IMPROVEMENT OF EXPANSIVE SOILS BY USING PHOSPHOGYPSUM

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ABSTRACT

STABILIZATION OF AN EXPANSIVE SOIL
USING PHOSPHOGYPSUM

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Expansive soils are a worldwide problem that poses several challenges for civil engineers. Such soils swell when given an access to water and shrink when they dry out. The most common and economical method for stabilizing these soils is using admixtures that prevent volume changes. Studies for treatment of expansive soils with phosphogypsum are very limited in literature. In this study the effect of phosphogypsum (PG) in reducing the swelling potential is examined. The expansive soil was prepared in the laboratory by mixing kaolinite and bentonite. Recycled PG was added to the soil at 5 to 25 percent by weight. Grain size distribution, Atterberg limits and swell percent were determined. Using PG improved the performance of the expansive soil by reducing the swelling potential.

The addition of more than 15% phsophogypsum did not affect the swelling potential. Therefore, the optimum phosphogypsum content was found as 15%. Additionally, phosphogypsum was used with lime and ground granulated blast furnace slag (GGBFS). The addition of %15 PG and %15 GGBFS decrease the swelling potential

from % 43.16 to % 9.16. When the curing period for sample including PG and GGBFS increase, the swelling potential for this sample declined.

Key Words: Expansive Soil, Soil Stabilization, Swelling Potential, Phosphogypsum

ÖZ

FOSFOJİPS KULLANARAK ŞİŞEN ZEMİNLERİN
STABİLİZASYONU

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Şişen zeminler inşaat mühendisleri için çeşitli zorluklar teşkil eden dünya çapında bir sorundur. Bu tip zeminler su ile birleştiklerinde şişer ve kuruduklarında ise büzüşürler. Bu tip zeminleri sağlamlaştırmak için kullanılan en yaygın ve ekonomik yöntem hacim değişikliğini önleyen katkı maddeleri kullanmaktır. Literatürde şişen zeminlerin fosfojips ile iyileştirimesi için yapılan çalışamalar sınırlıdır. Bu çalışmada şişme kapasitesini azaltan fosfojips (PG) etkisi incelenmiştir. Şişen zeminler kaolinit ve bentonitin karıştırılmasıyla laboratuvarda hazırlanmıştır. Fosfojipsin zemine ağırlığının yüzde 5 ile 25'i oranında eklenmiştir. Tane büyüklüğü dağılımı, kıvam limitleri, şişme yüzdesi ve karışımların şişme oranları tespit edilmiştir. Fosfojipsin kullanımı şişme potansiyelini azaltarak performansı arttırdığı görülmüştür. %15 fosfojips'den daha fazla kullanılması şişme potansiyelini etkilememektedir. Böylece optimum fosfojips içeriği % 15 olarak bulunmuştur. Ayrıca, fosfojips kireç ve öğütülmüş granule yüksek fırın cürufu (GGBFS) ile birlikte kullanılmıştır. %15 PG ve % 15 GGBFS ilavesi şişme potansiyelini % 43.16'dan % 9.16'ya düşürmüştür. PG ve GGBFS içeren numuneler için kür süresi arttıkça, şişme potansiyeli düşmüştür.

Anahtar kelimeler: Şişen Zemin, Zemin Stabilizasyonu, Şişme Potansiyeli, Fosfojips

To My Family

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TABLE OF CONTENTS

ABSTRACT v
ÖZ vii
ACKNOWLEDGMENTS x
TABLE OF CONTENTS xi
LIST OF TABLES xiiii
TABLE OF FIGURES xivv
LIST OF ABBREVIATIONS xviii
CHAPTERS
1. INTRODUCTION 1
1.1. General 1
1.1. Objective of Study 5
1.2. Scope of Study 6
2. LITERATURE REVIEW 7
2.1. Clay Mineralogy 7
2.1.1. Kaolinite Group 12
2.1.2. Illite Group 13
2.1.3. Smectite Group 15
2.2. Diffuse Double Layer 16
2.3. Clay Structure 17
2.4. Cation Exchange Capacity (CEC) 18
2.5. Mechanism of Expansion 20
2.6. Factors Influencing of Soils 22
2.7. Treatment of Expansive Soils 25
2.7.1. Chemical Additives 26
2.7.2. Pre-wetting 33
2.7.3. Soil Removal and Replacement 34
2.7.4. Compaction 34
2.7.5. Surcharge Loading 35

2.7.6. Thermal Methods 35
3. EXPERIMENTAL STUDY 37
3.1. Purpose 37
3.2. Material 37
3.3. Preparation of Test Samples 40
3.4. Properties of Samples 43
3.5. Test Procedure for Free Swell Test Using Oedemeter 58
3.5.1. Compaction of Specimen 58
3.5.2. Free Swell Test Using Bishop Apparatus 58
3.6. Test Results 61
3.7. SEM Analysis 66
4. DISCUSSION ON TEST RESULTS 75
4.1. Effect of Additives on Specific Gravity 75
4.2. Effect of Additives on Liquid Limit 75
4.3. Effect of Additives on Plastic Limit 76
4.4. Effect of Additives on Plasticity Index 77
4.5. Effect of Additives on Shrinkage Limit 77
4.6. Effect of Additives on Shrinkage Index 78
4.7. Effect of Additives on Grain Size Distribution 78
4.8. Effect of Additives on Activity 79
4.9. Effect of Additives on Swelling Potential 79
4.10. Effect of Cure on Swelling Potential 80
4.11. Effect of Additives and Curing on Rate of Swell 80
4.12. Discussions on SEM-EDX Analysis 82
5. CONCLUSION 87
REFERENCES 91
APPENDICES
A. CHANGES OF SOIL PROPERTIES 99
B. SWELL VERSUS TIME GRAPHS 101

LIST OF TABLES

TABLES

Table 2.1. Characteristic of some clay minerals(Nelson and Milner, 1992).....11

Table 2.2. Cation exchange capacity with respect to clay minerals
(Lambe and Whitman, 1968) ..... …….19

Table 2.3. Soil Properties Influencing Swell Potential(Nelson and Milner,1992)..23

Table 2.4. Environmental Factors Affecting Swell Potential
(Nelson and Miller,1992)……24

Table 2.5. Stress Conditions Affecting Swell Potential
(Nelson and Miller,1992)……25

Table 3.1. Chemical Composition of Kaolinite, Bentonite, Phosphogypsum,
Lime and Ground Granulated Blast Furnace Slag ……. ..... …….40

Table 3.2. Composition of Prepared Specimens by Dry weight…….41

Table 3.3. Properties of Samples…….57

Table 3.4. Samples used for SEM Analysis..... 67

Table A.1. Changes in Clay Content, Liquid Limit, Plastic Limit, Plasticity Index,
Shrinkage Limit, Shrinkage Index, Specific Gravity, Activity and
Swelling Potential.....100

TABLE OF FIGURES

FIGURES

Figure 1.1. Damage on a masonary wall due to the shrinkage in soil(source:http: //www.basementsystems.ca/foundation-repair/wall-cracks-repair/settlement.html).2
Figure 1.2. Damage on pavements due to the swelling soil (source: http://inspectapedia.com/structure/Frost_Heave_Expansive_Soil_Damage.php) .. 2
Figure 1.3. Figure 1.3. Phosphogypsum stack in Florida (http://www.cooperativeconservation.org/viewproject.asp?pid=655) 4
Figure 2.1. Basic Units of Clay Minerals (Craig, 2004) 8
Figure 2.2. Silica Sheet (Mitchell, 1993) 9
Figure 2.3. Alumina Sheet (Mitchell, 1993) 9
Figure 2.4. Kaolinite (Craig, 2004) 12
Figure 2.5. SEM of Kaolinite (source: http://claymin.geoscienceworld.org/content/37/1/1/F2/graphic-3.large.jpg) 13
Figure 2.6. Illite (Craig, 2004) 14
Figure 2.7. SEM of Illite (source:http://www.ssokinc.com/sheet_graphics/Illite.jpg) 14
Figure 2.8. Montmorillonite (Craig, 2004) 15
Figure 2.9. SEM of Montmorillonite (source: http://www.petrotech-assoc.com/images/smectite_2.jpg). 16
Figure 2.10. Distribution of cations and anions adjacent to a clay particle surface according to the diffuse double layer theory (Keijzer, 2000) 17
Figure 2.11. (a) Floccuated structure (b) Dispersed structure (Lambe and Withman, 1969) 18
Figure 2.12. Different Types of Exchange Sites on Clay Particles, Surface and Absorbed Ion Interlayer Sites..... 19
Figure 2.13. Mechanism of Swelling (Popescu, 1986) 21

Figure 2.14. A View of Stabilizing Expansive Soil with Lime (source: http://www.streumaster.com/technology/soil-stabilisation/) 28
Figure 2.15. A View of Stabilizing Expansive Soil with Cement (source: http://www.griffinsoil.com/soil_stabilization_manual_and_videos) 30
Figure 2.16. A View of Stabilizing Expansive Soil with Cement (source: http://www.trstabilisation.co.uk/Soil-Stabilisation.html) 30
Figure 3.1. Views of Materials Used in the Study 39
Figure 3.2. Preparation of Samples 42
Figure 3.3. Plasticity Chart 45
Figure 3.4. Classification Chart For Swelling Potential (After Seed, et al., 1962). 46
Figure 3.5. Grain Size Distribution Curves for Sample A and Phosphogypsum Treated Samples 47
Figure 3.6. Grain Size Distribution Curves for Sample A and Lime and Phosphogypsum Treated Samples 48
Figure 3.7. Grain Size Distribution Curves for Sample A and Phosphogypsum and Ground Granulated Blast Furnace Slag Treated Samples 49
Figure 3.8. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Specific Gravity of the Samples 50
Figure 3.9. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Liquid Limit of the Samples 51
Figure 3.10. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Plastic Limit of the Samples 52
Figure 3.11. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Plasticity Index of the Samples 53
Figure 3.12. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Shrinkage Limit of the Sample 54
Figure 3.13. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Shrinkage Index of the Samples 55
Figure 3.14. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Activity of the Samples 56

Figure 3.15. A View from Static Compaction with the Hydraulic Jack 58
Figure 3.16. Free Swell Test Setup Drawing (İpek, 1998) 60
Figure 3.17. Bishop Apparatus..... 60
Figure 3.18. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Swelling percent without Curing 62
Figure 3.19. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Swelling percent with Curing 63
Figure 3.20. Change of Swelling percent with addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag..... 64
Figure 3.21. Change of Swelling percent with addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag with curing 65
Figure 3.22. SEM image of Sample A (magnification factor=40000)..... 67
Figure 3.23. SEM image of Sample A+14%PG+1%L (magnification factor=40000) without curing 68
Figure 3.24. SEM image of Sample A+14%PG+1%L (magnification factor=40000) 7 days curing 68
Figure 3.25. SEM image of Sample A+14%PG+1%L (magnification factor=40000) 28 days curing..... 69
Figure 3.26. SEM image of Sample A+15%PG+15%GGBFS (magnification factor=40000) without curing 69
Figure 3.27. SEM image of Sample A+15%PG+15%GGBFS (magnification factor=40000) 7 days curing 70
Figure 3.28. SEM image of Sample A+15%PG+15%GGBFS (magnification factor=40000) 28 days curing 70
Figure 3.29. EDX Diagram of Sample A 71
Figure 3.30 EDX Diagram of Sample A+14%PG+1% L without curing..... 72
Figure 3.31. EDX Diagram of Sample A+15%PG+15% GGBFS without curing 73
Figure 4.1. Effect of additive materials on t50 of the Samples.....81
Figure 4.2. Effect of curing period on t50 of the Samples.....82

Figure 4.3. Sem images of soils (Ismaiel, 2006).....85
Figure B.1. Swelling percent versus Time Graph for Sample A
and Phosphogypsum Treated Samples without Curing 102
Figure B.2. Swelling percent versus Time Graph for Sample A
and Phosphogypsum and Lime Treated Samples without Curing..... 103
Figure B.3. Swelling percent versus Time Graph for Sample A
and 10 % Phosphogypsum Treated Samples with Curing..... 104
Figure B.4. Swelling percent versus Time Graph for Sample A
and 14 % Phosphogypsum, 1% Lime Treated Samples with Curing 105
Figure B.5. Swelling percent versus Time Graph for Sample A,
15 % Phosphogypsum and 15 % Grand Granulated Blast Furnace Slag Treated
Samples with Curing..... 106

LIST OF ABBREVIATIONS

ASTM: American Society for Testing and Materials

CEC: Cation exchange capacity

CH: Clay with high plasticity

DDL: Diffuse double layer

EDX: Energy Dispersive X-Ray

GGBFS: Ground Granulated Blast Furnace Slag

h: Initial height of the specimen

Hi:Initial height of the sample

Hf: Final height of the sample

K: Hydraulic conductivity

L: Lime

LL: Liquid limit

m: Mass of Sample

METU: Middle East Technical University

Pb: Lead

PI: Plasticity index

PG: Phosphogypsum

PL: Plastic limit

Po: Polonium

Ra: Radium

SEM: Scanning Electrom Microscope

SI: Shrinkage Index

SL: Shrinkage Limit

TS: Turkish Standard

U: Uranium

USCS: Unified Soil Classification System

V_1: Initial Volume

V_2: Final volume

ρ: Density

ΔH: Height difference which is between initial height and final height

CHAPTER 1

INTRODUCTION

1.1. General

Expansive soils swell when they are exposed to water and shrink when they dry. Therefore, soil moisture content is primary factor that affects the change of its volume. Expansive soils generally exist in semi arid regions which are located in the tropical and temperate climate zones in all five continents. For this reason, expansive soils are a worldwide problem (Chen,1988).

Swelling and shrinkage phenomena occur in expansive clay when moisture rises and declines, respectively. Due to swelling and shrinkage, severe destructions in light weight structures (Fig. 1.1), in pavements and buried structures in expansive clay soils have been reported (Fig 1.2). In recent years, many researchers have focused on the phenomena and improving constituve and computational models applied to the engineering behaviour of expansive soils (Hong, 2008).

In the U.S.A. at least 2.3 billions of dollars are spent for damages which expansive soils cause in light structures and roads every year. The damages caused by expansive soils are more expensive than the damages caused by other natural disasters, like eartquakes, floods, hurricanes (Jones and Holtz, 1973).

In Turkey, expansive soils exist in some part of the Central Anatolia region, West Anatolia region, Southeast Anatolia region and East Anatolia region (Çokça,1991). Ankara, which is the capital of Turkey, is a rich area in terms of expansive soil as

Figure 1.1. Damage on a masonary wall due to the shrinkage in soil (source:http://www.basementsystems.ca/foundation-repair/wall-cracks-repair/settlement.html)

Figure 1.2. Damage on pavements due to the swelling soil (source:http://inspectapedia.com/structure/Frost_Heave_Expansive_Soil_Damage.php)

approximately two-thirds of the settlement of this city is founded on a clayey soil called Ankara Clay. This clay is classified as medium to highly plastic expansive soil. Damages in the light weight structures such as light one-storey buildings, garden walls, roads and buried utilities such as water pipes resulted from excessive heave of Ankara Clay were reported particularly at the southwestern part of the city, where there is a rapid dense urbanization (Erguler and Ulusay, 2003).

Phosphogypsum is a primary byproduct of the wet-acid process for producing phosphoric acid from phosphate rock. It is largely calcium sulfate and has been given the name phosphogypsum. In Turkey, 3 million tons of PG are produced annually (Değirmenci et al., 2006). It is generally stored at open space areas (Fig 1.3). This situation causes many envorimental pollution problems (Lin et al.,1990).

Phosphogypsum contains impurities of free phosphoric acid, phosphates, fluorides and organic matter that adhere to the surface of gypsum crystals and also substituted in the crystal lattice of gypsum. Phosphogypsum also contains radioactive elements such as U^{238}, U^{234} (U: Uranium), Ra^{226} (Ra: Radium), Pb^{210} (Pb: Lead) and Po^{210} (Po: Polonium), which are derived from the phosphate rocks (Singh, 2001).

Although the chemical composition of PG is the same with the chemical composition of natural gypsum, its use as natural gypsum is limited due to the some impurities. The amount of impurities depends on the production type and phosphate rock which is used. The water-soluble impurities are washed several times with water and milk of lime (whitewash). The impurities in the crystal lattice are calcined at 140°-150°. The presence of impurities puts restrictions on the use of PG in industry. For this reason, impurities are removed from PG or the effects of impurities is reduced to the minimum level before the PG is used (Nanni and Chang, 1985).

Figure 1.3. Phosphogypsum stack in Florida (Source: http://www.cooperativeconservation.org/viewproject.asp?pid=655)

In this study, phosphogypsum procured from Bandırma fertilizer factory was used to improve the swelling properties of an expansive soil. A small amount of this waste is used in soil and road stabilization and the remaining of it usually is deposited in open areas or dumped to river and sea, around the world. Researches have been recently directed towards the use of phosphogypsum with the other wastes such as fly ash, GGBFS etc. In our country, the lack of utilization possibility of phosphogypsum causes an economic loss and also environmental pollution.

Gezer et al. (2012), determined the natural radionuclide content of disposed PG (phosphogypsum) produced from PFP (Phosphorus Fertilizer Plant) A and PFP B, in Turkey, and assessed the radiological hazard of the PG samples on the basis of total annual effective doses and radium equivalent activity. The mean annual effective doses assessed for members of the public and workers are lower than the annual limit of 1 mSv y^{-1}. The results of the annual effective dose and radium equivalent activity index show that the PG samples examined can be used as building raw material and filling material without additional radiological risk.

Considering the amount of natural stabilization materials, which decreases day by day, phosphogysum as a by-product which can be easily obtained in abundance from the fertilizer plants can be used as an alternative expansive soil stabilization material.

1.2. Objective of the Study

The basic objective of this research is to investigate the suitability of phosphogypsum as a stabilizer to reduce the swelling potential of an expansive soil. In addition to this, the long term performance (7 days and 28 days of curing) of phosphogypsum on reducing the swell potential of an expansive clay is investigated. Morever, the effects of phosphogypsum used with lime and ground granulated blast furnace slag are also observed in this study.

1.3. Scope of the Study

In the scope of this thesis, the expansive soils and phosphogypsum are introduced in Chapter 1. Literature review on expansive clays is given in Chapter 2. Chapter 3 contains the experimental work, standard proctor, Atterberg limit, hydrometer, sieve analyses, specific gravity, and free swell tests with and without curing were performed. The effects of adding stabilizers such as phosphogypsum, lime and ground granulated blast furnace slag were investigated. The results obtained from this study are assessed in Chapter 4. The final chapter, Chapter 5, presents the conclusions of this study.

CHAPTER 2

LITERATURE REVIEW

Expansive clays exists all around the world, especially in arid and semi arid regions. They are problem soils for engineering structures because of swelling in wet season and shrinkage in dry season (Mishra et al., 2008). Donaldson (1969) mention that there are many countries which have expansive soil in the world. They are Argentina, Australia, Burma, Canada, Cuba, Eithopia, Ghana, India, Israil, Iran, Mexico, Morocco, Zimbabwe, South Africa, Spain, Turkey, U.S.A., Venezuela. In addition to this, such soil can exist anywhere in the world. Much of expansive soil problems are not recorded in underdeveloped countries. When these problems is determined in these countries, the countries which have expansive soil will increase (Chen, 1988).

An expansive soil, which is made up of montmorillonite mineral, is used as a drilling fluid in drilling boreholes. The expansive soils have been proposed as engineered barriers in the geological disposal systems for waste since these materials form a low permeable zone around wastes by swelling.

2.1. Clay Mineralogy

Clays are classified with respect to their physical and chemical properties. Their chemical properties, especially micro structure, are investigated for understanding engineering behaviour of clay.

Clay particles are very small sized and their shape is determined by arrangement of the thin crystal lattice layers which they form, with many other elements which can become incorporated into the clay mineral structure (hydrogen, sodium, magnesium, sulphur).

The basic structural unit of clay minerals is composed of a silica tetrahedron block and an aluminum octahedron block (Fig. 2.1.). Various clay minerals are formed by a variety of combinations and arrengements of these blocks. The sheet structure is formed by combining the basic structural unit. Silica sheet is the combination of silicon-oxygen tetrahedrons (Fig. 2.2.). Alumina sheet is formed by combination of alumina octahedrons (Fig. 2.3.).

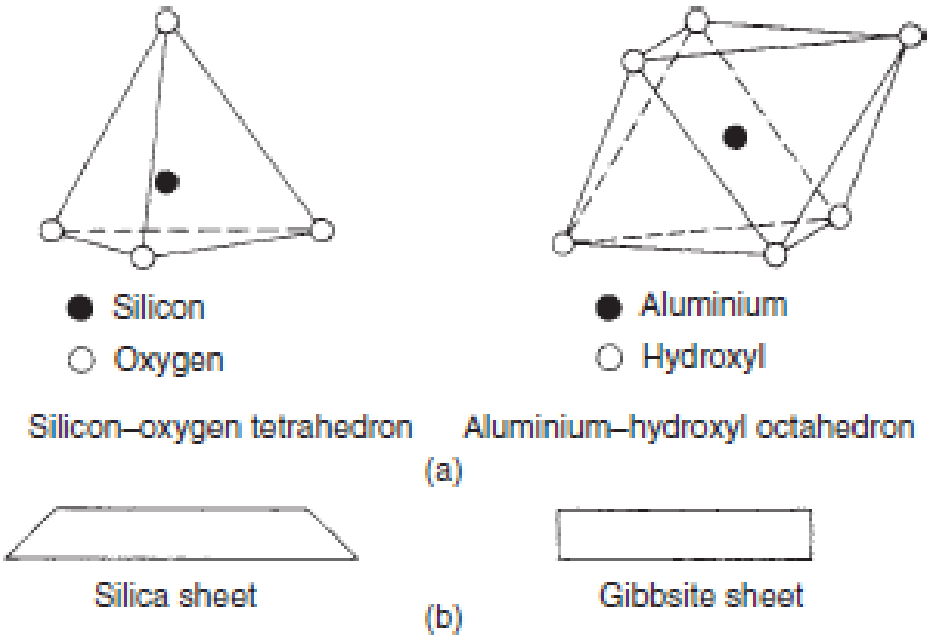


Figure 2.1. Basic Units of Clay Minerals (Craig, 2004)

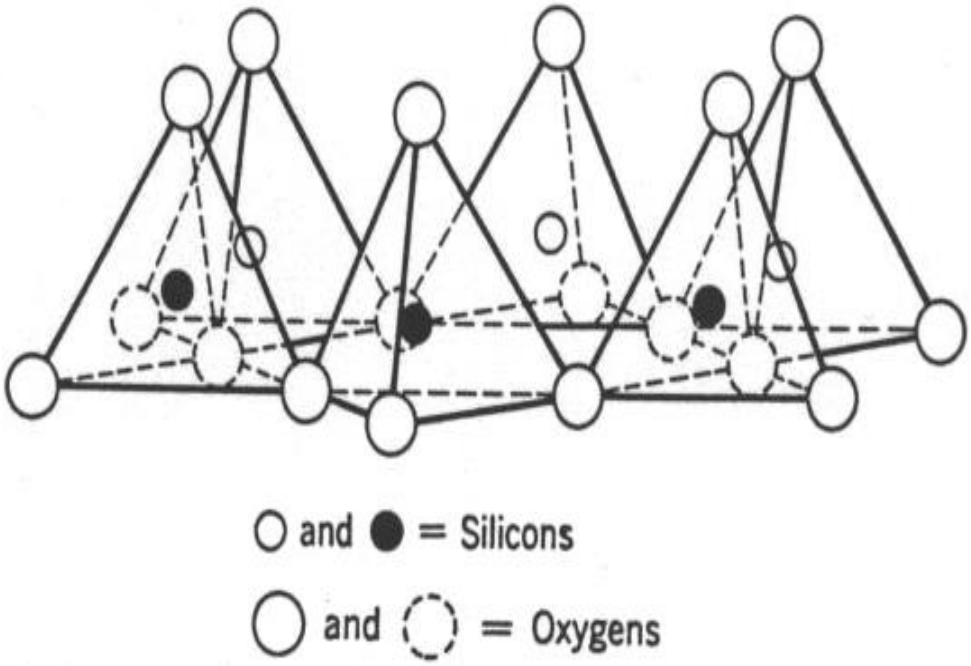


Figure 2.2. Silica Sheet (Mitchell, 1993)

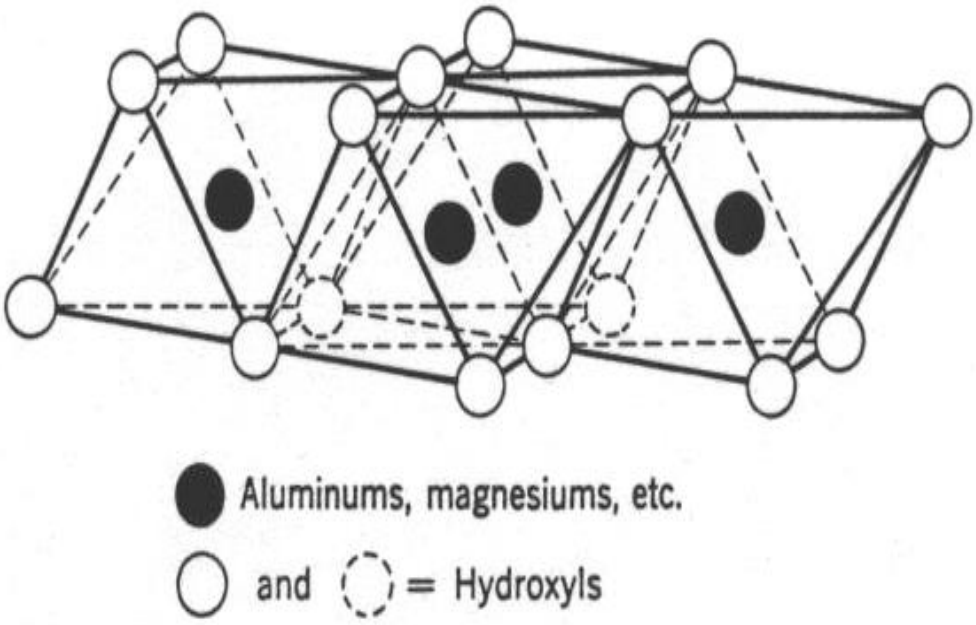


Figure 2.3. Alumina Sheet (Mitchell, 1993)

Kaolinite, illite and montmorillonite are the main groups of clay minerals. Characteristics of some clay minerals are given in Table 2.1

All three of these groups have layered crystal structures. The mineralogical distinction is based on the physical arrangement of the different layers and the manner by which the individual structural units are bonded together. These bonds are hydrogen bonds, potassium bonds and van der Waals bonds.

The specific surface is defined as the total area of the surface of the grain expressed in square centimeters per gram or per cubic centimeter of the dispersed phase. The specific surface increases from kaolinite mineral to montmorillonite mineral.

These types of clay minerals is classified with respect to Atterberg limits, which are liquid limit, plastic limit and shrinkage limit. While the values of the liquid limit and plastic limit of montmorillonite group are the highest in clay minerals, this group has the lowest shrinkage limit in the clay minerals.

