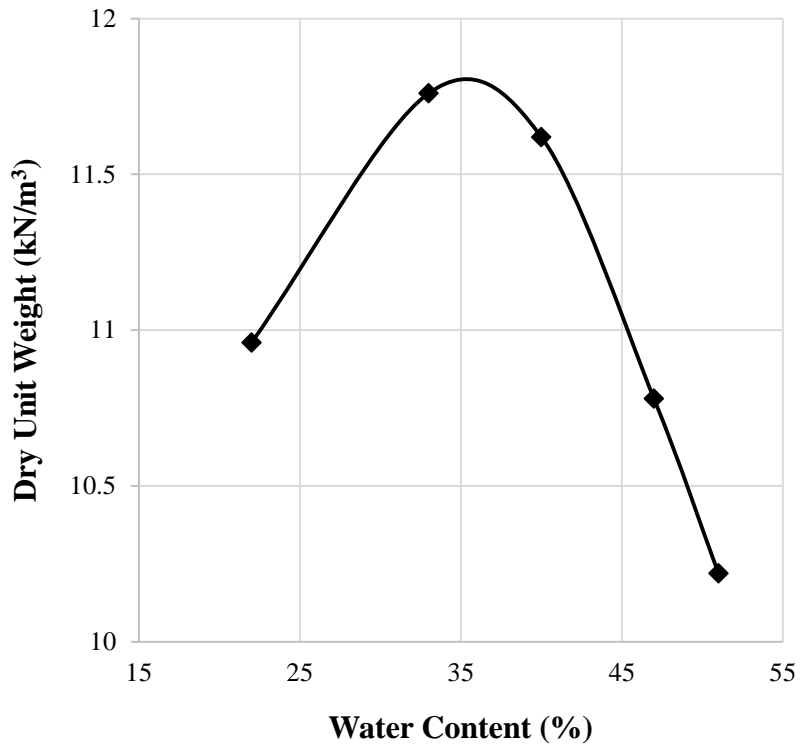
**Figure A.1.2 :** Standard proctor compaction test of bentonite – attempt 2  
(  $w_{opt} = 36\%$  ;  $\gamma_{d\ max} = 12.35\text{ kN/m}^3$  )



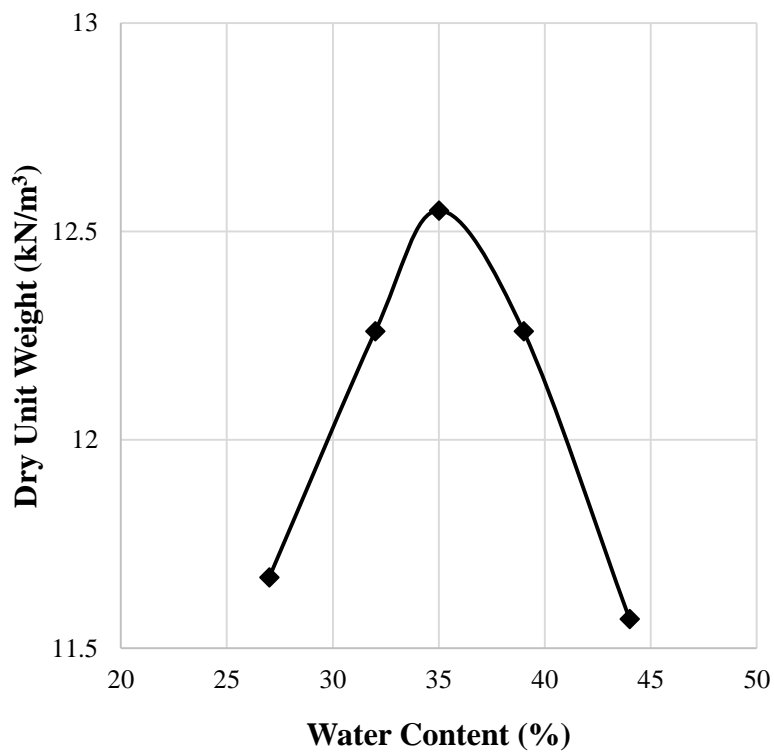
**Figure A.1.3 :** Standard proctor compaction test of bentonite – attempt 3  
 (  $w_{opt} = 35\%$  ;  $\gamma_{d\ max} = 12.20\text{ kN /m}^3$  )



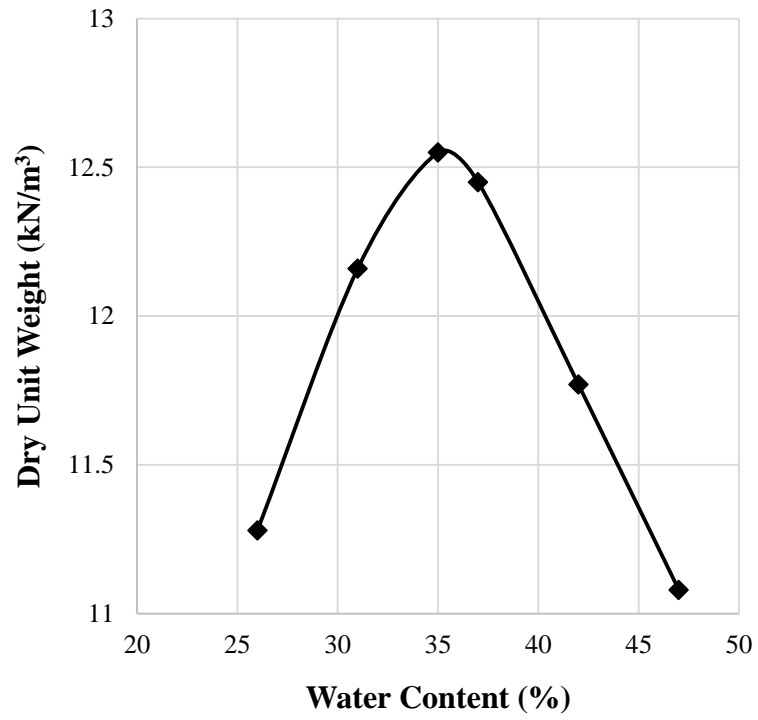
**Figure A.1.4 :** Standard proctor compaction test of bentonite – attempt 4  
 (  $w_{opt} = 35\%$  ;  $\gamma_{d\ max} = 11.96\text{ kN /m}^3$  )



