IMPROVEMENT OF EXPANSIVE SOILS BY USING
CEMENT KILN DUST

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**IMPROVEMENT OF EXPANSIVE SOILS BY USING
CEMENT KILN DUST**

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ABSTRACT

IMPROVEMENT OF EXPANSIVE SOILS BY USING CEMENT KILN DUST

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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. In this study, effect of using cement kiln dust (CKD) in reducing the swelling potential was examined. The expansive soil was prepared in the laboratory by mixing kaolinite and bentonite. Cement kiln dust (CKD) was added to the soil at 0 to 12 percent by weight. Grain size distribution, Atterberg limits and swell percent and rate of swell of the mixtures was determined. Specimens were cured for 7 and 28 days. As a result of the experimental study, the effect of cement kiln dust (CKD) on swelling potential and rate of swell of the artificially prepared expansive soil specimen at laboratory conditions was seen. Tests were repeated with lime (3%, 6% and 9%), cement (3%, 6% and 9%) and sand (3%, 6%, 9%, 12% and 15%) instead of CKD for comparison. With the addition of 12 % cement kiln dust, swell percentage of

expansive soil decreased from 36.63 % to 6.74 % without curing, from 35.58 % to 6.32 % with 7 days curing and from 33.63 % to 6.00 % with 28 days curing. Based on the favorable results obtained, it can be concluded that the expansive soil can be successfully stabilized by cement kiln dust.

Key Words: Expansive Soil, Soil Stabilization, Swelling Potential, cement kiln dust (CKD), lime, cement, sand.

ÖZ

ŞİŞEN ZEMİNLERİN ÇİMENTO FIRINI TOZU KULLANILARAK İYİLEŞTİRİLMESİ

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Mayıs 2014, 113 sayfa

Şişen zeminler dünya çapında bir problem olup, inşaat mühendislerine çeşitli sorunlar teşkil etmektedir. Bu tür zeminler suya maruz bırakıldıklarında şişip, kurutulduklarında büzüşürler. Bu zeminlerin stabilizasyonu için en yaygın ve ekonomik metod hacim değişikliklerini önleyici katkılar kullanmaktır. Bu çalışmada çimento fırını tozunun (ÇFT) şişme potansiyeline etkisi incelenmiştir. Kullanılan şişen zemin numunesi laboratuarda kaolin ve bentonit karıştırılarak hazırlanmıştır. Çimento fırını tozu ağırlıkça % 0 ila % 12 oranında katılmıştır. Örneklerin dane çapı dağılımı, kıvam limitleri, şişme yüzdeleri ve şişme hızı tespit edilmiştir. Ayrıca 7 ve 28 gün kür uygulanmıştır. Yapılan deneysel çalışma sonucunda, çimento fırını tozu eklenmesinin laboratuar ortamında yapay olarak hazırlanmış şişen zeminin şişme potansiyeline ve şişme hızına etkisine bakılmıştır. Deneyler karşılaştırma amacıyla, şişen zemin numunesine CKD yerine kireç (%3, %6 ve %9), çimento (%3, %6 ve %9), ve kum (%3, %6, %9, %12 ve %15) katılarak da tekrar edilmiştir. Bu çalışmada, % 12 çimento fırın tozu kullanılarak

şişen zeminin şişme yüzdesi kürsüz % 36.63'den % 6.74'e, 7 gün kür ile % 35.58'den % 6.32'ye ve 28 gün kür ile % 33.63'den % 6.00'ya düşmüştür. Deneyler sırasında elde edilen olumlu sonuçlar neticesinde, şişen zeminlerin çimento fırın tozu kullanılarak iyileştirilebileceği sonucuna varılmıştır.

Anahtar Kelimeler: Şişen Zemin, Zemin Stabilizasyonu, Şişme Potansiyeli, çimento fırını tozu, kireç, çimento, kum.

To My Family

ACKNOWLEDGMENTS

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LIST OF ABBREVIATIONS

ASTM: American Society for Testing and Materials

C: Cement

CH: Clay with high plasticity

CKD: Cement Kiln Dust

C_C: The coefficient of curvature

C_U: The Coefficient of uniformity

D_{10}: The grain diameter at 10 % passing

D_{30}: is the grain diameter at 30 % passing

D_{60}: is the grain diameter at 60 % passing

h: Initial height of the specimen

L: Lime

LL: Liquid limit

m: Mass of Sample

METU: Middle East Technical University

MH: Silt with High Plasticity

PI: Plasticity index

PL: Plastic limit

S: Sand

SI: Shrinkage Index

SL: Shrinkage Limit

TS: Turkish Standard

t_{50}: Time to reach fifty percent swell

UCSC: Unified Soil Classification System

V_0: Initial volume of the sample

V_1: First Volume reading

V_2: Last Volume reading

w_n: Natural moisture content of the soil

ρ: Density

ΔV: Change in volume (with respect to initial volume, V_0)

CHAPTER 1

INTRODUCTION

Expansive or reactive soil is a soil composed predominantly of clay. Clay undergoes significant volume change in response to changes in the soil moisture content. This volume change is realized by swelling upon wetting, and shrinkage upon drying. Being constructed on expansive soils, buildings are frequently prone to severe movement caused by non-uniform soil moisture changes with consequent cracking and damage related to the distortion (Figure 1.1, Figure 1.2, and Figure 1.3). Rainfall and evaporation, garden watering, leaking water pipes, or tree root activity may trigger these moisture changes (Li, et al., 2014).

Figure 1.1. A View of Soil Undergoing Shrinkage (source: http://www.geology.ar.gov/geohazards/expansive_soils.htm)

Figure 1.2. A View of Road Undergoing Swelling (source:
http://geosurvey.state.co.us/hazards/Swelling%20Soils/Pages/Definition.aspx)

Figure 1.3. A View of Damage of a Building due to Expansive Soil (source:
http://www.montrealhomeinspectorpro.ca/en/home-inspection-
articles/view/560/Visual-Inspection-of-Concrete)

Having a very small particle, a large specific surface area and a high Cation Exchange Capacity (CEC) are properties of expansive soils. The expansion of specified type of clay depends on: the engineering factors of the soil, and local environmental conditions, and geology. Geology is the main factor of the presence in the soil of these types of expansive clay minerals. Plasticity, dry density, and soil moisture content are included in the engineering factors. Furthermore, the most important local environmental conditions to take into account are the amount of the clay fraction in the soil, its initial moisture conditions, and confining pressure (Seco, et al., 2011).

If expansive soils have been prevented from reacting to changes in their moisture condition by mitigating measures, for example drain systems, which are provided to come forward, the results is still low shear strength and large secondary compression should be imposed upon soils. However many engineering structures especially geotechnical and geo-environmental ones are designed by positioning them on point of bearing although these problems are obviously known. The ignorance of these problems probably originates from the indispensability of that usage thanks to their prevalence. Geotechnical engineers opt either to excavate and replace the entire soil or to improve the geotechnical properties of the soil to attuned to the potential problem rather than trying to change the whole system. Using soil additives including lime, cement, rice husk ash, asphalt, and limestone ash has widely become one of the most popular methods to improve the geotechnical properties of poor construction materials like expansive soils (Ene & Okagbue, 2009). In addition to that, cement kiln dust can be used as an additive in order to improve the expansive soils.

During the manufacture of cement clinker, cement kiln dust (Figure 1.4), a particulate mixture of calcined and unreacted raw feed, clinker dust and ash, enriched alkali sulfates, halides and other volatiles, is attained as the product of kiln. The exhaust gases facilitate to capture aforementioned particulates which are gathered in cyclones, baghouses and electrostatic precipitator, called as particulate matter control device, shown in Figure 1.5 (Adaska & Taubert, 2008). The

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precipitator, which the cement kiln dust is obtained, is marked with red dash rectangle on the Figure 1.6.

Figure 1.4. A View of Cement Kiln Dust Used in the Study

Figure 1.5. Cement Kiln Dust Control Device (Adaska & Taubert, 2008)

Two types of cement kiln processes are split by their acceptance of feed materials behaviors. Wet-process kilns accept feed materials in a slurry form, while dry-

process kilns do in a dry and ground form. Therefore dust collections of two processes are also differentiated into two ways. First, a certain amount of dust is to be reserved and added to the kiln from cyclone as the dust collection system that is the closest one to the kiln. Second, recycling or discarding process is to be realized for the total amount of produced dust (Rahman, et al., 2011).
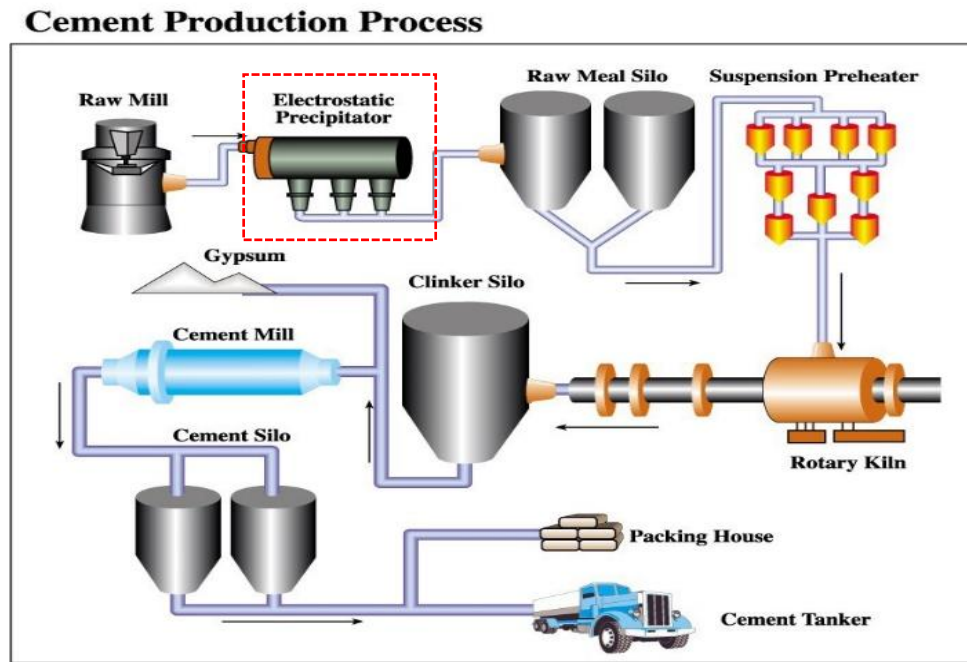
Figure 1.6. A Schematic Diagram of Cement Production Process (source: http://climatetechwiki.org/technology/energy-saving-cement)

It is estimated that generation of CKD is at approximately 30 million tons worldwide per year, and at more than 4 million tons per year in the US (Naik, et al., 2003). In the cement manufacturing process, recycling ever-increasingly demands more amounts of CKD. At the same time, landfilling or stockpiling at considerable expense is continuously required by considerable amount of CKD. In order to illustrate, according to Sreekrishnavilasam et al. (2007), the estimation

made in 1983 showed that CKD present in landfills and stockpiles was well in excess of 100 million tons.

Production amount of cement kiln dust can be roughly estimated by considering the produced clinker amount. In fact, cement kiln dust generation can be assumed 7 % of the clinker generation by weight (Naik, et al., 2003).

In Turkey, the clinker production amount in 2013 was approximately 60 million tons (source: http://www.tcma.org.tr/index.php?page=icerikgoster&menuID=38). In the light of the study of Naik, et al., (2003), the generation of cement kiln dust in Turkey can be roughly estimated as 4.2 million tons in 2013.

1.1. Objective of Study

The main objective of this study is the stabilization of expansive soil with cement kiln dust. To investigate the long-term performance (7 days and 28 days of curing) of CKD on reducing the swell potential of expansive clay is another objective of this study. In order to make comparison, the experiments were repeated with the chemicals / additives of whose improvement effects on expansive soils are proven such as lime, cement, and sand. With this study, the improvement of expansive soils in the active zone is aimed. Moreover, by stabilizing the expansive soils with cement kiln dust (the waste material), the reduction in waste amount is provided which is very important for the environmental aspect (i.e one of the objectives of this study is to reduce the storage volume of CKD).

1.2. Scope of Study

In the scope of this thesis, in Chapter 1, the expansive soils and cement kiln dust are introduced. Literature review on expansive clays and previous studies related to stabilization of expansive clays with CKD are given in Chapter 2. Chapter 3

contains the experimental studies, in this study, standard proctor test, Atterberg limit tests, hydrometer tests, sieve analyses tests, specific gravity tests, and free swell tests with and without curing were performed. The effects of adding stabilizers such as cement kiln dust, lime, cement and sand were investigated. In chapter 4, the results of the tests are discussed. Finally, chapter 5 contains the conclusions of the study and recommendations for the future study.

CHAPTER 2

LITERATURE REVIEW

The basic and primary problem that originates with regard to expansive soils is that elastic deformations are significantly less than that type of soils' deformations and the predictions about that cannot be made by classical elastic or plastic theory. As Nelson & Miller (1992) states, movement is usually in an uneven pattern and of such a magnitude as to cause extensive damage to the structures and pavements on them.

More than any other natural hazards including earthquakes and floods, expansive soils are the cause of greater cost of repair to structures, particularly light buildings and pavements (Jones & Holtz, 1973).

2.1. Clay Mineralogy

There are basically only two fundamental crystal sheets, the tetrahedral or silica and octahedral or alumina sheets in the structure of a clay material. Mainly a combination of silica tetrahedral units, the tetrahedral, consists of four oxygen atoms at the corners, surrounding a single silicon atom. The octahedral sheet is basically a combination of octahedral units including six oxygen or hydroxyls enclosing aluminum, magnesium, iron, or other atom. If all the anions of octahedral sheet are hydroxyls and two-thirds of the cation positions are filled with

aluminum, then the material is called gibbsite, Holtz & Kovacs (2011) states and stresses that if magnesium was substituted for the aluminum in the sheet and it filled all the cation positions, then the mineral was called brucite.

Figure 2.1 and Figure 2.2, present a silica tetrahedron and a silica sheet, also an octahedron and an octahedron sheet respectively.
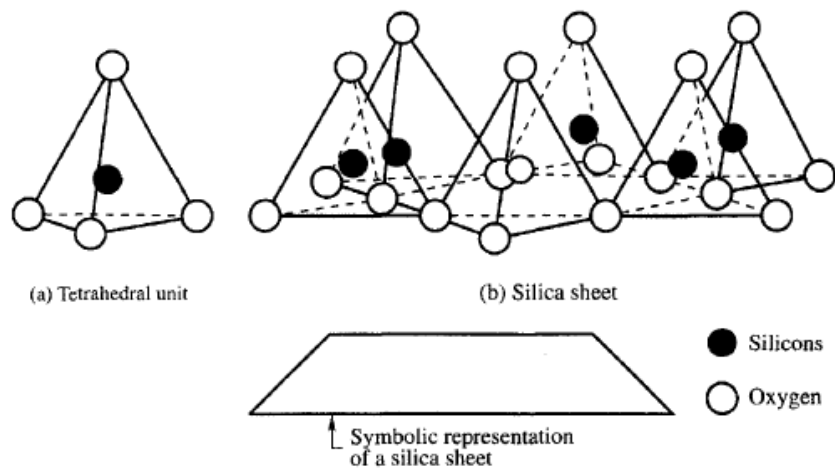
Figure 2.1. Basic Structural Units in the Silica Sheet (Murthy, 2002)
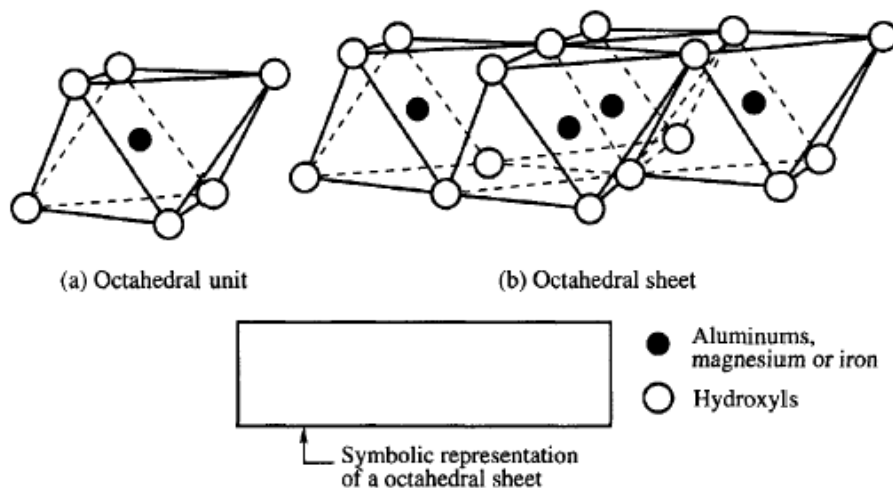
Figure 2.2. Basic Structural Units in the Octahedral Sheet (Murthy, 2002)

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The classification of clay minerals is made by three main groups, as follows:

    i.   Kaolinite group

    ii.   Illite group

    iii.   Smectite group

2.1.1. Kaolinite Group

Some clay minerals consist of repeating layers of two-layer sheets. For example, a combination of a silica sheet with a gibbsite sheet or a combination of a silica sheet with a brucite sheet can be defined as two-layer sheets. Hydrogen bonding and secondary valence forces hold the repeating layers together. The most important clay mineral of that type is kaolinite (Das, 2008).

The bonding between layers is sufficient; therefore there is no interlayer that can cause any swelling in the presence of water (Mitchell & Soga, 2005). However it has a low swelling possibility, which may be realized by low liquid limit, low activity, and yields hydraulic conductivity of 10^{-6} cm/s or higher (Oweis & Khera, 1998). Structure of kaolinite is given in Figure 2.3.

Ionic bond

Hydrogen bond

Gibbsite sheet

Silica sheet
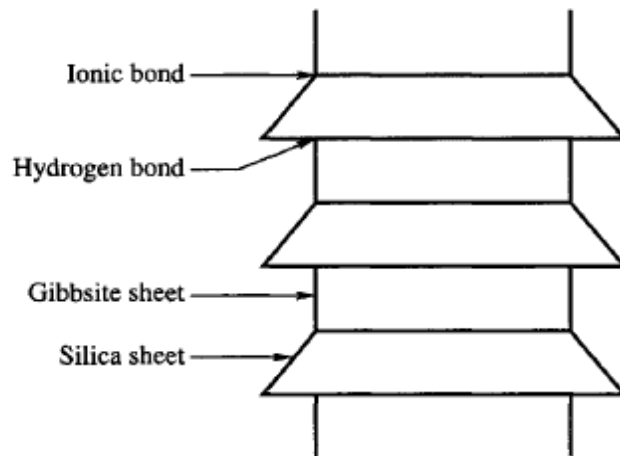
Figure 2.3. Structure of Kaolinite Layer (Murthy, 2002)

2.1.2. Illite Group

The main structural unit of illite is similar to that of montmorillonite, although there is a tiny difference. For the main structure of illite, aluminum atoms always replace some of the silica the resultant charge deficiency is balanced by potassium ions. The potassium ions exist between layers of the unit. As Murthy (2002) states, the bonds with the nonexchangeable K^+ ions are weaker than the hydrogen bonds, but stronger than the water bond of montmorillonite.

Illite is very stable and is common in soils and sediments. Its swelling potential is moderate, liquid limit is higher than kaolinite, it yields hydraulic conductivity of 10^{-7} cm/s or lower (Oweis & Khera, 1998). Structure of illite is given in Figure 2.4.
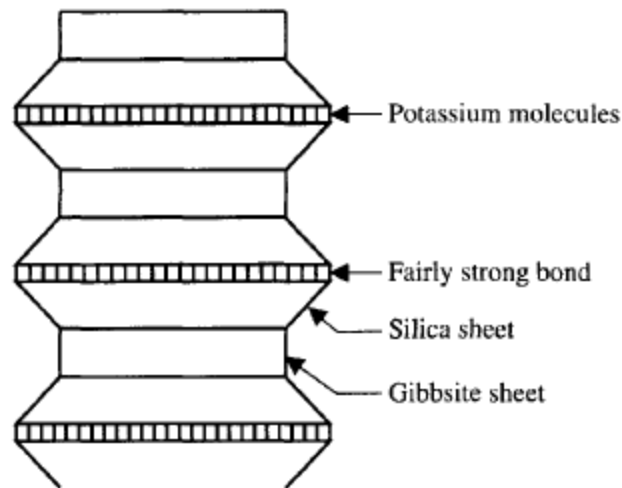
Figure 2.4. Structure of Illite Layer (Murthy, 2002)

2.1.3. Smectite Group

2.1.3.1.   Montmorillonite

As stated above, montmorillonite has the same basic structure as illite. In the octahedral sheet, aluminum is partially substituted by magnesium. Water molecules and exchangeable cations other than potassium occupy the space between the combined sheets. These ions cause a weak bond between the combined sheets (Craig, 2004). This bond is prone to be broken by water or other polar or cationic organic fluids entering between the sheets due to its weakness. The entrance of water molecules is easily realized through the layers expanding substantially and bearing much smaller particles with a very large specific surface (Oweis & Khera, 1998).

For that type of clays, swelling potential, activity, and liquid limit are at the utmost extent. Structure of montmorillonite is given in Figure 2.5.
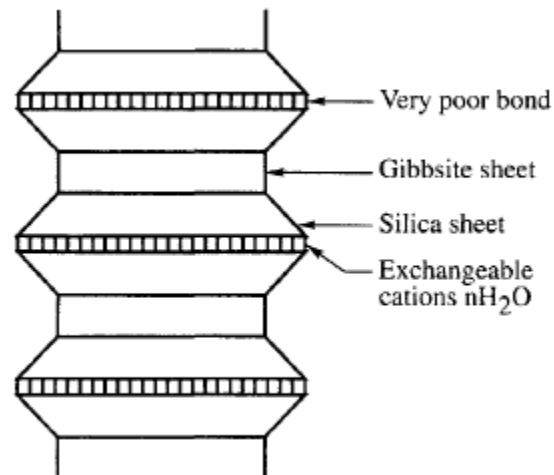
Figure 2.5. Structure of Montmorillonite Layer (Murthy, 2002)

2.2. Clay Structure

Dispersed and flocculated structures (Figure 2.6) are the two elementary structures of clay particles. If the magnitude of the attractive force is lesser than the magnitude of the repulsive force, the final result will exactly be repulsion. The individual clay particles will settle and form a dense layer at the bottom; however, they will separately remain from their surroundings. This is referred to as the dispersed state of the soil. On the other hand, flocs will be formed and these flocs will settle to the bottom, if the net force between the particles is attraction, which is called flocculated clay (Das, 2008).
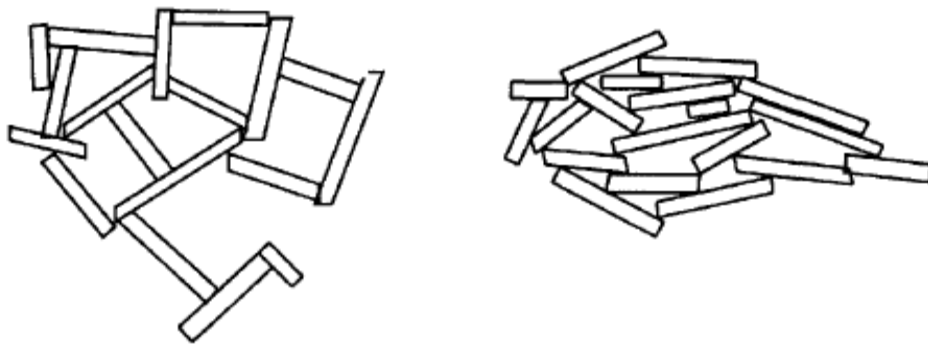
Figure 2.6. Flocculated and Dispersed Type Soil Structures respectively (Murthy, 2002)

2.3. Diffuse Double Layer

Every soil particle is found in nature as surrounded by water. The molecules gaining the specialty of being dipole take place in the centers of positive and negative charges of water molecules and prevent matching up. As a natural result of that, hydrogen (positive end) of the water molecules is attracted by the negative

charge on the surface of the soil particle. As shown in Figure 2.7, the definite pattern of the arrangement of the water molecules are achieved in immediate vicinity of the boundary between solid and water. A considerable and attractive force, which prevents increasing the distance of water molecule from the surface, helps more than one layer of water molecules to stick on the surface. Then the clay particle is surrounded by the electrically attracted water. That phenomenon is called as the diffused double-layer of water (Murthy, 2002).

The cation valence, electrolyte concentration, temperature, and dielectric constant (the measure of ease with which molecules can be polarized and oriented in an electric field) of the medium are determined the thickness of the diffuse double layer (Oweis & Khera, 1998).
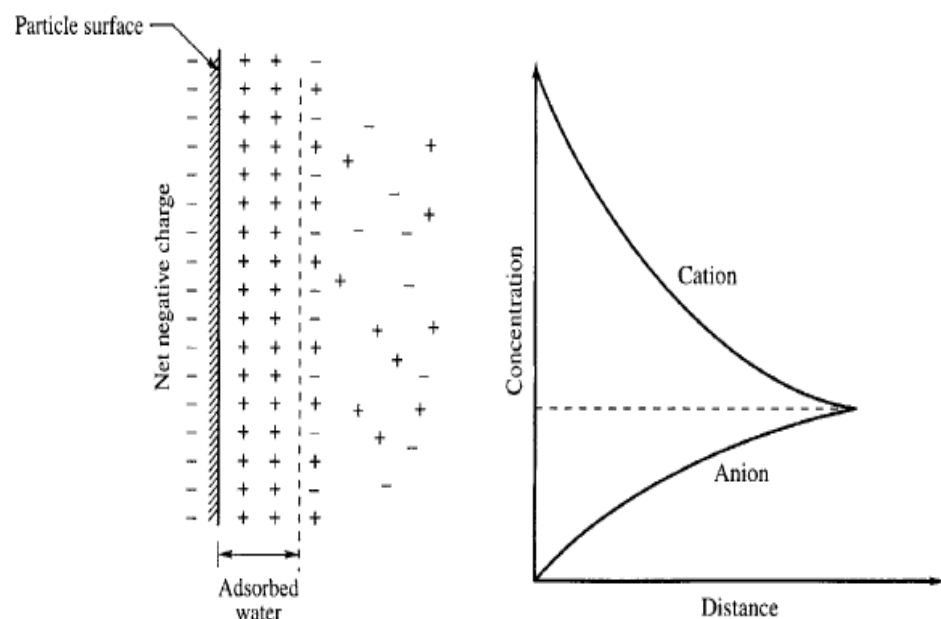
Figure 2.7. Distribution of cations and anions adjacent to a clay particle surface according to the diffuse double layer theory (Keijzer, 2000).

2.4. Cation Exchange Capacity (CEC)

The cations can be arranged in a series in terms of their affinity for attraction as below that shows some cations are strongly more attracted than the others:

$Al^{3+} > Ca^{2+} > Mg^{2+} > NH^{4+} > K^+ > H^+ > Na^+ > Li^+$, (Das, 2008).
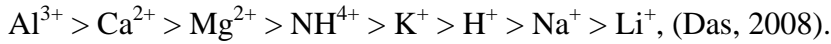
This series illustrate that a hydrogen clay can be transformed to sodium clay by a constant percolation of water including dissolved Na salts. Such changes can facilitate decreasing the permeability of a soil. However, all adsorbed cations are not exchangeable. The quantity of exchangeable cations in a soil determines exchange capacity (Murthy, 2002).

CEC is term that referred to the quantity of exchangeable cations required to balance the charge deficiency on the surface of the clay particles. Higher CEC, higher surface activity and consequently higher water absorption potential usually belong to clays with larger specific surface area. Cation exchange capacity (CEC) values have been used to explain the effect of the new pozzolanic reaction products on the particle size and the swell potential of the treated soils (Nalbantoğlu, 2004). Typical values of CEC for three basic clay minerals are tabulated in Table 2.1 as follow:

Table 2.1. Exchange Capacities of Some Clay Minerals (Murthy, 2002)

Mineral group	Exchange capacity (meq per 100 g)
Kaolinites	3.8
Illites	40
Montmorillonites	80

2.5. Mechanism of Expansion

The notion of clay swelling has basically two main mechanisms. The first of that is the swelling that realizes between particles. In clay accumulations that can be evaluated as dry, capillary gap between clay crystals holds these clay crystals together by its water vacuum force. When the clay unit is exposed to water, that force will discharge and the unit will swell. The second swelling mechanism that is generally seen in montmorillonit type of clays can be defined by the percolation through weak-bonded singular surfaces that form crystals as well as the crystals when the clay is exposed to water and the swelling of clay unit with greater volumetric increase (Popescu, 1986). Mechanism of swelling is presented in Figure 2.8.

