Improvement of Clay Soils using Lime Piles

Abiola Ayopo Abiodun

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

Master of Science
in
Civil Engineering

Eastern Mediterranean University
January 2013
Gazimağusa, North Cyprus

Approval of the Institute of Graduate Studies and Research

Prof. Dr. Elvan Yılmaz
Director

I certify that this thesis satisfies the requirements as a thesis for the degree of Master of Science in Civil Engineering.

Asst. Prof. Dr. Mürüde Çelikağ
Chair, Department of Civil Engineering

We certify that we have read this thesis and that in our opinion it is fully adequate in scope and quality as a thesis for the degree of Master of Science in Civil Engineering.

Assoc. Prof. Dr. Zalihe Sezai
Supervisor

Examining Committee

1. Prof. Dr. Özgür Eren
2. Assoc. Prof. Dr. Zalihe Sezai
3. Asst. Prof. Dr. Huriye Bilsel

ABSTRACT

The challenges posed by expansive soils to civil engineering structures in terms of damages and billions of dollars loss to repair and maintenance have been the major concern of 21st century researchers. In the last decades, lot of research works had been done to find feasible solutions to these problems. The challenges that the problematic soils caused are linked to their inappropriate engineering properties such as swelling, shrinkage, strength, and compressibility. The deficient properties of the expansive soils are attributed to the nature of their mineral type, structure, pore sizes, pore ionic solution etc. In order to transform the deficient properties to desirable properties required for engineering designs and applications, there is a need for a safe, economical and long lasting deep soil stabilization technique(s). In this study, a detail laboratory program was performed on a typical expansive soil to examine its physical and engineering properties in its natural state and when stabilized with a lime-pile technique. Five lime-piles of 3 cm each in diameter and 30 cm in height were constructed to stabilize the soil block which is 40 cm in diameter and 36 cm in height. The lime (Ca^{2+} ions) migrated from the piles into the soil block at different curing periods of 28, 90 and 120 days and produced different stabilized soil aggregates at different lime-pile distances. Extensive laboratory tests such as CBR and electrical resistivity were performed on the natural and the stabilized soils extracted at different lime-pile distances and curing periods and the test results were compared and discussed. The results of the tests indicated significant changes in the physical and engineering properties of the stabilized soils compared to the natural soil. The CBR test results suggest that the stabilized soil in 120 days of curing is fair enough to be used as both subbase and subgrade for highway and foundation constructions. The electrical resistivity measurements of the natural and lime-pile modified soils indicated that with lime treatment and curing time, the electrical resistivity of the lime treated soils decreased due to the particle aggregation and flocculation. The electrical resistivity (ER) test results suggest that the ER measurements can be used as a monitoring technique for lime diffusion in in-situ lime-pile applications.

Keywords: aggregation, electric resistivity, expansive soil, flocculation, lime piles.

ÖZ

Şişen zeminlerin mühendislik yapılarına yarattığı zararlar açısından ve harcanan milyarlarca dolar bakım ve onarım zararı açısından bu konu 21. yüzyıl araştırmacılarının en önemli konuları olmuştur. Son yıllarda, yapılan araştırma çalışmalarının çoğu bu sorunlara uygun çözümler bulmak için yapılmıştır. Sorunlu toprakların neden olduğu problemler, problemli toprakların şişme, büzülme, mukavemet ve sıkışma gibi mühendislik özellilerine ilişkilendirilmiştir. Şişen zeminlerin eksik özellikleri doğal mineral türü, yapısı, gözenek boyutları, gözenek iyonik çözümü gibi özelliklere atfedilmektedir. Yetersiz mühendislik özelliklerini, mühendislik tasarımı ve uygulaması için gerekli olan ve arzu edilen özelliklere dönüştürebilmek için, güvenli, ekonomik ve uzun süreli etkili olabilecek bir derin toprak stabilizasyon teknik(lerine) ihtiyaç vardır. Bu çalışmada, zeminin doğal halinde ve bir kireç-kazık tekniği ile stabilize edilmiş durumda, tipik bir şişen zemin üzerinde, fiziksel ve mühendislik özelliklerini incelemek üzere kapsamlı bir laboratuvar programı yürütülmüştür. 40 cm çapında ve 36 cm yüksekliğindeki toprak bloğu stabilize etmek için beş kireç-kazık, herbiri 3 cm çapında ve 30 cm yüksekliğinde inşa edilmiştir. Kazıklardan, farklı kür dönemlerinde 28, 90 ve 120 günde zemin bloğa yayılan kireç (Ca^{2+} iyonları) farklı kireç kazık mesafelerinde, farklı stabilize toprak agrega üretmiştir. Farklı kireç kazık mesafelerinde ve farklı kür sürelerinde elde edilen doğal ve stabilize toprak üzerinde Kaliforniya taşıma oranı, ve elektrik özdirenç gibi kapsamlı laboratuvar deneyleri gerçekleştirilmiş ve deney neticeleri karşılaştırılmış ve tartışılmıştır. Stabilize deney sonuçları doğal toprak ile karşılaştırıldığında, stabilize toprakların fiziksel ve mühendislik özellikleri önemli değişiklikler göstermiştir. Doğal ve kireç-kazık stabilize edilmiş topraklarda, elektriksel direnç ölçümleri, kireç stabilize edilmiş ve değişik kür sürelerinde, kireç stabilite edilmiş toprakların elektrik özdirenç değerlerinin partikül agregasyonu ve flokülasyonu nedeni ile azaldığını göstermiştir. CBR testi sonuçları 120 gün kür yapılan stabilize zeminin otoyol ve temel yapılar için alt temel malzemesi olarak kullanılmasının yeterince uygun olduğunu göstermektedir. Doğal ve kireç-kazık stabilize edilmiş topraklarda, elektriksel direnç ölçümleri, kireç stabilize edilmiş ve değişik kür sürelerinde, kireç stabilite edilmiş toprakların elektrik özdirenç değerlerinin partikül agregasyonu ve flokülasyonu nedeni ile azaldığını göstermiştir. Elektriksel direnç deney sonuçları, elektriksel direnç ölçümlerinin, arazide kireç kazık uygulamalarında, kireç difüzyon izleme tekniği olarak kullanılabileceğini göstermektedir.

Anahtar Kelimeler: agregasyon, elektrik özdirenç, flokülasyon, kireç kazıkları, şişen zemin

I dedicate this thesis to God Almighty and my Lord (Jesus Christ).

To the memory of my late parents, late Pa Gbolahan and late Mrs. Motoni Abiodun, and my late parental Guardian (Owodunni Olubamiro Michael) may you continue to rest in the bosom of our Lord. To my beloved family, many adorable friends and church family for their support, encouragement and prayers.

To my adorable fiancée (Adepeju) thanks for being a best friend when I needed you most. I appreciate your love, care and prayers.

I love you all!

ACKNOWLEDGMENT

I want to express my heartfelt gratitude to Assoc. Prof. Dr. Zalihe Sezai for her kind gesture and guidance from the beginning to the end of this study. It was with her invaluable and diligent supervision that made this study to be successful.

My sincere appreciation goes to all the academic and non-academic staff of Civil Engineering Department, EMU North Cyprus for their immeasurable assistance, encouragement and tutoring in my studying year. My sincere gratitude goes to all the lecturers that added outstanding academic values to my life in the course of my studies. I am also obliged to appreciate all my supportive friends and colleagues for their help throughout my studies. Sincere gratitude goes to Engineer Ogun Kılıç for his support and technical assistance during this study.

I will always be grateful to the wonderful family of Abioduns’, Olubamiros’ Adeifes’ and others– for their love, care and prayers.

I love you all.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS

ASTM	American Society for Testing and Materials
CBR	California bearing ratio
CH	inorganic clay of high plasticity
CP	central pile to pile distance
C_c	compression index
C_r	rebound index
C_v	coefficient of consolidation
C_u	undrained shear strength
^oC	degree celsius
D	diameter
DDL	diffuse double layer
E	void ratio
EC	electrical conductivity
ER	electrical resistivity
G	gram
G_s	specific gravity
H	height
i	hydraulic gradient
k	hydraulic gradient
kg	kilogram
kPa	kilopascal
1/K	diffuse double layer thickness
Lavg	average length

Lo	initial length
Lf	final length
LL or WL	liquid limit
LI	liquidity index
LLn	liquid limit of natural soil
LS	linear shrinkage
LSn	linear shrinkage of the natural soil
m	metres
m_v	coefficient of volume changes
MDD	Maximum dry density
OMC	optimum moisture density
ρ_b	In situ bulk density
ρ_d	In situ dry density
ρ_d (max)	maximum dry density
pH	degree of acidity or alkalinity
PI	plasticity index
PL	plastic limit
PLn	plastic limit of natural soil
PP	pile to pile distance
PVC	polyvinyl chloride
q_u	shear stress
r_f	final calculated radius
r_o	initial calculated radius
SS	stabilized soil
t_{90}	average degree of consolidation

UCS	unconfined compressive strength
USCS	unified soil classification system
V_f	final volume
V_o	initial volume
VS	volumetric shrinkage
V	velocity
w	in situ water content
w_{opt}	optimum moisture content
yp	unit weight of fluid
ε	shear displacement
μ	micro particles size
σ'	effective stress
σ*p*'	preconsolidation pressure
π	pi
ρ	electrical resistivity

Chapter 1

INTRODUCTION

1.1 Aim of the Thesis

The damages induced by problematic soils to civil engineering structures in the last decades, resulted in billions of dollars for repair and maintenance (Sridharan et al., 1997). North Cyprus with a land mass of 3,299 km^2 is occupied with deficient soils such as swelling clays, karstic bedrocks, alluvial and collapsible soils. The ancient buildings and the modern civil engineering structures were built on these deficient soils with no or inadequate modification or improvements. One of the challenges encountered in North Cyprus is that these structures are built on very fine to medium silty to clayey expansive soils of low shear strength, high compressibility and excessive heave which resulted in damages to ancient buildings and posed enormous threats to modern civil engineering structures in North Cyprus. These induced geotechnical failures are visible as cracking on buildings, bulging of roads, movement of foundations etc. These challenges are of keen interest to 21st century researchers by providing feasible solutions that are safe, long-lasting, economical and effective to upgrade the properties of these deficient soils.

The adequate stabilization of problematic soils in North Cyprus is paramount because the demand for land has increased tremendously in the major cities like Mağusa and Lefkoşa due to the increase in construction activities which is one of the booming industries in North Cyprus, just like other regions of the world.

Clay soils preoccupied these deficient soils and control their physical and engineering properties. Geotechnical engineers have studied the clay soil crystalline structures at the micro and macro level to understand clearly how to modify, improve and/or stabilize their properties to yield desirable properties suitable for engineering designs and applications (Rogers et al., 1997; Tonoz et al., 2003; Larson et al., 2009).

Improvement in the physicochemical stabilization of these deficient soils with chemical binders using deep ground (chemical) stabilization techniques such as lime column, lime pile and lime slurry injection have been proved to be more safer, effective and economical on the long term application. This is due to a reduction in maintenance costs with satisfactory improvement in their engineering properties such as the shear strength, swelling and bearing capacity etc. (Prabakar et al., 2003; Rajasekaran and Rao, 1997; Hausmann, 1990).

Based on the background information, this research work examines the effects of lime piles on engineering properties of a typical expansive clay soil in North Cyprus. The investigation involves the detailed findings of the significant modification in the soil properties such as Atterberg limits, linear shrinkage, shear strength, California bearing ratio, hydraulic conductivity, swelling, consolidation and electrical resistivity. Though the effects of stabilizing the deficient soils using lime piles (or lime columns) have been investigated and documented in the past by some researchers at different regions (Rogers et al., 1991, 1997; Tonoz et al., 2003; Zalihe, 2006; Nalbantoglu, 2001, 2004; Larson et al., 2009), this is not yet utilized in North Cyprus.

In the present study, firstly, all physical tests had been carried out on the disturbed natural soil sample and engineering tests were conducted on the natural soil. This soil was prepared and compacted in the test tanks using the in situ moisture content and in-situ dry density to attain the similar field conditions. Lime piles were constructed and systematically installed in the test tanks housing the compacted soil blocks. The physical and engineering tests were repeatedly performed on the samples extracted from the test tanks at different lime pile distances after 28, 90 and 120 days of curing.

The soil index tests were conducted to evaluate the linear shrinkage, plastic limit, liquid limit and plasticity index properties of the soil. Hydrometer test was performed to determine the percent fines content. The one-dimensional swell and consolidation tests were conducted to study the swell potential, swell pressure and compressibility characteristics such as compression index C_c, expansion index C_r, coefficient of consolidation $C_{v.}$ The saturated hydraulic conductivity, k_{sat} was indirectly determined to evaluate the pore size changes. Then changes in undrained shear strength were determined by performing a series of unconfined compression tests to establish the effect of lime-pile. Finally, the electrical resistivity (ER) test to evaluate changes in pore fluid concentration of the soil and California bearing ratio (CBR) test were carried out to ascertain its stability and suitability.

1.2 Background

1.2.1 Clay Mineralogy

The threats posed by the deficient clay soils require an in depth understanding of their microstructural and physicochemical behaviors. Clay soils are predominated by the hydrous alumino-silicates minerals with stratified crystal orientations. Their solid particles have size ranges smaller than 2 micrometers (μm). They are microscopic in size, cohesive, colloidal and plastic in nature with net negative electrical charges, high weathering resistance and their properties are governed by external surface forces (Mitchel, 1976).

The modification of the clay properties that is associated with the clay-lime reactions is influenced mainly by the migration of Ca^{2+} ions within the voids of the particles. The Ca^{2+} ions from the quicklime displace monovalent cations, Na^{+} and K^{+} at the negatively charged zone of the clay minerals. This changes their mineralogy within a short duration and with further increase in concentration of Ca^{2+} and curing time, the clay particles coagulate and change to a more granular soil (Larsson et al., 2009).

Their particles possess hexagonal flat minerals with different fluid content and cations bonded within their mineral structures by polar pull. This attraction is a function of their residual negative charges induced by isomorphous substitution. The effects of their residual negative charges are nullified with the adsorption of positive ions from the solution. The structure of entire clay minerals is made up of two basic structural units.

The silica tetrahedron is as expressed in Figure 1.1a and aluminium or magnesium octahedron as shown in Figure 1.1b. Silica tetrahedron is bonded tetrahedrally to four oxygen atoms (Figure 1.1c) while aluminium octahedron is bonded octahedrally to six oxygen atoms (Figure 1.1d). These two units are the building blocks of different types of clay minerals.

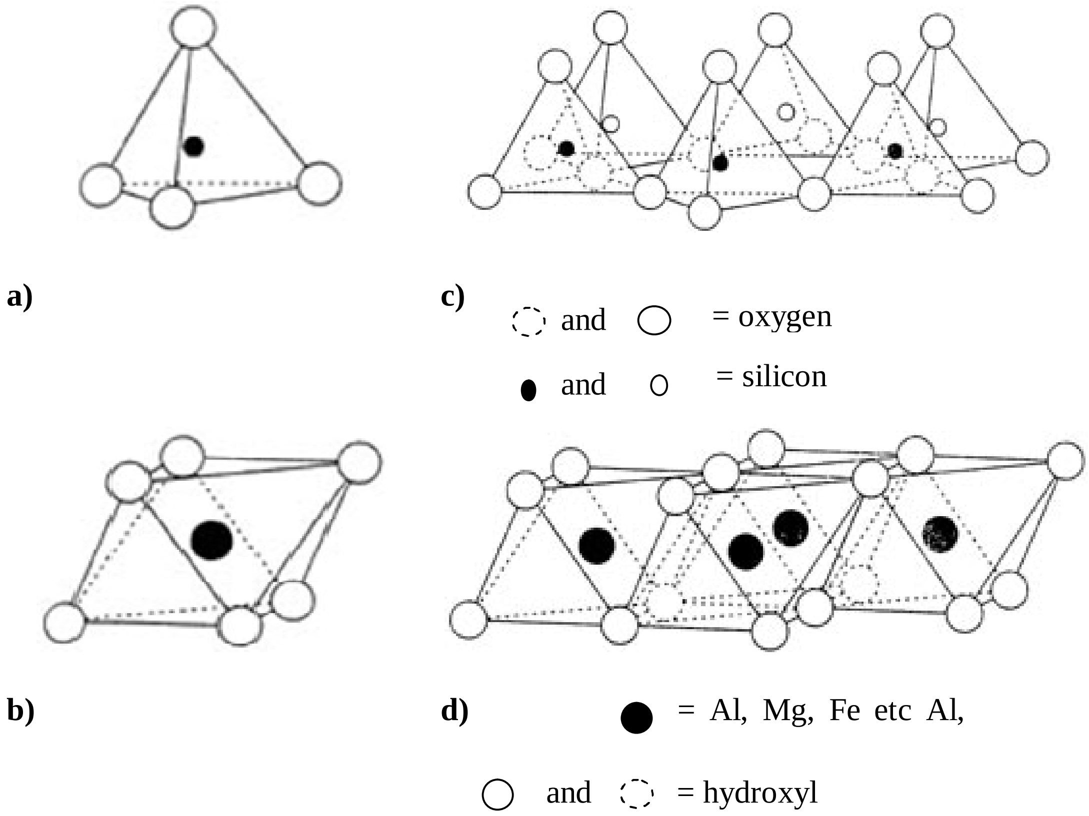


Figure 1.1: Silicate crystals: (a) a basic silica tetrahedron, (b) a sheet crystal of silica arranged in a hexagonal structure, (c) a basic alumina or magnesia octahedral unit and (d) sheet formation of alumina octahedral unit (Grim, 1968)

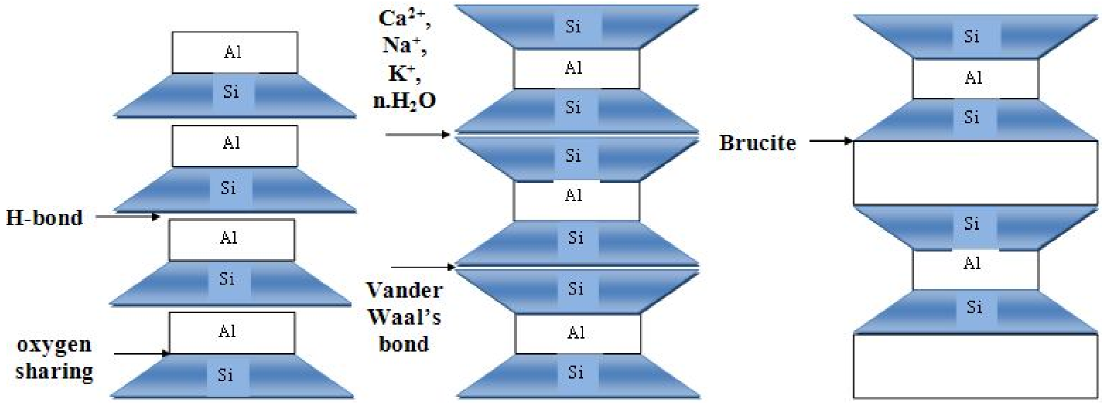


Figure 1.2: Clay mineral structures: (a) kaolinite group, (b) smectite group and (c) Chlorite group based on stacking block of unit silicate layer (Craig, 1992)

The grouping of clay minerals is based on the size of the unit cell, stacking distribution of layers and composition. Clay minerals have three main classes: 1.1, 2.1 and 2.1.1 categories representing the kaolinite, smectite and chlorite groups respectively as shown in Figure 1.2.

Smectite minerals exhibit extensive isomorphous substitution of silicon, Si or aluminium, Al by other cations. The minerals become unstable and highly plastic when come in contact with water (Mitchell et al., 2005; Little, 1995). The enormous amount of unbalanced substitution cause high cation exchange capacity (80 to150 meq/100 g). Therefore, the clay minerals have large specific surface area (500 to 800m^2/g) and a very weak interlayer bonding condition. Hence, expansive soils composed high percent of smectite minerals.

Generally, their mineralogy is crucial in geotechnical engineering and it is the micro-structural function for their physical and engineering properties. It controls the engineering behavior of soil such as surface chemistry, strength and swelling potential .

1.2.2 Fabric and Structure of Clay Soil

The fabric of soil is described as the geometric distribution of solid particles and pore spaces in a soil while the soil structure comprised the soil fabric and the interparticle forces which act between them **(**Holtz et al., 1981; Quighley et al., 1966). The clay colloidal particles have large specific surface area to mass ratio and exhibit different forms of fabric geometric associations such as dispersed, flocculated and aggregated (Note: E : edge, F: face; as indicated in Figure 1.3).

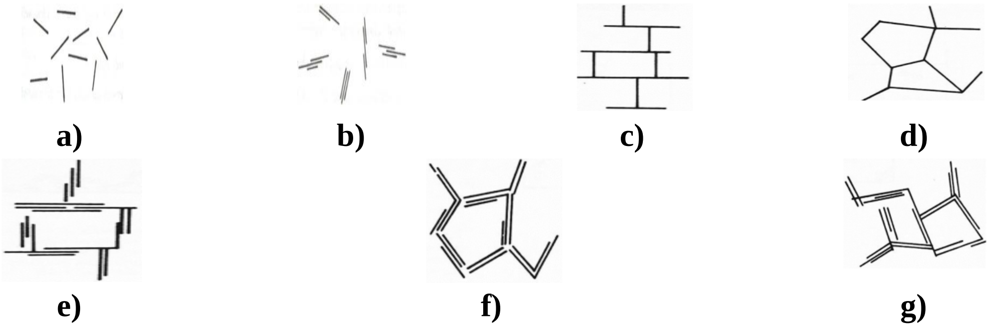


Figure 1.3: Clay particles association (a) Deflocculated and Dispersed (b) Aggregated but deflocculated, (c) EF flocculated but dispersed, (d) EE flocculated but dispersed, (e) EF and FF aggregated and flocculated (f) EE aggregated and flocculated, (g) EF and EE aggregated and flocculated (Mitchell, 1976).

1.2.3 Lime (Quicklime)

Lime is made up of a quicklime or slaked lime with precise content of calcium oxide in high proportions. Its utilization in deep ground stabilization techniques is found to be efficient in decreasing the swell-shrink capability, provides considerable strength and workability in deficient soils over time (Rogers et al., 1997; Nelson et al., 1992; Chen, 1988).

The lime reacts with clay at physicochemical and micro-structural levels and alters the physical and engineering properties of expansive soils. This chemical alteration changes mineralogy, effectively increase their shear strength and load bearing capacity due to long-term cementing reactions (Wilkinson et al., 2004a; Graves, 1996).

1.2.4 Clay-Lime Physicochemical Reactions

In recent years, the clay-lime interactions have received great interest from geotechnical engineers due to the necessity to solve the threats posed by expansive clay soil. Therefore inorganic materials interactions with clay particles have been utilized for the development of new stabilization techniques. The adsorption of the cation ions from lime by clay particles easily alters, modifies and stabilizes the expansive clay soils.

In this study, quicklime was utilized as the stabilizing agent and soil sample used is the expansive clay soil obtained from the South campus area of EMU in North Cyprus.

In the present study, the lime-pile was utilized to change the geomechanical properties of the clay soil, to improve its undesirable properties to acceptable properties at a remarkable depth. Many researchers have indicated that four mechanisms are involved in chemical interactions of lime-clay materials (Locat et al., 1990; Nelson et al., 1992; Little, 1995; Rogers et al., 1997, Larson et al., 2009).

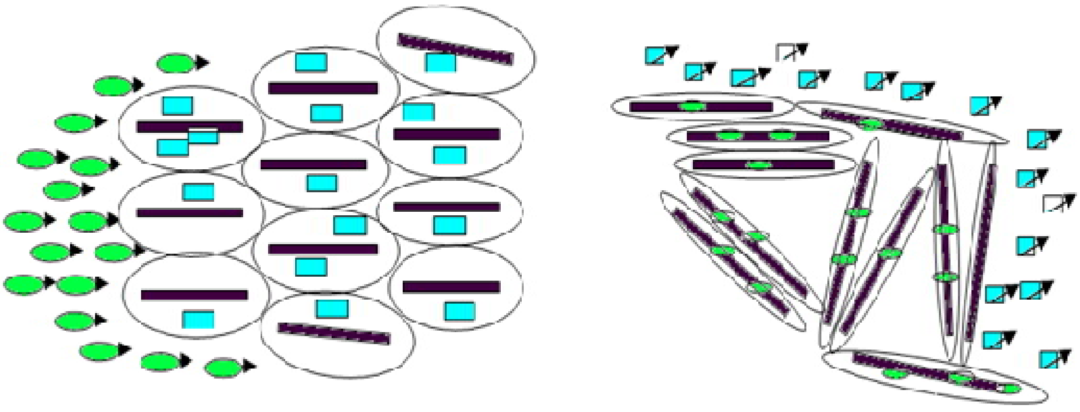
Table 1.1: The clay-lime reactions with their corresponding phase reactions

Supported by all researchers Clay-lime physicochemical reactions	Larsson et al. (2009) Clay-lime reactions	Supported by all researchers Clay-lime reactions phases
Cation Exchange	Modification resulting from ion exchange	First Phase Reactions
Hydration	Hydration	
Pozzolanic	Solidification resulting from pozzolanic reaction	Second Phase Reactions
Flocculation and Agglomeration		
Carbonation		

According to these researchers, these clay-lime physicochemical reactions are complex in nature and can be summarized in two phases. In Table 1.1, these mechanisms are generally categorized as (i) cation exchange, (ii) hydration, (iii) aggregation and flocculation, (iv) pozzolanic, and (v) carbonation reactions.

1.2.4.1 Cation Exchange and Flocculation

Basically, all fine to medium sized clay particles display quick cation exchange, flocculation and agglomeration reactions when subjected to lime treatment with the availability of water. The divalent calcium ions (Ca^{2+}) in the lime migrated into the clay and preferentially displaced weaker ions such as Mg^{2+}, K+, Na^{+}, H^{+}, Al^{3+} etc. precisely at the negatively charged zones of the clay minerals. The ions are adsorbed on their surfaces; reduce affinity for water thereby decreasing the diffused layer thickness (Eades and Grim, 1960; Diamond and Kinter, 1965; Broms, 1979, 1984; Paul and Rao, 1997; Larsson et al., 2009). The migration mechanism is highly complicated and controlled by the concentration gradient of the diffusing ions. In the case of the lime-pile techniques, the dominant flow mechanism is diffusion with the migration of Ca^{2+} ions (Tonoz et al., 2003).



Na^{+1} Saturated

Ca^{2+} Saturated

(Ca^{2+} 🟢, Na^{+} or K^{+} 🟦, clay particle ▬, water molecule ⬭)

Figure 1.4: Diagram showing cation exchange (Ca^{2+} ions replaced Na^{+} or K^{+} ions)

1.2.4.2 Hydration Reaction

In this phase, aftermath the clay-lime soil admixture, the exchange of cations begins followed by hydration reaction. This causes the formation of calcium hydroxide. This exothermic reaction generates heat and uses up some of the moisture in the soil (Tonoz et al., 2003).

$$CaO + H_2O \rightarrow Ca\,(OH)_2 + \text{HEAT (280 Cal/gr of CaO)} \qquad (1.1)$$

This is followed by flocculation and agglomeration (Figure 1.4) of clay particles (Herzog and Mitchell, 1963) causing a change in fabric and reduces the amount of fines (Little, 1995; Chen, 1988). Because of its cohesive nature, it changes to a more granular particle and considerably improved its strength (Rogers et al., 1996a).

1.2.4.3 Pozzolanic Reaction

The pozzolan such as clay soil interacts with calcium ions and water to produce cementing effects. The ASTM C340 standard defines pozzolans as siliceous or aluminous materials containing Si^{2+} and Al^{3+} ions, which exhibits low or no cementation value. The clay particles will, mineralogically breakdown in a high pH environment, produces silica in lightly dispersed form in the presence of moisture. This chemical reaction at room temperatures produce compounds exhibiting cementitious properties which bind the clay particles together (Jacobson, 2003).

$$Ca^{2+} + 2OH^{-} + SiO_2 \rightarrow \text{CSH: calcium silicate hydrate} \tag{1.2}$$

$$Ca^{2+} + 2OH^{-} + Al_2O_3 \rightarrow \text{CAH: calcium aluminate hydrate} \tag{1.3}$$

Pozzolanic reactions are functions of time and temperature. The build-up of ultimate cured strength of lime stabilized soil is slow and spontaneous for several years (Ormsby et al., 1973; Glenn et al., 1963; Eades et al., 1960).

1.2.4.4 Carbonation

Carbonation is the chemical reaction of lime with carbon (IV) oxide in the air to produce calcium-carbonate compound which is relatively insoluble. This chemical reaction is advantageous only when lime is properly handled during the field execution of lime columns or piles techniques. After mixing, a bit-by-bit carbonation process and production of cementitious products produces a continuing increase in strength (Arman et al., 1970). Carbonation is the last phase reaction in and occurs simultaneously with pozzolanic reaction which significantly improves and stabilizes the clay soil.

1.2.4.5 Electrical Conductivity

Electrical conductivity, EC is the quantity that determines the ability of a material to exhibit a conductance (resistance^{-1}) property by allowing the transmission of electric current through it. It is a function of the length and cross-sectional area of the material through which the electric current flows. It finds application in an intensive field mapping for agricultural purposes and geophysical mapping in earth science (Hartsock et al., 2000).

The flow of electric current in the soil mass is a complex process. Electrons migrate in the soil mass, via the solution in the micro and macro pores, along the surface of the minerals in the soil, via admixtures of particle and solution interfaces (Rhoades et al., 1989). Electrical conductivity is controlled by properties such as micro and macro pore connectivity, bulk density, soil aggregation, electrolytes in soil solution etc. (De Jong et al., 1979; Rhoades et al., 1981). The main properties strongly attributed to electrical conductivity are the intensity of the exchangeable ions (Ca^{2+} and Mg^{2+}) within the soil minerals and their solution (Hartsock et al., 2000).

Electrical conductivity (EC) and electrical resistivity (ER) are inversely proportional to each other. Therefore an indirect approach to determine the EC value is possible. The electrical resistivity of the soil is easily calculated using Ohm's law, with the multiplication of potential difference, V. This is determined using the units of voltage and current which flow through the soil to give the resistance of the soil in unit ohms. This is mathematically illustrated below:

$$\mathrm{R} = \frac{\Delta \mathrm{V}}{I} \qquad (1.4)$$

Hence, the electrical resistivity is determined using the mathematical expression below:

$$\rho = \frac{\mathrm{RA}}{L} \qquad (1.5)$$

Where ρ represents the soil electrical resistivity, R its resistance, A its cross-sectional area and L the distance between the electrodes. The electrical conductivity, σ with the SI unit Siemens per meter ($S \cdot m^{-1}$) which is the reciprocal of the electrical resistivity, ohm·meter ($\Omega \cdot m$) is then calculated as $\rho - 1$.

$$\sigma = \frac{1}{\rho} \tag{1.6}$$

1.3 Research Outline

This study comprises five chapters. The first chapter discusses the aim of the study and the basic information on the expansive clay soils detailing their interactions with a stabilizing agent such as quicklime. The second chapter provides an in depth literature review and consolidate with topics covered in Chapter 1. The general overview of the effects of soil stabilization using the lime-pile on the engineering and physical properties of the clay soil is presented in this chapter. A step-wise procedure for the small-scale laboratory model of lime-pile and the entire methodology in the current study are narrated in Chapter 3. The results obtained from the laboratory studies are analyzed and discussed extensively in Chapter 4. This includes the descriptions of all laboratory tests with their corresponding results obtained from both the natural and stabilized soils. To conclude, Chapter 5 summarizes the findings of the study with conclusions and propositions for additional study.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

Comprehensive literatures are accessible on the improvement of problematic soils using different stabilizing agents such as lime, cement, fly ash in different regions of the world (Jacobson, 2003). In North Cyprus, until now, there has been little study on the improvement of expansive clay soils using a deep ground stabilization technique. In this context, it is crucial to study the effects of the typical deep ground (chemical) stabilization technique, for instance lime piles, on the physical and engineering properties of an ideal expansive clay soil in North Cyprus using a small scale laboratory model. This chapter provides the basic information on the problematic soils, their deficient properties, their challenges, stabilization techniques, deep-mixing lime stabilization techniques and their corresponding effects on the engineering properties of clay soils in relation to clay-lime physicochemical reactions as propounded by different researchers.

2.2 Problematic Clay Soils

Problematic soils in civil engineering are described as an extensive type of challenging soil deposits that are products of various geologic activities. These soils are characterized by deficient properties that are uneconomical and unsafe for civil engineering structures. Researchers in civil engineering fields have reported that these soils pose threats to all kinds of civil engineering structures and are the basis for most geotechnical failures (Ola, 1983).

These deficient soils are unsuitable construction earth materials to be built-on, built-in, and supported-with, for civil engineering structures such as foundations, highways, embankments, bridges, underground tunnels etc. They possess unsatisfactory properties such as low strength, excessive heave, high compressibility (Nalbantoglu, 2004; Tonoz et al., 2003).

Rogers et al. (1997) emphasized that the deficient characteristics of the soils are linked to their engineering characteristics such as plasticity, volume change, and hydraulic conductivity, chemical and mineralogical compositions. These flawed properties can also be attributed to the nature of their soil pore fluid chemistry, surcharge, particle size distribution, temperature, pH, organic composition and aging (Ahnberg, 2006).

Some of the major problematic soils are soft clay deposits, collapsible soils, high sensitive clays, dispersive clays, quick clays, high expansive clays, etc. Liquefiable and quicksands are the types of usually problematic sandy soils known to researchers. In this section, only expansive clay soils will be considered in detail.

2.2.1 Expansive Clay Soils

The first recognition of the threats incurred by expansive soils was noticed by researchers in the early 1930‘s (Chen, 1988). These soils are characterized as highly plastic, comprising silicate minerals such as montmorillonite that attracts and absorbs a great amount of water. Within the clay mineral class, smectites are responsible for major damages, although Illite also has some threat potential. In contact with water, the voids in their mineral plates entrap the water molecules. The usual absorption of water, forces their plates apart causing the soil pressure and volume to increase (Glenn et al., 1963).

The periodic fluctuations in the field environmental conditions of the in-situ expansive clays often lead to their unconventional properties. The considerable changes in the soil climatic conditions, water table depth, deforestation due to numerous human activities and insufficient drainage pathways are examples of the environmental conditions (UFC, 2004).

In addition, the recurrent wetting and drying climatic conditions in the desert or semi-desert regions have also influenced the properties of expansive clay soils located around or beneath most civil engineering structures. This often triggers various types of geotechnical hazard such as movement of foundation or underlying subgrades, soil instability and distress to super structures (Puppala et al., 2007; Rao et al., 2006, 2002).

The swell, shrinkage, cracks, collapse, deformations, bulges and slips are the most common hazards observed in structures established at these zones of distress. The most obvious manifestations in buildings occur as sticking doors, craggy floors, fractured foundations, open walls, weak ceilings and windows. The cost of repair may sometimes outweigh the value of engineering structures with tremendous damage. These damages often lead to high maintenance cost, lost of property and loss of life **(**Ventkataswamy et al., 2003).

The global development resulting from an increase in world population, urbanization and industrialization demands for more land occupation. Therefore, civil engineers are forced to utilize unsuitable land occupied with expansive clay soils. This urgent need requires soil improvement with technique that can alter, modify and stabilize the physical and geotechnical properties of the deficient soils (Nelson et al., 1992).

This has prompted researchers to seek for structural alternative techniques to minimize the threats posed to mini and superstructures due to unconventional expansive soil behaviors. It has been postulated that all the ground modification techniques stabilize the soil mechanically except the chemical and thermal methods which modify/alter their engineering properties. There are severe drawbacks in the applications of mechanical stabilization methods such as high maintenance costs in long term performance (Punthutaecha, 2002; Nelson et al., 1992).

2.3 Stabilization Techniques for Expansive Soils

Lime stabilization has been the most prominent method used to stabilize expansive clay soils. It is highly applicable in the region that lacks good soil aggregates or satisfactory soils by transforming them to sound earth construction subbase and subgrade. These deficient soils incur damages to many civil engineering structures more than to many other natural disasters like earthquakes, landslides and floods etc. The term modification implies a minor alteration in the properties of the deficient soils while stabilization means adequate alteration in the properties of the soil to allow field construction to take place. Therefore, soil stabilization is a key tool to improve deficient soils for better utilization and applications (Krohn et al., 1980; Jones et al., 1973).

Generally, soil stabilization is the state of the art technique utilized by civil engineers for the improvement of engineering properties of an in-situ deficient soil to be built-on, built-in, and supported-with, by the use of mechanical compaction, thermal techniques and/or addition of chemical additives to attain the desirable engineering properties suitable for various civil engineering structures. Other alternatives include strengthening of foundations, excavating the problematic soils and relocating the engineering projects. However, these alternatives are considered impractical, time consuming and costly when compared to the chemical stabilization techniques (Puppala et al., 2007).