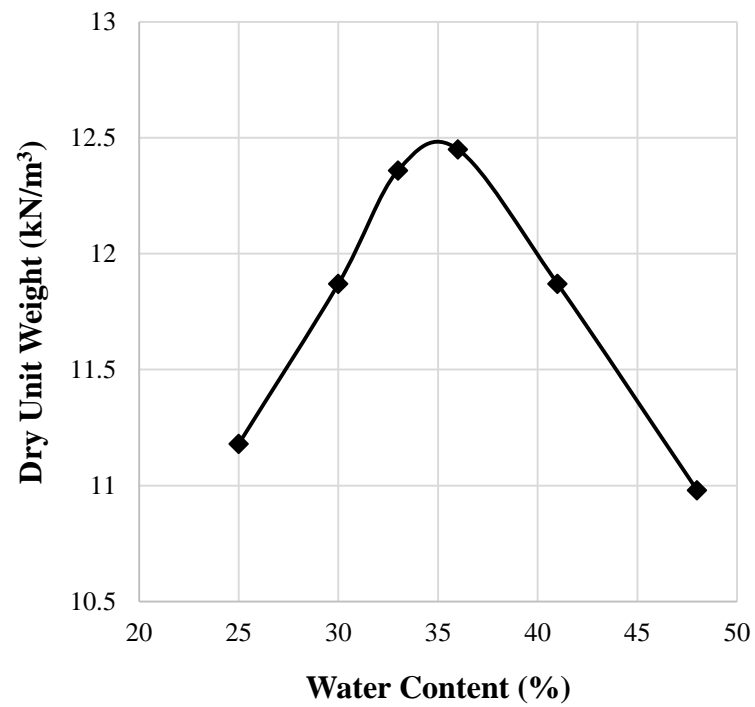
**Figure A.1.5 :** Standard proctor compaction test of bentonite – attempt 5  
 (  $w_{opt} = 36 \%$  ;  $\gamma_{d \max} = 11.80 \text{ kN/m}^3$  )



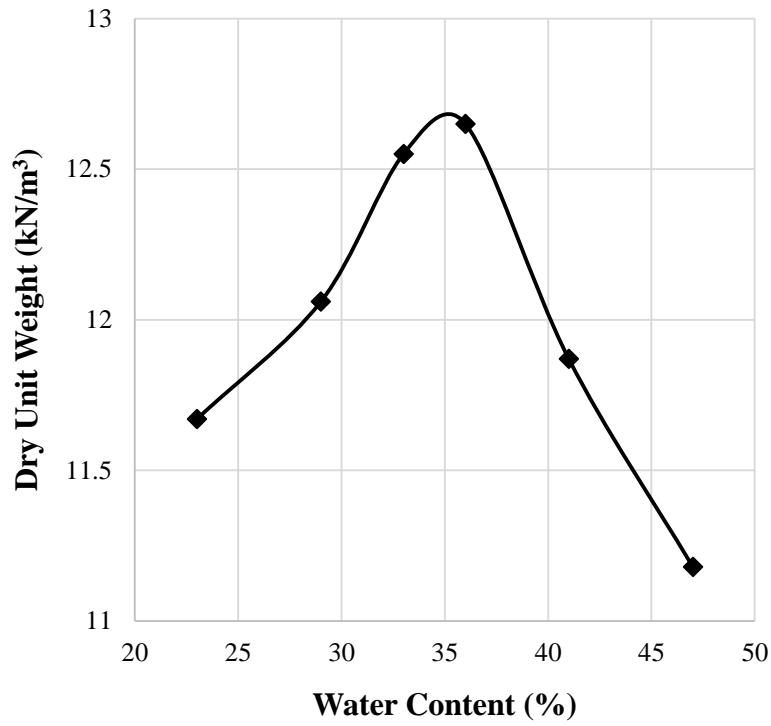
**Figure A.1.6 :** Standard proctor compaction test  
 B + CP 0.1 % ;  $w_{opt}: 35 \%$  ,  $\gamma_{dmax}: 12.55 \text{ kN/m}^3$



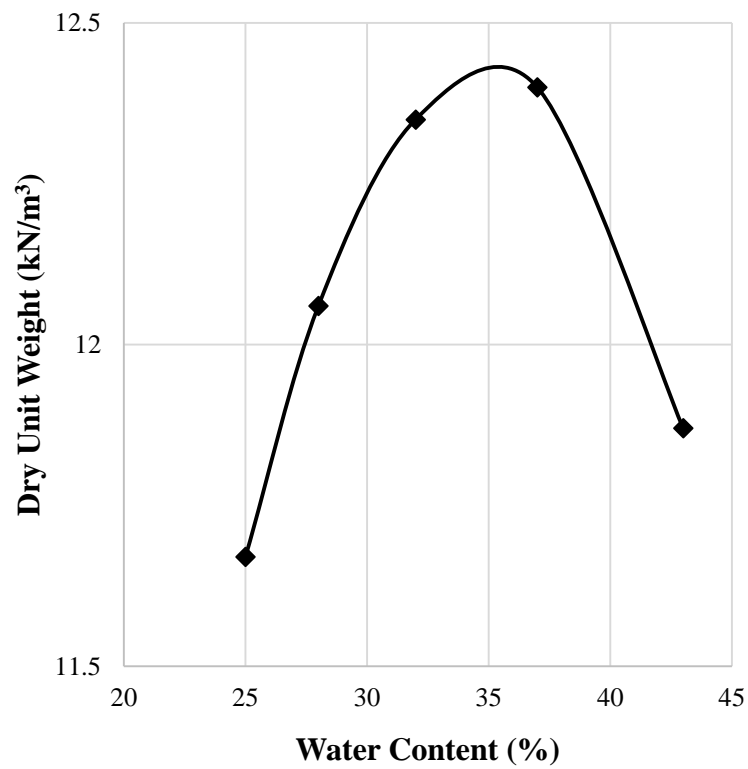
**Figure A.1.7 :** Standard proctor compaction test  
 B + CP 0.2 % ;  $w_{opt}$ : 35 % ,  $\gamma_{dmax}$ : 12.55 kN/m<sup>3</sup>



**Figure A.1.8 :** Standard proctor compaction test  
 B + CP 0.3 % ;  $w_{opt}$ : 35 % ,  $\gamma_{dmax}$ : 12.50 kN/m<sup>3</sup>