HYDRATION VOLUME CHANGES

interparticle or intercrystalline

NEAR DRY

Reversible

WET

INTERPARTICLE

Effective for all clay minerals

— Relict capillary water

Single crystal —

— Relict capillary water

— Strong bonds
— Single crystal
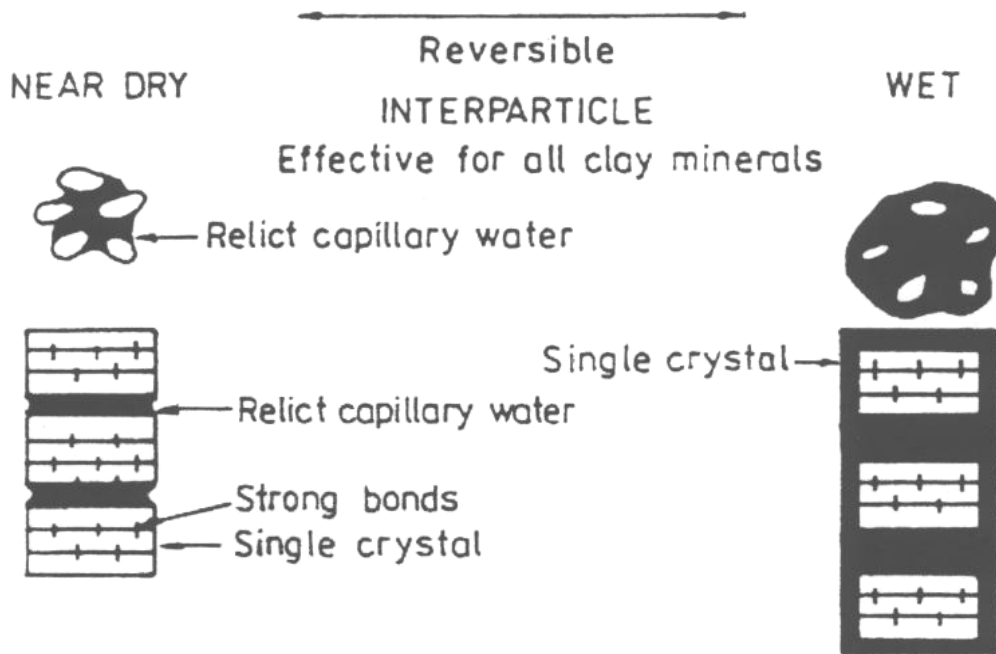
INTRACRYSTALLINE

Effective for montmorillonite, including that which can
be interlayered with other clay minerals

— Single crystal

—intercrystalline water also —
present

Single crystal
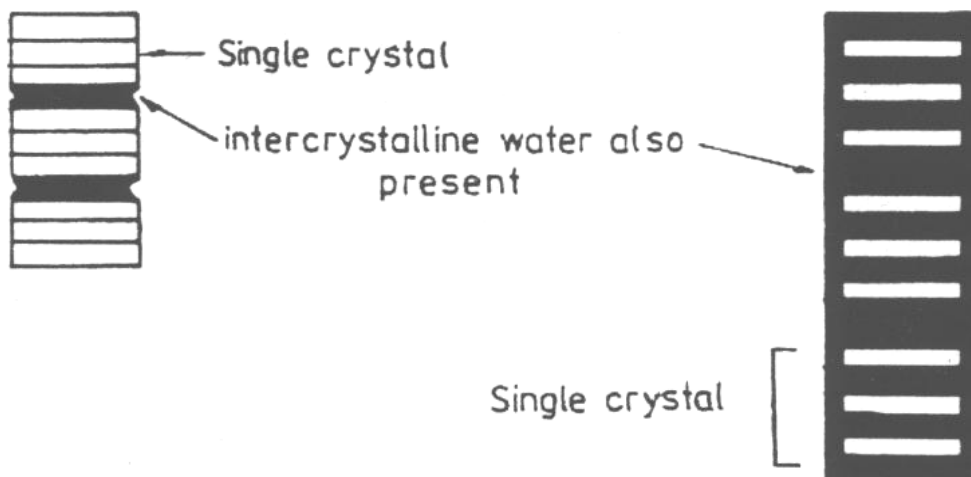
Figure 2.8. Mechanism of Swelling (Popescu, 1986)

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2.6. Factors Affecting Expansion Behavior of Soils

Factors affecting the expansion behavior of soils are summarized below in section 2.6.1, 2.6.2 and 2.6.3 (Nelson & Miller, 1992);

2.6.1. Soil Properties Influencing Swell Potential

2.6.1.1. Clay Mineralogy

Montmorillonites, vermiculites, and some mixed layer minerals are the clay minerals which cause volume changes. Infrequently expansive minerals are illite and kaolinites, but can trigger volume alteration if particle sizes are very fine (less than a few tenths of a micron).

2.6.1.2. Soil Water Chemistry

Increased cation concentration and increased cation valence repress swelling. To illustrate, Mg^{2+} cations in the soil water would outcome by less swelling than Na^+ cations.

2.6.1.3. Soil Suction

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

2.6.1.4.   Plasticity

Behaving in the way of plastic over wide-ranging moisture substance and having high liquid limits potential soils have usually tended to have the makings of swelling and shrinkage. The potency of swelling is showed by plasticity.

2.6.1.5.   Soil Structure and Fabric

Dispersed clays have a higher tendency for expansion than flocculated clays. Swelling can be reduced by cemented particles. Compaction at high water content or remolding alters fabric and structure. Kneading compaction has been displayed to create dispersed structures with lower swelling potential compaction statically at lower moisture contents.

2.6.1.6.   Dry Density

Higher densities generally associated with closer particle spacing and which mean greater repulsive forces between particles and larger swelling potential.

2.6.2. Environmental Factors Affecting Swell Potential

2.6.2.1.   Initial Moisture Content

Desiccated expansive soil absorbs water highly and attracts more water comparing to the soil at higher moisture. On the other hand, a relatively dry initial profile has a more resistance capacity than a wet soil profile when it comes to exposure to drying effects. Additionally, a wet soil one shrinks more than a relatively dry

initial one. The correlation between the expected range of final suction conditions and the vacuum effect of the initial soil must be taken into consideration.

2.6.2.2.   Moisture Variations

Changes in moisture in the active zone near the upper part of the profile mainly define heave, which is in those layers that the widest variation in moisture and volume change will be realized.

2.6.2.2.1.   Active Zone Depth

A substantial criterium of evaluating the swelling surface problems is the active zone depth, the greatest depth of water content waving (Coduto, 2005). Water content below that depth can be accepted as constant and swelling would not be occurred. Coduto (2005) states active zone depths for some cities: Houston (1.5-3.0m), Dallas (2.1-4.2m), Denver (3.0-4.2m), and San Antonio (3.0-9.0m). It is complicated to determine the active zone depth of a field. It can be ascertained by taking the change of LI (liquidity index) value in terms of depth into account (Das, 1999; Güngör, 2002).

LI= (wn-PL) / PI where,

LI: Liquidity index of the soil

wn: Moisture content of the soil

PL: Plastic limit of the soil

PI: Plasticity index of the soil

After the calculation of LI from specified formulation above, active zone depth can be estimated from Figure 2.9.
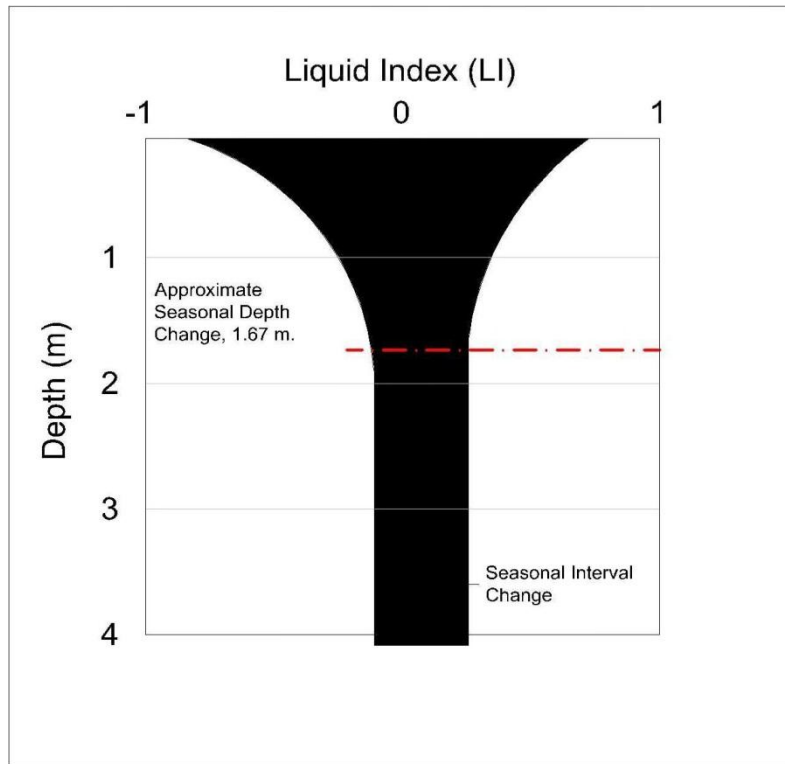
Figure 2.9. Approximate Determination of Active Zone Depth (Das, 1999; Güngör, 2002)

2.6.2.3.   Climate

The moisture availability and depth of seasonal moisture fluctuation are greatly influenced by amount and variation of precipitation and evapotranspiration. Greatest seasonal heave occurs in semiarid climates which have short wet periods.

2.6.2.4.   Groundwater

Shallow water tables show source of moisture and fluctuating water tables contribute to moisture.

2.6.2.5.   Drainage

Ponding around a poorly graded house foundation can be given as an example to surface drainage features that are to be resource of water at the surface. Therefore soil is able to access to water at greater depth thanks to leaky plumbing.

2.6.2.6.   Vegetation

Trees, grasses and shrubs consume moisture from soil through transpiration, which differentiate the soil in areas of varying vegetation.

2.6.2.7.   Permeability

Especially, because of cracks and fissures in the field soil mass, higher permeable soils accelerate absorption of water and increase the rates of swell.

2.6.2.8.   Temperature

Moisture can diffuse to cooler areas beneath pavements and buildings due to increase in temperature.

2.6.3. Stress Conditions Affecting Swell Potential

2.6.3.1.   Stress History

An over-consolidated soil is more expansive than the same soil at the same void ratio, but normally consolidated. Swelling pressures can increase in aging of

compacted clays, but amount of swelling under light loading has been indicated to be unaffected by aging. Repeated wetting and drying tend to reduce swelling in laboratory samples, but after ongoing wetting-drying cycles, swelling is unaffected.

2.6.3.1.1.   Cyclic Swelling Shrinkage Behavior

Up and down movement of foundations built on swelling surfaces due to swelling-shrinkage cycle causes fatigue and cracking in the structure. The studies on that issue show that particularly wetting and drying cycles has a greater influence on swelling ability of swelled surfaces (Tripathy & Subba Rao, 2009). On the other hand, there are clear differences between outcomes (Türköz, 2009). Many researchers studying on swelling-shrinkage behavior of clays differently evaluate the problem that is caused by this behavior. Some researchers argue that in case of repeatedly swelling of clay samples and exposure of that to shrink in such a way that the beginning comes through water content, the sample will show fatigue phenomenon and consequently causes less swelling. However other researchers remark that swelling potential increases by the amount of wetting and drying cycles in case of that sample is exposed to water content which is below the limit of sample's shrinkage. The researchers studying on that issue agree on the fact that swelling reaches to balance after the certain cycle of swelling-shrinkage (Türköz, 2009). Türköz (2009) finds that relating increase in cycle amount, swelling ability of surface that has high swelling potential decreases correlating with increasing particle size when cycling effect on swelling potential of surfaces is evaluated.

2.6.3.2.   In-situ Conditions

Estimation for the state of the initial stress in a soil must be made to evaluate either the probable result of loading the soil mass or changing the proportion of moisture.

However laboratory environment tests, in-situ measurements and observations are not the best ways to determine the initial effective stresses because they roughly give accurate results.

2.6.3.3.   Loading

The amount of volume change that will occur for given moisture content and density is determined by the magnitude of surcharge load. Inter-particle repulsive forces are acted to be balanced by an externally applied load and reduced by swelling.

2.6.3.4.   Soil Profile

Potential movements are considerably influenced by the thickness and location of potentially expansive layers in the profile. The greatest movement will occur in profiles that have expansive clays extending from the surface to depths below the active zone. If expansive soil is overlain by non-expansive material or overlies bedrock at shallow depth, lesser movement will be realized.

2.7. Factors Affecting the Magnitude of Swelling

Footings experience lifting due to the swelling or heaving of the soil if they are built in expansive soil. The amount of total heave and the rate of heave of the expansive soil on which a structure founded are very complex. The heave estimate depends on many factors which cannot be readily determined. Some of the major factors that contribute to heaving are: (Murthy, 2002)

i. Climatic conditions involving precipitation, evaporation, and transpiration affect the moisture in the soil. The depth and degree of desiccation affect the amount of swell in a given soil horizon.

ii. The thickness of the expansive soil stratum is another factor. The thickness of the stratum is controlled by the depth to the water table.

iii. The depth to the water table is responsible for the change in moisture of the expansive soil lying above the water table. No swelling of soil takes place when it lies below the water table.

iv. The predicted amount of heave depends on the nature and degree of desiccation of the soil immediately after construction of a foundation.

v. The single most important element controlling the swelling pressure as well as the swell potential is the in-situ density of the soil. On the completion of excavation, the stress condition in the soil mass undergoes changes, such as the release of stresses due to elastic rebound of the soil. If construction proceeds without delay, the structural load compensates for the stress release.

vi. The permeability of the soil determines the rate of ingress of water into the soil either by gravitational flow or diffusion, and this in turn determines the rate of heave.

2.8. Treatment of Expansive Soils

Being available for stabilizing expansive soils before and after construction of structures and highways treatment, procedures include (Nelson & Miller, 1992);

1. Chemical additives,

2. Prewetting,

3. Soil removal replacement,

4. Moisture control and compaction,

5. Surcharge loading,

Since the objective of this study is stabilization of expansive soils with cement kiln dust, lime, cement and sand, chemical additives will be examined more in detail compared to other treatment methods.

2.8.1. Chemical Additives

Chemical additives are mostly industrial waste or by-products materials, selected for their high mineral content. The dual aim is to improve in the properties of expansive soils, together with an increase in value of the by-products and waste materials, and to decrease of the environmental problems currently involved in their disposal (Seco, et al., 2011). Lime, cement, fly ash and salt are the additives used for stabilization of expansive soils in general. Otherwise, cement kiln dust has been recently used.

Chemical additives are greatly used in civil engineering operations such as road construction, embankments, foundation slabs and piles. By adding chemical additives to clay soils with water, a number of reactions including cation exchange, flocculation, carbonation and pozzolanic reaction lead to the improvement of soil properties. Between the cations associated with the surfaces of the clay particles and calcium cation of the chemical additives, a cation exchange has been occurred. Clay particles become closer to each other due to the effect of cation exchange and attraction and that movement forms flocs. This process is called flocculation, which is primarily responsible for the modification of the engineering properties of clay soils when treated with such additives (Al-Rawas, et al., 2005).

2.8.1.1.   Lime Stabilization

Lime is defined as the high-temperature product when the calcination of limestone is occurred.

Several studies explain the improvements in the mechanical and hydraulic properties of expansive soil due to the addition of lime, mainly from a macroscopic point of view. Most of these researches focuses on studying the changes associated with the Atterberg limit values (plastic and liquid limits) and the soil properties, such as the volume change (i.e., swell pressure, swell potential, heave), shear strength and the coefficient of permeability of the stabilized expansive soils (Al-Mukhtar, et al., 2012).

Figure 2.10 and Figure 2.11 show the before/after lime treated soil and the application of treating lime, respectively.

Figure 2.10. A View Illustrating before and after Lime Treated Soil (source: http://www.limestabilization.com/lime-uses)

After applying lime slurry to prepared soil, the machine is run in reverse to ensure thorough mixing to the specified depth.

Figure 2.11. A View of Stabilizing Expansive Soil with Lime (source: http://www.theholbrookcompany.com/university-of-dallaz-student-housing-152600)

Nalbantoglu and Tuncer (2001) conducted swelling laboratory tests on an expansive clayey soil (plasticity index equal to 46%) treated with different lime contents (i.e., 0%, 3%, 5% and 7% of lime by dry weight of soil). They indicate that the soil swelling potential, i.e. ratio between increase in height due to wetting to initial height (swelling strains measured after saturation in an oedometric cell at the Proctor optimum conditions) was drastically decreased from 20% for the untreated sample to 1.5% for the sample treated with 2% lime with no curing period. They also argue that the higher the lime content, the longer the curing time and the lower the swelling potential of the treated expansive soil (Stoltz, et al., 2012).

The chemical reaction between soil and lime is presented in Figure 2.12.
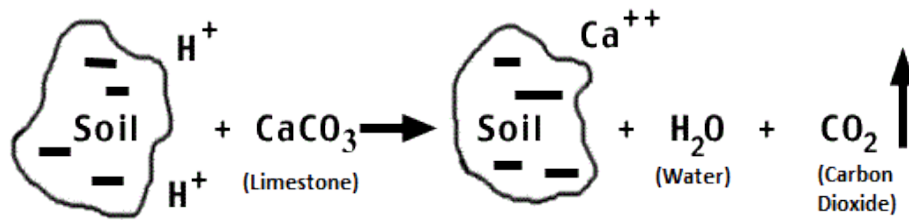
Figure 2.12. Expression of Chemical Reaction with Soil and Lime (source: http://hubcap.clemson.edu/~blpprt/acidity2_review.html)

2.8.1.2. Cement Stabilization

It can be claimed that cement is not relatively effective, when it is compared to lime, in stabilizing highly plastic clays. As Mitchell and Raad (1973) conclude, some clay soils have such a high affinity for water that the cement may not hydrate sufficiently to produce the complete pozzolanic reaction. It can be usually advantageous to use cement in the case of that soils are not reacted by lime.

The hydration of cement can be defined as complex pozzolanic reaction that results in a varied different compounds and gels (Figure 2.13). The result of mixing cement with clay soil has similar outcomes with that of lime. The mix of cement and clay soil causes a decrease in the liquid limit, the plasticity index, and the potential for volume change, while it, however, makes an adjuvant effect on the shrinkage limit and shear strength (Chen, 1975).

Cement in soil stabilization strengthens the mixture. In fact, in clay soils the chemical reaction of cement and soil is responsible for soil improvement. Calcium Silicate Hydrate and lime, the hydration process of two major elements of cement produces, are two new products in this process. Therefore, the interaction between this lime and clay minerals may also be essential effective in terms of the soil cement interaction process (Yong & Ouhadi , 2007).

Abdullah and Alsharqi (2011) states that Portland cement was added to medium expansive soil in four cement percentages (1%, 2%, 3% and 4%) all with a 7- day

curing period and a 28-day curing period. Since the 28-day period with 2% cement content was sufficient to reduce swell potential drastically, 28-day curing period was only conducted for the first two percentages. Large amount of reduction of free swelling was observed even for small percentage of cement content as small as 1%, where highest swell value was reduced from 7.4% to about 4% and about 3% cured for 7-day period and 28-day period, respectively.
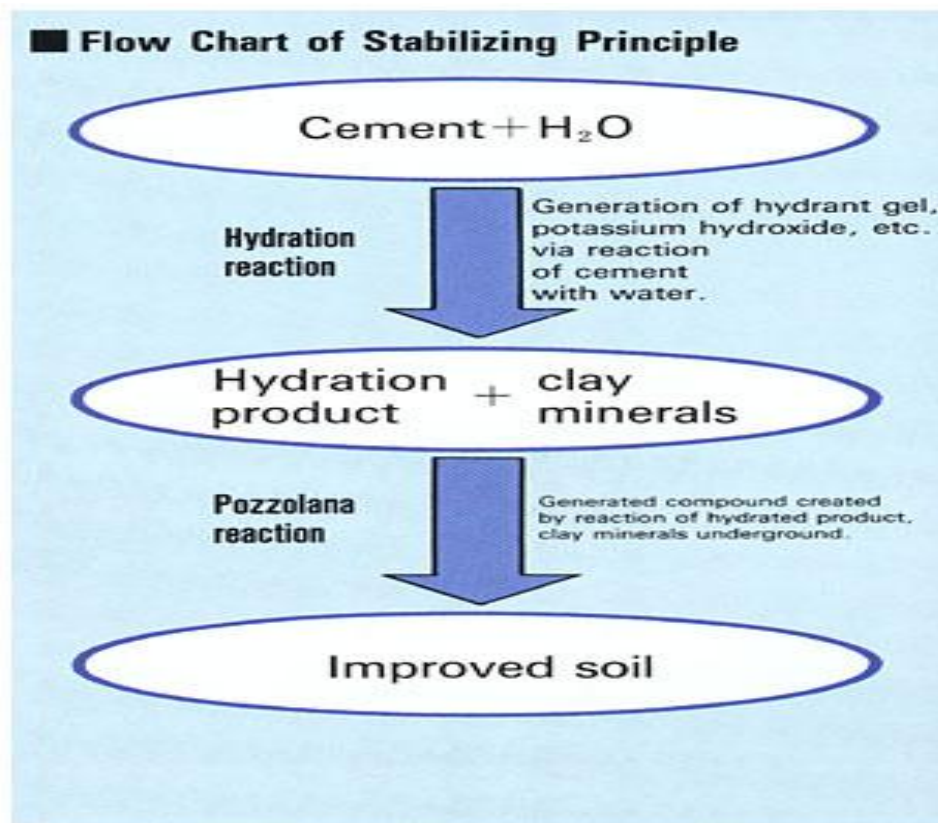
Figure 2.13. Improvement Process of Soil with Cement (source: http://cdm-gr.com/english/what.html)

The application of cement treatment and the schematic diagram of changes in clay-cement structure during curing are illustrated in Figure 2.14 and 2.15, respectively.

Figure 2.14. A View of Stabilizing Expansive Soil with Cement (source:
http://www.ctre.iastate.edu/research/detail.cfm?projectID=-373342403)

(a) As Compacted Condition

(b) After Short Curing Period

Indistinguishable Clay and Cement Phase

(c) After Long Curing Period
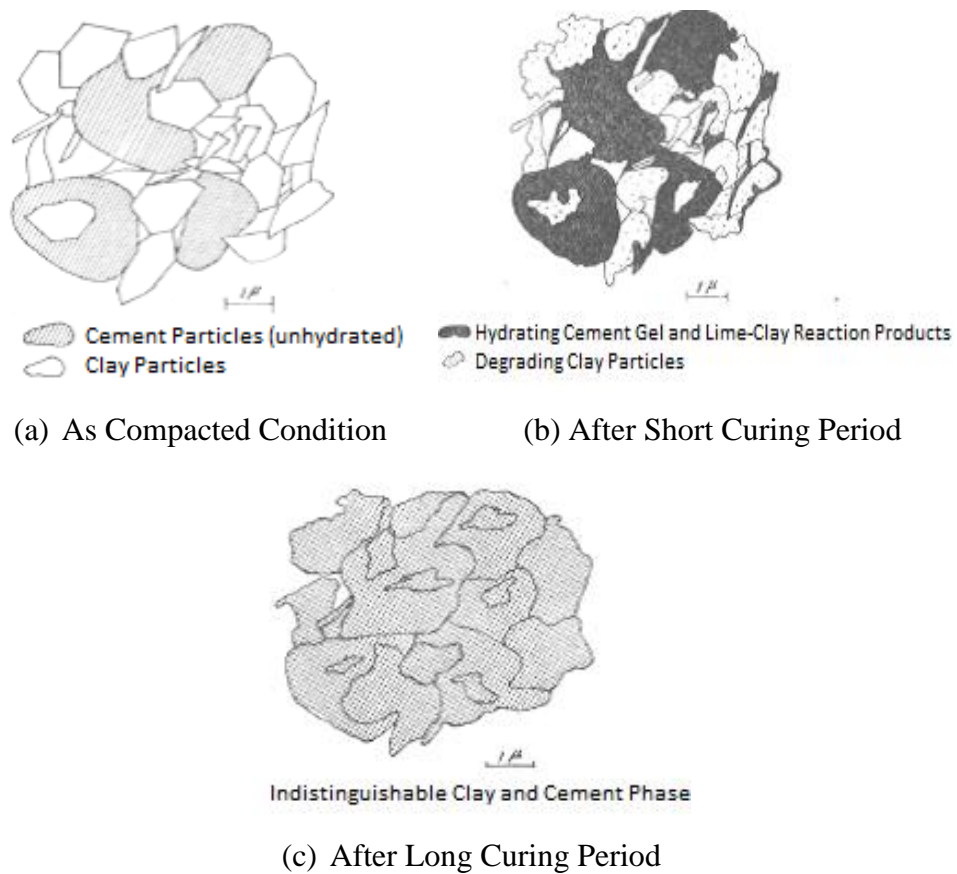
Figure 2.15. Diagram of Changes in Clay-Cement Structure during Curing
(Mitchell & Jack, 1966)

2.8.1.3.   Fly Ash Stabilization

Fly ash is the indicator of the particulate matter captured from exhausted gases of coal burning thermoelectric power plants by electrostatic precipitators.

In recent years, fly ash has been beneficially reused in soil stabilization and that re-usage is a growing trend and has increased significantly throughout the nation. In addition it is expected that the trend is to grow in future years. Several laboratory and field studies have shown that cementitious fly ashes (Class C) highly affect to improve the geotechnical properties of fine-grained soils. Moreover, the stabilization of high plasticity soft and/or expansive soil(s) with fly ash causes significant reduction of plasticity and shrink–swell potential (Shafique, et al., 2010).

Fly ash can provide a sufficient array of divalent and trivalent cations (Ca^{2+}, Al^{3+}, Fe^{3+}, etc.) under ionized conditions providing flocculation of dispersed clay particles. Therefore, expansive soils can be potentially stabilized effectively by cation exchange using fly ash (Çokça, 2001).

As the amount of fly ash increases, it is clear that a reduction in modified dry density & free swell index and an increase in optimum moisture content are observed. It can be concluded that in order to tackle with the problem of shrinkage, swelling and unequal settlement, the mixing fly ash in specific proportion with the expansive soil can be implemented (Malhotra & Naval, 2013).

Çokça (2001) states that changes in the physical properties and swelling potential of expansive soil due to the addition of fly ashes is a result of additional silt size particles to some extent and due to chemical reactions that cause immediate flocculation of clay particles.

There are, however, a wide variety of types of fly ash having different mechanical and chemical properties. Nelson and Miller (1992) say that for a specific

application, therefore, a comprehensive testing program would be needed to determine the design criteria necessary for fly ash stabilization.

2.8.1.4. Salt Stabilization

The cost common salts used in soil stabilization are sodium chloride and calcium chloride. The effect of sodium chloride on soil properties varies and generally has a greater effect in soils with a high liquid limit. Subject to the soil type, sodium chloride may have an additive effect on the shrinkage limit and shear strength. For soils reacting with calcium chloride, some beneficial control of frost heave may be realized (Nelson & Miller, 1992).

For expansive soils, low concentration of salt can reduce the swelling rate only by 20 % and 18 % for the swelling pressure. For a high concentration, reducing the swelling rate is around 60 % and 80 % for the swelling pressure. Moreover, we should note that for a low concentration, the reduction of swelling of expansive soil is about 40 % and about 70 % to swelling pressure, while for high concentration, reductions in swelling parameters are more important, it attains a reduction of more than 90% for the swelling pressure. It can be concluded that salt (NaCl) is more effective in reducing the swelling pressure as the rate of swell (Gueddouda, et al., 2011).

2.8.1.5. Cement Kiln Dust (CKD) Stabilization

CKD "is particulate matter that is collected from cement kiln exhaust gases and consists of entrained particles of clinker, unreacted and partially calcined raw materials, and fuel ash enriched with alkali sulfates, halides and other volatiles" (Sreekrishnavilasam, et al., 2006).

The chemical and physical properties of CKD can be influenced by several factors. Because plant operations differ considerably with respect to raw feed, type of operation, dust collection facility, and type of fuel used, the use of the terms typical or average CKD when comparing different plants can be misleading. The dust from each plant can vary markedly in chemical, mineralogical and physical composition (Keerthi, et al., 2013).