As activity is one of the descriptive property of clay minerals, the activity is used for classification of clay mineral. The activity value of clay minerals depends on plasticity index and percentage of clay particle in soils and as activity increases, the swell potential increases.

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

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

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

2.1.1. Kaolinite Group

Kaolinite, which has the general chemical composition of $2SiO_2Al_2O_32H_2O$, is produced from the weathering of the parent rocks that have orthoclase feldspar.

Kaolinite consists of a structure based on a single sheet of silica combined with a single sheet of alumina octahedrons (Figs 2.4, 2.5). Hydrogen bonding is between silica sheet and alumina sheet. When kaolinite sheets are stacked on each other, the hydroxyl of octahedron sheets are drawn to the oxygen of the silica tetrahedron sheet by means of oxygen bonds. Such bonds, which are ionic and covalent, are strong, but not as strong as the primary bonds and, therefore, cleavage occurs. These sheets can expand substantially in two directions and generally such crystals are 70 to 100 layers thick (Oweis and Khera, 1998).

Figure 2.4. Kaolinite (Craig, 2004)

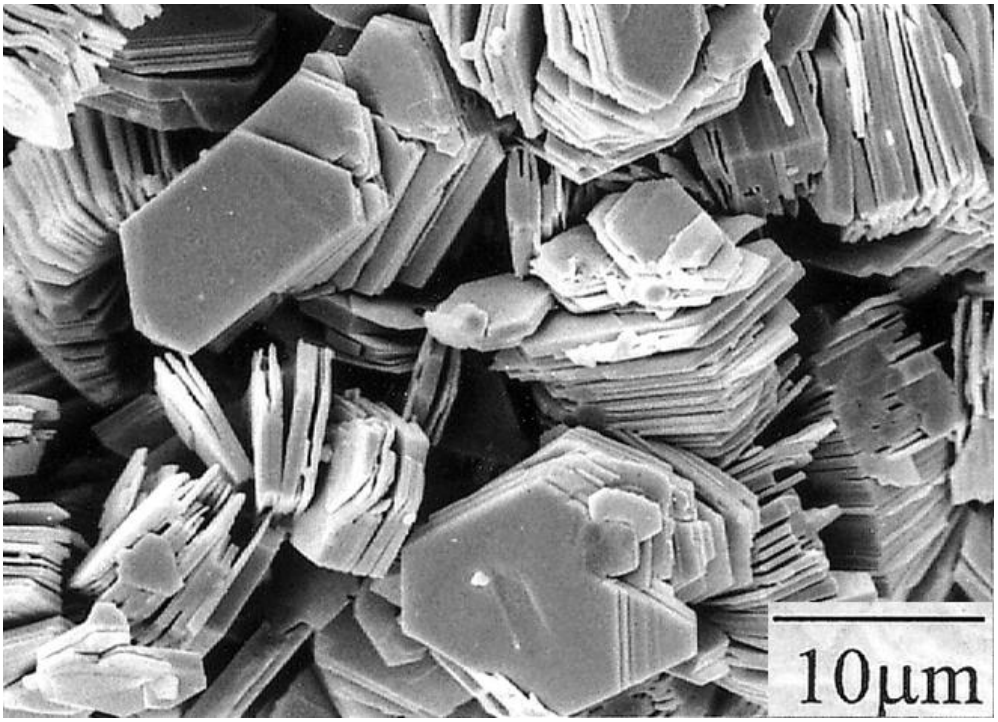


Figure 2.5. SEM of Kaolinite
(source: http://claymin.geoscienceworld.org/content/37/1/1/F2/graphic-3.large.jpg)

2.1.2. Illite Group

Illite is obtained from the weathering of the micas with the major parent mineral of muscovite. Illite has a basic structure consisting of a sheet of alumina octahedrons between and combined with two sheets of silica tetrehedrons. Potassium ions, fairly weak bonding, is between combined sheets (Figs 2.6, 2.7). Although the illite group of minerals has the same structural arrangement such as the montmorillonite group, illite mineral has less swelling potential than montmorillonite mineral due to the presence of potassium as the bonding material between units.

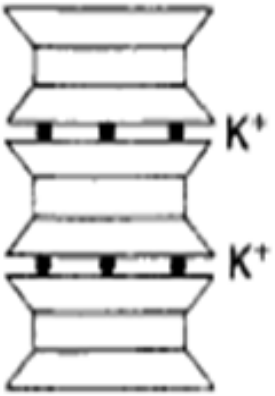


Figure 2.6. Illite (Craig, 2004)

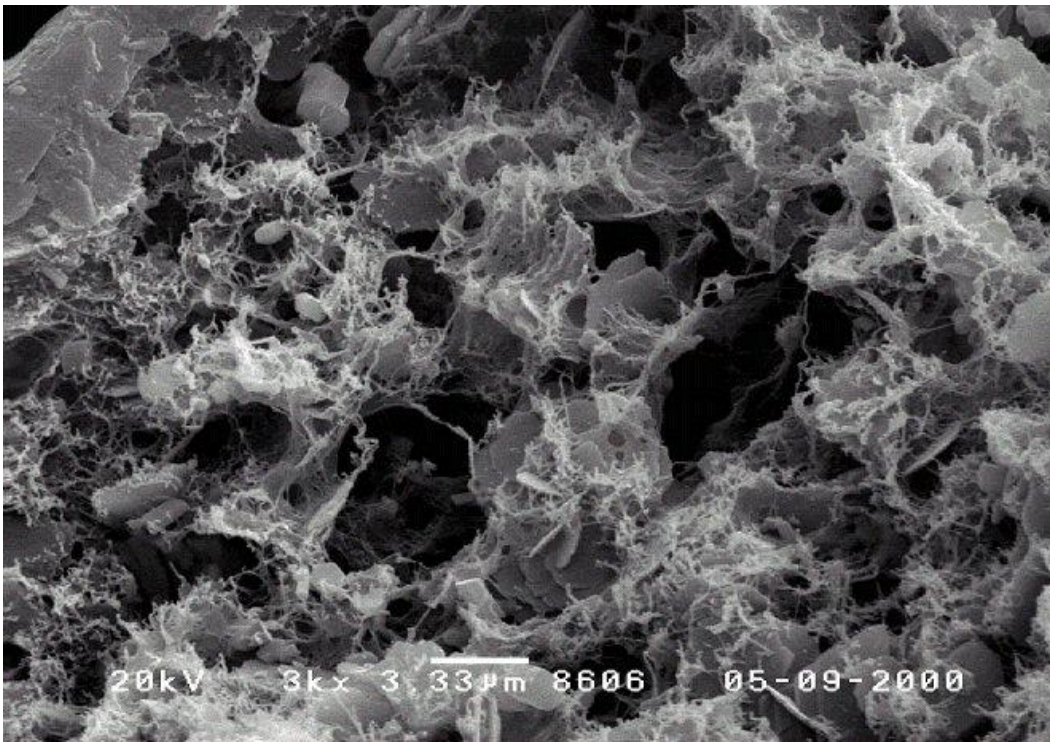


Figure 2.7. SEM of Illite

(source:http://www.ssokinc.com/sheet_graphics/Illite.jpg)

2.1.3. Montmorillonite Group

Montmorillonite has the same basic structure as illite. The water molecules and exchangable cations occupy the space between the combined sheets (Figs 2.8, 2.9). Montmorillinite is the main constituent of bentonite, derived by weathering of volcanic ash. It can expand due to its expandable nature when it comes incontact with water. The soils including a significant amount of montmorillonite minerals will exhibit high swelling and shrinkage characteristics.

Montmorillonite has the highest swelling potential, activity and liquid limit in clay soils. Montmorillonite including calcium has lower cation exchange capacity and swelling potential with regards to sodium montmorillonite. Bentonite, which is a type of montmorillonite, is extensively used for stability of boreholes in subsurface investigation and slurry wall construction, includes both sodium bentonite and calcium bentonite. However, the proportion of sodium bentonite is higher than the proportion of calcium bentonite (Oweis and Khera, 1998).

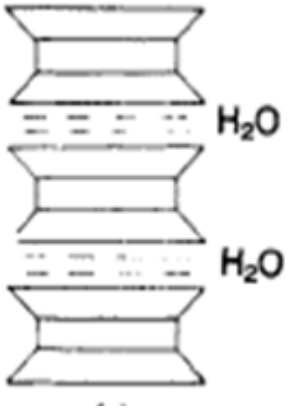


Figure 2.8. Montmorillonite (Craig, 2004)

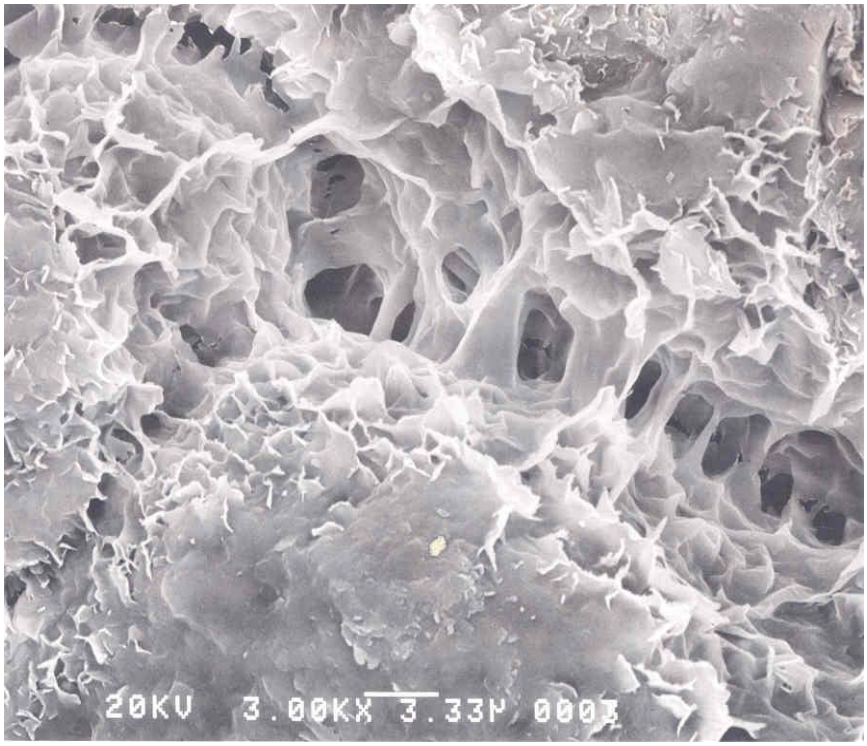


Figure 2.9. SEM of Montmorillonite

(source:http://www.petrotech-assoc.com/images/smectite_2.jpg)

2.2. Diffuse Double Layer

In a soil system, the cations present in the pore water are electrostatically attracted to the negatively charged surfaces of clay particles. At the same time, these cations tend to diffuse back to the bulk pore fluid (Figs 2.10), where their concentration is smaller (Van Olphen,1977). These two opposite trends give rise to a spacial ionic distribution in the liquid surrounding the charged surface, called the diffuse double layer (DDL).

The valence and concentration of cations are the main factors that affect the thickness of diffuse double layer. The cation that has higher ionic valence has smaller the thickness of the double layer. The cations that has lower valence cause bigger thickness of DDL. For example, smectite with Na^{+} has higher swelling capacity in comparison to that of smectite with Ca^{2+}.

The lower concentration of cations result in increase in DDL and swelling (Mitchell, 1976). The repulsive force between DDL system is formed by cations that is high concentration near the surface of clay particle (Chen, 1988).

The other factor that affect the thickness of DDL is temperature. When temperature increases, the thickness of DDL increases.

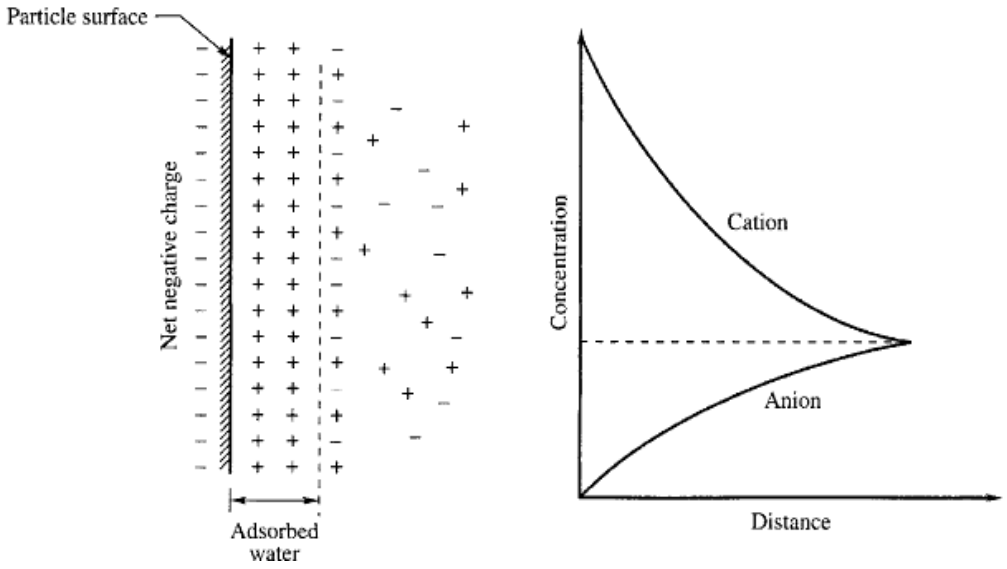


Figure 2.10. Distribution of cations and anions adjacent to a clay particle surface according to the diffuse double layer theory (Keijzer, 2000).

2.3. Clay Structure

The orientation of the soil particles in the soil mass and the spacing between the particles will affect the manner in which the particles interact. The structures of clay particles are divided into two groups, dispersed structure which formed when the net particle force is repulsive and floccuated structure which formed when the net particle force is attractive (Figs 2.11). The spacing between the particles is larger in the flocculated structure than in the dispersed structure. For this reason, flocculated clays tend to be more expansive than dispersed clays.

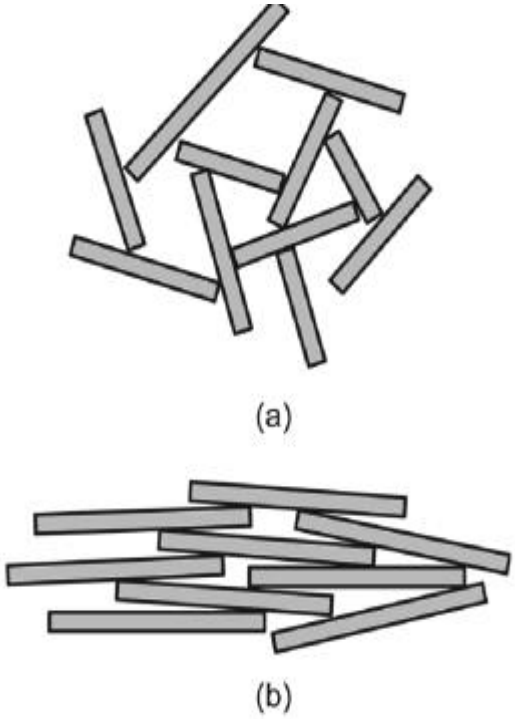


Figure 2.11. (a) Floccuated structure (b) Dispersed structure (Lambe and Withman, 1969)

2.4. Cation Exchange Capacity (CEC)

The cation exchange capacity can be defined as the ability of a mineral to adsorb external cations. The CEC of a soil is the number of cations in miliequivalents that neutralize one hundred grams of dry clay (meq/100 g). One miliequivalents is one miligram of hydrogen or any ion that will combine with one miligram of hydrogen or displace it (Oweis and Khera, 1998). The value of the CEC is different with respect to types of the clay minerals (Table 2.2).

CEC is one of the significant factor for determining clay mineral properties. The surface area and the charge on this surface area, which are two of the fundamental properties of clays, are measured by CEC. The surface of a clay is investigated two parts as external and internal (Figure 2.12).

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

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

The external exchange capacity depends on the number of bonding sites of cations on the external surfaces. The external CEC is a direct function of the crystal size: for a given volume or mass, the bigger the external surfaces, the smaller the crystal size. Therefore, the measurement of the external CEC gives information on the mean crystal sizes. The internal exchange capacity reflects the overall charge imbalance on the layer structure and the absorption capacity of the clays.

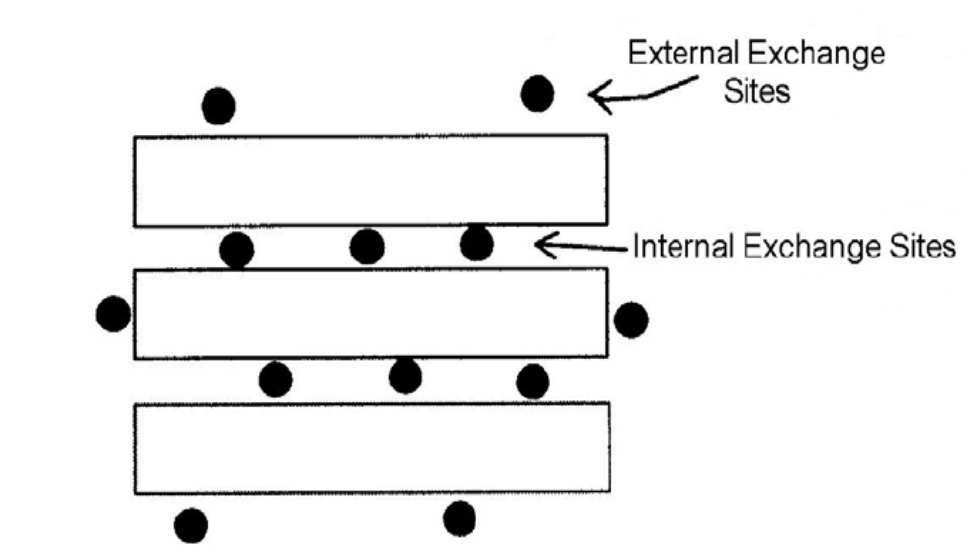


Figure 2.12. Different Types of Exchange Sites on Clay Particles, Surface and Absorbed Ion Interlayer Sites

2.5. Mechanism of Swelling

The swelling of clay sustains two primarily basic mechanisms (Fig 2.13). One of them is the swelling between soil particles. This mechanism is seen in all clay minerals. Capillary hole between clay crystals in clay deposits which can be assessed as dry bonds, keeps these crystals together by its water vacuum force. Swelling of clay occurs as a result of clay unit is subject to water. The other mechanism is simply defined as swelling occurred in clay crystal. This swelling is usually happened in montmorillonite group clay. When clay comes in content with under water, water comes in both clay crystals and poor-bonded singular layer formed these crystals. Therefore, swelling occurs due to increasing of volume of the clay soils (Popescu, 1986).

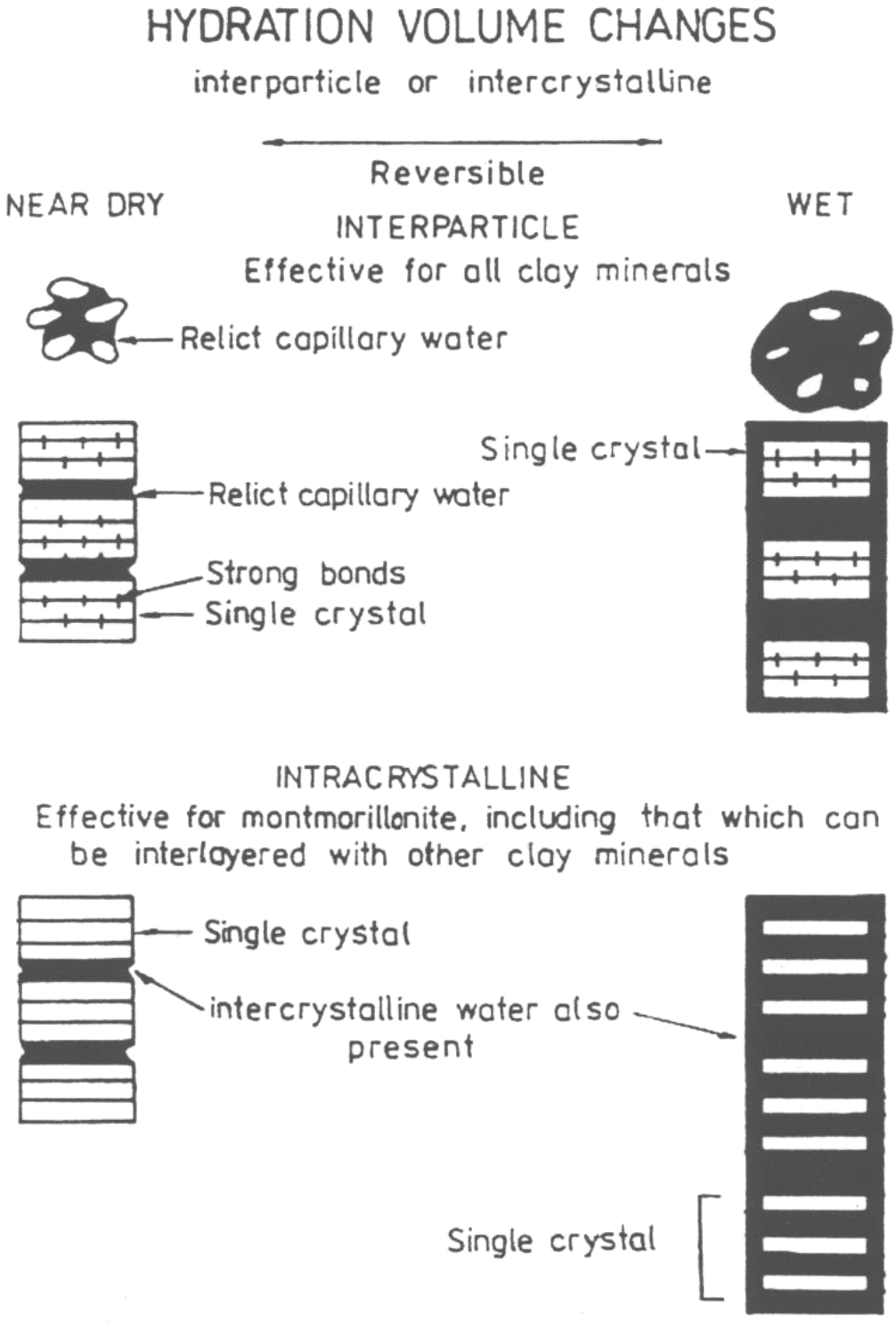


Figure 2.13. Mechanism of Swelling (Popescu, 1986)

2.6. Factors Influencing Swelling

Several factors, which are the type and amount of clay minerals present, the initial dry density (void ratio) and water content of the soil specimen, the nature of pore fluid, the type of exchangable cations, the overburden pressure and wetting and drying effects, influnce the swell potential and swelling pressure (Yong and Warkentin, 1975).

The factors which affect the swelling potential of an expansive soil can be investigated in three main groups (Table 2.3, 2.4, 2.5). These groups are soil properties influencing the nature of the internal force field, the environmental factors those may change the internal force system and the state of stress present on the soil.

Table 2.3. Soil Properties Influencing Swell Potential (Nelson and Miller, 1992)

FACTOR	DESCRIPTION
Clay Mineralogy	Three groups of clay minerals are kaolinite, illite and montmorillonite. Montmorillonite mineral is the highest swelling potential in expansive clay.
Dry Density	Higher densities cause closer particle due to the greater repulsive forces between particles, which cause larger swelling potential.
Soil Water Chemistry	Cation concentration, cation valence are negative effect for swelling. Na^{+} cations in the soil water causes more swelling than Mg^{2+} cations.
Plasticity	Soils which have higher plasticity index and higher liquid limit usually have more swelling potential than the others.
Soil Structure and Fabric	Dispersed clays are less expansive than flocculated clays. The soils compacted at higher water content or remolding cause the changes of fabric and structure.

Table 2.4. Environmental Factors Affecting Swell Potential (Nelson and Miller, 1992)

FACTOR	DESCRIPTION
Climate	Soil moisture is considerably affected by amount and variation of precipitation and evapotranspiration. Semiarid and arid regions are the most suitable regions for expansive soils .
Drainage	Poor surface drainage cause increase of moisture of soil.
Groundwater	Superficial and fluctuating water tables may cause heave.
Initial Moisture Content	Natural clayey soils, have smaller water content and high liquid limit, have a potential to swell.
Moisture Variations	The volume of expansive clay changes when moisture content of expansive soil in active zone alters.
Permeability	Soils, which have fissures and cracks, have high permeabilities. This case leads to higher rates of swell.
Temperature	An increase in temperature leads moisture to dissipate towards cooler areas, especially under pavements and buildings.
Vegetation	Soil moisture is depleted by vegetation, thus cause differential wetting of soil.

Table 2.5. Stress Conditions Affecting Swell Potential (Nelson and Miller, 1992)

FACTOR	DESCRIPTION
Stress History	overconsolidated clays have more swelling potential than the normally consolidated clays.
In Situ Conditions	The initial effective stress must be determined to assess the probable outcomes of loading or chancing the soil moisture.
Loading	External loading is either equal to or more than swelling pressure, soils do not expand.
Soil Profile	The thickness and location of potentially expansive layers in the profile considerably influence potential movements. Greatest movement will occur in profiles that have expansive clays extending from the surface to depths below the active zone. Less movement will occur if expansive soil is overlain by nonexpansive material or overlies bedrock at shallow depth

2.7. Treatment of Expansive Soils

Due to the increase in population and subsequent urbanization pressure, the use of expansive soil for construction purposes has been encouraged since 1930's. The various treatment options for improving expansive soils before and after construction of structures and highways are listed below (Nelson and Miller, 1992);

1. Chemical additives,
2. Prewetting,
3. Compaction,
4. Soil removal replacement,

5. Surcharge loading,
6. Thermal methods

2.7.1. Chemical Additives

Chemical stabilization involves application of chemical admixtures to improve the behavior of soil. Chemical stabilization in the form of lime and cement materials and a combination of them is generally used in soil stabilization. In addition to this, the waste materials such as phosphogypsum, fly ash and ground granulated blast furnace slag are used in stabilization of expansive soil. It is used for reducing the permeability of the soil, improving the shear strength, increasing bearing capacity, decreasing the settlement and expediting the construction.

Treatment of expansive soils is either to replace the soil to a rigid granular mass, the particles of which are adequately strongly bound to resist the internal swelling pressure of the clay, or to slow down moisture movement within the soil, provided the retardation is sufficient to overcome normal seasonal changes that is sufficient for practical purposes. Impervious membranes are used but if the area is totally isolated with these membranes, they will be barely to extend the path and hence time of the moisture movement. To improve the mechanical properties of a soil, densification is usually a practical method if additional stabilization systems are used. Chemical admixture stabilization has been widely applied in both shallow and deep stabilization to treat inherent properties of expansive soils (Sivapullaiah, 2006)

2.7.1.1. Lime Stabilization

Lime stabilization method is a common method all over the world for a long time (Chen, 1988). Lime treatment in cohesive soils generally causes a decrease in

plasticity, dispersion, and volume change potential, and an increase in particle size, permeability, and strength (Nalbantoglu and Gucbilmez, 2001).