Deep lime stabilization is a globally accepted ground modification technique for the benefit of improving, modifying and stabilizing the engineering properties of various deficient soils such as expansive soils. Generally, soil stabilization improves the soil shear strength and bearing capacity (Rogers et al., 1997; Tonoz et al., 2003; Larsson et al., 2009) and it has been utilized for controlling erosion (Macham et al., 1977).

2.3.1 In-Situ Lime Stabilization Techniques

Lime is the most dominant and globally accepted stabilizing agents in the engineering practice since time past with its effectiveness over an extensive range of soils, to control swelling and improve their strength. It is applicable on fine to medium grained clay soils (Petry et al., 2002; Little, 1995).

Soil improvement is aided by the development of a series of electro and physicochemical reactions which acting at a microstructural level, stabilize from a macroscopic point of view the physical and engineering characteristics of the soil. The mechanisms involved in the soil-lime reactions are hydration, cation exchange, flocculation, pozzolanic reactions and carbonation (Tonoz et al., 2003; Larsson et al., 2009).

Chemical stabilization known as the oldest method of improving the soil is basically the mixing of the natural soils with binders of dry powder, liquid or slurry with the desire of improving, controlling and maintaining their volume-change properties, shear strength, stability, stress-strain behaviors, permeability and durability (Tedesco et al., 2006).

Jacobson (2003) in his literature reported that lime is formed from natural limestone and that the distinct form of lime is a function of its production process. He reported five basic kinds of lime which comprises:

- High-calcium quicklime (CaO),
- Dolomitic quicklime ($CaO + MgO$),
- Hydrated high-calcium lime ($Ca (OH)_2$),
- Normal hydrated dolomitic lime ($Ca(OH)_2 + MgO$),
- Pressure-hydrated dolomitic lime ($Ca(OH)_2 + Mg(OH)_2$).

The most extensively accepted and best effective limes in soil modification of expansive soils are high-calcium quicklime and hydrated lime. Over the years, research has indicated that the former type usually provides a more desirable stabilizing effect (Basma et al., 1998; Little, 1995). Quicklime yields higher curing temperatures and absorbs more moisture than slaked quicklime due to his large surface area, resulting in a quick strength gain in the clay soil (Ahnberg et al., 1995; Jacobson, 2003; Tonoz et al., 2003).

The other stabilizing agents utilized by researchers are bitumen, cement, fly ash, industrial wastes, phosphates, chlorides, thermoplastic or geosynthetics materials, agricultural wastes such as groundnut shell ash, baggasse ash, rice husk ash, wood ash. Researchers have clarified that as far as stabilization is concerned, lime was found to be a higher quality stabilizing chemical additive (Tonoz et al., 2003; Basma et al., 1998). Lime has been extensively utilized in construction of roads by intimate mixing with clay subgrades (shallow mixing technique) which is highly exceptional for lessening swell-shrink potential, plasticity and provides considerable strength and workability in expansive soils with time (Rogers et al., 1997; Nelson et al., 1992).

The numerous studies by Nalbantoglu et. al., (2001), Little, (1995), Basma et al. (1991), indicated that the modification in soil-lime admixtures were directly proportional to many variables like soil type, lime content, lime type, curing time, water content and unit mass (Tonoz et al., 2003, Bozbey and Garaisayev, 2009).

In his literature, Jacobson (2003) also reported that the lime stabilizing power depends on the curing time, curing temperature, curing humidity, confining pressure and freeze/thaw cycles. Chemical stabilization of expansive soils with lime has been proved to be effective, economical and safe. They are categorized into shallow and deep mixing stabilization techniques.

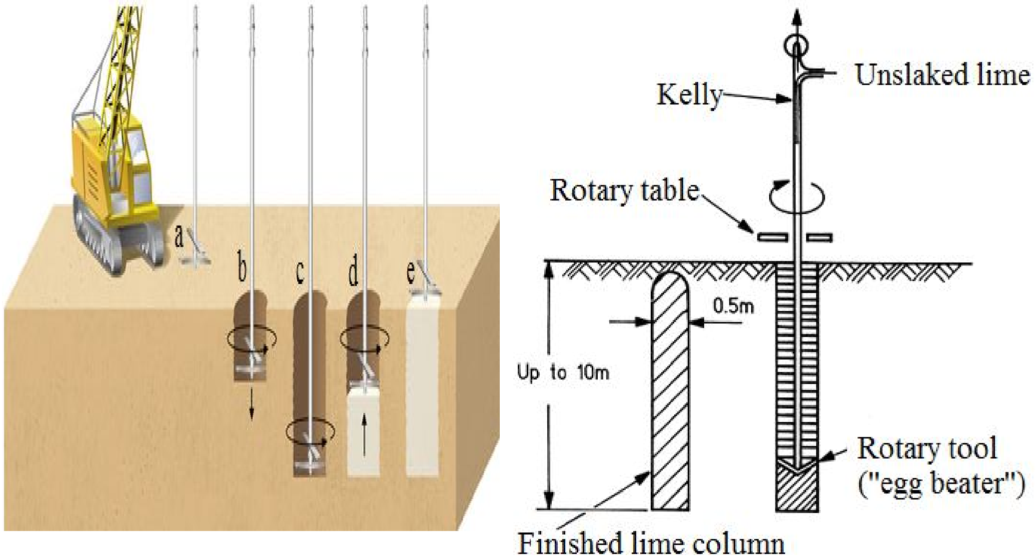
2.3.2 Deep Mixing Techniques

Lime stabilization has been utilized in shallow improvement techniques prior to the construction of highways and foundation subgrades through an in-situ mass mixing and recompaction to improve workability, strength and bearing capacity. These techniques penetrate the very low depth and they are limited to subsurface stabilization for subgrade applications (Puppala et al., 2007; Nelson et al., 1992).

Modern deep ground stabilization techniques capable of improving the deficient soils to greater depth are crucially required. Deep ground mixing is a basic terminology for a high range of techniques in which chemical agents, mostly lime, cement, or fly ash etc. is mechanically injected into the soil (Larsson et al., 2009; Terashi, 2003; Tonoz et al., 2003; Porbaha, 1998; Rogers et al., 1997). Bruce (2002) stated that their admixtures yielded entirely a higher strength and decrease compressibility than the natural soil. They further substantiated that the properties obtained from the stabilized soils depend on the properties of the native soil, the mixing techniques and the binders utilized.

Al-Tabbaa (2002) summarized the applications of deep mixing methods as groundwater control, foundation stabilization, liquefaction mitigation, fixation of contaminant etc. Other benefits are low noise pollution prior to or during construction, utilization on wide soil conditions, shortening of project duration, reduction in off-site waste disposal problems etc. Many researchers (Broms et al., 1975; Porbaha, 1998; Tonoz et al., 2003; Larsson et al., 2009) have extensively reported about the deep-mixing lime stabilization techniques as universally recognized ground modification techniques with remarkable successes.

For instance, lime columns are utilized to reinforce soft clay deposit for deep foundations, highway subgrades etc. The basic idea of this method is to form an in-situ vertical holes of up to 0.5m in diameters and 10m or greater in depth of thoroughly mixed quicklime and soft clay**,** which interact together to produce columns of admixtures with greater strength and lower compressibility than native soil, Figure 2.1 (Rogers et al., 1997).



(a) (b)

Figure 2.1: (a) Schematic illustration of a Field Deep Mixing Method for Ground Improvement (http://rebar.ecn.purdue) (b) Schematic diagram of the laboratory technique for lime columns construction (Broms et al., 1979; Rogers et al., 1997)

Secondly, the lime slurry pressure injection (LSPI) techniques involve the introduction of lime gel or slurry into the ground, by forcing the slurry into the pores, cracks and fissures of deficient clay soils under very high pressure as shown in Figure 2.2. It is utilized in deteriorated retaining walls and embankment slopes aiding treatment by migration due to the entrance of slurry (Rajasekharan et al., 1997).

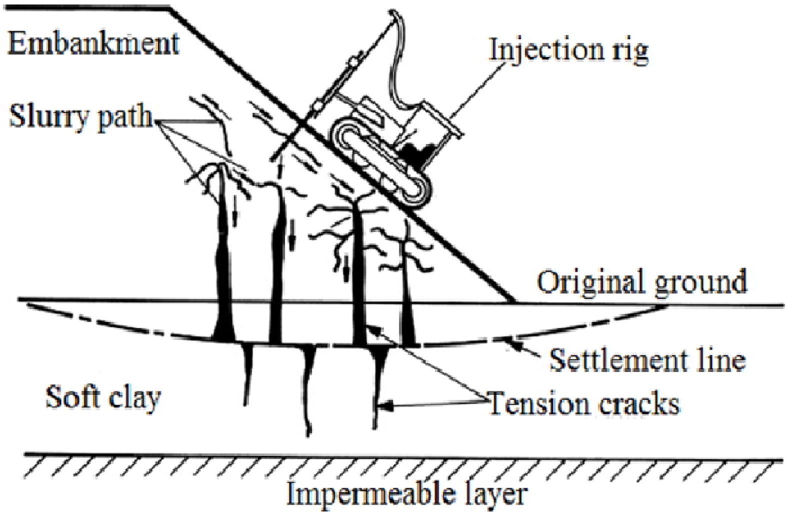


Figure 2.2: Schematic diagram of slope stabilization using (LSPI) technique (National Lime Stabilization, 1985; Rogers et al., 1997)

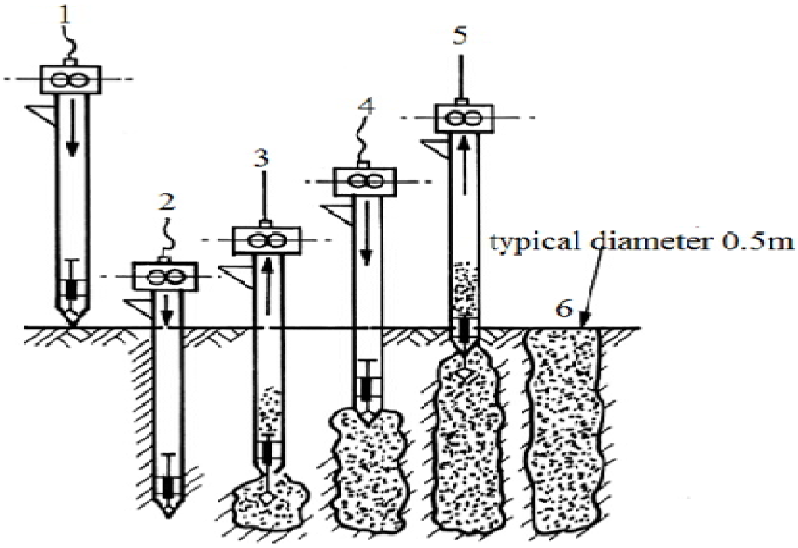


Figure 2.3: Schematic diagram of procedure for lime piles assembly in deficient soils (Ingles et al., 1972; Rogers et al., 1997)

Finally, Figure 2.3 indicates the lime pile technique which comprises columns in the ground filled with lime. This technique has been applied in the USA and Scandinavia countries as a technique of slope stabilization and in Scandinavia countries as a ground modification mechanism for soft soil. Interchangeably, the terms lime-column and lime pile have been used synonymously, in many laboratory studies by various researchers in the past (Rogers et al., 1997; Kitsugi and Azakami, 1982).

The choice of deep improvement techniques for any site is unique and depends on the structure, stress history of the clay and on the objective for which stability and swelling improvement are required. According to the past researchers, the soil improvement is also attributed to lime hydration and the geomechanical principle of this modification is a transfer of interparticle forces between the piles or columns and surrounding natural soil (Rogers et al., 1997; Bozbey and Garaisayev, 2009).

Rogers et al., (1997) stated that the stabilization mechanisms produced by the lime-piles are governed mainly by lime diffusion. They further reported that due to the low hydraulic conductivity of the clay soil, lime (calcium and hydroxide ions) migrates from the piles or columns slowly into the soil. The lime reacts with the soil causes highly alkalinic condition and stabilizes the surrounding soils. The highly alkalinic environment produced compounds of alumina-silicates, a highly precipitated hydrated binding agent. These end products resulted in flocculation by cementing individual clay soil particles together and as a curing time increases the surrounding clay soil gets more strengthened. Such pozzolanic reactions are dependent on time and the strength developed slowly and spontaneously over a long period of time (Rogers et al., 1997; Tonoz et al., 2003; Larsson et al., 2009).

2.4 Geotechnical Properties of Stabilized Expansive Clays

A recap of geotechnical and physicochemical properties of deep mixing lime stabilization techniques (lime columns, lime piles and lime slurry) on deficient soils conducted by various researchers are discussed in this section.

2.4.1. Index Properties of Stabilized Expansive Clays

The Atterberg limits of clay soils such as plastic, liquid and shrinkage limits have correlations with their engineering properties such as shear strength, swell-shrinkage, hydraulic conductivity, compressibility etc.

In the literature, comprehensive investigations are restricted to the persistent characteristics of the naturally deficient soils when stabilized with lime at a greater depth. Researchers have emphasized that the Atterberg limits are dependent on the moisture content, kind and quantity of clay minerals (Rogers et al., 1997). According to Bell (1996) and Sridharan et al. (1997), the Atterberg limits of clay soil also depend on the shearing resistance and the size of the diffuse double layer (DDL). Atterberg limits give basic information on the improvement of index properties of stabilized clay soils (Rogers et al., 1997; Tonoz et al., 2003; Ahnberg, 2006; Larsson et al., 2009).

Agus et al., (2006) in their investigation of shear strength around soft clay surrounded by lime columns, stated that the moisture content of the surrounding soil decreased after producing the lime columns. They attributed this to the usage of water for lime-soil chemical reactions. Larsson et al., (2009) also provided a detailed scenario of hydration reactions with a reduction in the water content of lime stabilized kaolin clay at different lime-column distances and different curing periods ranging from 7, 14, 28 and 56 days respectively.

Tonoz et al. (2003) investigated extensively the performance of the lime-column on the Ankara clay by comparing the engineering properties of the stabilized and native soils in terms of lime-column distances and curing time. In their study, they established that the longer the curing time for clay-lime reactions and shorter the lime-column distance, the higher the flocculating effects which reduced the percent of clay particles and decreased the plasticity. They emphasized that the clay particles exhibited the form of floccular aggregates after stabilizing with lime. The aggregate exhibit the behavior of silt particles, became more granular and easily worked with. They concluded that the distinct reduction in the clay particles of lime-stabilized soils range in between 20 and 40 % which depends on the curing time and effective within a distance twice the radial dimension of the lime columns.

Larsson et al. (2009) reported that lime-column stabilization also favored coagulation and resulted in reduced plastic and liquid limits. According to their findings, the saturation of kaolin with Ca^{2+} ions generated open-structured aggregates which resulted to increase in the liquid limit as the water became encapsulated within their voids. They concluded that the liquid limit was far more sensitive to the alteration of the cation concentration than the plastic limit of the same kaolin utilized in their investigation.

Most researchers concluded the liquid and plastic limit data of the lime treated soil samples were satisfactory after 28 days of curing and produced a remarkable improvement. The plasticity greatly reduced with an extension of the days. They proposed that this is achievable at a distance same as twice the diameter of the utilized lime column(s) or lime pile(s). The overall outcome was the reduction in the liquid limit, plastic limit and plasticity index, with close distance towards the lime column(s) and vice versa with the subsequent increase in distance away from the lime column(s). They stated that it is a time-dependent process (Tonoz et al., 2003; Larsson et al., 2009).

Paige-Green et al. (1999) evaluated various bar linear shrinkage tests and established them to be more accurate and precise test to indicate the capability of material than the common Atterberg limits. Cerato et al., (2001) in their own study stated that the most precise method is the direct measurement of linear shrinkage from the bar linear shrinkage test, but suggested it would be better to acquire at least one measurement using the ASTM standard D-427 or BS 1377 : 1970.

2.4.2. Volume Change Behavior of Stabilized Soil

Katti (1978) indicated that the expansive soils exhibit swelling properties when come in contact with water and shrinkage when subjected to drying conditions. These deficient properties in clay soil cause tremendous damages to engineering structures due to recurrent volume change resulting from periodic moisture fluctuations. The volume change mechanism is linked entirely to the availability of smectite clay minerals in the soil (Lambe et al., 1962, 1979; Komine et al., 1996).

Terzaghi (1925) in his study of quantitative description of compression in relation to the effective stress of soil stated that volume change behaviors are important indices in determining the degree of settlement, strength and deformation properties which indirectly influence the soil stability. He stated that the compressibility characteristics of pure clays are significantly dependent on DDL repellent forces. These intraforces between the particles are due to the availability of cations exchangeable (Mitchell, 1993).

Researchers (Nalbantoglu et al., 2001, 2006; Kate, 1998 and Cokca, 2001, Katti, 1978) reported the decrease in swell potential in various expansive clays using lime and/or fly ash admixtures for a reasonable period of time. The reduction in swelling percent as a result curing is associated with the pozzolanic and self-hardening properties which are directly time-dependent.

Tonoz et al. (2003) has indicated that lime-column production resulted in a sudden decrease in swell pressure. They emphasized that due to lime-clay physicochemical reactions there was formed of floccular aggregates which produced a chemically produced preconsolidation effect, causing an exceptional reduction in the compressibility properties. In their study, reduction in swelling pressure of between 40% and 75% were obtained within the dimension of twice the lime-column(s) diameter.

Kitsugi et al. (1982) presented a case study in which lime-pile technique was utilized to reduce settlements beneath considerable high embankment. They stated that the maximum decrease was obtained within a distance same as the lime-column diameter and slightly increases away from the column.

Rao et al. (1997) and Ventkataswamy et al. (2003) concluded that the swelling potential drastically reduced from lime stabilization due to increased pore salinity and exchange calcium ions which subsequently caused a decrement in the dimension of the diffused ion layer. Ventkataswamy et al. (2003) further elaborated in his study that the swell potential is significantly lower at a distance five times greater than the lime-column size used.

Rajasekaran et al. (2000) in his study reported that lime slurry pressure injection (LSPI) reduced the swelling potential of expansive soils.

Nevertheless, some researchers have reported that sulphate-bearing expansive clay soils treated with lime may result in the formation of highly crystalline expansive minerals ettringites, and thaumasite which can cause enormous heave, expansion and compressibility in the soil. This phenomenon is tagged ‘sulphate attack’ (Mitchell, 1976).

2.4.3. Hydraulic Conductivity of Stabilized Soil

Rajasekaran et al. (2000) extensively discussed the hydraulic transmission of the lime stabilized marine clay. In their literature, the hydraulic conductivity was defined as a measure of the ease in which fluid travels through the soil particles. They evaluated the stabilizing potential of lime in connection to stability and settlement analysis. Hydraulic conductivity is interpreted by Darcy’s law as:

$$v = ki \tag{2.1}$$

where v, k, and i symbolizes the velocity, hydraulic gradient and hydraulic conductivity respectively. Budhu et al. (1991) propounded the determination of the coefficient of hydraulic conductivity using the indirect method from a one-dimensional consolidation test with the relationship below:

$$k = c_v.\mathrm{m}_v.y_p/ (1 + e) \tag{2.2}$$

They stated that this indirect approach saves time and gives high precision and accuracy. They postulated the hydraulic conductivity is a function of consolidation coefficients (c_v), compressibility coefficient (m_v), unit weight (y_p) of the fluid and void ratio (e). Mitchell (1976) propounded the hydraulic conductivity of a soil phase is also dependent on its grain size, electrolyte concentration, external applied pressure and fabric arrangement of soil particles. Ranganatham (1961) indicated that the micro-structural distribution of soil and soil pore orderliness and distribution influence the hydraulic conductivity of fine grained soils. He stated that these factors depend on mineralogical composition, cations exchange and pore fluid chemical interactions of soil systems.

Bujang et al. (2010) in their compressibility study of behavior of peat treated by a deep mixing method reported compressibility coefficients reduced with increased in stabilizing agents, curing time and closer distance to the lime-column used. They stated that this was due to the formation of aggregate particles during hydration, pozzolanic, and cation exchange reactions after the chemical additives used to interact with the soil-water system.

Rajasekaran et al. (2000) in their permeability investigation of lime improved marine clay, achieved a radial increase in both permeability and shear strength of the treated soils. They emphasized that calcium chloride and quicklime produced the best improvement for soil engineering properties and concluded that lime-treated marine clay had 15 times and 12 times improvement in hydraulic conductivity at 8cm and 12 cm respectively from the quicklime-clay columns used after 45 days of curing.

2.4.4. Shear Strength of Stabilized Soil

The unconfined compressive strength is one of the recommended tests for determining the needed quantity of binder(s) to be used in the stabilization of soils. The use of lime, cement and fly ash in soil stabilization has remarkably proved to be more effective and economical and have provided the required strength required for vast engineering works (Singh et al, 1991).

Agus et al. (2006) in their study of strength distribution in soft clay surrounded by the lime columns stated the lime column improved the strength of soil surrounding it in both the circular and vertical directions. They emphasized that the strength gained was more pronounced in the vertical direction than in the radial direction. They concluded that lime migrated from the column up to four times the column diameter (4 x D) in radial direction and penetrated deeper eight times (8 x H) in vertical direction into the soil.

They showed that the main effective zone was within two times (2xD) diameter in radial direction and four times (4xD) deeper in vertical direction. Greater strength was achieved close to the column and reduced steadily with the distance from the column and this is dependent on the migration of lime.

Larson et al. (2009) in their study commented that migration of Ca^{2+} ions from the utilized lime-cement columns had a remarkable impact on modification of the undrained shear strength characteristics of kaolin surrounding the columns. Rogers et al. (1997) in their study on the improvement of clay soil with lime-pile attributed the improvements of expansive soil bearing capacity mainly to strength from the lime-pile. This strength improved was catalyzed with an addition of calcium silicate or aluminate with the lime and it is dependent on the confining pressure of the surrounding soil.

Tonoz et al. (2003) further emphasized that the greater the smectite content in the clay soil, the greater is the strength obtained when treated with lime. They also showed lime-columns chemically incurred preconsolidation effect causing an increase in the vertical effective stress. These are pronounced at the distance twice the diameter of the column. Conclusively, they reported there was a 80% increase in strength gained. Their stress-strain curves also proved the stabilized soil behaved in a slightly brittle feature resulting from physicochemical reactions.

Rajasekaran et al. (2000) in their permeability investigation of lime improved marine clay, determined the strength of the lime stabilized marine clay utilizing a falling cone technique and indicated a 10 times increase in strength gained within 30 – 45 days of curing.

2.4.5. Electrical Resistivity and Electrical Conductivity of Compacted Soil

Electrical resistivity measures the impediment of electric current through a material, while the electrical conductivity as a reciprocal measure the ease in which electric charge flow through a material. Both quantities have correlations and applications in the evaluation of the index and geotechnical engineering properties of compacted soils. They have profound applications in resistivity imaging for subsurface site mapping and geophysical techniques and in the study of compacted soil behaviors **(**Abu-hassanein et al., 1996).

Electrical conductivity had been studied since the time past by soil scientist in the fields of agricultural (soil) science (Rhoades et al., 1981; De Jong et al., 1979). These two fundamental properties, electrical resistivity and electrical conductivity govern the ability of all materials to transmit ions and electric charges. In their dry state, soil impedes the flow of electric current; therefore, conductance property of a material is only visible in electrolytic solutions, water bearing soils and rocks via the ions in the solution (Abu-hassanein et al., 1996).

Many factors influence the electrical resistivity such as temperature, degree of saturation, pore size, shape and distribution and structure, water composition, organic content, geologic formation, and operating frequency. The surface charge of clay minerals and moisture interactions caused the ionic content and soil fabric to have a significant effect on electrical conductivity (Abu-hassanein et al., 1996).

Indirect approach of determining the electrical conductivity had been performed on a compacted and stabilized soil block in this study by remodeling the laboratory setup of Abu-hassanein et al. (1996). In their study, they determined the vertical electrical resistivity of the compacted soil with a simple laboratory apparatus. There has been limited research conducted on electrical conductivity of the compacted soil and lime-stabilized soils.

According to Archie (1942) principle, the electrical resistivity (ER) (ρ) in the compacted saturated soil is a function of its porosity (n) and the soil or rock types denoted with constants a and m. Abu-hassanein et al. 1996 emphasized in their investigation stated that the ER of the soil and rock types might have different cementation factor (m), even when exhibit equal pore fluids, orientation, structures and porosity (n) exist in them.

$$\rho = a\rho_w n^{-m} \tag{2.3}$$

Many researchers consider the electrical conduction via the fluid in the pores of the clean sand and gravel. However, having considered that in nature, the conditions of the soils in the field, is a typical admixture of clay, sand and gravel; therefore more research works are required. In clay dominated soils, electrical conduction can transmit both through the pores and significantly through the stern layers of each charged particle of clay minerals (Rhodes et al. 1976; Urish 1981, Mitchell, 1993; Sadek, 1993). The degree of saturation (S) also alters the electrical resistivity in the soil (Keller et al. 1996; McNeil, 1990).

$$\frac{\rho}{\rho\text{sat}} = S^{-B} \tag{2.4}$$

It has been recorded that the higher the level of saturation, the lower the electrical resistivity. According to Abu-hassanein et al. 1996, the electrical resistivity of a soil is dependent on the amount of clay size fraction (especially smectite minerals) present in the clay. In their study of electrical resistivity of compacted clays, it was observed that the higher the liquid limit (LL) and plasticity index, the lower the determined electric resistivity. When the granular fraction of the soil was increased, they recorded higher electrical resistivity.

Therefore, many more researchers also agreed to these basic facts, that the higher the amount of fines, the greater the specific surface area which proportionally improves the surface conductance. They also stated that on the other hand the electrical resistivity increases as the dimension of the granular size fraction increases (Kwader, 1985; Keller et al., 1966).

Chapter 3

MATERIALS AND METHODS

3.1 Introduction

In this thesis, the last two chapters discussed the preliminary information about this study. These chapters revealed the basic review of clay-lime physicochemical reactions, the alteration in the geomechanical properties and engineering properties of the soils.

The comprehensive laboratory program was set up to investigate the influence of lime-piles on the properties of expansive clay soils. The laboratory program comprises extensive laboratory tests with the objective of identifying, comparing, and evaluating the modification in different physical and engineering properties of the natural and the stabilized soils using lime piles. Generally, the laboratory experiments utilized in this study had been performed in conformity with the American Standards, ASTM. This chapter presents the abcs details of the methodology utilized in this study.

3.2 Materials

3.2.1 Experimental Soil

Different laboratory experiments were performed in this study on an expansive clay soil extracted from communication school backyard at the south campus of EMU, Famagusta North Cyprus. The clay soil was excavated from the depth of approximately 1.5 m to 3 m below the ground surface during the winter season.

This location was selected because there were visible damages such as cracks on the wall of the surrounding buildings coupled with the fact that it harbors a naturally occurring soil of relatively high plasticity. The geographic location (Latitude 35.14 and Longitude 33.89) of the selected area is indicated in Figure 3.1 and the physical properties of the native soil are given in Table 2. The in-situ dry density and water content values of the native soil were determined from the soil samples collected from the cylindrical metal tubes, extracted in the field.

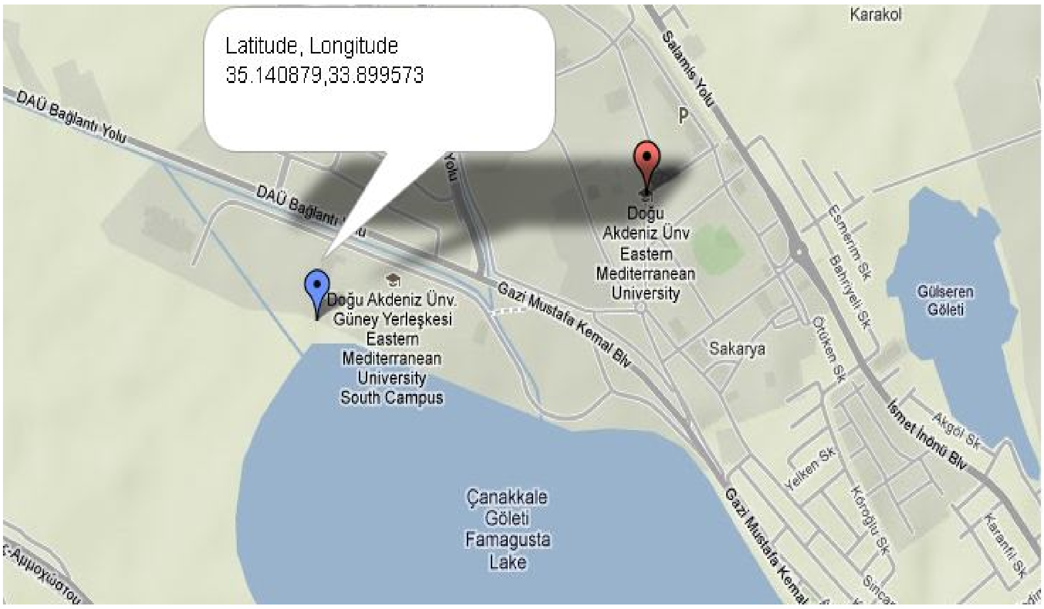


Figure 3.1: Geographic location of the clay soil

The excavated natural soil was filled in about 12 wide buckets and transported to the soil mechanic laboratory. The soil was spread in wide trays and placed in the laboratory oven at 50^{o} C for a period of seven days without altering its mineralogical and chemical compositions. The oven-dried soil was pulverized mechanically using a small laboratory mechanical grinder. The dry mass sample was mixed with water at known different percentages by mass and the specimens were kept in an air tight plastic bag for nearly 24 hours to avoid loss of water and proper curing of soil-water admixture. The 24 hour curing period facilitated an intimate curing of the soil which yielded high consistency, accuracy and precision in the measurement of the index characteristics of the natural soil.

The Standard Proctor compaction, SPC test was conducted in conformity to ASTM D 698-07, method A. From the SPC curve, the compaction characteristics, optimum moisture content (OMC) and maximum dry density (MDD) of the soil were determined.

Table 3.1: Physical and index properties of the natural expansive clay soil

Soil index properties	Quantities
In situ bulk density, ρ_b (gr/cm^3)	1.88
In situ dry density, ρ_d (gr/cm^3)	1.45
In situ water content, w (%)	30.00
Clay size fraction (< 2 μm)^a (%)	64.00
Silt size fraction (2 μm – 74 μm)^a (%)	26.00
Sand size fraction (> 74 μm)^a (%)	10.00
Fines fraction (< 74μm)^a (%)	94.00
Coarse fraction (>74μm)^a (%)	6.00
Specific gravity^b, (Gs)	2.56
Maximum Dry Density ^{c}, $\rho_{d(max)}$ (gr/cm^3)	1.49
Optimum moisture content, w_{opt} (%)	25.00
Liquid limit, LL^e (%)	68.00
Plastic limit, PL^e (%)	33.00
Plasticity Index, PI^e (%)	35.00
Liquidity index, LI^e	0.96
Activity^e	0.55
Linear shrinkage, LS (%)	20.00
pH value	8.11
Electrical conductivity σ (S·m^{-1})	368
Classification	CH

a According to ASTM D 422 - 98
b According to ASTM D 854 - 06
c According to ASTM D 698 - 07
d According to ASTM D 2487 - 00 (Unified Soil Classification System)
e According to ASTM D 4318

For the purpose of this investigation, the in situ moisture content and in situ dry density were chosen in preparing the soil for swell, consolidation, shear strength and other suitable tests. The soil was tested using ASTM standards to find its physical and engineering properties. The natural soil index properties were determined and given in the Table 3.1.

In this investigation, different types of tests were performed on the natural soil and the stabilized soils extracted from the test tanks at various lime-piles distances and different curing periods. Quicklime was selected as the stabilizing agent to study the performance of lime-piles on the properties of clay soil. The chemical composition and index properties of the lime used in this study are provided in Table 3.2.

3.2.2 Quicklime

The quicklime (CaO) is an odorless white to pale yellow/brown, caustic, alkaline and crystalline powder or solid produced from natural limestone. It is a special type of binder used in chemical stabilization techniques. The most widely utilized and most effective limes are the high-calcium quicklime and hydrated slaked limes. Research has clearly indicated that quicklime has preferable stabilizing power and produces a better stabilization effect on a long term performance.

Quicklime yields higher curing temperatures, absorbs more water and produces higher strength than hydrated lime in the clay soil around the piles or columns (Ahnberg et. al., 1995). It is a non-polar hydrophobic compound that reacts chemically with water to produce heat energy by the production of a hydrated lime, a compound that is slightly soluble in water. It is represented with a reversible chemical reaction:

$$CaO\ (s) + H_2O\ (l) \rightleftharpoons Ca(OH)_2\ (aq)\ (\Delta H_r = -63.7\ kJ/mol\ of\ CaO) \qquad (3.1)$$

Table 3.2: Chemical composition and physical properties of the quicklime

Chemical compound/attribute	values
CaO	87.20%
MgO	2.13%
Loss on ignition	1.25%
Size	12.26% (+90 μ)
Density	1148 kg/m^3

The quicklime was utilized in this study to produce the set of lime-piles in order to stabilize the clay soil in three separate test tanks for 28, 90 and 120 days of curing. The quicklime provided the divalent calcium ions needed for the clay-lime physicochemical reactions.

3.2.3 The Test Tanks

The three circular test tanks utilized in this study are made up of steel coated with silver paint to prevent rusting during the test program. The test tanks have equal dimensions of 40 cm in both the diameter and height, as shown in Figure 3.10. The test tanks have a moveable steel plate cover of 39.5 cm in diameter which readily fit into the test tanks.

The purpose of the steel cover plate was to prevent dehydration and contamination of the soil samples during the testing programs.

3.3 Experimental programs

3.3.1 Sample Preparation

3.3.1.1 Preliminary Sample Preparation

Firstly, the initial sample preparation was conducted using two small molds of 11.6 cm in height and 10.1 cm in diameter as shown in Figure 3.2. This was conducted in order to have a preliminary understanding of how the compacted clay soil would react with the lime-piles in the circular steel test tanks of larger dimensions.

Figure 3.2: The steel molds used for preliminary setup

The air dried pulverized natural clay soil sample was mixed with a known percentage by mass of equivalent in-situ water content. The wet soil sample was left aside to cure for 24 hours and was then compacted in the molds to its corresponding in-situ bulk density, leaving a few centimeters spaces in the mold above the compacted clay soil as indicated in Figure 3.3x.

Figure 3.3: The compacted clay soil in the molds

The main idea was to achieve the same field conditions using the calculated in-situ water content and bulk density. The purpose of the space left above the compacted soil sample in the mold was to provide for filling up with water to saturate the sample. Four columns were constructed in the compacted soil using a 1 cm hollow tube with openings at both ends as shown in Figure 3.4. In Figure 3.5, the columns were filled with dry quicklime powder of equal amount of weight to produce the lime piles.

After the lime piles installation, one of the molds was fully filled with distilled water in the initial space provided to make it fully saturated and the other was being flashed with the distilled water at hourly intervals which makes it an unsaturated sample. The spraying water bottle was used to accomplish the task in the latter sample.

Figure 3.4: Pictures showing four columns installation

Figure 3.5: Pictures showing the lime-piles installation

In Figure 3.6, it was observed that there were visible cracks in the partially saturated lime-treated clay soil after 24 hours. This was as a result of the lateral expansion of the lime-piles, while the fully saturated lime stabilized clay had little or no visible cracks as indicated in Figure 3.7. The clay-lime reaction requires more water that it was provided in the partially saturated sample. Installation of the lime-piles caused drying of the clay platelets, since moisture was absorbed from them to react with the lime-piles. As the hydration reaction began, the unrestrained lime-piles expanded laterally. This lateral expansion led to the lateral consolidation of the surrounding clay soil

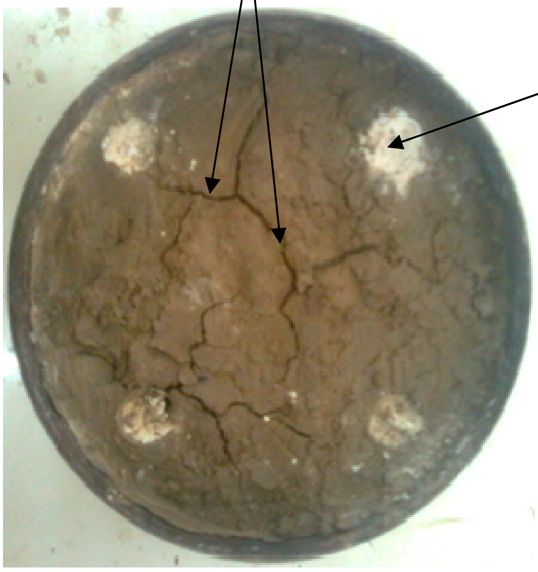


Figure 3.6: Picture indicating formation of cracks in partially saturated clay

Figure 3.7: Picture showing saturated clay with no visible crack formation

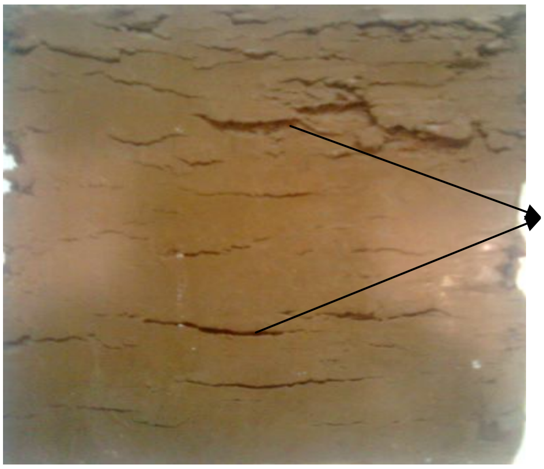


Figure 3.8: Vertical cross sectional view of cracks in the partially saturated clay

In Figure 3.8, the cracks formed in the partially saturated clay were visible from the surface to the bottom of the clay when the sample was removed from the mold and divided into two equal halves. The whole idea was to establish the facts about how the clay would behave when lime-piles were constructed in the expansive clay before the main laboratory model was set up. This was also done to determine the best approach to prevent cracks in the clay during the main experimental setup. The formation of cracks in the soil block sample is disadvantageous to this study.