A number of CKDs and clay-type soils were used in order to study the soil stabilization. Soil- CKD mixes including 3, 8, and 10 % of CKD were tested for various engineering properties like the unconfined compressive strength, moisture-density relationship, liquid limits (LL), plastic limit (PL), plasticity index (PI), and shrinkage limit (Rahman, et al., 2011).

Compared to lime, the increase in CBR (California Bearing ratio) due to CKD was much greater; however, lime is proved to be better at reducing the PI, giving a PI reduction from 28 % to nearly 0 % at 5 % lime content. The available literature indicates that at given the proper conditions, CKD can be an effective soil stabilizer (Miller & Azadb, 2000).

It has been displayed that incorporation of the cement kiln dust induced extensive changes in the physico-chemical properties of Na-montmorillonite clay. The strength of CKD-treated and moist-compacted Na-montmorillonite clay specimens increased substantially even after one day, and progressively with further curing (Peethamparan, et al., 2009).

Ismaiel (2013) states that CKD having self-cementing characteristics reacts with soil in a manner similar to Portland cement. Typically, CKD has approximately one-third of the amount of cement oxides (CaO, Al_2O_3, SiO_2, and Fe_2O_3) present in Portland cement. The primary value of cement kiln dust (CKD) is its cementitious property. Depending on the concentration of free lime (CaO), CKD can be highly cementitious. Therefore, it can be used as a replacement for cements.

The formed cementitious compounds (Figure 2.16) obtained as a result of the chemical reactions between the silica and the alumina existing in the soil and the additives reduced the volume of the void spaces and participate in the soil particles (Ismaiel , 2013)

CaO + H_2O → [Hydration of Quick Lime] → $Ca(OH)_2$

$Ca(OH)_2$ → [Ionization of Quick Lime] → Ca^{+2} + $2(OH)^-$

Ca^{+2} + $2(OH)^-$ + Clay → [Adsorption and/or Cation Exchange / Pozzolanic reaction] → CSH + CAH+ CASH

$$Ca^{++} + 2(OH) + S_iO_2 \rightarrow CSH \text{ Calcium silicate hydrate}$$
$$\text{(Silica)} \qquad \text{(Gel)}$$
$$Ca^{++} + 2(OH) + Al_2O_3 \rightarrow CAH \text{ Calcium aluminumhydrate}$$
$$\text{(Alumina)} \qquad \text{(Fibrous)}$$
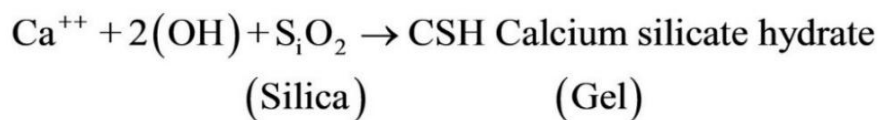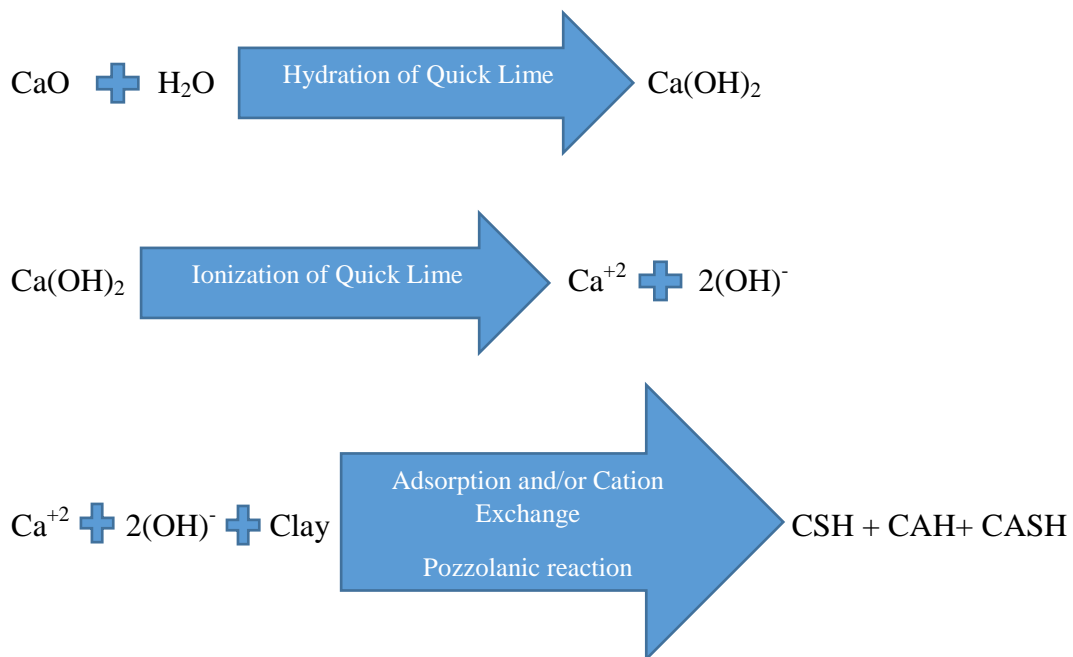
Figure 2.16. Hydration Products of Cement Kiln Dust (Ismaiel, 2013)

2.8.2. Pre-wetting

Pre-wetting is based on the theory that increasing the moisture content in the expansive foundation soils which will be the cause of heave to occur prior to construction and thereby eliminate problems afterward. This procedure may have serious drawbacks that hinder its application. Expansive soils typically exhibit low hydraulic conductivity and the time required for adequate wetting can be up to several years (Nelson & Miller, 1992).

2.8.3. Soil Removal and Replacement

In order to provide stable foundation material, removal of expansive soils and replacement with non-expansive soils can be used as a method. Backfill materials should be impermeable and preferably non-expansive (silts, clayey silts, silty clays, or some clays). Backfill material, particularly remolded in situ soil, should be replaced and compacted with careful moisture and density control (Ardani, 1992).

2.8.4. Moisture Control and Compaction

The swell potential plunges down as the compaction moisture content increases. However at specific compaction moisture content, the swell potential increases as the compaction energy increases (Rashid, et al., 2013).

2.8.5. Surcharge Loading

Swelling can be prevented by the fact that expansive clays can be loaded with a surcharge large enough to counteract the expected swell pressures. That can be generally applied only for soils with low to moderate swelling pressures (Nelson & Miller, 1992).

CHAPTER 3

EXPERIMENTAL STUDY

3.1. Purpose

The purpose of this study is to examine the improvement of an expansive soil by addition of cement kiln dust. In fact, for the expansive soil, material definition tests were performed and swelling potential was investigated. The effect of 0, 7 and 28 days of curing on the swelling potential was also considered. Improved properties of the expansive soil by adding cement kiln dust were compared with the samples which were obtained by the addition of lime, cement and sand.

3.2. Material

Kaolinite, bentonite, cement kiln dust, lime, cement and sand were used for the investigation.

Kaolinite: Kaolinite (Figure 3.2 (a)) was a production of Kalemaden Endüstriyel Hammaddeler Sanayi ve Ticaret A.Ş. The name of the provided kaolinite from the factory was Kaolinite 186. It was sieved through # 40 sieve before usage. Material properties of the kaolinite were listed in Table 3.1 below. Chemical composition of the kaolinite that obtained from supplier is given in Table 3.3.

Table 3.1. Material Properties of Kaolinite

Density, ρ (Mg/m³)	2.63
Liquid Limit, LL (%)	44
Plastic Limit, PL (%)	29
Plasticity Index, PI (%)	15

Bentonite: Bentonite (Figure 3.2 (b)) was a production of Karakaya Bentonit Sanayi ve Ticaret A.Ş. Bentonite was sieved through # 40 sieve before usage. Material properties of the bentonite were listed in Table 3.2 below. Chemical composition of the bentonite that obtained from supplier is given in Table 3.3.

Table 3.2. Material Properties of Bentonite

Density, ρ (Mg/m³)	2.53
Liquid Limit, LL (%)	403
Plastic Limit, PL (%)	35
Plasticity Index, PI (%)	367

Cement Kiln Dust: Cement kiln dust (Figure 3.2 (c)) was acquired from Baştaş Başkent Çimento Sanayi ve Ticaret A.Ş. It was sieved through # 40 sieve before usage. Density of the cement kiln dust was defined as 3.22 Mg/m³. Chemical analysis of cement kiln dust was performed by General Directorate of Mineral Research and Exploration (MTA). The results are presented in Table 3.3.

Lime: Lime (Figure 3.2 (d)) was obtained from Akyüz Kireç Sanayi Ltd. Şti. It was sieved through # 40 sieve before usage. Density of the lime was defined as 2.41 Mg/m³. Chemical composition of the lime that obtained from supplier is given in Table 3.3.

Cement: Cement (Figure 3.2 (e)) was obtained from Limak Çimento Sanayi ve Ticaret A.Ş. It was sieved through # 40 sieve before usage. Density of the cement was defined as 3.13 Mg/m³. Chemical composition of the cement obtained from supplier is given in Table 3.3.

Sand: Particle size distribution of the sand used in the study (Figure 3.2 (f)) was presented in Figure 3.1. It was sieved through # 40 sieve before usage. Density of sand was defined as 2.69 Mg/m³.
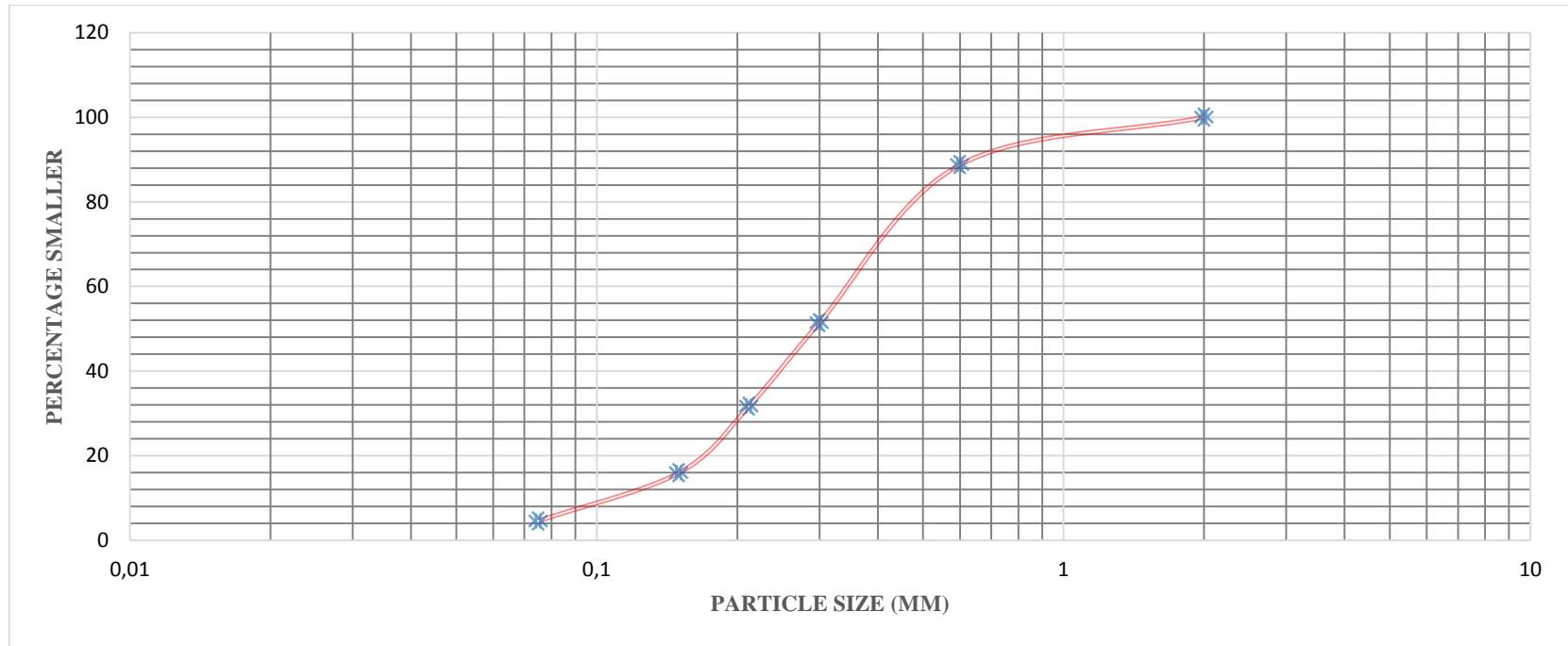
Figure 3.1. Particle Size Distribution Curve for Sand

D_{10}: The grain diameter at 10 % passing= 0.10 mm   Fines Content (<0.074 mm)= 4.59 %

D_{30}: The grain diameter at 30 % passing = 0.21 mm   USCS= SP (Poorly graded sands)

D_{60}: The grain diameter at 60 % passing = 0.35 mm

C_U: Coefficient of uniformity= 3.5

C_C: Coefficient of curvature= 1.26

(a) Kaolinite

(b) Bentonite
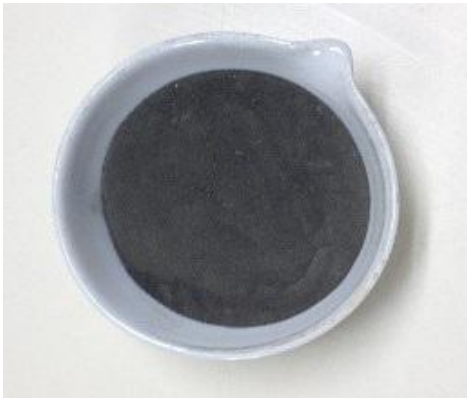
(c) Cement Kiln Dust

(d) Lime

(e) Cement

(f) Sand

Figure 3.2. Views of Materials Used in the Study

Table 3.3. Chemical Composition of Kaolinite, Bentonite, Cement Kiln Dust, Lime and Cement

Composition (%)	Kaolinite	Bentonite	Cement Kiln Dust	Lime	Cement
SiO_2	47.89	61.28	19.70	< 0.10	21.00
Al_2O_3	36.75	17.79	3.90	< 0.10	5.30
TiO_2	0.61	*	0.20	*	0.31
Fe_2O_3	0.40	3.01	3.20	< 0.10	3.30
CaO	0.39	4.54	66.50	95.55	65.60
MgO	0.09	2.10	1.20	0.90	1.10
Na_2O	0.01	2.70	0.70	*	0.03
K_2O	0.75	1.24	1.30	*	0.79
SO_4	0.37	*	*	*	*
P_2O_5	*	*	0.10	*	0.09
MnO	*	*	< 0.10	*	0.05
Loss on Ignition	12.73	*	0.40	3.20	0.90

*Not determined

The chemical compositions of kaolinite, bentonite, lime, and cement were provided from the suppliers. Chemical composition of kaolinite was obtained from Kalemaden Endüstriyel Hammaddeler Sanayi ve Ticaret A.Ş. For bentonite, it was provided from Karakaya Bentonit Sanayi ve Ticaret A.Ş. For lime, it was obtained from Akyüz Kireç Sanayi Ltd. Şti. and finally the chemical composition of cement was provided from Limak Çimento Sanayi ve Ticaret A.Ş.

3.3. X-Ray Diffraction Analysis

X-Ray diffraction analysis was performed by General Directorate of Mineral Research and Exploration (MTA). The results are presented in Table 3.4 and Figure 3.3.

Table 3.4. Mineral Composition of the Cement Kiln Dust

Mineral Name
Hatrurite
Grossular
Amorphous Material
Clay Group Mineral
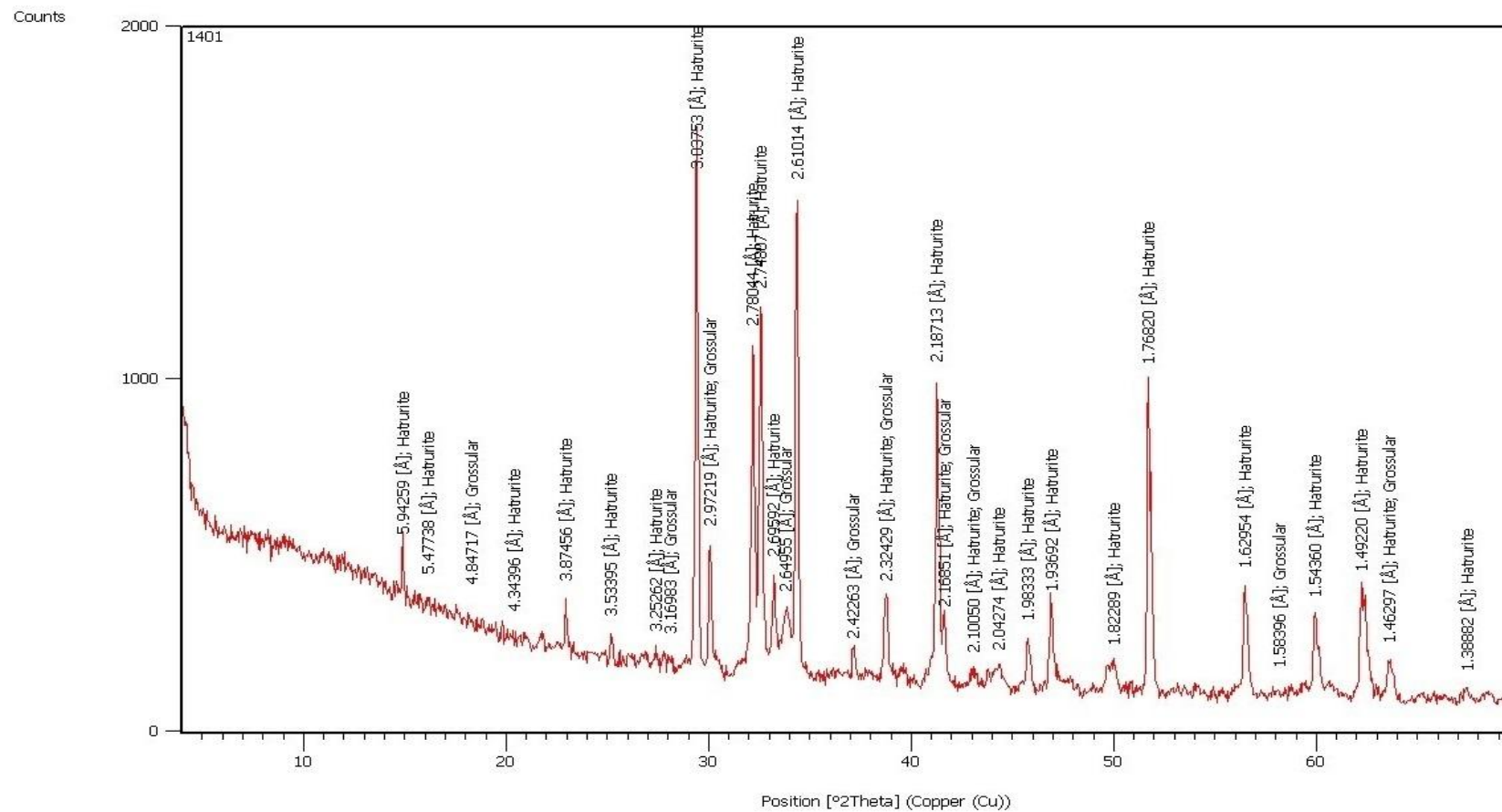
Figure 3.3. X-Ray Diffraction Patterns of Cement Kiln Dust

3.4. Determination of Free Lime Content of Stabilizers

Pozzolonic activity of a stabilizer is one of the main effect which decreases swell potential of expansive soils. Moreover, free lime content ($Ca(OH)_2$) of the stabilizer is the main factor determining the pozzolonic activity.

Tests were conducted according to ASTM C 25 (2011) (Standard Test Methods for Chemical Analysis of Limestone, Quicklime and Hydrated Lime). The process of the test is explained below;

40 g sugar was dissolved in 100 ml CO_2-free water in order to obtain sucrose solution. Then, a few drops of 4 % phenolphthalein indicator and 0.1 N sodium hydroxide was added to this mixture until pink color was observed.

The next step was stirring 2.804 g of sample with 40 ml CO_2-free water and 100 ml sucrose solution.

For chemical reactions, by swirling at 5 minutes intervals, the mixture was left to stay for 15 minutes.

4 - 5 drops of 4% phenolphthalein indicator was mixed to mixture after 15 minutes.

Finally, mixture was titrated with hydrochloric acid until the pink color got lost which takes 3 seconds.

Available lime content (%) = (N x V x 3.704) / W

N= 1 for this study (normality of acid solution)

V= (1.0 N), ml (standard hydrochloric acid)

W= 2.804 g in this study

The percentages of available lime contents for cement kiln dust, lime and cement determined from the specified method and formula as 1.6 %, 38 %, and 5.8 % respectively.

3.5. Preparation of Test Samples

The expansive soil was constituted artificially in laboratory by mixing kaolinite and bentonite with a certain amount of water. The artificial expansive soil, which was stated as Sample 1 in this study, comprised of 85 % kaolinite and 15 % bentonite by dry mass. These percentages were selected because the appropriate swell percentage for the study was obtained with these amount of materials.

Firstly, all materials, kaolinite, bentonite, cement kiln dust, lime, cement, and sand, used in the study were placed in an oven at 50 ℃ for 24 hours to ensure complete dryness. The pulverization step followed the drying. Pulverization was provided by a plastic hammer. The materials were passed through # 40 sieve before dry mixing. After mixing, the dry mixture was passed again through # 40 sieve in order to obtain a homogeneous mixture.

For performing swelling potential, the required amount of compacted Sample 1 for mold was determined by considering the maximum dry unit weight which was determined as 1.54 Mg/m³. The maximum dry unit weight and optimum moisture content of Sample 1 were calculated from standard proctor test (Figure 3.4). In order to be realistic, 95 % compaction was assumed. According to this assumption, the dry mixture amount was calculated as 88.03 gr. From Figure 3.4, the amount of water added to Sample 1 was 27 % of mass of dry mixture. Preparation process of the Sample 1 and treated samples are shown by the help of a flow chart in Figure 3.5.
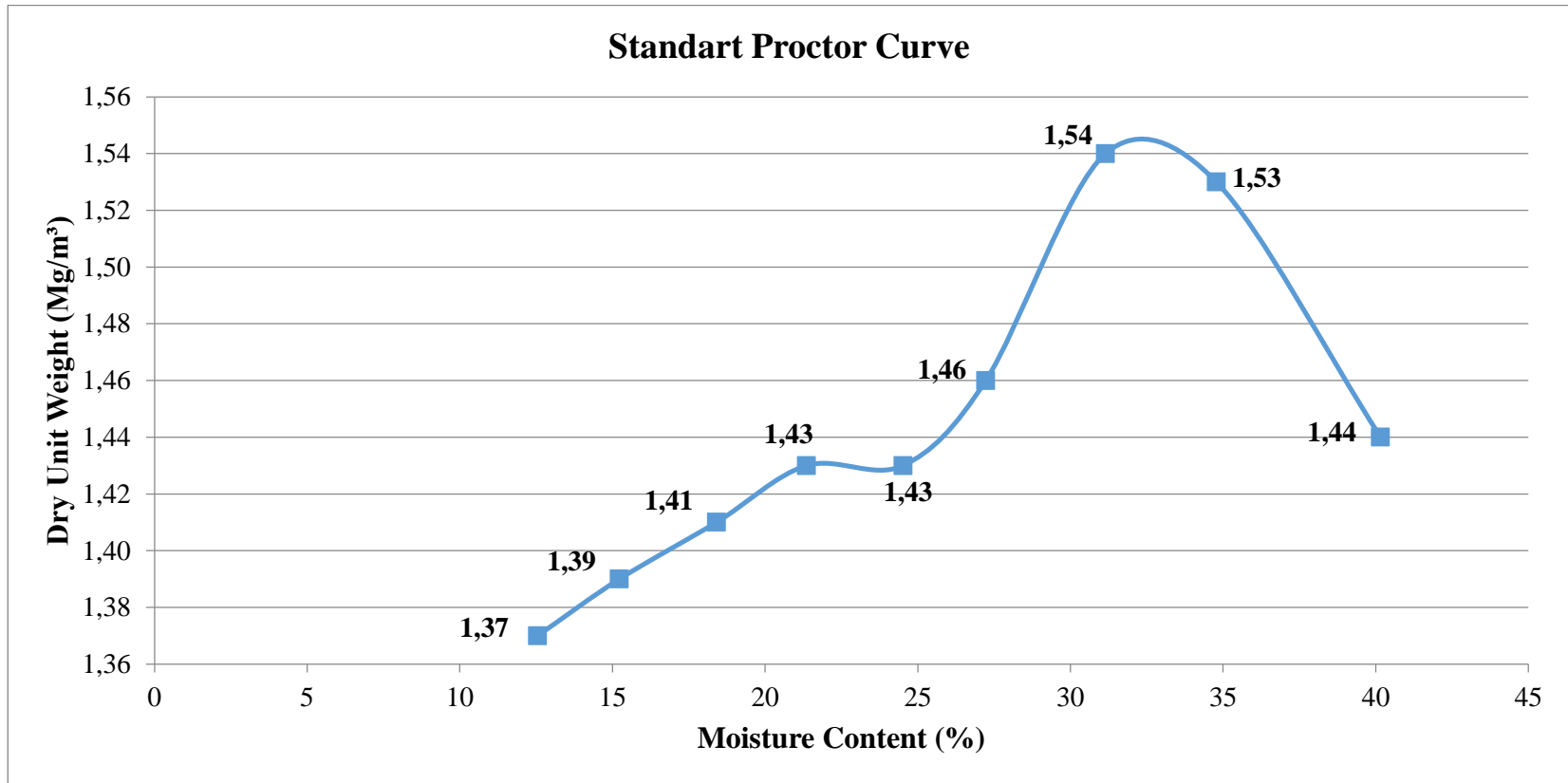
Figure 3.4. Compaction Curve for Sample 1

For curing, the prepared specimens were placed in a plastic bag and then a completely air-tightness was provided. They are allowed to cure at 22 $^{\circ}$C and 70% relative humidity for periods of 7 and 28 days.

Percentages of the prepared samples for the study are tabulated in Table 3.5.

