ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

INFLUENCE OF MOLDING WATER CONTENT ON THE ENGINEERING PROPERTIES OF LIME STABILIZED EXPANSIVE SOIL

A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfilment of the Requirements for the Degree of Master of Science in Civil Engineering (Geotechnical Engineering)

By

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Abbreviation and Notes

OMC	Optimum Moisture Content
MDD	Maximum Dry Density
UCS	Unconfined Compressive Strength
USC	Unified Soil Classification system
AASHTO	American Association of State Highway and Transportation
NDS	Newly Developed Stabilizer
CBR	California Bearing Ratio
RHA	Rice Husk Ash
NLA	National Lime Association
HMA	Hot Mix Asphalt
ASTM	American Society for Testing and Materials
CH	Clays of High Plasticity
PL	Plastic limit
LL	Liquid Limit
P.I	Plastic Index

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Abstract

Expansive soil is considered defective due to the large volume change it undergoes for any moisture fluctuation. There are several engineering solutions to the problems these soils pose to the structures built on them. At times when removing the defective soil is uneconomical or impractical, stabilizing the soil with lime has proven to be effective. Past researches on lime stabilization of expansive soils identified many factors influencing the stabilization process. However, all the factors that contribute to best results are not yet obtained.

Common practice in lime stabilization is to compact the mixture of soil and lime at optimum moisture content (moisture content at maximum dry density). If the soil is expansive, at maximum dry density, large amount of expansive clay minerals (montmorillonite) will fill a given volume. On the other hand, using the dry side of the optimum moisture content may affect water dependent reactions that takes place during lime stabilization. If one uses the wet side of the optimum moisture content to mix the soil with lime, the clay mineral will absorb more water and go through large volume change. Therefore, the optimum amount of water required for lime stabilization of an expansive soil must be determined through laboratory experiments.

This research tends to identify the properties of an expansive soil. Lime is then added in proportions of 0%, 2%, 4%, and 6% to the soil sample. After a series of laboratory tests the optimum amount of lime that obtains desirable effect on engineering properties of the soil is determined to be 6%. Then, the stabilized soil property at the wet and dry side of the optimum moisture content has been investigated.

The results of the research have shown that molding water content does not affect the swelling pressure. However, unconfined compressive strength improves significantly on soil specimens

prepared on the wet side of optimum moisture content of soil-lime mixture. The improvement in compressive strength on the wet side of optimum moisture content is found to be dependent on the curing time.

Chapter One

Introduction

1.1. General/Background

Soil stabilization is a method of effectively and economically solving problems associated with defective soil properties. There has been much advancement on researches focused on identifying materials that can be used to improve soil properties either by mechanical or chemical stabilization. Industrial wastes, agricultural bi-products and commercially available chemicals are typical materials being commonly researched for successful soil stabilization. But one has to investigate the contributing factors to fully understand the mechanism of each of these stabilization methods [3].

Soil stabilization can be accomplished by several methods. All these methods fall into two broad categories [3] namely;

a) Mechanical Stabilization

Under this category, soil stabilization can be achieved through physical process by altering the physical nature of native soil particles by either induced vibration or compaction or by incorporating other physical properties such as barriers and nailing.

b) Chemical Stabilization

Under this category, soil stabilization depends mainly on chemical reactions between stabilizer (cementitious material) and soil minerals (pozzolanic materials) to achieve the desired effect. e.g**.** Lime stabilization, Cement stabilization, fly ash stabilization, Rice Husk Ash, etc...

Lime stabilization is one of the methods of improving the properties of soils especially for cohesive soils. In general, fine-grained clay soils (with a minimum of 25 percent passing the #200 sieve (74mm) and a Plasticity Index greater than 10) are considered to be good candidates for stabilization by lime. The mineralogical properties of the soils will determine their degree of reactivity with lime and the ultimate strength that the stabilized soil will develop [1].

According to National Lime Association's report in 2001, lime is added to a reactive soil to generate long-term strength gain through pozzolanic reaction. This reaction produces stable calcium silicate hydrates and calcium aluminate hydrates as the calcium from the lime reacts with the aluminates and silicates solubilized from the clay. The full-term pozzolanic reaction can continue for a very long period of time, even for decades as long as enough lime is present and the pH remains high (above 10) [1].

1.2. Problem Statement

When an expansive soil is stabilized with lime, the water added while mixing the soil with lime also known as ***molding water content***, plays a contradicting role. The usual trend in projects involving lime stabilization is to use the optimum moisture content (also known as moisture content at maximum dry density) as the molding water content. This will result in large amount of clay minerals to fill a certain volume, which may amplify the concentration of the swelling minerals (montmorillonite) at the given volume.

If the wet side of the optimum moisture content is used as molding water content, the clay minerals may absorb the water and go through large volume change, hence, providing high swelling pressure. Alternatively, one may consider using the dry side of the optimum moisture content as molding water content. However, in doing so, the chemical reactions that occur during lime stabilization may get affected since most of these reactions are dependent on the amount of water present during the stabilization process.

1.3. Research question:

What effect does the *molding water content* have on the engineering properties of lime stabilized expansive soil?

1.4. Objective:-

The aim of this research is to identify the geotechnical property of lime treated expansive soil on the wet and dry side of the optimum moisture content of the soil lime mix

Specific Objectives

- To determine the optimum amount of lime that stabilizes the expansive soil
- To identify the effect of molding water content on the swelling pressure after treatment by lime
- To identify the effect of molding water content on unconfined compressive strength of the soil after treatment by lime

1.5. Scope and Limitations of the study

- The mineral content of the soil is a factor one should consider during lime stabilization. Its effect in relation to molding water content was not studied in this research.
- The study focuses on the effect of molding water on swelling pressure and unconfined compressive strength only.
- To identify the amount of lime that stabilizes soil, lime is added to the soil at a rate of 2% by weight. Effect of lower rate of lime addition on the soil was not studied.