Though minute cracks were noticeable in the fully saturated clay soil as shown in Figure 3.6 due to minor lateral expansion, this is still disadvantageous to this study. However, it was suggested that an additional lime-pile should be provided in the center of the column. This would probably limit the formation of the cracks developed from the lateral expansion of the lime piles in the soil block.

Figure 3.9: Picture showing the five columns installation

Figure 3.10: Picture showing the lime pile installation

The test setup was finally repeated and the fifth column and its lime pile were installed at the center of the mold as indicated in Figure 3.9 and Figure 3.10. The procedure was repeated by filling the space above the compacted soil in the mold with distilled water and it was observed that there was no crack formation. The fifth pile had neutralized the effect of lateral expansion caused by the evenly distributed four lime-piles in the compacted soil block.

3.3.1.2 Sample Preparation in the Circular Steel Test Tanks

Four experimental setups were utilized in this investigation. The setups were conducted in the typical circular steel test tanks of 40 cm in diameter and in height as shown in Figure 3.11. The known amount of the pulverized dry mass of natural soil sample was mixed with the known amount of distilled water using the initially determined in-situ moisture content. The prepared wet soil was packed in an airtight polythene bag for 24 hours which allowed an intimate and proper curing of the soil and hence prevented water loss.

In this investigation, it was required to prepare homogenous compacted soil samples in the test tanks in order to achieve the initial field conditions of the natural soil in the laboratory. In the study, 40 cm in diameter and 36 cm in height were chosen as the required dimensions to compact the wet soil sample. The required amount of the wet soil sample by weight needed to be compacted in the tank with the chosen dimensions was calculated to be approximately 85 kg. In Table 3.3, the initially determined in-situ water content and in-situ dry density were provided. This was to produce a uniform field conditions in the test tanks. The typical steel test tank and the compacted soil block are shown in Figure 3.11 and Figure 3.12 respectively.

The tank was further partitioned into four equal parts with a marker to exactly 9 cm size by height and the compaction of the wet soil was conducted in four successive layers using a tamping-static method to achieve the desired in situ bulk density.

Four centimetres in height by space was left above the soil block in the circular test tanks to allow the moveable steel plate cover, to fit into the tank for proper control of moisture.

Table 3.3: Dimension properties of the compacted wet soil in the circular steel test tanks

Dimension properties	Values
In-situ water content, (w) (%)	30
In-situ bulk Density, ρ_b (g/cm^3)	1.88
Dimension of the steel test tank (cm)	D = 40, H = 40
Dimension of the compacted soil block in the test tank (cm)	D = 40, H = 36
Volume of the soil sample in the tank (cm^3)	45238.93
Mass of the compacted soil in the test tank (gr)	85049.20
Bulk density achieved, ρ_b (g/cm^3)	1.88 ± 0.025

*H = Height *D = Diameter

Figure 3.11: The circular steel test tank

Figure 3.12: The compacted natural soil

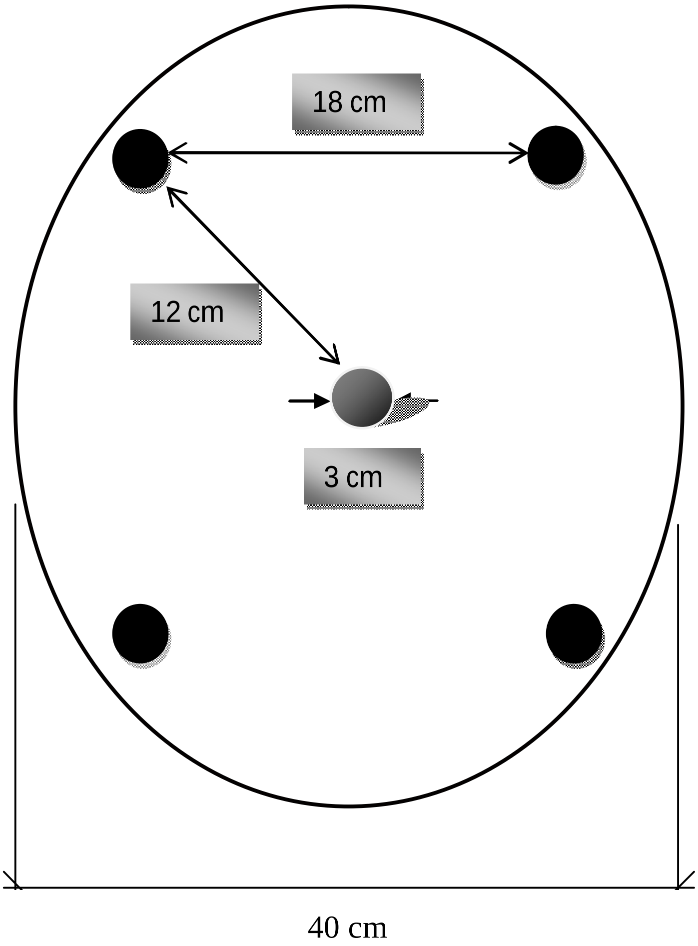


Figure 3.13: Schematic diagram showing the tank and the lime-pile dimensions

3.3.2 Lime Piles Installation

Tonoz et al. (2003) have indicated that prior to raining season is the most suitable period for the in situ installation of lime columns or lime piles, to aid the migration of ion from the column or pile to the surrounding soil. The pile dimensions used were based on the compilation of data from previous laboratory and in situ studies of lime piles and lime columns reported by various researchers. In the field application, a hollow tube is forced into the ground to the desire depth and stabilizing agent is applied forcefully into the holes by air pressure as the tube is being retracted. The laboratory simulation of this technique was conducted with the installation of five columns (Figure 3.13) in the compacted soil blocks, already prepared in the test tanks as shown in Figure 3.12. In Figure 3.14, the proposed points of installation were marked prior to the lime installation. Five columns with 3 cm in diameter and 30 cm in height were installed in each of the compacted soil blocks as shown in Figure 3.15, using a hollow polyvinyl chloride (PVC) pipe with openings at both ends.

The PVC pipe has an internal diameter of 3 cm with a greater length of approximately 40 cm. The higher length was to provide an easy penetration into the soil blocks, creating the columns and extracted undisturbed samples from the tanks. One of the advantages of using a PVC pipe is that it created smooth holes in the soil block without having to clean them with a spiral brush prior to filling with lime, this would have been done if a small hand auger was used.

The bulk density of the extracted soil samples from the soil block in the test tanks were determined to have a range within 1.88 ± 0.025 g/cm^3. The powdered form of quicklime of uniform mass was introduced into each column in definite subsequent layers and each layer was lightly compacted to form the lime piles as shown in Figure 3.16.

After the installation of the lime piles in the compacted soil block, the samples were protected with a thin perforated fiber cloth and sandy soil of size 2 mm were poured on top of the compacted soil up to the 37 cm mark by height as shown in Figure 3.17 and Figure 3.18. This was to simulate slow absorption rate of water by the quicklime, to avoid a sudden hydration reaction, in order to minimize the lateral expansion of the lime-piles and formation of unwanted cracks in the soil block. The fastened moveable steel plate of 0.5 cm thickness was positioned on the setup as shown in Figure 3.19.

Figure 3.14: The position of the four columns marked at 18 cm to each other and 12 cm to the central column

Figure 3.15: The columns of 3 cm in diameter constructed with PVC pipe in the clay sample block

Figure 3.16: Pattern of the lime-piles installation in the soil block

Figure 3.17: Showing the placement of the thin porous fiber cloth

Deformation dial gauges were positioned on top of the fastened moveable steel plate to measure any change in volume (swelling) after saturating the sample with distilled water as indicated in Figure 3.20. The empty upper part of the tank was filled with water up to a point of 40 cm mark in the test tanks. The dry lime-pile absorbed the water, released the calcium ions into the solution and diffused them into the surrounding clay soil, which aided the clay-lime physicochemical reactions. The Ca^{2+} ions then migrated from each pile position at a distance into the surrounding soil. The soil underwent a series of physicochemical reactions; hydration, pozzolanic reactions, flocculation, agglomeration and carbonization.

Figure 3.18: Showing the layer of sand soil on the porous fibre cloth

Figure 3.19: Showing the movable steel plate to cover the compacted soil block

Figure 3.20: The complete set up with deformation gauges

Figure 3.21: The treated soil after 90 days of curing with quicklime

The swelling measured by the deformation gauges was neglected in this study, as they were small values when compared to the size of the compacted soil block samples. The soil was stabilized for different curing periods of 28, 90 and 120 days. The stabilized soil became more friable, brittle and flocculated in nature. The stabilized soil after 90 days of curing is shown in Figure 3.21. At the end of each curing period, samples were extracted with suitable molds to examine the change in physical and engineering properties such as the strength, swelling, consolidation and California bearing ratio of the treated soils.

3.4 Sample Extraction

The sample extraction was conducted by using molds that are suitable for each test. Steel molds of different dimensions were pushed slowly into the soil block with the aid of a hydraulic compressive jack in the EMU structures laboratory as it is indicated in Figure 3.22. In Figure 3.23, the diameter of the molds are symbolized with a (75 mm), b (152 mm) and c (38 mm). These are the representative dimensions of the stabilized soil sample utilized in one dimensional swell/consolidation test, CBR and UCS tests respectively. The major concern during the extraction was to prevent the samples from cracking. The samples extracted from the molds were kept in airtight polythene bags before conducting the tests to prevent water loss and change in sizes.

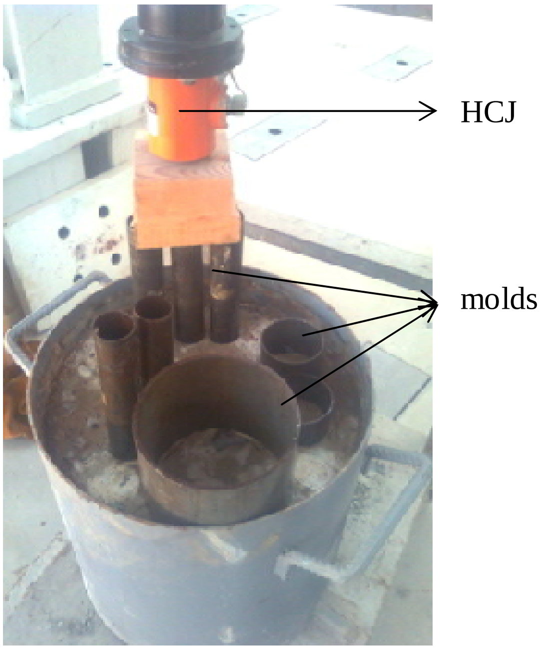


Figure 3.22: Hydraulic compressive jack (HCJ) and molds used for extraction

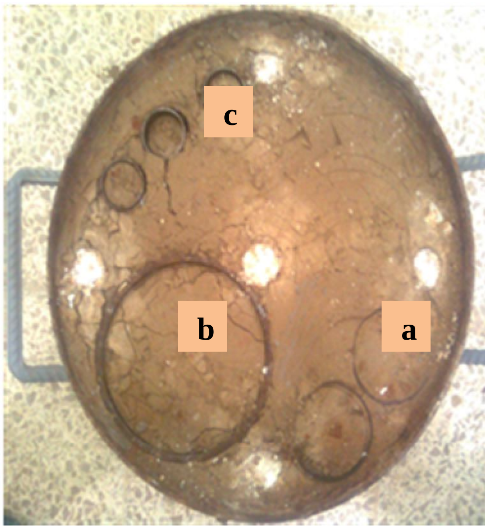


Figure 3.23: The dimension of the molds

3.5 Methods

3.5.1 Moisture Content Determination

The test was performed in conformity with ASTM D 2216. The soil specimens taken from samples extracted with a steel test tube in the field were weighed, dried in oven at 110^{o} C for one day and weighed again. The moisture content of the sample was calculated as the difference in mass of the wet and dry samples divided by the mass of the dry sample.

3.5.2 Specific Gravity Determination

The specific gravity of the soil was measured using a designation ASTM D 854. The values were determined by weighing an empty 100 ml pycnometer bottle, thereafter filled with distilled water. The chosen weight of an oven dried soil was put in pycnometer; water was added to it, till it was filled up and re-weighed again.

The mass of moisture displaced by the solid particles was calculated. The specific gravity was finally determined as the mass of dry soil divided by the mass of displaced water.

3.5.3 pH Test

The pH value of the native soil was performed in accordance to ASTM 2976-71for calculating the degree of acidity or alkalinity of the soil having fluid in its voids. Precisely 3 g by the mass of the 50 ^{o}C oven-dried soil was placed in 100 ml glass tube and 50 ml of distilled water was added into it. The mixture was stirred for 30 minutes and its pH was determined by a pH probe device.

3.5.4 Hydrometer Test

Hydrometer test was conducted in accordance to the British Standard BS 1377. The 50 g by weight of an air dried pulverized natural soil sample was mixed with the 100 ml of Calgon solution, thoroughly mixed and lightly heated in glassware. The solution was poured into a glass jar and it was shaken using a mechanical shaker for a minimum of one quarter hour.

The mixture was poured into the sedimentation glass jar and water was added until the maximum of 1000 ml in volume was reached. The tests were repeatedly performed on samples from 3 cm, 6 cm and 9 cm away from the lime-piles at different curing periods of 28, 90 and 120 days.

3.5.5 Atterberg Limit Tests

In order to determine the soil plasticity, the plastic limit, liquid limit, plasticity index, liquidity index and activity of the soil were measured in conformity to ASTM D4118-05. The Atterberg limit apparatus was utilized to quantify the liquid limit and the plasticity was determined by making 3 mm rods of soil until they began to fracture.

The texture of the natural and the lime treated soil was shown in Figure 3.24. The samples used were taken at 3cm, 6cm and 9 cm away from the lime pile in the test tanks at different curing periods of 28, 90 and 120 days. This was conducted to ascertain the effects of lime-pile on expansive clay soil based on the lime-pile distances.

(a) (b)

Figure 3.24: The textural appearance of the soil (a) Natural soil (b) Stabilized soil

3.5.6 Linear Shrinkage Test

The linear shrinkage was conducted in accordance with BS-1377: 90. The air dried soils were mixed with initial water content of 71%. This is the water content at precisely the 15 blows above the liquid limit of 61% at 25 blows derived in the Atterberg limit test.

The samples of both the natural soil and stabilized soil samples extracted from the test tank were prepared in the shrinkage molds of 140 mm in length and 25 mm in diameter. The soil was put in the brass bar in three successive layers and lightly banged lightly against a smooth surface to remove the air bubbles in the soil.

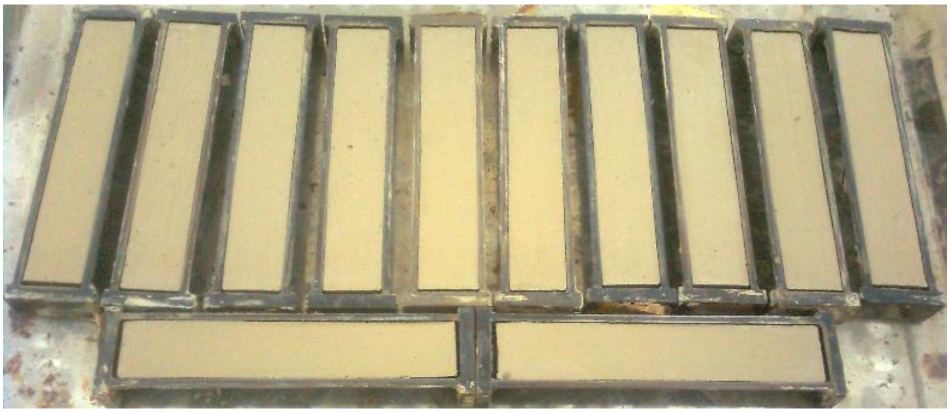


Figure 3.25: The linear shrinkage test set-up

The complete set-up is given in Figure 3.25. This is then followed by air drying the soil for twelve hours at regular hour intervals. After the 12 hours with readings being taken at successive hours, the readings were also taken at the 24, 48, 72, 96 and 120 hours before finally put in the oven to get the final change in length. The test compared the average oven dry length or diameter (change in volume) of the soil sample after shrinkage to the initial length or diameter (initial volume).

3.5.7 Standard Proctor Compaction Test

This test was conducted in conformity with ASTM D 698 to determine the compaction characteristics of the natural soil and the lime stabilized soils. The oven-dried pulverized natural soil was efficiently mixed with a calculated quantity of water and the wet soil was left for 24 hours in plastic bags to achieve an evenly distributed of moisture in the soil.

The soil was compacted with the aid of an automatic electronic compaction device in successive three layers. The undisturbed samples were used to measure the compaction characteristics of the natural and stabilized soils from the dry density versus moisture content curve.

3.5.8 Unconfined Compression Test

These tests were performed in conformity with ASTM D 2166-06 to measure the shear strength parameters of both the natural and the stabilized soils. The pulverized natural soil was dried with oven and then effectively mixed with moisture content of 30% which corresponds to the in situ water content. The moist soil was kept for 24 hours in plastic bags to achieve an evenly distributed of moisture in the soil.

The natural soil samples were compacted in the mold size of 76 mm in height and 38 mm in diameter using a static compaction method. The stabilized soil was extracted with a long steel tube at 3 cm and 6 cm distance from the lime-pile and tested at different curing periods of 28, 90 and 120 days.

3.5.9 One-dimensional Swell Test

Standard consolidometers were used to conduct the swelling tests on the native soil according to ASTM D4546-08 (method B). The samples were prepared in the ring molds with dimensions of 75 mm in diameter and 20 mm in height using a static compaction approach at the in situ water content. The stabilized clay soil samples were extracted from the test tanks at 3cm and 6cm distances from the lime pile at different curing periods of 28, 90 and 120 days. The samples were placed in the molds and saturated with distilled water.

The samples were positioned under vertical loading of 7 kPa to determine one-dimensional swelling induced for nearly 10 days. This was conducted to determine the swelling potential of stabilized soil using lime-piles based on the piles distance and curing periods.

3.5.10 One dimensional Consolidation Test

These tests were performed after the completion of the swell tests according to ASTM D 2435-04. The standard vertical loading was a day for every applied load of 1 to 32 kg and unloading of 32 to 0 kg. The alteration in the height of the inundated samples was determined at time change of 0.25 to 24 hours of each loading. The moisture content of the samples was determined after the final unloading up to 0 kg by the oven-drying method. The consolidation test for each specimen lasted for approximately 10 days.

3.5.11 California Bearing Ratio (CBR) Test

The CBR test of the laboratory compacted cohesive soil having maximum grain sizes less than (19 mm) was conducted using a designation ASTM D 1883. The soil was subjected to a surcharge pressure using a loading machine as shown in Figure 3.26, that produced the intensity of loading required for a specific maximum penetration. The penetration piston with 49.63 mm in diameter and not less than 101.6 mm long was set on the compacted specimen and the load was applied through the piston with the penetration rate of 1.27 mm/min.

The load readings at different penetrations intervals between 0.64 mm to 12.70 mm were recorded. The test was carried out on the natural soil and the tested soil in 28, 90 and 120 days curing periods. This is an index test for the determination of the shear strength and bearing capacity of the compacted soil at the chosen in situ moisture content and dry density.

(a) CBR loading machine (b) CBR molds (Soralump et al., 2006)

Figure 3.26: CBR test apparatus

3.5.12 Electrical Conductivity Tests

The apparatus used for measuring the electrical conductivity of the compacted natural and stabilized clay soils is given in Figure 3.27. The apparatus comprises a fiber cylinder mold with a size of 76 mm in height and 38 mm in diameter. This is the same as the size required for the UCS test samples. The size was chosen to be able to extract enough samples within lime-pile distances and the material used (electrical insulator) was chosen to prevent their interference with the current from the sample during the test. The electrical resistivity is the main quantity measured with the equipment, while the inverse values were calculated to determine the corresponding electrical conductivity.

The schematic diagram of the mold used and the complete laboratory set-up is given in Figure 3.26. In the figure, the alternating current (AC) used to provide an electrical voltage of 34.2 V. The differential voltage difference is measured between the copper rods (of diameter 0.5 mm) that are pinned through the samples at the center position.

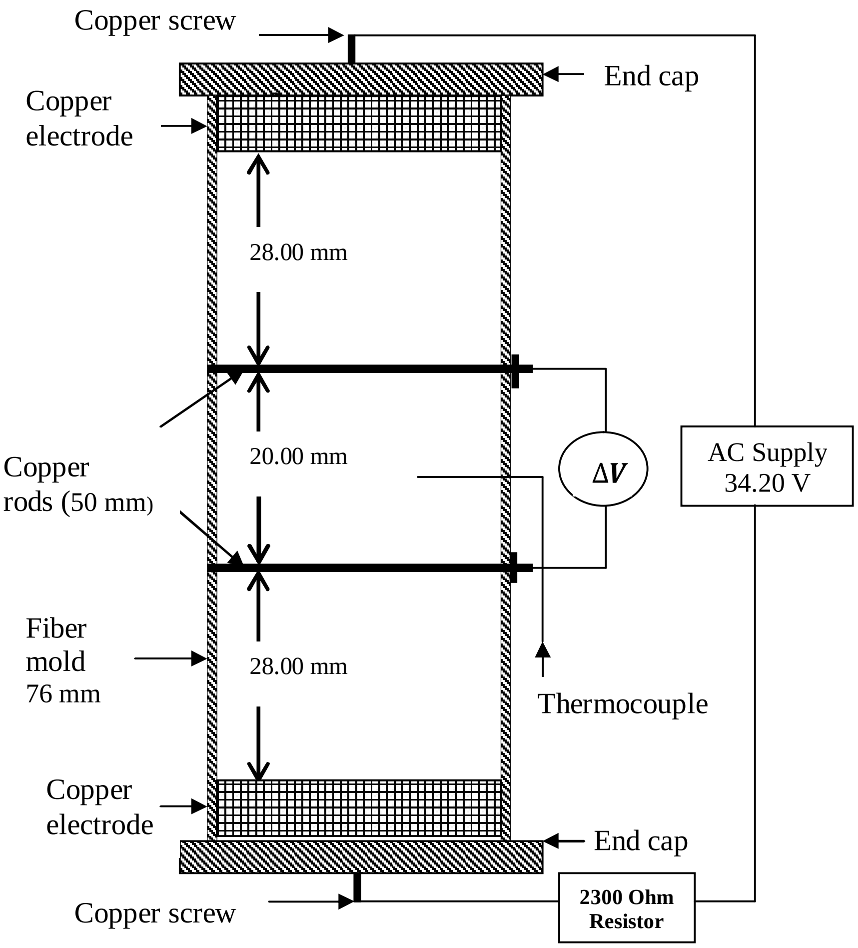


Figure 3.27: Schematic diagram of the modified apparatus used for measuring ER of the stabilized soil (Abu-hassanein et al., 1996)

A voltmeter device of high accuracy was used to determine the potential difference between the copper rods distances (20 mm). The electrical resistance (R) was determined using Ohm's law with the consideration of one dimensional electric field. The electric current, I is measured indirectly by measuring the reduction of voltage in the 2.3kΩ resistor connected to the soil samples.

$$R = \frac{\Delta V}{I} \tag{3.1}$$

The vertical electrical resistivity (ER) of the soil is calculated via electrical resistance using:

$$\rho = \frac{RA}{L} = \frac{\Delta VA}{IL} \qquad (3.2)$$

Where L = length between the copper rods and A = the copper rods cross-sectional area.

Chapter 4

RESULTS AND DISCUSSIONS

4.1 Introduction

The laboratory program was conducted with respect to the methodology explained in Chapter 3. In this chapter, comprehensive results and analysis of all experimental investigations are given in details. The results of experimental analysis are divided into two sessions. The first session presents the physical and engineering properties of the native soil used. The last section provides comprehensive information about the effects of the lime-pile on the physical and engineering properties of the extracted stabilized soil samples based on curing periods and lime-pile distances.

4.2 Properties of the Natural Soil

The index characteristics of the native soil were determined from the soil sample obtained from the field, while the engineering properties were determined from the compacted soil block prepared in the test tanks with the chosen density and moisture content indicated in Table 3.3. The comprehensive results and discussions of the hydrometer analysis, standard Proctor compaction test, Atterberg limits test, linear shrinkage test, unconfined compression test, swell and one-dimensional consolidation test, pH test, CBR test and electrical resistivity test are all presented in this section.

4.2.1 Hydrometer Test

The results of hydrometer test are given in Figure 4.1. As it is indicated on the graph, the native soil comprises 64% of clay size particles, 26% silt, and 10% sand. The results of the test show that the soil comprises more than 50% of fine grained soil fractions of silt and clay.

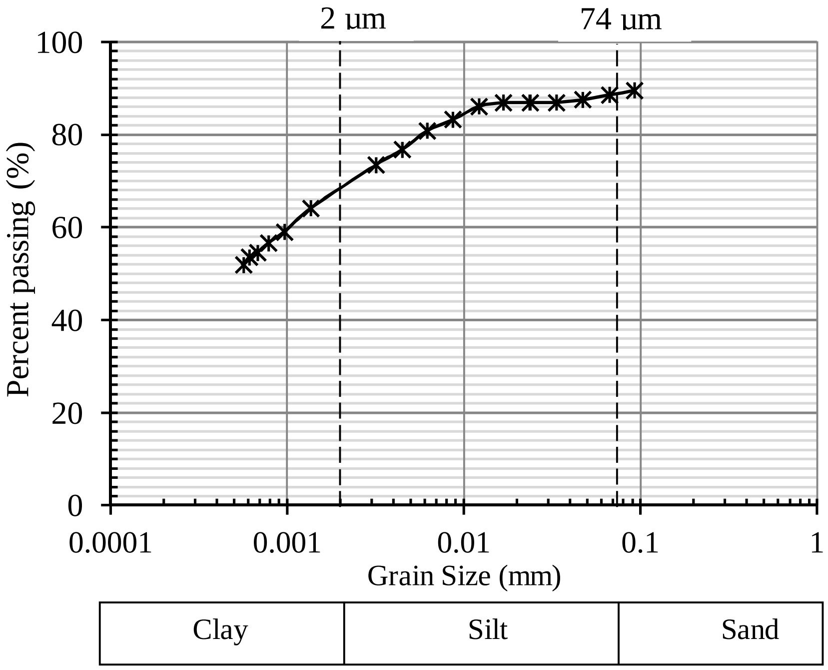


Figure 4.1: Grain size distribution of the native soil by hydrometer analysis

4.2.2 Atterberg Limit Test

Atterberg limit tests conducted in conformity with ASTM D4318 - 05 to determine Atterberg limits of the natural clay soil. The plastic limit and liquid limit of the natural soil were determined as 33% and 68% respectively. The plasticity index of the soil was determined to be 35.

In the USCS, the native soil was located above the A-line as indicated in Figure 4.2. Conclusively, this illustrates that the native soil falls in CH group which is predicted to be inorganic clays of high plasticity.

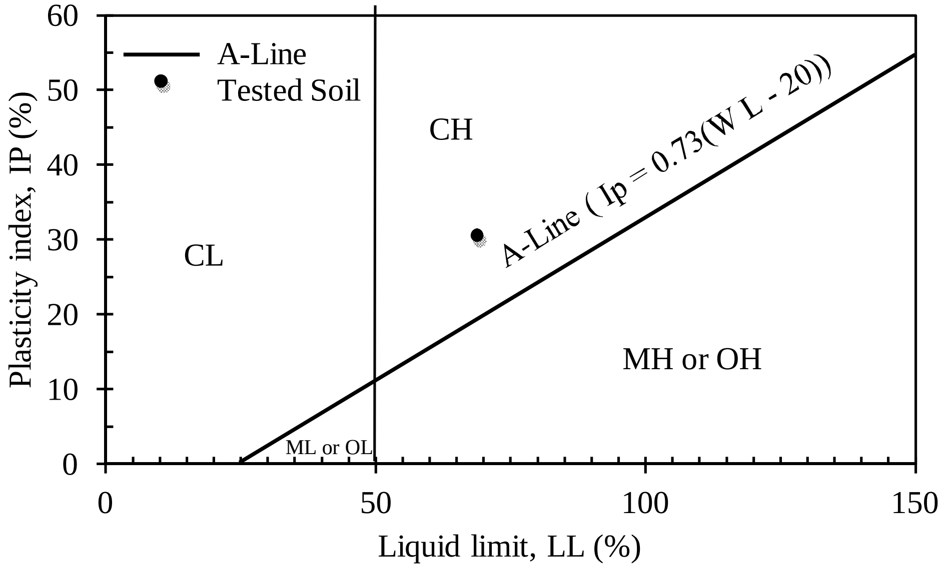


Figure 4.2: USCS plasticity chart for the natural soil

4.2.3 Linear Shrinkage Test

The linear shrinkage limit test was performed in accordance with BS-1377: 90 to calculate one-dimensional linear shrinkage. In Figure 4.3, the linear shrinkage curve shows the relationship between the calculated linear shrinkage values versus the water content of each change in length. The linear shrinkage of the natural soil was determined to be 20% using equation 4.1and indicated in Figure 4.3. The volumetric shrinkage was calculated to be 48% using equation 4.2.

The change in length and diameter of the soil was determined three times with digital Vernier's caliper to determine the average dimension which was used to measure linear shrinkage using equation 4.1.

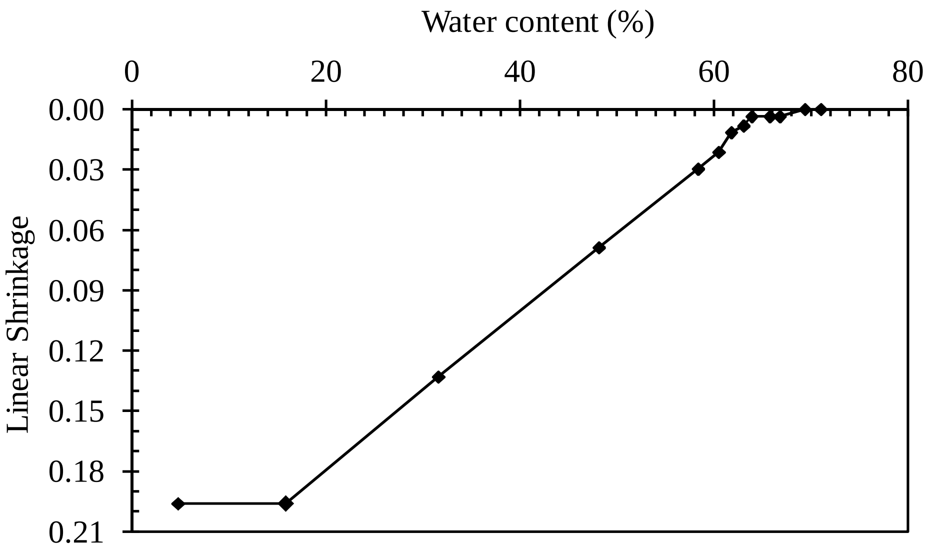


Figure 4.3: Linear shrinkage limit curve of the natural soil

$$\text{LS} = (1 - \frac{Lavg}{Lo}) \times 100 \qquad (4.1)$$

where:

LS = Linear shrinkage (%)

L_{avg} = Average length (mm)

L_o = Initial length of brass mold (mm)

The volumetric shrinkage was calculated from the linear shrinkage test with the assumption of uniform 3-D shrinkage using the equation 4.2.

$$\text{VS} = (1 - \frac{Vf}{Vo}) \times 100 \qquad (4.2)$$

where VS = Volumetric shrinkage (%).

$$V_o = 0.5\pi r_o^2 L_o = \text{initial volume (mm}^3) \qquad (4.3)$$

where:

L_o = initial measured length (mm)

π = constant = 3.14

r_o = initial calculated radius assuming 3D shrinkage (mm)

$$V_f = 0.5\pi r_f^2 L_f = \text{initial volume (mm}^3\text{)} \tag{4.4}$$

where:

L_f = final measured length (mm)

r_f = final calculated radius assuming 3D shrinkage (mm)

4.2.4 Standard Proctor Compaction (SPC) Test

In Figure 4.4, the compaction curve of the native soil was given.

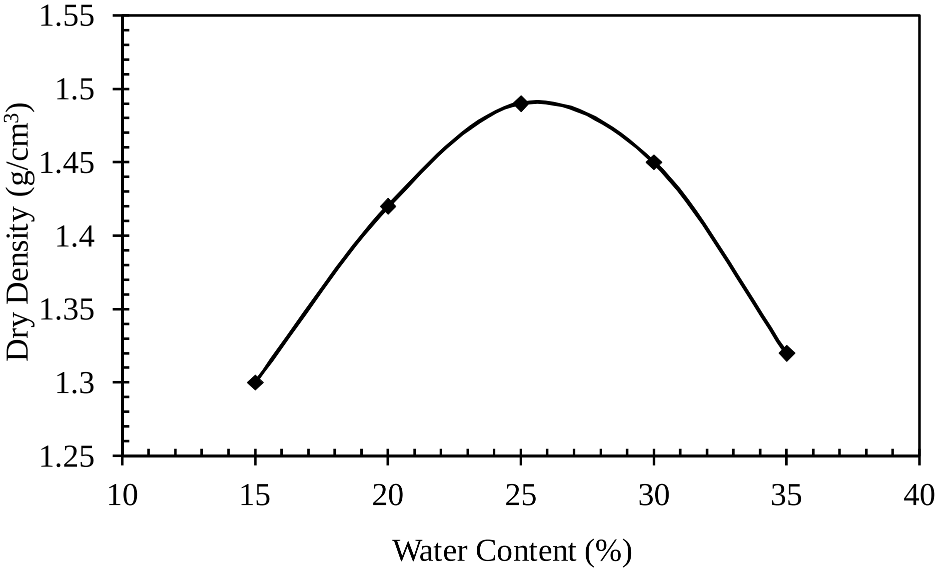


Figure 4.4: SPC curve of the natural clay soil

The results of this test were utilized to measure the compaction characteristics of the soil which are the OMC and MDD. The MDD and OMC were determined to be 1.49 g/cm^3 and 25% respectively.

4.2.5 Unconfined Compression (UC) Test

The UC test was conducted in accordance with ASTM D 2166-06 to measure the undrained cohesion of the soil and shear strength. In Figure 4.5, the curve obtained from the UC test was given. In Figure 4.5, it is indicated that the UCS, q_u of the natural soil compacted at the in situ water content was 195 kPa.

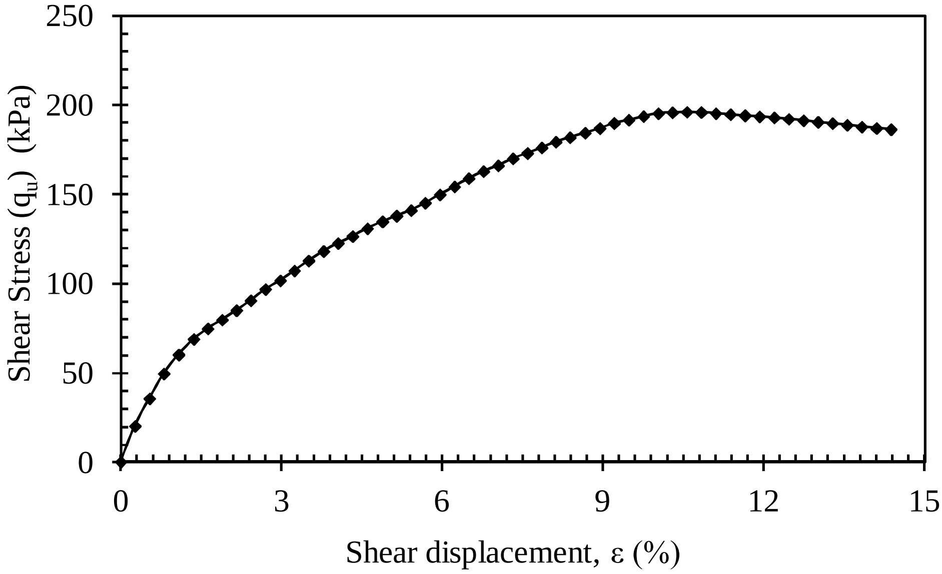


Figure 4.5: Stress-strain diagram for the natural clay

The undrained shear strength parameter of the soil (c_u) which is equal to the half value of the unconfined compressive strength ($q_u/2$) was calculated as 98 kPa.