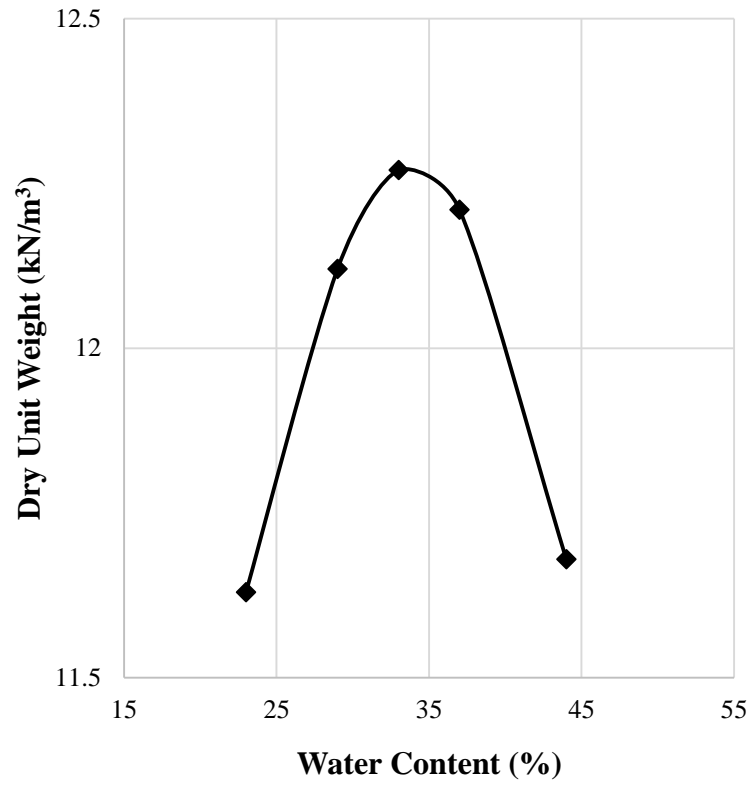


**Figure A.1.9 :** Standard proctor compaction test  
 B + CP 0.5 % ;  $w_{opt}$ : 35 % ,  $\gamma_{dmax}$ : 12.69 kN/m<sup>3</sup>

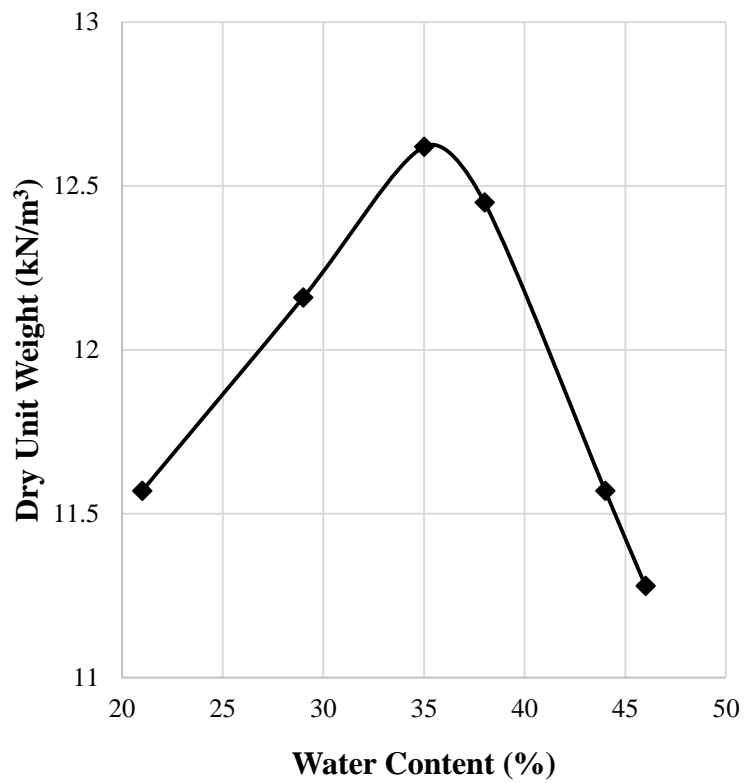


**Figure A.1.10 :** Standard proctor compaction test  
 B + CP 0.7 % ;  $w_{opt}$ : 35 % ,  $\gamma_{dmax}$ : 12.43 kN/m<sup>3</sup>