1.6. Organization of the thesis

Chapter one : Introduction
Background, objective and a brief summary of the thesis work is presented

Chapter Two: Literature Review
Researches on expansive soils, lime stabilization of various soil types, Mechanism of lime stabilization and other topics are reviewed

Chapter Three: Research materials and methods
Materials and Instruments used along with methodolgy of the thesis is discussed in this chapter

Chapter Four: Laboratory Test results and Discussions
Engineering properties of the expansive soil and improvements due to addition of lime and molding water content is discussed

Chapter Five: Conclusion and Recommendation
Conclusions drawn from the test results are discussed, recommendations on lime stabilization practices and future research areas are pointed out

Chapter Two

Literature Review

2.1. Introduction

This chapter presents previous researches on the use of lime alone or in combination with other materials for stabilization of different types of soils. It tends to critically review the previous works in light of their importance for this research.

2.2. Lime Stabilization

When lime is added to a clay soil, it reacts with the soil minerals to produce decreased plasticity, increased workability, reduced swell and increased strength. As a general guide, treated soils shall increase in particle size by cementation, reduction in plasticity, increased in internal friction among the agglomerates, greater shear strength, and increased workability due to the textural change from plastic clay to friable, sand like material [8].

Lime Stabilization can be classified as traditional stabilization and modified Stabilization based on time soil-lime mixture compaction. Traditional lime stabilization is defined as lime mixed into the soil and immediately compacted without allowing the lime/soil mixture to sit or mellow for an extended period of time before compaction. Modified lime stabilization is a process of soil stabilizing that advocate ***Mellowing*** (a process allowing lime-stabilized soil to remain in a soft, loamy state for a period of one to several days for chemical reaction to take place before compacting to final density) [2].

2.3. Mechanism of Lime stabilization

Lime is generally considered as a mixture of the oxides and hydroxides of calcium and magnesium, the materials commonly used for lime stabilization are calcium hydroxide ($Ca(OH)_2$) and dolomite ($Ca(OH)_2$ + MgO). Calcium hydroxide is a fine, dry powder formed by 'slaking' calcium oxide, CaO with water also known as quicklime. Quicklime is produced by heating natural limestone (calcium carbonate, $Ca(CO)_3$) in a kiln until carbon dioxide is driven out [10].

The actual mechanisms of lime-soil stabilization are not agreed upon. However, general agreement does exist that four basic reactions do occur [11]:

a) Cation exchange,

b) Flocculation and agglomeration,

c) Pozzolanic reaction and

d) Carbonation

Lime is used extensively to change the engineering properties of fine-grained soils and the fine-grained fractions of more granular soils. It is most effective in treating plastic clays capable of holding large amounts of water [17]. The particles of such clays have highly negative-charged surfaces that attract free cations (i.e., positively charged ions) and water dipoles. The addition of lime to a fine-grained soil in the presence of water initiates several reactions. The two primary reactions, cation exchange and flocculation agglomeration, take place rapidly and produce immediate improvements in soil plasticity, workability, uncured strength, and load-deformation properties [11].

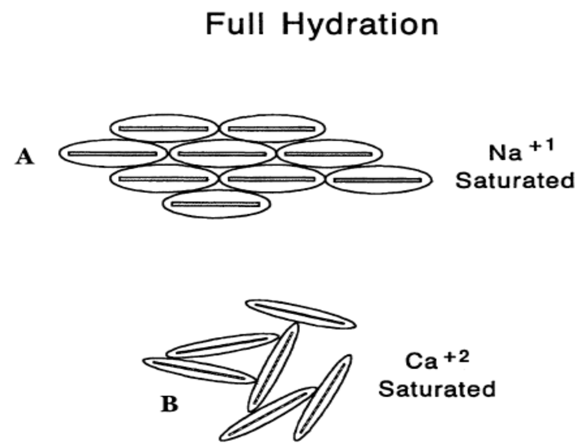

Fig 2. 1: Textural change in clay minerals due to lime addition [11]

The figure above illustrates occurrence of the textural change due to cation exchange, flocculation and agglomeration (A) illustrates low strength clay soil, where particles are separated by large water layers. (B) The addition of lime (Calcium) shrinks the water layer allowing the plate like particles to flocculate [11]

Cation exchange

The cation exchange occurs because divalent calcium cations can normally replace cations of single valence, and ions in a high concentration will replace those in a lower concentration. Lyotropic series which generally states that higher valence cations replace those of a lower valence, and larger cations replace smaller cations of the same valence. The Lyotropic series is written as [11]:

$$Li+ < Na+ < H+ < K+ < NH/ << Mg++ < Ca++ << AI+++$$

Flocculation and Agglomeration

These reactions result in a change in texture, the clay particles "clumping" together into larger sized "aggregates". They are effected by the increased electrolyte content of the pore water and also as a result of ion exchange by the clay to the calcium form. They are also primarily responsible for the changes in plasticity, shrinkage, and workability characteristics of lime-soil mixtures. ‘These beneficial changes were noted for all soils studied and relatively small percentages of lime were required to achieve the changes’ [18]

Carbonation

Lime reacts with carbon dioxide to form the relatively weak cementing agents calcium and magnesium carbonate, depending on the type of lime used. Although carbonation does produce weak cementing agents, it is an undesirable re-action and steps should be taken to minimize carbonation during construction operations and also following construction [27]

Pozzolanic reaction

It is a reaction between soil silica and/or alumina and lime resulting in several cementing agents. These cementing agents are the major source of the strength increment noted in lime-soil mixtures. Typical soils that are possible sources of silica and alumina include clay minerals, quartz, feldspars, micas, and other similar silicate or alumina silicate minerals. [18]

2.4. Lime stabilization of different soil types

A research in university of Kentucky that focused on lime stabilization has experimented on Unconfined Compressive Strength improvement due to lime addition. The soil was classified as A-6 and CL according to AASHTO and USC classification system respectively. At 2% hydrated lime product additive, the UCS of the soil increased from 207 kPa to 414 kPa. This strength gain was achieved through 7 days of curing time at room temperature. Higher percentages were less than 690 kPa, the minimum strength required by the Kentucky transportation cabinet for stabilized grade. [6]

The addition of lime to clay materials increases their optimum moisture content and reduces their maximum dry density for the same compactive effort [25].The reduction in dry density could be due to an immediate formation of cementitious products which reduce compactibility and hence the density of the treated soil [26]. However, the strength gain in the soil will normally more than compensate for the changes in compaction optima, and they should not be regarded as disadvantageous [25].