4.2.6 Swell Test

This was performed in conformity with ASTM D 4546-08 (method B) by utilizing the mechanism of the one-dimensional odometer apparatus under 7 kPa surcharge weight. The percent swelling was calculated by dividing the dial gauge top deformation values by the original height of the specimen in the oedometer ring. This test was conducted for a period of approximately 10 days until the equilibrium was reached. The maximum primary and secondary swell potentials of the natural clay soil are represented in Figure 4.6 and determined to be 4.0% and 4.4%.

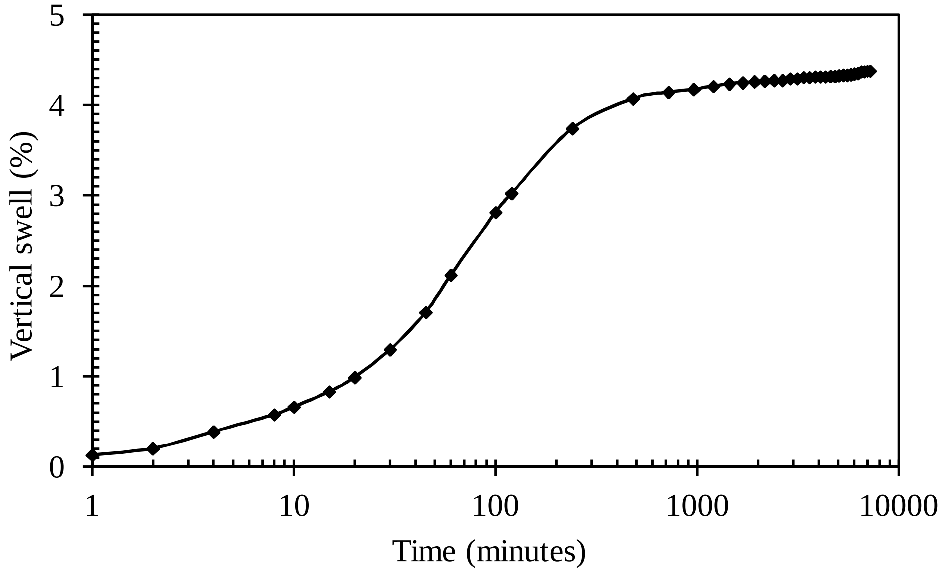


Figure 4.6: Swell-time curve for natural clay soil

The swell pressure was determined to be 180 kPa and the preconsolidation pressure was determined to be 95 kPa from a consolidation curve given in Figure 4.6 and Table 4.1.

4.2.7 One-dimensional Consolidation Test

This test was conducted on the native soil using a standardized one dimensional-oedometer in conformity with ASTM D 2435-04. The change in void ratio was calculated for each applied stress after the duration of 24 hours.

The values obtained were plotted using semi-logarithmic scale and the void-log pressure curve was drawn. The coefficient of consolidation (C_v), coefficient of volume changes (m_v), coefficient of compression (C_c) and rebound index (C_r) was determined from the consolidation curve as indicated in Figure 4.7 and the corresponding values were given in Table 4.1.

Table 4.1: Volume-change properties of the natural soil

Compressibility characteristics	Values
Compression index, C_c	0.22
Rebound index, C_r	0.10
Coefficient of consolidation, C_v (m^2/s)	6.80E-08
Coefficient of compressibility, m_v (m^2/kN)	1.86E-04
Degree of consolidation, t_{90} (min)	13.2
Coefficient of permeability, k average (m/s)	7.05E-11
Preconsolidation pressure, $\sigma p'$ (kPa)	95
Swell pressure, (kPa)	180

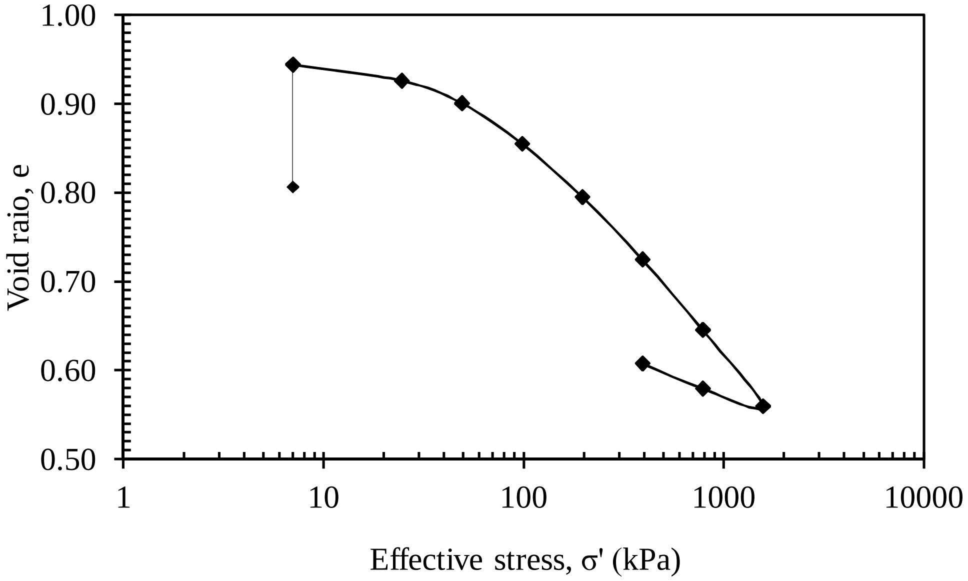


Figure 4.7: Consolidation curve for the natural soil

4.2.8 California Bearing Ratio Test, CBR

The CBR tests were performed in conformity with ASTM D 1883 as discussed in chapter 3. The California division of highways developed this test in 1929 to classify the compatibility of a soil to be used as a subgrade or subbase course material in highway construction. It determines the shear resistance of a soil under the influence of moisture and density conditions. The CBR test for a natural sample was conducted using the compaction properties provided in Table 3.3.

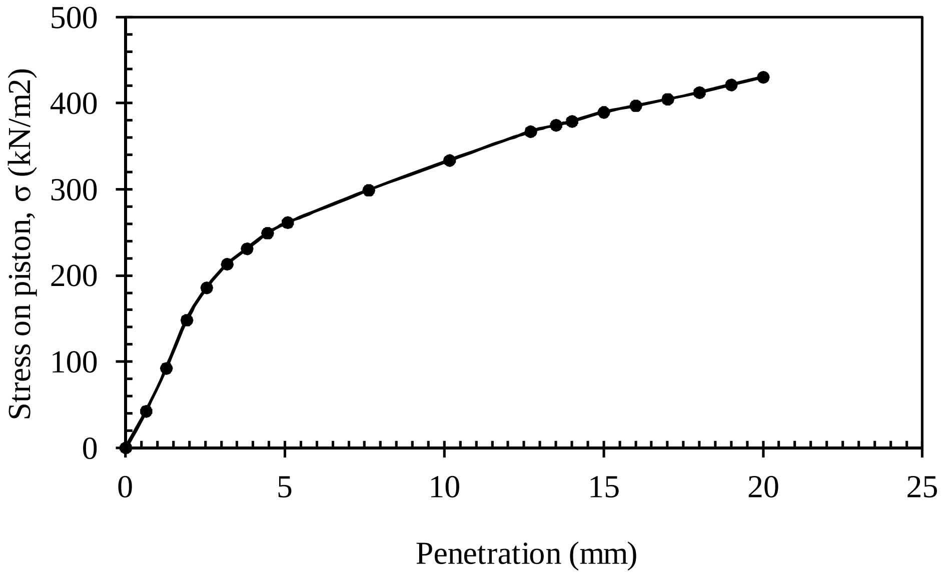


Figure 4.8: California bearing ratio test for natural soil

In the equation 4.5, the CBR of the natural soil was determined as the ratio of the unit load (kN/m^2) needed to provide a depth of penetration (2.54 mm) of the penetration piston (with an area of 0.001935 m^2) to the standard unit load (6894.76 kPa) required to obtain the same depth of penetration on a standard sample of crushed stone. In Figure 4.8, the CBR value of the natural soil was determined to be 2.7%.

$$\% \text{ CBR} = \frac{\text{Stress on piston (test unit load)}}{\text{Standard stress (standard unit load)}} \text{ x } 100 \qquad (4.5)$$

4.2.9 pH Test

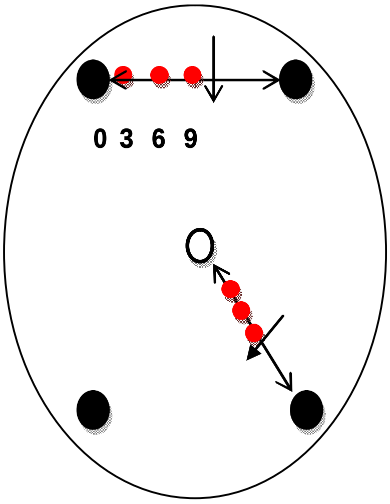
This test was used for the determination of the degree of acidity of the native soil. The pH was conducted in conformity with ASTM 2976 – 71. The soil pH was measured to be 8.11 which indicate the natural soil to be a basic soil.

4.2.10 Electrical Conductivity and Electrical Resistivity Test

The electrical conductivity and electrical resistivity tests were performed according to the laboratory method described in Chapter 3. The electrical resistivity of the natural soil was determined to be 0.27153 ohm centimeter (Ω·cm) and its electrical conductivity was determined to be 368 Siemens per meter (S·m^{-1}).

4.2.11 The Position of the Molds for the Tests at Specific Lime-pile Distances

In this section, Figure 4.9 to Figure 4.11 shows the clear view for the chosen positions of the molds used for the extraction of the stabilized soils at different lime-pile distances in the test tanks.



● = Lime-piles (P) diameter = 3 cm, ○ = Central lime-pile dimension = 3 cm

● = Locations of extraction in the test tank: 0 – 3 cm, 3 – 6 cm and 6 – 9 cm

●↔● Pile to pile (**PP**) distance = 18 cm

○<- ->● Central pile to pile (**CP**) distance = 12 cm

Figure 4.9: Schematic diagram indicating locations of stabilized soil samples extracted for Hydrometer test, Atterberg limit test and linear shrinkage test

Three samples were extracted for the hydrometer test, six samples for both the linear shrinkage test and the Atterberg limit test, five samples for the unconfined compressive strength within a different pile to pile distances and curing days. The legends for the piles and points of extraction are duly represented.

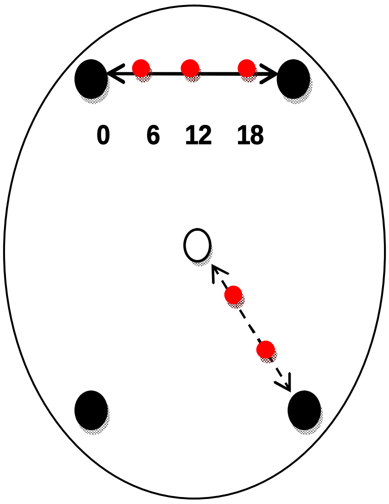
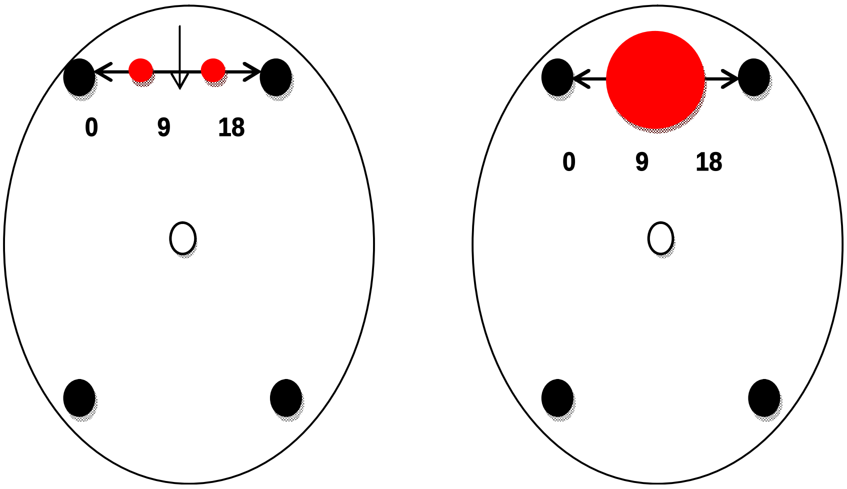


Figure 4.10: Schematic diagram indicating locations of stabilized soil samples extracted for unconfined compression test



(a)

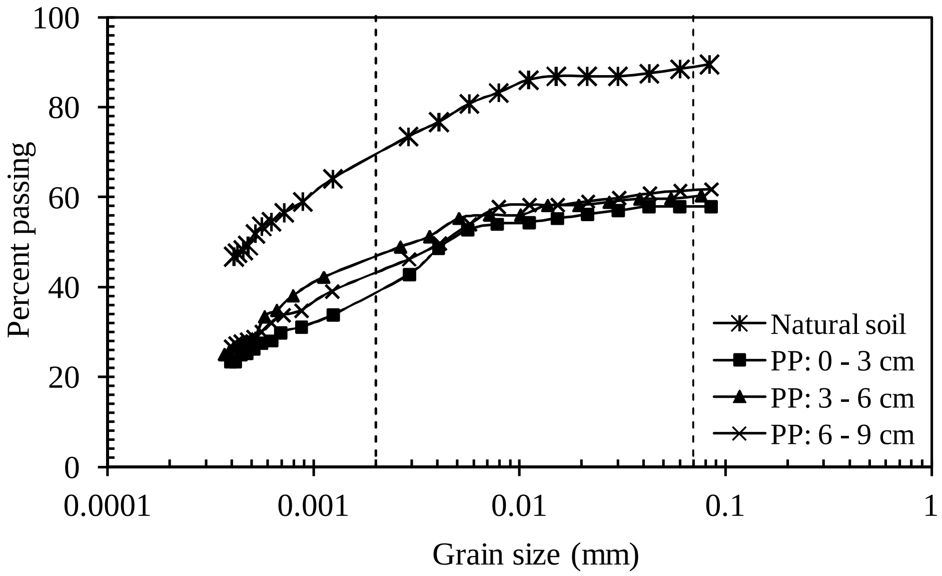
(b)

Figure 4.11: Schematic diagram indicating locations of stabilized soil samples extracted for (a) swell test and one-dimensional consolidation test and (b) CBR test

4.3 Effects of Lime Pile on Engineering Properties of the Native Soil

4.3.1 The Effect of Lime Piles on the Grain Size Distribution of the Stabilized Soil

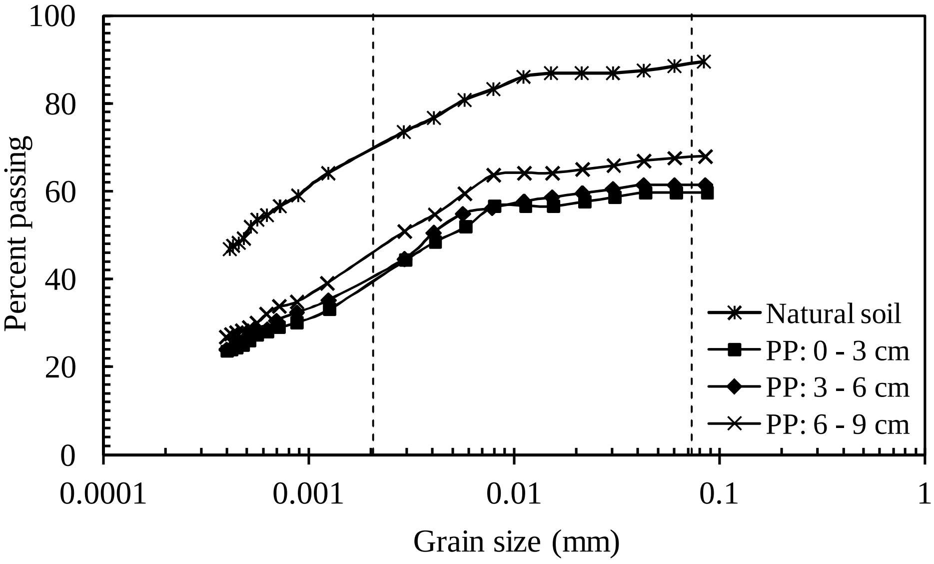
Figure 4.12 to Figure 4.14 showed the results of hydrometer analyses which indicated changes in the grain size distribution of the natural and stabilized soils in terms of lime-pile distances and curing time. The migration of Ca^{2+} ions from the lime-piles subjected the clay particles to physicochemical reactions which produced aggregated particles and caused a drastic reduction in the amount of clay particles. Lime diffused through the clay particles and caused the soil to exhibit the characteristics of cohesionless fine particles of silt.



Clay < 0.002	Silt 0.002 – 0.075	Sand 0.075 – 2.00

Figure 4.12: Grain size distribution variation in natural and stabilized soils at different curing distances in 28 days of curing

In Figure 4.12, it can be observed that there was a reduction in the amount of clay particles from 64% to 39% within a lime-pile distance of 0 – 3 cm in 28 day curing period. Within the radial distances of 3 - 6 cm and 6 – 9 cm, the reduction recorded was to 42% and 44% respectively. The soil samples extracted within 0 - 3 cm and 3 - 6 cm circular distances between two piles showed better modification with improved index properties than the samples extracted within the distance of 6 - 9 cm, which is the distance where lime could not effectively migrate.



Clay < 0.002	Silt 0.002 – 0.075	Sand 0.075 – 2.00

Figure 4.13: Grain size distribution variation in natural and stabilized soils at different curing distances in 90 days of curing

In Figure 4.13, it can be noticed that there was a reduction in the amount of clay particles to 39%, 40% and 42% within lime pile distances of 0 – 3 cm, 3 – 6 and 6 – 9 cm respectively in 28 days curing period. In Figure 4.14, the reduction in the amount of clay particles achieved in 120 days of curing were to 38%, 38% and 40% for the same lime pile distances.

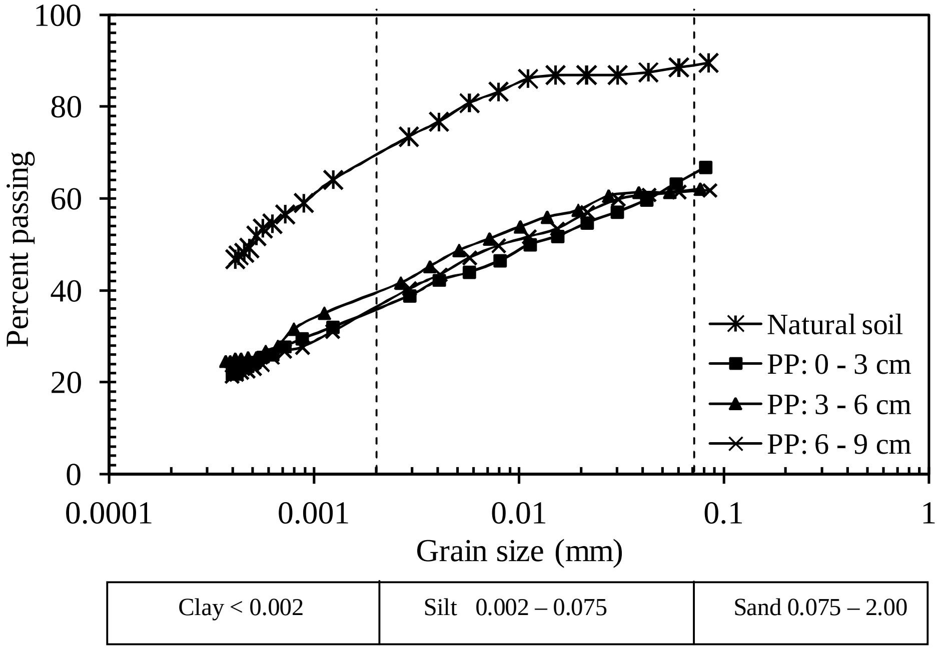


Figure 4.14: Grain size distribution variation in natural and stabilized soils at different curing distances in 120 days of curing

The effect of lime migration can be visibly observed from each pile into the surrounding soil, with the Ca^{2+} ions and hydroxyl ions being diffused from high to low concentration as the distance to the pile increases as indicated in Figure 4.13 to Figure 4.15. The OH^- ions improved the soil alkalinic condition. The OH^- ions aided the clay minerals to break down for easy cation exchange of Ca^{2+} ions with monovalent ions such as Na^+ and K^+ (Larson et. al., 2009). The results obtained substantiated that the lime migrated twice the radial dimension of the pile.

It was evident that the diffused double layer of the clay platelets had collapsed, the attractive forces between the water and clay particles had decreased and as a result, the rate of settling time of the treated soil increased. The lime stabilized soil became flocculated and yielded more dense particles with a granular texture.

In Figure 4.12 to Figure 4.14, it can be inferred from that there was a reduction in amount of clay particles was more significant in 28 days curing periods with subsequent and gradual reduction in 90 and 120 curing days. These findings are in good agreement with work of Tonoz et al. (2003). In their investigation, they also observed a remarkable reduction in the amount of clay particles after the clay-lime reactions. They stated that 28 days of curing were significant enough to achieve a distinct reduction in the amount of clay particles. The highest reduction of clay particles occurred at a distance twice the diameter of the pile which is the effective zone and in 120 days curing periods. However, beyond the effective zone and under the specified number of curing days, the quantity of clay particles tends to increase and advanced towards that of the natural clay sample.

4.3.2 The Effect of Lime Piles on Atterberg Limits of the Stabilized Soil

The effect of Ca^{2+} ions on the plasticity of the soil is dependent on various factors such as the curing period, curing temperature, ionic concentration, type of lime used and soil utilized. In this study, the Atterberg limits test results of the stabilized soil were determined to ascertain lime (Ca^{2+} ions) migration from the lime-piles in terms of the lime-pile distances and curing periods. The soil samples were extracted within the required lime-pile distances of 0 – 3 cm, 3 – 6 cm and 6 – 9 cm from the stabilized soil blocks in different curing periods and the Atterberg limit tests were performed.

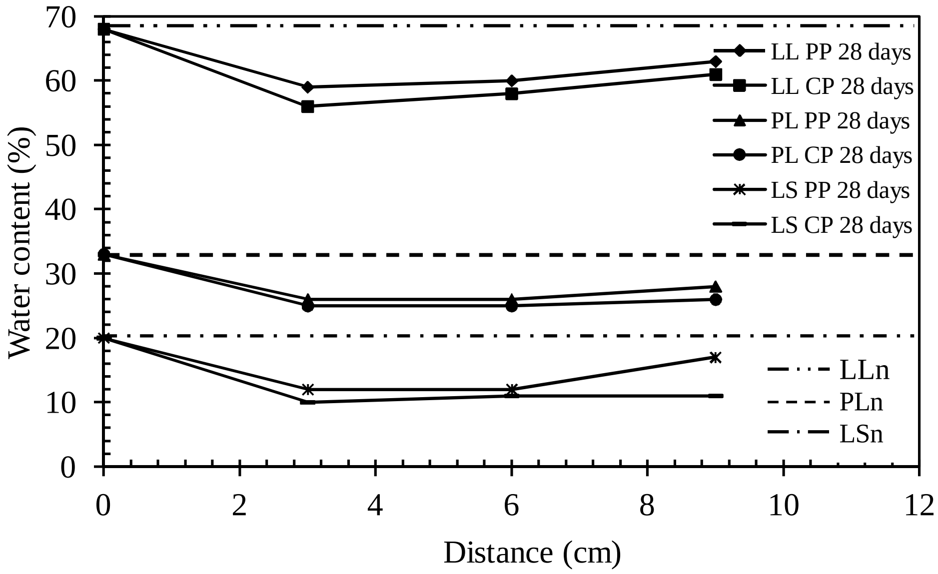


Figure 4.15: Atterberg limit distribution of the stabilized soil at different curing distances in a curing period of 28 days

Table 4.2: The Atterberg limits and linear shrinkage of treated soil from pile to pile in 28 days of curing

Distance from the pile to pile (cm)	0	0 - 3	3 - 6	6 - 9
Liquid limits (LL) (%)	68	59	60	63
Plastic limits (PL) (%)	33	26	26	28
Plasticity index (PI)	35	33	34	35
Linear shrinkage (LS) (%)	20	11	12	17

Table 4.3: The Atterberg limits and linear shrinkage of treated soil from pile to pile in 90 days of curing

Distance from the central-pile to pile (cm)	0	0 - 3	3 - 6	6 - 9
Liquid limits (LL) (%)	68	56	58	61
Plastic limits (PL) (%)	33	25	25	26
Plasticity index (PI)	35	31	33	35
Linear shrinkage (LS) (%)	20	10	11	11

In Figure 4.15, the LL and PL result of the stabilized soils showed that their values recorded highest reduction of 13% and 24% within the pile to pile distances in 28 curing days. The Atterberg limit results were more significant at the distance close to the lime piles. The LL and PL of the stabilized soils within the central pile to pile distances were reduced by 18% and 30% in the same curing periods at similar radial distances.

In Table 4.2 and Table 4.3, the detailed results of the Atterberg limits for 28 curing days are provided. The PI of the stabilized soil was remarkably reduced by 6% and 23% of the pile to pile distances and central pile to lime-pile distances respectively which is quite significant improvement.

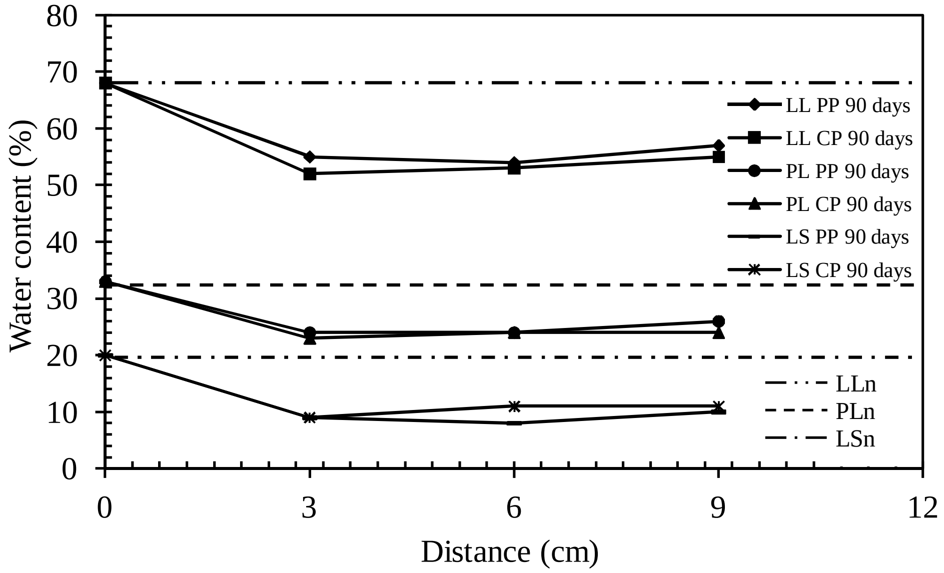


Figure 4.16: Atterberg limits of the treated soils extracted from different curing distances at curing periods of 90 days

In Figure 4.16, the reduction in liquid limits and plastic limits result of the stabilized soils showed the values to be 21% and 27% within the pile to pile distances in 90 curing days. The LL and PL of the stabilized soils within the central pile to lime-pile distances were reduced by 24% and 30% in the same curing periods at similar pile to pile radial distances. In Table 4.4 and Table 4.5, the detailed results of the Atterberg limits of 90 curing days are provided. The PI of the stabilized soil was reduced significantly by 14% and 17% for the pile to pile distances and central pile to lime-pile distances respectively.

Table 4.4: The Atterberg limits and linear shrinkage of treated soil from pile to pile in 90 days of curing

Distance from the pile to pile (cm)	0	0 - 3	3 - 6	6 - 9
Liquid limits (LL) (%)	68	55	54	57
Plastic limits (PL) (%)	33	24	24	24
Plasticity index (PI)	35	31	30	33
Linear shrinkage (LS) (%)	20	09	08	10

Table 4.5: The Atterberg limits and linear shrinkage of the treated soil from the central pile to pile in 90 days curing

Distance from the central-pile to pile (cm)	0	0 - 3	3 - 6	6 - 9
Liquid limits (LL) (%)	68	52	53	53
Plastic limits (PL) (%)	33	23	24	24
Plasticity index (PI)	35	29	29	29
Linear shrinkage (LS) (%)	20	09	11	11

In Table 4.6 and Table 4.7, the comprehensive results of the Atterberg limits in different lime-pile distances of 3 cm, 6 cm and 9 cm in 120 days curing periods are presented. In Figure 4.17, the plasticity index of the stabilized soil was reduced significantly by 11% and 20% for the pile to pile distances and a central pile to lime pile distances. The reduction in liquid limits and plastic limits result of the stabilized soils were determined to be 28% and 40% within the pile to pile distances in 120 curing days.

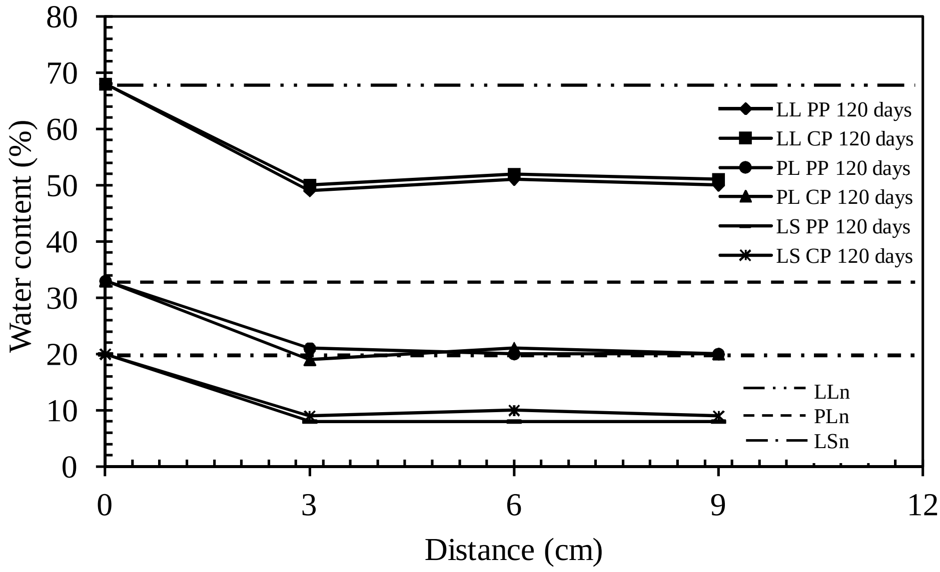


Figure 4.17: Atterberg limits of the treated soils extracted from different curing distances at curing periods of 120 days

Table 4.6: The Atterberg limits and linear shrinkage of treated soil from pile to pile in 120 days of curing

Distance from the pile to pile (cm)	0	0 - 3	3 - 6	6 - 9
Liquid limits (LL) (%)	68	49	51	50
Plastic limits (PL) (%)	33	21	20	20
Plasticity index (PI)	35	28	31	30
Linear shrinkage (LS) (%)	20	08	08	08

Table 4.7: The Atterberg limits and linear shrinkage of the treated soil from the central pile to pile in 120 days of curing

Distance from the central-pile to pile (cm)	0	0 - 3	3 - 6	6 - 9
Liquid limits (LL) (%)	68	50	52	51
Plastic limits (PL) (%)	33	19	21	20
Plasticity index (PI)	35	31	31	31
Linear shrinkage (LS) (%)	20	09	09	09

Evidently, there was a remarkable reduction in the PI of the treated soil within curing periods of 28 days with further slow improvement in reduction within 90 and 120 days of curing. In Figure 4.15 to Figure 4.17, it was observed that in the effective zone of the lime-pile distance of 0 – 6 cm distance, which is twice the diameter of the lime-piles, there was a greater reduction in LL, PL and PI in the stabilized soil. The Atterberg limits reduced as the lime diffused into the surrounding soil within the effective zone and gradually increased away from the lime pile distances, that is, away from the effective zone and tended toward the natural soil properties.

With the migration of the Ca^{2+} ions from the lime-piles, the Atterberg limit showed a regular trend in the reduction of PI from location of high lime concentration and tends to approach the plasticity of the natural soil, away from the pile. With longer curing days, the lime migrated to almost every part of the stabilized soil block and caused a reduction in its plasticity.

The Atterberg limit results indicated that the chemical effects of lime migration in the stabilized soil block caused dehydration, agglomeration and flocculation of clay particles which caused a reduction in the clay plasticity. The introduction of lime-piles to the compacted soil block provided calcium and hydroxyl ions which aided migration by diffusion and this caused spontaneous physicochemical reaction which altered the textural characteristics of the soil and yielded soil particles with lesser plasticity.

The change in the plasticity, texture and fabric of the stabilized soil with the application of the quicklime piles can also be attributed to the collapse of the diffused double layer (DDL) which is the thickness between the clay minerals surface and soil water solution. The high salt concentration (double positive charge calcium ions), dehydration process, dielectric constant and pH are all contributing factors to the collapse of DDL, with the elimination of the electrical repulsive forces and the reduction in the soil plasticity.

4.3.3 The Effect of Lime-Piles on the Linear Shrinkage of the Stabilized Soil

The linear shrinkage (LS) of the stabilized soil was investigated in order to correlate the lime diffusion with the lime-pile distances and curing periods.

Table 4.8: The linear shrinkage of the stabilized soils at different lime-pile distances and curing periods

Distance within lime-pile distance (cm)	0 - 3	3 - 6	6 - 9
Natural soil	20	20	20
PP: 28 days curing period	11	12	17
CP: 28 days curing period	10	11	11
PP: 90 days curing period	09	08	10
CP: 90 days curing period	09	11	11
PP: 120 days curing period	08	08	08
CP:120 days curing period	09	09	09

**PP: pile to pile distance **CP: central pile to pile distance

To achieve this, samples were extracted from lime-pile distances within 0 – 3 cm, 3 – 6 cm and 6 – 9 cm in the test tanks in different curing periods. The LS tests were performed on the extracted stabilized soils in curing periods of 28, 90 and 120 days.

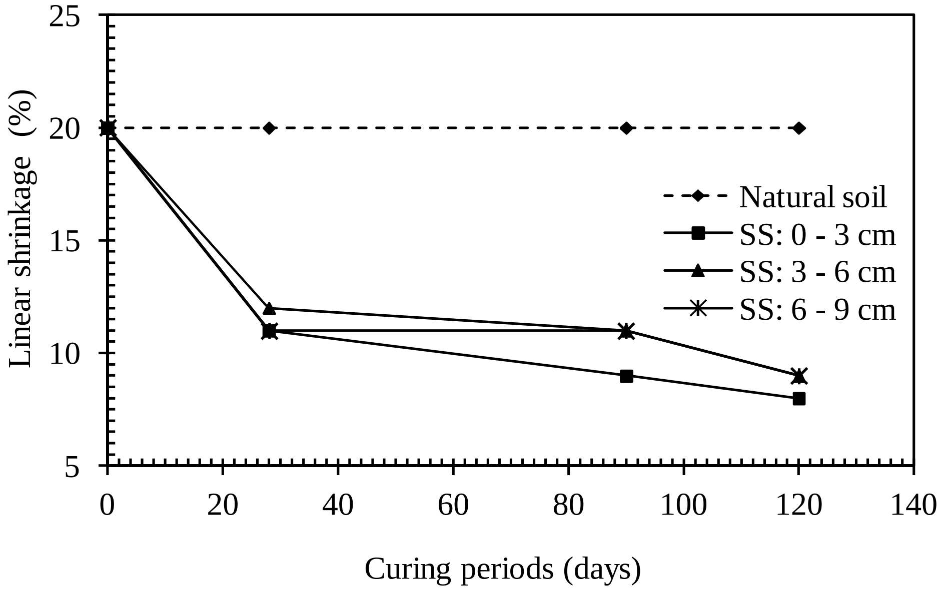


Figure 4.18: Linear shrinkage versus curing periods

The linear shrinkage for the natural soil was determined to be 20% and for the stabilized soils were determined to be 11%, 9% and 8% from the lime-pile distance of 3 cm in curing periods of 28, 90 and 120 days respectively as indicated in Table 4.8 and Figure 4.18.