Table 3.5. Composition of Prepared Specimens by Dry Weight

Sample	Kaolinite+ Bentonite (%)	Cement Kiln Dust (%)	Lime (%)	Cement (%)	Sand (%)
Sample 1	100	-	-	-	-
3% CKD	97	3	-	-	-
6% CKD	94	6	-	-	-
9% CKD	91	9	-	-	-
12% CKD	88	12	-	-	-
3% L	97	-	3	-	-
6% L	94	-	6	-	-
9% L	91	-	9	-	-
3% C	97	-	-	3	-
6% C	94	-	-	6	-
9% C	91	-	-	9	-
3% S	97	-	-	-	3
6% S	94	-	-	-	6
9% S	91	-	-	-	9
12% S	88	-	-	-	12
15% S	85	-	-	-	15

At the first stages of the study, the preliminary tests were done for Sample 1 and cement kiln dust treated sample in order to see the level of effect of cement kiln dust on the stabilization of Sample 1. After the preliminary tests, the results indicated that cement kiln dust could be used as a stabilizer for Sample 1.
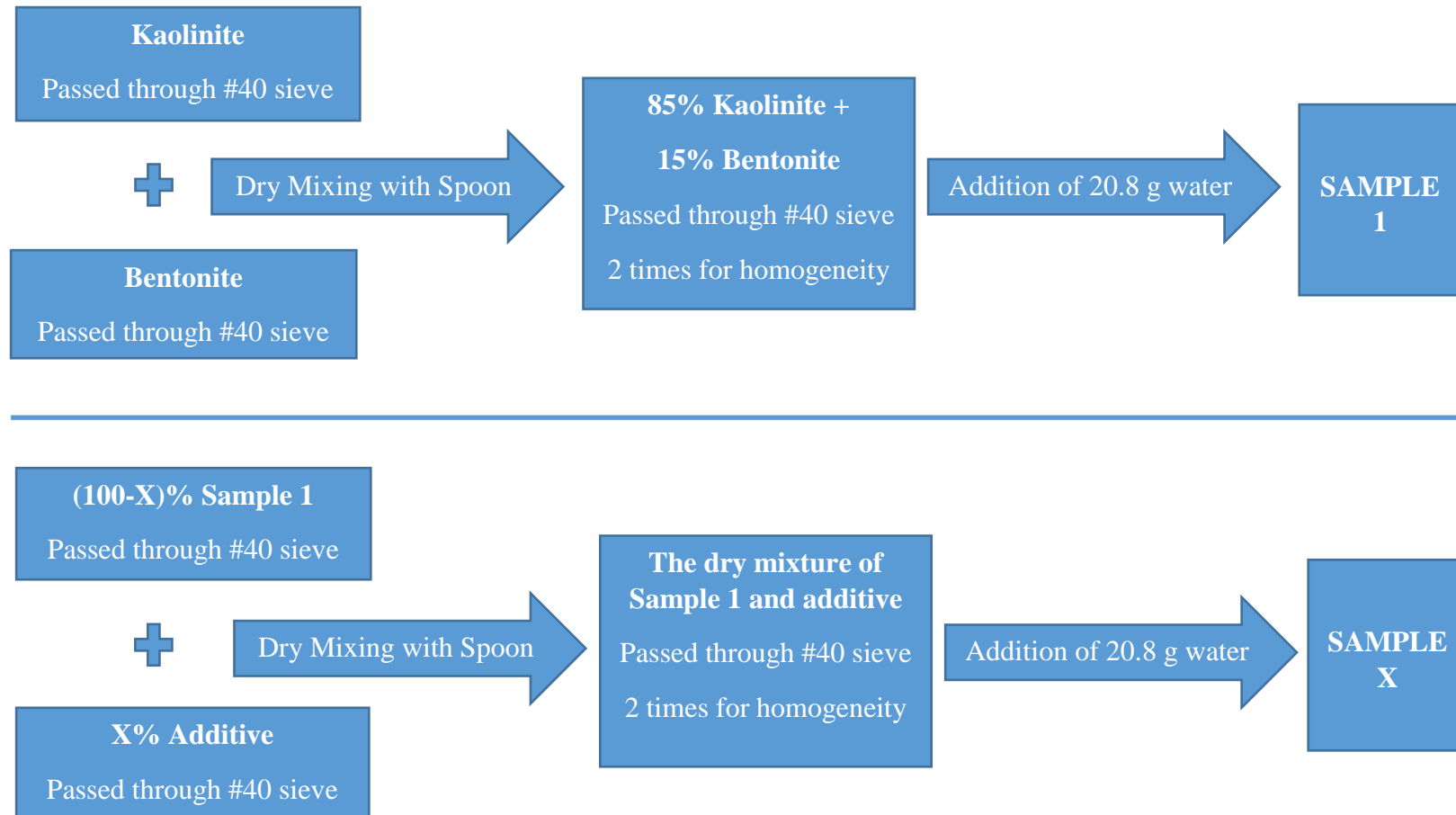
Figure 3.5. Preparation of Samples

3.6. Properties of Samples

In order to determine the material properties of the samples, standard proctor test, Atterberg limits tests (liquid limit (LL), plastic limit (PL), plasticity index (PI), shrinkage limit (SL), shrinkage index (SI)), hydrometer tests, sieve analysis tests and density tests were conducted. Standard proctor test was performed according to ASTM D 698-12 (2012). Moreover, liquid limit, plastic limit and plasticity index were determined according to ASTM D4318 (2010). ASTM C356-10 (2010) was used for shrinkage limit which was calculated by considering linear shrinkage. Sieve analysis and hydrometer tests were performed according to ASTM D422 (2007).

Hydrometer tests in order to define the grain size distribution curves for cement kiln dust, lime, cement and sand could not be conducted due to rapid settling. On the other hand, this problem was not encountered for the treated samples except cement treated samples. Despite trying for three times, chemical reactions and flocculation after approximately one hour prevented to perform hydrometer test for cement treated samples. Due to these obstacles, the tests could not be conducted. Especially, the flocculation could be seen clearly for 6 % and 9 % cement treated samples (Figure 3.6).

| 3 % | 6 % | 9 % |
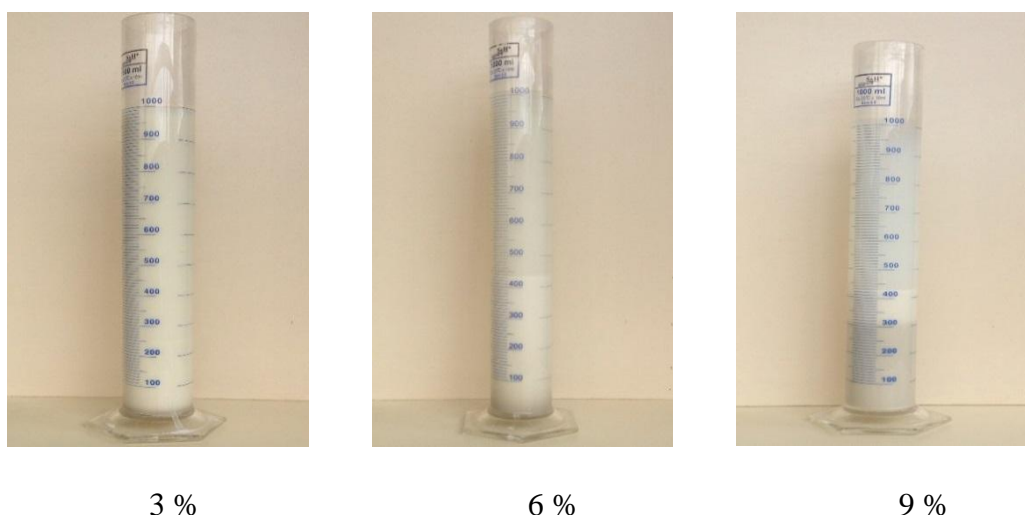
Figure 3.6. A View from Hydrometer Test for Cement Treated Samples

Liquid limits were determined by two different methods, namely Casagrande and fall cone test methods. The purposes of this approach were to compare the two methods and to ensure the accuracy of the results. The mostly used method for determination of liquid limit is Casagrande method. However, the results obtained by the Casagrande Apparatus method are very dependent on the operator, thus it is hard to replicate the results when the same test is carried out on the same material (Grønbech, et al., 2011). On the other hand, the Fall Cone is also widely known throughout the world, and the accepted standard for finding the liquid limit in many countries. The liquid limit is defined as the moisture content of the clay at which the cone penetrates 20 mm from its original position in 5±0.5 sec (Haulsbay, 1982). The Fall Cone has the advantage over the Casagrande apparatus that the operation of the apparatus is not affected by the operator, and the results are thereby comparable independent of the user. When using the Fall Cone Apparatus, one should be aware of the state of the cone, since a worn cone can affect the fall depth, and thereby the results of the Liquid Limit. The soil should be compacted carefully because air pockets trapped in the soil around the point of impact can also influence the measured fall depth. Fall cone test method was conducted according to BS 1377 (2010) (Figure 3.7).
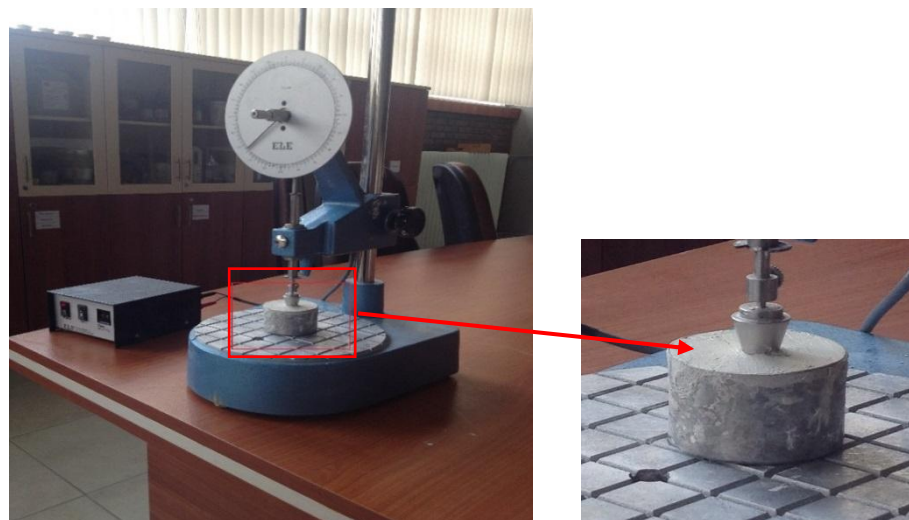
Figure 3.7. A View of Fall Cone Test

Specific gravity tests for the samples could not be performed because the samples were frozen due to the chemical reactions while the test was being conducted. Therefore density of the additives and the samples were determined according to TS EN 196-2 (2013). Le Chatelier bottle was filled with gas oil to the point which was between 0 to 1 millimeters. The bottle was placed in a water pool at room temperature. The bottle was remained in the pool until it reached to the room temperature. Then, volume (V_1) was read. After reading, the sample was added to the gas oil filled bottle through funnel carefully. Cover of the bottle was placed and it was rotated on its own axes (Figure 3.8 (a)) in order to provide well mixture and avoid from air bubbles. Finally, it was again placed in to the room temperature water pool (Figure 3.8 (b)) for approximately 1 hour, then, volume (V_2) was read.

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

where m: Mass of the sample (gram)

$\qquad$ V_1: First read volume (cm^3)

$\qquad$ V_2: Last read volume (cm^3)

$\qquad\qquad$ (a) $\qquad\qquad\qquad\qquad\qquad\qquad\qquad$ (b)

Figure 3.8. A View of Density Test

Unified Soil Classification System (USCS) was used in order to classify the soils. Definition of soil classes were done by considering plasticity chart by using liquid limit and plasticity index values of samples (Figure 3.9).

Prediction of swell potentials were done by using classification chart recommended by Seed et al. (1962) (Figure 3.10). In that figure, the number obtained by dividing plasticity index values (PI) to clay size fraction represents the activity. Moreover, swell potentials of the samples were also predicted by chart recommended by Van Der Merwe (1964) (Figure 3.11).

Grain Size distribution curves for cement kiln dust, lime, and sand treated samples were shown in Figure 3.12, 3.13 and 3.14, respectively.

Improvement effects of stabilizers on density, liquid limit, plastic limit, plasticity index, shrinkage limit, and shrinkage index are represented in Figure 3.15, 3.16, 3.17, 3.18, 3.19, and 3.20, respectively.