A lateritic soil in Nigeria was tested for the effect of lime variation on moisture content and dry density. The lime concentrations used were 2.5%, 5%, 7.5% respectively and a total of five specimens were used for each concentration to obtain moisture density relation. The maximum dry density ranges from 1.63 kg/dm^3 to 1.89 kg/dm^3 and the moisture content ranges from 2.2 % to 17.2 % for the samples under consideration. The dry density of the sample decreases with increase in lime concentration with the rate of reduction being more between 0 % and 2.5 % lime content while the moisture content increases with increase in lime content [7].

The effects of lime on geotechnical property of an expansive soil have been investigated by F.G Bell. The soil was mixed with lime contents of 3%, 6%, 9%, 12% and 15%. Free swell and free swell index of the stabilized samples decrease significantly with increasing lime content. Swelling pressure swelling potential and volume change of the treated samples also reduce markedly with the increase in lime content. With the increase in lime content, maximum dry density decreased while the optimum moisture content increased. Compared with the untreated sample, unconfined compressive strength of the treated samples increases significantly, depending on the lime content and curing age. It was found that strength development index increases with increasing curing age and lime content [26].

A comprehensive study has been made on the strength of lime treated soft clays. The three major factors that affect the strength of the soils (lime content, curing time, and curing temperature) are analysed and quantified via proposed empirical equations that are verified against experimental data. A general strength criterion, unifying the influence of all the three factors into a single equation has been proposed. The capacity of the general equation is also demonstrated. It is seen that the proposed strength equations can provide a useful means for predicting the strength of lime treated clays under various conditions. The strength of the limed treated soil increases significantly with curing time and curing temperature. For the influence of lime content, peak strength is reached at an optimum lime content which for many soils lies within the range of 3 to 9%. The variations in soil strengths with the three factors are analysed and quantified via proposed empirical equations [20].

A research has been conducted on improvements on the engineering property of frequently occurring minerals in clay soil by the addition of small percentage of lime (Bell, 1996). These minerals were namely Kaolinite, montmorillonite, and quartz. The series of tests that these minerals were subjected to provided the following results: “the plasticity of montmorillonite was reduced whilst that of kaolinite and quartz was increased somewhat. All materials experienced an increase in their optimum moisture content and a decrease in their maximum dry density, as well as enhanced California bearing ratio, on addition of lime. Some notable increases in strength and Young's Modulus occurred in these materials when they were treated” [26]

2.5. Stabilization of soils with lime and other stabilizers

A newly developed stabilizer (NDS) produced by incinerating various proportions of industrial wastes, combined with lime, has self cementing characteristics that resembles Ordinary Portland Cement. It has satisfactory results in stabilizing loam soils especially in strength development, due to the fixation of at least 34% of hydration water. This leads to an increase in dry density and decrease *w/c* ratio in the pore [5].

The effect of lime and fly ash on the geotechnical characteristics of expansive soil is was experimented on by a study [9]. Lime and fly ash were added to the expansive soil at 4% - 6% and 40% - 50% by dry weight of soil, respectively. Chemical composition, grain size distribution, consistency limits, compaction, CBR, free swell and swell capacity were determined from testing specimens. It is identified that expansive soil texture changes when lime and fly ash are mixed with expansive soil. "As the amount of lime and fly ash is increased, there are an apparent reduction in maximum dry density, free swell and swelling capacity under 50 kPa pressure, and a corresponding increase in the percentage of coarse particles, optimum moisture content and CBR value" [9].

The Compaction curves of the expansive soil and all types of mixtures (Soil, Lime and Fly ash) and their M.D.D and O.M.C values are shown in the figure below.

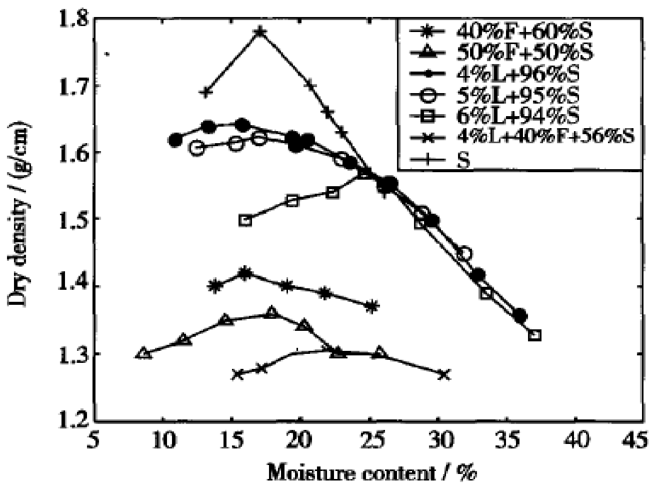

Fig 2. 2: Compaction curves of expansive soil, fly ash-soil and Lime-soil mix [9].