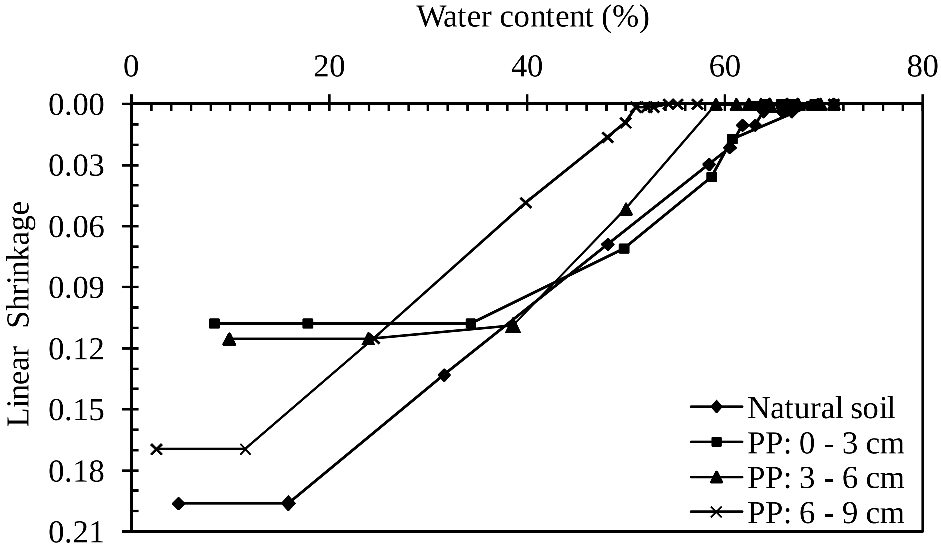


Figure 4.19: Linear shrinkage versus water content for natural and stabilized soils within different lime pile distances and in 28 days of curing

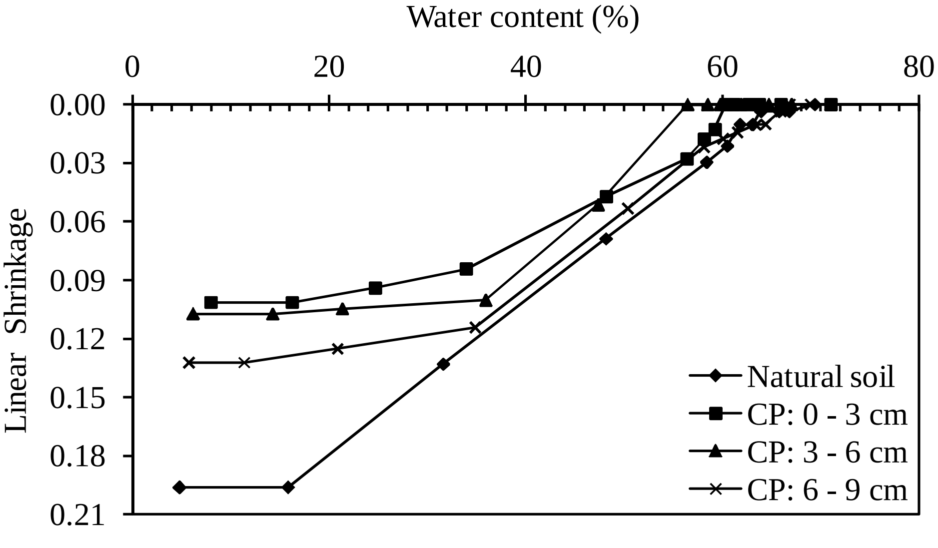


Figure 4.20: Linear shrinkage versus water content for natural and stabilized soils at different central lime pile to pile distances and in 28 days of curing

The change in the linear shrinkage versus water content is represented in Figure 4.19 to Figure 4.24. The curves showed a linear drying path of the change in length with respect to the reduction in water content, until a distinct point at which moisture content kept reducing with no further change in length (Amy et al., 2006).

In Figure 4.19 and Figure 4.20, the linear shrinkage was reduced from 20% (the natural soil) to 11% for stabilized soil after 28 days of curing. In Table 4.8, the change in linear shrinkage values with respect to pile to pile distances and central-pile to pile distances are given in details. The linear shrinkage reduction was due to the clay-lime physicochemical reactions, with subsequent gradual improvement in long curing time.

The variations in the textural characteristics of the stabilized soils due to the chemical effects of the quicklime migration to different distances in the soil block. The curves showed that the linear shrinkage value of the stabilized soils determined at different drying periods reduced and the highest reduction occurred within the effective zone distance, twice the radial size of the piles (i.e. 0 – 6 cm).

In Figure 4.19 to Figure 4.24, the linear shrinkage values measured at different curing periods reduced from 20% (natural soil) to 9% and 8% for the stabilized soils in 90 and 120 days of curing respectively. In Table 4.8, comprehensive change in the linear shrinkage values with respect to pile to pile distances and central-pile to pile distances are given in details.

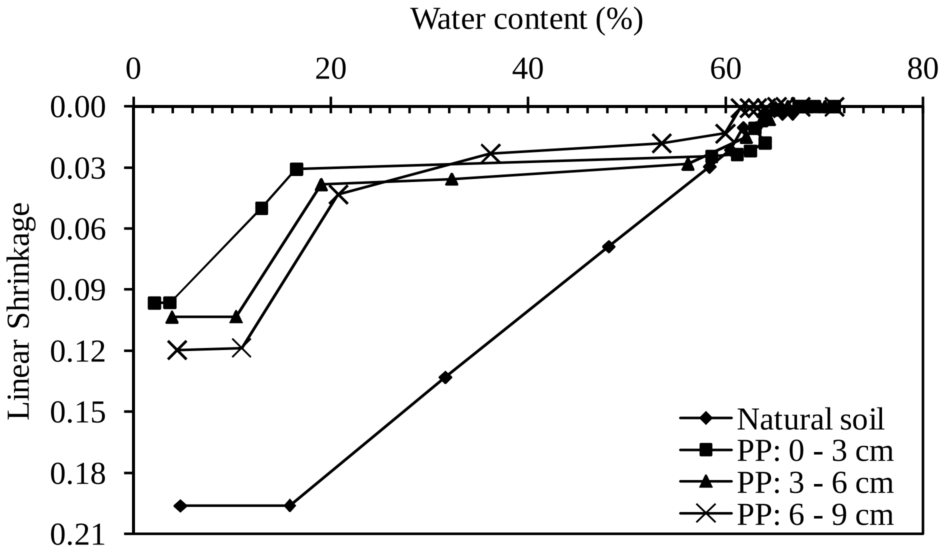


Figure 4.21: Linear shrinkage versus water content for natural and stabilized soils at different lime pile distances and in 90 days of curing

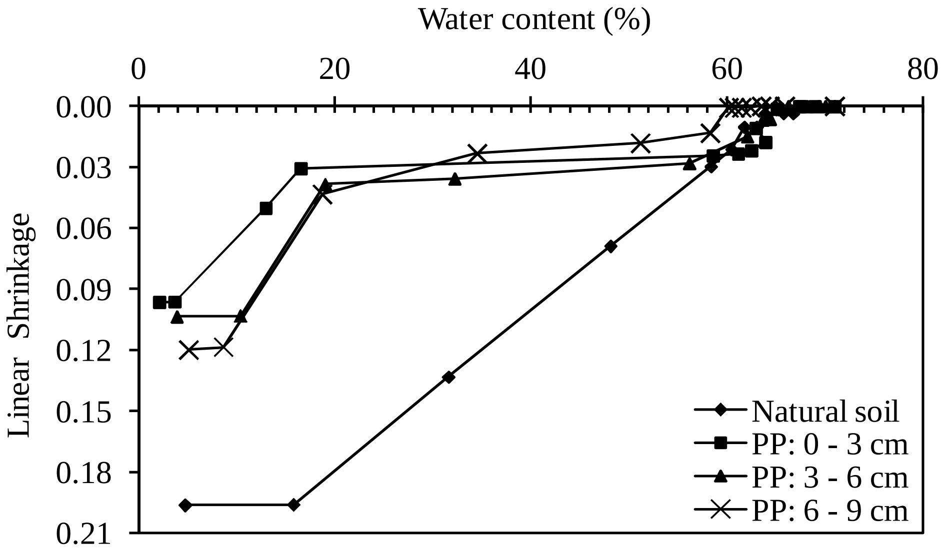


Figure 4.22: Linear shrinkage versus water content for natural and stabilized soils at different central lime pile to pile distances and in 90 days of curing

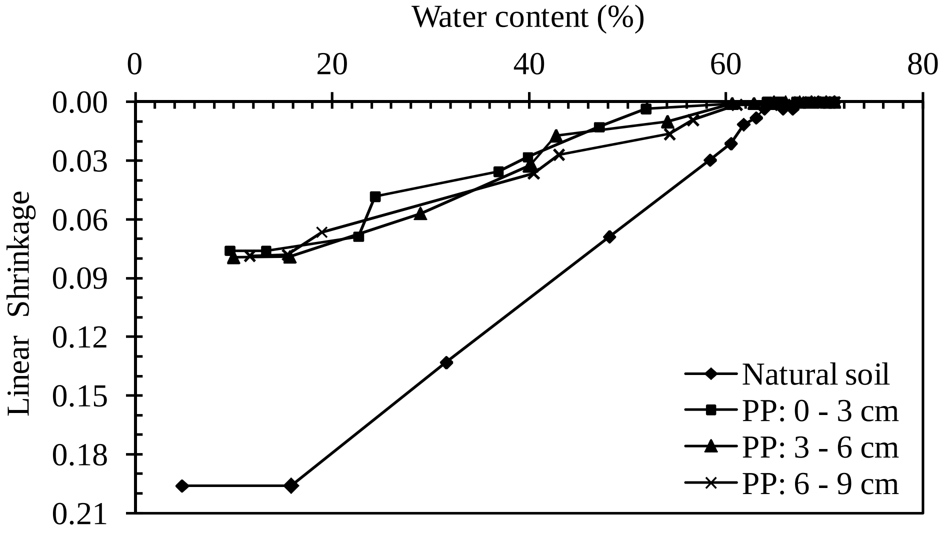


Figure 4.23: Linear shrinkage versus water content for natural and stabilized soils at different lime pile distances and in 120 days of curing

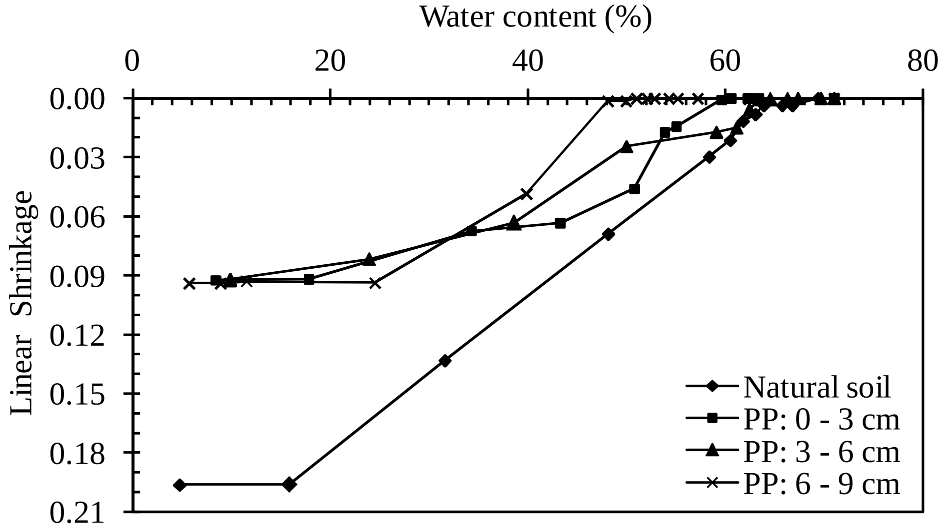


Figure 4.24: Linear shrinkage versus water content for natural and stabilized soils at different central lime pile to pile distances and in 120 days of curing

In Figure 4.25 to Figure 4.30, there was a significant loss in moisture content in the stabilized soil due to the high Ca^{2+} ions content and distortion of the DDL and an increase in the pore sizes. Nevertheless, with rapid loss in moisture and at lower water content in the stabilized soil, remarkable decrement in the linear shrinkage of the stabilized soil was obtained. This is more significant at the distance close to the pile and the central-pile. The better result was attained within the 6 cm distance to the lime-pile and in 120 days of curing. This is a remarkable modification in the shrinking properties of the stabilized soil which is time dependent and the higher the curing periods, the better the stabilization.

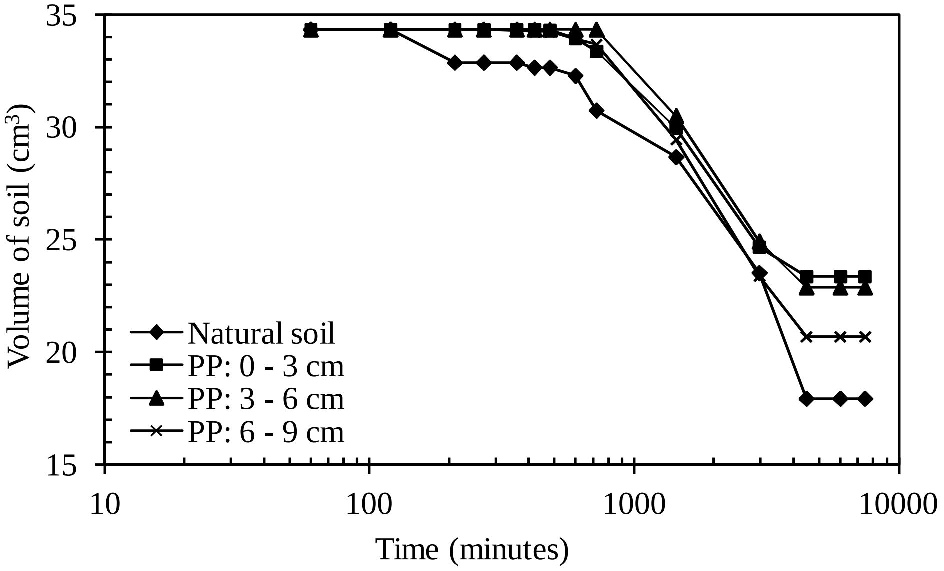


Figure 4.25: Change in volume of soil versus time curves for the natural and stabilized soils at different lime pile distances in curing periods of 28 days

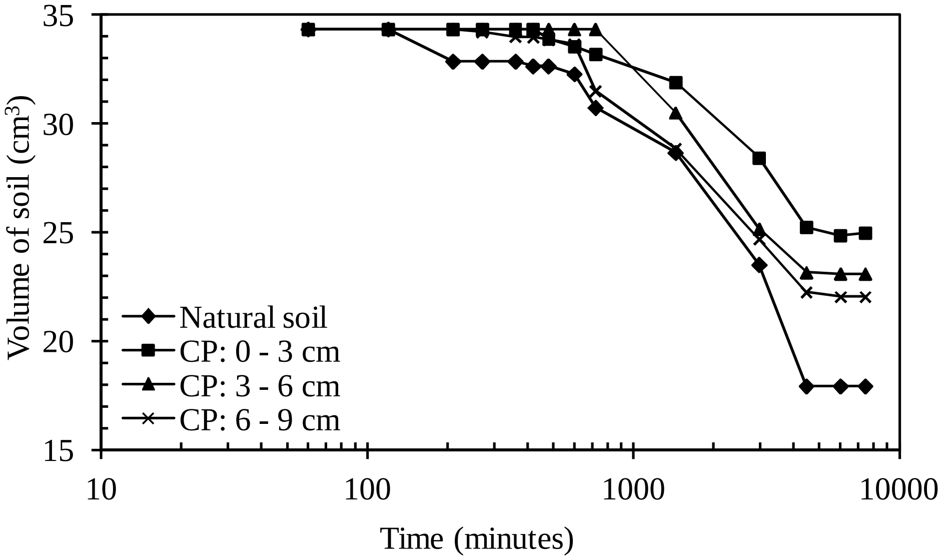


Figure 4.26: Change in volume of soil versus time curves for the natural and stabilized soils at different central pile to pile distances in curing periods of 28 days

In Figure 4.25 to Figure 4.30, the relationships between the changes in volume with respect to log-time were established for the natural and the stabilized soils at different curing periods and pile to pile distances. The variation in the change in volume with respect to time was observed as the number of curing days increased. The stabilized samples close to the piles, showed smaller amount of volume changed due to lime migration.

In the natural soil, the volume shrinkage in terms of the change in length and diameter was determined to be 48% using the equation 4.2. The stabilized soils showed 31% and 27% change in volume in 28 and 90 days of curing respectively. This reduction in change in volume with respect to time can be attributed to the reduction in the amount of the clay minerals due to lime diffusion.

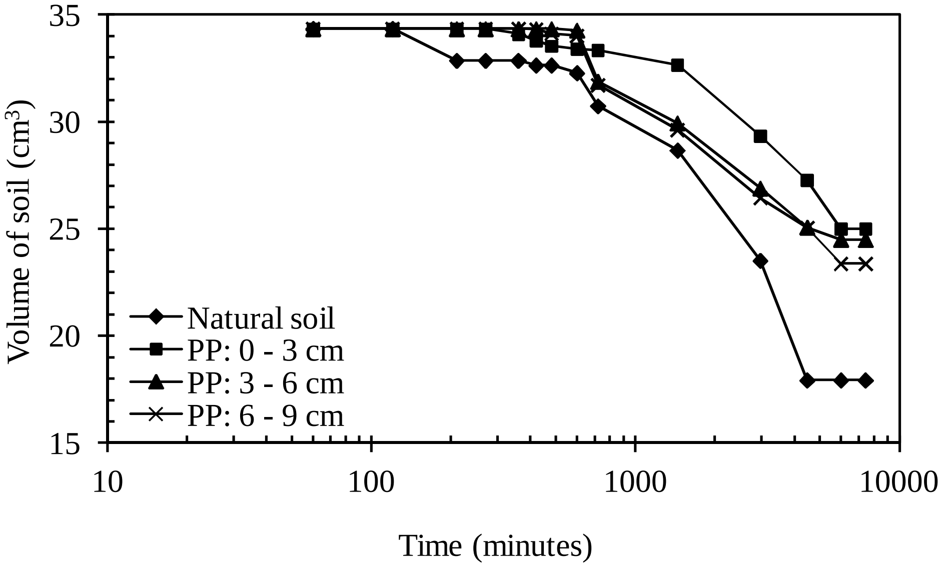


Figure 4.27: Change in volume of soil versus time curves for the natural and stabilized soils at different lime pile distances in curing periods of 90 days

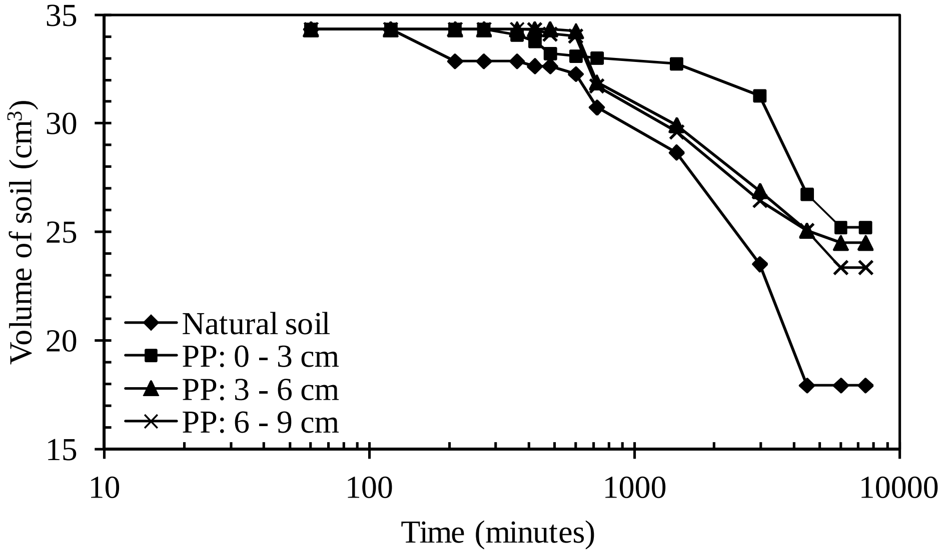


Figure 4.28: Change in volume of soil versus time curves for the natural and stabilized soils at different central pile to pile distances in curing periods of 90 days

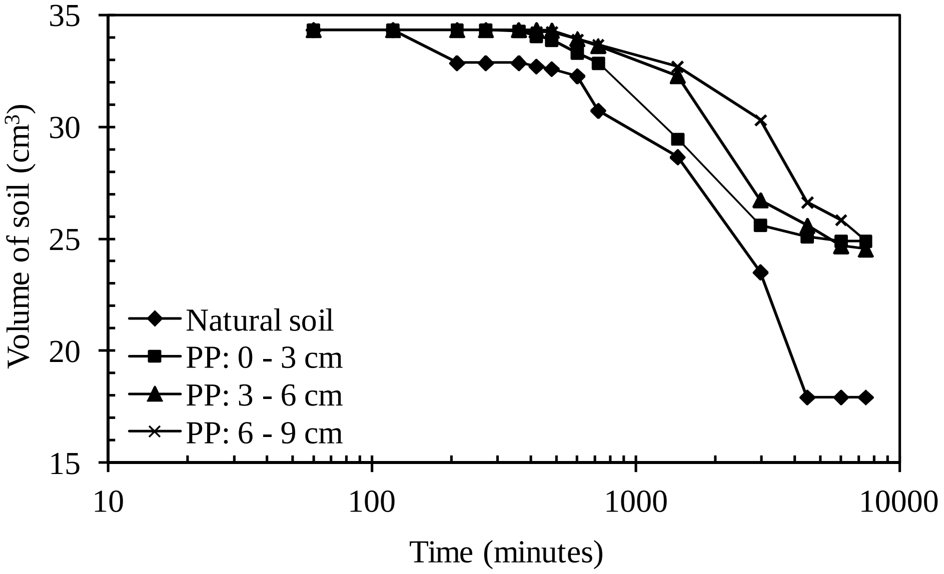


Figure 4.29: Change in volume of soil versus time curves for the natural and stabilized soils at different lime pile distances in curing periods of 120 days

The closer the stabilized soil to the pile, the lower the rate at which the volume changed. All these evidence substantiated the fact that the stabilized soil has become more friable and granular. In conclusion, the stabilized soils recorded highest percentage of reduction in volume change at 120 curing days by 23% within a lime-pile distance of 6 cm.

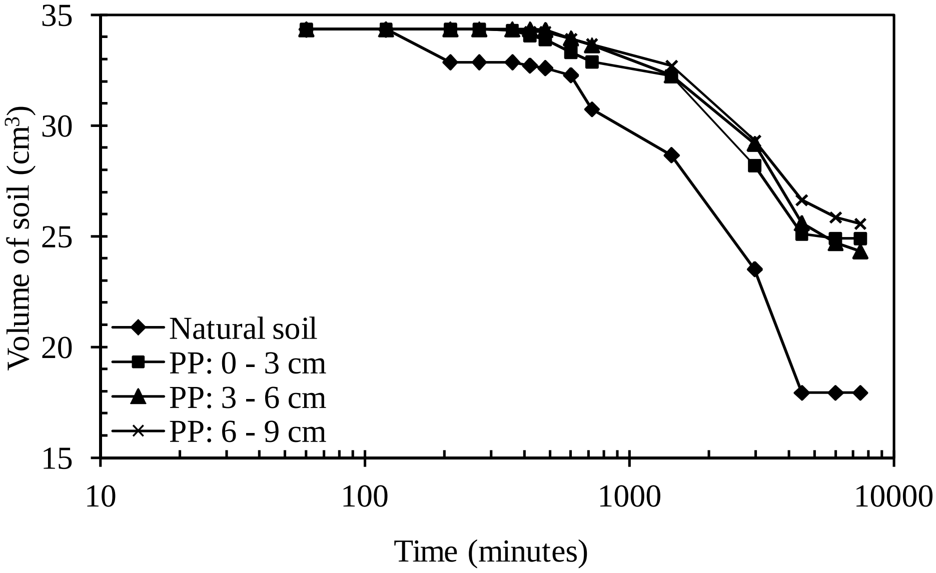


Figure 4.30: Change in volume of soil versus time curves for the natural and stabilized soils at different central pile to pile distances in curing periods of 120 days

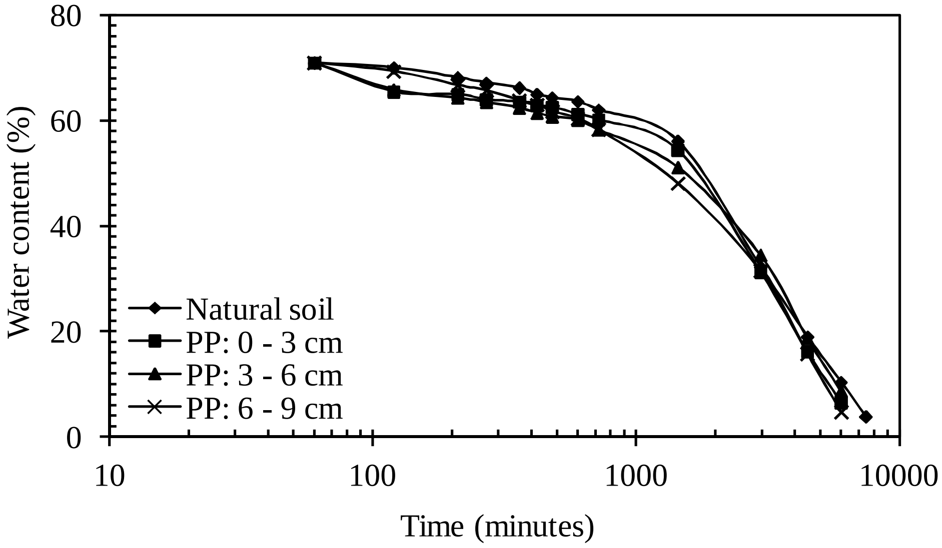


Figure 4.31: Change in water content versus time curves for the natural and stabilized soils at different lime pile distances and in 28 days of curing

In Figure 4.31 and Figure 4.36, the curves indicate the variations in change in moisture content versus time during the linear shrinkage test. In Figure 4.31 and Figure 4.32, higher variations of loss in the moisture from the stabilized soils at different lime piles and central pile distances at curing time smaller than 120 days were observed. There was a distinction in the water loss of the natural soil compared to the stabilized soils. But as the curing period increased, the variation in moisture loss within lime-pile distances reduced. This indicated an effective progression of lime in the soil block by effective cation exchange in the clay minerals.

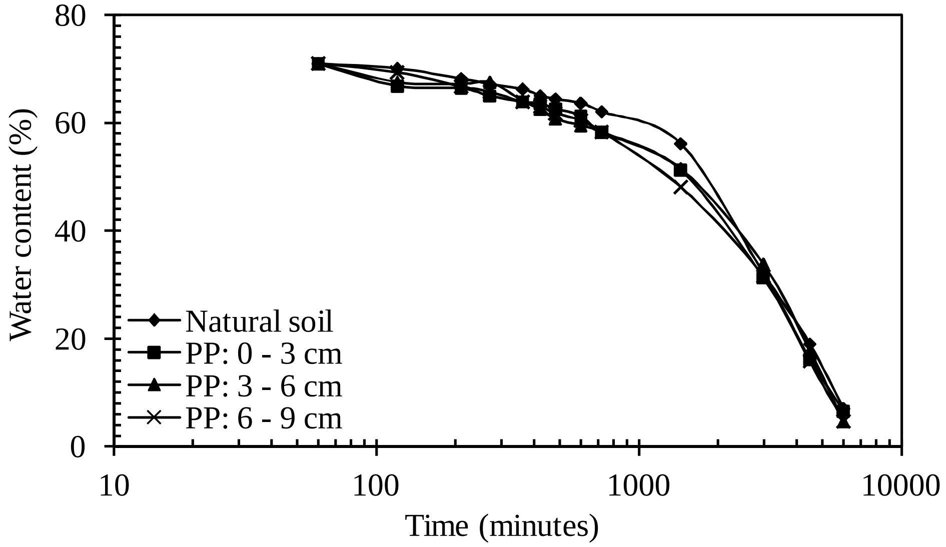


Figure 4.32: Change in water content versus time curves for the natural and stabilized soils at different central pile to pile distances and in 28 days of curing

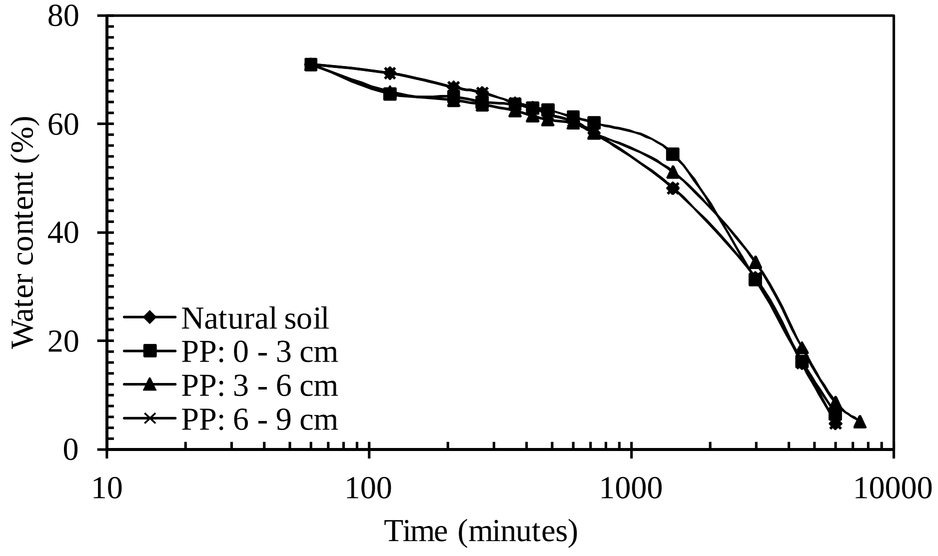


Figure 4.33: Change in water content versus time curves for the natural and stabilized soils at different lime pile distances and in 90 days of curing

In Figure 4.31 to Figure 4.34, the change in water content variations in the stabilized soil was high for 28 and 90 days of curing, while the variations in the stabilized soil tend to be equilibrium for curing periods of 120 days as shown in Figure 4.35 and Figure 4.36. This also indicated complete lime diffusion as the logarithmic curve was tending to be linear in 120 days of curing for stabilized soils extracted at different lime pile and central pile distances.

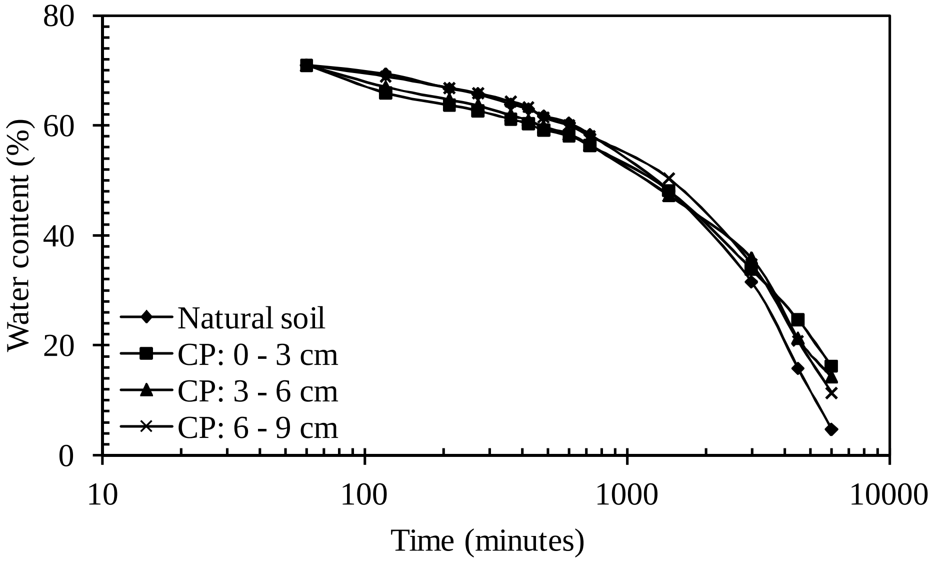


Figure 4.34: Change in water content versus time curves for the natural and stabilized soils at different central pile to pile distances and in 90 days of curing

The more close the stabilized soil to the lime piles, the better the shrinkage properties achieved, while the farther the distance the properties tended towards the natural soil. The moisture content reduced with higher curing periods and the volume of change in the stabilized soil was gradual which indicated a remarkable improvement.

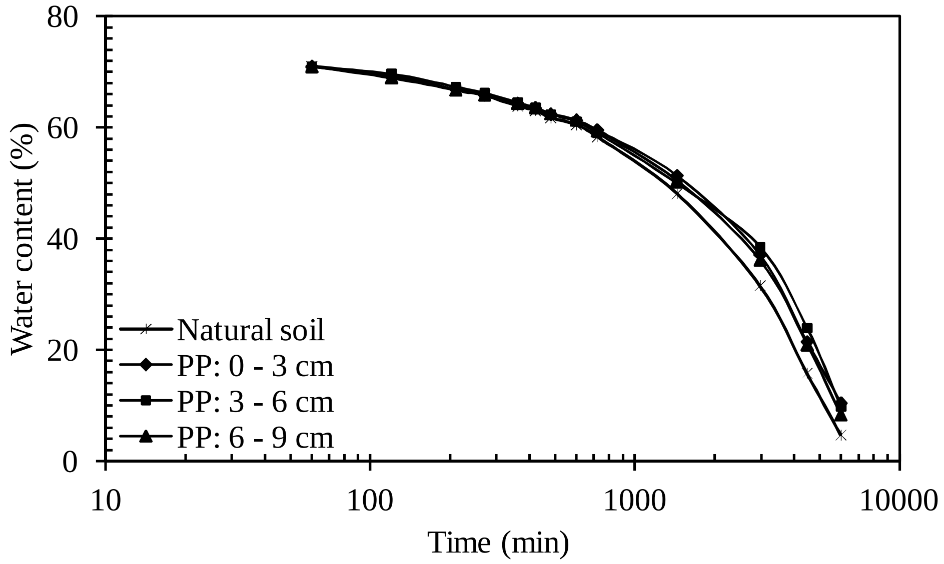


Figure 4.35: Change in water content versus time curves for the natural and stabilized soils at different lime pile distances and in 120 days of curing

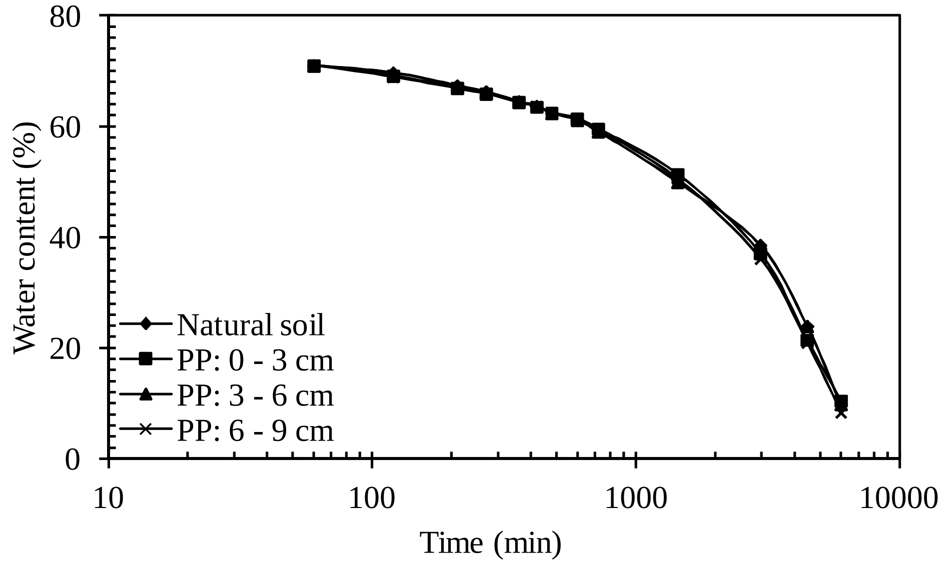


Figure 4.36: Change in water content versus time curves for the natural and stabilized soils at different central pile to pile distances and in 120 days of curing

4.3.4 The Effect of Lime Piles on the Compaction Characteristics of the Stabilized Soil

The compaction curves of the stabilized soils are given in Figure 4.37.

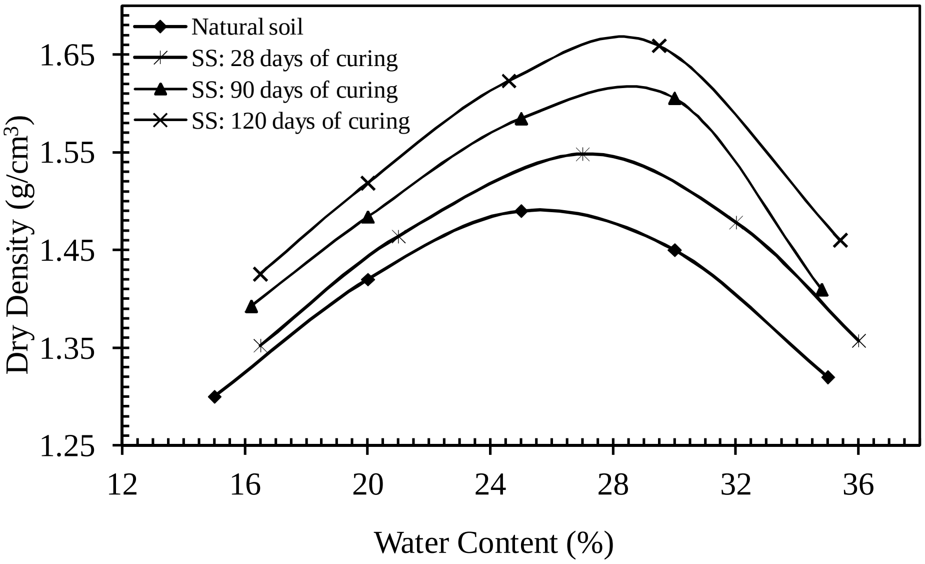


Figure 4.37: Compaction characteristics of natural soil and soil stabilized with lime piles in 28, 90 and 120 days curing periods

The samples extracted from the stabilized soil block in the test tanks were subjected to the standard Proctor compaction test at different curing time and the compaction characteristics, the maximum dry density and optimum moisture content of the stabilized soil which changed in response to the lime treatment after different curing periods were determined.