Properties of samples are tabulated in Table 3.6.
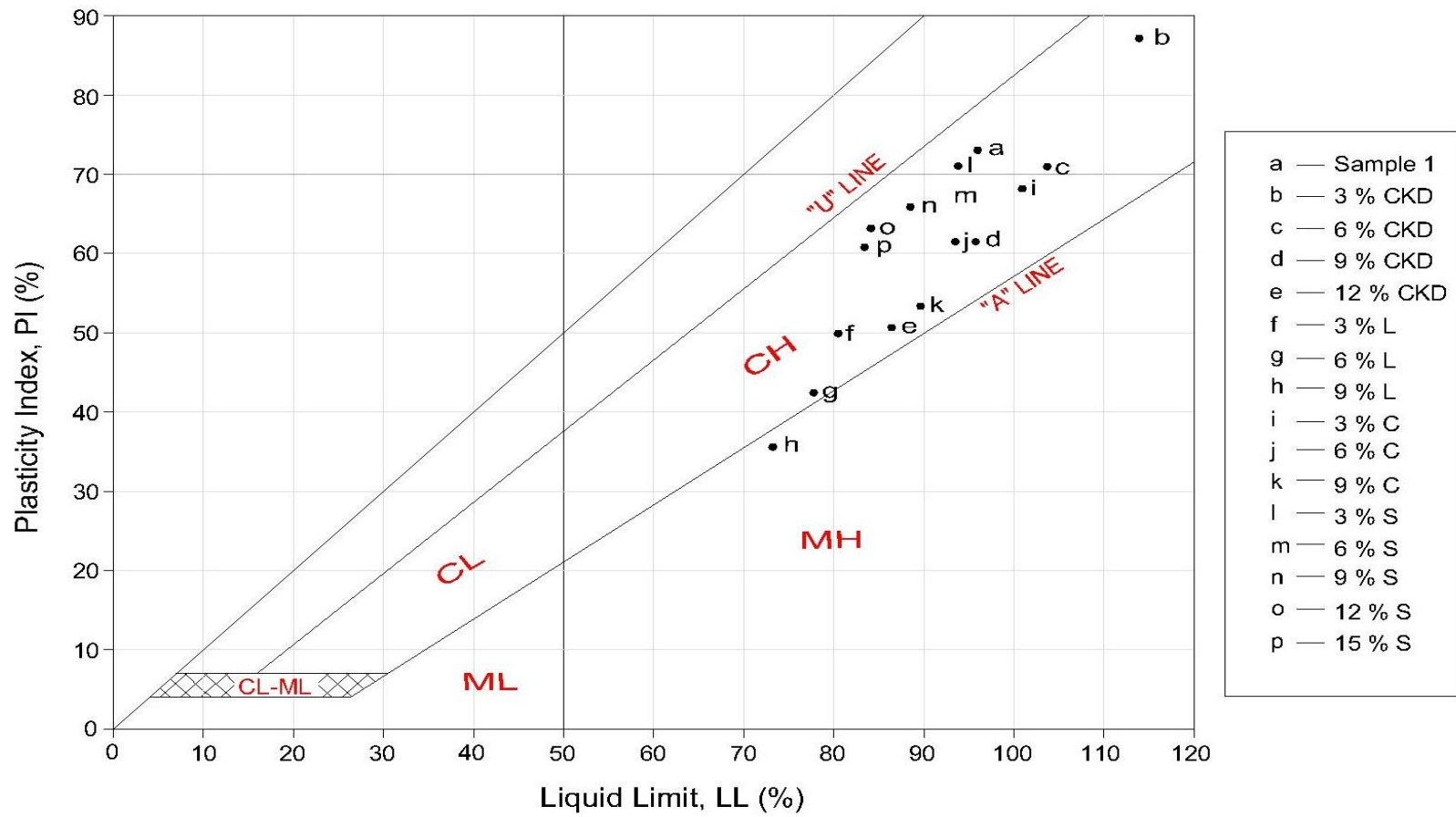
Figure 3.9. Plasticity Chart
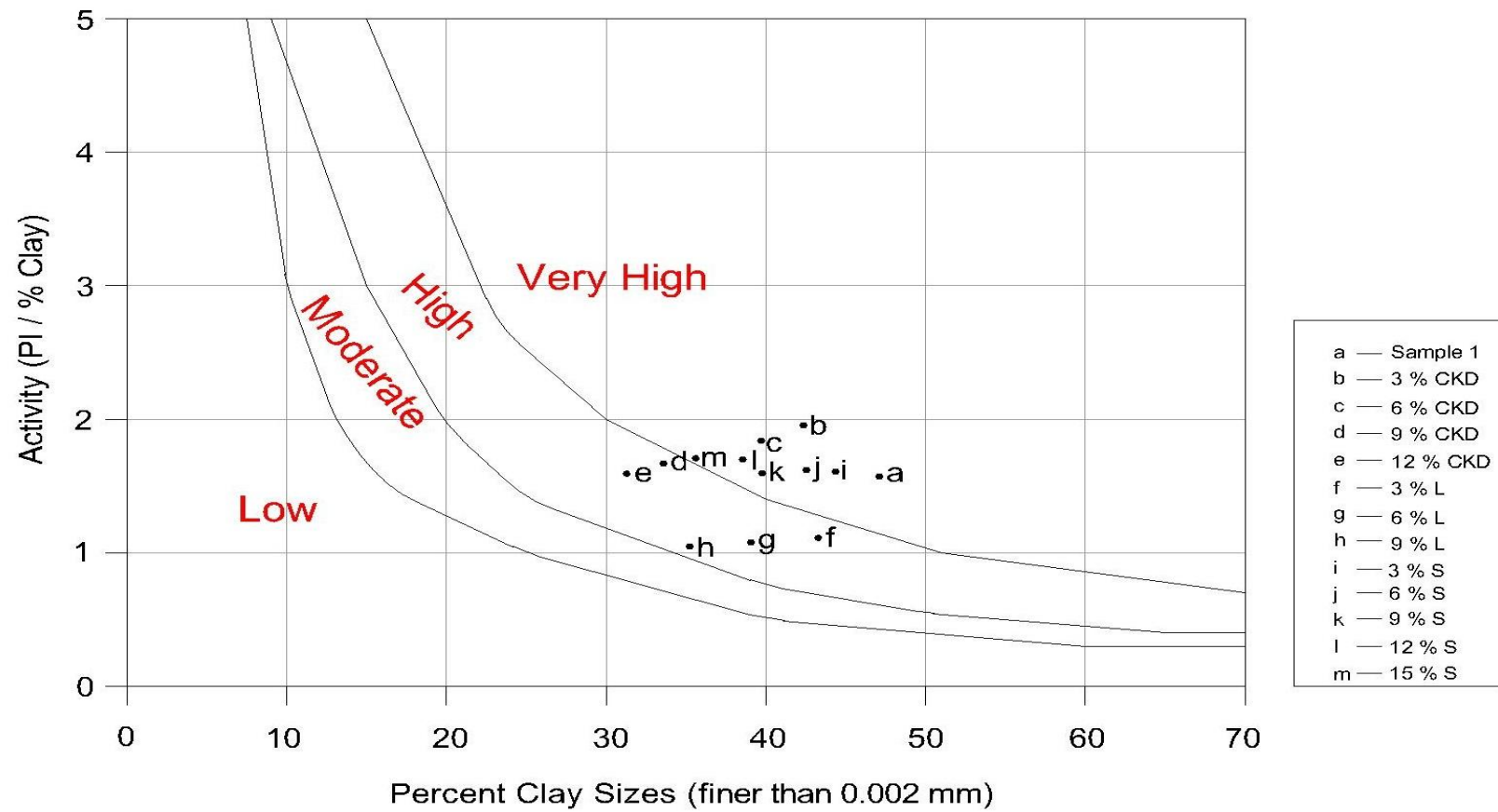
Figure 3.10. Classification Chart for Swelling Potential (After Seed, et al., 1962)
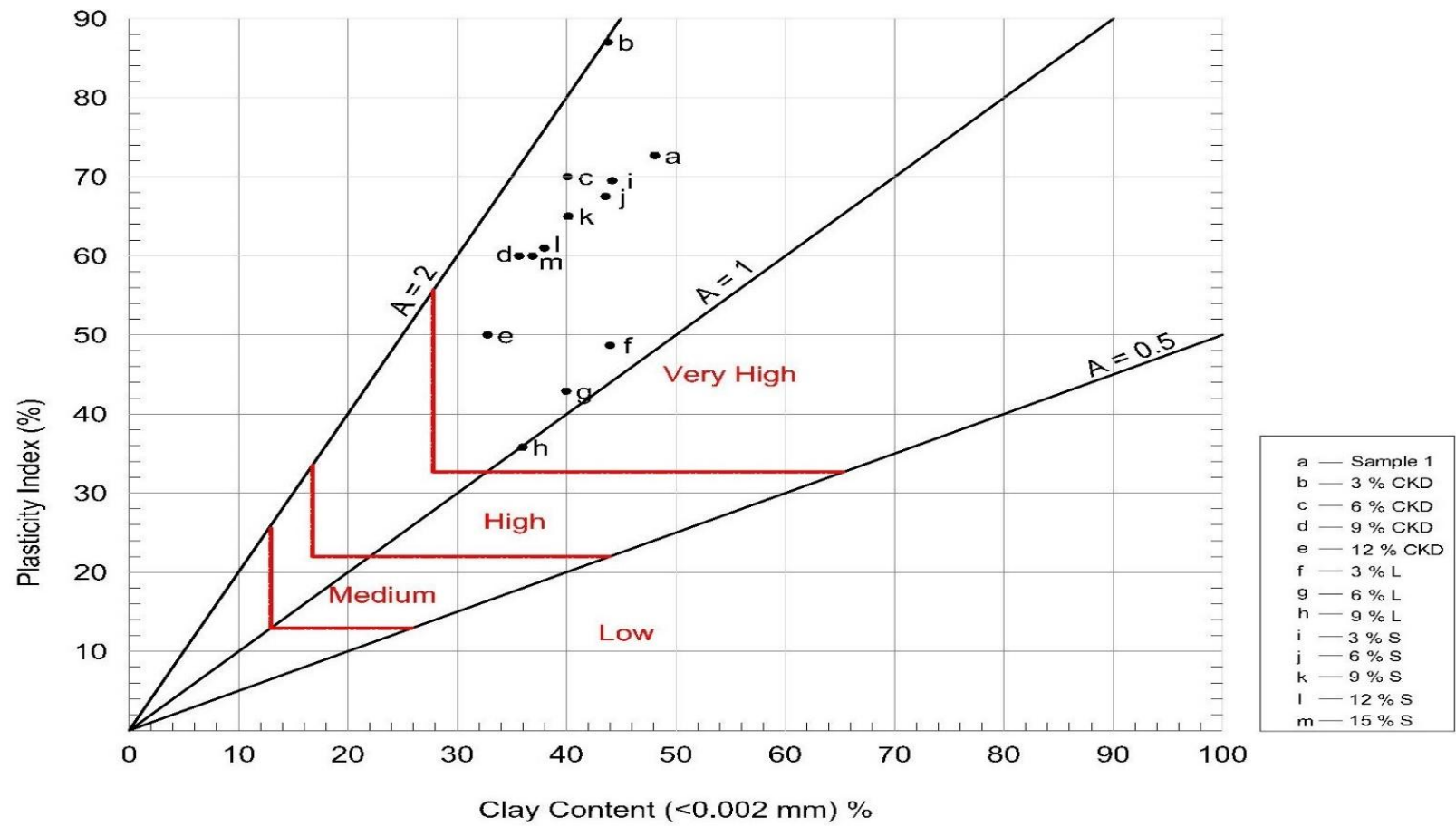
Figure 3.11. Classification Chart for Swelling Potential (After Van Der Merwe, 1964)
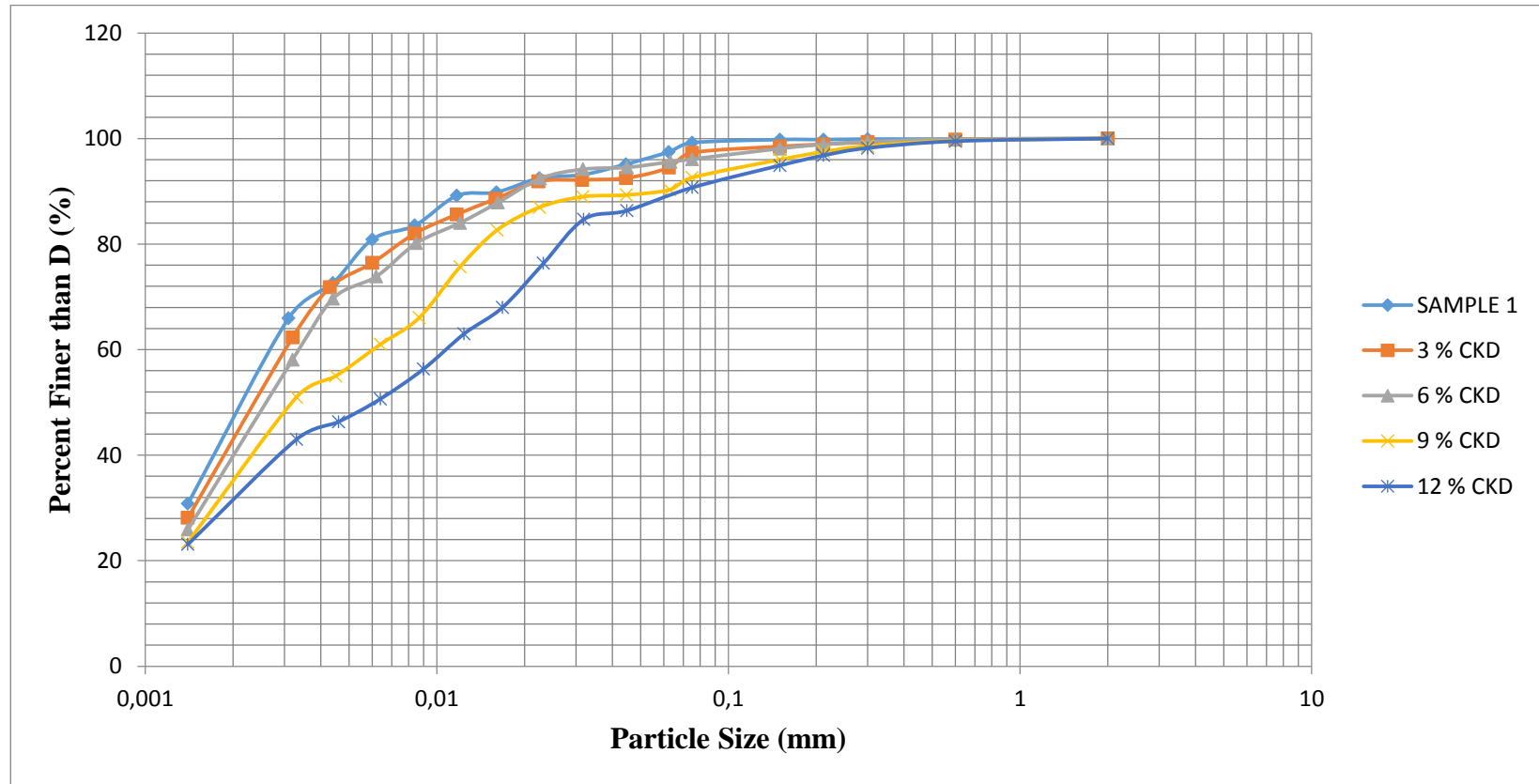
Figure 3.12. Grain Size Distribution Curves for Sample 1 and Cement Kiln Dust Treated Samples
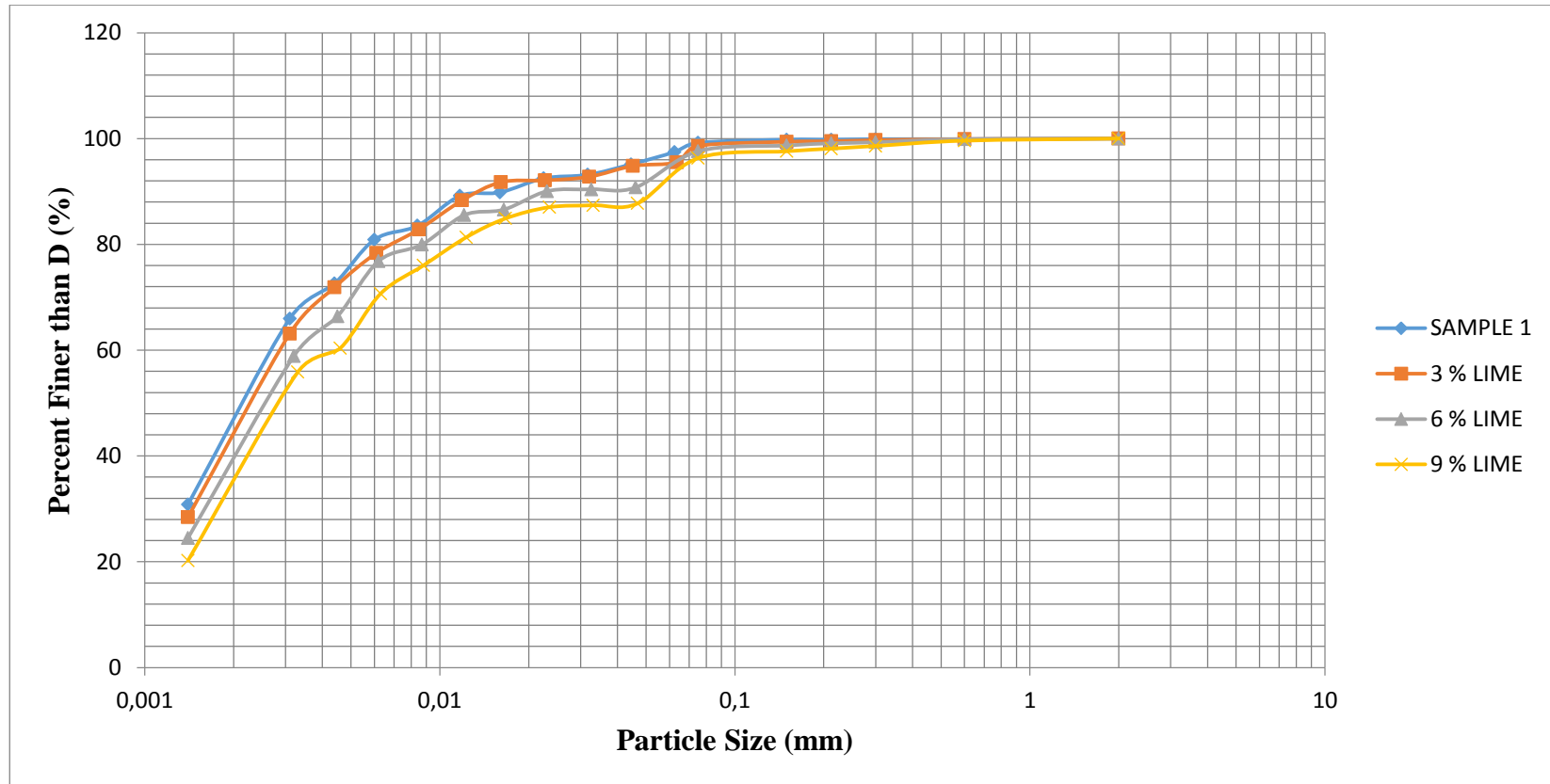
Figure 3.13. Grain Size Distribution Curves for Sample 1 and Lime Treated Samples
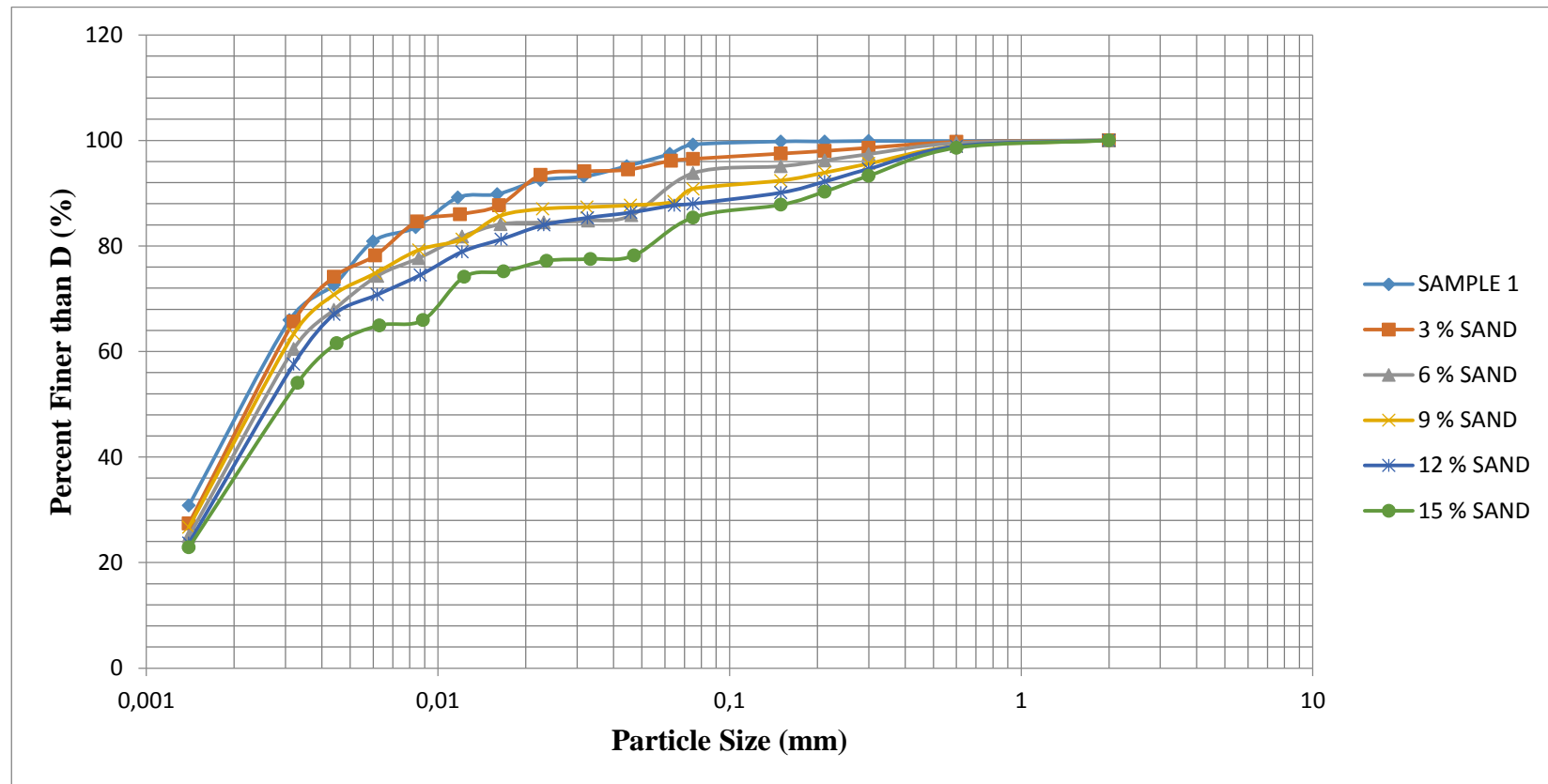
Figure 3.14. Grain Size Distribution Curves for Sample 1 and Sand Treated Sample
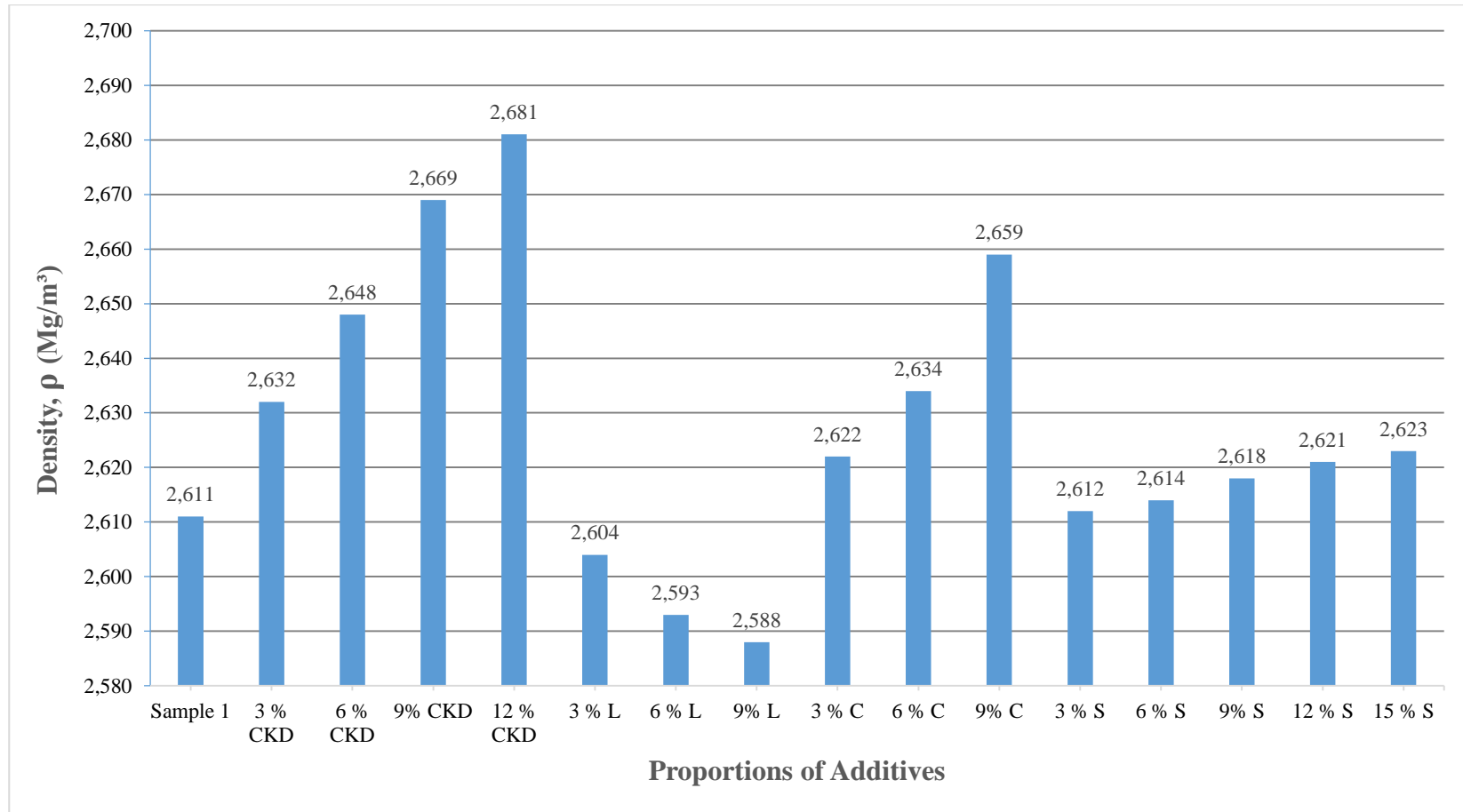
Figure 3.15. Effect of Addition of Cement Kiln Dust, Lime, Cement and Sand on Density (ρ) of the Sample
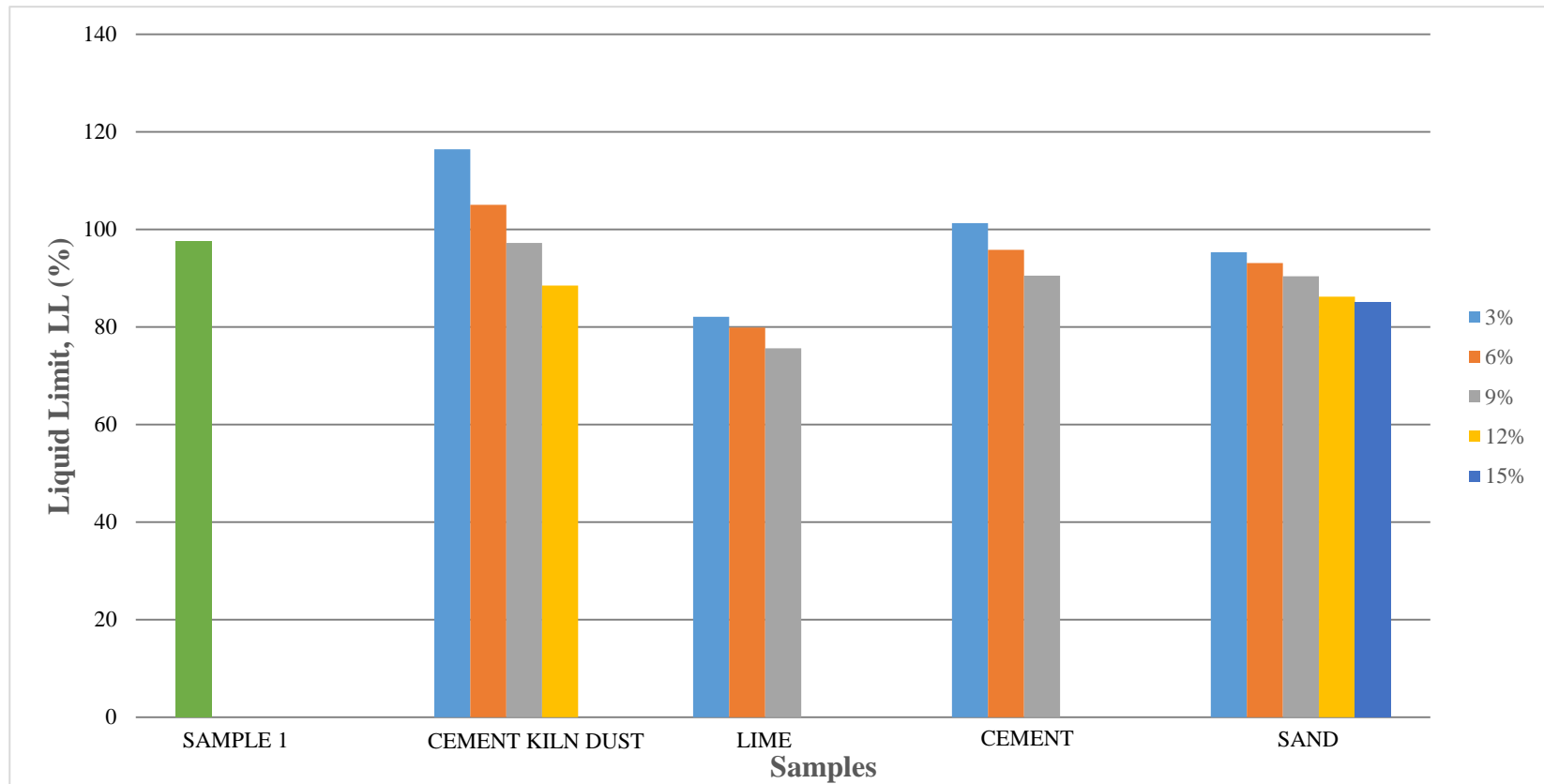
Figure 3.16. Effect of Addition of Cement Kiln Dust, Lime, Cement and Sand on Liquid Limit (LL) of the Samples
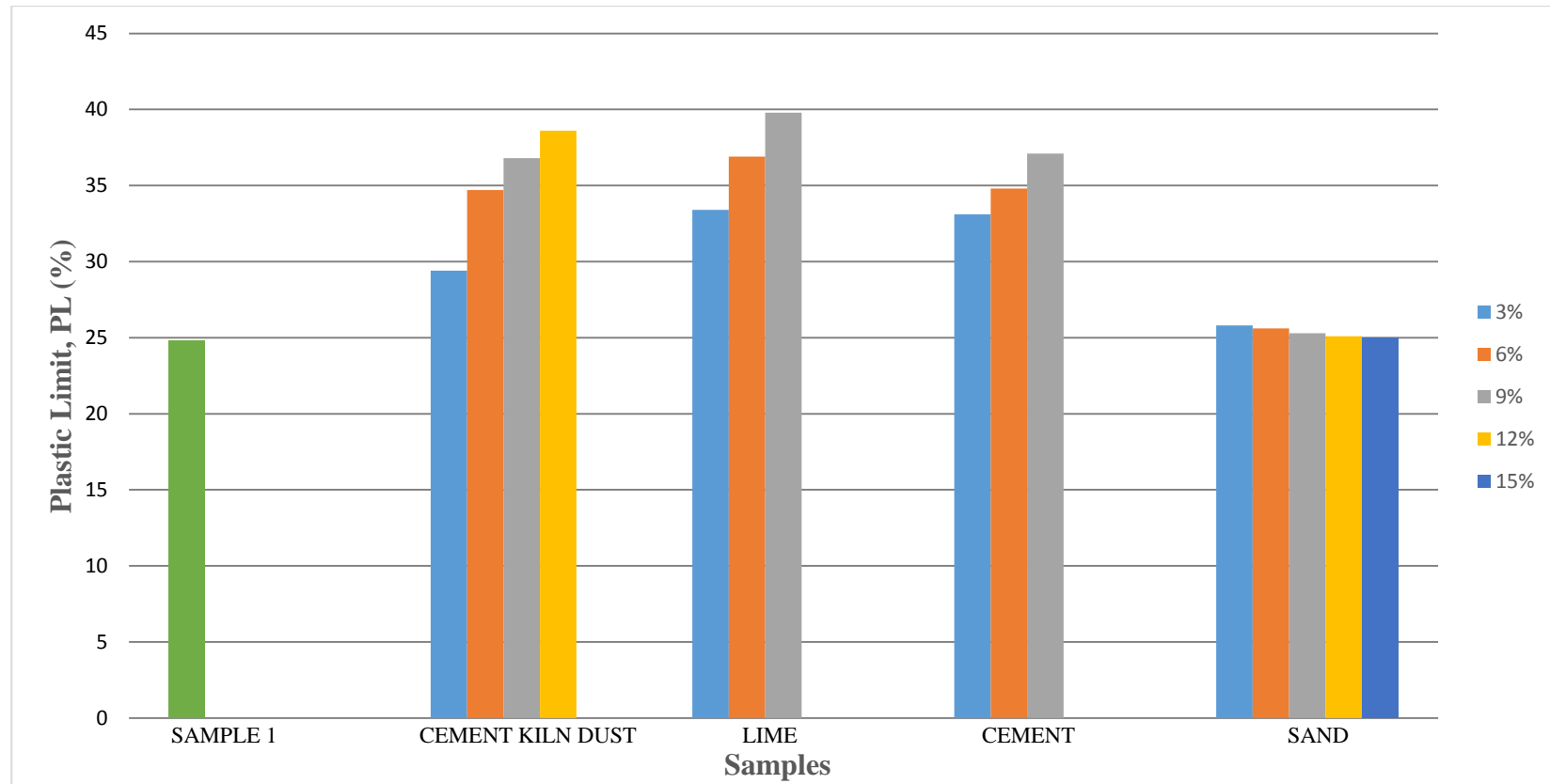
Figure 3.17. Effect of Addition of Cement Kiln Dust, Lime, Cement and Sand on Plastic Limit (PL) of the Samples
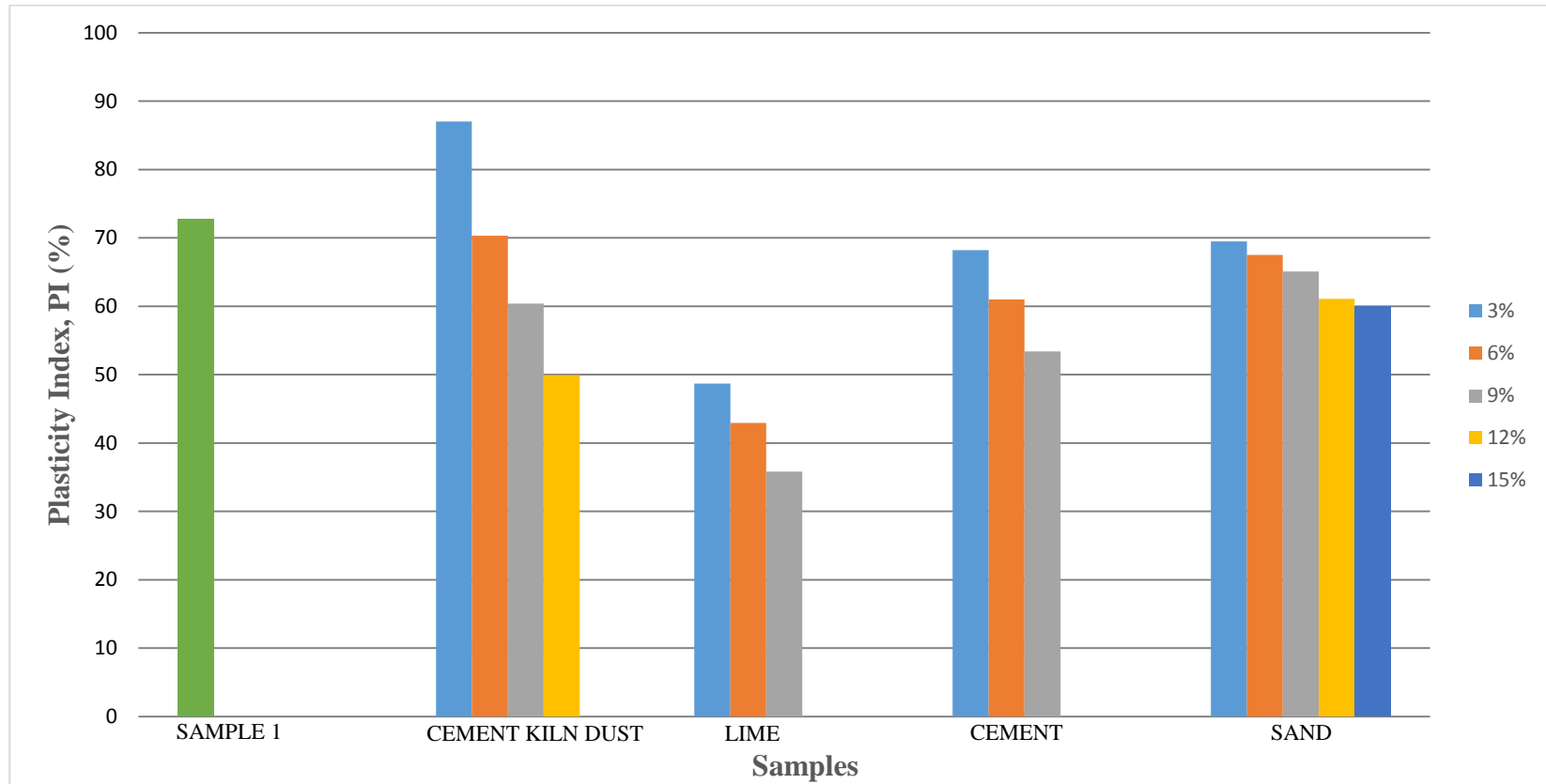
Figure 3.18. Effect of Addition of Cement Kiln Dust, Lime, Cement and Sand on Plasticity Index (PI) of the Samples
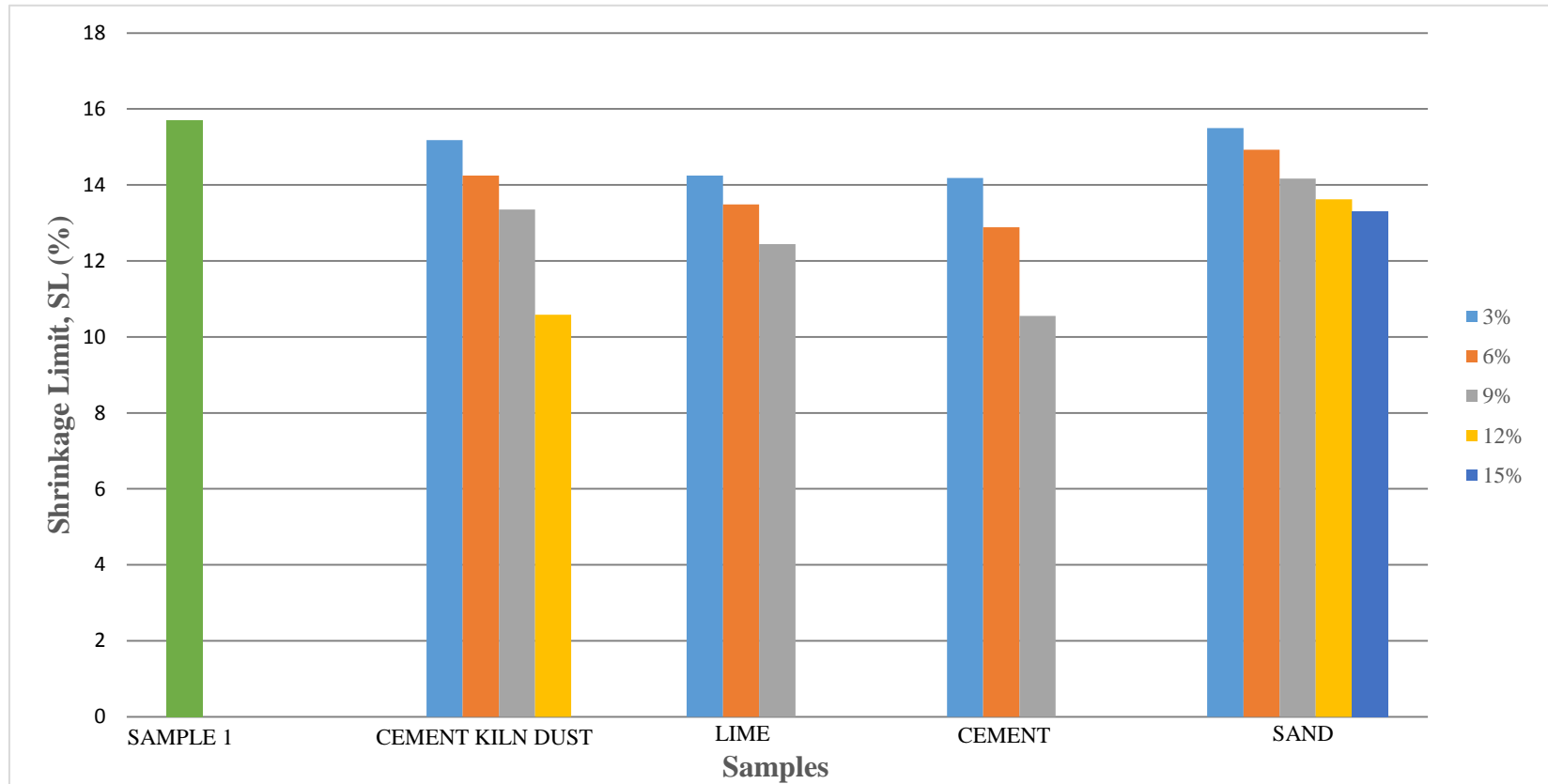
Figure 3.19. Effect of Addition of Cement Kiln Dust, Lime, Cement and Sand on Shrinkage Limit (SL) of the Samples
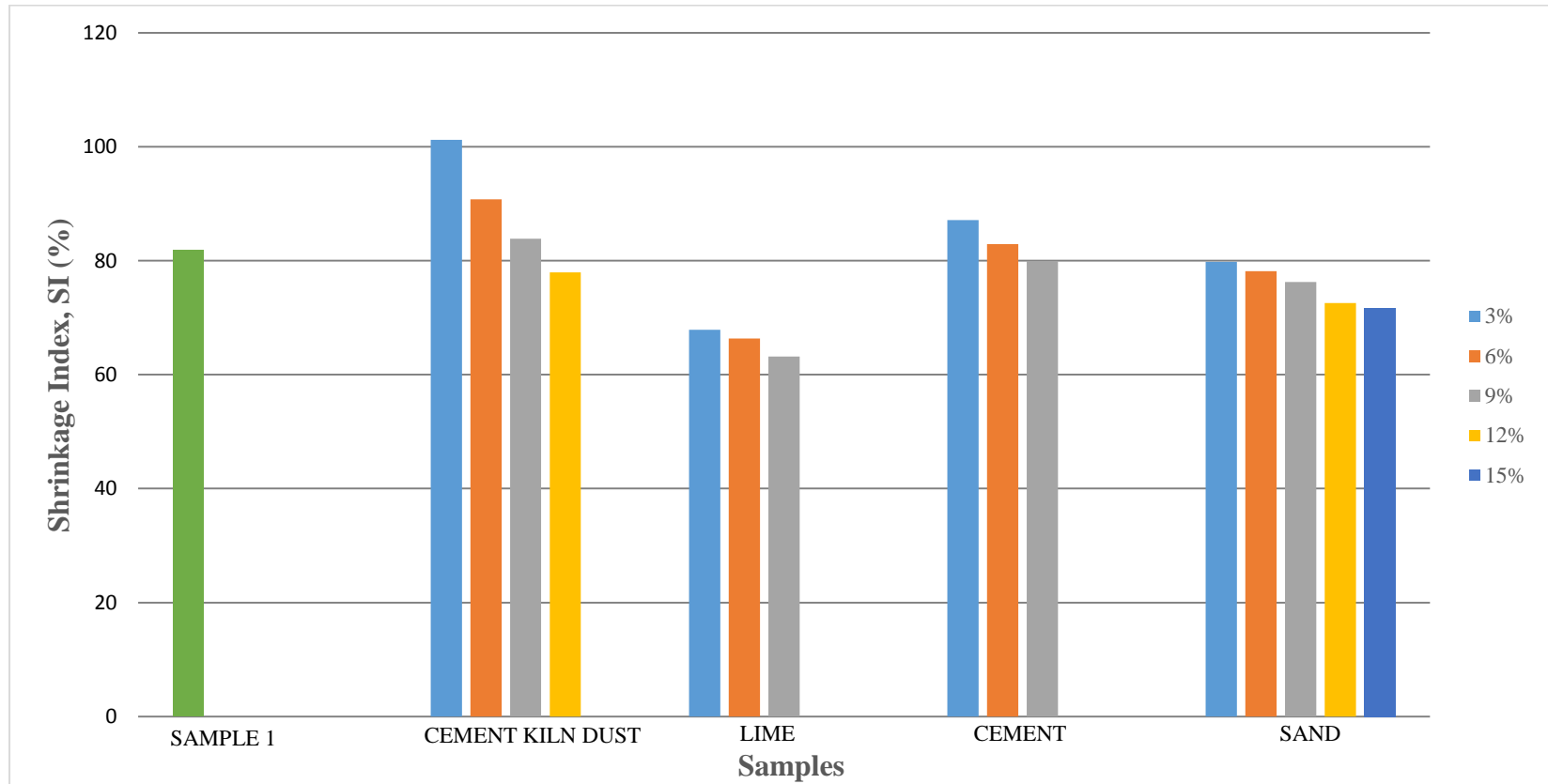
Figure 3.20. Effect of Addition of Cement Kiln Dust, Lime, Cement and Sand on Shrinkage Index (SI) of the Sample

Table 3.6. Properties of Samples

Sample	Clay (%)	Density (Mg/m³)	LL (%)	PL (%)	PI (%)	SL (%)	SI (%)	UCSC	Activity	Swelling Potential (Seed et al.)	Swelling Potential (Van Der Merwe)
Sample 1	48.10	2.611	97.50	24.80	72.70	15.70	81.80	CH	1.51	Very High	Very High
3 % CKD	43.80	2.632	116.40	29.40	87.00	15.18	101.22	CH	1.98	Very High	Very High
6 % CKD	40.10	2.648	105.00	34.70	70.30	14.25	90.75	CH	1.75	Very High	Very High
9 % CKD	35.70	2.669	97.20	36.80	60.40	13.35	83.85	CH	1.69	High	Very High
12 % CKD	32.80	2.681	88.50	38.60	49.90	10.58	77.92	CH	1.52	High	Very High
3 % L	44.00	2.604	82.10	33.40	48.70	14.25	67.85	CH	1.11	High	Very High
6 % L	39.95	2.593	79.84	36.90	42.94	13.49	66.35	CH-MH	1.07	High	Very High
9 % L	36.10	2.588	75.60	39.80	35.80	12.44	63.16	MH	0.99	High	Very High
3 % C	-	2.622	101.30	33.10	68.20	14.18	87.12	CH	-	-	-
6 % C	-	2.634	95.80	34.80	61.00	12.89	82.91	CH	-	-	-
9 % C	-	2.659	90.50	37.10	53.40	10.55	79.95	CH	-	-	-
3 % S	44.20	2.612	95.30	25.80	69.50	15.50	79.80	CH	1.57	Very High	Very High
6 % S	43.60	2.614	93.10	25.60	67.50	14.93	78.17	CH	1.55	Very High	Very High
9 % S	40.20	2.618	90.40	25.30	65.10	14.17	76.23	CH	1.62	Very High	Very High
12 % S	38.10	2.621	86.20	25.10	61.10	13.62	72.58	CH	1.60	Very High	Very High
15 % S	36.00	2.623	85.00	25.00	60.00	13.30	71.70	CH	1.67	Very High	Very High

3.7. Test Procedure for Free Swell Test Using Oedemeter

3.7.1. Compaction of Specimen

Samples were compacted with a dry density of 1.46 Mg/m³ which is determined by assuming 95% of compaction (Figure 3.4). The compaction process was done by the help of a hydraulic jack (Figure 3.21). The samples are compacted in a mold having a 19 mm height and 63.5 mm diameter (Figure 3.22). According to dry density and the dimensions of mold, the required amount was calculated.