The pozzolanic cementitious strength gaining reaction between a given fly ash and hydrated lime can be characterized or measured with the plastic mortar cube test method. The graded standard ASTM sand is used as the aggregate for 73.3% of the soil mix. Lime constitutes 8.9% and fly ash constitutes 17.8% of the dry mixture, a 1:2 portion. The amount of water added is governed by the fixed flow method. Water is added to get a standard flow of water or fluidity (i.e slump cone test) to be within a certain range after 10 blows on the drop table. Some fly ashes will demand more water to get this fluidity depending upon their fineness and surface characterstics [19].

The effect of using a combination of lime and natural pozzolana in geotechnical characterstics of two types of cohesive soils (namely: red clay and grey soil) has been studied. Lime and Natural pozzolana were added at ranges of 0-8% and 0-20% respectively. Compaction and shear tests were conducted on test specimens cured for 1, 7, 28, and 90 days. Based on the experimental results, increasing the natural pozzolana from 0 to 20% decreased the O.M.C and increased the M.D.D. The addition of natural pozzolana alone had a negligible effect on shear stress. The research further states that M.D.D of lime stabilized soils decreased with increases in lime content, in contrast with natural pozzolana stabilized soils. When a combination of lime and natural pozzolana was used, the maximum dry density increased for the grey soil and decreased for the red soil, the optimum moisture content decreased for the grey soil and increased for the red soil [14].

Effect of rice husk ash, lime and gypsum on compaction, strength, CBR and free swell index properties of expansive soil has been studied. The rice husk ash appears to be an inert material with the silica in the crystalline form suggested by the structure of the particles, it is very unlikely that it would react with lime to form calcium silicates. It is also unlikely that it would be as reactive as fly ash, which is more finely divided. It was observed from the study that the liquid limit of the expansive soil has been decreased by 22% with the addition of 20% RHA+5% lime. It was also observed that the unconfined compressive strength of the expansive soil has been increased by 548% with addition of 20% RHA+5% lime + 3% gypsum after 28 days curing. [15]

2.6. Application of lime Stabilization

Lime plays a great role in rigid pavement construction when expansive sub-grade soils are encountered. When expansive soil underlies the pavement, water percolates the joint areas, causing the subsoil to swell. As a result, the slab warps and high joints are created. Lime stabilization solves this problem by creating a moisture barrier during stabilization. Lime stabilization has also been used to treat glacial clay soils. It can reduce frost heave by providing considerable strength to the soil layer itself, and by making the soil relatively impervious to water. Another advantage of using lime in concrete pavements is the improving of plastic, borderline gravels for use as granular sub-base, similar to cement- treated base. In this application 1 to 2 percent lime is added to the gravel and mixed in a pug mill prior to placement with an aggregate spreader. This type of treatment reduces both the PI and the percentage of fines [13].

Lime has an extensive history as a soil treatment option for airport construction. Slurry lime is becoming the most often specified lime-based treatment option due to the potential for dry lime dusting of airplanes and mechanical equipment. Airport construction often proceeds under time constraints. The use of lime to dry and modify marginal and poor soils can assist in keeping projects on schedule during wet weather by providing a working table that sheds water and allows return to work more quickly after rain events [1].

New construction of large stores or shopping centers with the accompanying parking areas is an increasingly common application for lime stabilization. Location of these facilities tends to be based on customer accessibility, not on soil characteristics. Unstable soils may be present. Sites may be in wet, low-lying areas. Rarely are sites level or on grade. The contractor must cut and fill the site and compact soils to prescribed soil densities. Material excavated for building pads can be limed in lifts as it is removed and stored in a stockpile for a few weeks. These treated soils should have water content 1 to 3 percent above optimum to ensure that the lime reaction has enough water for completion. This practice saves construction time as the mellowing is occurring in the material stockpile. The treated and mellowed material can then be compacted in lifts without delay as it is returned to the building pad [1].

Recent studies have shown that lime also has other effects in Hot Mix Asphalt (HMA). Specifically, lime acts as active filler, anti-oxidant, and as an additive that reacts with clay fines in the Hot Mix Asphalt. These benefits of lime in HMA are significantly large considering its effect as it reduces stripping, acts as a mineral filler, improves fracture toughness, favourably alters oxidation kinetics and interacts with products of oxidation to reduce their deleterious effects [8].

Lime can also be used as a stabilizer for base materials. One has to understand the purpose of lime is to interact with the fine material, normally finer than 75 μm, to form a matrix that will provide improved strength for the aggregate base. Adding lime to the fines matrix will decrease plasticity as well as increase strength, and it can generally be surmised that if an acceptable target strength is achieved, that the plasticity of the fines will be appropriately altered as well. However, it is prudent to test the plasticity of the minus no. 40 sieve fraction with the target lime content to verify the impact of lime on the plasticity of the fines [17].

2.7. Factors affecting lime stabilization

Many factors influence the lime soil pozzolanic reaction. Important factors include natural soil properties, lime type, lime percentage, curing conditions, and density [18].

Natural Soil Properties

In an extensive study of typical Illinois soils, it was found that the ability of a soil to participate in the lime-soil pozzolanic reaction was determined primarily by natural soil properties [18]. The degree to which the lime-soil pozzolanic reaction is measured and proceeded in terms of lime-reactivity which was defined as the difference in the unconfined compressive strengths of the natural soil and the maximum strength developed by a 3, 5, or 7 per cent lime-soil mixture after a 28-day curing period at 73◦F. Some soils did not display significant reactivity and others reacted to produce strength in-creases ranging up to several hundred percent. Pertinent findings of Thompson's work are summarized below:

1) Soil organic matter retarded the pozzolanic reaction if it was present in large quantities. None of the A horizon soils reacted with lime and some of the B horizons, particularly Brunizems with organic carbon contents greater than approximately 1 percent, also did not

react. The retardation of the reaction was attributed to a "masking effect" of the organic matter on the clay surfaces and/or an organic matter chelation reaction.