To decrease swelling potential and swelling pressures in clays, lime stabilization has been commonly used. Generally the strength of wet clay is developed with the addition of lime to soil. The treatment in strength is partly due to the decrease in plastic properties of the clay and partly to the pozzolanic reaction of lime with soil, which produces a cemented material that increases in strength with time. Lime-treated soils have generally greater strength and a higher modulus of elasticity than untreated soils. Recommended percentages of lime for soil stabilization for soils with more than 50 percent silt-clay fraction the percent of lime should lie between 5 and 10 (Murthy, 2002).

The amount of calcium ions (Ca^{+2}) and magnesium ions (Mg^{+2}) increases during the addition of lime to clay. These ions tend to displace other common cations such as sodium (Na^{+1}) or potassium (K^{+1}), in a process known as cation exchange. The plasticity index of clay crucially decreases due to the replacement of sodium or potassium ions with calcium. A reduction of plasticity generally results in the reduction of swelling potential. In addition to this, the addition of lime causes increment of soil PH, which also rises the cation exchange capacity (Çokça, 2001).

Nalbantoglu (2006) states that the microstructure of stabilized soil changed after mixing a reactive clay with chemical additives. Reactions between soil–lime–water resulted in attack at sites of structural weakness of clay mineral and caused the crowding of calcium ions on the clay mineral surface. Soil engineering properties and visible changes at the microstructure level of the stabilized soils was changed.

The chemical theory involved in the lime reaction is complex. The primary reactions include cation exchange, flocculation-agglomeration, lime carbonation, and pozzolanic reaction. All fine-grained soils can be modified to some degree to exhibit less plasticity and improved workability using lime treatment. The strength characteristics of a lime-treated soil depend primarily on soil type, lime type, lime percentage, and curing conditions (time-temperature) (Nelson & Miller, 1992).

A View of Stabilizing Expansive Soil with Lime is presented in Figure 2.14.

Figure 2.14. A View of Stabilizing Expansive Soil with Lime (source: http://www.streumaster.com/technology/soil-stabilisation/)

2.7.1.2. Cement Stabilization

Portland cement releases a large amount of lime during the hydration. For this reason, the result of mixing cement with clay soil is similar to that of lime. Although it decreases liquid limit, plasticity index, swelling potential, it increases the shrinkage limit and shear strength (Chen, 1988). However, lime is more effective than the portland cement for the stabilization of the highly plastic clay. Cement may not hydrate sufficiently to produce the complete pozzolanic reaction due to the some clays which have such a high affinity for water. If soils are not lime reactive, cement stabilizations are usually advengetous (Mitchell and Raad, 1973).

Cement stabilization of expansive soils or soft soils, which involves mixing of soils with cement and water (Figures 2.15 , 2.16), subsequent compaction of this mix to a

high density makes the material more resistant to variations in physical, thermal and chemical stresses (Winterkorn and Pamukcu, 1991). Cement stabilization is limited due to the some factors; these factors are high cost, possible corrosion of the soil environment, brittle failure and low temperature cracking as a result of the hydration and moisture loss, and proneness to sulphates attack (Punthutaecha, 2002).

Cement stabilization develops from the cementitious bonds between the calcium silicate (three-calcium silicate) and aluminate hydration products of cement and the soil particles. The action of cement declines both the interlayer and the intermicellar swelling (Van der Kerkhof, 2001).

Cement-stabilization applied either at surface or at deeper depths has been used for many various applications including foundations which is built for stopping the settlement of existing structures, retaining structures which are constructed for temporary support of excavations, liquefaction mitigation (to cut off water infiltration into excavations and sewer lines), water control (foundation curtains that reduce water loss under structures), and pollution control by using stabilization/solidification (S/S) techniques that aim to immobilize the source of contaminants (remediation of contaminated land prior to redevelopment) (Stavridakis, 2006).

The cement requirement depends on the gradation of the soil. Recommended percentages of cement for plastic clay soils require 13 percent or more (Murthy, 2002).

Figure 2.15. A View of Stabilizing Expansive Soil with Cement (source: http://www.griffinsoil.com/soil_stabilization_manual_and_videos)

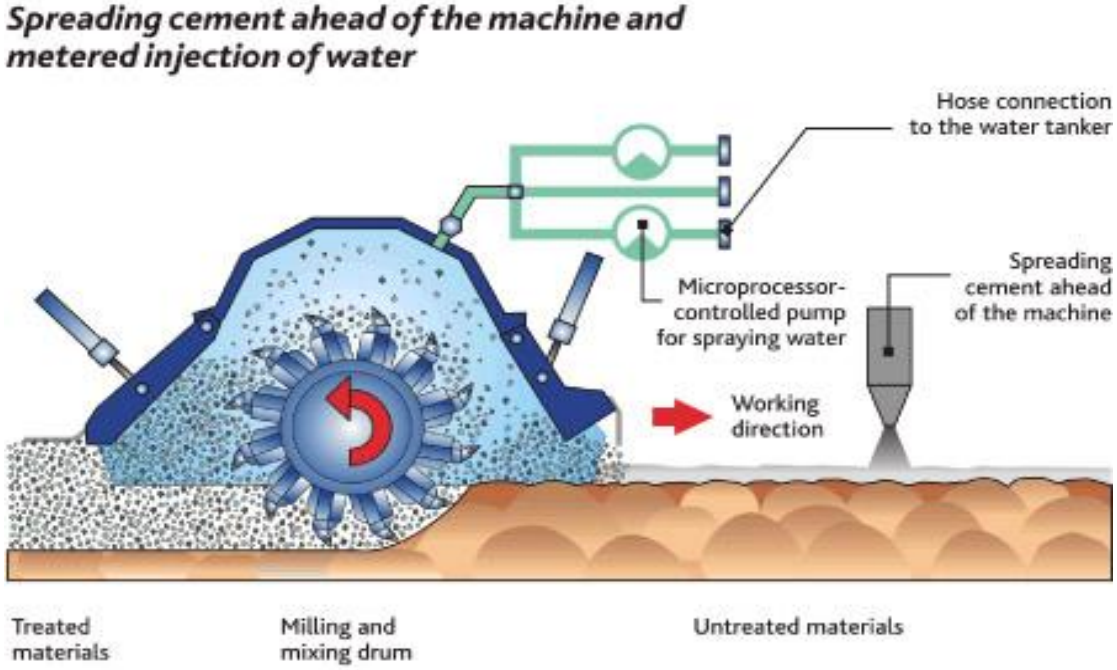


Figure 2.16. A View of Stabilizing Expansive Soil with Cement (source: http://www.trstabilisation.co.uk/Soil-Stabilisation.html)

2.7.1.3. Fly Ash Stabilization

Fly ash is obtained during the combustion of ground or subbituminous coals. It can be adapted to a wide range of stabilization applications due to the exhibiting self-cementing characteristics. Generally, applications for fly ash treatment can be divided into several categories: drying soils to facilitate compaction, treatment of soils to reduce shrink-swell potential, and stabilization to improve engineering properties of a material such as increased strength, durability, or stability (Stavridakis, 2006). Fly ash treatment can effectively reduce the swell potential of expansive soils and increase support capacity of pavement subgrades (Kate, 1998).

Stavridakis (2006) stated that fly ash is divided into two groups, Class C and Class F. Class C is produced the results of the combustion of the subbituminous coal. This fly ash is rich in terms of calcium. For this reason, this fly ash exhibits so self-cementing characteristic that additive materials such as lime and portland cement do not need to add. Class F is poor in calcium with regards to Class C. Therefore, it is necessary that the addition of materials which cause the pozzolanic reactions for the production of cementitous products.

The fly ash consists primarily of CaO and SiO_2 which affect intramicellar (interlayer) and intermicellar expansion, which are the two fundamental types of swelling, with respect to the amount of these oxides. The reduction of swell potential by using fly ash is generally comparable to lime and cement stabilization. This reduction based on the amount of CaO and SiO_2, appears to be a result of bonding of the soil particles by the cementitious products formed during hydration of the fly ash in addition to flocculation of the clay minerals. As a result, a skeletal cementation can be occured with the assistance of portland cement, so this condition provides the increasing of the strength and the durability of swelling material.

2.7.1.4. Phosphogypsum Stabilization

In this study, easily available industrial by-product (Phosphogypsum) is used for the enrichment of expansive soil properties, since the studies for treatment of expansive soils with phosphogypsum are very limited in literature.

The use of Phosphogypsum (PG) in soil stabilization can lead to low-cost construction and can provide an environmentally friendly means of its disposal and also enhance the engineering properties of the expansive soil.

The reaction sequence of the stabilization of a clay with phosphogypsum can be as follows:

In the wet process the phosphate content of the rock is converted by concentrated sulphuric acid to phosphoric acid and a calcium sulphate residue (phosphogypsum) in either dihydrate ($CaSO_4.\ 2H_2O$) or hemihydrate form ($CaSO_4.1/2H_2O$):

$$[Ca_3\ (PO_4)_2]_3CaF_2+10H_2SO_4+10nH_2O \rightarrow 6H_3PO_4+10(CaSO_4\ nH_2O)\ +2HF$$

Phosphate Ore + Sulphuric Acid → Phosphoric Acid + Phosphogypsum

Phosphogypsum is mainly $CaSO_4 \cdot 2H_2O$ but also contains impurities such as free phosphoric acid, phosphates, fluorides and organic matter (that adhere to the surface of the gypsum crystals**).**

The reaction sequence of the stabilization of clay with phosphogypsum and ettringite formation is presented below (Ismaiel, 2006).

$$Al_4\ Si_4\ O_{10}\ (OH)_8 + 4(OH)^- + 10\ H_2O \longrightarrow 4\ Al\ (OH)_4^- + 4\ H_4\ Si\ O_4$$

(Dissolution of kaolinite at Ph > 10.5)

$$Ca\,SO_4 \cdot 2\,H_2O \longrightarrow Ca^{+2} + SO_4^{2-} + 2\,H_2O$$

(Dissolution of gypsum)

$$6\,Ca^{+2} + 2\,Al(OH)_4^- + 4(OH)^- + 3\,(SO4)^{2-} + 26\,H_2O \longrightarrow$$

$$Ca_6\,[Al(OH)_6]_2\,(SO4)_3\,26\,H_2O$$

(Formation of ettrengite)

Hunter (1988) told that lime treated sulphate bearing clays swelled and disintegrated after a few years when used for road constructions. The possible swelling mechanisms associated with ettringite formation caused this problem.

2.7.1.5. Ground Granulated Blast Furnace Slag Stabilization

During the steel manufacturing, ground granulated blast furnace slag (GGBFS) is produced as a byproduct by cooling the molten slag using high pressure water jets. GGBFS has cementitious properties when this material is crushed to obtain very fine cement sized particle. It mainly consists of silicates, alumina-silicates and calcium alumina-silicates (Bhuyan and Singh, 2010). GGBFS is used in many applications for good durability, high resistance to chloride penetration, resistance to sulphate attack and protection against alkali silica reaction (RahmanOuf, 2001).

2.7.2. Pre-wetting

The purpose of prewetting is to allow desiccated foundation soils to swell before the construction. Ponding or submerging of an area in water is one of the most widespread wetting methods. However, wetting the foundation subsoil by ponding

may require many months or even years to increase the water content to the required depths due to the clay chemical structures. Prewetting with the grid of vertical wells may be enabled before the flooding. This can decrease the time neccesary to arrange the soil moisture content at point where maximum heave will occur within a few months of flooding (Stavridakis, 2006). Prewetting method is more economic than the other methods. However, as the soil becomes saturated, bearing capacity of the soil reduces.

2.7.3. Soil Removal and Replacement

Soil removal and replacement with non-expansive soils is one method of the stabilization of expansive soil. If non-expansive soil is compacted at higher density than expansive clay, it produces high bearing capacity. Expansive soil should be removed and replaced by non-expansive soil fill to a depth necessary to prevent excessive heave. Chen (1988) suggested that this depth should be minimum 1 to 1.3 m. Removal and replacement require less delay to construction than some other procedures such as prewetting (Nelson & Miller, 1992).

2.7.4. Compaction

The swelling potential of expansive soil can be decreased by reducing the dry density of this soil. In other words, the expansive soil with a low dry density may have less swelling potential with respect to expansive soil with a higher dry density. Holtz (1959) stated that compaction of expansive soil, which is compacted at lower density and at water content above the optimum moisture content produces less swell potential than compaction of this soil, which is compacted at high density and low moisture content. Some soils have such a high potential for volume change that compaction control did not reduce swell potential significantly (Nelson & Miller, 1992).

2.7.5. Surcharge Loading

Surcharge loading can be effective in conditions such as when swell pressures are low and some deformations can be tolerated. The depth of the active zone and the maximum swell pressures can be determined with soil testing program before the surcharge loading. In addition to this, drainage control can be done during the surcharge. The surcharge method is most effective when swell pressures are low and some heaving can be tolerated in the construction project, such as in a secondary highway system (Nelson & Miller, 1992).

2.7.6. Thermal Methods

Little work has been done in the United States to study or apply thermal treatment to expansive soil. Heating clays to approximately 200°C can significantly reduce the potential for volume change. However, practical, economical methods have not yet been developed (Nelson & Miller, 1992).

CHAPTER 3

EXPERIMENTAL STUDY

3.1. Purpose

The purpose of the experimental study is to investigate the effects of addition of phosphogypsum on Atterberg limits, swelling potential and rate of swell of an expansive soil. In addition to this, the effect of curing on swelling potential and rate of swell of an expansive soil treated with phosphogypsum, granulated blast furnace slag and lime are investigated.

3.2. Material

Kaolinite, bentonite, phosphogypsum, lime and ground granulated blast furnace slag (GGBFS) were used for the investigation (Figure 3.1).

Kaolinite: Kaolinite is taken from Kaolin Industrial Minerals Company in the form of gravel sized grains. These grains are crushed and passed through # 40 sieve before the study. The specific gravity was 2.57 and liquid limit, plastic limit and plasticity index were found as 46%, 28% and 18%, respectively.

Bentonite: Bentonite is obtained from Karakaya Bentonite Factory which is situated in Esenboğa/ Ankara. These grains are crushed and passed through # 40 sieve before the study. The specific gravity was 2.53 and liquid limit, plastic limit and plasticity index were found as 410, 34 and 376 respectively.

Phosphogypsum: Phosphogypsum was a production of Bandırma Gübre Fabrikaları A.Ş. (Bandırma Fertilizer Plant). The specific gravity of this sample was 2.24.

Lime: Lime was obtained from Akyüz Kireç Sanayi Ltd. Şti. (Akyüz Lime Industry). It was sieved through # 40 sieve before usage. Specific gravity of the lime was defined as 2.41. Chemical composition of the lime that was obtained from supplier is given in Table 3.1.

Ground granulated blast furnace slag: Ground granulated blast furnace slag was a waste material of Ereğli Demir ve Çelik Fabrikaları T.A.Ş. (Ereğli Iron and Steel Factory). Before this study, this material was crumbled with hammer and was sieved through # 40 sieve. The specific gravity of this sample is 2.71.

a) Kaolinite

b) Bentonite

c) Phosphogypsum

d) Lime

e) Grand granulated blast furnace slag

Figure 3.1. Views of Materials Used in the Study

Table 3.1. Chemical Composition of Kaolinite, Bentonite, Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag

Composition (%)	**Kaolinite**	**Bentonite**	**PG**	**Lime**	**GGBFS**
SiO_2	51.00	61.28	0.44	< 0.10	29-38
Al_2O_3	33.50	17.79	0.88	< 0.10	14-22
TiO_2	0.45	*	*	*	*
Fe_2O_3	0.85	3.01	0.32	< 0.10	*
CaO	0.15	4.54	32.04	95.55	31-40
MgO	0.25	2.10	*	0.90	7-11
Na_2O	0.15	2.70	0.13	*	*
K_2O	1.00	1.24	*	*	*
SO_3	*	*	44.67	*	*
P_2O_5	*	*	0.50	*	*
F	*	*	0.79	*	*
Loss on Ignition	12.65	*	21.06	3.20	*

*Not determined

3.3. Preparation of Test Samples

Sample which is a potential expansive soil was prepared by mixing kaolinite and bentonite in the laboratory (Fig 3.2 and Table 3.2). This sample was named as Sample A. Sample A consists of 85% kaolinite and 15% bentonite. 10% phophogypsum was added to Sample A to determine the effect of stabilizer, the preliminary test was done before the main tests. The preliminary test result was positive (i.e. swelling potential decrease by adding phosphogypsum), than twelve samples were prepared by adding phosphogypsum, lime and ground granulated blast furnace slag with different percentages to Sample A. All materials, which

could pass through No.40 sieve and were oven-dried at 50°C for one day, were used in this study. They were mixed with trowel and then were sieved two times through No.20 sieve to obtain homogeneous mixture at each mixture.

Each mixture was prepared at specific dry density and water content. So, each mixture was prepared 90 gr. Then 10.35 gr water (distilled water) was added to sample. The dry density and water content of sample were 1.495 gr/cm^3 and 11.5%, respectively. During the study, twelve samples were prepared by adding phosphogypsum (varying from 5% to 25%), phosphogypsum&lime and phosphogypsum&ground granulated blast furnace slag. Each test was repeated at least two times and average values reported.

For curing, samples were put into plastic bag and put into moisture room with 70% moisture and 22-25°C temperature.

Table 3.2. Composition of Prepared Specimens by Dry Weight

Sample	**Kaolinite+ Bentonite (%)**	**PG (%)**	**Lime (%)**	**GGBFS (%)**
Sample A	100	-	-	-
5% PG	95	5	-	-
10% PG	90	10	-	-
15% PG	85	15	-	-
20% PG	80	20	-	-
25% PG	75	25	-	-
5% PG + 5% L	90	5	5	-
7% PG + 3% L	90	7	3	-
9% PG + 1% L	90	9	1	-
9.5% PG + 0.5% L	90	9.5	0.5	-
14% PG + 1%L	85	14	1	-
15% PG + 15% GGBFS	70	15	-	15

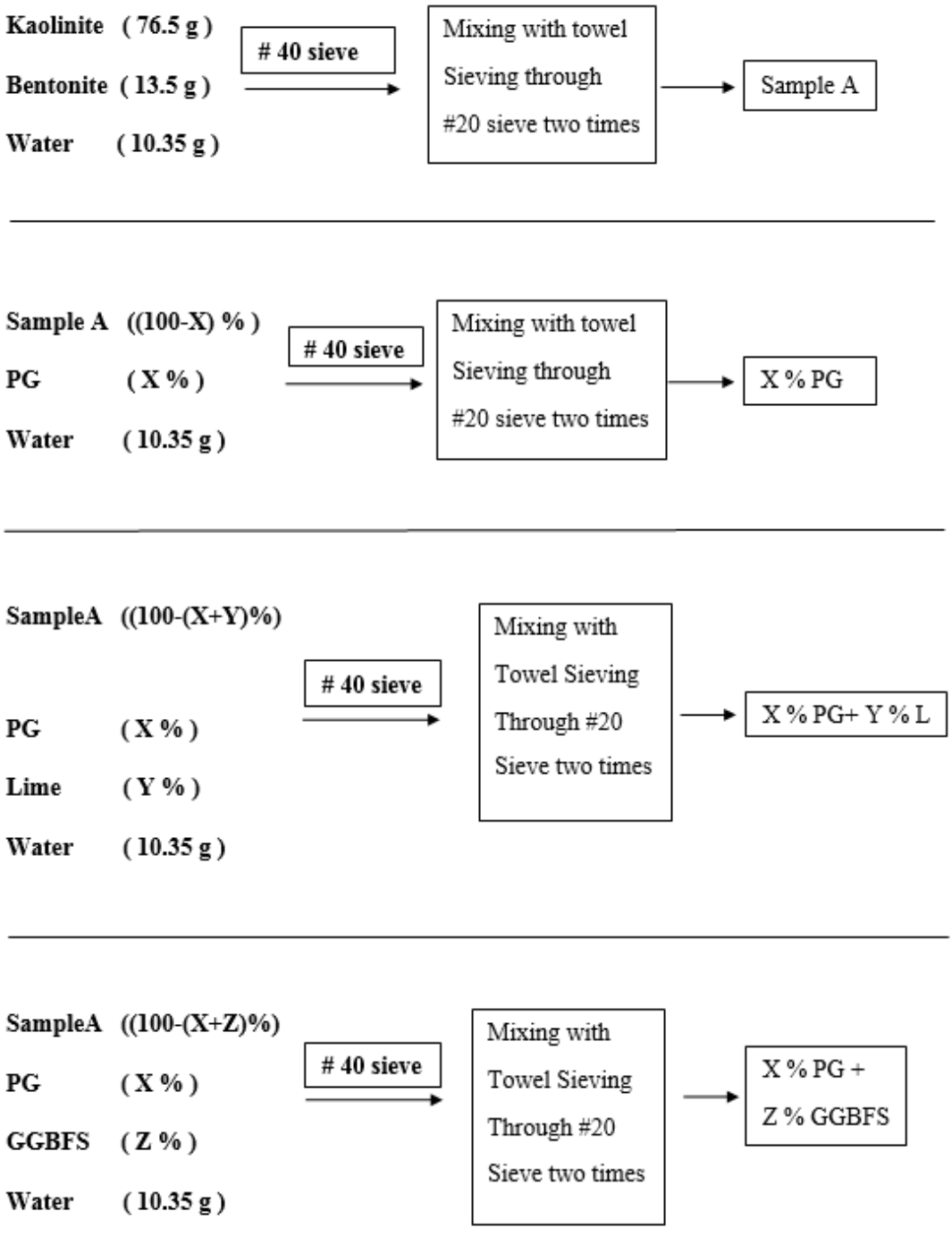


Figure 3.2. Preparation of Samples

3.4. Properties of Samples

Atterberg limits tests: liquid limit (LL), plastic limit (PL), shrinkage limit (SL), hydrometer tests, and specific gravity tests were done. Atterberg limits, shrinkage limit and hydrometer test were done according to ASTM D4318, ASTM C356-10 and ASTM D422, respectively.

Grain size distribution curves of samples treated with phosphogypsum and phosphogypsum&lime, phosphogypsum&ground granulated blast furnace slag were obtained. According to ASTM D422, hydrometer readings were taken continously for first 4 hours and the final reading was taken after 24 hours from the start of the test. However, grain size distribution curves for samples including phosphogypsum and ground granulated blast furnace slag could not be obtained due to the rapid settlement. This problem was solved by the increasing the mixture duration and performing the mixing process more carefully.

Atterberg limits were performed with respect to Casagrande method. The samples which were used in these tests passed through No.40 sieve. Before the liquid limit test and plastic limit test, the samples were mixed thoroughly with distilled water in the humid room and left for one day for soaking. The plastic limit test and liquid limit test in this study were done according to ASTM procedure, each test was repeated two times.

Specific gravity tests of the samples were done by adding gas oil instead of water. Because when water is added to clay soils, clay soils absorb water due to the chemical structures and this process cause excessive volume change. The specific gravity tests were done with respect to TS EN 196-2 (2013) in this study. This test procedure is as follows;

1. 50 gr dry soil sample was prepared.
2. Le Chatelier bottle was filled with gas oil to the point which was between 0 to 1 millimeters.

3. The bottle was waited in a water pool at room temperature until its temperature reached to room temperature.
4. After initial volume (V_1) was determined, the sample was added to this bottle with cone carefully.
5. When the cap of the bottle was closed, it was rotated on its own axes to obtain well mixture and avoid from air bubbles.
6. After the mixing process, the bottle was waited in water pool for 1 hour.
7. Finally, final volume (V_2) was determined.

Density of the Sample, $\rho = m / (V_2-V_1)$

where m: Mass of the sample (gram)

V_1: Final volume (cm^3)

V_2: Initial volume (cm^3)

According to Unified Soil Classification System (USCS), the samples were classified (Figure 3.3). Before the classification, the liquid limit and plastic limit of each sample were determined.

Swell potentials for each samples are estimated (Figure 3.4) with respect to classification chart recommended by Seed et al. (1962).

Grain Size distribution curves for sample A and treated samples were shown in Figure 3.5, 3.6 and 3.7, respectively.

The effect of addition of materials on specific gravity is shown in Figure 3.8.

The changes of liquid limit, plastic limit, plasticity index, shrinkage limit, shrinkage index and activity are illustrated in Figures 3.9, 3.10, 3.11, 3.12, 3.13 and 3.14, respectively.

Properties of samples are demonstrated in Table 3.3.