The reduction in low plasticity index due to lime diffusion caused the soil to become more friable and granulated in nature. Therefore, this aided more soil to be added to the block during compaction and increased the optimum moisture content of the stabilized soil as the curing periods increased.

Table 4.9: Compaction characteristics of the natural and stabilized soils in different curing periods

Soil type Curing periods	Maximum dry densities (g/cm^3)	Optimum water content (%)
Natural soil	1.49	25
SS: 28 days of curing	1.55	27
SS: 90 days of curing	1.62	28
SS: 120 days of curing	1.66	30

*SS: Stabilized soil

In Table 4.9, the details of the maximum dry density (MDD) of the stabilized soil which increased from 1.49 g/cm^3 to 1.66 g/cm^3, while optimum moisture content (OMC) increased from 25% to 30% in curing period of 120 days are given.

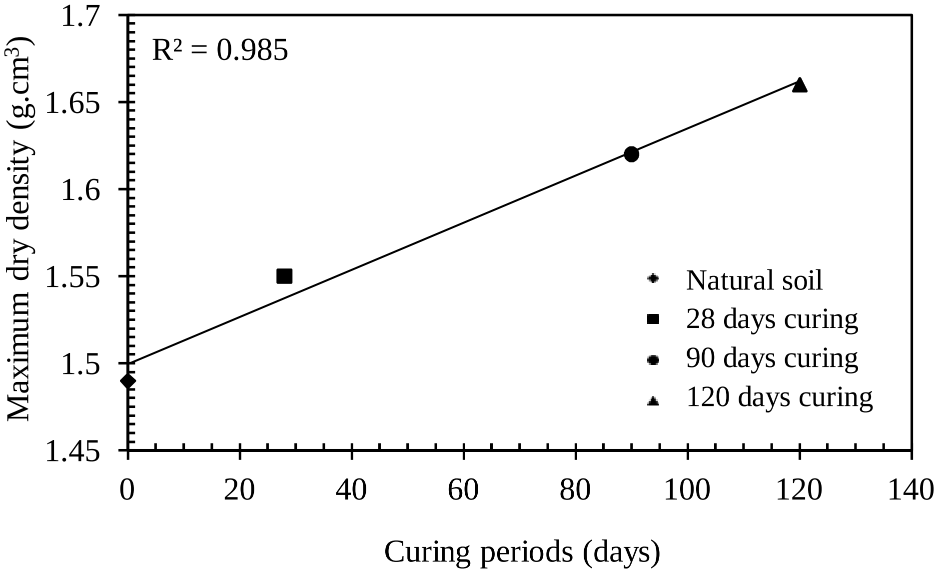


Figure 4.38: Maximum dry density versus curing periods for the natural and stabilized soils

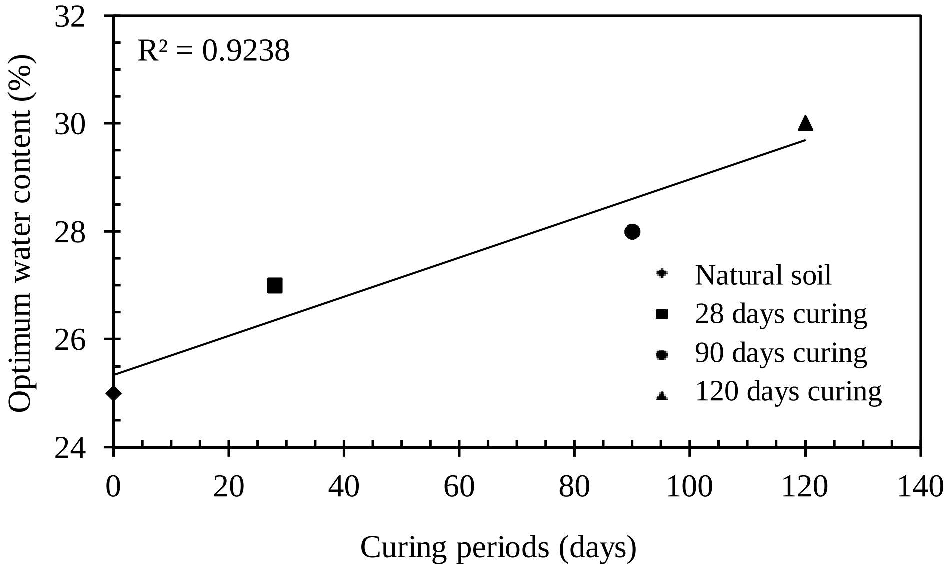


Figure 4.39: Optimum water content versus curing periods for the natural and stabilized soils

In Figure 4.38 to Figure 4.39, it can be inferred that the compaction characteristics of the stabilized soil increased after modifying the soil properties with the lime-piles. The increase in the maximum dry density and optimum moisture content can be attributed to changes in the mineralogy and texture of the stabilized soils due to lime diffusion.

4.3.5 The Effect of Lime-Piles on the Unconfined Compressive (UC) Strength of the Stabilized Soil

The stress-strain curves drawn from the unconfined compressive tests were represented in Figures 4.40 to Figure 4.49. The samples were extracted from the soil block in the test tanks within radial distances of 0 - 6 cm, 6 - 12 cm and 12 - 18 cm from one pile to the other at curing periods of 28, 90 and 120 days. In addition, samples were also extracted from the same distances and curing periods within the central pile to surrounding pile. The tests were performed in conformity with the ASTM standard (ASTM D 2166-06).

In Figures 4.40 to Figure 4.50, the greatest strength was achieved within the effective zones (0 – 6 cm and 12 – 18 cm) of the lime-pile distances in 120 days of curing.

In Figure 4.40, higher curing periods indicated higher strength. In 4.42 to Figure 4.50, the lime-pile radial distance of 0 - 6 cm is as close to the other pile extracted within 12 – 18 cm.

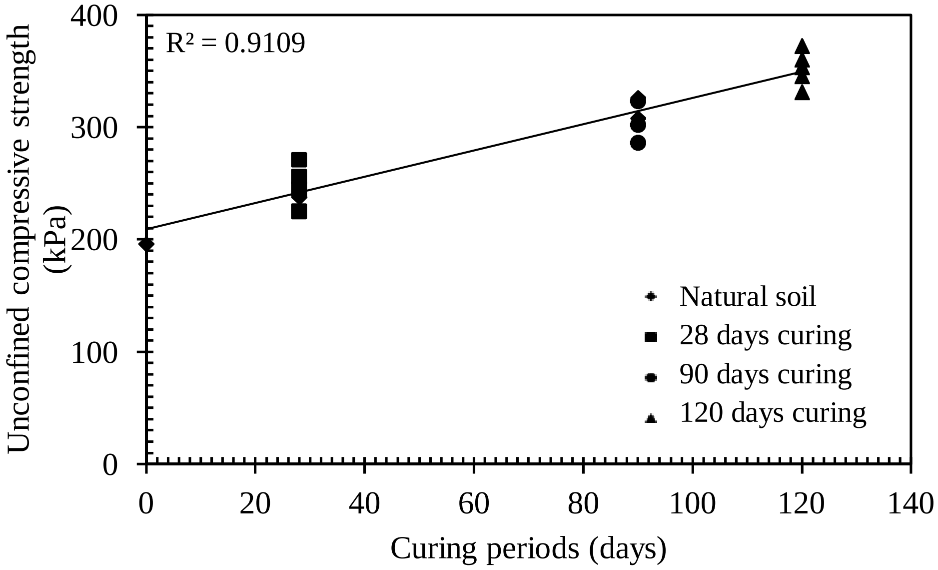


Figure 4.40: Unconfined compressive strength versus curing periods

In Figure 4.41, it can be inferred that the gained in strength within the effective zones from pile to pile distances of radial distance 0 – 6 cm and 12 – 18 cm were 39% and 26% respectively and for the ineffective zone within 6 – 12 cm, it was 15%. The increment in the strength from the central pile to pile distances of radial dimensions within 0 – 6 cm and 6 – 12 cm were 25% and 22% respectively as indicated in Figure 4.42. The combined strength increase is given in Figure 4.43.

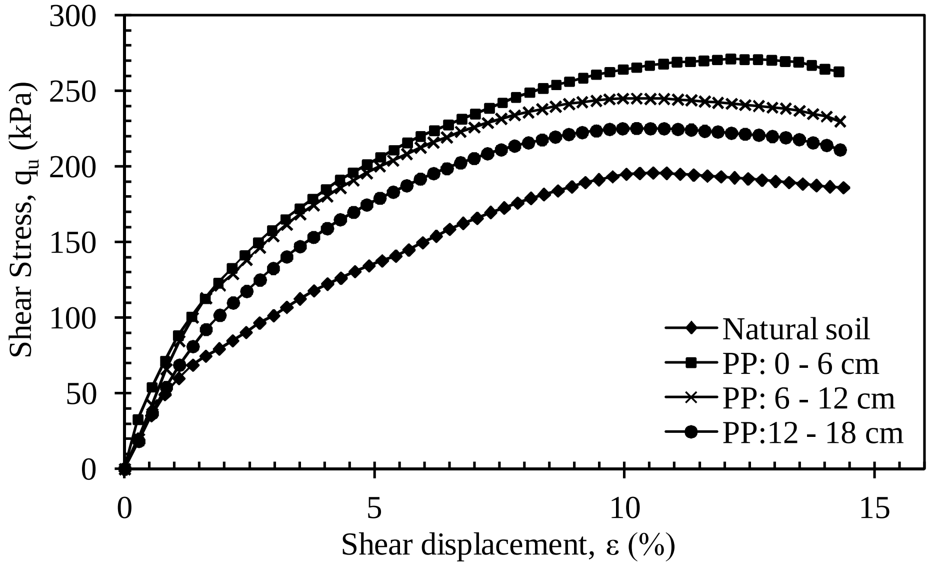


Figure 4.41: Stress-strain curves for the stabilized soil at different lime pile distances in 28 days curing

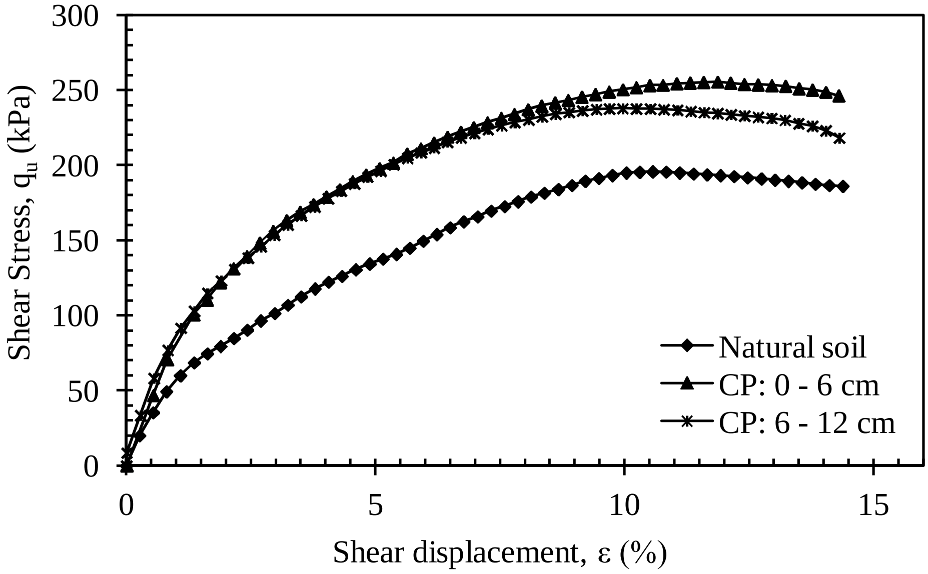


Figure 4.42: Stress-strain curves for the stabilized soil in curing periods of 28 days at different central pile to pile distances

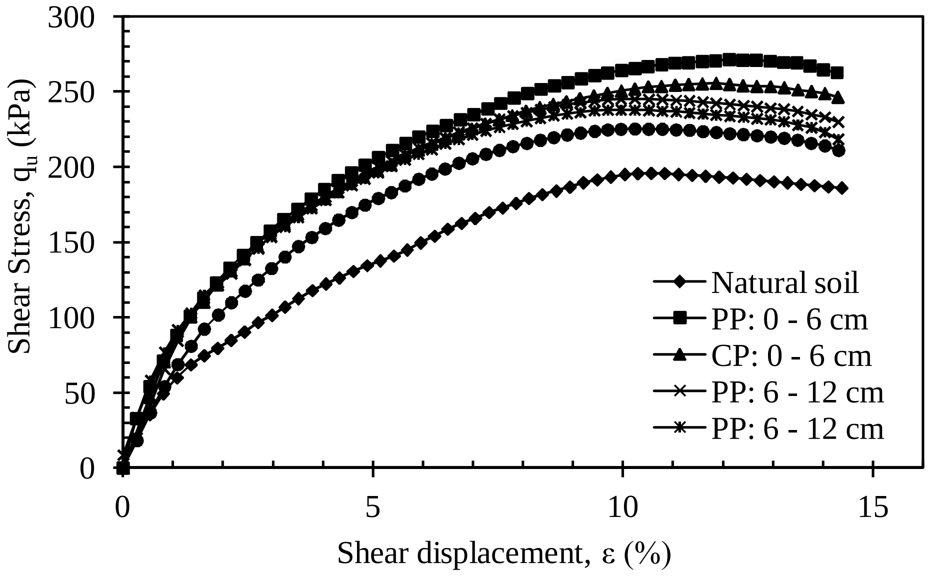


Figure 4.43: Stress-strain curves for the stabilized soil for curing periods of 28 days at different lime pile distances

Therefore, the gained in strength within 28 days curing periods was considerably enough in the stabilized soil when compared to the natural soil. The soil geomechanical properties changed due to quicklime modification. The lime modified the textural, mineralogical and physiochemical properties of the stabilized soil and caused a significant increase in durability and stability of the soil. The changes in the unconfined compressive strength of the lime treated soils in terms of curing period and lime-pile distances are attributed to the dissolution of alumina and silica in a highly alkalinic environment, followed by the exchangeable monovalent ions of sodium, potassium etc. with divalent calcium ions to form cementitious compounds of calcium (aluminate and silicate) hydrate (CAH and CSH).

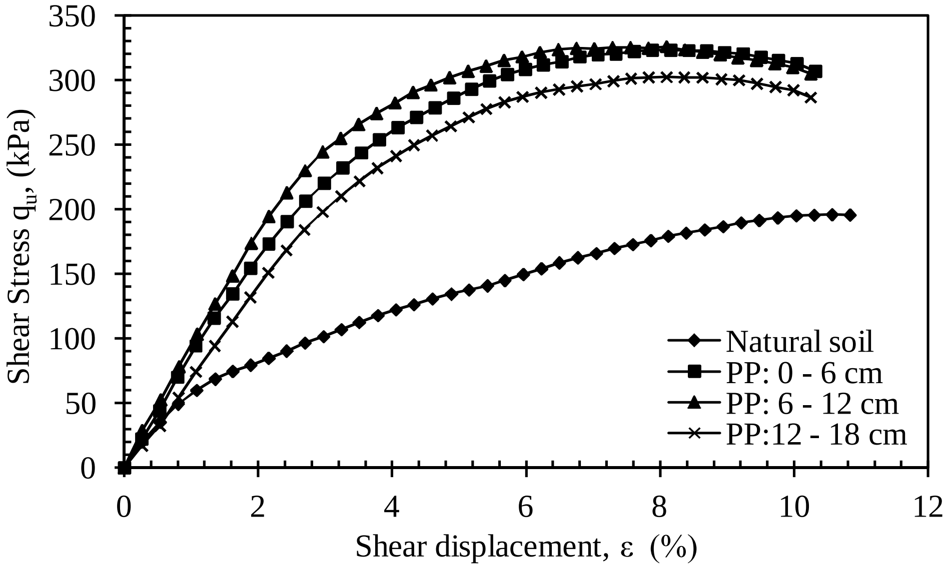


Figure 4.44: Stress-strain curve for the stabilized soil at different lime pile distances in curing periods of 90 days

In Figure 4.44 to Figure 4.46, the highest strength gained from stabilized soil within pile to pile distances was 66% and within the central pile to pile distances, the strength gained was 60% for 90 days curing periods. The stabilized soil became more granular and less plastic in nature due to reduction in DDL thickness and subsequent flocculation of the soil particles. The binding materials coagulated the clay platelets together, increased their angle of internal friction and bond energy which resulted in a remarkable increase in strength, stiffness and durability.

While the highest strength achieved in 120 days of curing periods was approximately 90% within lime-pile distances, as recorded in Figure 4.47 to Figure 4.48.

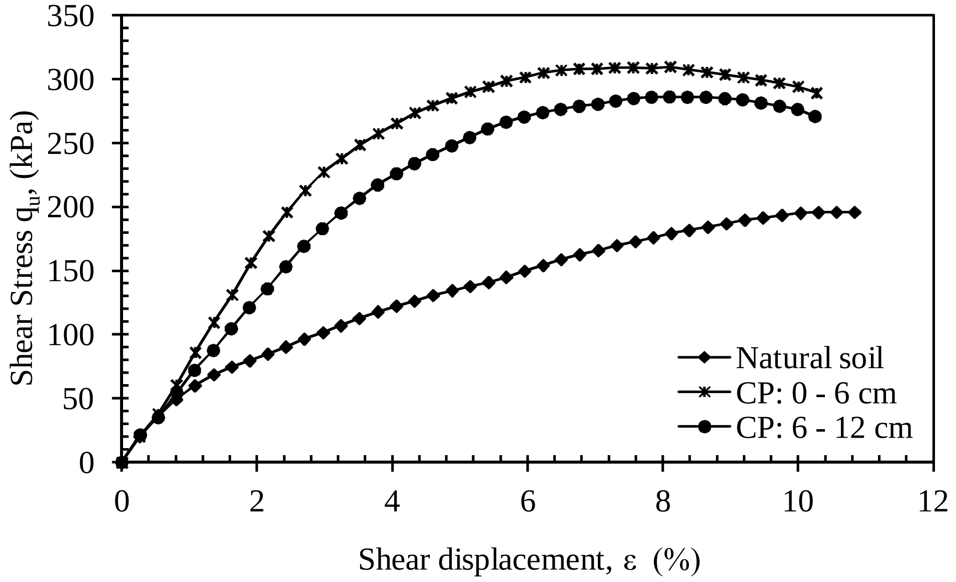


Figure 4.45: Stress-strain curve for the stabilized soil for curing periods of 90 days at different central pile to pile distances

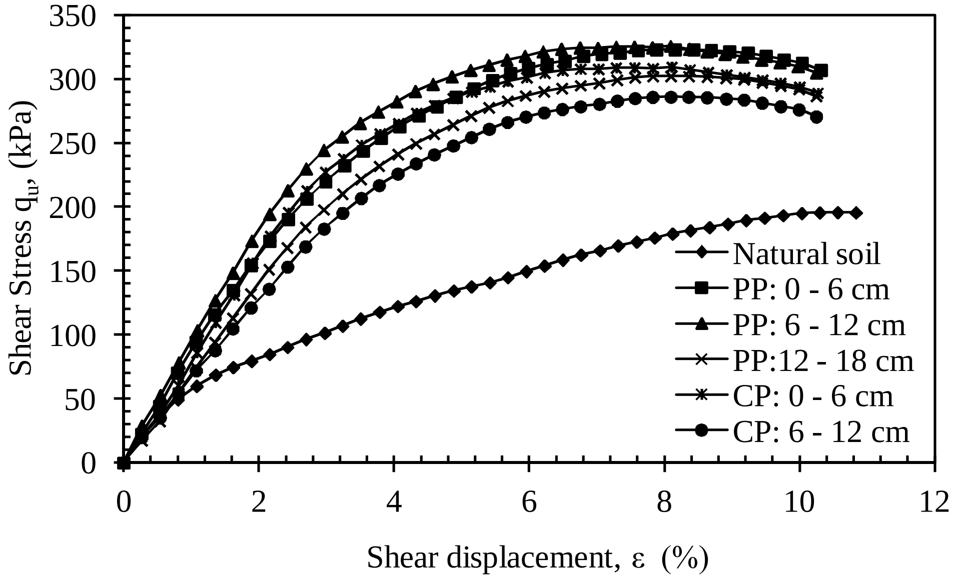


Figure 4.46: Stress-strain curve for the stabilized soil for curing periods of 90 days at different lime pile distances

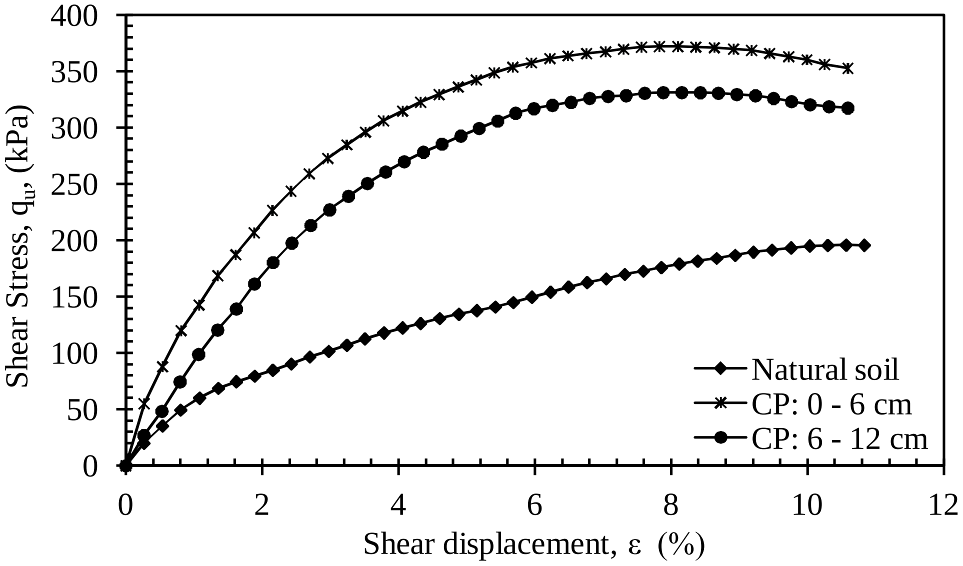


Figure 4.47: Stress-strain curve for the stabilized soil for curing periods of 120 days at different central pile to pile distances

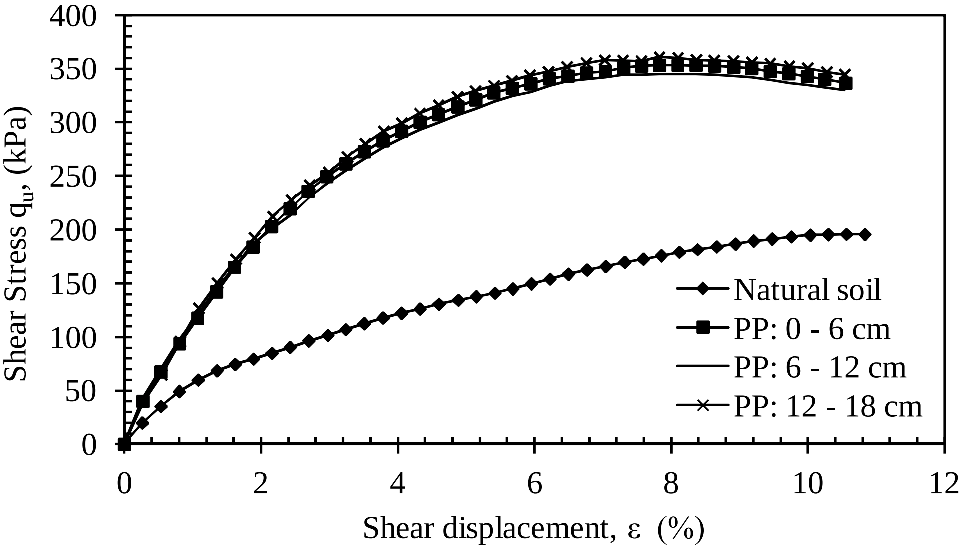


Figure 4.48: Stress-strain curve for the stabilized soil at different lime pile distances in curing periods of 120 days

In conclusion, the stress-strain curves for the stabilized soils presented the increment in strength at the effective zones of lime-pile radial distances within the range of 0 – 6 cm and 12 – 18 cm, while the stabilized soil within the range of 6 – 12 cm showed only a moderate gain in strength. The lime could not effectively migrate beyond the radial distance of 60 mm within 28 days curing period, but became more effective at curing periods of 120 days, hence produced a better strength and stiffness. This is due to the fact that the lime had been evenly distributed at 120 days of curing period. The undrained shear strength of the stabilized soil increased by 41%, 67% and 87% respectively for curing periods of 28 days, 90 and 120 days.

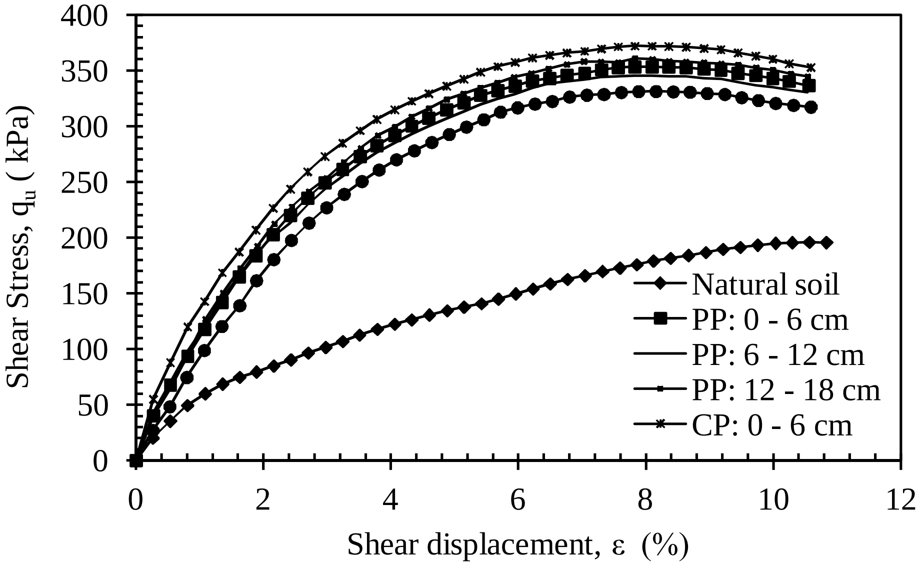


Figure 4.49: Stress-strain curve for the stabilized soil for curing periods of 120 days at different lime pile distances

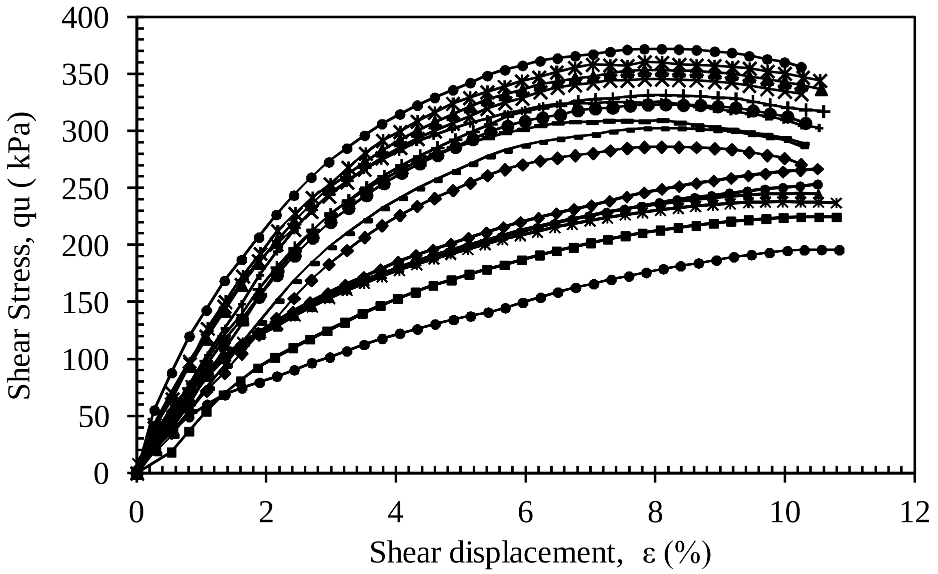


Figure 4.50 Stress-strain curves for the stabilized soils at different curing periods and lime pile distances

Figure 4.44 to Figure 4.50 indicated that there were more significant peaks in the stress-strain curves at longer curing periods (90 and 120 days) than it was for the short curing period. This increase is attributed to the higher binding force of clay plates by pozzolanic products formed by clay-lime reactions. The most significant improvement in strength was achieved at a distance close to lime-pile peripheral and in a curing period of 120 days. The samples extracted at radial distance of 0–6 cm from the lime-pile in curing periods of 120 days produced the highest strength in unconfined compressive test.

The increment in shear strength of stabilized soil is attributed to clay-lime physicochemical reactions (cation exchange capacity, pozzolanic reaction, flocculation) caused by the lime migration and decrease in amount of clay which invariably reduced the cohesive properties of the soil and increased its angle of internal friction (Tonoz et. al., 2003).

4.3.6 The Effect of Lime Piles on One-Dimensional Swell Potential of the Stabilized Soil

The most representative one dimensional swell curve of the stabilized soils extracted from the soil block in the test tanks are given in Figure 4.52 to Figure 4.61. In these curves, the percent vertical swell was plotted against arithmetic time and logarithmic time in minutes. These figures provide the behavior of swelling characteristics of the stabilized soils extracted from different radial locations of 0 – 9 cm and 9 – 18 cm in curing periods of 28, 90 and 120 days.

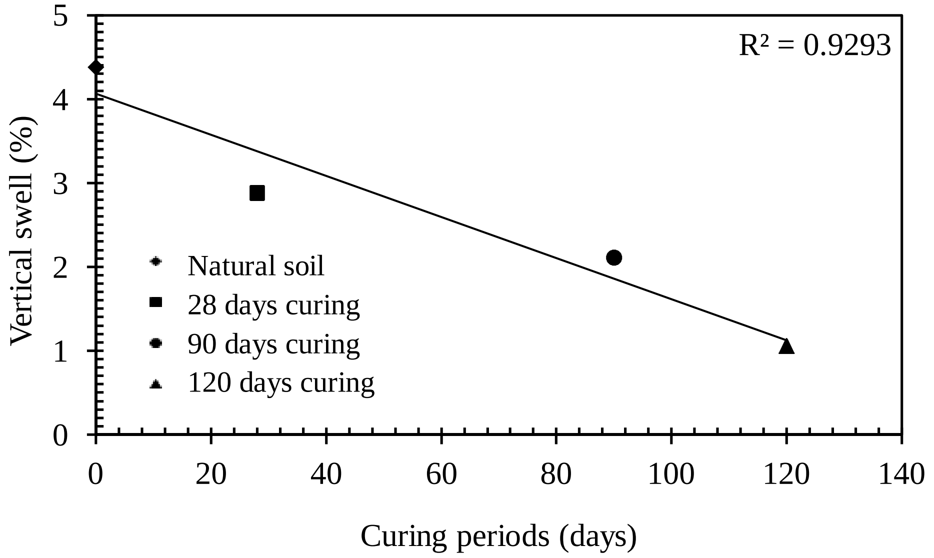


Figure 4.51: Percent vertical swell versus curing periods for the natural and stabilized soils

In Figure 4.51, it can be deduced that the higher the curing period, the lower the percent vertical swell.

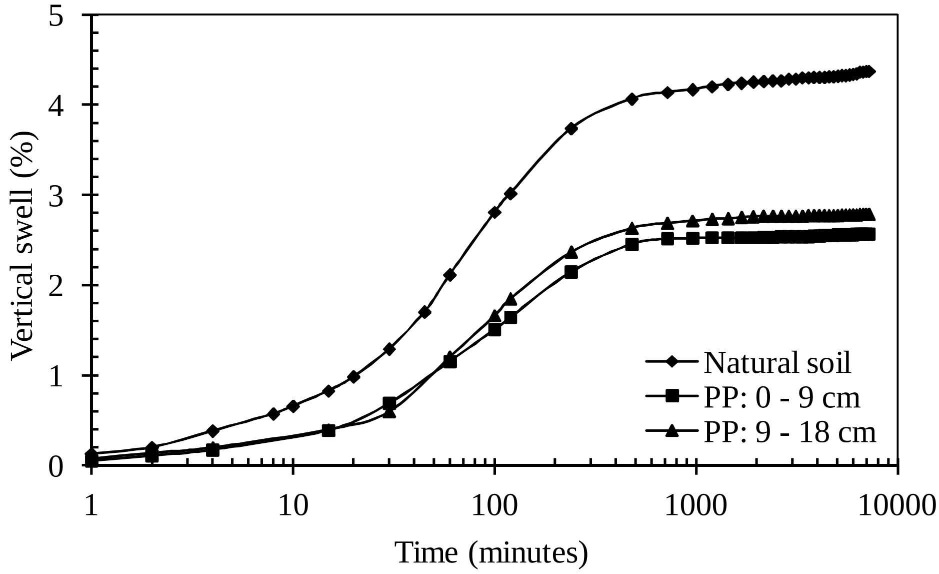


Figure 4.52: Swell-time curves of the stabilized soil at different curing distances in curing periods of 28 days

In Figure 4.52, the percent secondary vertical swell recorded for the stabilized soils in 28 days of curing were 2.57% and 2.79%. The percent primary vertical swell were determined to be 2.50% and 2.15%. Therefore, secondary swell potential of the stabilized soil was reduced by 42% and 36% and primary swell by 38% and 46% respectively within the specified lime pile radial distances. This is a distinct modification in the swell properties of the stabilized soil when compared with the natural soil. The test results showed that due to the physicochemical reactions between clay and lime, the swell percentage of the soils was reduced because of the collapse of the DDL and the reduction in the amount of clay particles.

In Figure 4.53, the swell curves of the stabilized soils for 90 days curing are provided. The reduction in the swell potential of the stabilized soil extracted at the radial distances of 0 – 9 cm and 9 – 18 cm were by 50% and 52% respectively. The secondary swell potential values were determined from the vertical percent swell of 2.22% and 2.38% , while primary swell potential values were 1.55% in 90 curing days. The reduction in primary swelling was determined to be 61%.

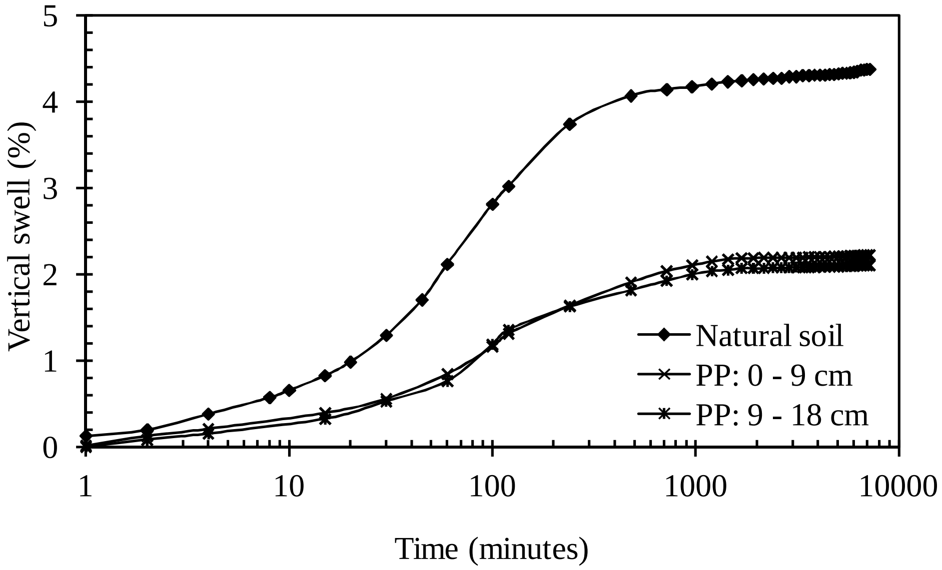


Figure 4.53: Swell-time curves of the stabilized soil at different curing distances in 90 days of curing periods

Furthermore, the secondary swell percent of the stabilized soil extracted in 120 days curing periods are determined to be 0.92% and 1.05% with reduction of 75% and 78% at the lime pile distances of 9 cm and 18 cm, and the primary swell percent was determined to be 0.5% on average, with reduction of 88% indicated in Figure 4.54.