2) Although < 2p clay contents ranged from 7 to 65 per cent, it did not significant-ly influence lime-reactivity. However, some minimum quantity of clay is required toprovide adequate silica and/or alumina sources for the pozzolanic reaction
3) Clay mineralogy also effected lime-reactivity; mixed layer and montmorillonitic clays were most reactive
4) Soil chemical properties greatly influenced lime-reactivity. Highly significant correlations were obtained between natural soil pH and lime-reactivity. Soils with pH below approximately 7 had lime-reactivities less than 100 psi. Cation ex-change capacity, exchangeable bases, percent base saturation, and Ca/Mg ratios were not significantly correlated with lime-reactivity. The better reactivities of the higher pH soils were attributed to reduced weathering status of the soil minerals
5) Natural soil drainage was a good indicator of lime-reactivity. Soils with poor natural drainage displayed higher levels of lime-reactivity than better drained soils. All of the Humic-Gley soils, which are poorly drained, included in the investigation reacted very well. The increased reactivity of the poorly drained soils was attributed to minimal weathering of the soil minerals, thus the soil was a ready source of reactive silica and/or alumina. It was established that in-creased weathering and ferric oxide coatings on the soil mineral surfaces were responsible for the low reactivity of the better drained soils.
6) Based on data obtained from a study, soil pulverization quality considerably affected the unconfined compressive strength and secant elasticity modulus values [12]. As the amount of soil passing No. 4 sieve increase, the effectiveness of lime treatment also increases. The

results of the study clearly demonstrated that if enough time and effort were not given to soil pulverization process in lime stabilization works in field applications, lower performance and therefore increased environmental problems should be expected.

Lime Type

Many investigators, Laguros, J.G, et.al, [21], Lu.L.W, et.al [22] and Wang J.W, et.al, [23] have indicated that lime type significantly influences the lime-soil pozzolanic reaction. Monohydrated dolomitic limes generally produced greater strengths than hydrated calcitic limes. It was concluded from their study of nine soils that dolomitic limes produced higher strengths for montmorillonitic and illitic soils, but kaolinitic soils neither dolomitic nor calcitic limes consistently produced higher strengths

Lime Percentage

In most cases, for given curing conditions, a soil will achieve a maximum strength at some optimum lime content or will reach a lime content beyond which further increases of treatment level will not produce a significant strength increase. Herrin and Mitchell, [24] found that optimum lime contents are generally higher for dolomitic than for calcitic limes. The soil characteristics that significantly influence optimum lime contents have not been established, but they probably encompass such factors as chemical, physical, and mineralogical properties.

Curing Conditions

Herrin and Mitchell, [24] indicated in their literature survey that increased curing time and elevated temperatures produced substantial strength increases in lime-soil mixtures. Reports of other investigators published subsequent to their work further, Thomson [18], substantiates this fact.

Thompson has presented data, showing the influence of time and temperature on the strength of a typical Illinois soil

Density

Density of the compacted material also influences the cured strength of a lime-soil mixture. As stated by Herrin and Mitchell, [24] "The strength of a lime-soil mixture is increased materially when the mixture is compacted to a higher unit weight by a greater compactive effort."

Chapter Three

Research Materials and Methods

3.1. Description of the site

The soil samples used for this research were obtained from eastern Addis Ababa, around Gelan condominium on the Addis Ababa outer ring road project. The geographical location of Gelan is located at 8°52’ North latitude and 38°49’ longitudes with an average elevation of 2240m. It is located 9 km southeast from the Addis Ababa Bole international airport.

The Addis Ababa outer ring road project is 32 km long. The visual classification of the natural soil covered the project route is dark grey in color, cracks and fissures are observed on the surface.

3.2. Materials Used

The materials used for this study are: expansive soil samples, lime and water.

The expansive soil samples were taken at a depth of 2 m from the ground surface. This was facilitated by the construction work going on at the point of collection, which exposed the soil strata.

The hydrated lime used as the stabilizing agent for the research was provided from the reputable Derba Cement factory branch office in Nani Building 9th floor. The Lime was stored in a cool and dry place away from weather effects.

3.3. Methodology

The general approach of the research is as the following:

1) The engineering property of the expansive soil is identified by conducting laboratory tests.

2) The soil is mixed with various amount of lime to obtain the optimum amount that stabilizes the soil.
3) Specimens are prepared by mixing the soil with the optimum amount of lime and different moisture contents.
4) Using the specimens mentioned above tests were conducted to study effect of moisture variation on lime stabilization

3.3.1. Preparation of soil sample

The soil was air dried and pulverized to pass through a No. 4 sieve. The soil passing No. 4 sieve was used for conducting Standard Proctor compaction, Oedometer swelling pressure, and UCS tests. Index properties were determined on the soil fraction passing through a 425-μm sieve. Compacted soil specimens were prepared using the O.M.C and M.D.D obtained from the standard proctor compaction test.

3.3.2. Preparation of soil-lime mix sample

The soil-lime compacted specimens were prepared using the O.M.C and M.D.D for different amounts of lime. First, the soil, lime and water (added to produce O.M.C) were mixed and put in to a water tight plastic bag. The mix in the plastic bag was put in to a water tank for a period of 7 days to be cured. Then at the 7th day, the mix was taken out of the water tank and out of the plastic bag. It was compacted using the standard compaction mold.

3.4. Laboratory testing methods

A series of tests were performed on the untreated soil, on specimens with lime added in the varying percentages, and finally on lime stabilized specimens with varied moisture contents

Unconfined Compressive Strength test

The standard used for this test is ASTM D 2166 (Standard Test Method for Unconfined Compressive Strength of Cohesive Soil). The specimen was placed centrally on the lower platen of the compression testing machine. The force was applied with a controlled strain rate of approximately 1 mm/minute. The force was recorded during the test until the specimen yielded.