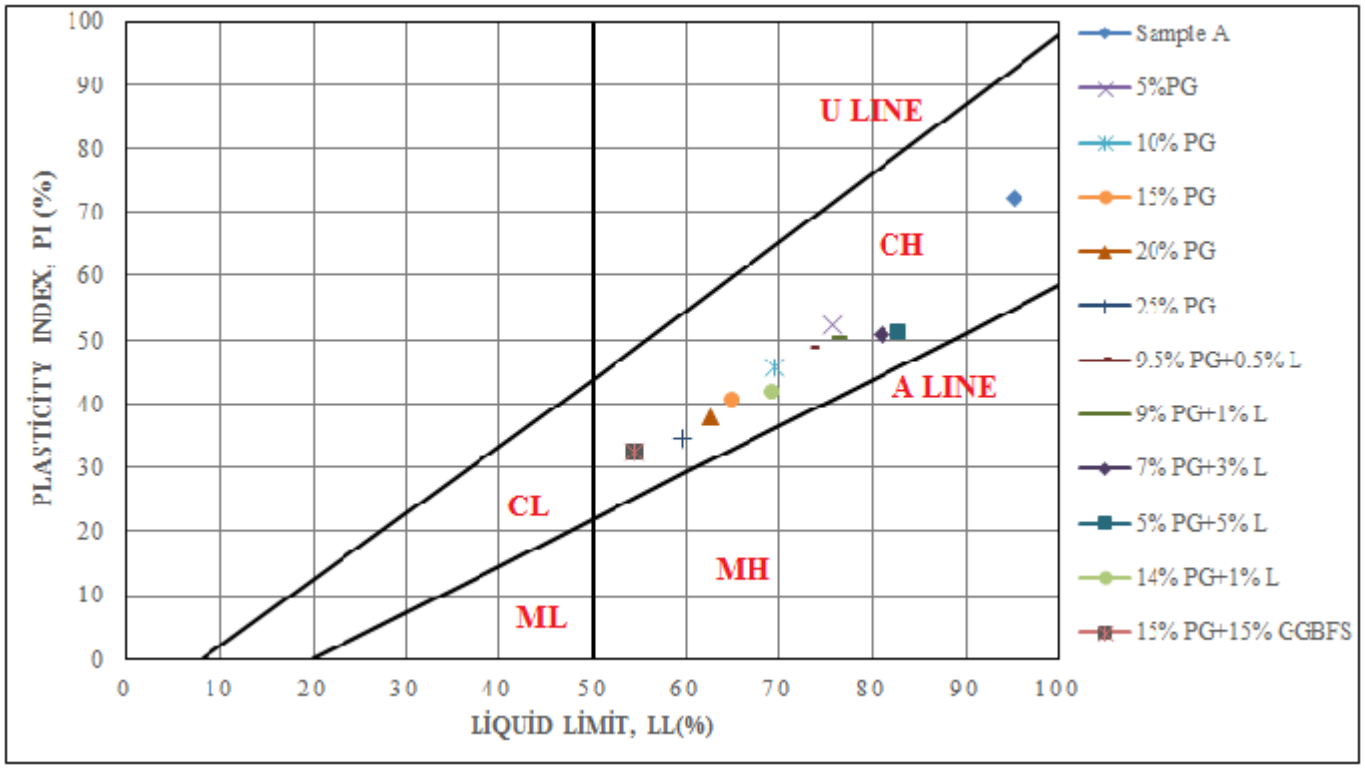


Figure 3.3. Plasticity Chart

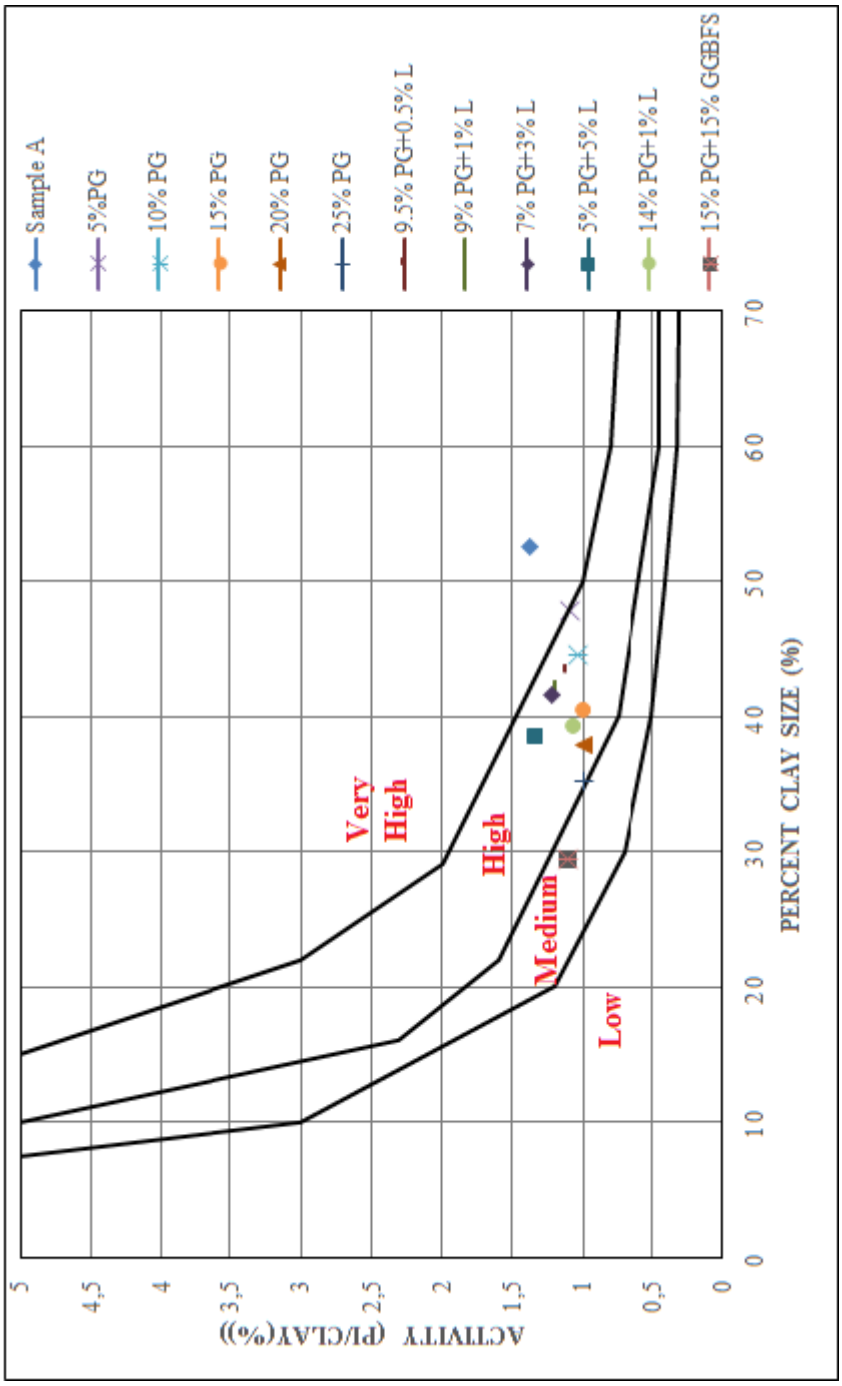


Figure 3.4. Classification Chart for Swelling Potential (After Seed, et al., 1962)

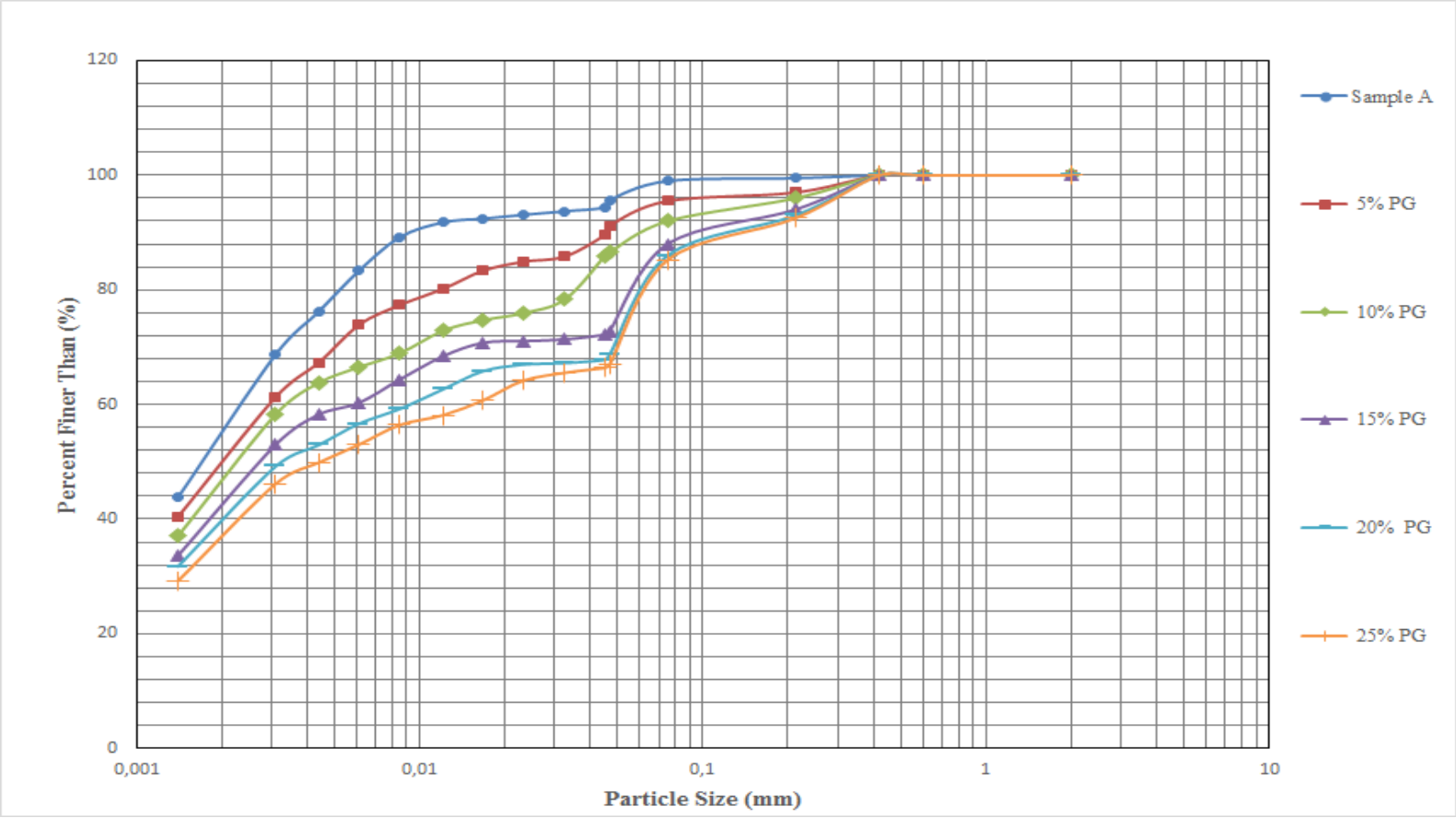


Figure 3.5. Grain Size Distribution Curves for Sample A and Phosphogypsum Treated Samples

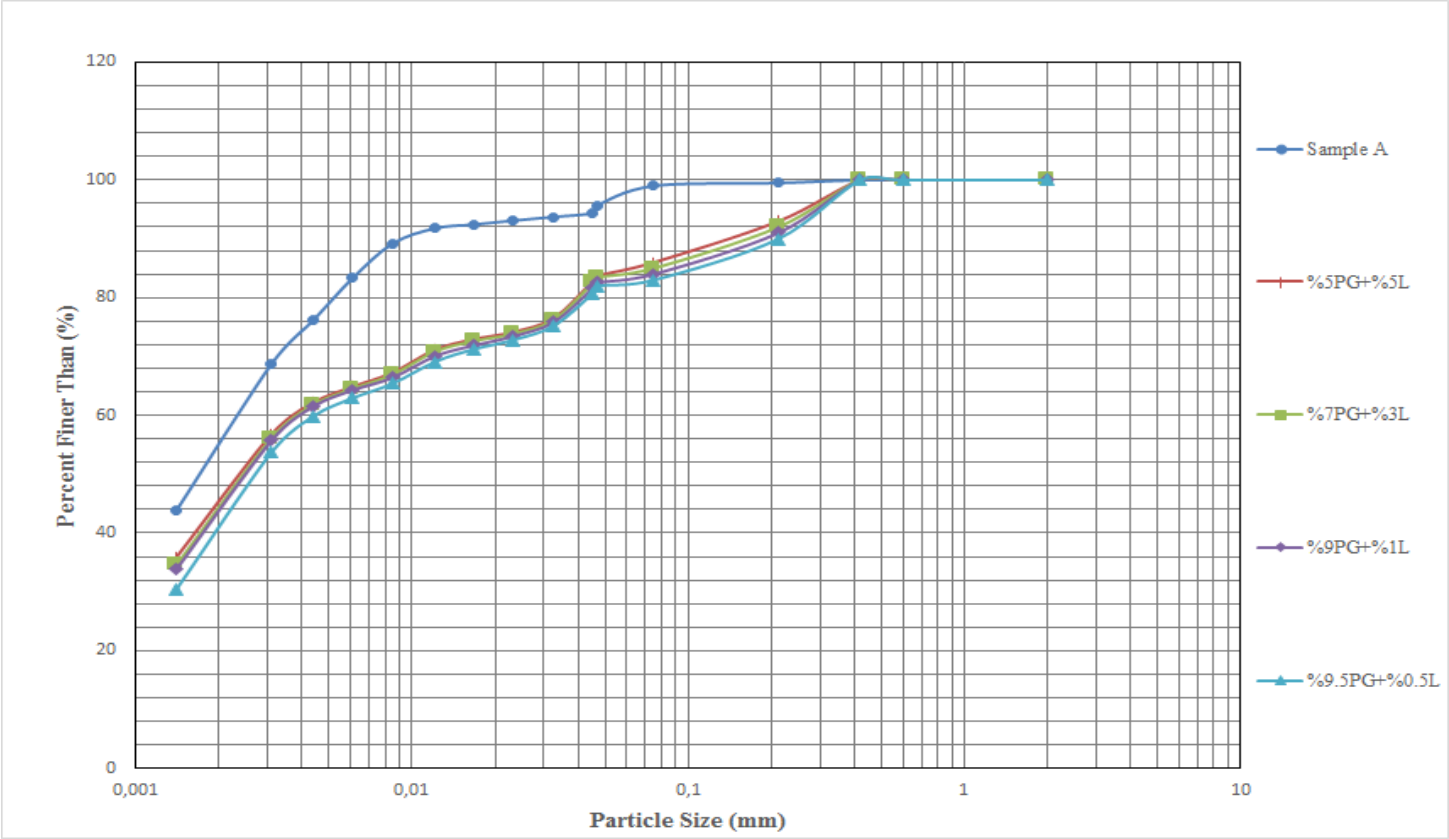


Figure 3.6. Grain Size Distribution Curves for Sample A and Lime and Phosphogypsum Treated Samples

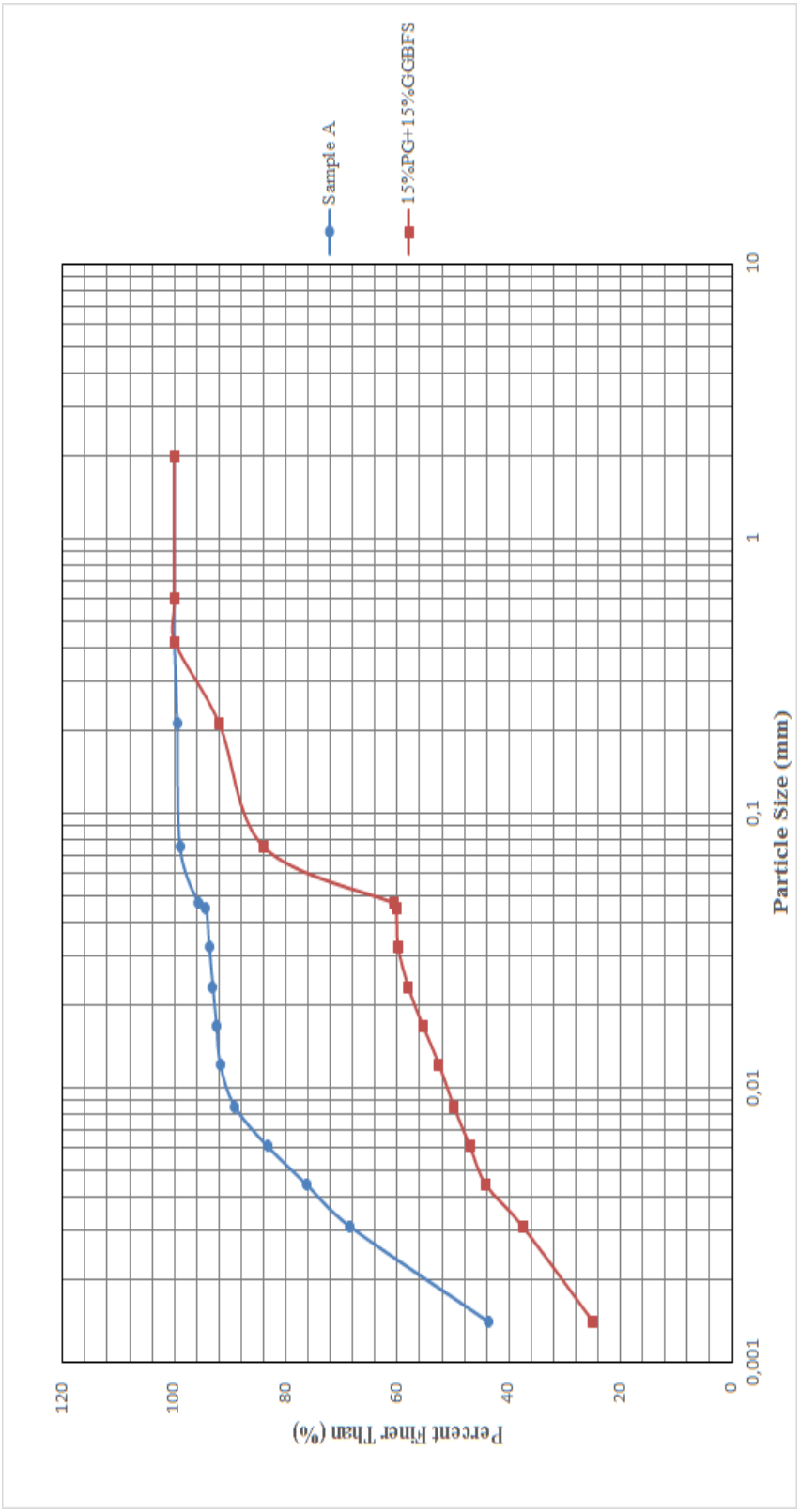


Figure 3.7. Grain Size Distribution Curves for Sample A and Phosphogypsum and Ground Granulated Blast Furnace Slag Treated Samples

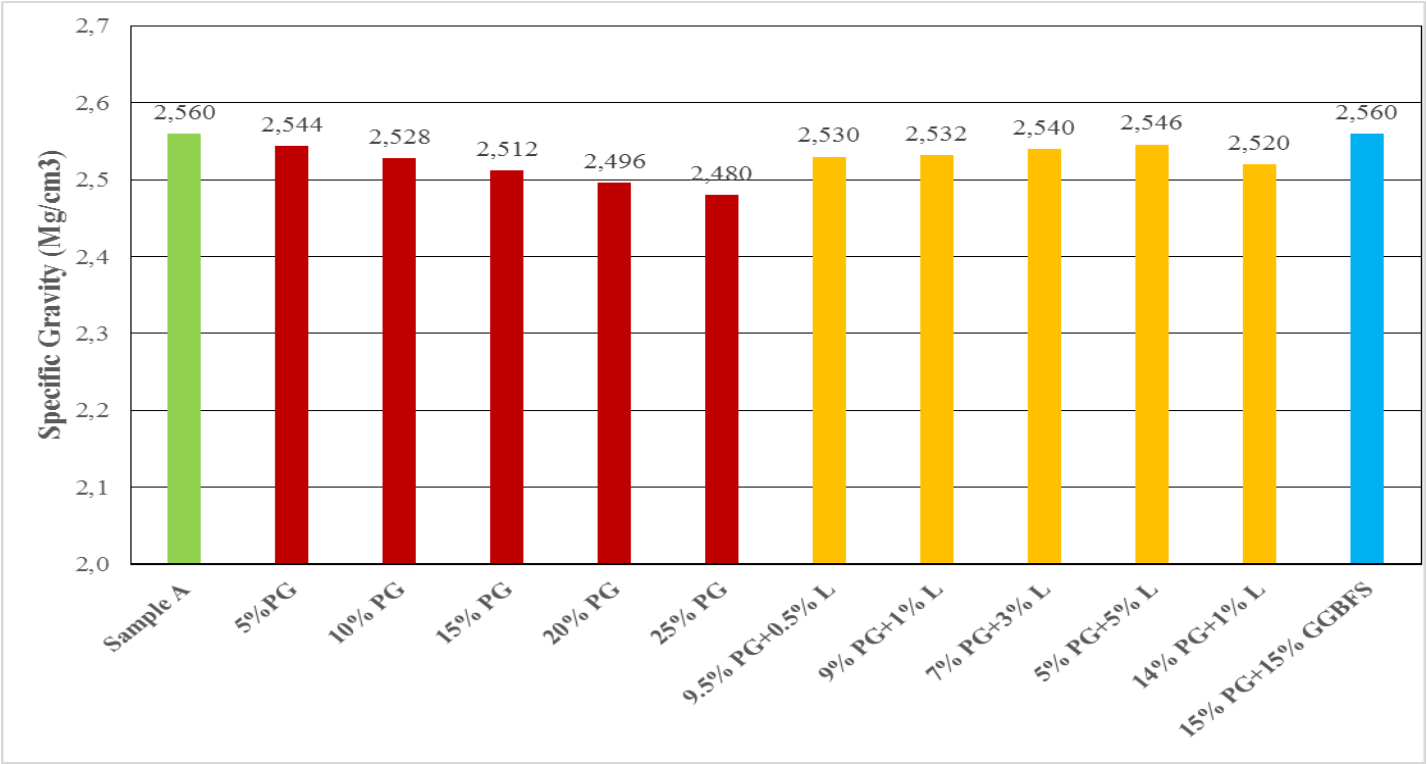


Figure 3.8. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Specific Gravity of the Samples

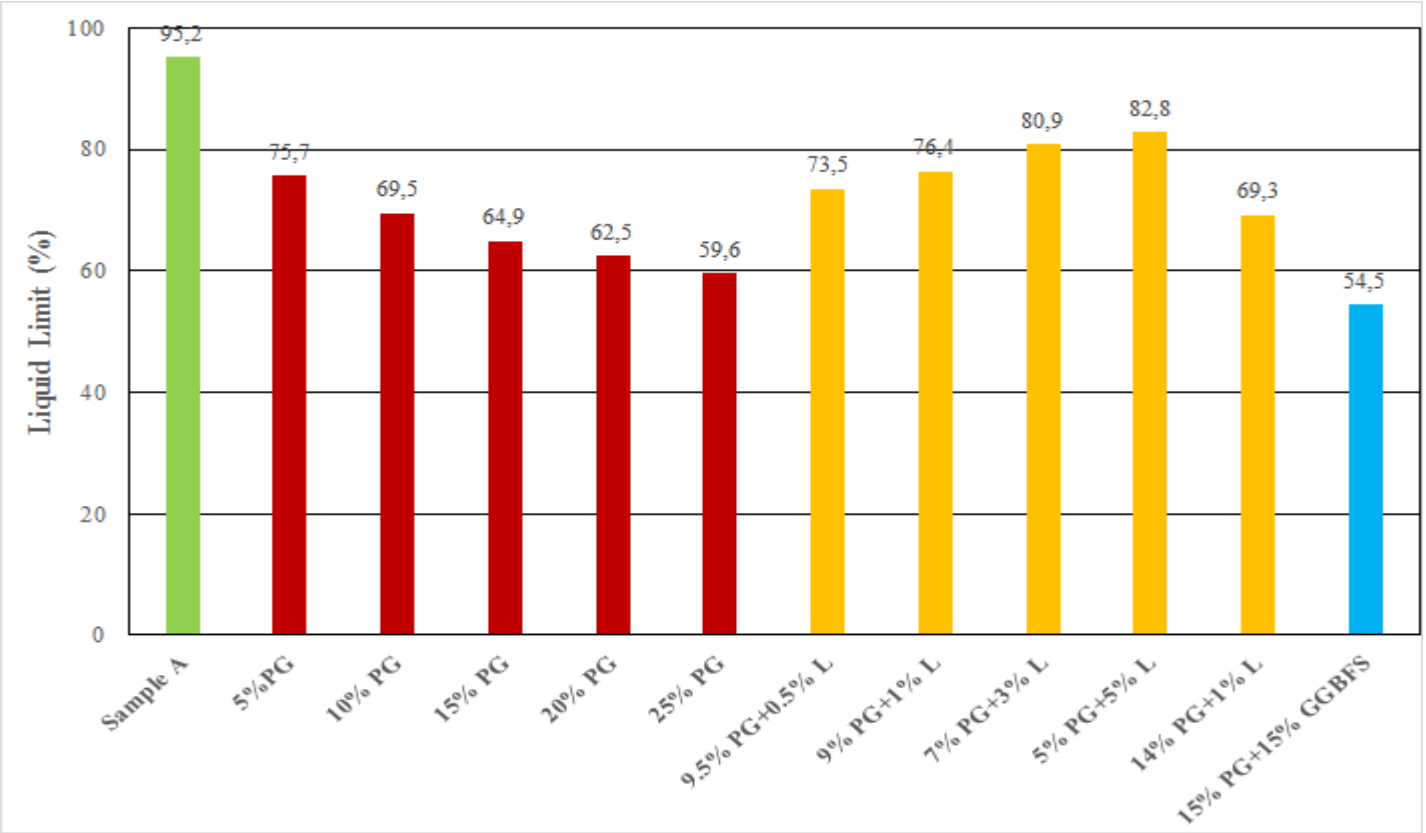


Figure 3.9. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Liquid Limit of the Samples

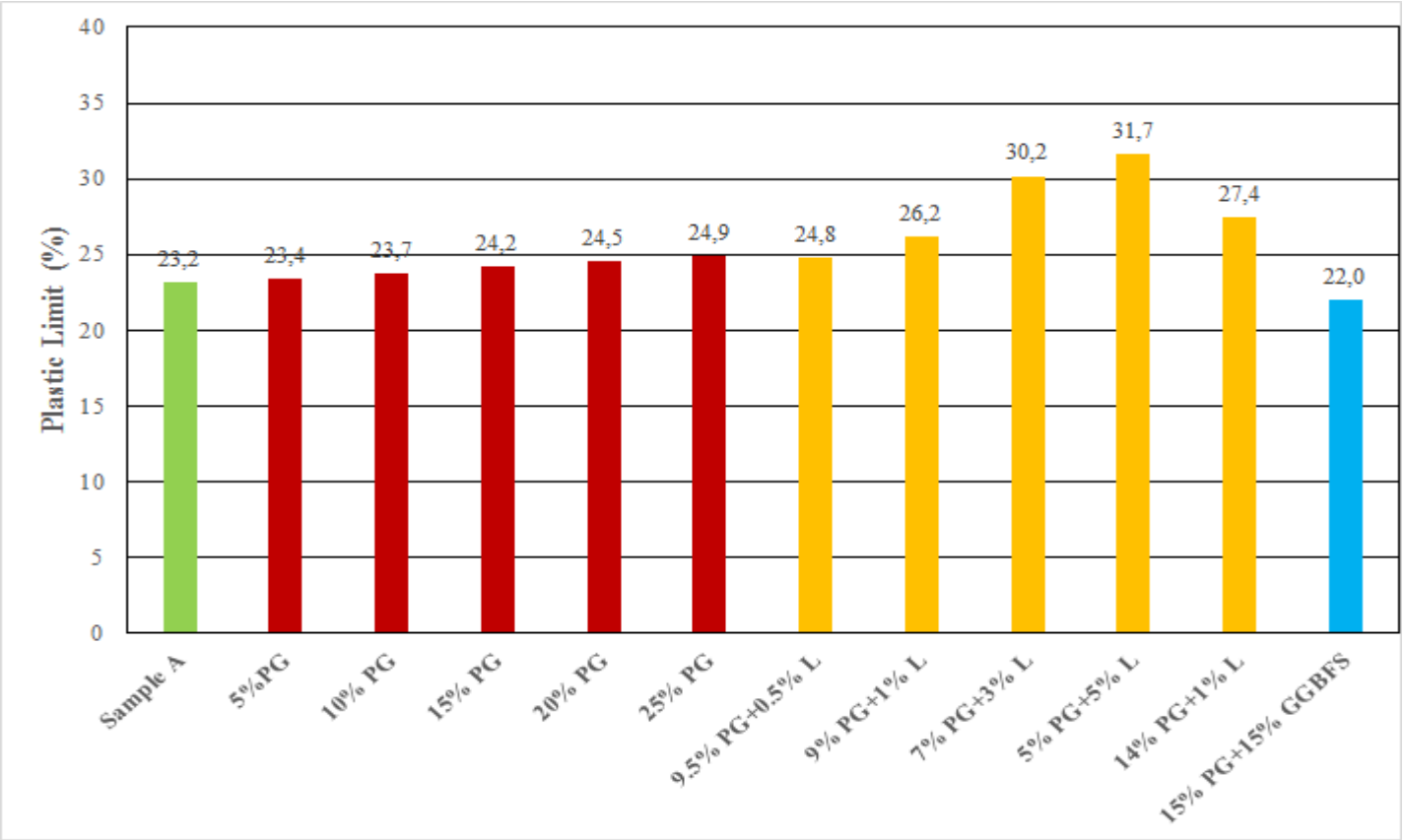


Figure 3.10. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Plastic Limit of the Samples