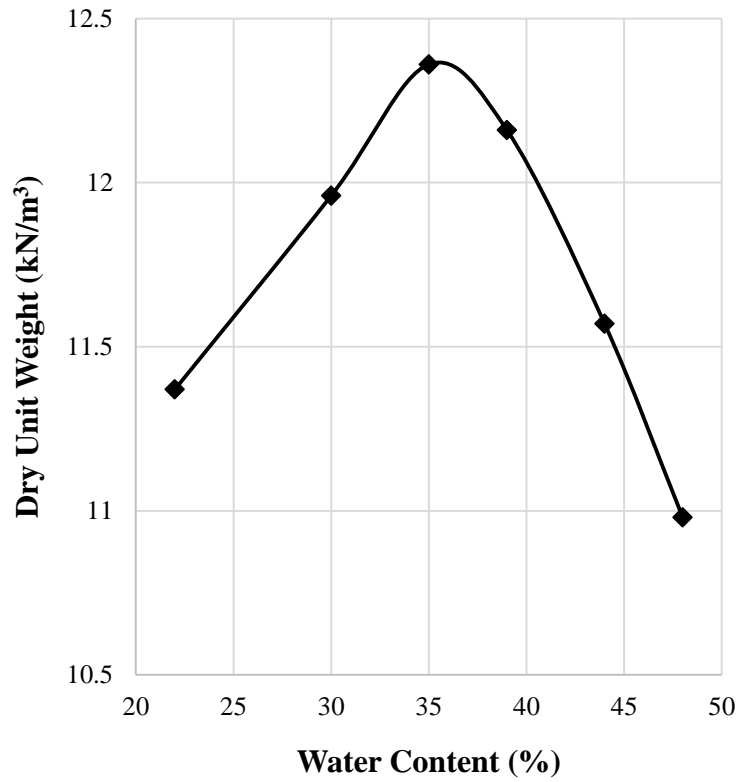




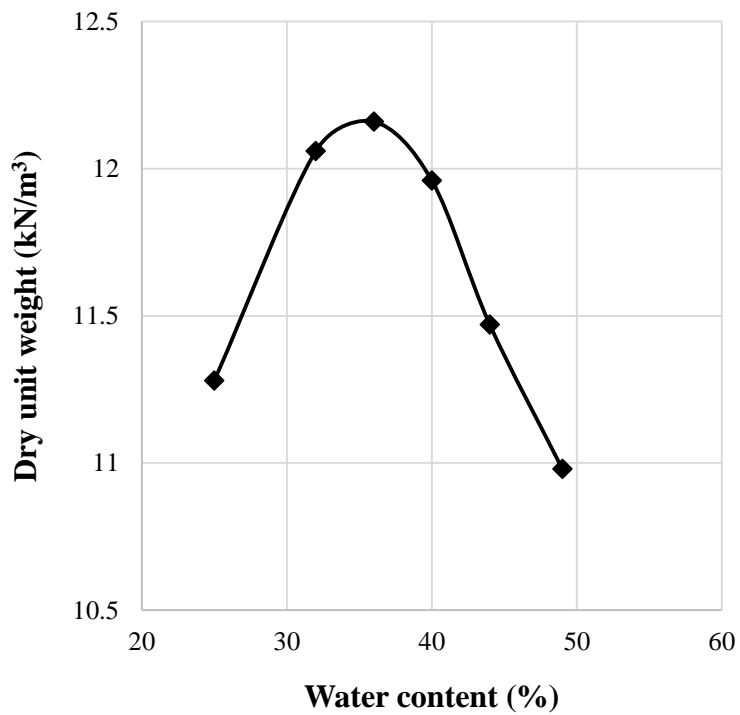
**Figure A.1.11 :** Standard proctor compaction test  
 B + CP 1 % ;  $w_{opt}$ : 34 % ,  $\gamma_{dmax}$ : 12.28 kN/m<sup>3</sup>



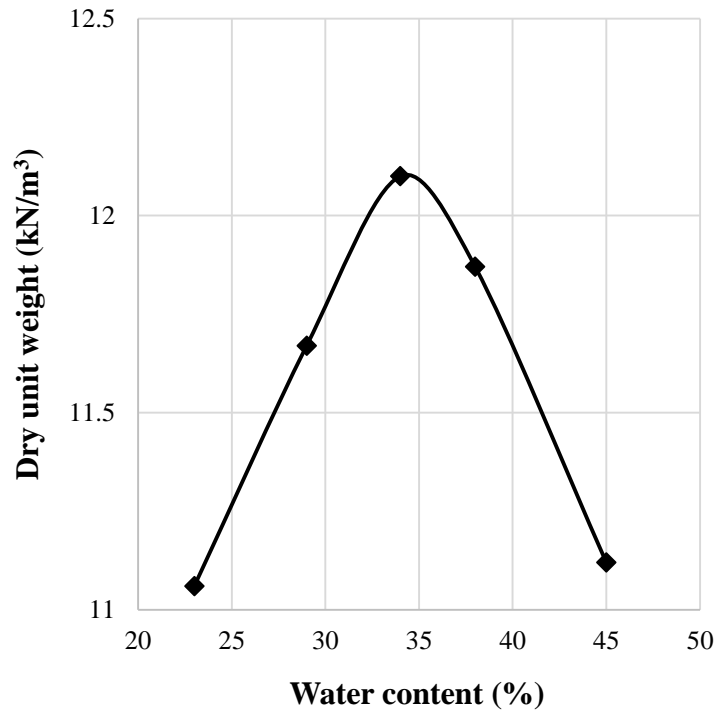
**Figure A.1.12 :** Standard proctor compaction test  
 B + HPP 0.1 % ;  $w_{opt}$ : 35 % ,  $\gamma_{dmax}$ : 12.62 kN/m<sup>3</sup>



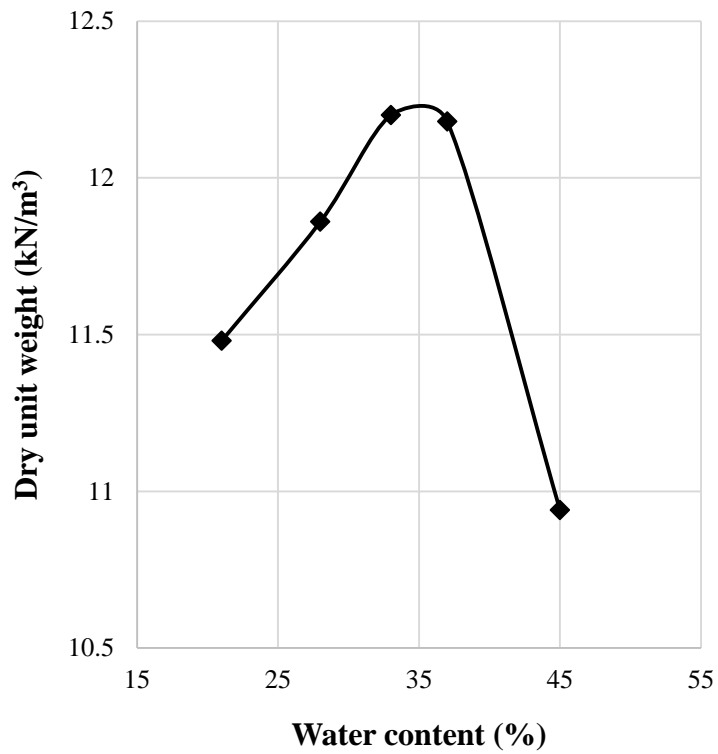
**Figure A.1.13 :** Standard proctor compaction test  
 B + HPP 0.2 % ;  $w_{opt}$ : 36 % ,  $\gamma_{dmax}$ : 12.35 kN/m<sup>3</sup>