The unconfined compressive strength is computed from the following formula:

$$q_u = \frac{P}{A\left[1-\frac{\Delta H}{H}\right]} \quad \text{....................................[3.1]}$$

Where:

- qu = The unconfined compressive strength in kPa
- P = Maximum load in kN.
- A = Initial cross-sectional area of test specimen in square millimeters.
- H = Initial height of test specimen.
- Δ H = Reduction in height of test specimen.

1 - (ΔH/H) = Correction for increased area assuming constant volume.

Swelling Pressure Test

swell- consolidation method, the re-compacted specimen is placed in the consoildometer and a normal setting pressure equal to 7 kN/m^2 is applied. Then sample is inundated and allowed to swell. For soil compacted at Maximum Dry Density, it will take longer time than that of undisturbed samples. The usual practice is to allow it to swell for 24 hours but in case of re-compacted specimens 6 days waiting period provided enough time for water to percolate through the pores. After the soil specimen complete swelling, weights are hung on the oedometer in the order of 1kg, 2kg, 4kg, 8kg, 16kg, respectively after every 24 hours. The pressure corresponding to the weight that return the swelled specimen to its original height is identified as the swelling pressure. Accordingly, one can determine the swelling pressure from the e log p curve of the swell-consolidation test result. The standard referred for this test is ASTM D 2435 (standard test method for one dimensional consolidation properties of soils)

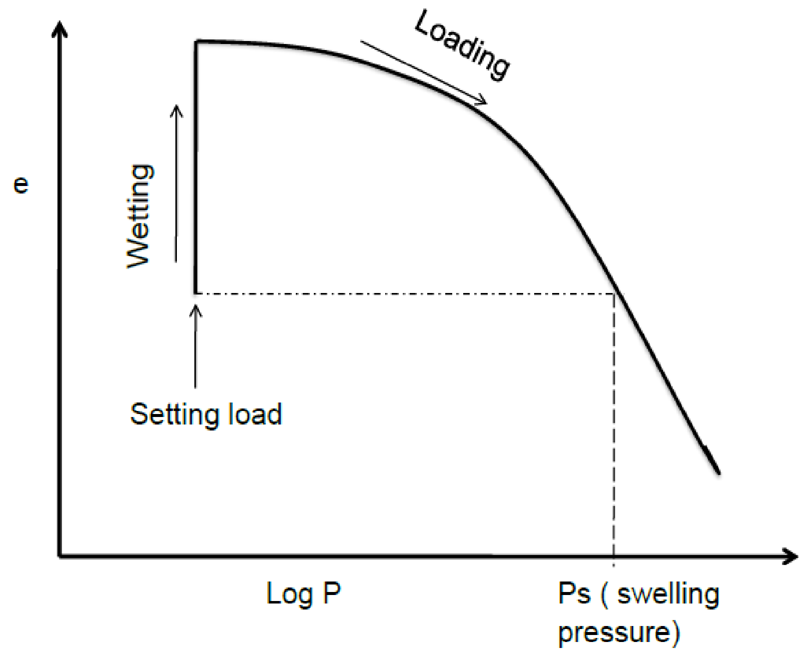

Fig 3. 1: Typical e - log P curve

Compaction test

This laboratory test is performed to determine the relationship between the moisture content and the dry density of the soil for a specified compactive effort. The compactive effort is the amount of mechanical energy that is applied to the soil mass. Three layers of the soils then were compacted in a standard four-inch mold using an automatic standard Proctor hammer. The weight of the soil sample as well as the weight of the compaction mold with its base is determined. Compute the amount of initial water to add from the following equation:-

$$\text{Water to add (in ml)} = \frac{(\text{Soil mass in grams}) \times \text{initial moisture content (\%)}}{100} \quad \ldots\ldots\ldots\ldots\ldots\ldots [3.2]$$

The moisture content of compacted soil specimen is calculated that is used to determine the dry density in the following formula:

$$\boldsymbol{\rho_d = \frac{\rho}{1+w}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots[3.3]$$

Where: w = moisture content in percent divided by 100, and ρ = wet density in grams per cm^3.

Index Properties

A fine-grained soil can exist in any of several states; which state depends on the amount of water in the soil system. When water is added to a dry soil, each particle is covered with a film of adsorbed water. If the addition of water is continued, the thickness of the water film on a particle increases. Increasing the thickness of the water films permits the particles to slide past one another more easily. The behavior of the soil, therefore, is related to the amount of water in the system. Approximately sixty years ago, A. Atterberg defined the boundaries of four states in terms of "limits" is dicussed below.

The laboratory test procedure referred to determine the Atterberg limits is ASTM D 4318 - Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

- **Liquid limit**

The water content at which the soil has such a small shear strength that it flows to close a groove of standard width when jarred in a casagrande apparatus. The Liquid Limit, also known as the upper plastic limit, is the water content at which soil changes from the liquid state to a plastic state.

- **Plastic Limit**

The water content at which the soil begins to crumble when rolled into threads of specified size. The Plastic Limit, also known as the lower plastic limit, is the water content at which a soil changes from the plastic state to a semisolid state. Plastic Limit (PL or wP) - the water content, in percent, of a soil at the boundary between the plastic and semi-solid states.

- **Plasticity Index (PI)**

The range of water content over which a soil behaves plastically is defined as "the range of consistency with in which the soil exhibit plastic properties". It is also defined as "the numerical difference between the liquid limit and plastic limit".

Mathematically, **Plasticity index = Liquid Limit – Plastic Limit**

It is denoted by I_p and, $I_p = LL - PL$

Chapter Four

Laboratory Test Results and Discussion

4.1. Introduction

This chapter will discuss the laboratory test procedures and results obtained that are instrumental for accomplishing the objectives of this research.

4.2. The Untreated soil properties

The engineering property of the soils is determined to identify soil property without any interference of lime stabilizing action.