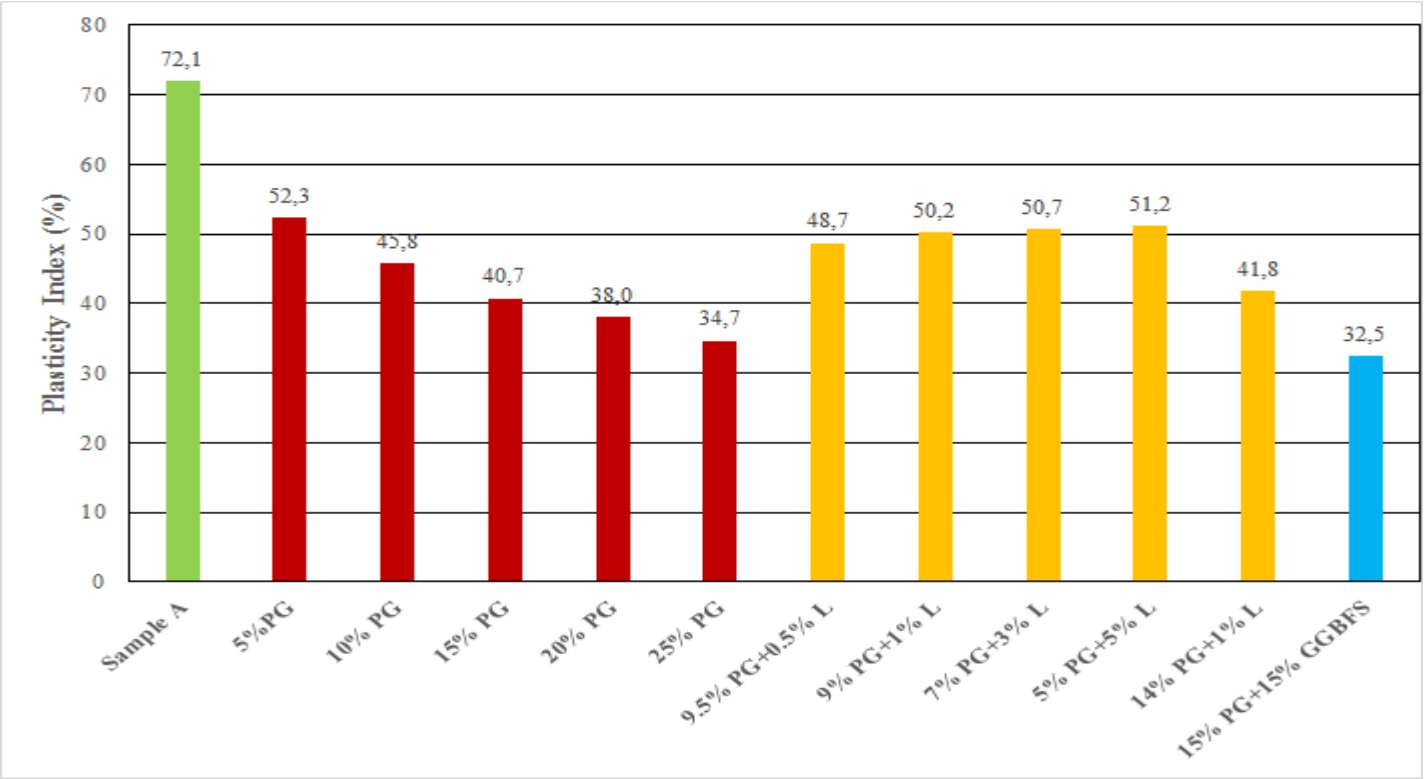


Figure 3.11. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Plasticity Index of the Samples

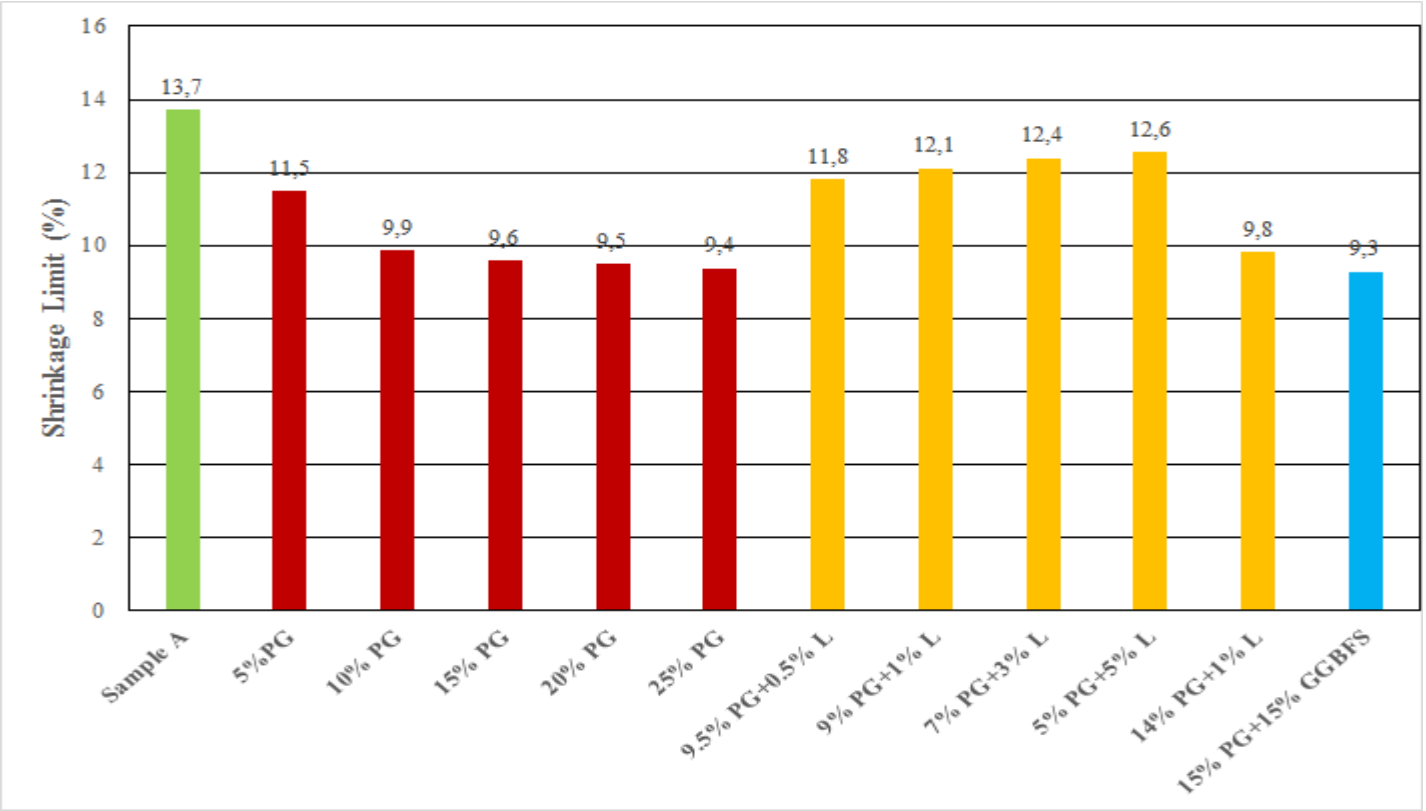


Figure 3.12. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Shrınkage Limit of the Samples

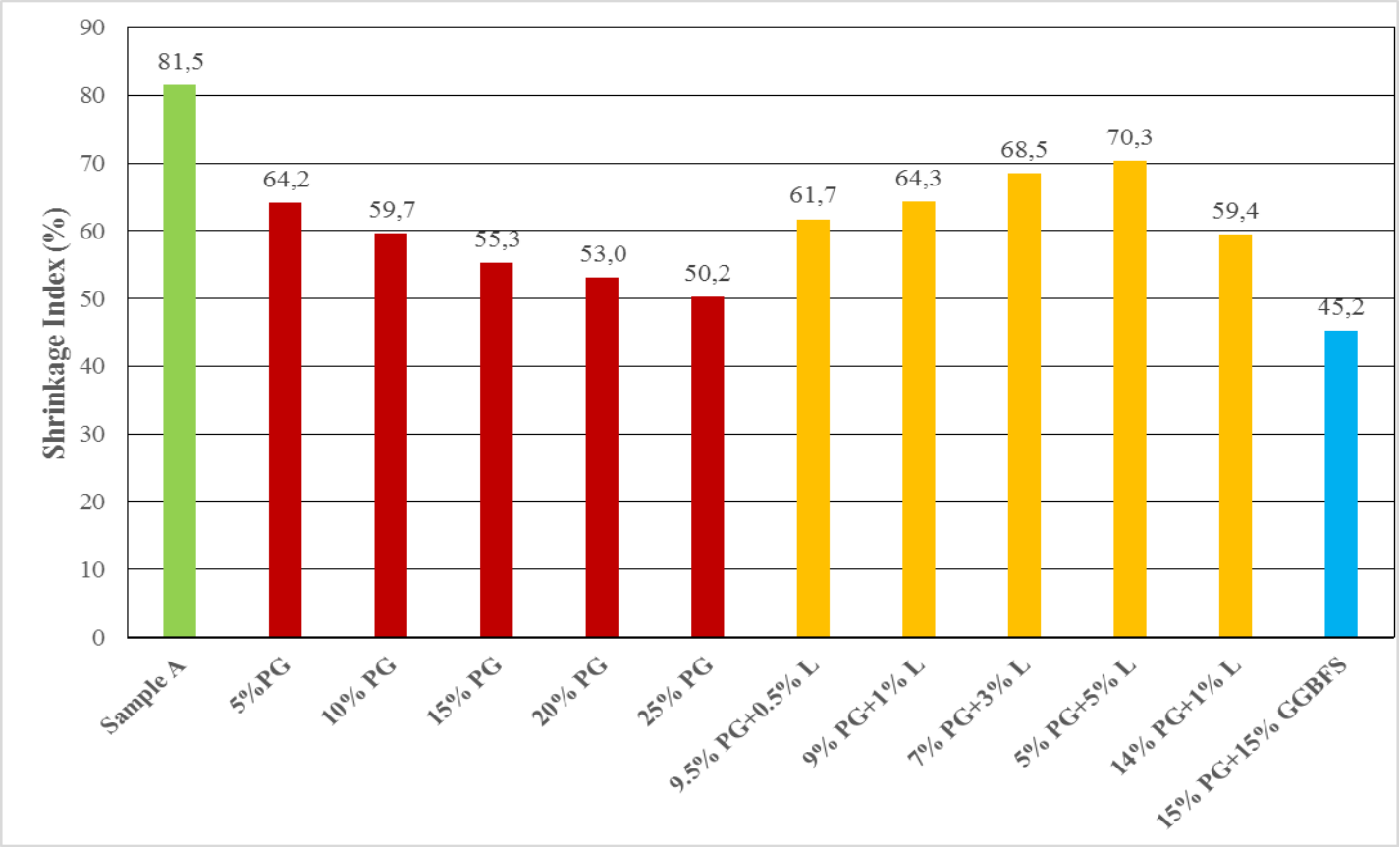


Figure 3.13. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Shrinkage Index of the Samples

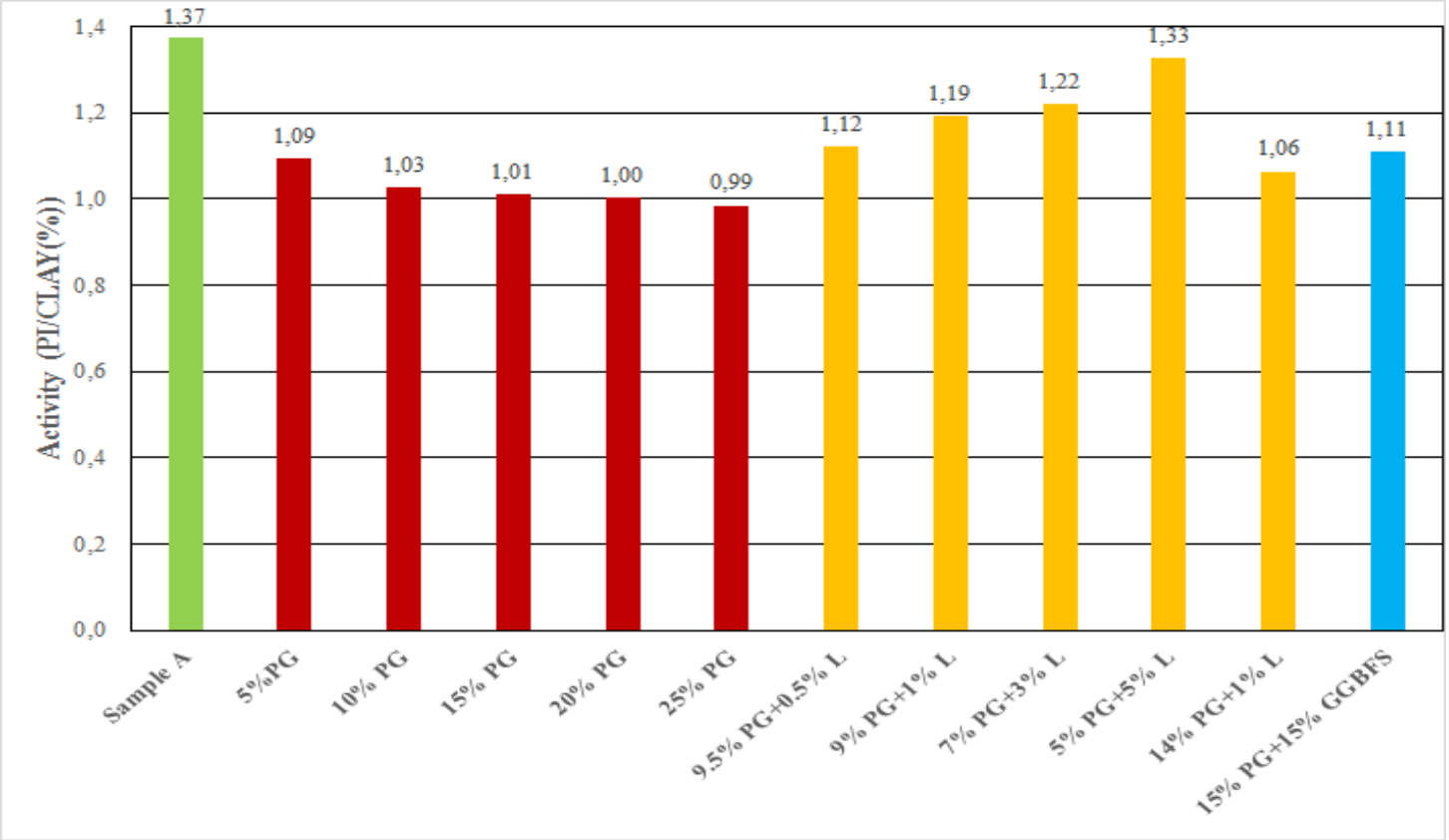


Figure 3.14. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Activity of the Samples

Table 3.3. Properties of Samples

Sample	Clay (%)	Gs (Mg/m³)	LL (%)	PL (%)	PI (%)	SL (%)	SI (%)	USCS	Activity	Swelling Potential (Seed et al.)
Sample A	52.5	2.560	95.2	23.2	72.1	13.7	81.5	CH	1.37	Very High
5%PG	47.8	2.544	75.7	23.4	52.3	11.5	64.2	CH	1.09	Very High
10% PG	44.6	2.528	69.5	23.7	45.8	9.9	59.7	CH	1.03	High
15% PG	40.5	2.512	64.9	24.2	40.7	9.6	55.3	CH	1.01	High
20% PG	37.9	2.496	62.5	24.5	38.0	9.5	53.0	CH	1.00	High
25% PG	35.2	2.480	59.6	24.9	34.7	9.4	50.2	CH	0.99	High
9.5% PG+%0.5% L	43.4	2.530	73.5	24.8	48.7	11.8	61.7	CH	1.13	High
9% PG+1% L	42.1	2.532	76.4	26.2	50.2	12.1	64.3	CH	1.19	High
7% PG+3% L	41.6	2.540	80.9	30.2	50.7	12.4	68.5	CH	1.22	High
5% PG+5% L	38.6	2.546	82.8	31.7	51.2	12.6	70.3	CH	1.33	High
14% PG+1% L	39.3	2.520	69.3	27.4	41.8	9.8	59.4	CH	1.06	High
15% PG+15% GGBFS	29.4	2.560	54.5	22.0	32.5	9.3	45.2	CH	1.11	Medium

3.5. Test Procedure for Free Swell Test Using Oedometer

3.5.1. Compaction of Specimen

Compaction of the each sample was done at specific dry density (1.495 gr/cm^3) and 11.5% water content. Samples were compacted in oedometer cutting ring which has 6.35 cm diameter and 1.9 cm height. Compaction method used in this study was static compaction. Hydraulic jack (Fig 3.15) was used to compact the samples.

Figure 3.15. A View from Static Compaction with the Hydraulic Jack

3.5.2. Free Swell Test

The swelling percent of these samples were obtained by free swell test (Fig 3.16, 3.17). Free swell tests in this study were done with respect to ASTM D-4546. This

test was performed using Bishop oedometer load frame. This test was performed according to these steps.

1. To decrease the friction between the cutting ring and sample, silicone grease was smeared on the inside of the ring.
2. Using a hydraulic jack, the samples were compacted statically. The sample was compacted in two layers to obtain uniform density.
3. The cutting ring was placed in the oedometer cell. Then, porous stone, filter paper were put on both top and bottom of the sample. The filter paper was used to protect the porous stones from soil contamination.
4. The cutting ring was fixed and cap placed over the porous stones.
5. The oedometer cell placed on the oedometer load frame. Some adjustments was made before the test; The load arm of the oedometer load frame was straigth and dial gage, which has 0.01 precision, was adjusted to zero.
6. The sample was inundated with distilled water. At the same time, the seating pressure was applied and chronometer was started.
7. The deformations at the specific time were recorded until free swell of sample was completed.
8. Swelling percent and time graph was plot with respect to Casagrande method.

According to test results, swelling percent was calculated for each sample. The following equation was used to calculate swelling percent;

Swell percent = $(\Delta H / H_i) * 100$

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

$\Delta H = (H_f - H_i)$

H_i = Initial height of the sample H_f = Final height of the sample

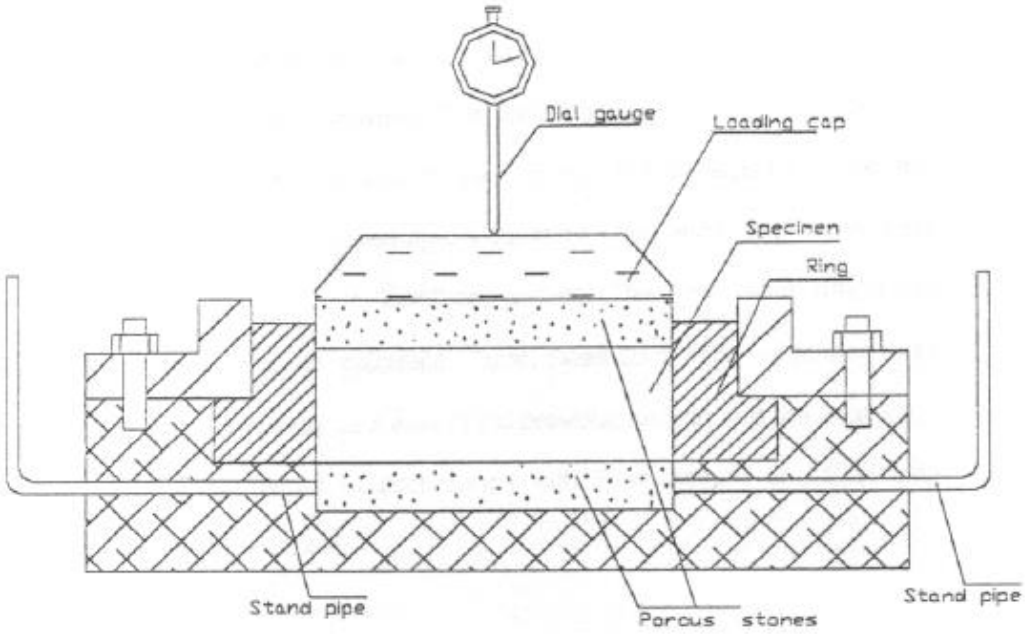


Figure 3.16. Free Swell Test Setup Drawing (İpek, 1998)

Figure 3.17. Bishop Apparatus

3.6. Test Results

Swelling percent of sample A, treated samples which treated with phosphogypsum, lime and ground granulated blast furnace slag were expressed in Figure 3.18.

Swelling percent of sample A, treated samples which treated with phosphogypsum, lime and ground granulated blast furnace slag with curing were shown in Figure 3.19.

Swelling potential of samples treated with phosphogypsum, lime and ground granulated blast furnace slag with respect to Sample A were shown in Figure 3.20.

Swelling potential of samples treated with phosphogypsum, lime and ground granulated blast furnace slag with curing with respect to Sample A were shown in Figure 3.21.

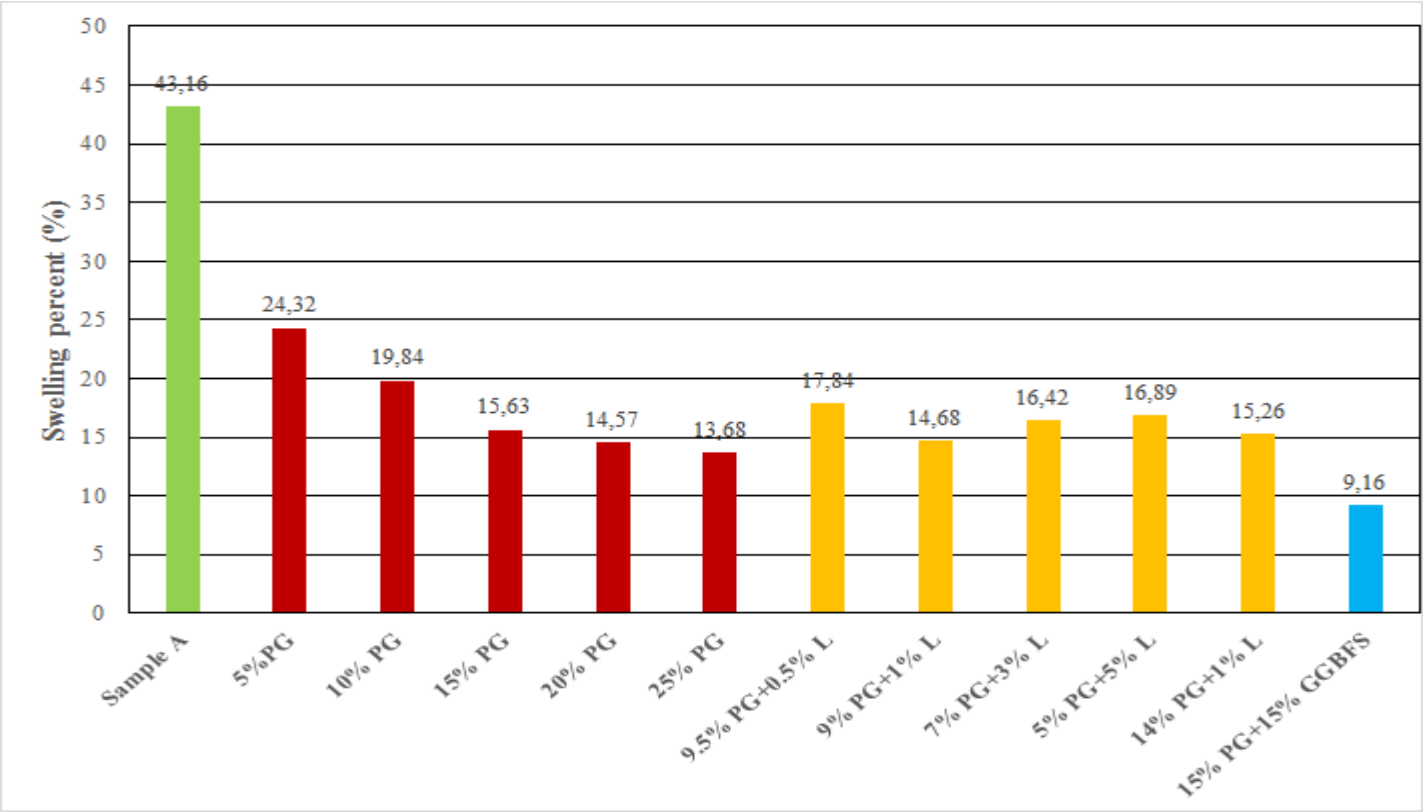


Figure 3.18. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Swelling Percent without Curing