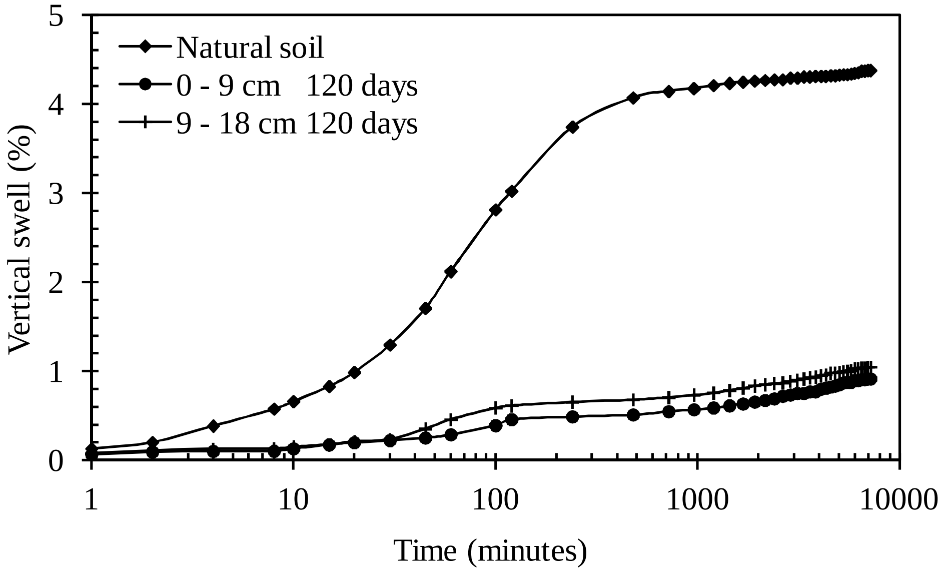


Figure 4.54: Swell-time curves of the stabilized soil at different lime pile distances in 120 days of curing periods

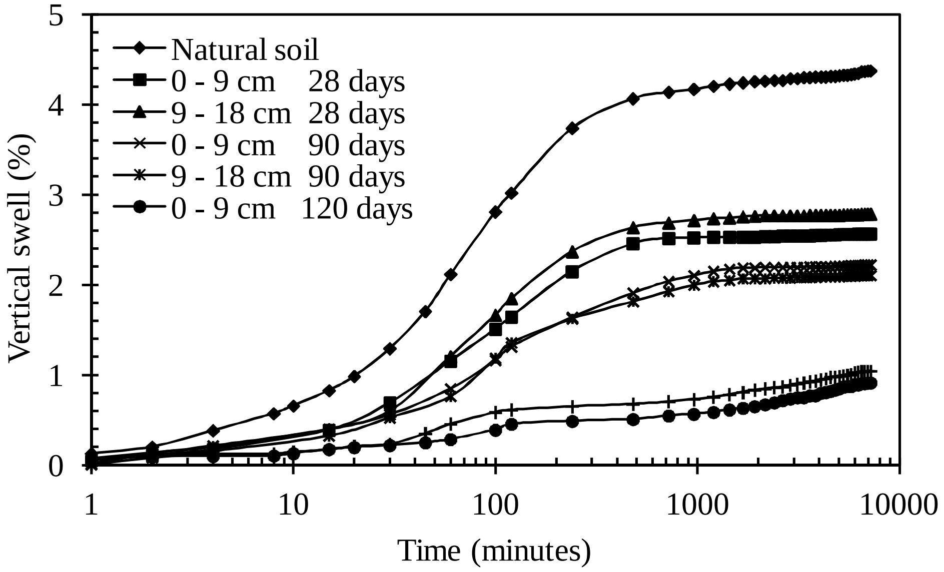


Figure 4.55: Swell-time curves of the stabilized soil at different lime pile distances and curing periods

These yielded an outstanding decrement in the swelling characteristics of the stabilized soil. The improvement in the swelling properties of the stabilized soil is highly commendable which makes the soil appropriate and suitable for foundation materials in engineering constructions.

The combined swell curves of the stabilized soils are presented Figure 4.55. As it can be observed from these curves, though the reduction in swell potential achieved in 90 and 120 curing days was higher than in the 28 curing days. It is evident that the significant reduction in swell potential actually occurred during the 28 days of curing when compared with the natural soil. This is in conformity with the work of Tonoz et al. (2003). This can be attributed to the rapid clay-lime physicochemical reactions which occurred within this curing period.

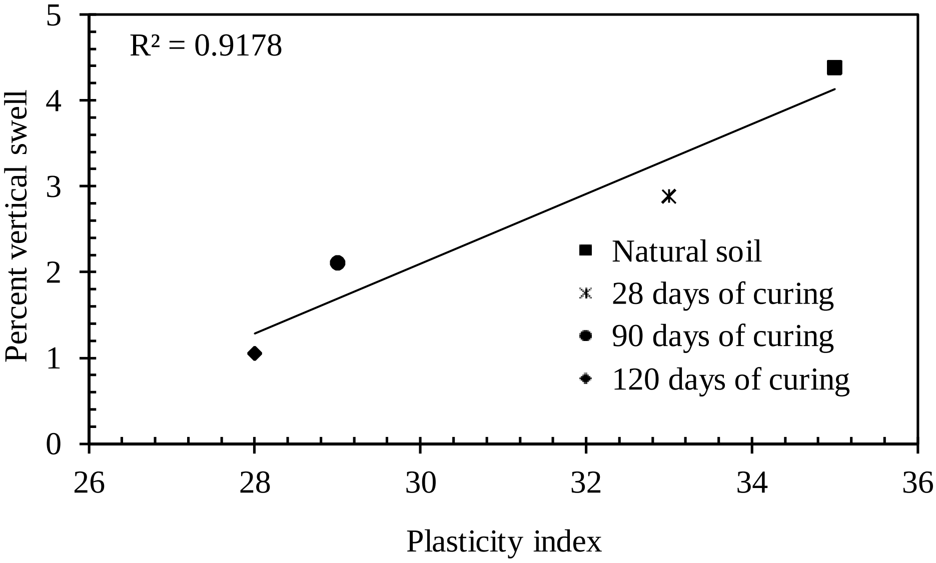


Figure 4.56: Percent swell versus PI of stabilized soil at different curing periods

Figure 4.56 represents the percent swell versus the plasticity index of the stabilized soils extracted at different locations and in different curing periods. It can be inferred from the curve that the swell potential of the stabilized soil reduced with a reduction in the PI values.

The amount of volume change in soil is proportional to the change in stress applied, electrolyte type and its concentration, type of cation present and the pore fluid dielectric constant. The kind and composition of clay minerals present invariably affect the intensity of the swell or shrinkage properties of soil (Sridharan, 2002). Swelling of soils is directly proportional to the plasticity properties, that is, the more the plastic the mineral contents, the more its swelling potential (Chen, 1975; Sridharan, 2002). The test results showed that the clay-lime interactions which reduced the amount of clay decreased the soil plasticity index (Figure 4.56), due to cation exchange of monovalent cations with divalent cation of Ca^{2+} ions and finally reduced the swelling percent of the soil.

According to the Gouy-Cahapman double layer thickness theory, the reduction in the percent swelling of the stabilized soil can also be linked to the collapse of the DDL, due to the presence of the Ca^{2+} ions, which increased the pore fluid concentration, and reduced the dielectric constant.

4.3.7 The Effect of Lime Piles on One-Dimensional Consolidation of the Stabilized Soil

The samples that were extracted from the stabilized soil blocks at different lime-pile distance and curing periods were subjected to one-dimensional consolidation test. These tests were performed to study the effects of quicklime on the compressibility properties of the stabilized soil as a result of lime migration from the piles into the surrounding soil. The variations in compressibility characteristics including preconsolidation pressure ($\sigma p'$) and swell pressure with respect to curing periods and distances to the lime-pile were carefully studied in details.

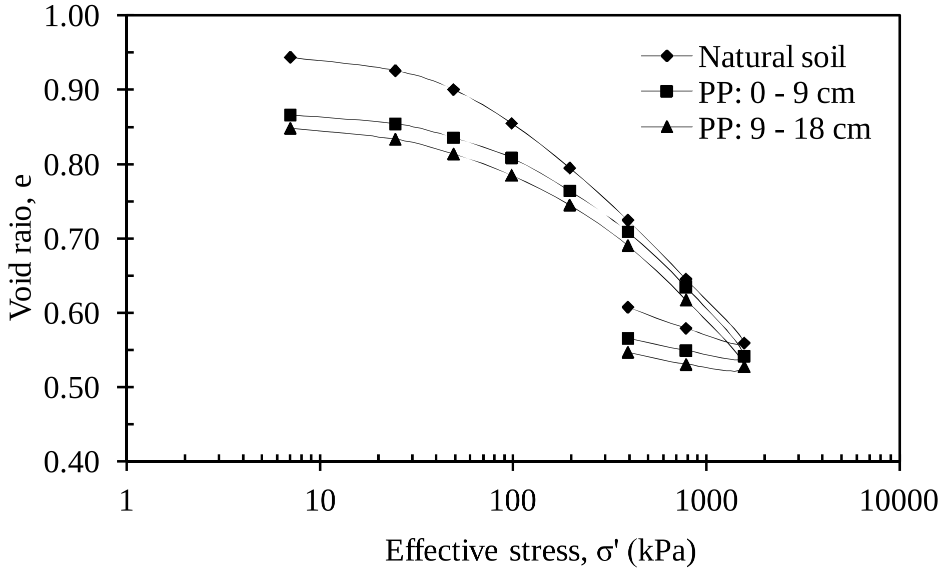


Figure 4.57: Void ratio-log pressure curves of the stabilized soil at different lime pile distances and 28 days curing periods

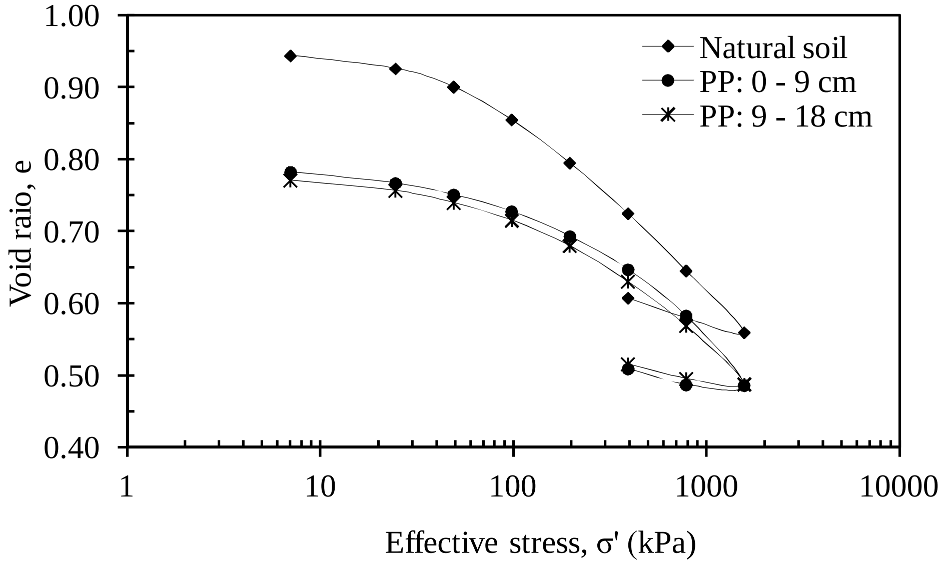


Figure 4.58: Void ratio-log pressure curves of the stabilized soil at different lime pile distances and 90 days curing periods

The *e-log p* curves for the quicklime treated soils at the specified curing periods are represented in Figure 4.57 to Figure 4.9. Each *e-log p* curves represents the compressibility characteristics of the stabilized samples which were extracted at different lime-pile distances of 0 – 9 cm and 9 – 18 cm respectively.

In Figure 4.57, the *e-log p* curves of stabilized soil extracted in 28 days are represented. It can be seen that the compression index, C_c was reduced from 0.26 to 0.20 within the radial distances of 0 - 9 cm which is about 23% reduction. Within the lime-pile distance of 9 – 18 cm, the compression index decreased by 12%. The reduction in rebound index achieved was 60% and 50% at the lime-pile distances of 0 – 9 cm and 9 – 18 cm respectively for the same curing period. In Table 4.10, it can be seen that there was a significant improvement in the consolidation coefficient and the average degree of consolidation of the stabilized soil.

In Figure 4.58, the compression index, C_c was reduced by 31% within the lime-pile distances in 90 days of curing. The reduction in rebound index achieved was 60% and 60% at the lime-pile distances of 0 – 9 cm and 9 – 18 cm respectively. In Figure 4.59, the void ratio versus log pressure curves for 120 days of curing were given.

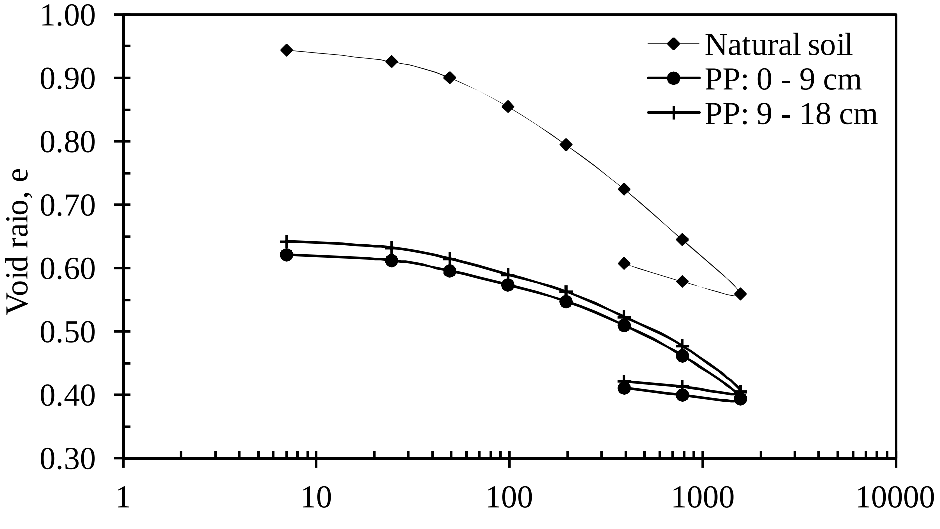


Figure 4.59: Void ratio-log pressure curves of the stabilized soil at different lime pile distances and 120 days curing periods

The compression index reduced by 62% and rebound index by 60%. The decrease in the compressibility properties of the stabilized soil close to the lime pile was achieved within curing periods of 28 days and this is in good agreement with the investigation of Tonoz et al. (2003) and Rajasekaran et al. (2000).

Table 4.11 indicated that there was also a significant reduction in the swell pressure value. The value decreased from 180 kPa to 60 kPa (approximately 67% reduction) for the samples taken within 90 mm radial distance. In Table 4.11, it can be seen that preconsolidation pressure, $\sigma p'$ increased from 95 kPa – 170 kPa (approximately 80% increments) for samples extracted from the same radial distance. The higher the curing periods, the greater the hydraulic conductivity obtained as indicated in Table 4.10. These findings can be attributed to an increase in the concentration of the pore fluid, increase in pore size, more granular soil aggregates and a decrease in void ratio.

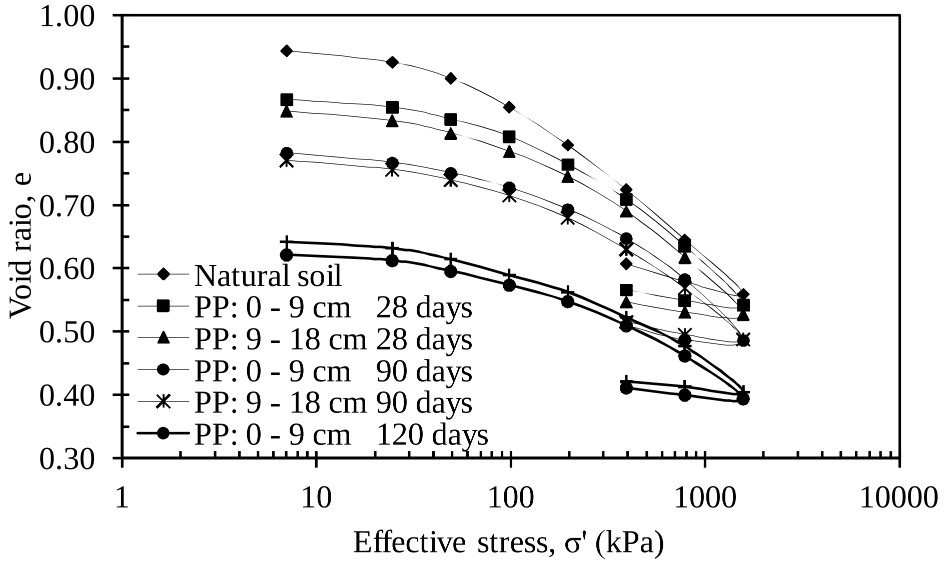


Figure 4.60: Void ratio-log pressure curves of the stabilized soil at different lime-pile distances and curing periods

Table 4.10: Compressibility characteristics of natural and stabilized soils extracted at different lime-pile distances and curing periods

Soil type: Curing distance Curing periods	Coefficient of consolidation C_v (m^2/s)	Compression index C_c	Rebound index C_r	Degree of consolidation t_{90} (min)
Natural Soil	6.80E-08	0.26	0.10	13.2
SS: 0 – 9 cm 28 days	1.31E-07	0.20	0.07	6.48
SS: 9 – 18 cm 28 days	1.18E-07	0.23	0.05	7.15
SS: 0 – 9 cm 90 days	5.39E-07	0.21	0.07	6.12
SS: 9 – 18 cm 90 days	3.19E-07	0.18	0.04	6.27
SS: 9 – 18 cm 120 days	9.52E-06	0.12	0.04	5.01
SS: 9 – 18 cm 120 days	9.97E-06	0.10	0.04	4.78

*SS = stabilized soil

Table 4.11: Swell pressure, preconsolidation pressure and hydraulic conductivity of natural and stabilized soils extracted at different lime-pile distances and curing periods

Soil type: Curing distance Curing periods	Swell Pressure p_s' (kPa)	Preconsolidation Pressure $\sigma p'$ (kPa)	Coefficient of hydraulic conductivity k (m^2/s)
Natural Soil	180	95	8.61E-11
SS: 0 – 9 cm 28 days	110	129	1.17E-10
SS: 9 – 18 cm 28 days	98	135	3.18E-10
SS: 0 – 9 cm 90 days	60	150	7.42E-10
SS: 9 – 18 cm 90 days	60	170	5.57E-10
SS: 0 – 9 cm 120 days	55	176	7.15E-09
SS: 9 – 18 cm 120 days	51	177	6.11E-09

*SS = stabilized soil

4.3.8 The Effect of Lime Piles on California Bearing Ratio of the Stabilized Soil

The CBR test was conducted in order to study the improvement the in the strength of the stabilized soil with respect to the migration of the calcium ions from the piles. In addition, the test was conducted to substantiate on the suitability of the stabilized soil for highway and foundation constructions. The test was performed on the stabilized soil extracted from the test tank with the mold illustrated in Figure 3.21 and Figure 3.22. Therefore, in each test tank, due to the dimension of the mold, and the limited space in the test tank, only one sample was extracted between the two lime-piles distances in the stabilized soil block.

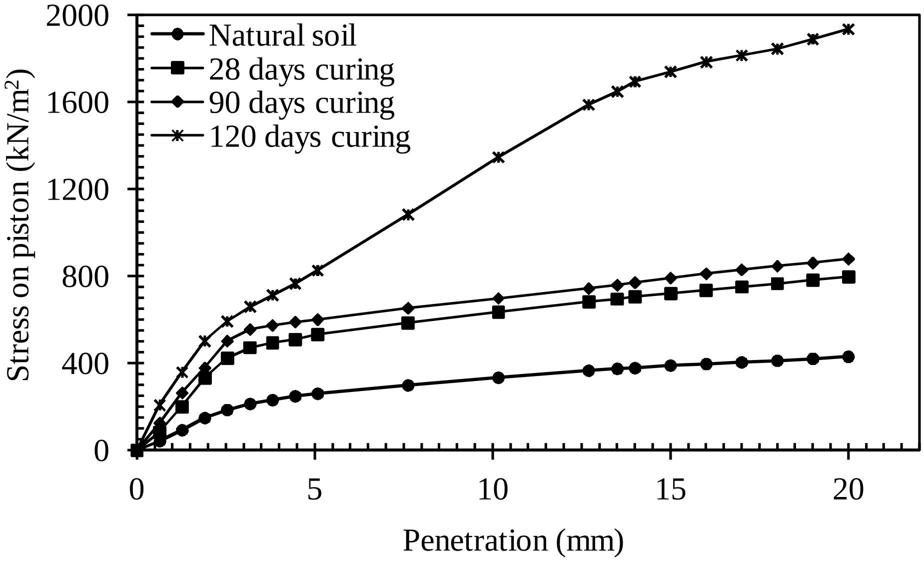


Figure 4.61: CBR curves of the stabilized soil at different curing periods

In Figure 4.61 and Figure 4.62, both curves indicate that the CBR number of the stabilized soils increased from 2.7% to 6.1%, 7.3% and 8.9% for curing period of 28, 90 and 120 days respectively. Figure 4.62 simply indicates that the higher the curing periods the higher the CBR number obtained.

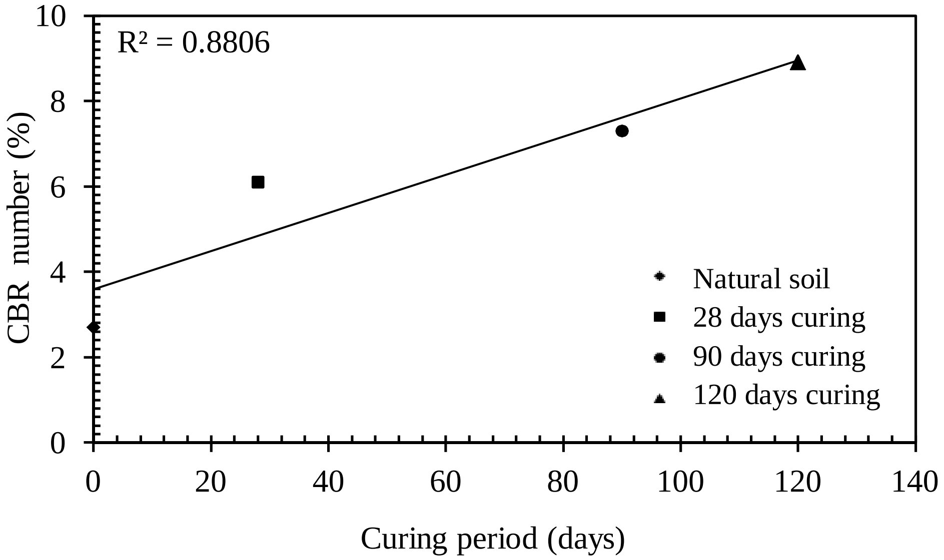


Figure 4.62: The CBR number versus curing periods

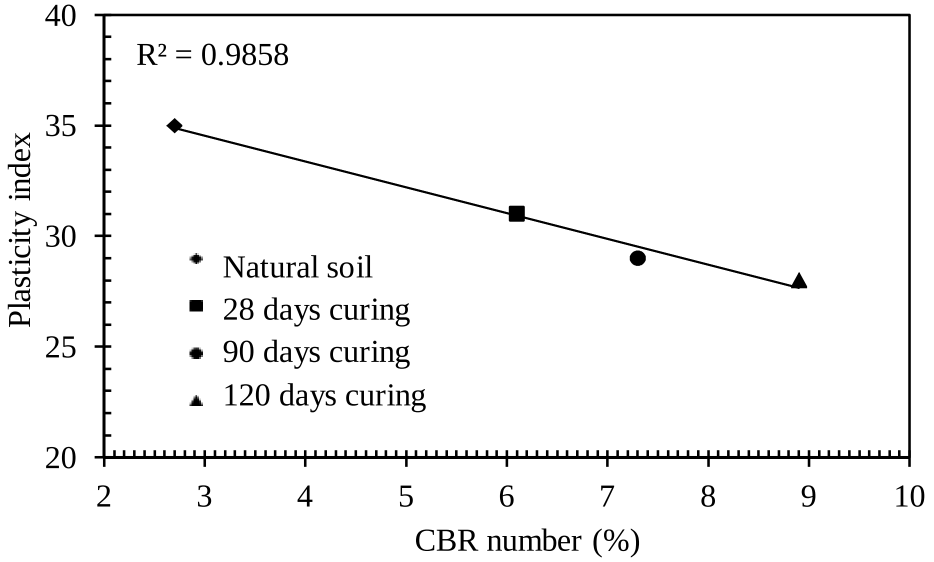


Figure 4.63: The CBR number versus plasticity index of the natural soil and the stabilized soils at different curing periods

In Figure 4.63 and Figure 4.64, it can be seen there was a significant increase in the properties of the stabilized soil. In Figure 4.63, the relationship between the CBR numbers versus the plasticity index values obtained from the natural and stabilized soils in each curing period are indicated. It was observed that as the plasticity index was reduced by increasing the curing periods, the CBR numbers increased.

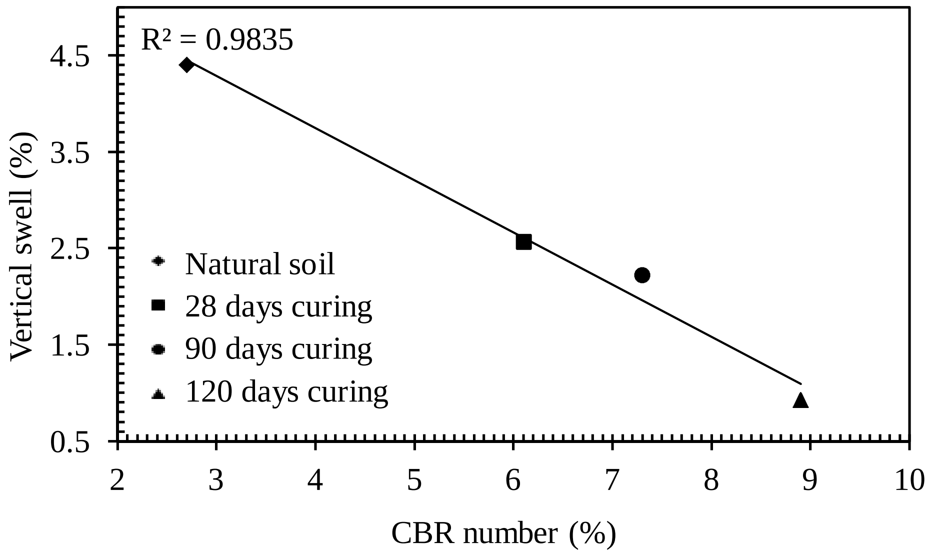


Figure 4.64: The CBR number versus percent vertical swell of the natural soil and the stabilized soil at different curing periods

In Figure 4.64, the CBR numbers versus the percent vertical swelling was presented. It can be examined that the lower the percent vertical swell attained, the higher the CBR number. This indicates that the clay size fraction had reduced, which invariably reduced the plasticity index and swelling of the stabilized soil, hence, yielded high stability and strength.

In Figure 4.65 and Figure 4.66, the higher the compaction characteristics of the soil, OMC and MDD obtained for each curing period, the CBR number increased.

In Table 4.12, the USCS of CBR numbers for the purpose of determining the suitable soil required for highway and foundation constructions is illustrated. In the UCSC general ratings, there was an indication that the natural soil was very poor. After improvement within 28, 90 and 120 curing days, the stabilized soil became poorly fair to fair materials respectively. The natural soil and the stabilized soil at curing period of 28 days can be used as both the subgrade and subbase materials. The stabilized soils at 90 and 120 days of curing can be used as both the subgrade and subbase materials in highway construction.

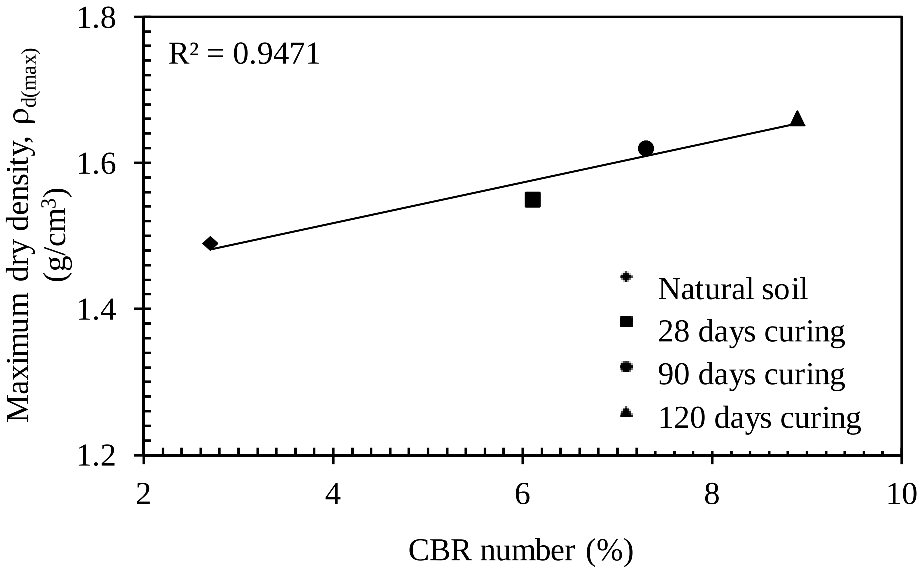


Figure 4.65: The CBR number versus maximum dry densities of the natural soil and the stabilized soils at different curing periods

In Figure 4.64, the CBR numbers versus the percent vertical swelling was presented. It can be examined that the lower the percent vertical swell attained, the higher the CBR number. This indicates that the clay size fraction had reduced, which invariably reduced the plasticity index and swelling of the stabilized soil, hence, yielded high stability and strength.

In Figure 4.65 and Figure 4.66, the higher the compaction characteristics of the soil, optimum moisture content and maximum dry density obtained for each curing period, the CBR number increased.

In Table 4.12, the USCS of CBR numbers for the purpose of determining the suitable soil required for highway and foundation constructions is illustrated. In the UCSC general ratings, there was an indication that the natural soil was very poor. After improvement within 28, 90 and 120 curing days, the stabilized soil became poorly fair to fair materials respectively.

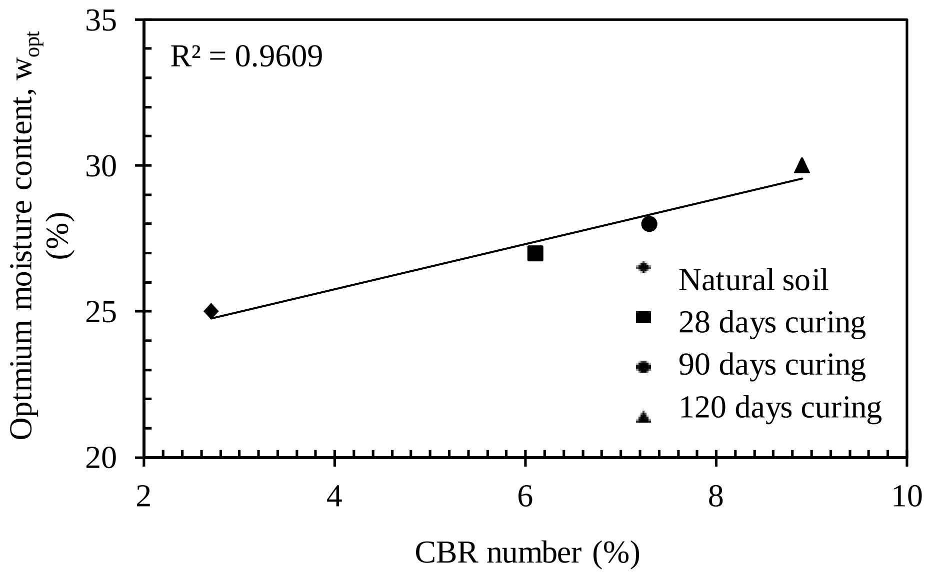


Figure 4.66: The CBR number versus optimum moisture content of the natural soil and the stabilized soils at different curing periods

The natural soil and the stabilized soil at curing period of 28 days can be used as both the subgrade and subbase materials.The stabilized soils at 90 and 120 days of curing can be used as both the subgrade and subbase materials in highway construction.

Table 4.12: The USCS classification system for CBR numbers (The Asphalt Institute, 1970)

CBR No. (%)	General rating	Uses	Unified
0 – 3	Very poor	Subgrade	OH, CH, MH, OL
3 – 7	Poor to fair	Subgrade	OH, CH, MH, OL
7 – 20	Fair	Subbase	OL, CL, ML, SC, SM, SP
20 – 50	Good	Base, subbase	GM, GC, SW, SM, SP, GP
> 50	Excellent	Base	GW, GM

4.3.9 The Effect of Lime-Piles Electrical Conductivity of the Stabilized Soil

In this study, the main controlling factors of electrical conductivity (EC) in the stabilized soil are the pore fluid, particle orientation and shape, cementitious products, the electrolyte concentrations (Ca^{2+} ions) and diffused double layer.

In Table 4.13, the EC of the stabilized soil increased as the number of curing days increases. The terminology electrical resistivity (ER) and electrical conductivity (ER) are used exchangeable in this study. In equation 1.6 given in Chapter one, the expression indicates they are two reciprocal electrical quantities. In Figure 4.67, shows that the EC increased as the curing days increased.

Table 4.13: Electrical resistivity, electrical conductivity and unconfined compressive strength values obtained at different lime pile distances and curing periods

Soil type: Curing distance Curing periods	Electrical Resistivity ohm centimeter $\Omega\cdot m$ (10^{-4})	Electrical Conductivity Siemens per meter Sm^{-1} ($x10^{2}$)	Unconfined Compressive strength (kPa)
Natural soil	27.15	3.68	196
PP: 0 – 6 cm 28 days	4.1093	21.67	271
PP: 6 – 12 cm 28 days	7.5463	20.94	225
PP: 12 – 18 cm 28 days	4.6174	23.38	245
CP: 0 – 6 cm 28 days	6.5742	21.31	256
CP: 6 – 12 cm 28 days	1.4499	20.73	238
PP: 0 – 6 cm 90 days	1.9772	68.97	323
PP: 6 – 12 cm 90 days	1.5356	50.58	302
PP: 12 – 18 cm 90 days	1.3338	65.12	326
CP: 0 – 6 cm 90 days	1.1160	74.74	308
CP: 6 – 12 cm 90 days	1.3115	72.95	286
CP: 12 – 18 cm 90 days	1.2028	76.25	310
PP: 0 – 6 cm 120 days	1.2535	81.75	353
PP: 6 – 12 cm 120 days	2.5242	79.77	345
PP: 12 – 18 cm 120 days	1.2054	82.96	360
CP: 0 – 6 cm 120 days	1.1800	84.34	372
CP: 6 – 12 cm 120 days	1.1899	83.69	331
CP: 12 – 18 cm 120 days	1.1186	84.29	-

*PP: pile to pile distances, CP: central pile to pile distance

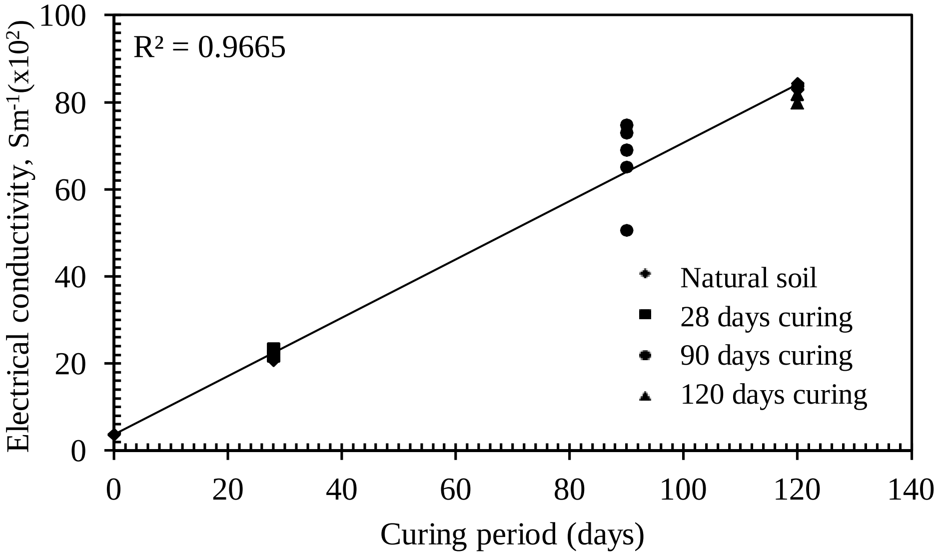


Figure 4.67: The Electrical conductivity values versus curing periods

Figure 4.68 to Figure 4.70, show that the EC increased as the curing days increased. This is due to the fact that because the lime-clay reaction increased the pore fluid concentration and the formation of cementious materials, increased the electrical conductivity of the soil at increasing curing periods.