4.2.1. Grain size Analysis

This test is performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method is used to determine the distribution of the finer particles.

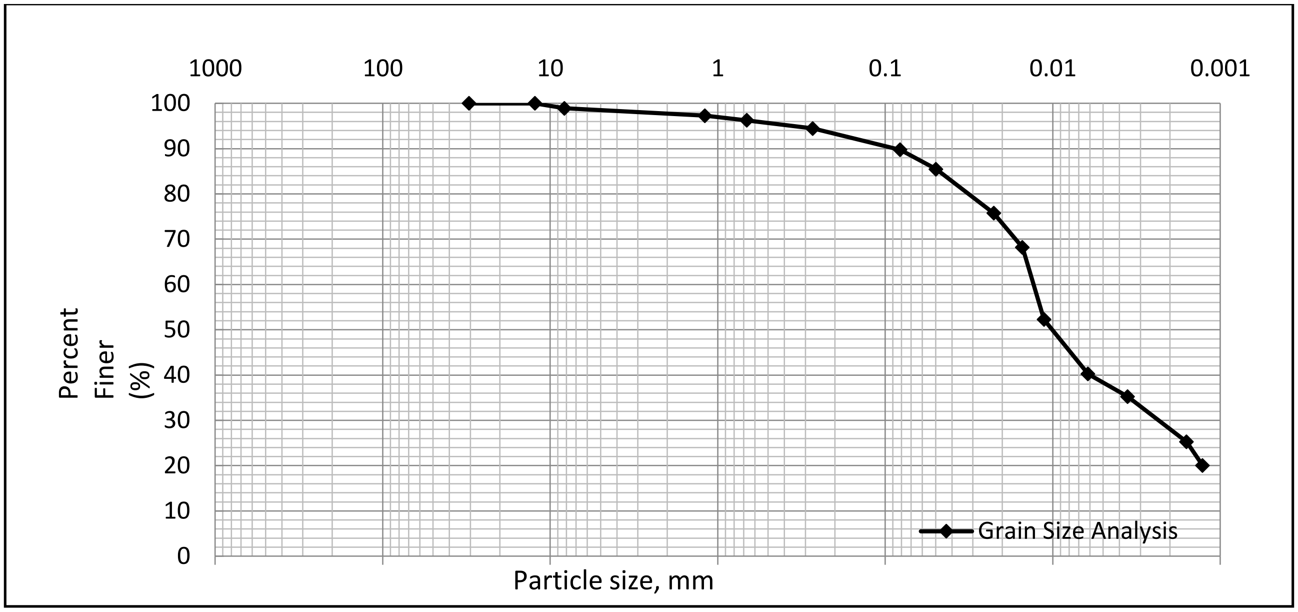

Fig 4. 1: Grain size distribution of the soil

The figure above shows the combined result of mechanical sieve analysis and hydrometer analysis. Based on the amount of grain sizes, there are different classification systems shown in the table below.

Table 4. 1: Classification of soil based on the grain sizes

Classification system	**Grain Size (mm)**			
Unified	**Gravel**	**Sand**	**Silt**	**Clay**
	4.75 – 75	0.075-4.75	clay and silt are identified by their plasticity	
AASHTO	2.00-75	0.05-2.00	0.002-0.05	<0.002
MIT	2.00-100	0.06-2.00	0.002-0.06	<0.002
ASTM	2.00-100	0.075-2.00	0.005-0.075	<0.005
USDA	2.00-75	0.05-2.00	0.002-0.05	<0.002

Percentage of Gravel (Particle Size 75 mm→4.75 mm) = 100% -97.82% = 2.18%

Percentage of Sand (Particle size 4.75 mm→0.075 mm) = 97.82% - 91.53% = 6.29%

Percentage of fine particles = 100% - (2.18+6.29)% = 91.53%

4.2.2. Compaction test

It is a method of experimentally determining the optimal moisture content at which a given soil type will become most dense and achieve its maximum dry density for a specified compactive effort.

This research will employ the tamping or impact compaction method using the type of equipment and methodology known as the Standard Proctor test. In this method, the soil is compacted by a 5.5 lb hammer falling a distance of one foot into a soil filled mold. The mold is filled with three equal layers of soil, and each layer is subjected to 25 drops of the hammer, generally consistent with the American Society for Testing and Materials (ASTM) standards

This test is especially useful to prepare the remolded test sample for strength tests and swelling pressure determination using oedometer test. The optimum moisture content and maximum dry density of the soil obtained from this test will be compared to that of the soil lime mix.