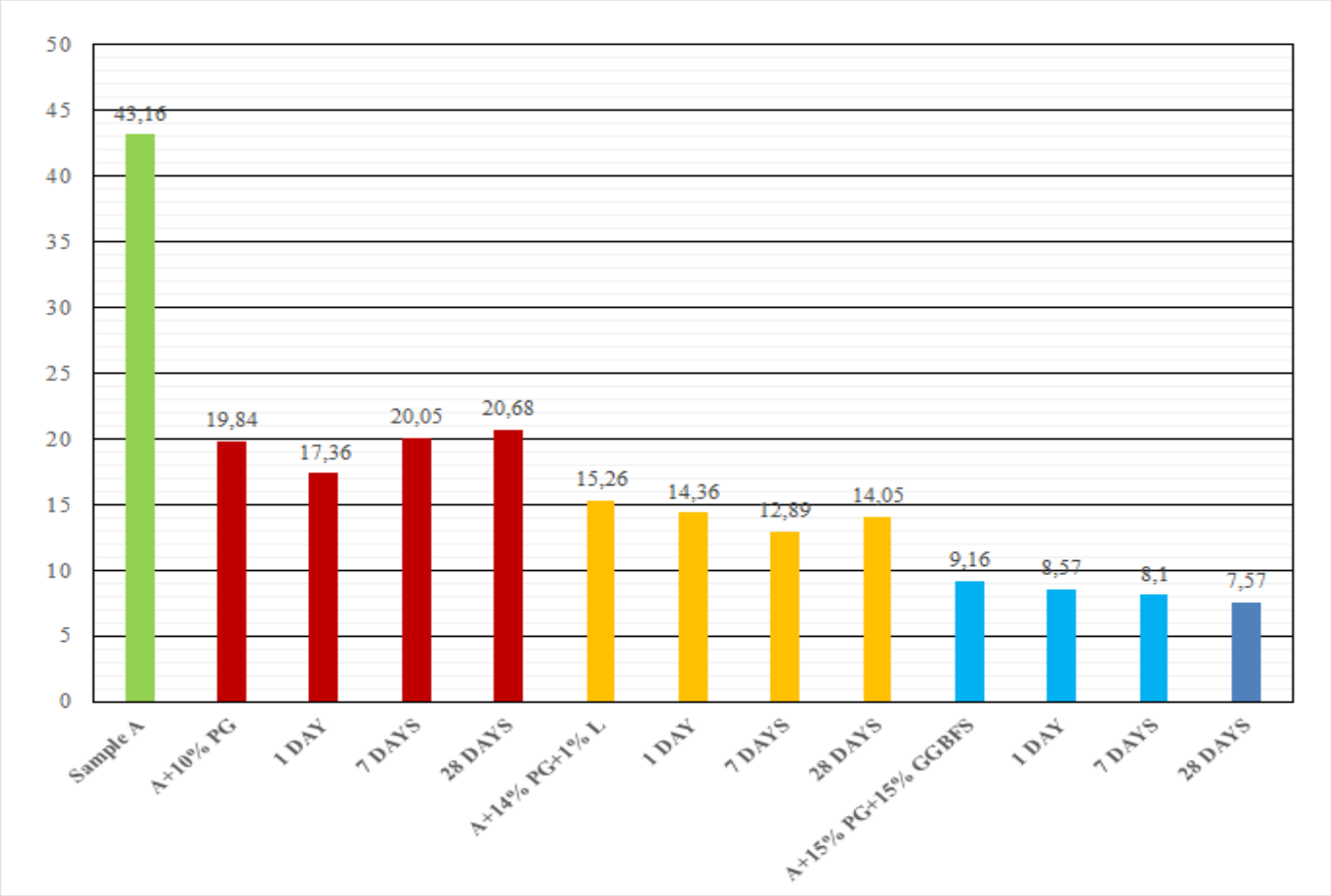


Figure 3.19. Effect of Addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag on Swelling Percent with Curing

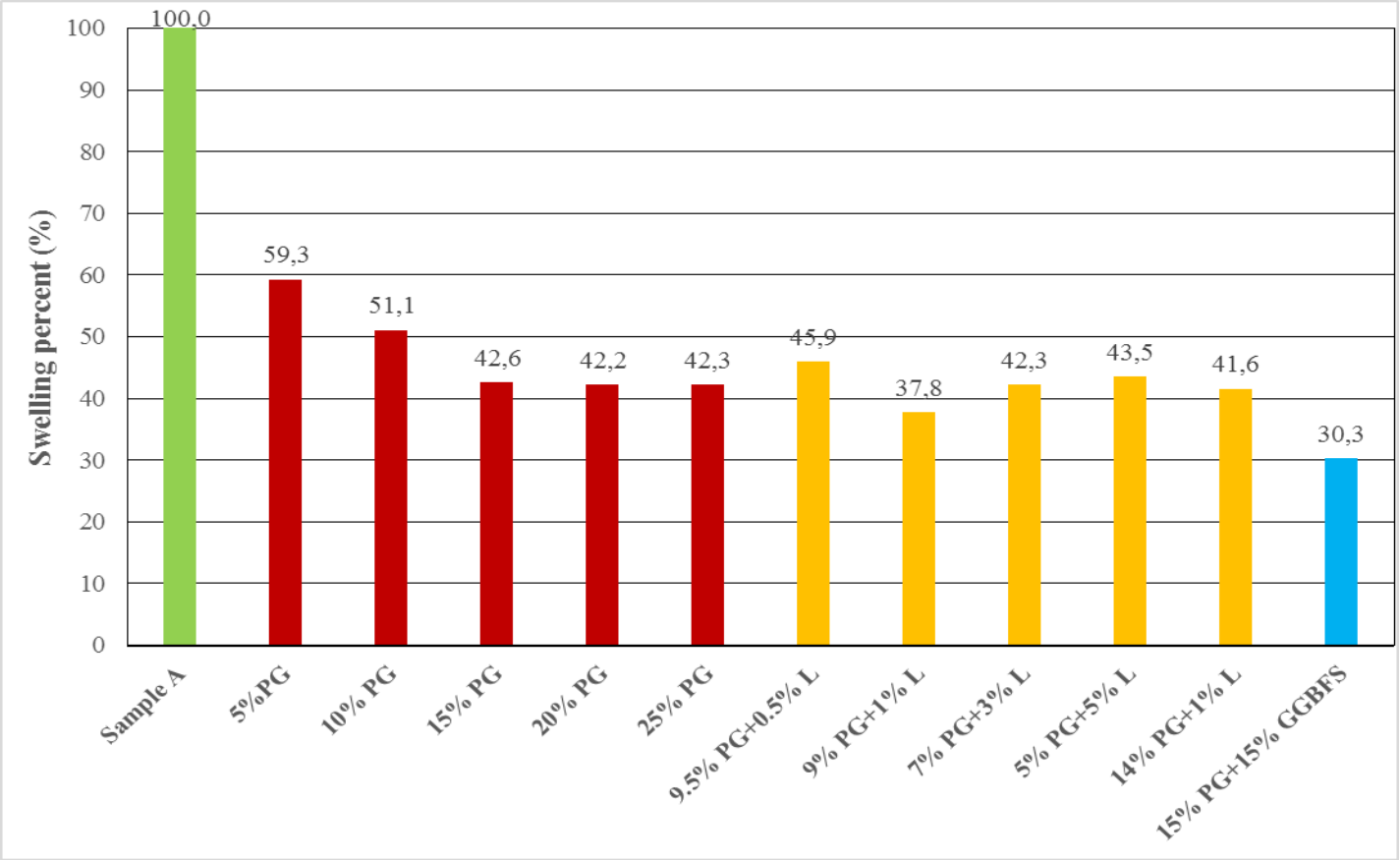


Figure 3.20. Change of Swelling Percent with addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag without curing

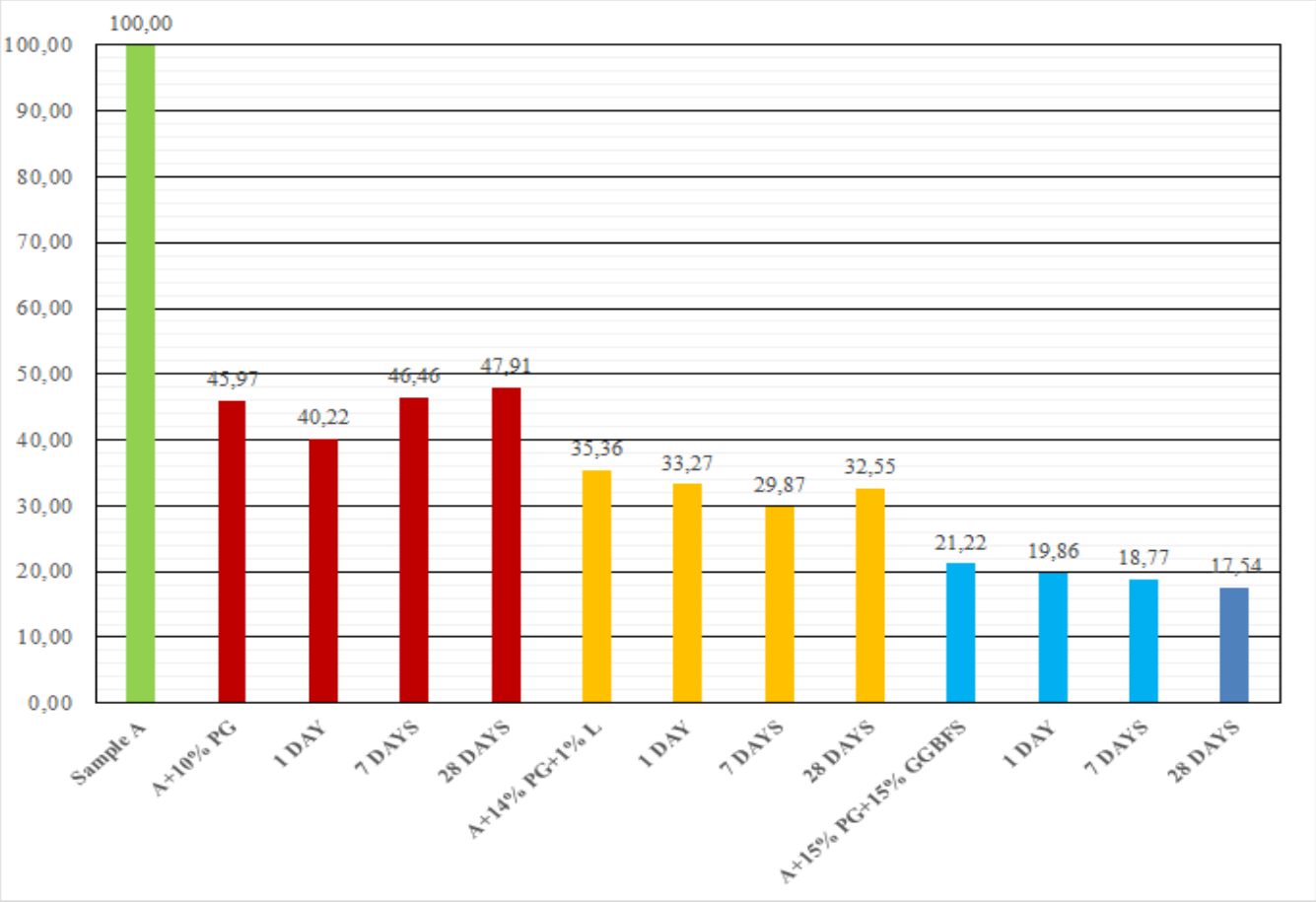


Figure 3.21. Change of Swelling percent with addition of Phosphogypsum, Lime and Ground Granulated Blast Furnace Slag with curing

3.7. SEM Analysis

The methods available for the estimation of the swelling–shrinkage potential are grouped into two categories: physico-chemical and mechanical methods. The common analytical techniques used in physico-chemical methods are the X-ray diffusion at small angles, X-ray diffraction, Scanning Electron Microscopy (SEM). On the other hand, the classic experimental techniques used in the mechanical methods are the consolidometer test and the soil suction test (Waddah et al., 1999).

Scanning Electron Microscope (SEM) is one of the most versatile equipment to examine and analyze the microstructure of soil and chemical composition. In other words, the detail information about the microstructure of soils and change in microstructure for chemically treated soils can be obtained by using scanning electron microscope.

SEM analysis was done at METU Central Laboratory. QUANTA 400F Field Emission Scanning Microscope was used during the SEM analysis in this study. Firstly, all samples used in this analysis were dried at 45°C and pulverized. Because water vapor in the sample may harm the scanning electron microscope. After the samples were cupped, they were coated with gold and palladium for SEM Analysis.

Samples used for SEM Analysis are given in Table 3.4.

Table 3.4. Samples used for SEM Analysis

Sample	Conditions
Sample A	Without curing
A+14% PG+1% L	Without curing
	7 days curing
	28 days curing
A+15% PG+15% GGBFS	Without curing
	7 days curing
	28 days curing

The figures from SEM Analysis are given in Figures 3.22-3.28.

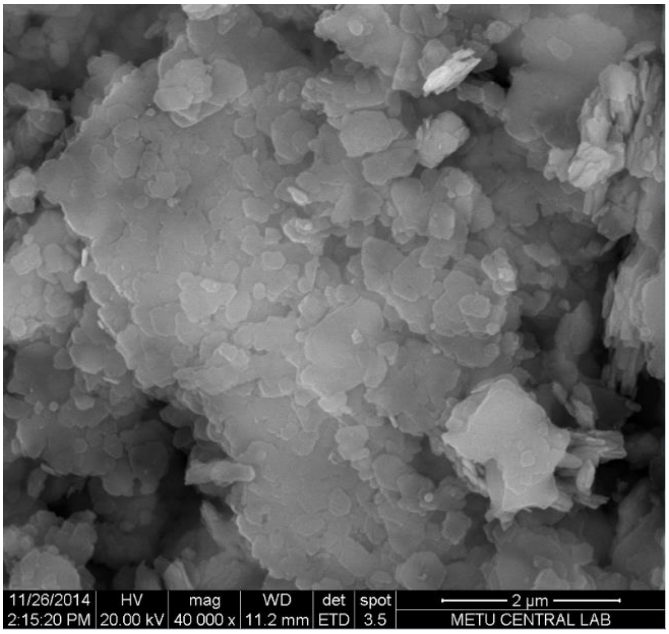


Figure 3.22. SEM image of Sample A

(magnification factor=40000)

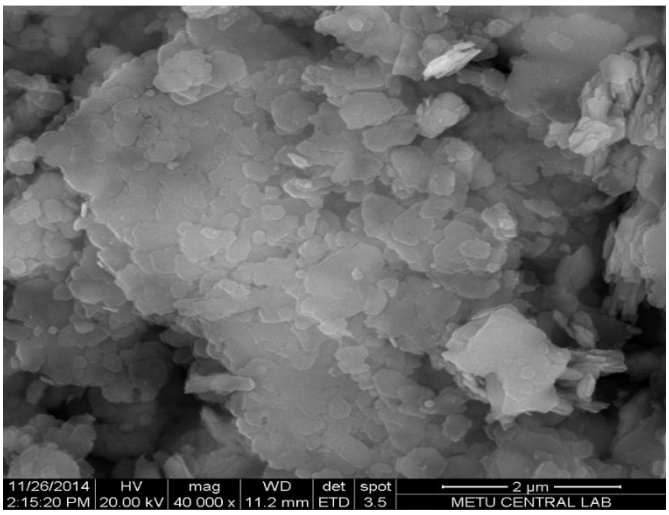


Figure 3.23. SEM image of Sample A+14%PG+1%L

(magnification factor=40000) without curing

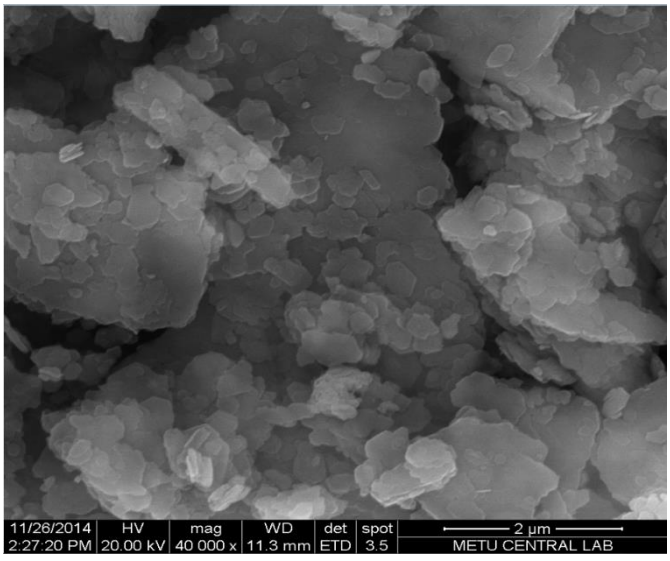


Figure 3.24. SEM image of Sample A+14%PG+1%L

(magnification factor=40000) 7 days curing

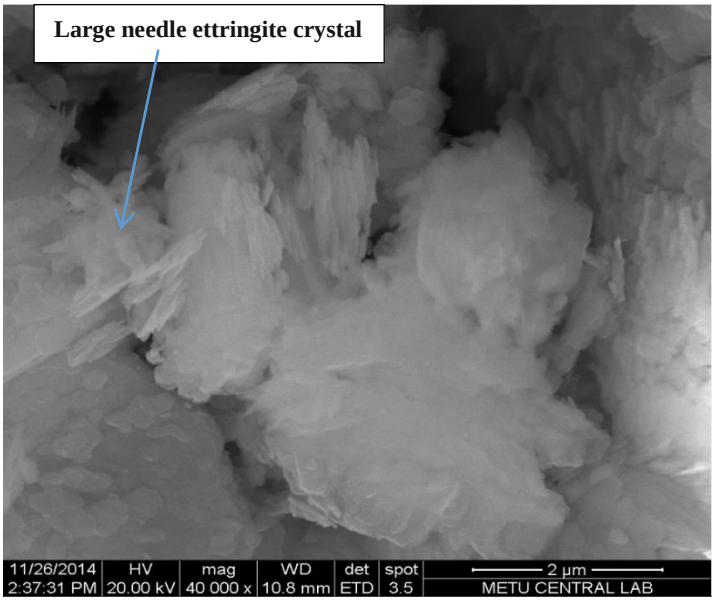


Figure 3.25. SEM image of Sample A+14%PG+1%L

(magnification factor=40000) 28 days curing

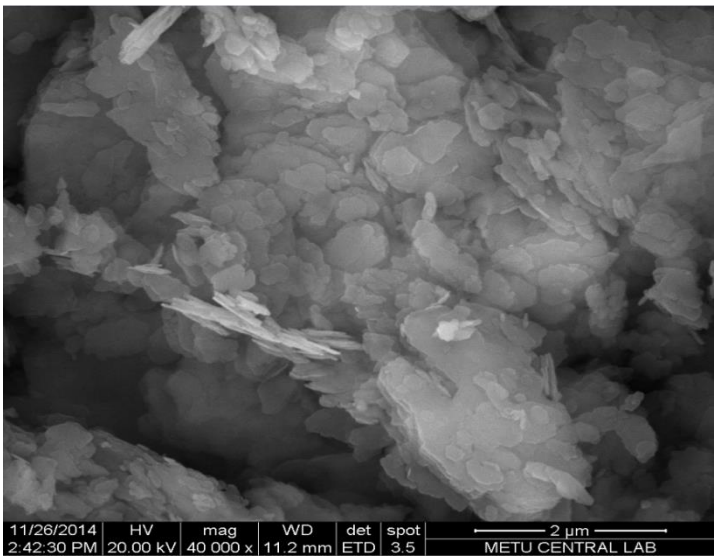


Figure 3.26. SEM image of Sample A+15%PG+15%GGBFS

(magnification factor=40000) without curing

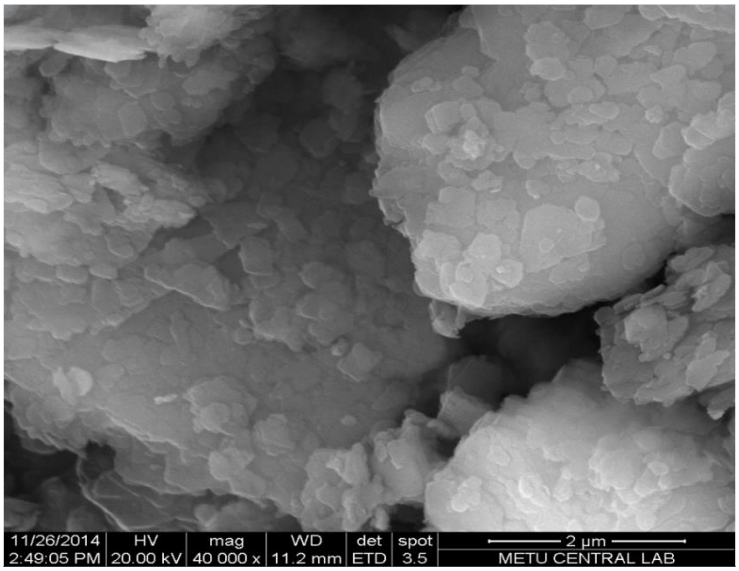


Figure 3.27. SEM image of Sample A+15%PG+15%GGBFS

(magnification factor=40000) 7 days curing

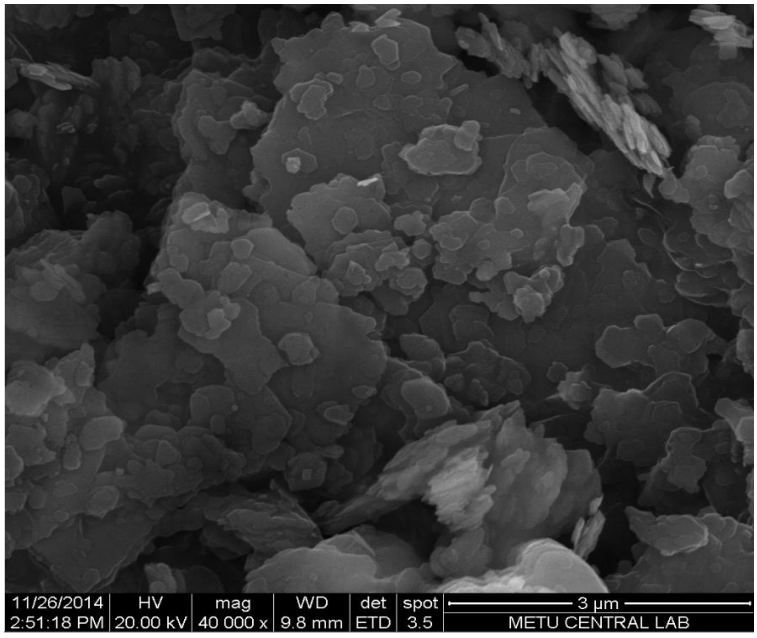
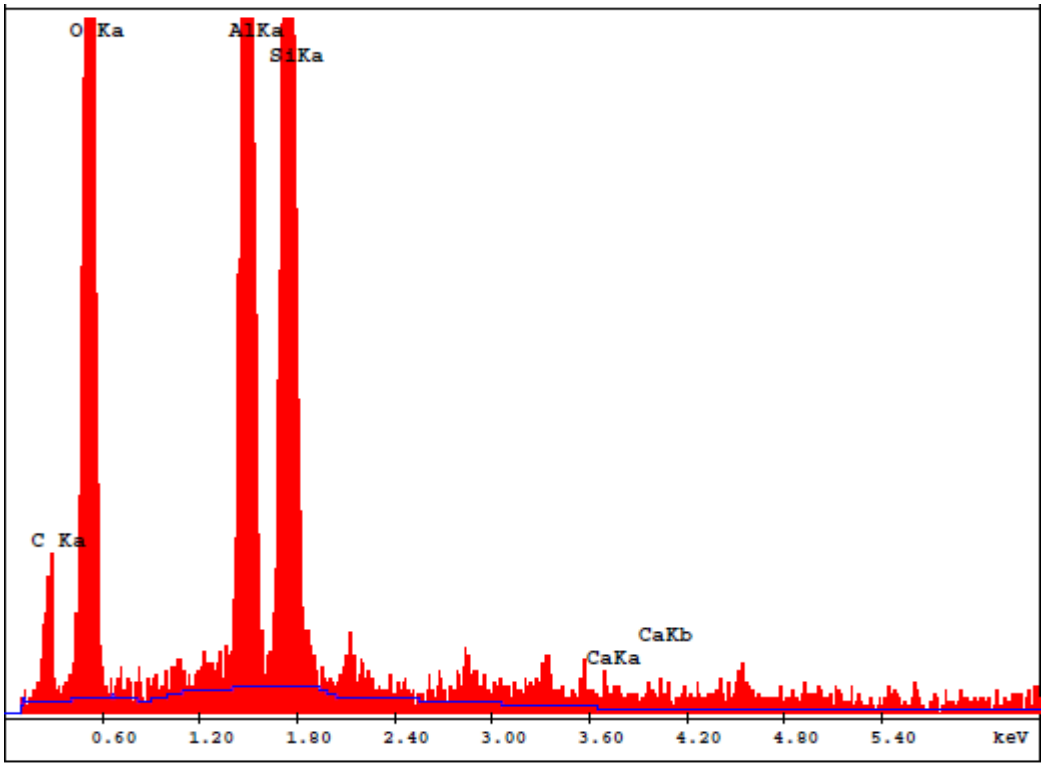


Figure 3.28. SEM image of Sample A+15%PG+15%GGBFS

(magnification factor=40000) 28 days curing

While SEM analysis, Energy Dispersive X-Ray (EDX) analysis was performed to take information about chemical characterization of these samples. EDX diagrams which were obtained from Sample A, Sample A+14%PG+1% L, Sample A+15%PG+15% GGBFS were given in Figures 3.29, 3.30 and 3.31 respectively.