Figure 3.21. A View of Static Compaction by the Help of Hydraulic Jack

Figure 3.22. A View from of the Specimen after the Compaction Process

3.7.2. Free Swell Test Using Bishop Apparatus

Firstly, free swell tests were conducted by simple oedometer test apparatus (Figure 3.23 (a)). For comparison, Bishop Apparatus was used (Figure 3.23 (b)). When the results of the two apparatus were compared, it was clear that Bishop Apparatus gave more accurate results than the simple oedemeter apparatus. Then, free swell tests were continued by using Bishop Apparatus.

Free swell method was used to determine the effectiveness of the cement kiln dust, lime, cement and sand on the swell properties of the Sample 1. Compacted samples with consolidation ring were placed into the Bishop apparatus after placing the filter paper on top and bottom of it. Air-dried porous stone plates were also placed after the filter paper (Figure 3.24). Then, the dial gauge measuring the vertical deflection was set to zero and, 6.9 kPa load was applied. At the same time with loading, the consolidation ring was filled with water.

Increments in the dial gauge were read for 0.25 min, 1 min, 2.25 min, 4 min, 6.25 min, 9 min, 12.25 min, 16 min, 20.25 min, 25 min, 36 min, 49 min, 64 min, 81 min, 100 min, 121 min, 144 min, 1 day, 2 days, 3 days and 4 days in order to plot swelling percentage and time graphs.

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After the completion of swelling the last reading was recorded. Finally, the swelling percentages were determined according to ASTM D4546 (2008).

Free Swell Percentage = (Δh / h)*100

where Δh: Change in height of the specimen

h: Initial height of the specimen

(a)   (b)

Figure 3.23. A View of (a) Simple Oedemeter and (b) Bishop Apparatus
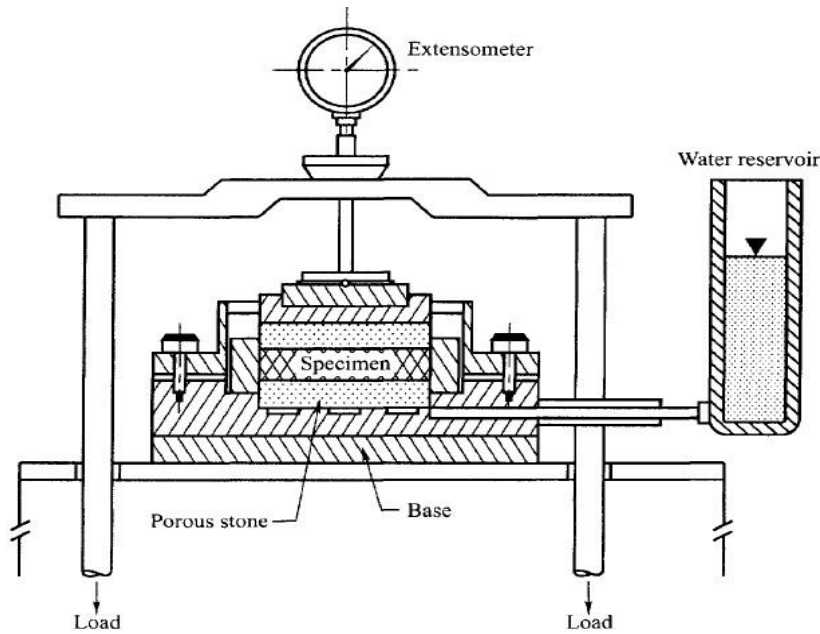
Figure 3.24. Schematic Drawing of Free Swell Test Setup (Murthy, 2002)

3.8. Test Results

Free swell values of cement kiln dust, lime, cement and sand treated samples without cure were presented in Figure 3.25. Free swell values of Sample 1 were shown in all figures in order to see the effect of the additives clearly.

Free swell values of cement kiln dust, lime, cement and treated samples with 7 days cure were presented in Figure 3.26.

Free swell values of cement kiln dust, lime, cement and treated samples with 28 days cure were presented in Figure 3.27.

Free swell values of cement kiln dust, lime, cement and sand treated samples with and without cure are given in Table 3.7.

Effect of curing period on treated samples was shown in Figure 3.28. Effect of curing was not studied for sand treated samples because sand is a neutral material

which does not cause a chemical reaction with clay. Therefore curing for sand treated samples does not affect the swell potential.

There is no readily available method for determining rate of swell. Thus, to determine the rate of swell of the Sample 1 and treated samples, the time necessary for fifty percent swell, t_{50}, was taken into account. Change in the rate of swell of samples with the addition of stabilizers and curing was shown in Figure 3.29.

Change in rate of swell, t_{50}, of cement kiln dust, lime, cement and sand treated samples with and without cure are given in Table 3.8.

Swell versus Time graphs were presented in Appendix B.

Table 3.7. Swell Percentages of the Samples

Sample	Swell Percentage (Without Cure) (%)	Swell Percentage (7 Days Cure) (%)	Swell Percentage (28 Days Cure) (%)
SAMPLE 1	36.63	35.58	33.63
3 % CEMENT KILN DUST	15.95	14.53	12.68
6 % CEMENT KILN DUST	12.42	11.05	10.05
9 % CEMENT KILN DUST	9.53	8.84	7.95
12 % CEMENT KILN DUST	6.74	6.32	6.00
15 % CEMENT KILN DUST	6.42	6.21	5.95
3 % LIME	16.50	15.42	14.42
6 % LIME	10.84	9.95	9.37
9 % LIME	8.37	7.89	6.05
3 % CEMENT	18.89	16.37	15.84
6 % CEMENT	11.58	11.00	10.16
9 % CEMENT	9.37	8.58	7.84
3 % SAND	33.58	-	-
6 % SAND	27.26	-	-
9 % SAND	24.11	-	-
12 % SAND	22.84	-	-
15 % SAND	20.26	-	-

Table 3.8. Change in Rate of Swell t_{50} of the Samples

Sample	Change in t_{50} (Without Cure) (%)	Change in t_{50} (7 Days Cure) (%)	Change in t_{50} (28 Days Cure) (%)
SAMPLE 1	0.00	0.00	0.00
3 % CEMENT KILN DUST	-54.72	-61.11	-69.05
6 % CEMENT KILN DUST	-71.70	-86.67	-86.79
9 % CEMENT KILN DUST	-83.02	-88.89	-88.33
12 % CEMENT KILN DUST	-84.91	-89.33	-88.57
15 % CEMENT KILN DUST	-85.47	-89.44	-88.62
3 % LIME	-79.25	-78.89	-82.98
6 % LIME	-85.66	-88.67	-88.10
9 % LIME	-86.04	-88.78	-88.57
3 % CEMENT	-65.09	-67.78	-67.86
6 % CEMENT	-82.08	-85.56	-86.67
9 % CEMENT	-85.85	-89.00	-88.33
3 % SAND	-1.89	-	-
6 % SAND	-4.72	-	-
9 % SAND	-5.66	-	-
12 % SAND	-6.60	-	-
15 % SAND	-8.49	-	-

"+": increase, "-": decrease
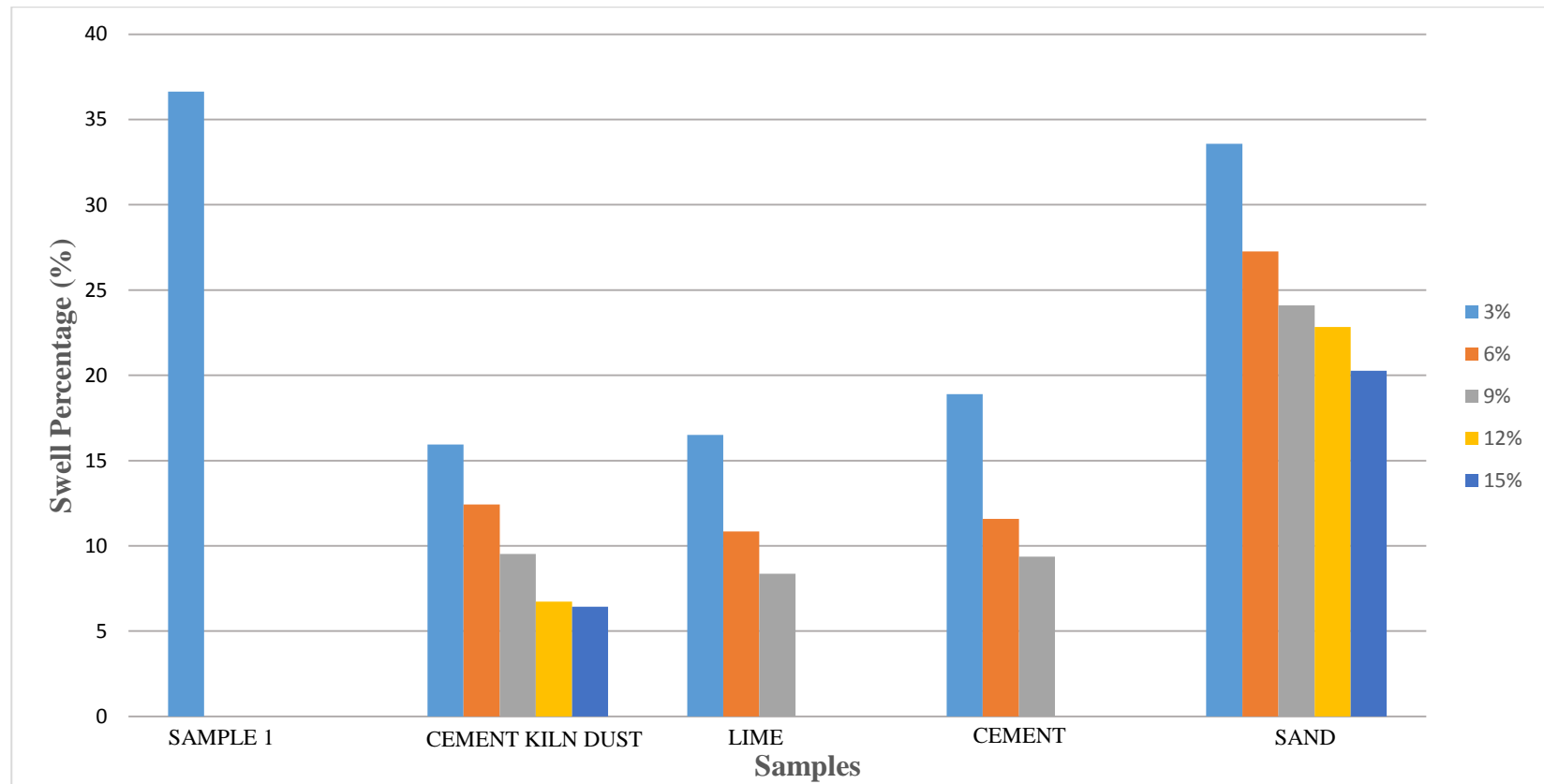
Figure 3.25. Effect of Addition of Cement Kiln Dust, Lime, Cement and Sand on Swelling Percentage without Curing
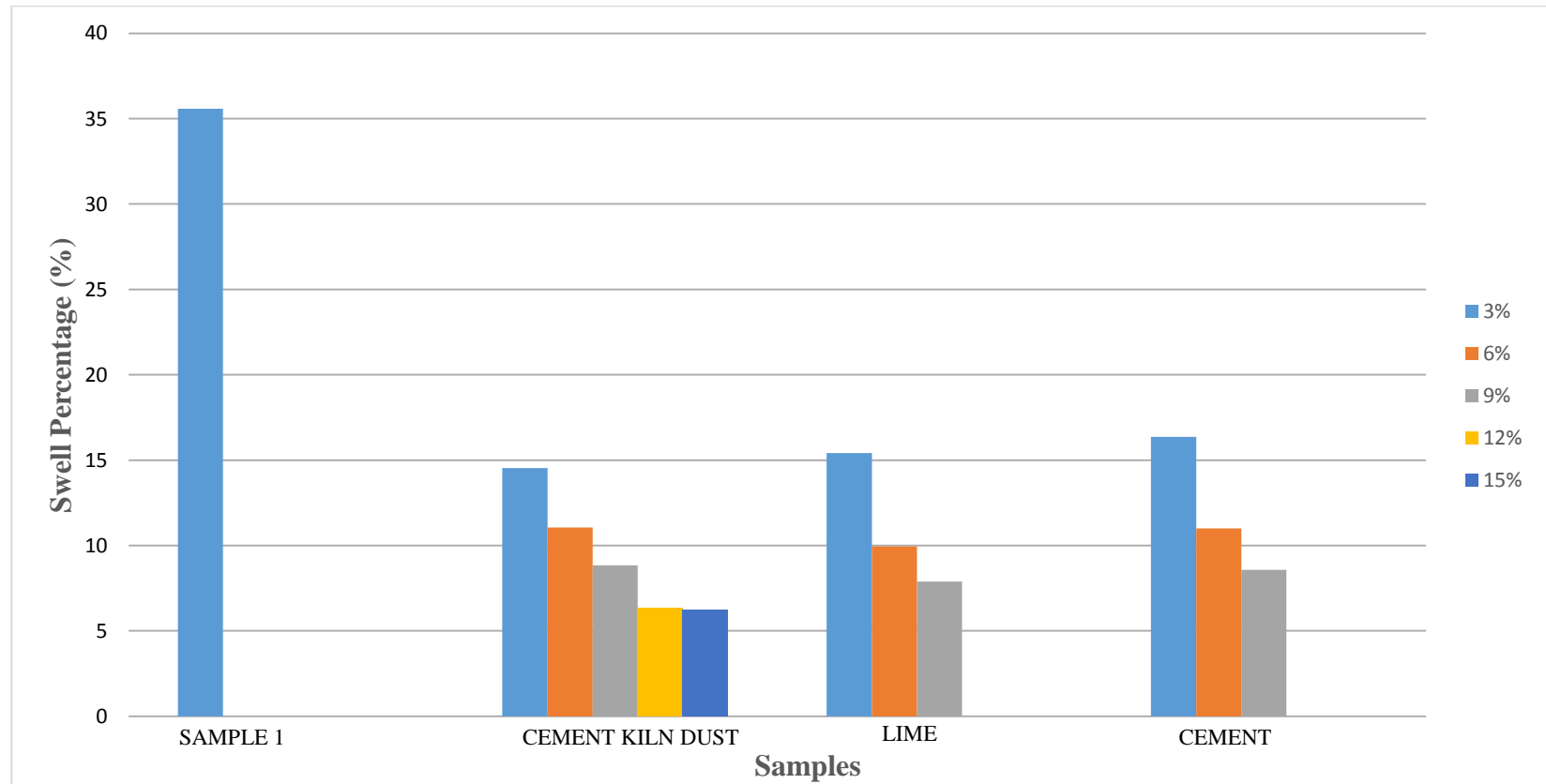
Figure 3.26. Effect of Addition of Cement Kiln Dust, Lime and Cement on Swelling Percentage with 7 Days Curing
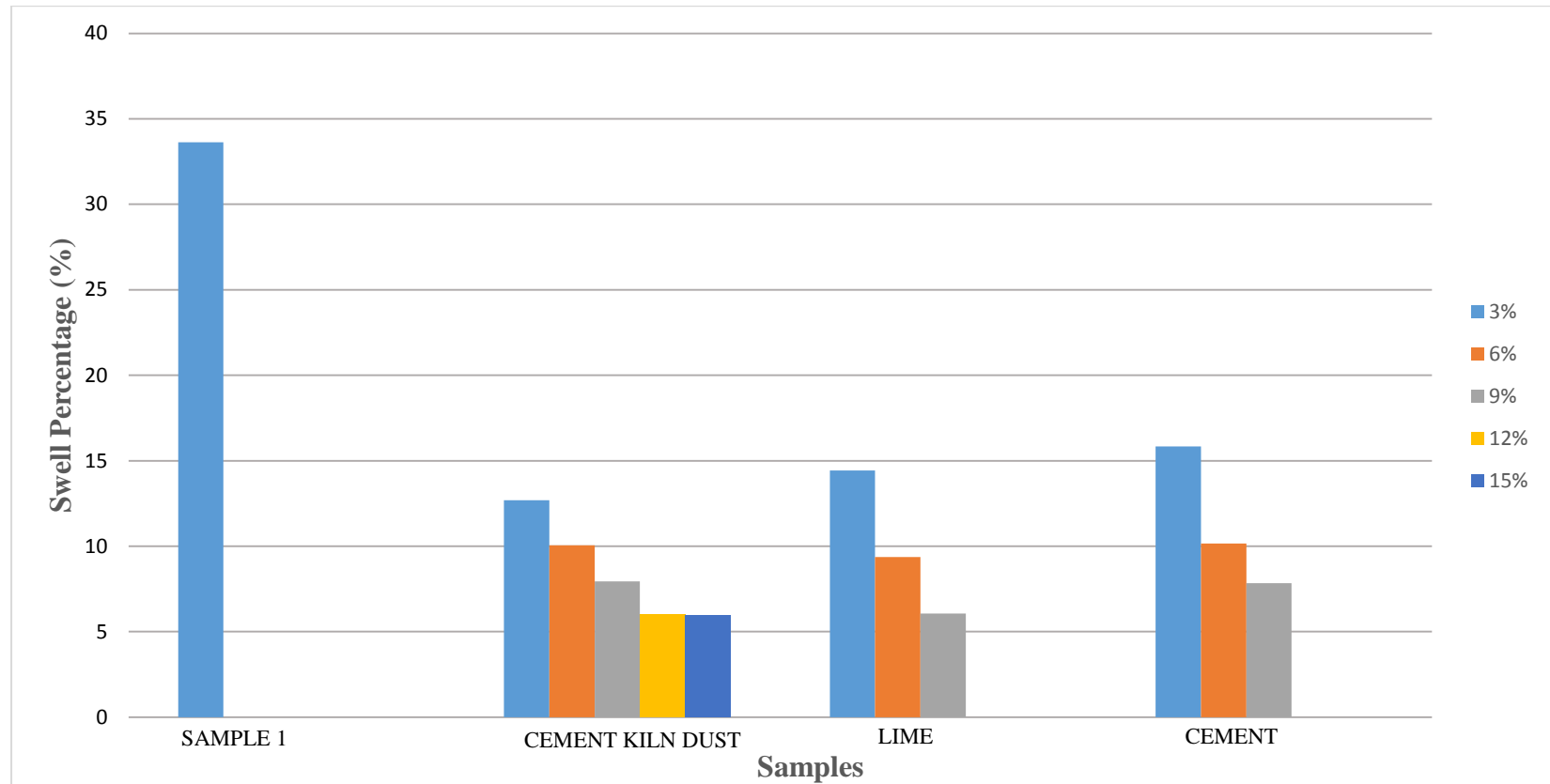
Figure 3.27. Effect of Addition of Cement Kiln Dust, Lime and Cement on Swelling Percentage with 28 Days Curing
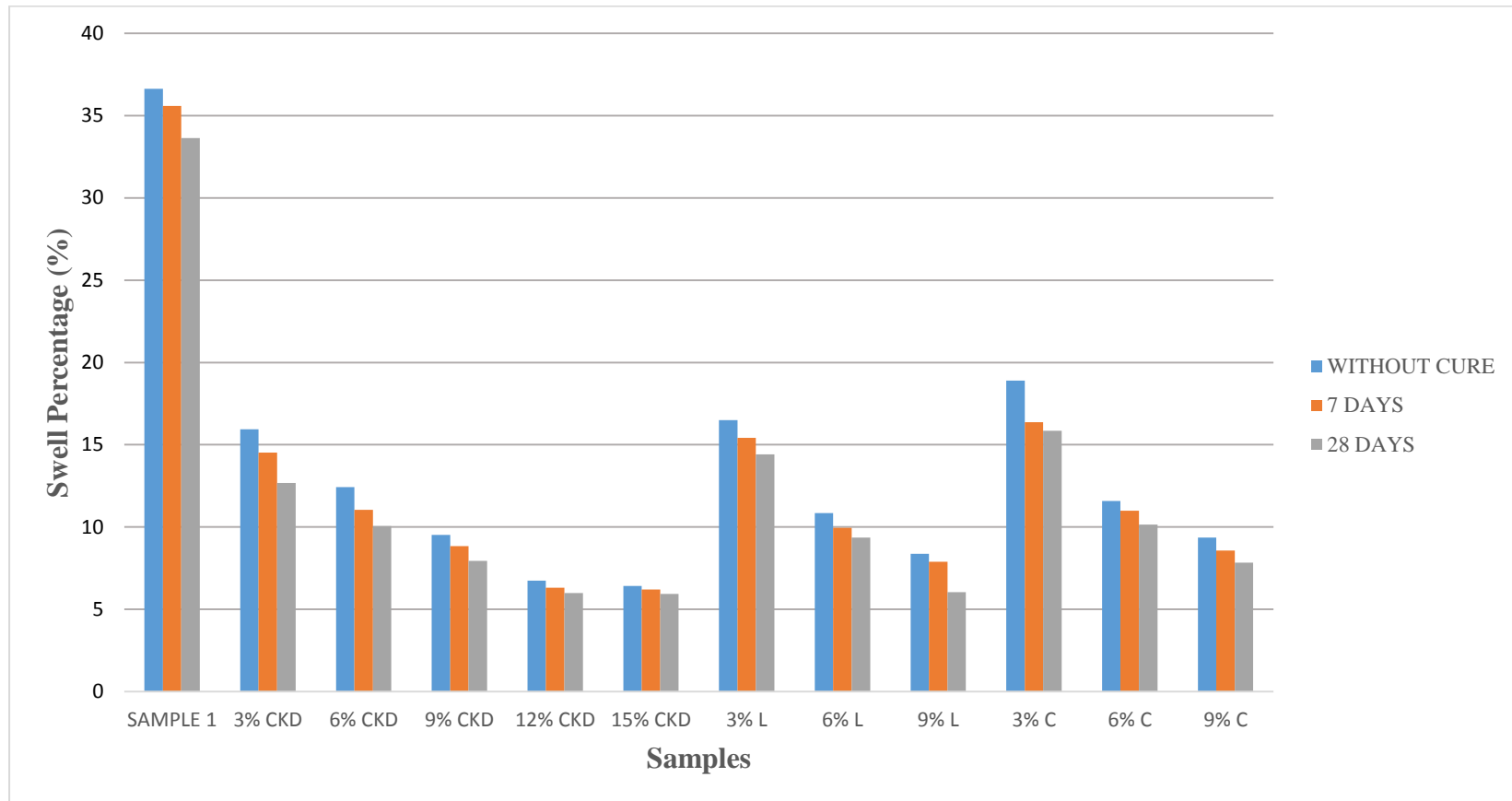
Figure 3.28. Effect of Curing on Swell Percentages
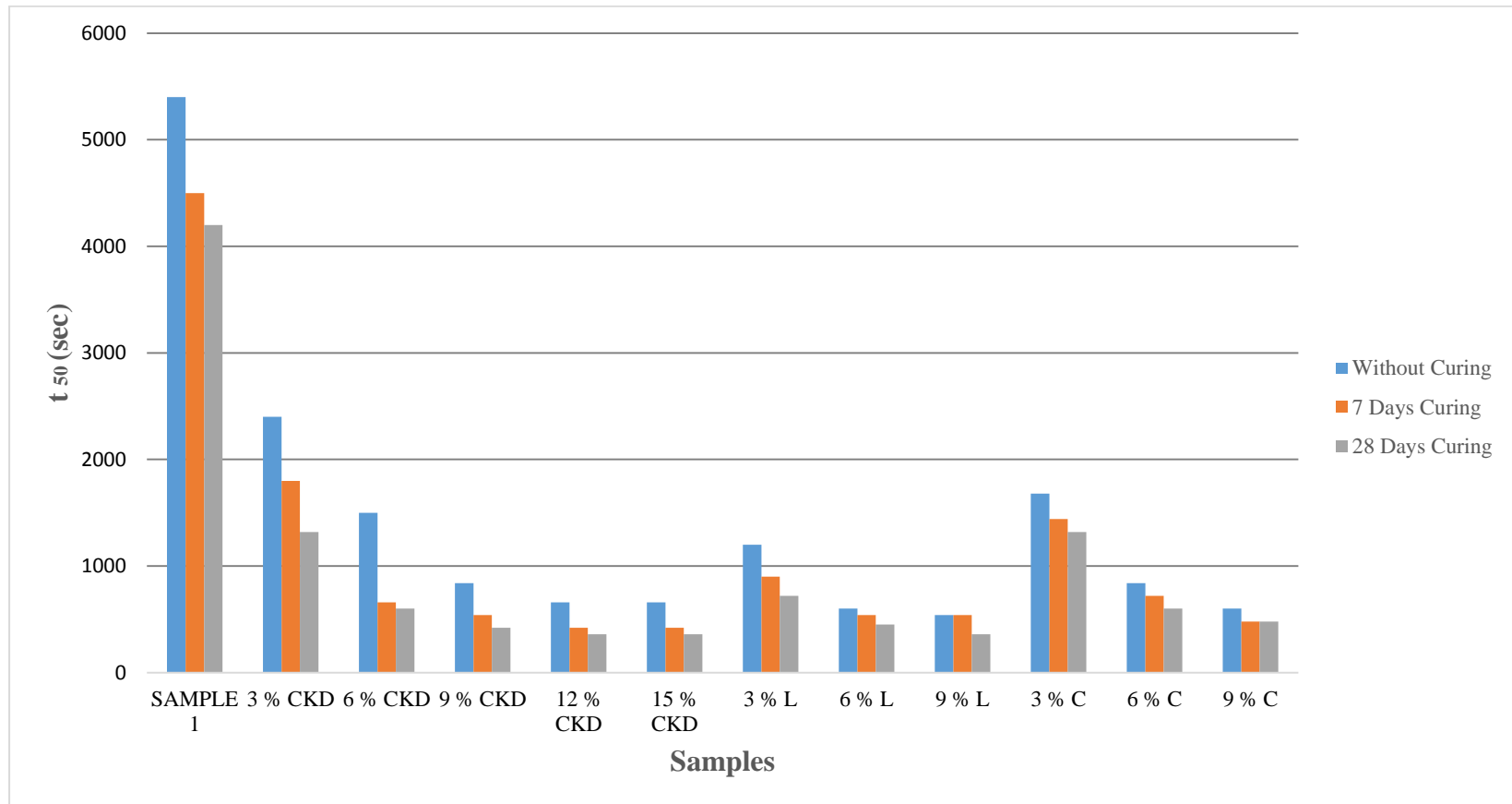
Figure 3.29. Effect of Curing on t_{50} of the Samples

CHAPTER 4

DISCUSSION ON TEST RESULTS

4.1. Effect of Additives on Density

Density of kaolinite, bentonite, cement kiln dust, lime, cement and sand were determined as 2.630 Mg/m³, 2.530 Mg/m³, 3.220 Mg/m³, 2.410 Mg/m³, 3.130 Mg/m³ and 2.690 Mg/m³, respectively. Moreover, the density of Sample 1 which was obtained by mixing kaolinite and bentonite was found as 2.611 Mg/m³. As it can be seen from Table 3.6, there is a gradual increase in density values while adding the stabilizers except lime. The decrease with addition of lime was anticipated because lime has a lower density than Sample 1. Increase in density for cement kiln dust and cement treated specimens could be caused by mixing higher density materials with Sample 1.