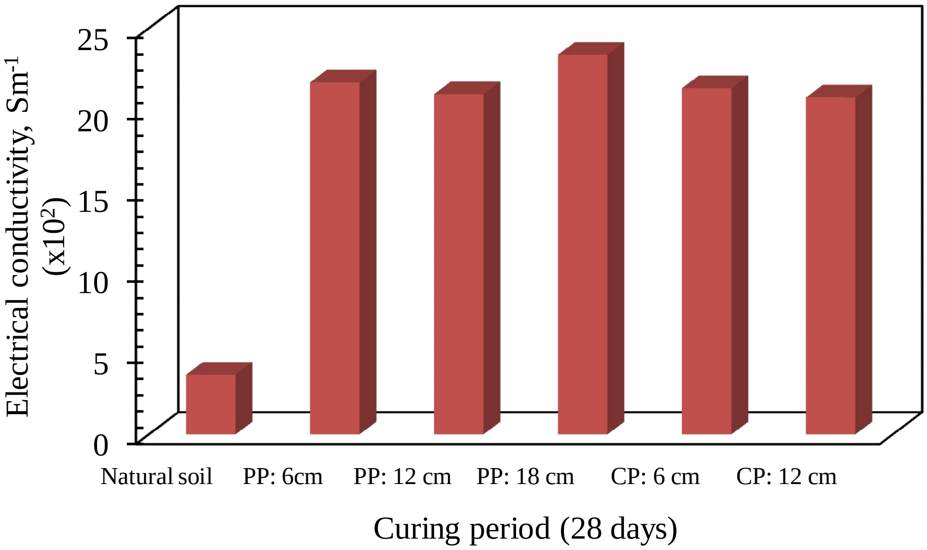


Figure 4.68: The Electrical conductivity at different lime-pile distances in 28 days of curing

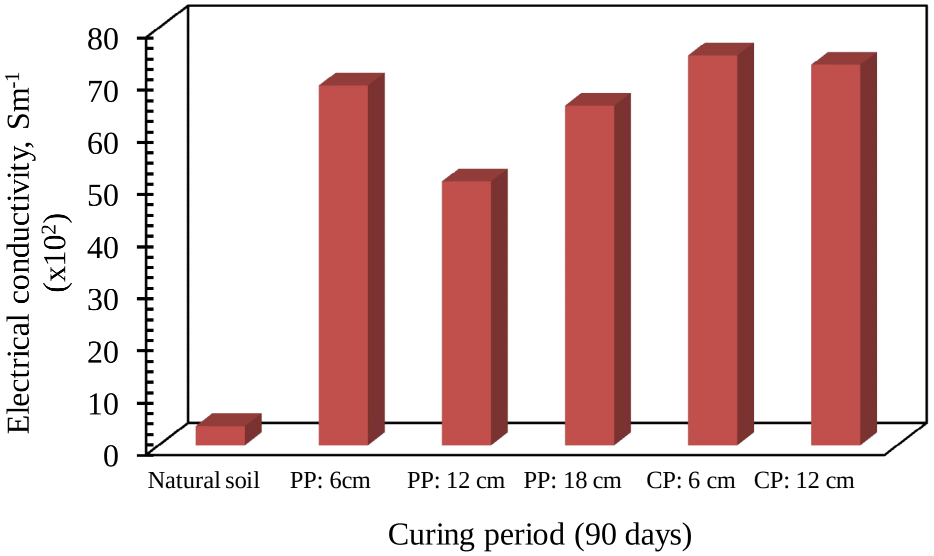


Figure 4.69: The Electrical conductivity at different lime-pile distances in 90 days of curing

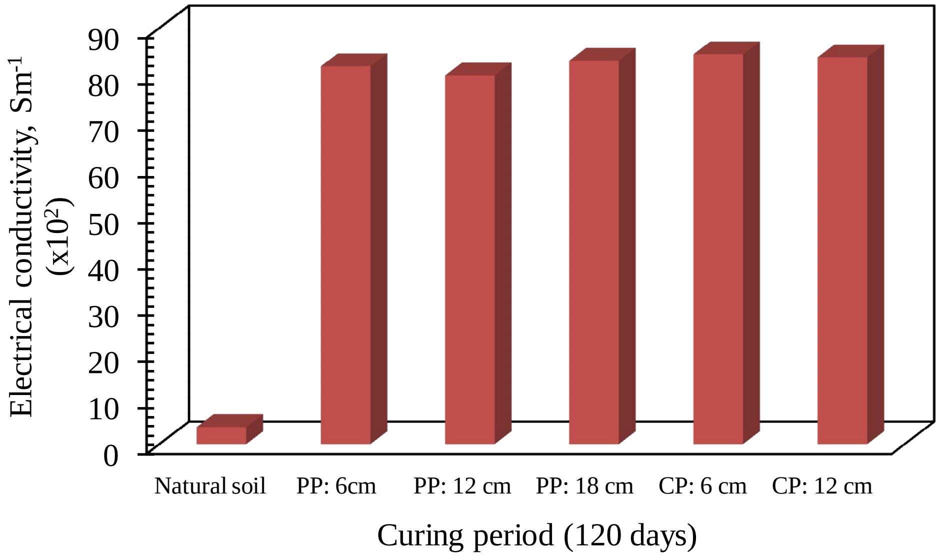


Figure 4.70: The Electrical conductivity at different lime-pile distances in 120 days of curing

It can be seen that the EC increased across as the curing days increased. This is due to the fact that the lime stabilized soil has become more granular in nature due to the flocculation and aggregation as a result of which the cementation effect between the particles increased the electrical conductivity.

The lime migration into the soil block increased the ionic concentration in the pores of the treated soils. It can be deduced that the higher the concentration of Ca^{2+} ions, the higher the EC.

In Figure 4.71 to Figure 4.73, the EC versus unconfined compressive strength (UCS) were correlated. The figures indicated that UCS of the soil increased with increment in the EC of the stabilized soil. The idea was to study the change in the amount of dissolved soluble Ca^{2+} ions in the soil. The higher the EC of the soil, the higher is the flocculating property of the soil and its aggregate stability and strength. The aggregate stability (flocculating power of cations) are dependent on the balance between divalent ions (Ca^{2+} and Mg^{2+}) and monovalent ions Na^+ as well as the soluble salts content (Mitchell, 2005).

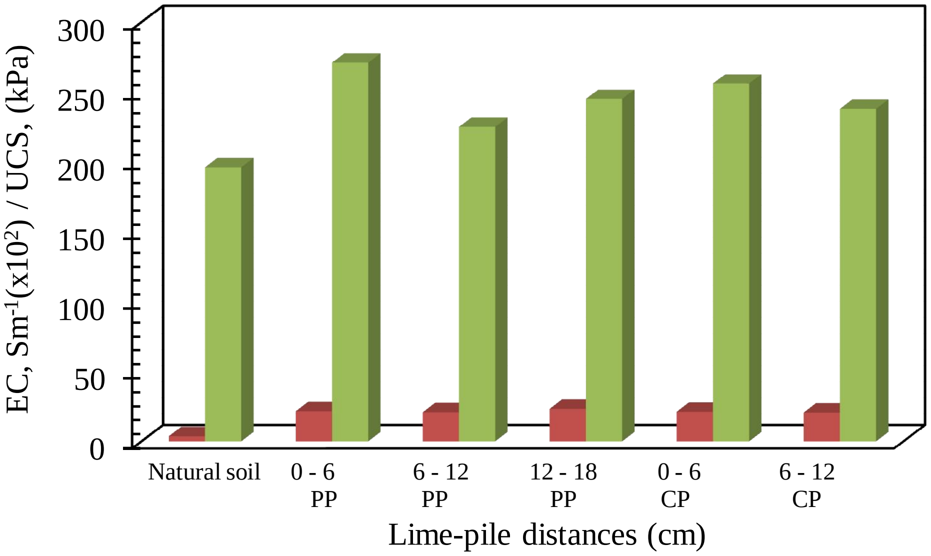


Figure 4.71: The Electrical conductivity versus unconfined compressive strength in 28 days of curing period

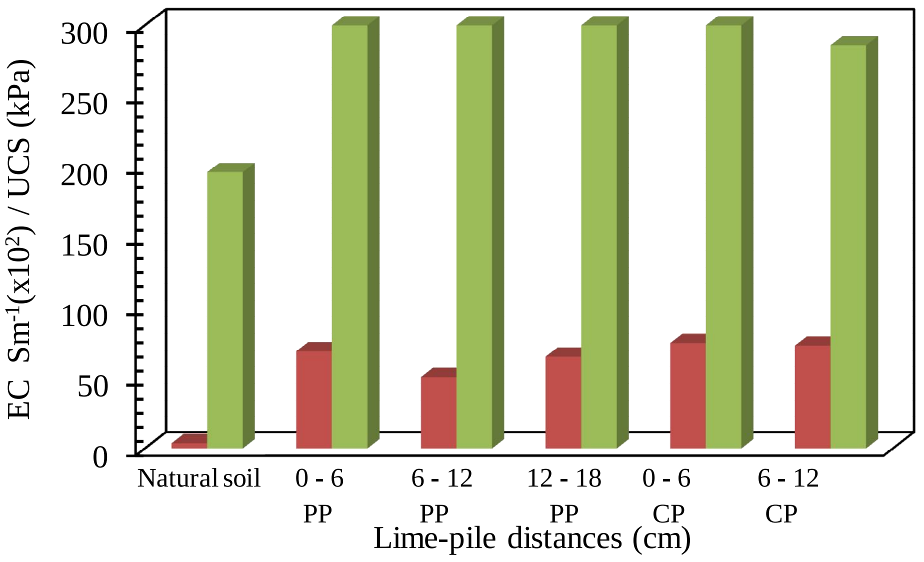


Figure 4.72: The Electrical conductivity versus unconfined compressive strength in 90 days of curing period

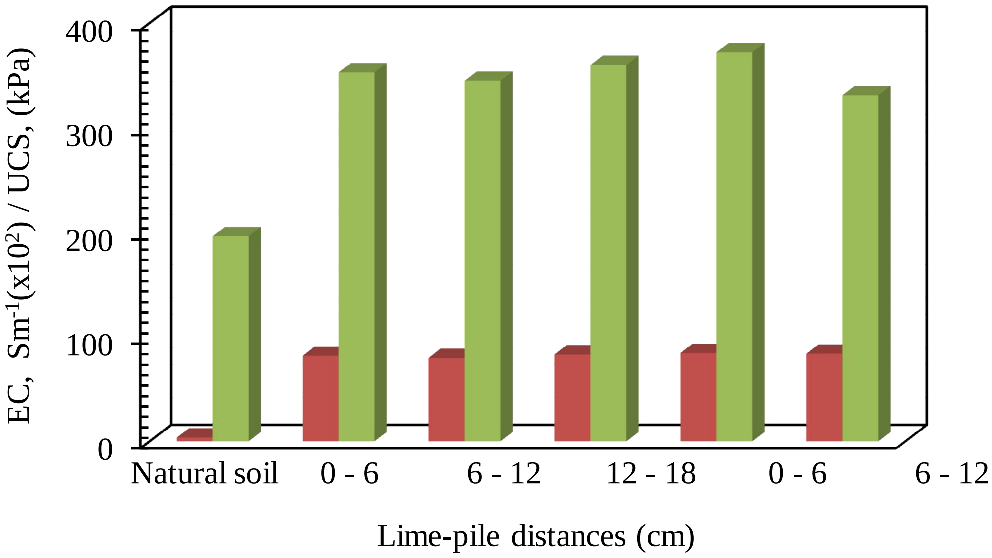


Figure 4.73: The Electrical conductivity versus unconfined compressive strength in 120 days of curing period

In Figure 4.74, the C versus UCS for natural and stabilized soils at different lime-pile distances and curing periods are given. As discussed earlier, the electrical conductivity of the stabilized soils increased as the curing periods increased. Because of the cementation products formed due to the lime-clay reaction, higher unconfined compressive strength values were obtained and as a result of cementation, the bonding between the solid particles created a bridge between the particles and that increased the conductivity of the treated soils.

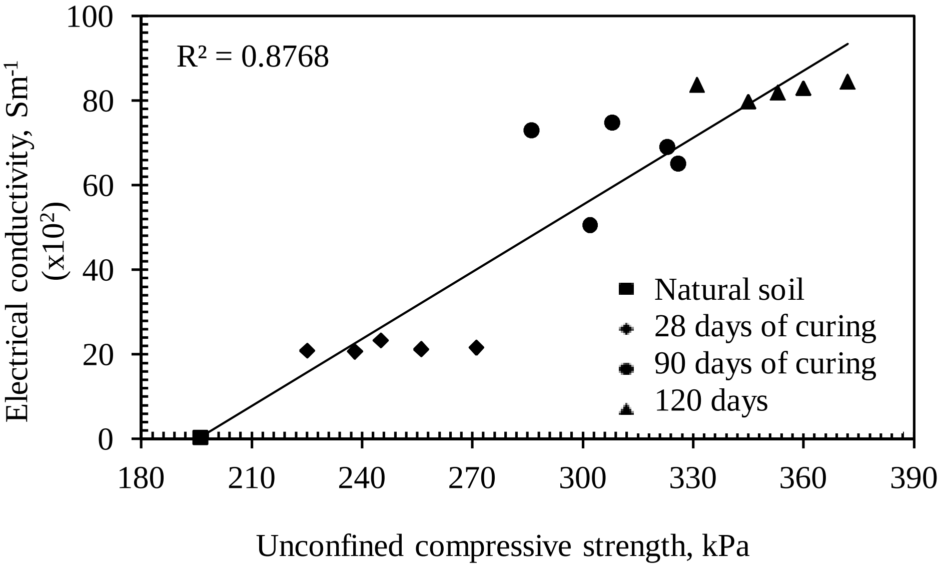


Figure 4.74: The Electrical conductivity versus unconfined compressive strength for natural and stabilized soils at different lime-pile distances and curing periods

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

Based on the experimental results and engineering properties of the stabilized soils in the current study, the following conclusions can be highlighted:

Increasing the pore fluid concentration of the soil (natural ionic fluid) to calcium ions supplied from the lime-piles caused a significant modification in the physical and engineering properties of the soil. It was observed that the clay mineralogy, its structure and all its properties seemed to be a function of its pore fluid content, its pore fluid concentration and the curing periods. These factors compelled a distortion of the diffuse or stern double layer of the clay which resulted in the formation of the flocculated and aggregated clay.

The increase in the curing periods and variation in the lime-pile distances resulted in the formation of different degree of stabilization using the lime-piles. The stabilized soil attained different engineering properties at different curing periods and tended to approach similar engineering properties as the curing periods increased over time. This indicated that the clay-lime physicochemical reactions is continuous in process and change the soil properties.

The changes in the electrolyte concentration resulted in alteration of the sedimentation time of the fines content in hydrometer test. With the collapse of the diffused double layer and dielectric constant due to lime clay migration, the stabilized, aggregated soil resulted in reduction in clay size particles.

With the increment in the curing periods which resulted in a higher electrolyte (lime) concentration, the plasticity of the clay reduced. The changes in the fabric nature of the clay to a more granular texture and reduction in its affinity for water caused a drastic reduction in the plasticity index of the soil. The liquid limit and plastic limit followed a regular reduction trend as the curing days increased in the stabilized soil which was close to the lime-pile distance.

The swelling and shrinkage characteristics of the soil decreased when subjected to the lime-pile treatment at different curing periods and lime-pile distances. The swelling and shrinkage of clay are the functions of its fines content. The stabilized soils with lower fines content had a significant reduction in their one dimensional vertical swell potential and longitudinal linear shrinkage. This was achieved with an increased curing period and the reduced distance to the lime-pile. The swelling pressure of the stabilized soil reduced remarkably

The change in the compressibility characteristics of the soil is proportional to its stress history, particle orientation, size and structure. The flocculation property of the stabilized soil made the soil to be more granular in nature and became less cohesive but attained more frictional properties. Hence, the stabilized soil became more incompressible. The degree of incompressibility of the stabilized soil increased as the curing periods were increased from 28, 90 and 120 days and at the close distance of the lime-piles. With the continuous increment in the concentration of calcium ions in the pore fluid over the curing periods, the average degree of consolidation (t_{90}) and the consolidation coefficient, C_v increased. This invariably reduced the compression and expansion indices of the soil as the curing periods increased. The results indicated an increase in the hydraulic conductivity of the soil with increasing curing periods. This can be attributed to an increment in the degree of flocculation and a consequent increase in the pore sizes of the stabilized soils.

The result of the analysis indicated a significant increase in the unconfined compressive strength of the soil with increasing curing periods at closer lime-pile distances. The improvement in the structure of the soil made the particles to form a stronger bond. The formation of the cementitious and pozzolanic materials also provided a better shearing resistance and stiffness. This produced a higher UCS and undrained shear strength of the treated soils.

The CBR number of the soil improved for each curing period. The natural soil which was categorized as a very poor soil. After improving the soil with lime piles, the CBR number became fair and the stabilized soil can be used as subgrade and subbase materials in highway construction. In addition, the clay-lime reactions are continuous; therefore, the stabilized soil can still attain a better condition in longer curing periods.

The electrical conductivity of the soil increased with increasing concentration of the electrolyte (Ca^{2+} ions) in the pore fluid. That was due to the formation of the cementation products in the lime-clay reaction. The bonding created from the formation of cementitious materials calcium silicate and aluminate hydrates increased the surface conductance of the treated soils. In addition, the increment in the concentration of the pore fluid increased the conductive properties of the stabilized soils and resulted in a reduction in electrical resistivity. Further reduction in electrical resistivity was obtained with the increase in curing time. The electrical resistivity test results suggest that the electrical resistivity measurements can be used as a monitoring technique for lime diffusion in in-situ lime-pile applications.

5.2. Recommendations

Many research works had been performed on the deep ground chemical stabilization on various problematic soils, but in spite of that, limited research studies have been done on the effect of electrical resistivity and electrical conductivity due to the lime migration of chemical binder(s) (e.g., lime) in treated soils.

Therefore, differences between the electrical resistivity and electrical conductivity in stabilizing soils using different chemical binders should also be studied.

In addition, further study on the migration of lime at microscopic level should be studied. Geotechnical engineering field applications should be developed for the right quantities of chemical binder(s) needed for deep mixing soil stabilization techniques and the right curing time required prior to construction.

REFERENCES

Agus, S. M. and Hung J. (2006). Shear strength around soft clay surrounded by lime columns. Dept. of Construction Engrg: Univ. Muhammadiyah, Yogyakarta, Indonesia and National Taiwan Univ. of Sci. & Tech., Taiwan.

Åhnberg, H., Johansson, S.E., Retelius, A., & Ljungkrantz, C. (1995). Cement and lime for deep stabilisation of soils—a chemical/physical study of stabilisation effects. Swedish Geotechnical Institute, Linköping. Report No. 48 (in Swedish).

Åhnberg, H (2006). Strength of Stabilized Soils: A Laboratory Study on Clays and Organic Soils Stabilized with Different Types of Binder, Swedish Deep Stabilization Research Center, Linköping, Sweden, Report No. 16.

Al-Tabbaa, A. (2002). Soil Mixing – Overview Current Practice and Needs. Cambridge University of Engineering Department, Cambridge UK. 21- 28.

Archie, G. (1942). Eletcrical-resistivity log as an aid in determining some reservoir characteristics. Trans. Am. Inst. of Min. Engr., 146, 318 – 319.

Arman, A. & Munfakh, G.A. (1970). Stabilization of Organic Soils with Lime. Division of Research Engineering, Louisiana State University, Baton Rouge, USA, Engineering Research Bulletin 103.

Basma, A. A., & Tuncer, E. R. (1991). Effect of lime on volume change and compressibility of expansive clays. Transportation Research Record No. 1295, pp 52–61.

Basma A. A., Al-Rawas A., Al-Saadi SN, Al-Zadjali T. F. (1998). Stabilization of expansive clay in Oman. Environ Eng Geoscience, Vol. 503–510.

Bell FG (1996). Lime stabilization of clay minerals and soils. Eng. Geol Vol. 223–237.

Bozbey, I. and Garaisayev, S. (2009). Effects of soil pulverization quality on lime stabilization of an expansive clay. Environ. Earth Sci. 60:Vol. 1137–1115.

Broms, B. B. (1984). Design Handbook, Third Edition. Lime Column AB pp, 1-51.

Broms, B.B., Boman, P., (1975). Lime stabilized columns. Proc. 5th Asian Regional Conference on Soil Mechanics and Foundation Engineering, Bangalore, vol. 1, pp. 227–234.

Broms, B., and Boman, P. (1979). Lime columns – a new foundation method. Journal of Geotechnical Engineering, ASCE, Vol. 105, GT4, pp.539-556, 1979.

Bruce, D. (2002). An introduction to deed mixind methods as used in geotechnical applications. Volume III. The verification and properties of treated ground. Report No. FHWA-RD-99-167, US Department of Transportation, Federal Highway Admistration, 2001.

Budhu, M., Giese, R.F., Campbell, G., & Baumgrass, L. (1991). The permeability of soils with organic soils. Can. Geotech. J. 28, 140–147.

Bujang, B. K. H., Sina, K., Arun, P., & Maassoumeh, B. (2011). Scientific Research and Essays Vol. 6 (1), pp. 196-204, 4 January, 2011.

Cerato, A. M. (2001). Influence of specific surface area on geotechnical characteristics of fine-grained soils. A Master degree thesis, Dept. of Civil. Envr. Engrg. Univ. of Massachusetts.

Chen, F.H., (1975). Foundation on Expensive Soil. Development in Geotechnical Engineering, 12. Elsevier Scientific Publishing Company, New York, USA.

Chen, F.H. (1988). Foundations on Expansive Soils. Developments in Geotechnical Engineering, Vol.12, Elsevier Publications, Netherlands.

Cokca, E. (2001). Use of Class F Ashes for the Stabilization of an expansive soils. Journal of Geotechnical Engr. Vol. 127 No. 7, pp. 568-573.

Craig, R.F. (1992). Soil Mechanics. Fifth Edition London. Chapman & Hall

De Jong, E., A.K. Ballantyne, D.R. Cameron, and D.W.L. Read. (1979). Measurement of apparent electrical conductivity of soils by an electromagnetic induction probe to aid in salinity surveys. Soil Sci. Soc. Am. J. Vol. 810-812.

Diamond, S., & Kinter, E. B. (1965). Mechanism of Soil Lime stabilization. An interpretative review. Presentation at the 44th Annual Meeting, Highway Research Board, Washington.

Edaes, J. L., & Grim, R. E. (1960). Reaction of hydrated lime with pure clay minerals in soil stabilization: Highway Research Board Bulletin 262 (N.A.S.-N. R.C. Publ. 771), pp.51-63.

Edaes, J. L., Nicholas, F. P., Jr., & Grim, R. E. (1962). Formation of new minerals with lime stabilization as proven by field experiments in Virgina. Highway Research Board, Bulletin 335, pp.31-39.

Graves, H.M. (1996). An introduction to Soil Stabilization. Proc. Seminar on Lime Stabilization, Loughborough University, Thomas Telford, pp5-12.

Glenn, G. R., and Handy, R. L. (1963). Lime-clay mineral reaction products: Paper presented at 42nd Annual Meeting, Highway Research Board, Washington, D.C., 1963.

Grim, R. E. (1968). The Clay Mineralogy 2nd edition New York: Mc Graw Hill, pp. 596.

Hartsock, N. J., T. G. Mueller, G. W. Thomas, R. I. Barnhisel, K. L. Wells and S. A. Shearer, . (2000). *Soil Electrical Conductivity Variability*. In. P.C. Robert et al. (ed.) Proc. 5th international conference on precision Agriculture. ASA Misc. Publ., ASA, CSSA, and SSSA, Madison, WI.

Hausmann, M. R. (1990). Engineering Principles of Ground Modification New York: Mc Graw Hill, 1990.

Holtz, R.D. & Kovacs, W.D. (1981). An Introduction to Geotechnical Engineering. New Jersey: Prentice Hall.

Ingles, O.G., Metcalf, J.B., (1972). Soil Stabilisation, Principles and Practice. Butterworths, London.

Inglef, O. G., & Metcalf, J. B. (1973). Soil Stabilization. Wiley and Sons, New York.

Jesse Jacobson. (2003). Factors Affecting Strength Gain in Lime-Cement Columns and Development of a Laboratory Testing Procedure. Master's Thesis,Virginia Polytechnic Institute and State University, Blacksbrug, Virginia.

Jones, D. E. J. and Holtz, W. G (1973). Expansive Soils – The hidden d isaster. Civil Engineering, Vol. 43, n.8, p. 49-51.

Kate, J.M. (1998). Behaviour of expansive soils treated with fly ash". Proc. International Symposium on problematic soils, Sendai (Japan), Vol. 1, 293- 296.

Katti, R. K. (1978**).** Search for solution to problems in black cotton soils. First IGS Annual Lecture at I.I.T, Delhi.

Keller, G. and Frischknetcht, F. (1966) Electrical methods in geophysical prospecting, New York, N.Y: Pergamon Press.

Kitsugi K. and Azakami H. (1982). Lime-column techniques for the improvement of clay ground. In: Proc Symp on Recent Developments in Ground Improvement Techniques, Bangkok, pp 105–115

Komine, H. and Ogata, N. (1996). Prediction of swelling characteristics of compacted bentonite, Can. Geotech. J. Vol. 33, 11-12.

Kwader, T. (1985). Estimating aquifer permability from formation-resistivity=factors. Ground Water, 23(6), 762 – 766.

Krohn, J. P. and Slosson, J. E. (1980). Assessment of Expansive Soils in the Conference on Expansive Soils, Denver, Colorado, pp. 596 – 608.

Lambe T. W. (1962). Soil Stabilization. Foundation Engineering. Leaonards, G. A., ed., McGraw-Hill, New York, pp.351-437.

Lambe, T.W. and Whitman, R.V. (1979). Soil Mechanics, John Wiley & Sons,New York, 4.45-4.51.

Larsson, S., Rothhamel, M., & Jacks, G. (2009). A laboratory study on strength loss in kaolin surrounding lime–cement columns. Applied Clay Science Volume: 44 Issue: 1-2: Vol. 116-126.

Larsson, S. (2005). State of Practice Report Session 6: Execution, monitoring and quality control. Proceedings of the International Conference on Deep Mixing, Best Practice and Recent Advances, Stockholm, vol. 2, pp. 732–785.

Little, D. N. (1991). X-Ray Diffraction, energy dispersive spectra and scanning electron microscopic evaluation of stabilized Denver clay soils containing sulphates. Report to Chemical Lime Company, Fort Worth, Texas.

Little, D. N. (1995). Handbook for Stabilization of Pavement Subgrades and Base Courses with Lime. National Lime Association, Kendall/Hunt Publishing Company, Dubuque, Iowa.

Locat, J., Berube, M. A. and Choquette, M. (1990). Laboratory investigations on the lime stabilization of sensitive clays: shear strength development. Can. Geotech. J. 27, Vol. 294 – 123.

Macham, L. R., Diamond, S., & Leo, E. (1977). Laboratory study of the effectiveness of cement and lime stabilization for erosion control. Transportation Research Record, No. 641, pp. 24-28.

Mcneil, J. (1990). Use of electromagnetic methods for groundwater studies. Geotech and Envir. Geophys., Vol. 1, S. Ward. Ed., 191 – 218.

Mitchell, J.K. (1976). Fundamentals of soil behaviour. John Wiley and Sons, Inc., New York, p.422.

Mitchell, J.K. (1993). Fundamentals of Soil Behavior, 2nd ed. John Wiley and sons, New York, N.Y.

Mitchell, J.K., Soga, K., (2005.) Fundamentals of Soil Behaviour, 3rd Ed. John Wiley, New York.

Mosavat, N., and Z. Nalbantoglu (2012). The impact of hazardous waste leachate on performance of clay liners, Waste Management & Research, 2012.

Nalbantoglu, Z. and Tuncer, E.R., (2001). Compressibility and Hydraulic Conductivity of a Chemically Treated Expansive Clay. Canadian Geotechnical Journal, Vol. 38, No. 1, pp.154-160.

Nalbantoglu, Z. & Gucbilmez, E. (2001). Utilization of an industrial waste in calcareous expansive clay stabilization. Geotechnical Testing Journal, GTJODJ, Vol. 25, No.1, 78-84.

Nalbantoglu Z. (2004). Effectiveness of Class C fly ash as an expansive soil stabilizer, Construction and Building Materials. Vol. 18, pp. 377-381.

Nalbantoglu, Z. and Tawfiq S., (2006.). Evaluation of the Effectiveness of Olive Cake Residue as an Expansive Soil Stabilizer. Environmental Geology, 50 (6), pp. 803-807.

Narasimha Rao, S., & Rajasekaran, G. (1994). Lime injection technique to improve the behaviour of soft marine clays. Ocean Engineering 21, 29–43.

Nasim M. (2010). Pore fluid effects on physicochemical and engineering properties of compacted clay. Master degree thesis, Eastern Mediterranean Univ. N. Cyprus.

National Lime Association, 1985. Lime slurry pressure injection. Bulletin 331, National Lime Association, Arlington, Virginia, USA.

Nelson, J. D., & Miller, J.D. (1992). Expansive soils: Problems and Practice in Foundation and Pavement Engineering.‖ John Wiley and Sons, Inc., New York.

Ola, S.A. (1983). The Geotechnical properties of the black cotton soils of North eastern Nigeria in: Tropical soils of Nigeria in Engineering practice, A.A. Balkema Rotterdam, Pp. 85-101.

Ormsby, W. C., & Kinter, E. B. (1973). Effects of dolomitic and calcitic limes on strength development in mixtures with two clay minerals. Public Roads, 37: 149-160.

Paige-Green, P. and Ventura, D. (1999). The bar linear shrinkage test – More useful than we think. 12th Regional Conference of the Geotechnics for Developing Africa. Ed. Blight and Fourie, Vol. 12, pp. 379-387.

Paul, K. M. and Rao, S. N. (1997). Journal of Geotechnical and Geoenvironmental Engineering - J Goetch. Geoenviron. Eng, vol. 123, no. 2, pp. 183-185.

Petry, T. M., & Little, D. N. (2002). Review of Stabilization of Clays and Expansive Soils in Pavements and Lightly Loaded Structures—History, Practice, and Future. Journal of Materials in Civil Engineering, ASCE, 14 (6), 447-460.

Petry, D. E., & Rich, C. I. (1971). Modification of certain soils by calcium hydroxide stabilisation. Soil Science Society of America Proceedings, 35: 834-834.

Porbaha, A. (1998). State of the art in deep mixing technology: part I. Basic concepts and overview. Ground Improvement 2, 81–92.

Prabakar K., Dendorkar N., & Morchhale R.K. (2003). Influence of fly ash on strength behavior of typical soil. Construction of Building Materials 18 (2004) 263-267.

Propesco, M. (1980). Behavior of expansive soils with crumb structure. Proceedings of the 4th International Conference of Expansive Soils. Vol 1, New York, pp. 158-171.

Punthutaecha, K. (2002). Volume change behavior of expansive soils modified with recycled materials. *Ph.D. thesis*, The University of Texas at Arlington, Arlington, Texas.

Puppala, A. J., Madhyannapu, R. S., Hoyos, L. R., & Yuan, D. (2007). Deep Mixing Technology for Mitigation of Shrink-Swell Behavior of Expansive Soils of Moderate to Deep Active Depths. Department of Civil and Environmental Engineering. The University of Texas at Arlington, Arlington, Texas 76019. Project Number 0-5179.

Quighley, R.M., Thompson, C.D. (1966). The fabric of anisotrophically consolidated sensitive marine clay. Can. Geotech. J. 3, 61–73.

Ranganatham, B.V., (1961). Soil structure and consolidation characteristics of black cotton clay. Geotechnique 11, 333–338.

Rajasekaran, G., & Rao, N. (1997). Lime Stabilization Technique for the Improvement of Marine Clay. Soils and Foundations, Japanese Geotechnical Society, 37 (2), 97-104.

Rajasekaran .G and Narasimha R. (1997). The microstructure of lime-stabilized marine clay. Ocean Eng 24(9): Vol. 867–878

Rajasekaran, G., & Narasimha Rao, S., (2000). Strength characteristics of lime-treated marine clay. Ground Improvement 4, 127–136.

Rajansekaran, G. R., & Rao, N. (2002). Permeability Characteristics of lime treated marine clay. Ocean Eng. 29:113-127.

Rao, S. M., and Venkataswamy, B. (2002). Lime Pile Treatment of Black Cotton Soils. Ground Improvement, 6 (2), 85-93

Rao, S. M., & Revanasiddappa, K. (2006). Influence of cyclic wetting and drying on collapse behaviour of compacted residual soil. Geotechnical and Geological Eng 24:725-734.

Rhoades, J., Raats, P., and Prather, R. (1976). Effect of liquid-phase electrical conductivity, water content, and surface conductivity on bulk soil electrical conductivity. Soil Sci. of Am. J., 40, 651 – 655.

Rhoades, J.D., N.A. Manteghi, P.J. Shouse, and W.J. Alves. (1989). Soil electrical conductivity and soil salinity: new formulations and calibrations. Soil Sci Soc. Am J. 53:433-439.

Rogers, C.D.F., & Glendinning, S. (1994). Deep-slope stabilization using lime piles. Transportation Research Record 1440, 63–70.

Rogers, C.D.F., & Glendinning, S. (1996a). Modification of clay soils using lime. In: Rogers, C.D.F., Glendinning, S., Dixon, N. (Eds.), Lime Stabilisation. Thomas Telford, London, pp. 99–114.

Rogers CDF, Bruce C. J. (1991) Slope Stability Engineering. ThomasTelford, London, 443 pp.

Rogers, C.D.F., & Glendinning, S. (1997). Slope stabilization using lime piles. In: Ground Improvement Geosystems: Proceedings Third International Conference on Ground Improvement Geosystems, Densification and Reinforcement. Thomas Telford, London, pp 174 – 180.

Sadek, M. (1993). A comparative study of the electrical and hydraulic conductivities of compacted clays. PhD thesis, Dept. of Civil Engrg., Univ. of California at Berkley, Berkley, Calif.

Singh, G.; and Singh, J. (1991). Highway Engineering. Standard Publishers Distributors, Nai Sarak, Delhi, India. pp. 608-10.

Soralump, S. and Kulsuwan, B. (2006) Landslide Risk Prioritization of Tsunami Affected Area in Thailand, International Symposium on Environmental Engineering and 5th Regional Symposium on Infrastructure Development in Civil Engineering,Philippines

Sridharan, A. Prashanth, J.P. & Sivapullaiah, P.V. (1997). “Effect of fly ash on the unconfined compressive strength of black cotton soil”. Ground Improvement Journal, vol.1, 169-175.

Tedesco D.V., Russo G. (2006). Effetti della stabilizzazione a calce sulla ritenzione idrica dei terreni. Incontro Annuale dei Ricercatori di Geotecnica-IARG 2006, Pisa.

Terashi, M., (2003). The state of practice in deep mixing methods. ASCE, Geotechnical Special Publication No. 120 vol. 1, 25–49.

Terzaghi K. (1925). Erdbaumechanik auf Bodenphysikalischer Grundlage. Wien, Deuticke.

Tonoz, M. C., Gokceoglu, C., & Ulusay, R. (2003). A Laboratory Scale Experimental Investigation on the Performance of Lime Columns in Expansive Ankara (Turkey) Clay. Bulletin of Engineering Geology and the Environment, Springer-Verlag, 62 (2), 91-106.

UFC Unified Facilities Criteria (2004). Foundations in expansive soils, US army Corps of Engineers, ufc-3-220-07.

U.S. Transportation Research Board. (1987). State of the Art Report 5: lime stabilization, reactions, properties, design and construction. Edited by L. Chou. U.S. Transportation Research Board. Washington D.C.

Ventkataswamy, B., Ghandi, S.R., K.S. Ghandi, & Sanghavi, K.V., (2003). Improvement of expansive clay by deep insitu technique. IGC 2003 ‘Geotechnical Engineering for Infrastructural Development’ Dec 18-20, 2003, Roorkee.

Urish, D. (1981). Electrical restistivity-hydraulic conductivity Relationships In glacial outwash aquifers. J. Geotech. Engrg., ASC, 121 (10). 736 – 739.

Wilkinson, A., Haque, A., Kodikara, J., Christie, D., & Adamson, J. (2004a). Stabilisation of Reactive Subgrades by Cementitious Slurry Injection – a review Australian Geomechanics Journal, Vol. 39(4), pp.81-93.

Williams, H. F. L. (2003). Urbanization Pressure Increases Potential for Soils-Related Hazards, Denton County, Texas. Environmental Geology, 44 (8), 933-938.

Zalihe N. (2006). Evaluation of the effectiveness of olive cake residue as an expansive soil stabilizer · Environmental Geology (2006) 50: Vol. 803-807.