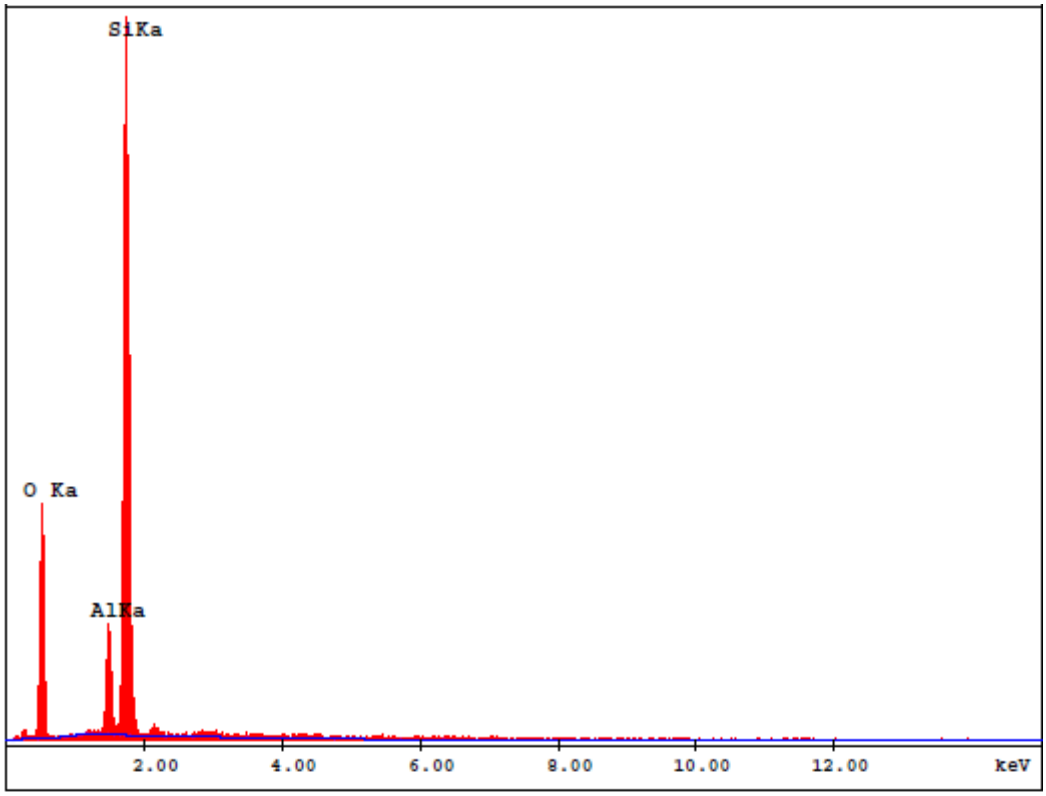


EDAX ZAF Quantification (Standardless)
Element Normalized
SEC Table : Default

Element	Wt %	At %	K-Ratio	Z	A	F
C K	15.97	23.53	0.0270	1.0334	0.1634	1.0005
O K	48.68	53.86	0.1543	1.0161	0.3119	1.0004
AlK	15.57	10.21	0.1084	0.9465	0.7307	1.0072
SiK	19.39	12.22	0.1284	0.9741	0.6796	1.0000
CaK	0.39	0.17	0.0035	0.9445	0.9493	1.0000
Total	100.00	100.00				

Element	Net Inte.	Bkgd Inte.	Inte. Error	P/B
C K	8.72	1.01	8.92	8.61
O K	130.62	1.35	2.10	96.79
AlK	130.39	3.15	2.13	41.41
SiK	143.89	3.54	2.02	40.62
CaK	2.42	0.73	19.32	3.31

Figure 3.29. EDX Diagram of Sample A

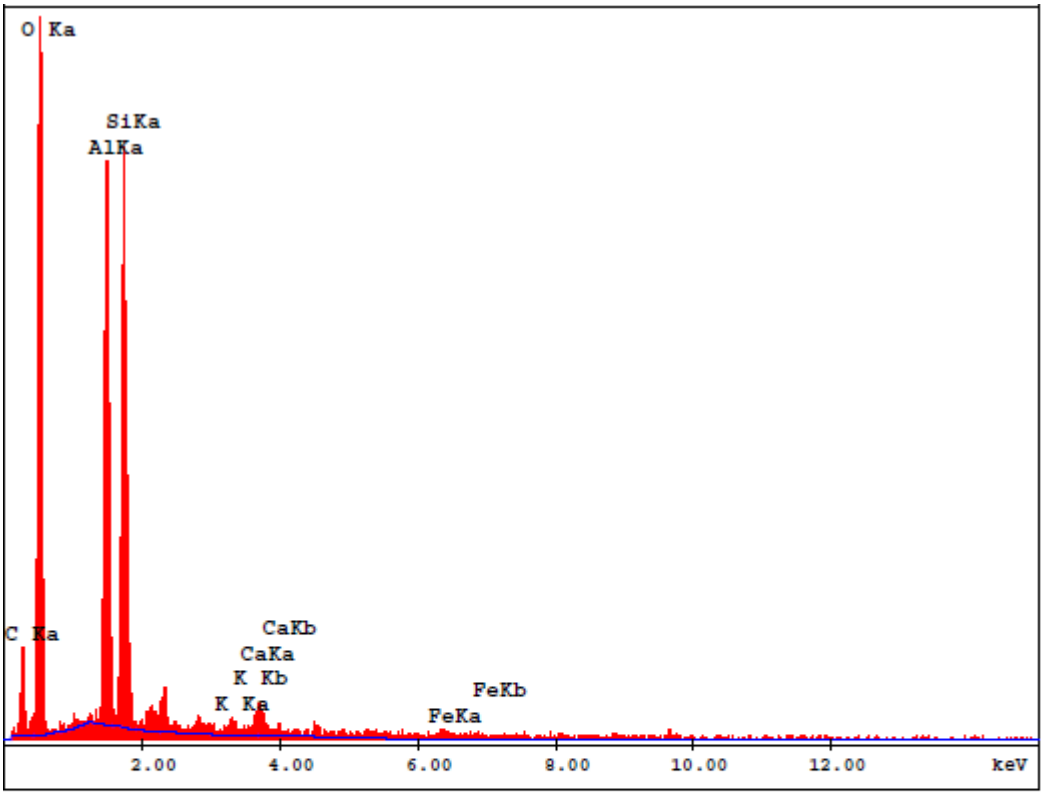


EDAX ZAF Quantification (Standardless)
Element Normalized
SEC Table : Default

Element	Wt %	At %	K-Ratio	Z	A	F
O K	39.58	53.37	0.1239	1.0277	0.3043	1.0006
AlK	7.29	5.83	0.0552	0.9570	0.7725	1.0244
SiK	53.13	40.80	0.4124	0.9849	0.7882	1.0000
Total	100.00	100.00				

Element	Net Inte.	Bkgd Inte.	Inte. Error	P/B
O K	82.43	0.94	1.80	88.11
AlK	52.23	3.82	2.39	13.67
SiK	363.41	3.59	0.85	101.33

Figure 3.30. EDX Diagram of Sample A+14%PG+1% L without curing



EDAX ZAF Quantification (Standardless)
Element Normalized
SEC Table : Default

Element	Wt %	At %	K-Ratio	Z	A	F
C K	19.81	28.57	0.0372	1.0319	0.1820	1.0005
O K	47.79	51.72	0.1399	1.0147	0.2884	1.0003
AlK	13.22	8.49	0.0902	0.9451	0.7173	1.0063
SiK	16.41	10.12	0.1106	0.9728	0.6928	1.0003
K K	0.62	0.27	0.0053	0.9211	0.9321	1.0028
CaK	1.37	0.59	0.0124	0.9431	0.9569	1.0005
FeK	0.77	0.24	0.0067	0.8560	1.0081	1.0000
Total	100.00	100.00				

Figure 3.31. EDX Diagram of Sample A+15%PG+15% GGBFS without curing

Plate like microstructure (Figures 3.22-3.28) of Sample A showed that main clay mineral in sample A is kaolinite. This is an expected result since Sample A contains 85% kaolinite and 15% bentonite. Also from Figure 3.23, A+14% PG+1% L sample with 28 days curing ettringite and CSH crystals were observed in the sample, in that sample ettringite formation during the soil-phosphogypsum reactions with the presence of sulfate, caused an increase in the swell potential of the sample

(Fig.3.18). The reaction sequence of ettringite formation is presented below (Ismaiel, 2006).

$$6Ca + 2Al(OH)^{-}_{4} + (OH)^{-} + 3(SO_4)^{2-} + 26H_2O \longrightarrow \text{Ettringite}$$

$$(Ca_6[(Al(OH)_6)]_2.(SO_4)_3.26H_2O)$$

But with the addition of GGBFS to the sample, A+15% PG+15% GGBFS, ettringate formation disappeared in the SEM (Figures 3.26-3.28) and a decrease in the swell potential of the sample was observed (Fig.3.19).

CHAPTER 4

DISCUSSION ON TEST RESULTS

4.1. Effect of Additives on Specific Gravity

Specific gravity of kaolinite, bentonite, phosphogypsum, lime and ground granulated blast furnace slag was obtained as 2.57, 2.53, 2.33, 2.41 and 2.71 respectively. The specific gravity of sample A was found as 2.56. The specific gravity of phosphogypsum and lime is lower than the specific gravity of sample A. For this reason, mixture which prepared by adding phosphogypsum and lime had lower specific gravity than sample A. In addition to this, ground granulated blast furnace slag has higher specific gravity than the other materials. So the mixture, which is prepared by adding ground granulated blast furnace slag, has the same specific gravity with sample A.

4.2. Effect of Additives on Liquid Limit

Liquid limit of sample A were found as 95.2%. When the ratio of phosphogypsum, lime, ground granulated blast furnace slag in mixture increased, liquid limit of mixture decreased. The lowest liquid limit (54%) is obtained from A+15%PG+15%GGBFS. The mixtures which consist of Sample A and phosphogypsum have lower liquid limits than the mixtures which consist of Sample A, phosphogypsum and lime.

The addition of the 15% PG decreased the liquid limit value from 95.2% to 64.9%. While the liquid limit value of sample including 14% PG+1% L was found as 69.3%, the liquid limit value for sample including 15% PG and 15% GGBFS was determined as 54.5% (this was the lowest liquid limit value obtained in this study). As the waste materials such as phosphogypsum and ground granulated blast furnace slag are non plastic materials, liquid limit values of the mixtures are reduced by adding these materials.

4.3. Effect of Additives on Plastic Limit

The values of plastic limit for these samples were between 22% and 31.7%. The lowest plastic limit (22%) was obtained from sample which consists of phosphogypsum and ground granulated blast furnace slag. However, the sample which has phosphogypsum (5%) and lime (5%), had the highest plastic limit (31.7%).

When the amount of phosphogypsum in mixture increases, the plastic limit value gradually rises. In this study, plastic limits of samples including only Sample A and phosphogypsum range from 23.4% to 24.9%.

When lime was added to phosphogypsum and Sample A, plastic limit raised more. While plastic limit for Sample A +10% PG was 23.7%, plastic limit for sample A+5% PG+5% L was 31.7%. Therefore, lime is more effective on plastic limit than phosphogypsum.

Ground granulated blast furnace slag dropped the plastic limit of Sample A. The lowest plastic limit was taken from Sample A+ 15% PG+ 15% GGBFS. Unlike phosphogypsum and lime, ground granulated blast furnace slag declined the plastic limit.

4.4. Effect of Additives on Plasticity Index

Plasticity Index values of Sample A was determined as 72.1%. This value decreased to 52.3% when 5% phosphogypsum was added to sample A. Though the percentage of phosphogypsum in mixture decreases this value, the optimum percentage of mixture was found as 15% for phosphogypsum. The plastic limit determined as 40.7% for this optimum mixture.

The plastic limit for samples, including lime and phosphogypsum, was greater than the sample including only phosphogypsum for same percentage. While the plastic limit of sample A+10% PG was 45.8%, the plastic limit of sample A+5% PG+5% L was determined as 51.2%.

Ground granulated blast furnace slag was more effective than the other materials. For instance, the value of plastic limit for sample A+15% PG+15% GGBFS was assessed as 32.5%.

4.5. Effect of Additives on Shrinkage Limit

Shrinkage limit value slightly decreased at these conditions applied to phosphogypsum. The shrinkage limit of sample A was 13.7% while this limit was determined 9.4% for sample treated with 25% phosphogypsum. The conditions applied to both phosphogypsum and lime increased with respect to applying to only phosphogypsum. This limit of sample including 10 % phosphogypsum was 9.9% while this limit was found as 12.6 % for sample A + 5 % PG + 5 % L.

The lowest shrinkage limit (9.3%) was obtained from sample A+15% PG+15% GGBFS. When ground granulated blast furnace slag compared with phosphogypsum, both of them had the same effect on shrinkage limit.

4.6. Effect of Additives on Shrinkage Index

Shrinkage index values of Sample A was decreased by adding phosphogypsum. The shrinkage index of sample treated 5 % phosphogypsum was sharply declined from 81.5%, which was the shrinkage index of Sample A, to 64.2%. However, the shrinkage index of samples, which treated 15%, 20% and 25% phosphogypsum, were 55.3%, 53.0% and 50.2% respectively.

When the samples were added both phosphogypsum and lime, shrinkage index values were bigger than the samples treated at the same amount only by phosphogypsum.

The sample treated phosphogypsum and ground granulated blast furnace slag had the lowest shrinkage index value, and this sample had the lowest liquid limit value in this study.

4.7. Effect of Additives on Grain Size Distribution

Grain size distribution curves for these samples were determined by doing both hydrometer test and sieve analysis test. When the amount of phosphogypsum increased for samples treated only with phosphogypsum, rapid settlements were observed after first 1 hour. For samples involving phosphogypsum and lime, grain size distribution curves were like sample A although these samples were coarser than sample A. Since the specific gravity of the sample with ground granulated blast furnace slag is high, rapid settlement of the sample was occured in hydrometer test. This problem was solved by the increasing the mixture duration and performing the mixing process more carefully.

The curves obtained from hydrometer tests were coarser than sample A as the additive materials had coarser particles such as silt than sample A.

4.8. Effect of Additives on Activity

Activity declined with the addition of phosphogypsum. Sample A treated by both phosphogypsum& lime and phosphogypsum&ground granulated blast furnace slag had higher activity than the sample A treated only by phosphogypsum.

4.9. Effect of Additives on Swelling Potential

The additive materials, which are phosphogypsum, lime and ground granulated blast furnace slag, in this study brought down the swell potential of sample A.

Swelling potential of sample A was gradually released by adding phosphogypsum. The samples added 5%, 10% and 15% phosphogypsum had less swelling potential due to the reduction of sample A and effect of phosphogypsum. However, the samples added 20% and 25% phosphogypsum had very little swelling potential than sample included 15% phosphogypsum. For this reason, the addition of more than 15% phsophogypsum did not affect the swelling potential. Therefore, the optimum phosphogypsum content was found as 15%.

Samples including only phosphogypsum were improved with lime. While swelling potential of sample with 10% phosphogypsum was 19.84%, addition of 5%, 3%, 1%, 0.5% lime decreased this value (19.84%) to 16.89%, 16.42%, 14.68% and 17.84% respectively. The more lime added to sample with phophogypsum, the less swelling potential was obtained. However, the 0.5% lime added to this sample was not enough for improvement of this sample. Therefore, the ideal mixture percentage for lime was found as 1%.

The best percentages of both phosphogypsum and lime for samples including these additives were determined as 14% and 1% respectively. This sample reduced both the swelling potential of sample A from 43.15% to 15.26%. The swelling potential of sample with 15% PG 15.63%.

For phosphogypsum and ground granulated blast furnace slag treated sample, had the lowest swelling potential, which was 9.15%.

4.10. Effect of Cure on Swelling Potential

In this study, samples were waited in the curing room, which was 70% moisture and 22-25 °C temperature for 1, 7 and 28 days.

The sample including only phosphogypsum for one day curing, had less swelling potential than the sample without curing. The swelling potentials of sample treated with 10 % phosphogypsum without curing and curing for 1 day, 7 days, 28 days were found 19.84 %, 17.36 %, 19.84 %, 20.68 %, respectively. However, when these samples were waited in the curing room for 7 days and more, the swelling potentials of these samples were higher than samples without curing.

When the samples improved with phosphogypsum and lime were waited in the curing room for 7 days, the swelling potential decreased. However, if the curing time is more than 7 days, the swelling potential increased.

The swelling potentials of the samples treated with phosphogypsum and ground granulated blast furnace slag reduced gradually with curing period. The swelling potential of sample including both phosphogypsum and ground granulated blast furnace slag without curing was 9.16 % while the swelling potential for same sample for 28 days curing was determined as 7.57 %.

4.11. Effect of Additives and Curing on Rate of Swell

The term t_{50} is used for the time necessary for 50% swell of total swell. The t_{50} value of sample A was found as 7980 seconds (Figure 4.1), this value reduced with additive materials.

The t_{50} values declined gradually from 5% PG to 15% PG. This value sligthly increased from 15% PG to 25% PG. The t_{50} for samples treated phosphogypsum and lime was smaller than the t_{50} for samples treated only phosphogypsum. For example, while t_{50} value for 15% PG was 1301 seconds, t_{50} value for 14% PG+ 1% L was 900 seconds. For sample including 15% PG and 15% GGBFS, t_{50} value was determined as 1215 seconds.

The effect of curing period on the rate of swell was investigated. When the curing period increased, the t_{50} values for sample treated phosphogypsum+lime and phosphogypsum+ ground granulated blast furnace slag increased slightly. However, while this value for sample treated 10% PG was found as 1980 seconds for 7 days curing, this value reduced to 1800 seconds for 28 days of curing (Figure 4.2).

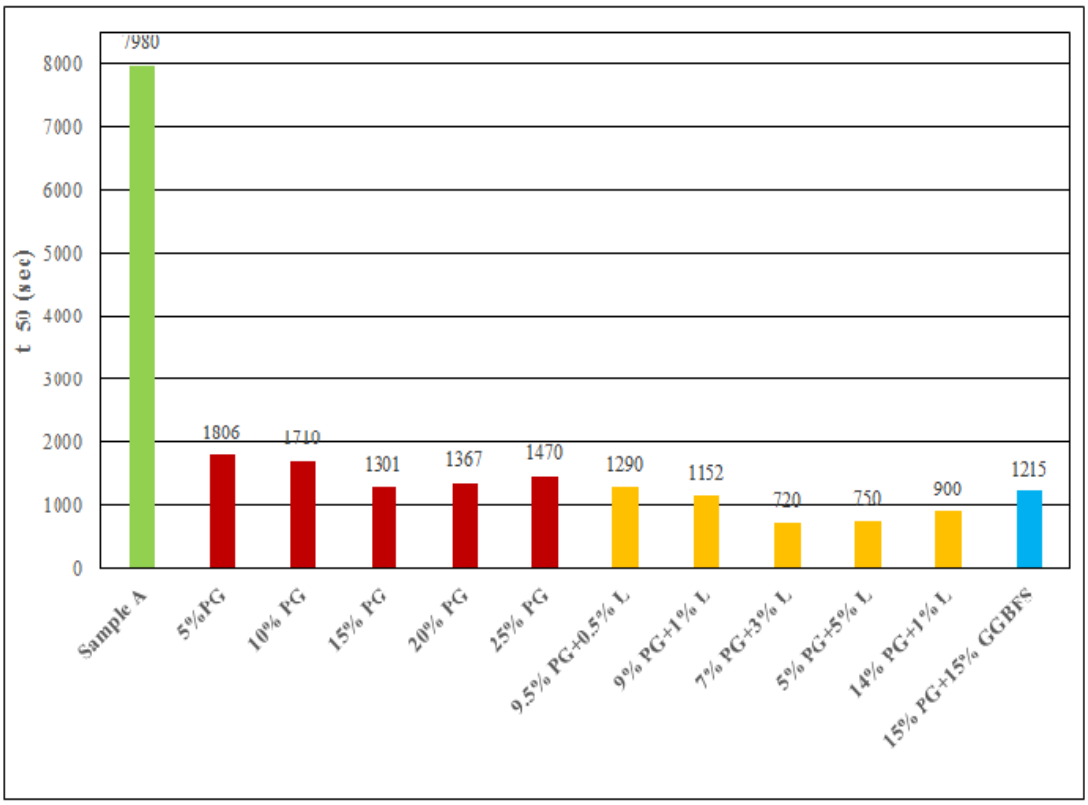


Figure 4.1. Effect of additive materials on t_{50} of the Samples

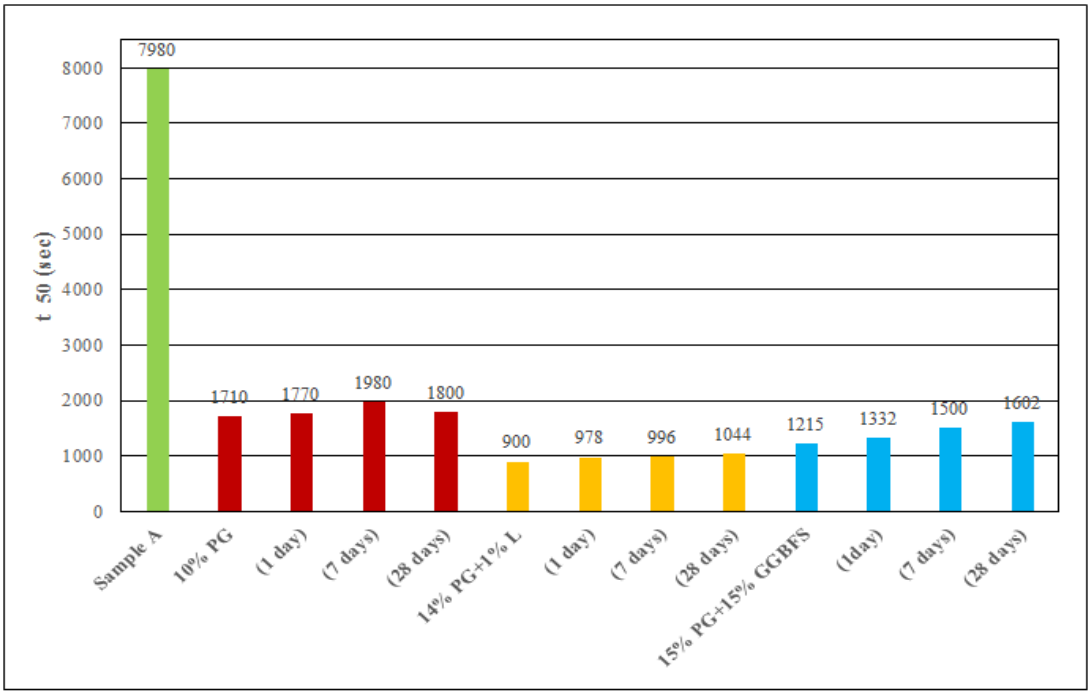


Figure 4.2. Effect of Curing period on t_{50} of the Samples

4.12. Discussions on SEM-EDX Analysis

There are three different types of samples in this analysis. They are Sample A, Sample A+14% PG+1% L and Sample A+15% PG+15 %GGBFS.

The SEM images obtained from Sample A showed that microstructure of this sample seemed as plate due to this sample formed 85% kaolinite and 15% bentonite. The image taken from Sample A+14% PG+1% L without curing looked like the images of Sample A.

As sulphates can react with lime, they cause serious problems such as swelling, heave and damages. Ettringite formation occurs because of excessive sulphate in the mixture. Ettringite, its shape is like a needle, leads to overly swelling. The formation of ettringite was observed at the images of Sample A+14% PG+1% L with curing of 28 days.

The images taken from Sample A+15% PG+15% GGBFS showed that this sample had microstructure which was similar to Sample A. Higgins et al. (1998) stated that ground granulated blast furnace slag was fairly prosperous to decrease swelling caused by sulphate. In addition to this, it also reduced the swelling potential of the samples. Veith (2000) stated that when the sulphate content is less than 1%, sulphate will not have any effect on swelling. For this reason, the mixture of lime and ground gronulated blast furnace slag may be resistant to swelling resulted from sulphate.

Similar findings were reported by Ismaiel (2006). Ismael (2006) stated that when soil is stabilized with calcium-based additive such as lime, portland cement, fly ash etc. ettrengite or thaumasite forms as a result of the reaction between sulphate in the soil and calcium and alumina from clay (in the presence of water). The formation of ettrengite or thaumasite cause considerable pavement heaving and loss of strength after the compaction process.

Sulphate leads to the alteration of soil-lime, soil-fly ash and soil-lime/fly ash reaction. The reactions between calcium in the lime or in the fly ash (in the absence of sulphate) and soil (in the presence of water) produce calcium silicate hydrate of varying calcium to silica ratios. In case of the presence of sulphate, these reactions are modified and ettringite or thaumasite are formed. Hunter (1988) described the sequence of the reaction as follows:

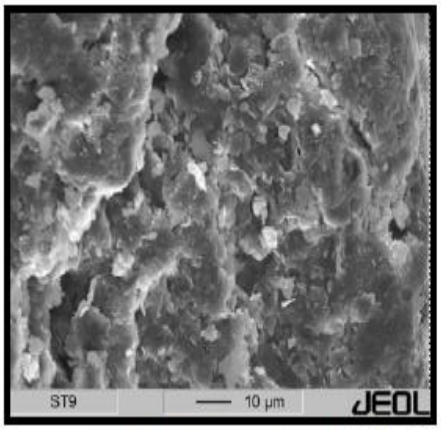
$$6Ca+2AL(OH)^{-}_{4} + (OH)^{-} +3(SO_4)^{2-} +26H_2O \longrightarrow Ca_6\,[(\,AL(OH)_6\,)]_2\,.\,(SO_4)_3\,.\,26H_2O$$

“ettringite”

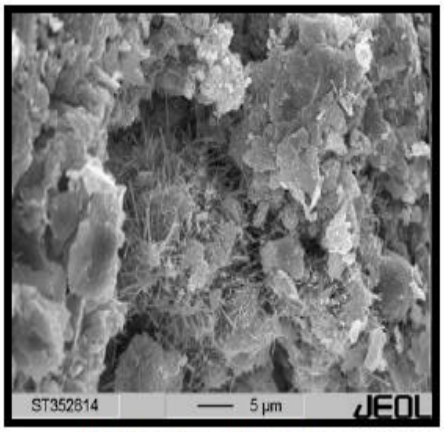
Figure 4.3 (b) shows a micrograph of weathered soil stabilized with 35% fly ash and cured for 28 days. The micrograph illustrates rod-like crystals “ettringite” (as hydration reaction product) growing on the relics of clay particles. The microstructure is highly porous where pore spaces are relatively large. This confirms the high value of the hydraulic conductivity (K-value = 1.0E-07 m/sec) at 28 days curing compared to K-value of the same mixture at 180 days curing. Figure 4.3 (c) shows the microstructural development of weathered soil stabilized with 35% fly ash and cured for 180 days.

The micrograph shows cementitious compounds (as pozzolanic reaction products) and massive formation of ettringite (rod-like crystals) joining together and filling the pore spaces. Fly ash and soil particles join together and strength rises as a result of this condition. Subsequently, this contributed to a reduction of the K-value (about 2.0E-08 m/sec) at the long-term curing. The micrograph of weathered soil stabilized with 3% lime and 35% fly ash together and cured for 28 days (Figure 4.3 (d)) illustrates the growth of ettringite crystals at the surface of clay-relics and fly ash particles nucleated with ettringite crystals. The micrograph also shows cementitous compounds coating the flocs. The microstructure seems highly porous where the pore spaces are relatively large compared to the pore spaces of the same mixtures after 180 days curing. This confirms an increase in the hydraulic conductivity at the maximum value (K-value = 4.1E-08 m/sec) after 28 days curing. The microstructural development of weathered soil stabilized with 3% lime and 35% fly ash together after 180 days curing (Figure 4.3 (e)) illustrates flocculated arrangement (due to the flocculation) with pore spaces relatively smaller than the pore spaces of the same mixture at 28 days curing. The reduction of pore spaces diameter resulted from the formation of a new cementitous compounds (new formation of mineral crystals as a result of pozzolanic reaction at long-term curing) surrounded and within the pore spaces (voids) between fly ash- and soil-particles. This led to an increase in the strength gain and a reduction of the hydraulic conductivity (K-value=2.6E-09 m/sec).

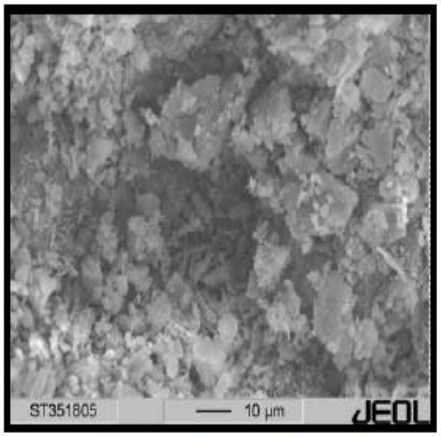
The microstructural development of weathered soil stabilized with 3% lime and 35% fly ash together after 180 days curing (Figure 4.3 (f)) shows massive formation of long rods of ettringite crystals after 180 days curing. The amount and the length of ettringite crystals are large in comparison to the fly ash-weathered soil mixture after 180 days curing. This shows that the reaction of fly ash with the soil is weaker than the reaction of lime and fly ash together and the addition of lime to sulfate-bearing soil leads to an increase in the ettringite-formation.