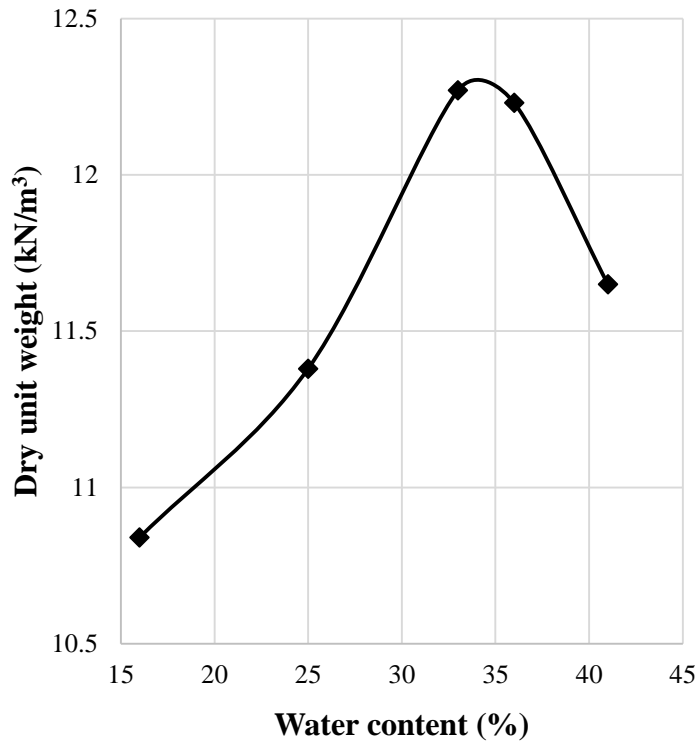
**Figure A.1.14 :** Standard proctor compaction test  
 B + HPP 0.3 % ;  $w_{opt}$ : 35 % ,  $\gamma_{dmax}$ : 12.17 kN/m<sup>3</sup>



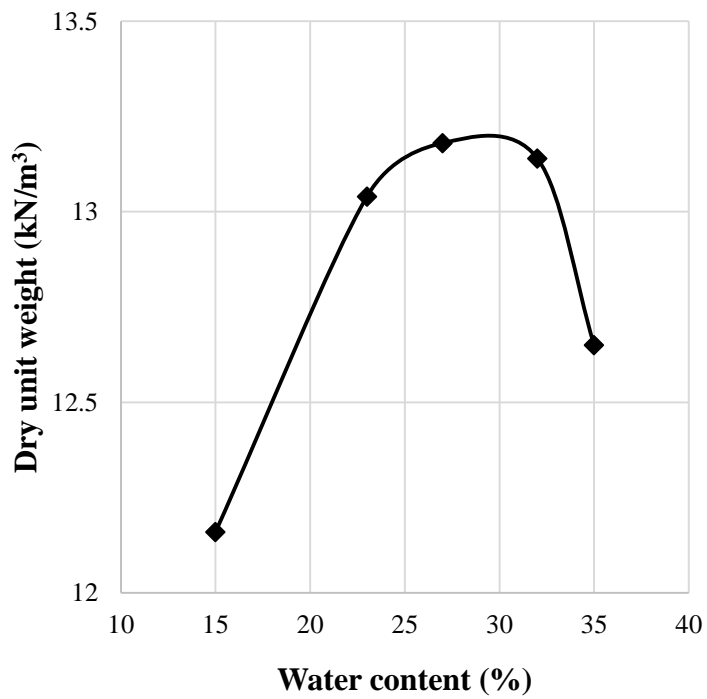
**Figure A.1.15 :** Standard proctor compaction test  
B + HPP 0.5 % ;  $w_{opt}$ : 34 % ,  $\gamma_{dmax}$ : 12.10 kN/m<sup>3</sup>



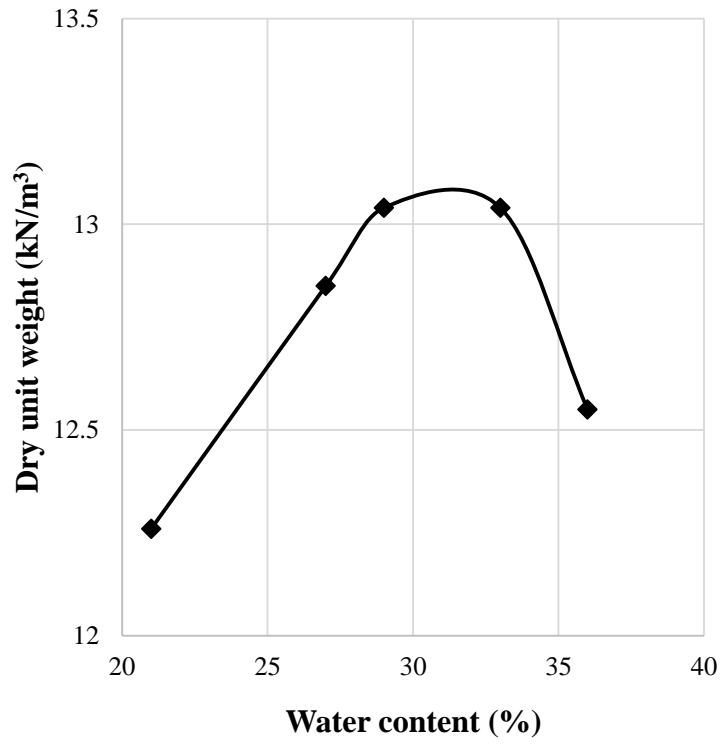
**Figure A.1.16 :** Standard proctor compaction test  
B + HPP 0.7 % ;  $w_{opt}$ : 35 % ,  $\gamma_{dmax}$ : 12.22 kN/m<sup>3</sup>



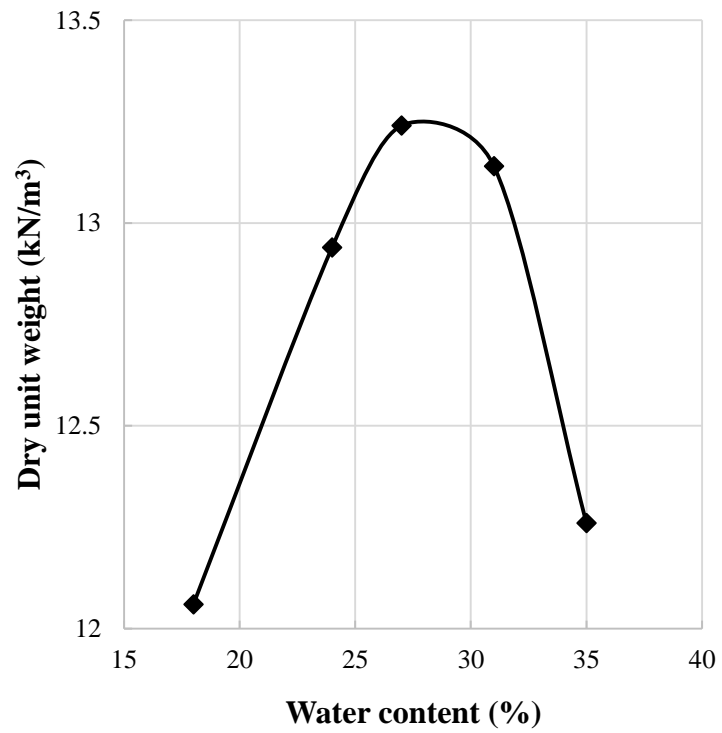
**Figure A.1.17 : Standard proctor compaction test**  
 B + HPP 1 % ;  $w_{opt}$ : 34 % ,  $\gamma_{dmax}$ : 12.30 kN/m<sup>3</sup>