4.2. Effect of Additives on Liquid Limit

Liquid limit values for 3 % and 6% cement kiln dust treated samples were calculated as 116.40 % and 105.00 % respectively which are higher than Sample 1 (Table 3.6). In addition, the liquid limit value of 3 % cement treated sample was determined as 101.30 % also higher than liquid limit value of Sample 1. In the study of Sreekrishnavilasam (2007), the increase trend in liquid limit was observed with the addition of cement kiln dust to expansive soil and this behavior was

related to the type of the expansive soil. Moreover, an increase in the liquid limit was also seen with the addition of 1 % lime to the expansive soil in the study of Sherwood (1993).

Before conducting the tests, the lower liquid limit values were expected for all treated samples. Except the specified samples above, the anticipated trend could be seen. To determine liquid limit values, the tests were conducted for 3 times with Casagrande apparatus. When the increase was noticed, in order to make comparison, the tests were performed with fall cone apparatus for 2 times for all treated samples and Sample 1. Approximately similar results were obtained and average values were used for the study. Therefore, the reason of the increase in liquid limit values for 3 % cement kiln dust, 6 % cement kiln dust and 3 % cement treated samples could be caused by chemical reactions. On the other hand, decrease in liquid limit values of the treated specimens could be explained by mixing material which is not plastic with Sample 1 and also flocculation of particles because of chemical reactions and reduction in specific surface area.

Changes in liquid limit of the treated samples are shown in Table 4.1.

4.3. Effect of Additives on Plastic Limit

Plastic Limit values of the samples increased with the addition of cement kiln dust, lime, cement and sand as it was expected (Table 3.6). Maximum plastic limit value was obtained by adding 9 % lime as 39.80 % where this value is 24.80 % for Sample 1. Minimum increase was observed for sand treated samples. It caused 1 % increase in plastic limit (Table 3.6). Negligible decrease in plastic limit values for sand treated samples was observed. It is understood that using sand as a stabilizer does not affect plastic limit significantly.

Changes in plastic limit of the treated samples are shown in Table 4.1.

4.4. Effect of Additives on Plasticity Index

Plasticity Index values of Sample 1 decreased with the addition of cement kiln dust, lime, cement and sand (Table 3.6). On the other hand, plasticity index value of 3% cement kiln dust treated sample increased due to high increase in liquid limit value. Except 3 % cement kiln dust treated sample, the anticipated reduction trend was provided. Maximum decrease in plasticity index was observed for 9 % lime treated sample. It has a plasticity index value as 35.8 % where this value for Sample 1 is 72.70 % (Table 3.6). Decrease in plasticity index of the samples could be related to mixing of non-plastic material as a stabilizer to Sample 1 and also flocculation of particles because of chemical reactions.

Changes in plasticity index of the treated samples are shown in Table 4.1.

4.5. Effect of Additives on Shrinkage Limit

Reduction trend was observed for shrinkage limit values of treated samples (Table 3.6). Maximum decrease was seen for 12 % cement kiln dust and 9 % cement treated samples which their shrinkage limits were 10.58 % and 10.55 % respectively. Sand had a less impact on reducing shrinkage limit when compared with other stabilizers used in this study.

Changes in shrinkage limit of the treated samples are shown in Table 4.1.

4.6. Effect of Additives on Shrinkage Index

Shrinkage index values of 3 % cement kiln dust, 6 % cement kiln dust and 3 % cement treated samples increased due to increase in their liquid limit values. Moreover, for 9 % cement kiln dust and 6 % cement treated samples, an ignorable increase was observed. On the other hand, the maximum decrease in shrinkage

index was seen for lime treated samples, especially for 9 % lime treated sample which was 63.16 % (Table 3.6).

Changes in shrinkage index of the treated samples are shown in Table 4.1.

Table 4.1. Changes in Swell Percentages in Liquid Limit, Plastic Limit, Plasticity Index, Shrinkage Limit and Shrinkage Index

Sample	Change in LL (%)	Change in PL (%)	Change in PI (%)	Change in SL (%)	Change in SI (%)
3 % CKD	+19.38	+18.54	+19.67	-3.31	+23.74
6 % CKD	+7.69	+39.91	-3.30	-9.23	+10.94
9 % CKD	-0.31	+48.38	-16.91	-14.97	+2.51
12 % CKD	-9.23	+55.65	-31.36	-32.61	-4.75
3 % LIME	-15.79	+34.68	-33.01	-9.23	-17.06
6 % LIME	-18.11	+48.79	-40.94	-14.08	-18.89
9 % LIME	-22.46	+60.48	-50.76	-20.76	-22.78
3 % CEMENT	+3.90	+33.47	-6.19	-9.68	+6.50
6 % CEMENT	-1.74	+40.32	-16.09	-17.89	+1.36
9 % CEMENT	-7.18	+49.59	-26.55	-32.80	-2.26
3 % SAND	-2.25	+4.03	-4.40	-1.27	-2.45
6 % SAND	-4.51	+3.22	-7.15	-4.91	-4.43
9 % SAND	-7.28	+2.02	-10.45	-9.75	-6.80
12 % SAND	-11.59	+1.21	-15.96	-13.24	-11.27
15 % SAND	-12.82	+0.81	-17.47	-15.29	-12.35

"+": increase, "-": decrease

4.7. Effect of Additives on Grain Size Distribution

Grain size distribution curves were drawn by using sieve analysis and hydrometer tests. Hydrometer tests were repeated for three times and the average value of the three values was used for the study. For cement kiln dust and cement treated samples, conducting the hydrometer tests were difficult due to cementitious effects of these stabilizers. In order to prevent rapid reaction between the additives and Sample 1, the speed of performing test was increased and mixing of samples was done more carefully to prevent over flocculation. Although these precautions were useful for cement kiln dust treated samples, hydrometer tests for 3 %, 6 % and 9 % cement treated samples could not be performed. Rapid reaction and flocculation problems could not be overcome for these samples.

Grain size distribution curves show that, while adding stabilizers the curve of Sample 1 has a tendency to shift to coarser side. For cement kiln dust treated samples, less tendency was observed for 3 % and 6 %, on the other hand, obvious shifting to coarser side was seen by adding 9 % and 12 % (Figure 3.11). Grain size distribution curve of 3% lime treated sample was not altered much but the alteration was observed for 6 % and 9 % treated ones (Figure 3.12). The expected trend was also provided for sand treated samples (Figure 3.13). The flocculation of particles due to the chemical reactions and mixing silt-sized particles with the expansive soil could be the reasons of shifting the coarser side.

4.8. Effect of Additives on Activity

Activity of Sample 1 decreased with the addition of lime. On the other hand, there was an increase in activity with the addition of 3 % cement kiln dust but reduction was observed in the activity as the cement kiln dust amount was increased. However, there was no consistent relationship between activity and sand treated samples (Table 3.6).

4.9. Effect of Additives on Swell Percentage

Reduction of swell percentage of Sample 1 was observed with the addition of cement kiln dust, lime, cement, and sand (Table 3.7).

For cement kiln dust treated samples, addition of 3 %, 6 %, 9 %, and 12% cement kiln dust decreased the swell potential of Sample 1 from 36.63% to 15.95 %, 12.42 %, 9.53 %, 6.74 % respectively (Table 3.7). Maximum reduction in swell percentage was observed for 12 % cement kiln dust treated samples. A dramatic drop in swell percentage appeared after addition of 3 % cement kiln dust to Sample 1. Then drops were also seen while increasing the additive amount but not as much as the first drop. This reduction could be explained by replacement of some percent of expansive material with non-expansive material and chemical reactions. Having high calcium content was an effect on reduction in swell percentage. In order to check whether the increase in swelling potential continued or not, 15 % cement kiln dust treated sample was put into Bishop Apparatus after 12 %. The results revealed that negligible alteration in swell percentage occurred when it was compared with 12 % cement kiln dust treated sample (Table 3.7). Therefore, the optimum cement kiln dust amount was determined as 12 %. As the lower swelling potential clay is used, the lower optimum cement kiln dust amount is anticipated.

For lime treated samples, swell percentages reduced from 36.63 % to 16.50 %, 10.84 % and 8.37 % for 3 %, 6 % and 9 % lime treated samples respectively (Table 3.7). This reduction could be explained by replacement of some percent of expansive material with non-expansive material and chemical reactions. Containing high calcium was an important parameter causing less swelling. As it is seen from Table 3.7, approximate achievement for improvement of expansive soil was obtained by adding 12 % CKD and 9 % lime to Sample 1.

For cement treated samples, mixing the Sample 1 with 3 %, 6 % and 9 % cement reduced the swell percentages from 36.63 % to 18.89 %, 11.58 % and 9.37 %

respectively (Table 3.7). This reduction could be explained by replacement of some percent of expansive material with non-expansive material and chemical reactions. Like cement kiln dust and lime, high calcium content of cement had an impact on the decrease in swell potential. Addition of cement as a stabilizer gave similar results with lime and cement kiln dust.

For sand treated samples, the decreasing trend in swell percentages was also observed but less when it was compared with cement kiln dust, lime and cement treated samples. 3 %, 6 % and 9 %, 12 % and 15 % sand treated samples were used. The swell percentages of these treated samples were 33.58 %, 27.26 %, 24.11 %, 22.84 %, and 20.26 % respectively (Table 3.7). As it is stated, this percentage was 36.63 for Sample 1. Reduction in swelling potential could be explained by replacement of some percent of expansive material with non-expansive material.

In order to see the clear difference in heights, the molds before and after undergoing swelling process are shown in Figure 4.1.

(a)   (b)
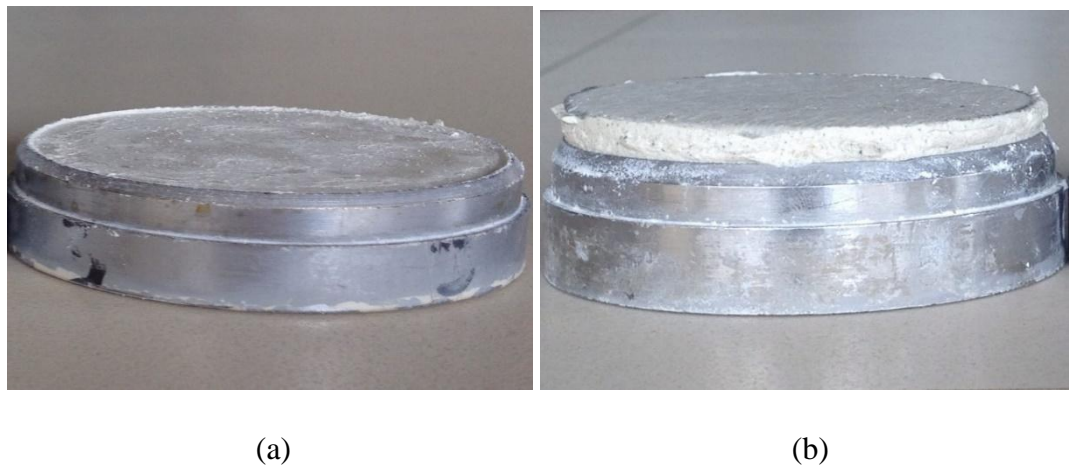
Figure 4.1. A View of Specimens before (a) and after (b) Swelling

4.10. Effect of Cure on Swell Percentage

Samples were put in the curing room for one day before compaction in order to allow water distribution homogenously. For every sample, the improvement effect of curing on swell percentage was observed (Table 3.7). This improvement may occur due to well distribution of water in sample and some pozzolonic reactions during curing time.

4.11. Effect of Additives and Curing on Rate of Swell

A dramatic reduction in the rate of swell was observed with the addition of 3 % additives, however this behavior was not valid for sand treated samples (Figure 3.37, Figure 3.38, and Figure 3.39). The time necessary for 50 % swell of total swell, t_{50}, was 5400 seconds for Sample 1. The maximum reduction in t_{50} was seen for 12 % and 15 % CKD whose t_{50} values were 360 seconds. On the other hand, sand treated samples acted similar to Sample 1.

The effect of 7 days and 28 days curing was investigated. It was clearly seen that curing decreased the rate of swell (Figure 3.40). The results revealed that 7 days curing generally had a significant impact on reducing the rate of swell when the t_{50} values of without and 7 days curing samples were compared. Moreover, results also indicated the effect of 28 days curing on reduction in the rate of swell.

Rate of swell decreased for each of the specimens with addition of stabilizer (Table 3.8). But curing had some effects for decreasing the rate of swell.

$$(t_{50})_{\text{without cure}} > (t_{50})_{\text{7 day cured}} > (t_{50})_{\text{28 days cured}}$$

CHAPTER 5

CONCLUSION

This study intends to investigate the suitability of using cement kiln dust as a stabilizer for expansive soils. Lime, cement and sand were also used in order to make comparison. The effects of specified stabilizers on Atterberg limits, density, grain size distribution and free swell percentages, rate of swell were studied. With regard to these studies, the following results could be achieved;

1. Density of Sample 1 increased with addition of cement kiln dust, cement and sand. On the other hand, reduction was observed for lime treated samples due to adding lower density material.

2. Grain size distribution curves tended to slide to coarser side with the addition of cement kiln dust, lime, and sand. Addition of silt size particles and chemical reactions caused this type of behavior. However, small variation in grain size distribution curves was observed for 3% cement kiln dust, 3 % lime, and 3 % sand treated samples.

3. In general, reduction trend was observed for liquid limit values as mixing additives with Sample 1. However, an increase in liquid limit value was seen for 3 % and 6 % cement kiln dust treated samples which were the same situation for 3 % cement treated sample. This discrepancy could be explained by chemical reactions.

4. Plastic limit values increased, on the other hand, shrinkage limit values (linear shrinkage values) decrease by the addition of stabilizers to Sample 1.

5. Plasticity index decreased with the addition of stabilizers except 3 % cement kiln dust. The exception was occurred due to increase in liquid limit.

6. Shrinkage Index decreased with the addition of cement kiln dust, lime, cement and sand. Increase in shrinkage index values were observed only for 3 % cement kiln dust, 6 % cement kiln dust and 3 % cement treated samples due to high increase in liquid limit values.

7. For activity, general trend could not be mentioned. Increases and decreases obtained for the activity by the addition of stabilizers. A high decrease was observed for lime treated samples. However, there is no consistent trend for the activity values.

8. Swell potential reduced significantly with the addition of stabilizers except sand. The similar amount of reduction behavior could not be observed for sand due to being neutral material. Reduction in swell percentage increased by the increase in the amount of additives. 12 % cement kiln dust, 9 % lime, 9 % cement, and 15 % sand treated samples decreased the swell percentage of Sample 1 from 36.63 % to 6.74 %, 8.37 %, 9.37 %, and 20.26 % without curing, respectively. The experiment in order to determine the swell percentage for 15 % cement kiln dust treated sample was also conducted. However, it was realized that there was a negligible difference between 12 % cement kiln dust treated sample.

9. The maximum decrease in swell percentage of treated samples was achieved with the addition of 12 % cement kiln dust. From this study, it can be concluded that with the addition 12 % of cement kiln dust the better results for swell percentage can be obtained when compared with 9 % lime and 9 % cement treated samples. This conclusion is also valid for 7 and 28 days cured samples.

10. The improvement effect of curing period, namely, 7 days and 28 days on the swelling percentage of the samples was observed.

11. The rate of swell increased with the addition of stabilizers. It also increased with the increasing curing time.

12. It can be obviously said from this study that cement kiln dust can be used as a stabilizer for improvement of expansive soils.

13. Utilization of CKD in this manner also has the advantage of reusing an industrial waste by-product without adversely affecting the environment or potential land use.

Recommendations for Future Study

In this study, cement kiln dust was examined as a stabilizing agent for expansive soils and swell potential tests were performed. Although performed swell tests are reliable, additional tests (scanning electron microscope and X-Ray diffraction studies) can be done to examine the microfabric and mineralogical characteristics of the specimens in order to reach to the full scale characterization of the specimens.

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APPENDIX A

CHEMICAL ANALYSIS REPORT OF CEMENT KILN DUST

Sample	L.o.I %	Al$_2$O$_3$ %	CaO %	Fe$_2$O$_3$ %	K$_2$O %	MgO %	MnO %	Na$_2$O %	P$_2$O$_5$ %	SiO$_2$ %	TiO$_2$ %
CKD	0.40	3.9	66.5	3.2	1.3	1.2	< 0.1	0.7	0.1	19.7	0.2

Figure A.1. Chemical Analysis Report of Cement Kiln Dust

APPENDIX B

SWELL VERSUS TIME GRAPHS

Swell percentage versus time graphs were presented for Sample 1, 3 % CKD, 6 % CKD, 9 % CKD, 12 % CKD, 15 % CKD, 3 % lime, 6 % lime, 9 % lime, 3 % cement, 6 % cement, 9 % cement, 3 % sand, 6 % sand, 9 % sand, 12 % sand, and 15 % sand treated samples without curing. Moreover, swell percentage versus time graphs for the same treated samples with 7 and 28 days curing were also presented in Appendix B except sand treated samples.
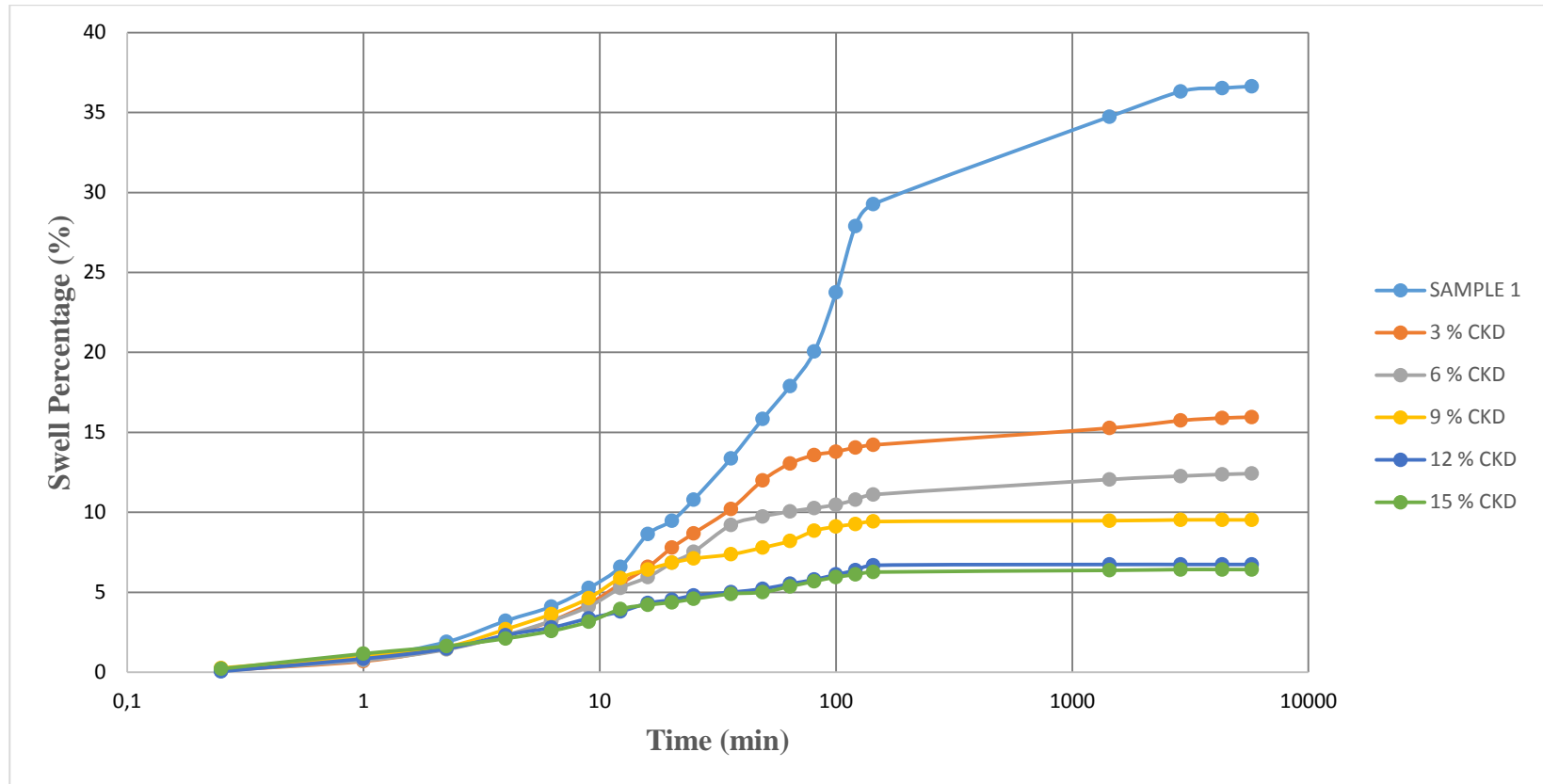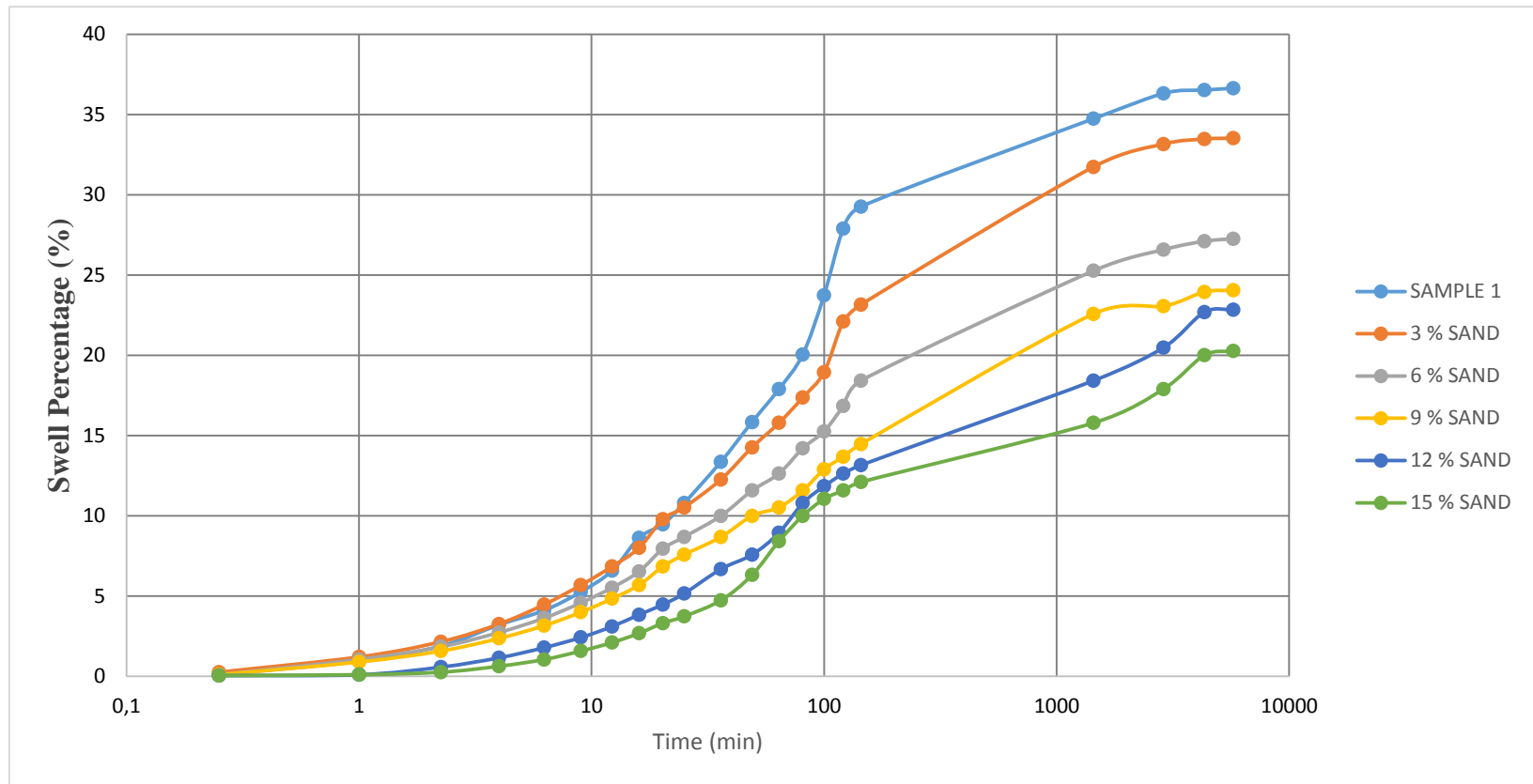
Figure B.1. Swell Percentage versus Time Graph for Sample 1 and Cement Kiln Dust Treated Samples without Curing
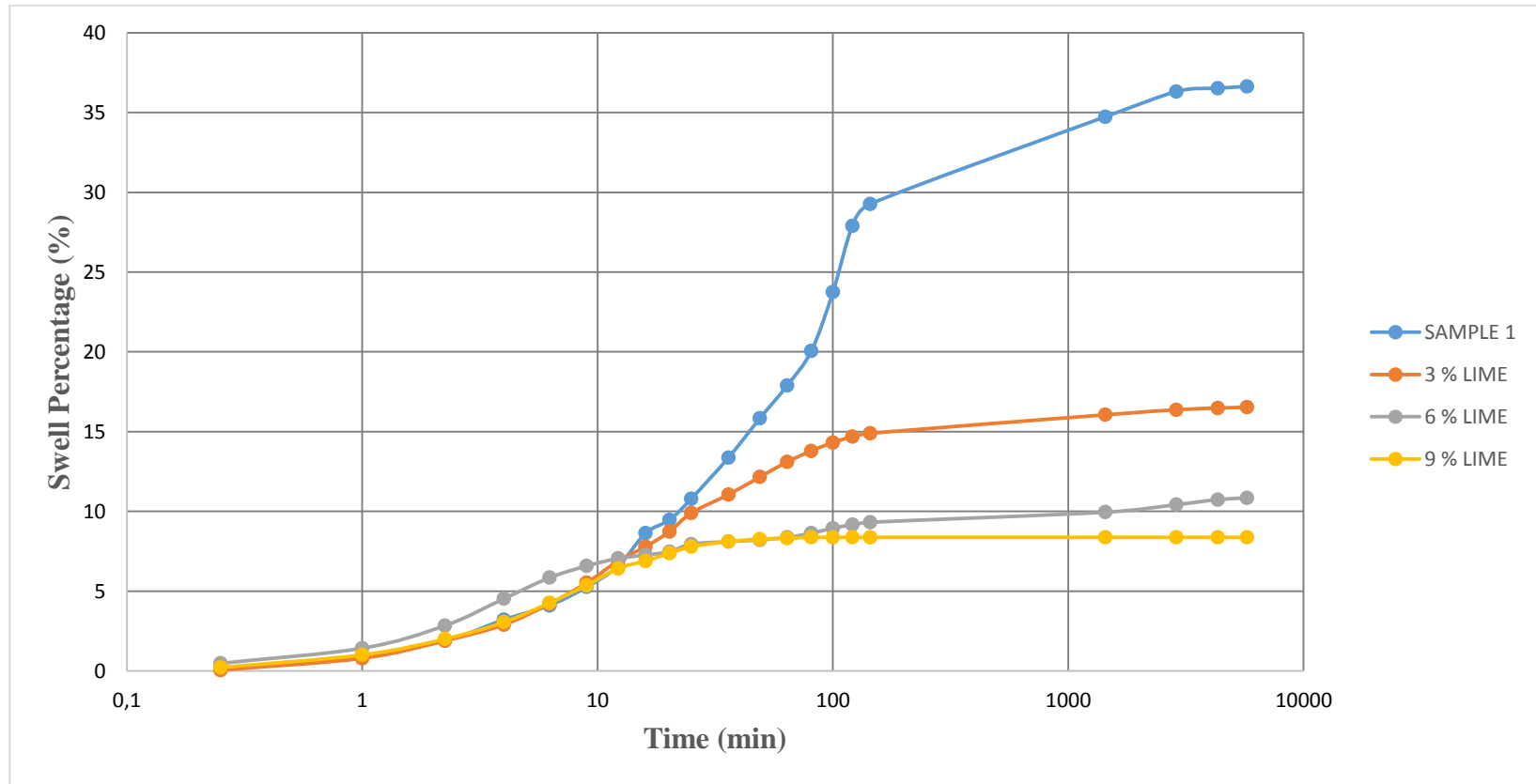
Figure B.2. Swell Percentage versus Time Graph for Sample 1 and Lime Treated Samples without Curing
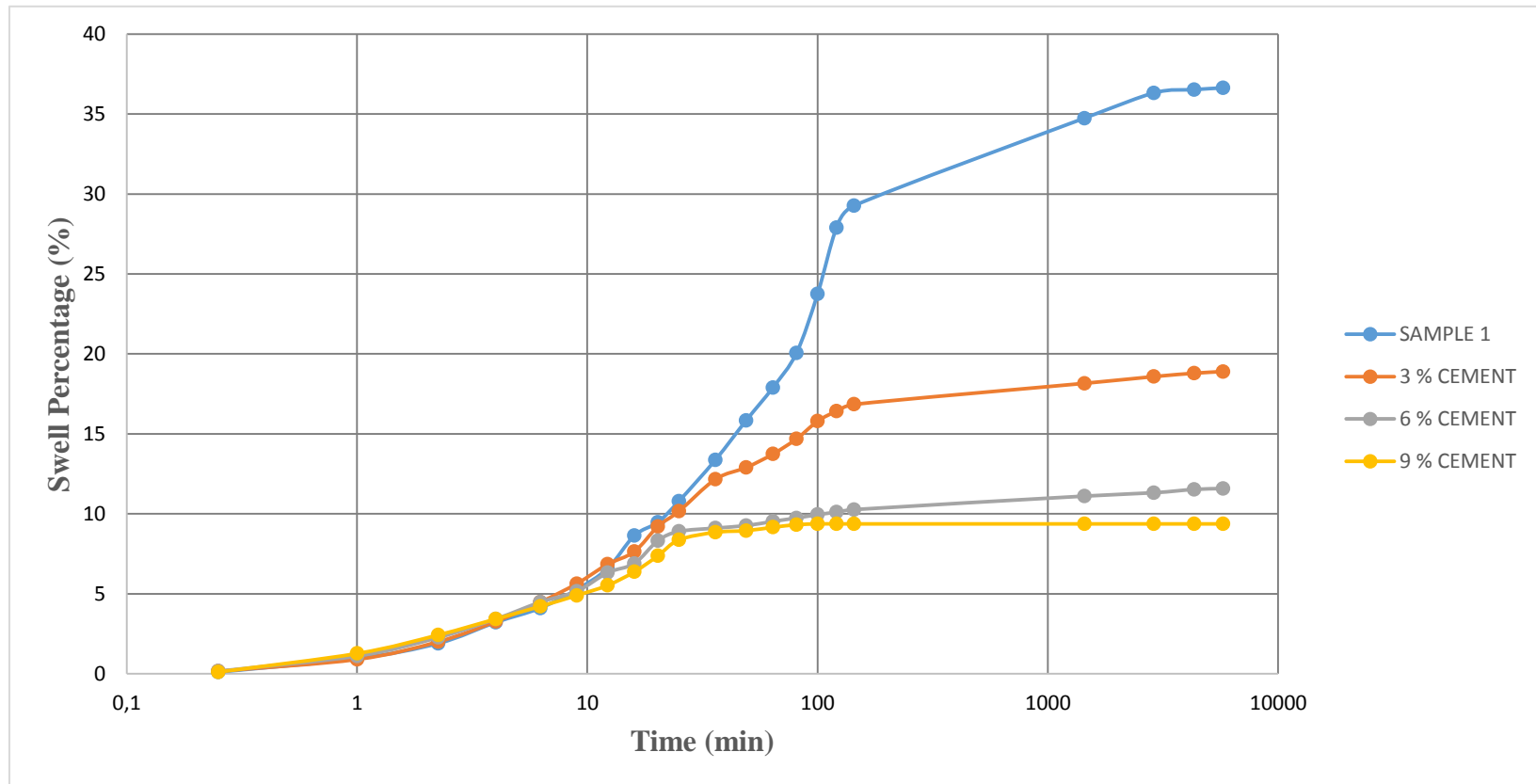
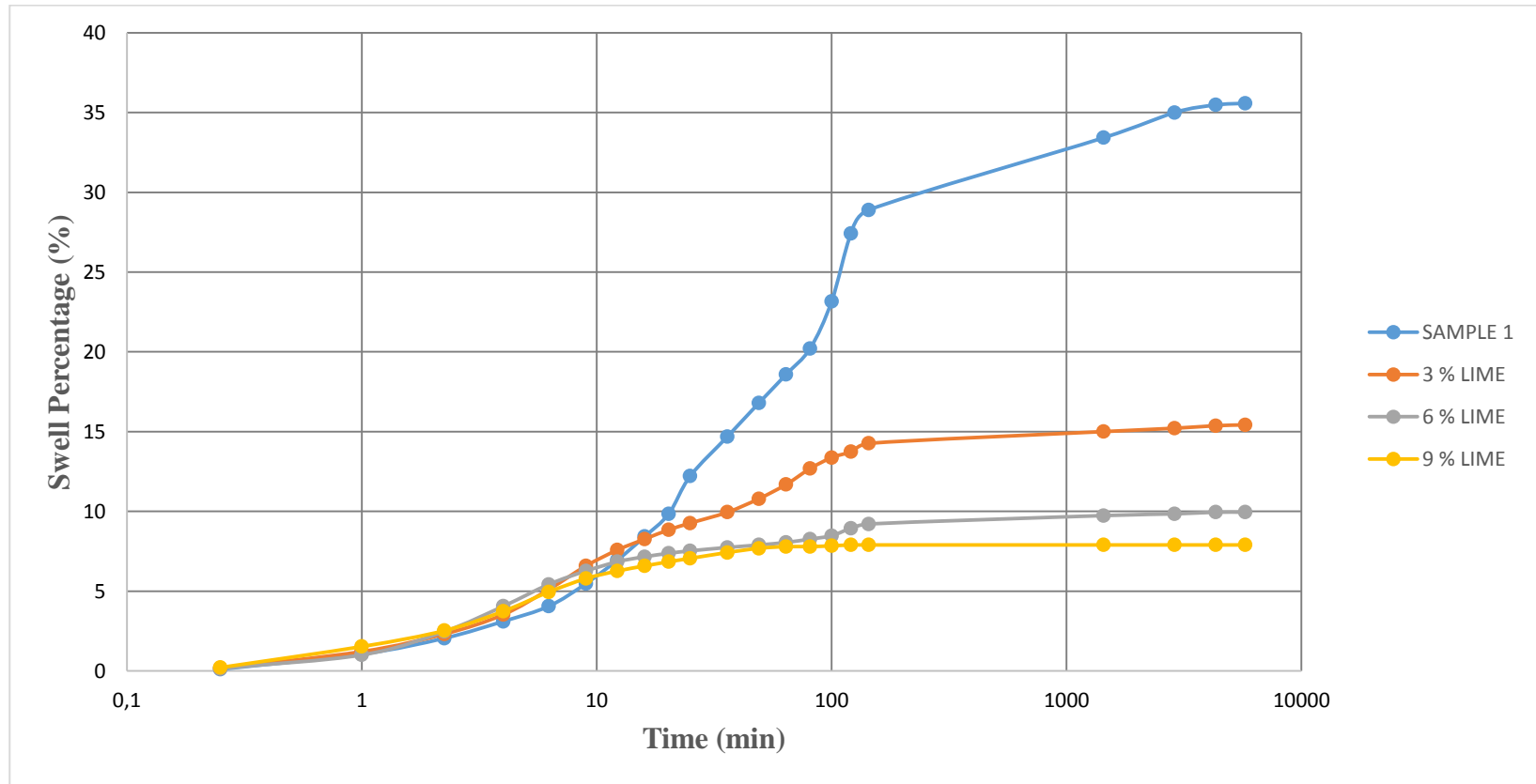**Figure B.3.** Swell Percentage versus Time Graph for Sample 1 and Cement Treated Samples without Curing