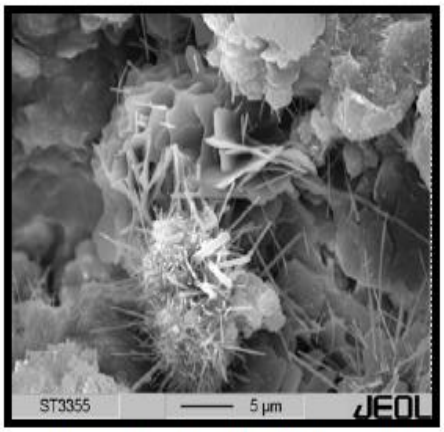
(a) Natural weathered soil



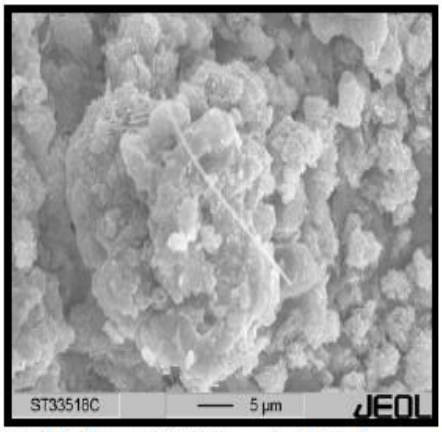
(b) 35% fly ash, 28 days curing



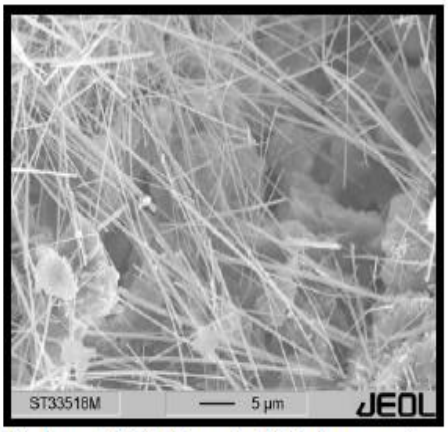
(c) 35% fly ash, 180 days curing



(d) 3% lime+35% fly ash, 28 days curing



(e) 3% lime+35% fly ash, 180 days curing



(f) 3% lime+35% fly ash, 180 days curing

Figure 4.3. Sem images of soils (Ismaiel, 2006)

According to Ismaiel study, SEM analysis done for the three tested stabilized soils shows that both fly ash and lime/fly ash stabilized weathered soil (after 28 and 180 days curing) include needle or rod like crystals. These crystals are like ettringite crystals. The occurrence of ettringire crystals were seen at SEM analysis.

CHAPTER 5

CONCLUSION

The effect of phosphogypsum, lime and ground granulated blast furnace slag addition on swelling potential of an expansive soil is investigated in this study. PG addition to reduce swelling potential was not studied much in the literature. So the results could be very valuable. In addition to this, the impact of these additive materials on specific gravity, Atterberg limits, grain size distribution and free swelling percents were also sought. With respect to results obtained from this study, the following conclusions can be accessed;

1. Specific gravity of sample A declined when the amount of phosphogypsum increased. However, the specific gravities of the samples including phosphogypsum and lime are greater than the sample stabilized only by phosphogypsum. The samples contaning ground granulated blast furnace slag has the highest specific gravity in this study.
2. Grain size distribution curves are changed by the addition of stabilizer materials and these materials increased the amount of coarse grained particles in the mixtures. This condition results from silt size particles and chemical reactions.
3. While liquid limits values of samples treated phosphogypsum and lime decreased, plastic limits of these samples increased gradually for samples with phosphogypsum and lime. On the other hand, these limits decreased for samples including ground granulated blast furnace slag. 15% pg and 15% ggbfs addition to sample A gave the lowest atterberg limit values.

4. Plasticity index, shrinkage index and shrinkage limit declined by the addition of phosphogypsum and ground granulated blast furnace slag. These indexes of samples including phosphogypsum and lime had increased when the amount of lime in samples raised.
5. Activity values of samples were decreased by addition of additive materials. However, the samples treated only by phosphogypsum had lower activity values than the other samples.
6. Swelling potential of samples was decreased when the amount of stabilizers in samples increased. Swelling potential of samples treated by phosphogypsum were gradually declined in accordance with the amount of phosphogypsum. Samples including phosphogypsum and lime had lower swelling potential than samples including only phosphogypsum. However, optimum lime content was found as 1 % by dry weight of sample. The minimum swelling potential in this study was obtained from the sample including phosphogypsum and ground granulated blast furnace slag.
7. The curing effects on samples in this study were distinct for each treated sample. The suitable curing time for samples treated by phosphogypsum was 1 day. The swelling potentials of these samples were greater than samples curing this period both longer and shorter. The curing period for samples improved with phosphogypsum and lime was 7 days. On the other hand, the curing period for samples treated phosphogypsum and ground granulated blast furnace slag decreased swelling potential progressively.

Waste material such as phosphogypsum and ground granulated blast furnace slag are detrimental to both human and environment and these waste materials can be used for the treatment of expansive soils. The utilization of phosphogypsum and ground granulated blast furnace slag, both provides a significant contribution to our economy and solution of the environmental pollution problems.

Recommendations for Future Study

It should be considered that this was a specific study for this phosphogypsum, ground granulated blast furnace slag and expansive soil. The chemical reactions depend on chemical composition and index properties of phosphogypsum, ground granulated blast furnace slag and expansive soil. Therefore, for better estimating the behaviour of phosphogypsum treated samples, different phosphogypsums should be used for different expansive soils having different mineralogical conditions. Also instead of distilled water, tap water can be used to see the effect of water on the test results. Radioactivty of phosphogypsum should be checked before using in soil stabilization.

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APPENDIX A

CHANGES OF SOIL PROPERTIES

Changes in clay content, atterberg limits, specific gravity, activity and swelling potential of the treated samples are shown in Table A.1.

Table A.1. Changes in Clay Content, Liquid Limit, Plastic Limit, Plasticity Index, Shrinkage Limit, Shrinkage Index, Specific Gravity, Activity and Swelling Potential

SAMPLE	Change in								
	Clay (%)	LL (%)	PL (%)	PI (%)	SL (%)	SI (%)	Gs (%)	Activity (%)	Swelling Percent(%)
A	0	0	0	0	0	0	0	0	0
A+%5 PG	-8.95	-20.51	0.86	-27.50	-16.13	-21.25	-0.62	-20,44	-43.65
A+%10 PG	-15.05	-26.97	2.16	-36.45	-28.03	-26.80	-1.25	-24,82	-54.03
A+%15 PG	-22.86	-31.83	4.18	-43.51	-30.00	-32.13	-1.88	-26,28	-63.79
A+%20 PG	-27.81	-34.33	5.73	-47.31	-30.80	-34.92	-2.50	-27,01	-66.24
A+%25 PG	-32.95	-37.39	7.41	-51.90	-31.53	-38.38	-3.13	-27,74	-68.30
A+%9.5 PG+%0.5 L	-17.52	-22.77	6.90	-32.43	-13.87	-24.27	-1.17	-18,25	-58.67
A+%9 PG+%1 L	-19.81	-19.73	12.84	-30.32	-11.61	-21.09	-1.09	-13,14	-65.99
A+%7 PG+%3 L	-20.76	-15.02	30.04	-29.64	-9.56	-15.94	-0.78	-10,95	-61.96
A+%5 PG+%5 L	-26.48	-13.00	36.42	-29.03	-8.39	-13.78	-0.55	-2,92	-60.87
A+%14 PG+%1 L	-25.14	-27.26	18.19	-41.98	-28.39	-27.07	-1.56	-22,63	-64.64
A+%15 PG+%15 GGBFS	-44.00	-42.79	-5.13	-54.99	-32.26	-44.56	0.00	-18,98	-78.78

"+": increase, "-": decrease

APPENDIX B

SWELL VERSUS TIME GRAPHS

Swelling percent versus time graphs were presented for Sample A. 5 % PG. 10 % PG. 15 % PG. 20 % PG. 25 % PG. 9.5 % PG+ 0.5 % L. 9 % PG+ 1 % L. 7 % PG+ 3 % L. 5 % PG+ 5 % L. 14 % PG+ 1 % L. 15 % PG+ 15 % GGBFS treated samples both without curing and with curing.

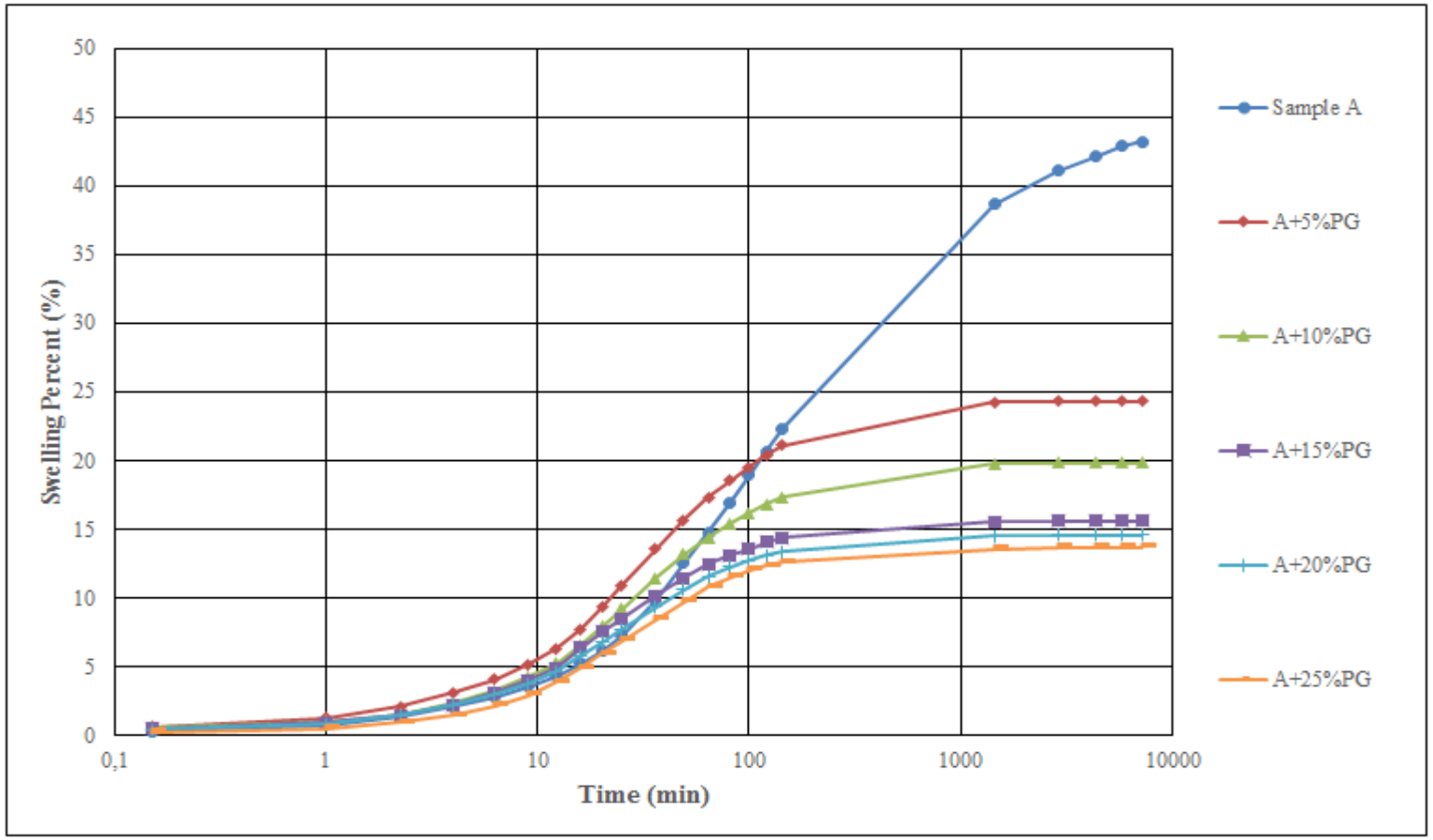

Figure B.1. Swelling percent versus Time Graph for Sample A and Phosphogypsum Treated Samples without Curing

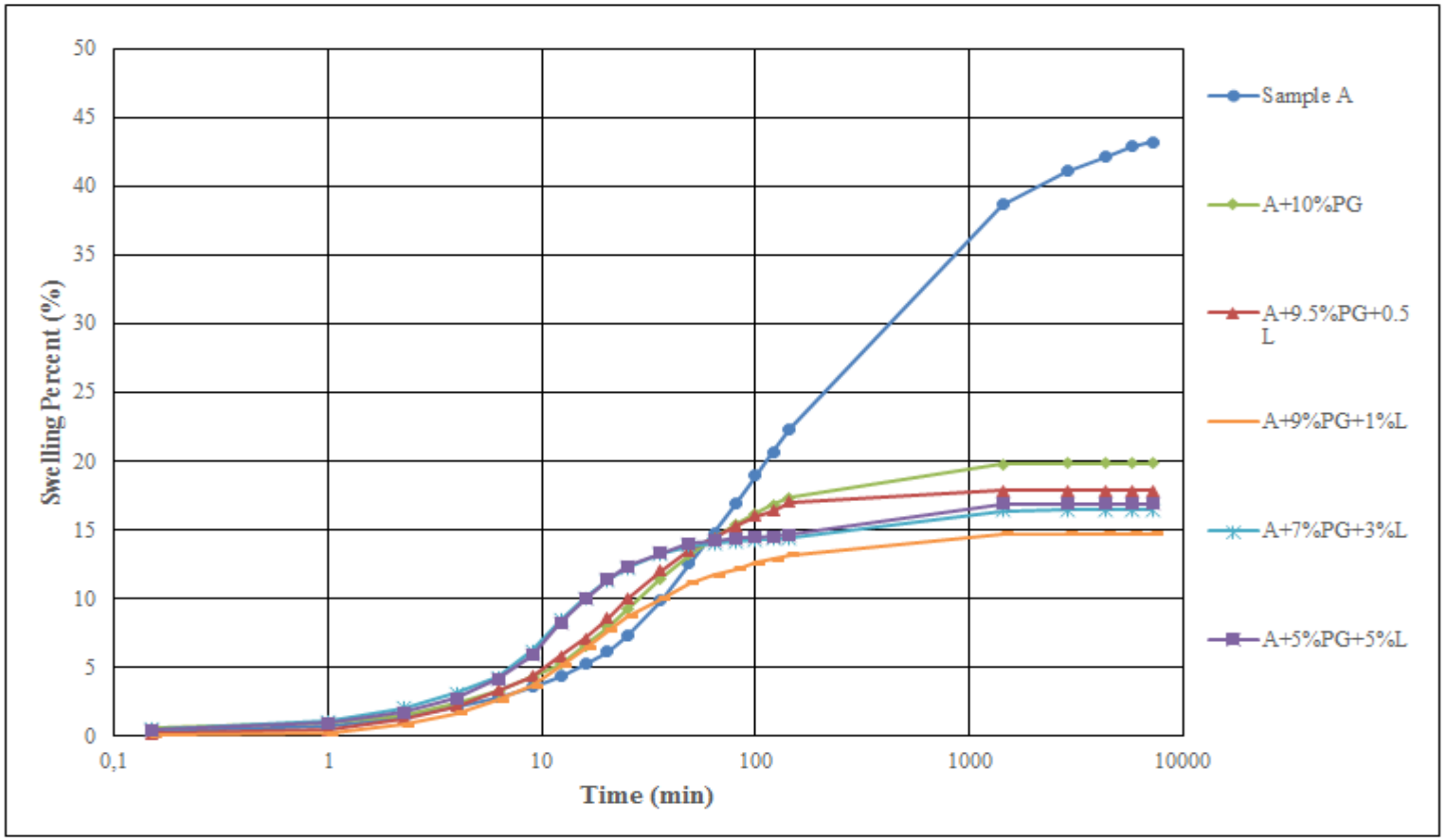

Figure B.2. Swelling percent versus Time Graph for Sample A and Phosphogypsum and Lime Treated Samples without Curing

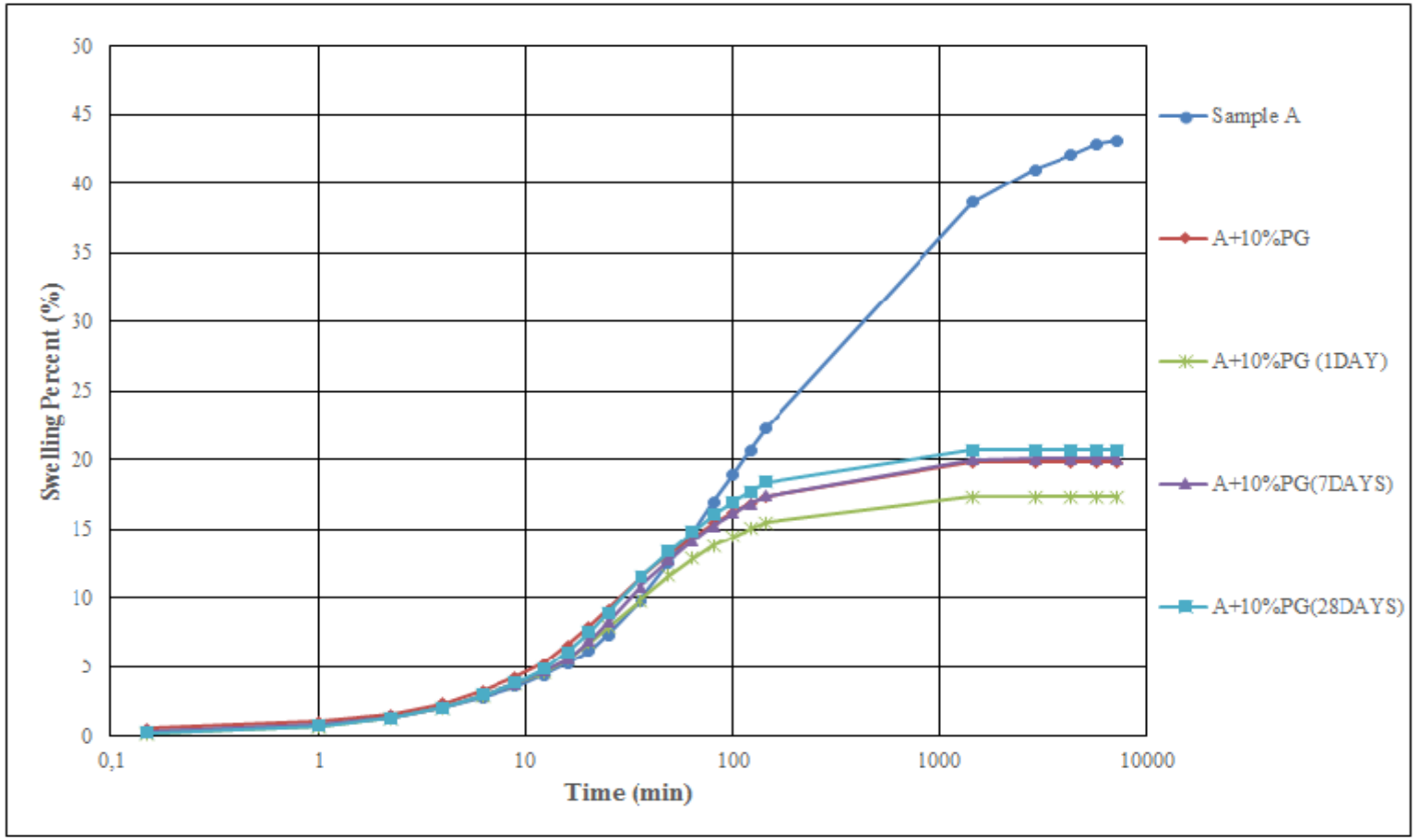

Figure B.3. Swelling percent versus Time Graph for Sample A and 10 % Phosphogypsum Treated Samples with Curing

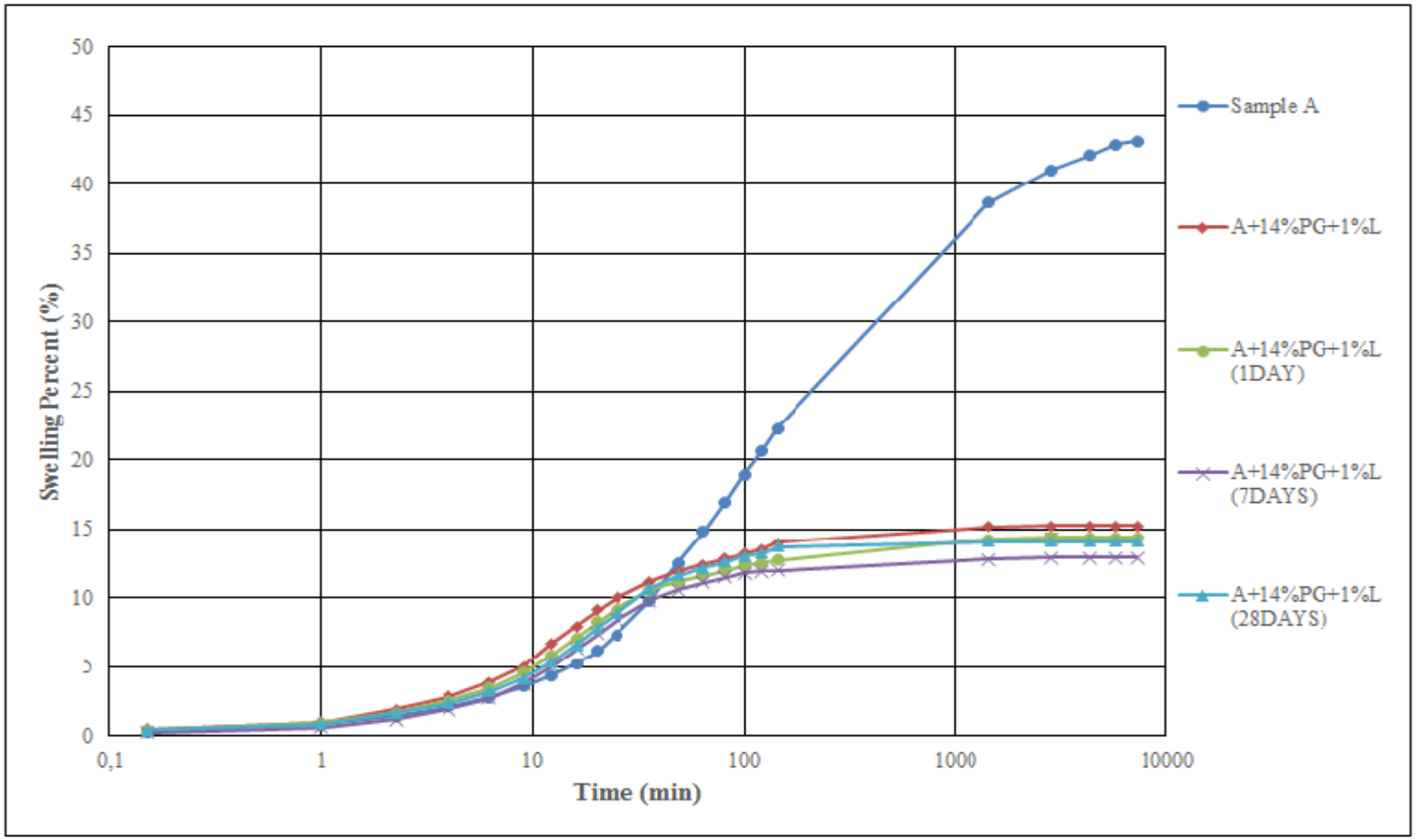

Figure B.4. Swelling percent versus Time Graph for Sample A and 14 % Phosphogypsum, 1% Lime Treated Samples with Curing

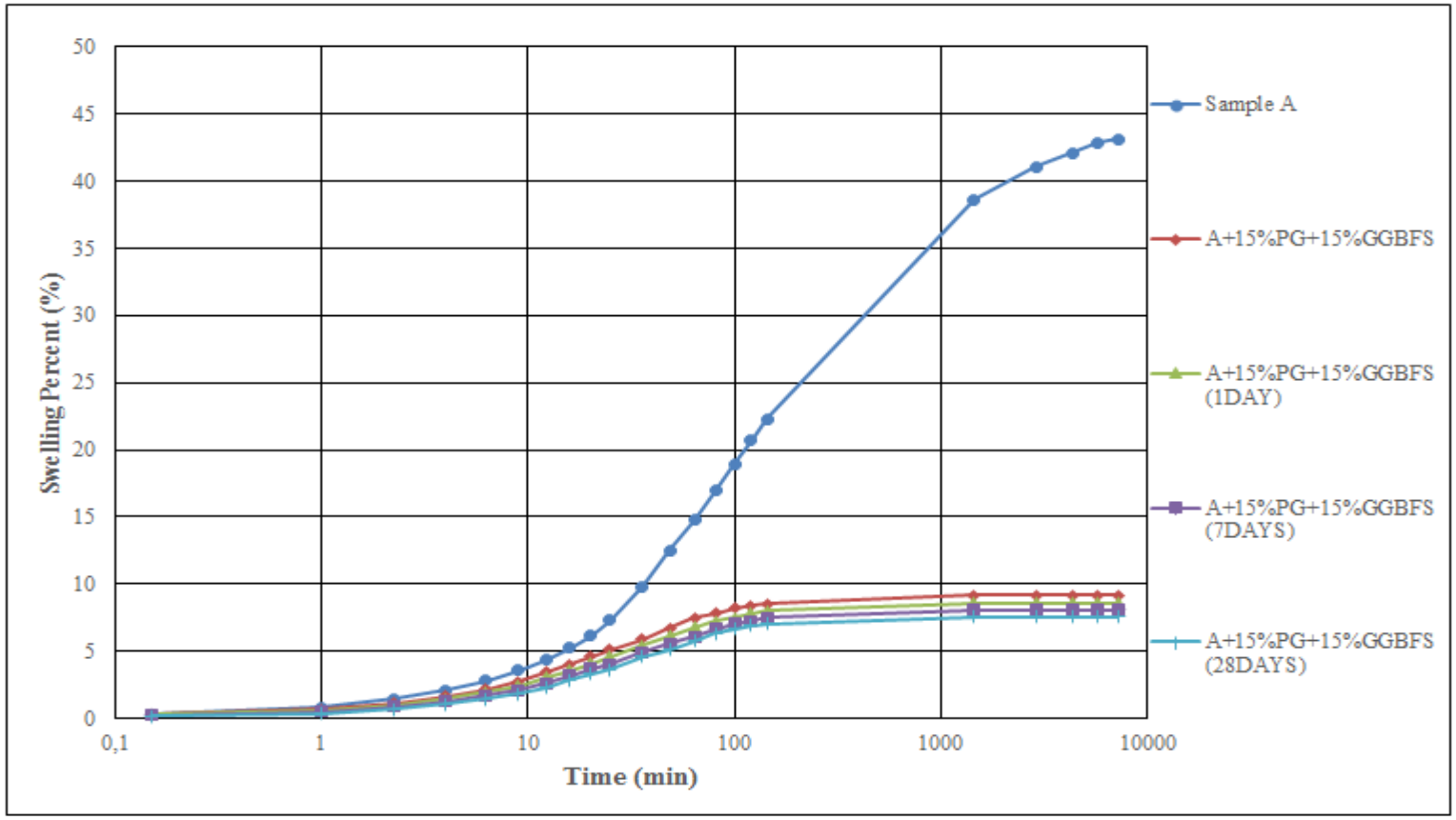

Figure B.5. Swelling percent versus Time Graph for Sample A, 15 % Phosphogypsum and 15 % Grand Granulated Blast Furnace Slag Treated Samples with Curing