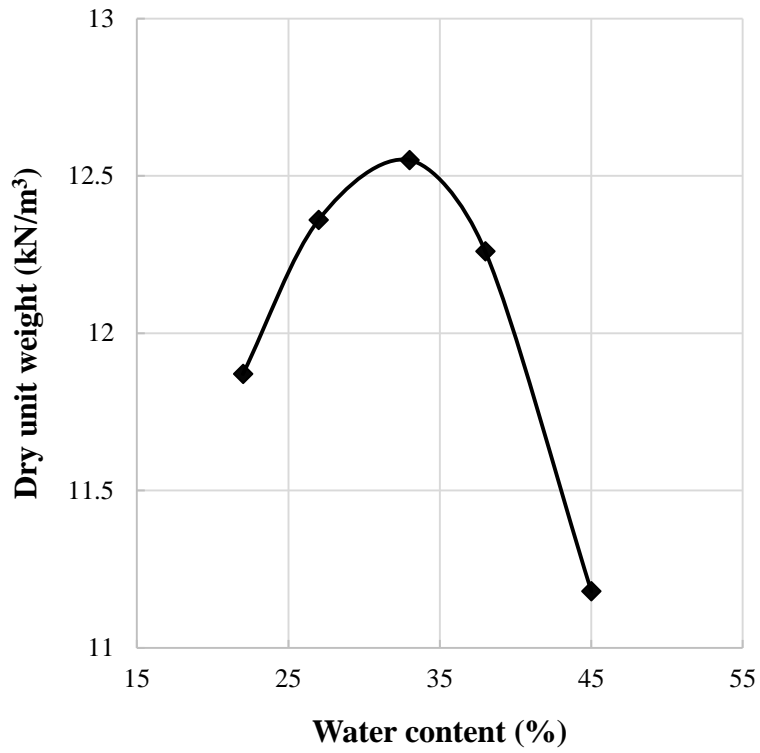
**Figure A.1.18 : Standard proctor compaction test**  
 B + F 5 % ;  $w_{opt}$ : 30 % ,  $\gamma_{dmax}$ : 13.20 kN/m<sup>3</sup>



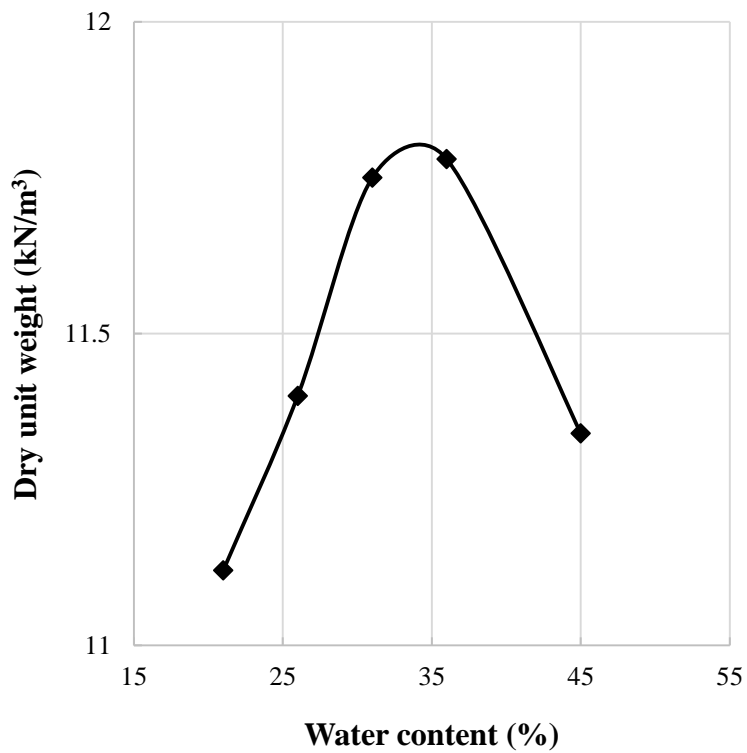
**Figure A.1.19 :** Standard proctor compaction test  
 B + F 10 % ;  $w_{opt}$ : 31 % ,  $\gamma_{dmax}$ : 13.09 kN/m<sup>3</sup>



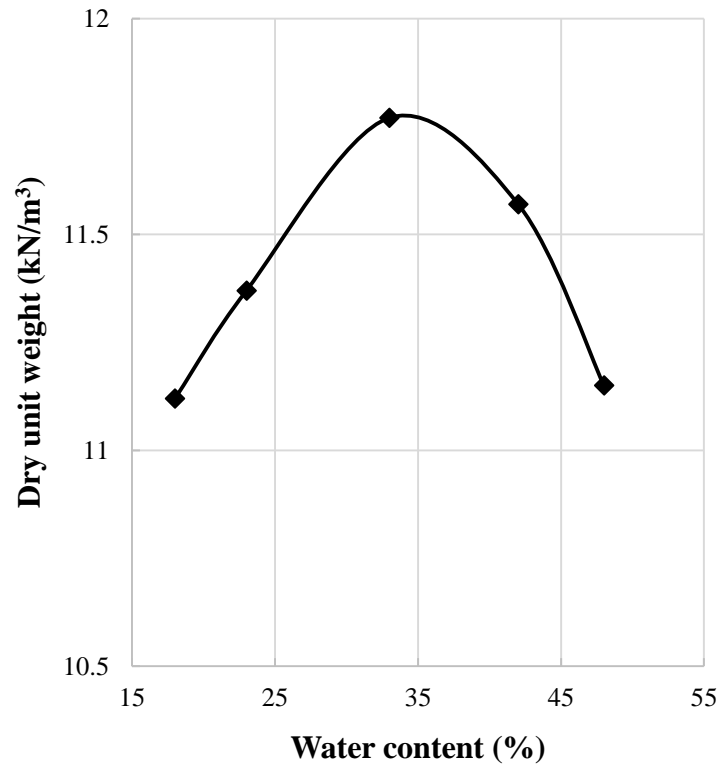
**Figure A.1.20 :** Standard proctor compaction test  
 B + F 15 % ;  $w_{opt}$ : 28 % ,  $\gamma_{dmax}$ : 13.25 kN/m<sup>3</sup>