Figure B.4. Swell Percentage versus Time Graph for Sample 1 and Sand Treated Samples without Curing
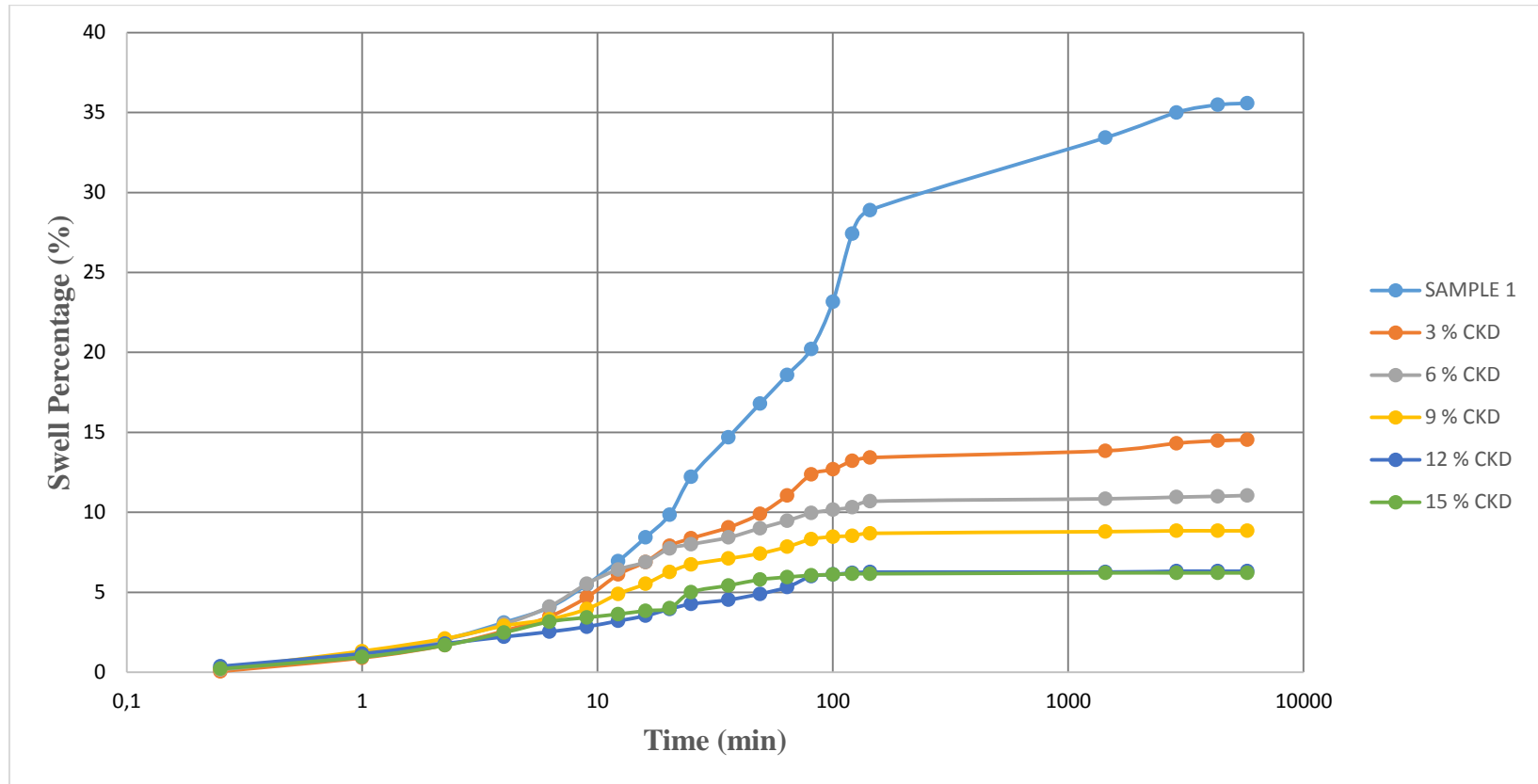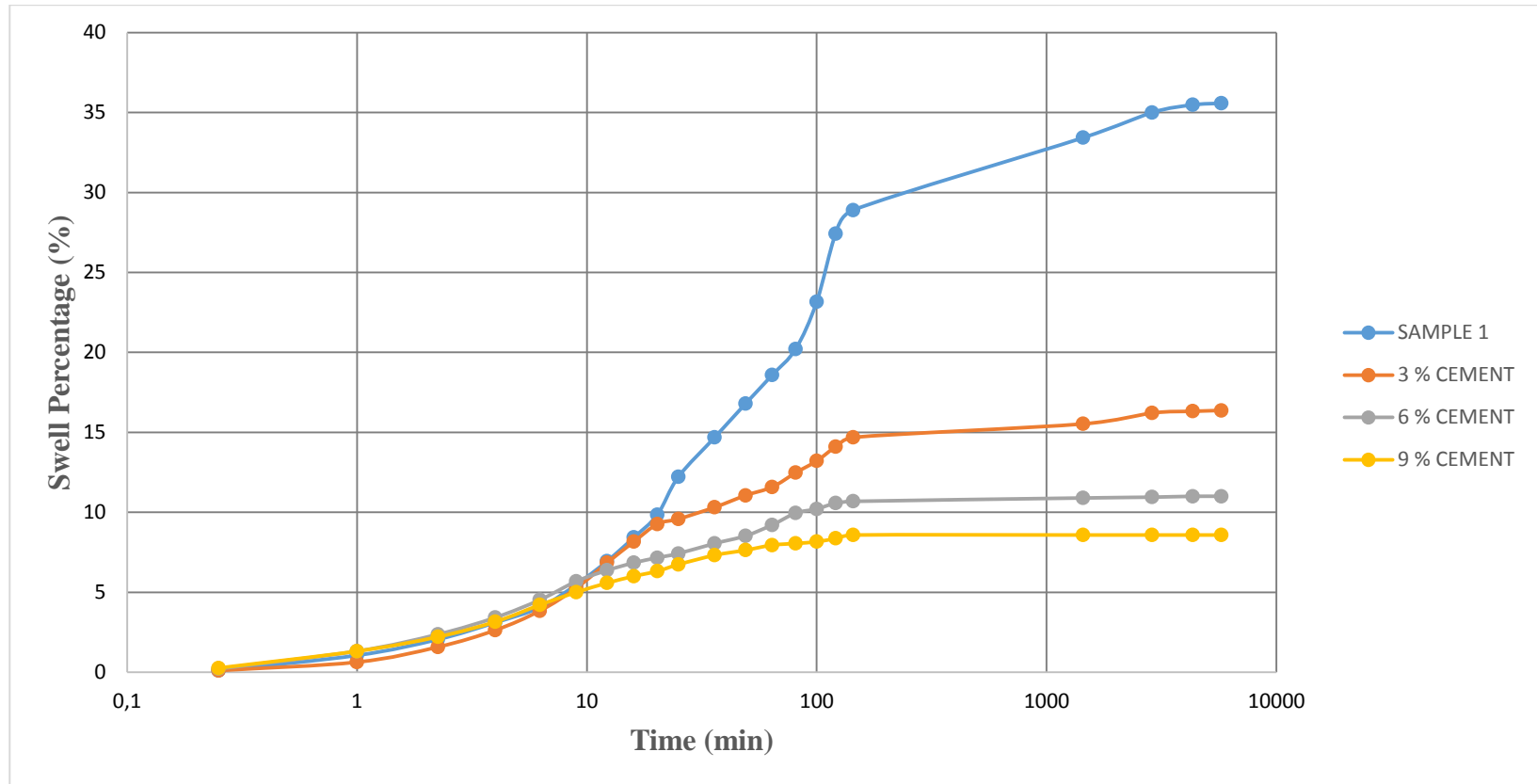
Figure B.5. Swell Percentage versus Time Graph for Sample 1 and Cement Kiln Dust Treated Samples with 7 Days Curing
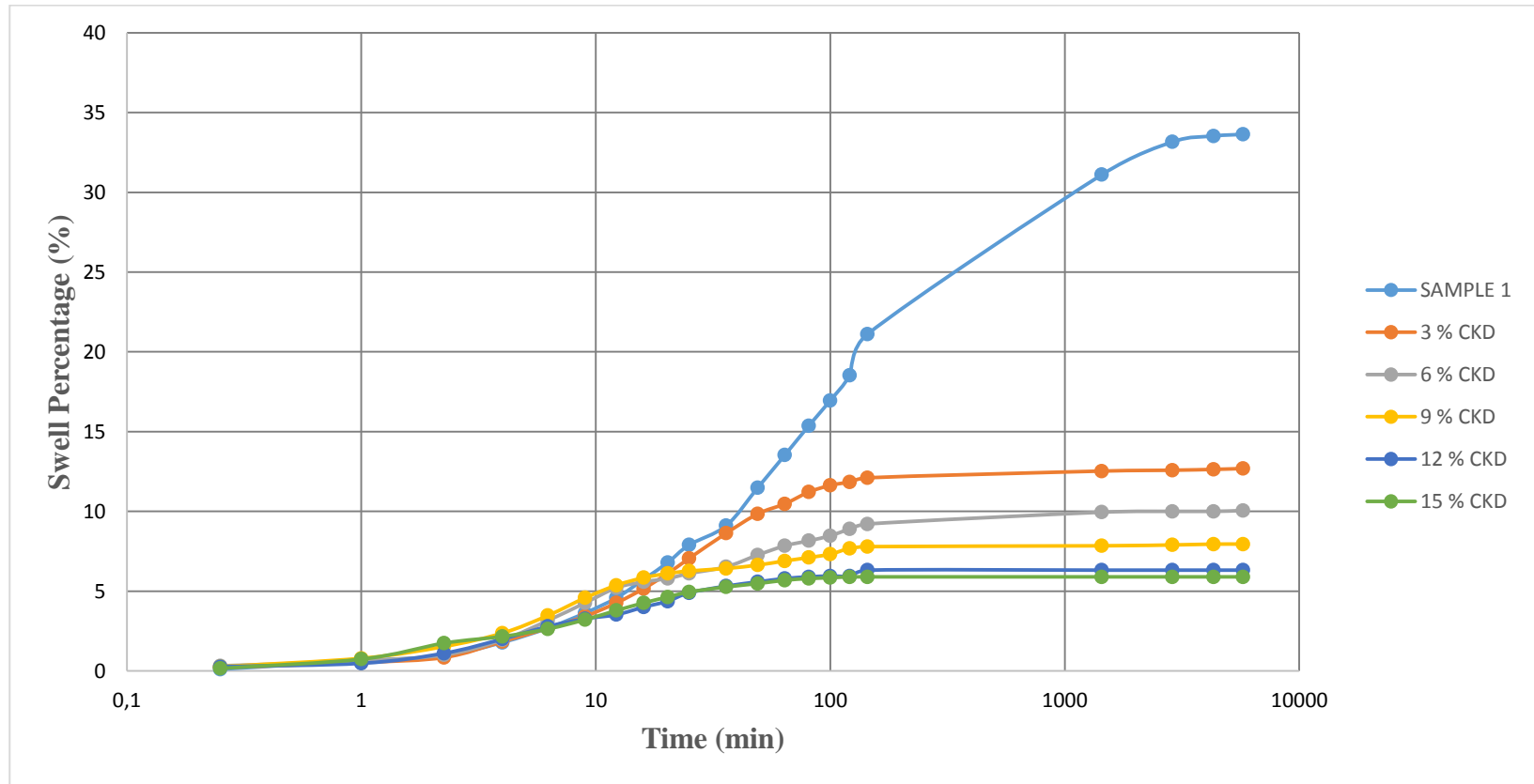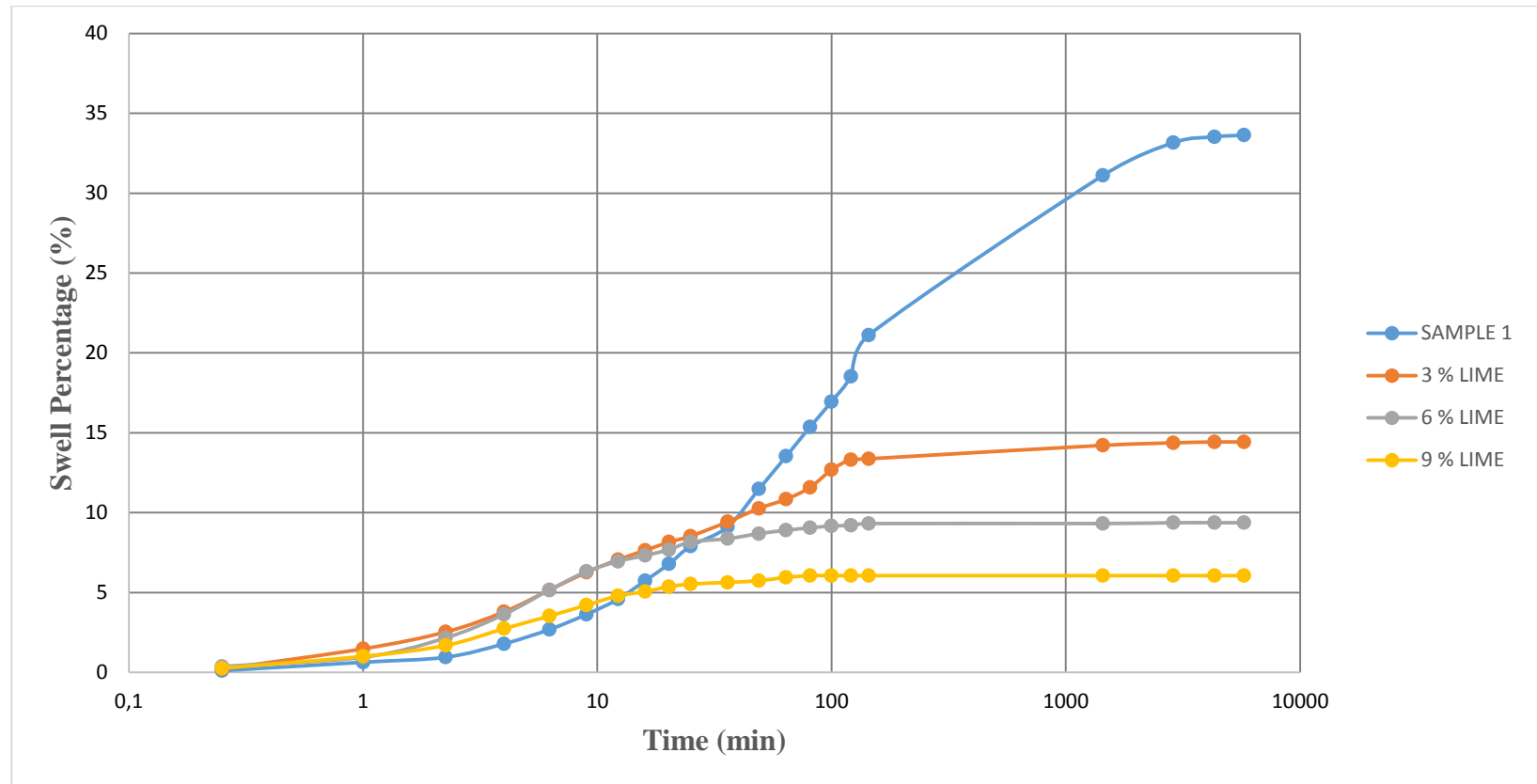
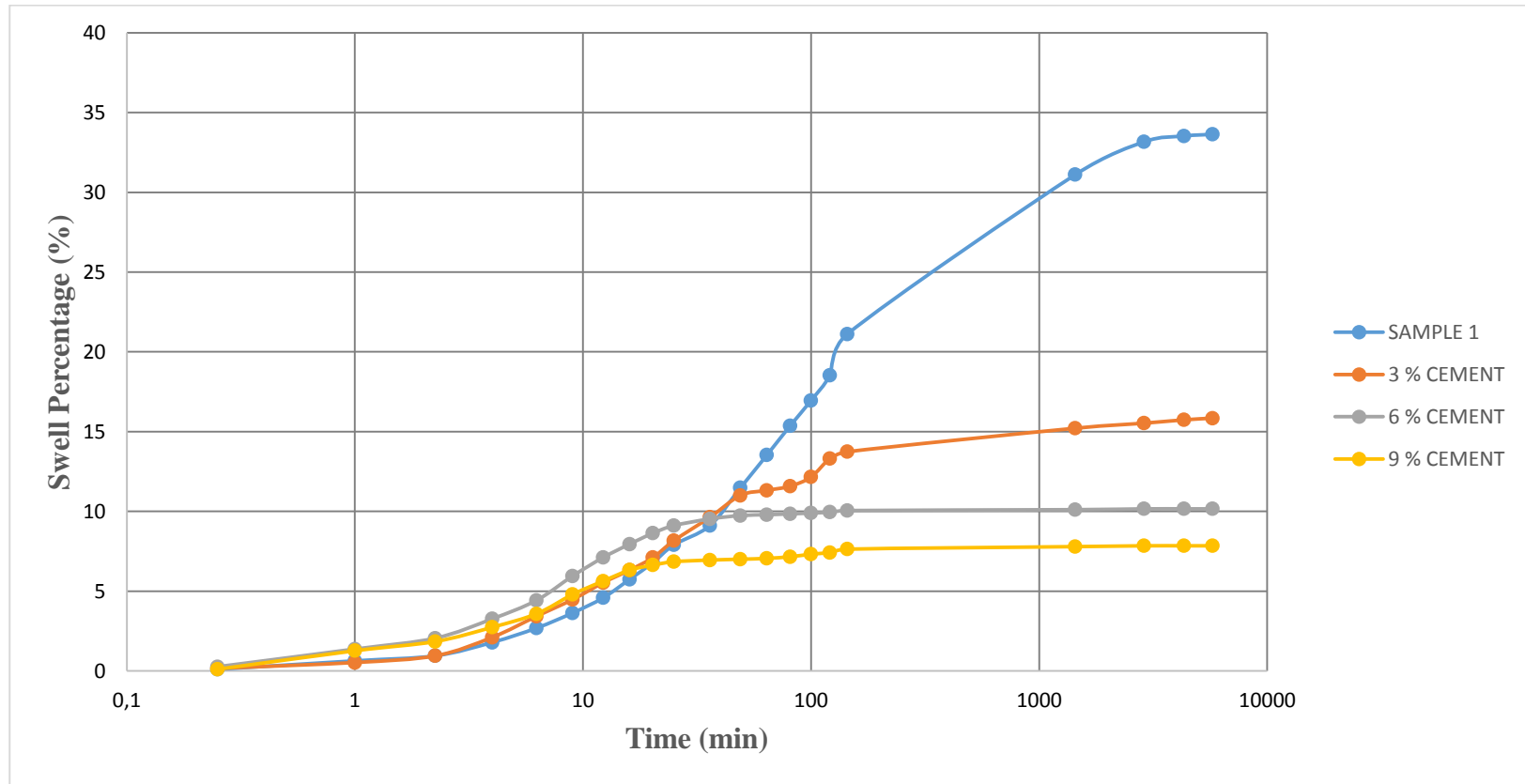**Figure B.6.** Swell Percentage versus Time Graph for Sample 1 and Lime Treated Samples with 7 Days Curing

Figure B.7. Swell Percentage versus Time Graph for Sample 1 and Cement Treated Samples with 7 Days Curing

Figure B.8. Swell Percentage versus Time Graph for Sample 1 and Cement Kiln Dust Treated Samples with 28 Days Curing

Figure B.9. Swell Percentage versus Time Graph for Sample 1 and Lime Treated Samples with 28 Days Curing

Figure B.10. Swell Percentage versus Time Graph for Sample 1 and Cement Treated Samples with 28 Days Curing