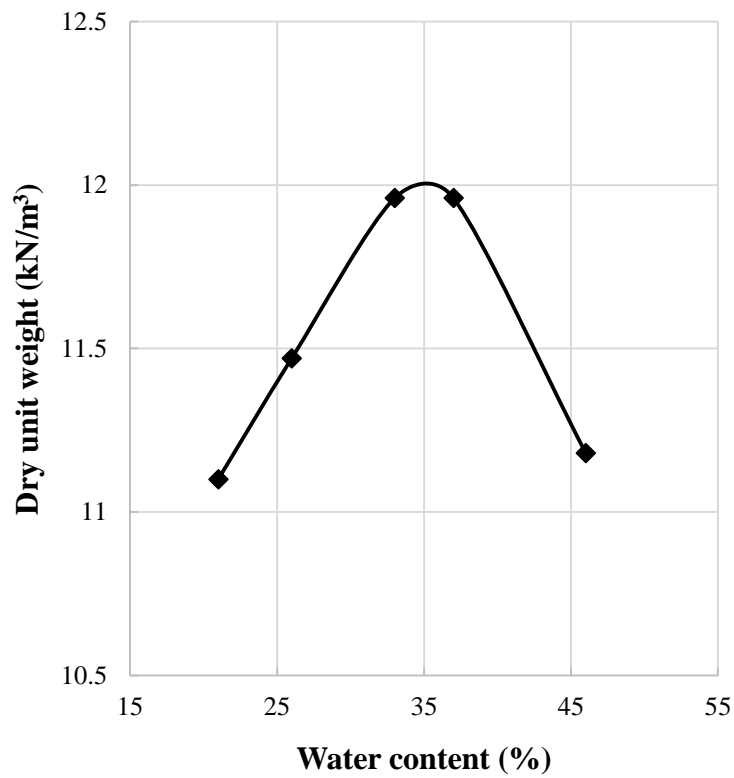
**Figure A.1.21 : Standard proctor compaction test**  
 B + L 1 % ;  $w_{opt}$ : 33 % ,  $\gamma_{dmax}$ : 12.25 kN/m³



**Figure A.1.22 : Standard proctor compaction test**  
 B + L 3 % ;  $w_{opt}$ : 34 % ,  $\gamma_{dmax}$ : 11.81 kN/m³

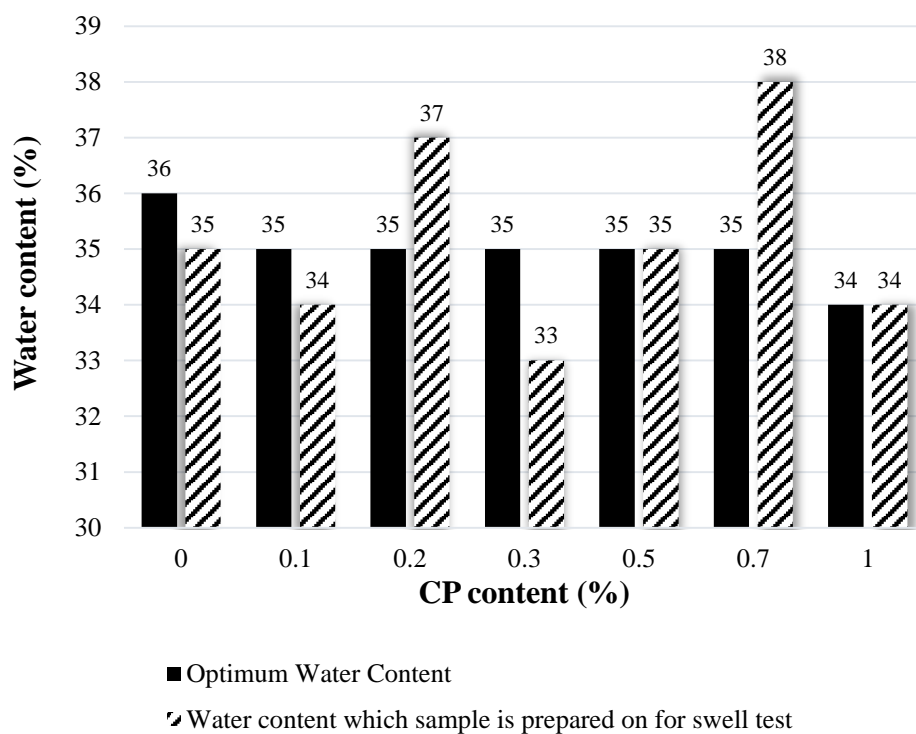


**Figure A.1.23 :** Standard proctor compaction test  
 B + L 5 % ;  $w_{opt}$ : 34 % ,  $\gamma_{dmax}$ : 11.78 kN/m<sup>3</sup>

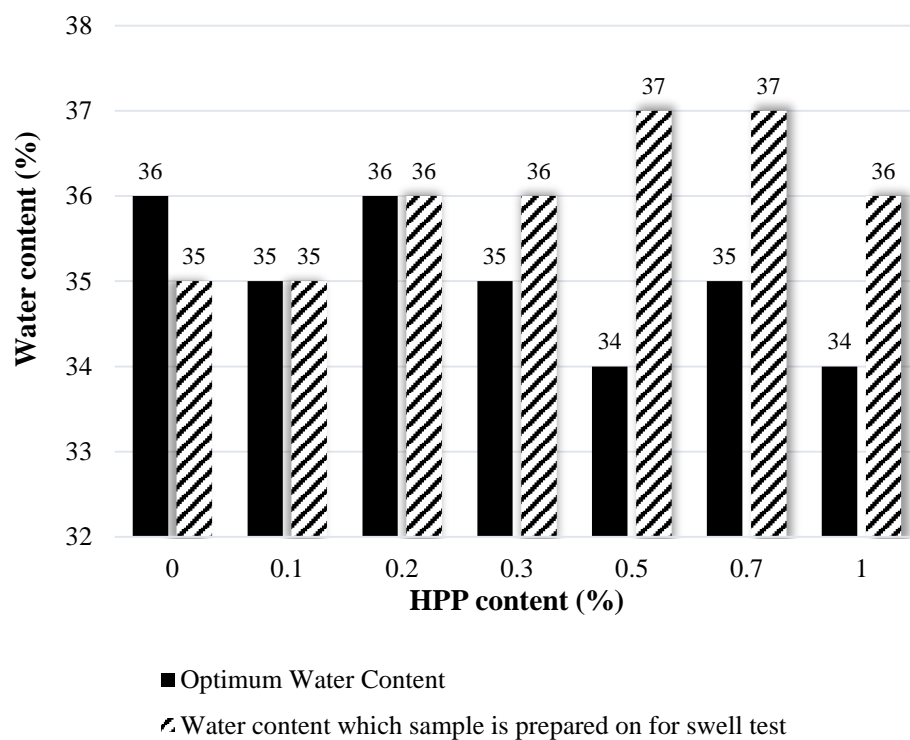


**Figure A.1.24 :** Standard proctor compaction test  
 B + L 7 % ;  $w_{opt}$ : 35 % ,  $\gamma_{dmax}$ : 12.00 kN/m<sup>3</sup>

## APPENDIX A.2

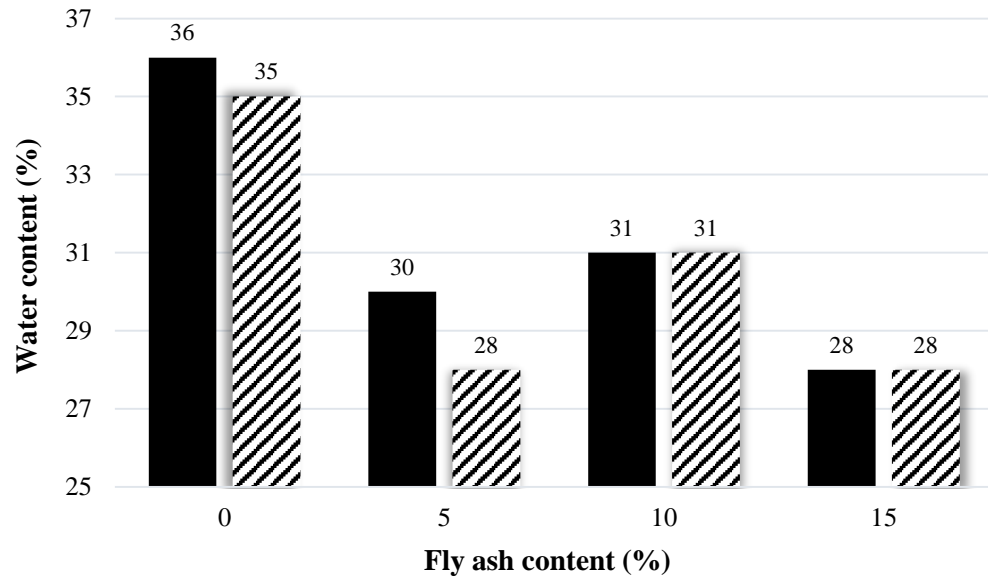


**Figure A.2.1 :** Water content control for bentonite – CP mixtures



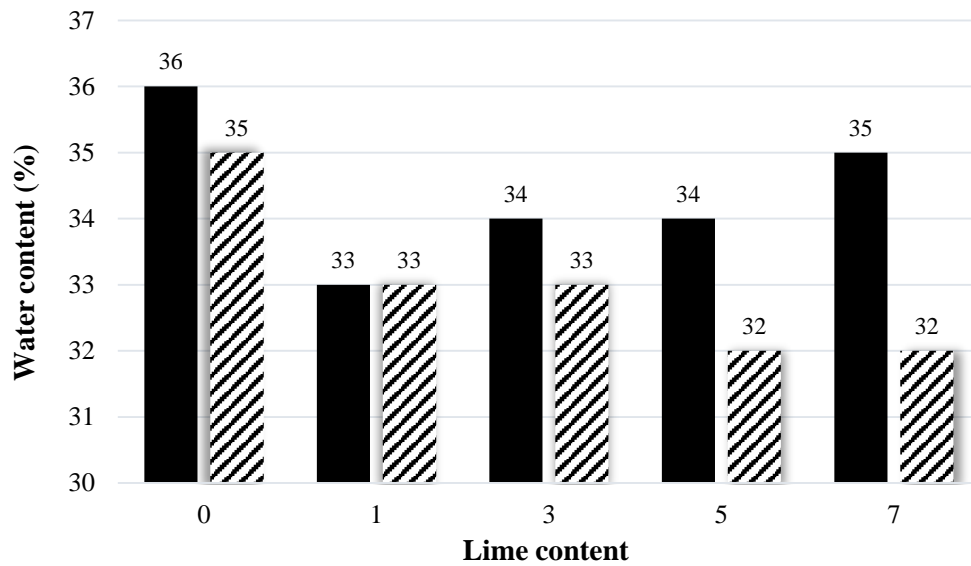
**Figure A.2.2 :** Water content control for bentonite – HPP mixtures





■ Optimum Water Content   ▨ Water content which sample is prepared for swell test

**Figure A.2.3 :** Water content control for bentonite – fly ash mixtures



■ Optimum Water Content

▨ Water content which sample is prepared on for swell test

**Figure A.2.4 :** Water content control for bentonite – lime mixtures



## **CURRICULUM VITAE**



**Name Surname** : Samad KAZEMI KHOSROWSHAHI  
**Place and Date of Birth** : TABRIZ/ 19, April, 1984  
**Address** : Golchin Alley., imam st., Khosrowshah-Tabriz/ IRAN  
**E-Mail** : samadkazemi.kh@gmail.com  
**B.Sc.** : Islamic Azad University of Shabestar/Civil Engineering