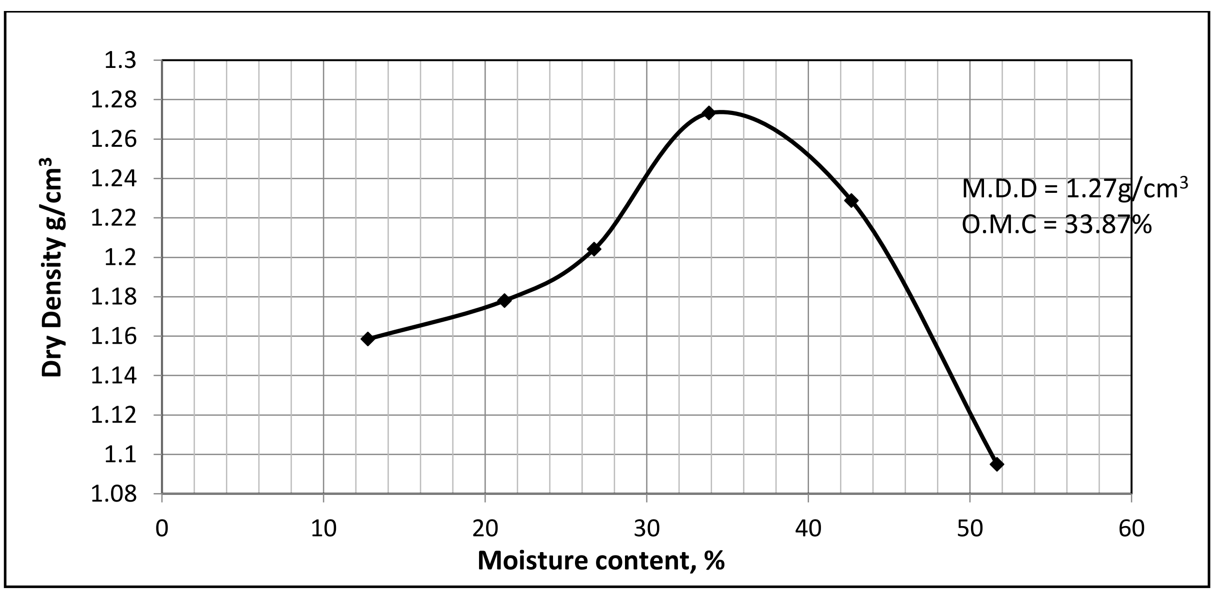

Fig 4. 2: Compaction test of the untreated soil

4.2.3. Unconfined Compressive Strength

The primary purpose of this test is to determine the unconfined compressive strength, which is then used to calculate the unconsolidated undrained shear strength of the clay under unconfined conditions. According to the ASTM standard, the unconfined compressive strength (q_u) is defined as the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test.

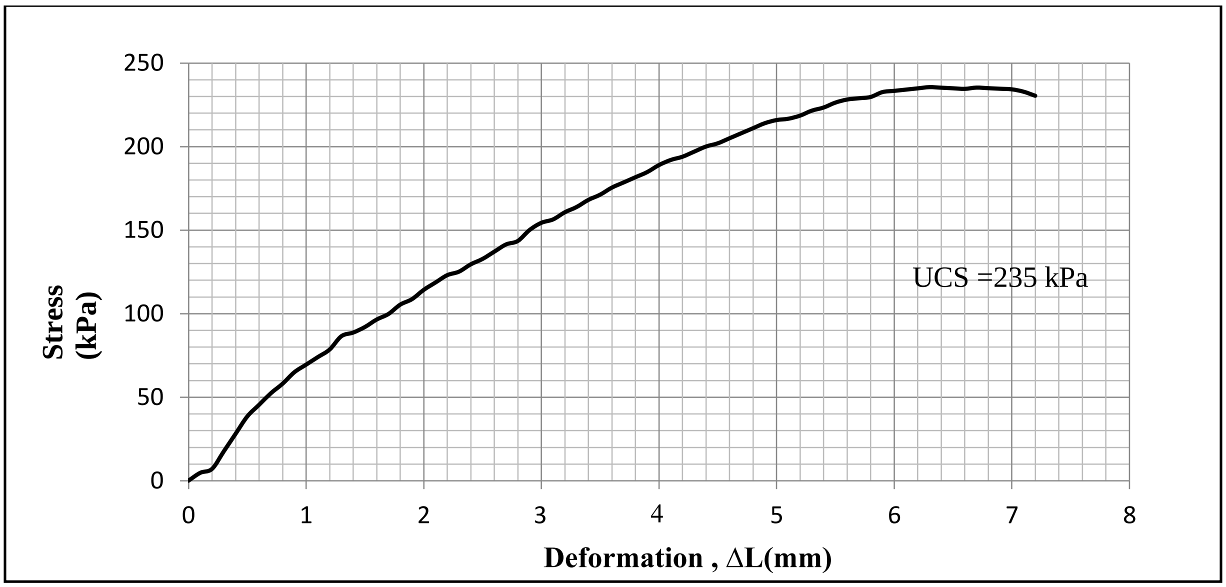

Fig 4. 3: Unconfined Compressive Strength of the untreated soil

The sample for this test is prepared using the O.M.C and M.D.D obtained from compaction test and remolded using the standard proctor compaction mold. Therefore the stress Vs. strain plot of the UCS test is shown above. The moisture content of the specimen is shown in the table below.

Table 4. 2: Moisture content of the untreated soil specimen for UCS

Sample No.	B2
MC = Mass of empty, clean can + lid (grams)	33.2
MCMS = Mass of can, lid, and moist soil (grams)	120.5
MCDS = Mass of can, lid, and dry soil (grams)	62.1
MS = Mass of soil solids (grams)	28.8
W = Water content, w%	33%

The UC S *(q_u)* of the soil, determined from the graph above is 235 kPa, Cohesion of the soil (S_u) is one half of the UCS of the soil (q_u/2) = 235/2 = 117.5 kPa.

4.2.4. Specific Gravity Test

It is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. The specific gravity test result in the table below indicates heaviness of the soil which indicates organic material is not present in the soil. Usually high specific gravity indicates the soil might be expansive clay, however cannot be solely used to conclude the soil is expansive

Table 4. 3: Specific gravity of the untreated soil

Pycnometer No.	A3
Weight of Pyc. + Soil + Water, W_{pws} (g)	158.6
Temperature (◦c)	22.8
Weight of Pyc. + Water, W_{pws} (g)	143.1
Weight of dry soil , W_s (g)	25
Conversion factor, K	0.99935
Specific gravity at 20 ◦c	2.63

4.2.5. Free swell test

The free swell test is one of the most commonly used simple tests for estimating soil swelling potential. This test is performed by pouring 10cc of dry soil, passing through sieve no 40 (0.425 mm diameter), into a 100 cc graduated cylinder. The cylinder is then filled with distilled water and the swelled volume of the soil is measured after the material settles. Results of the free swell tests of the soil is given in Table 3.3

Table 4. 4: Free Swell Test

	Initial	**After 24 hrs**	**Free swell**
Sample 1	10 ml	22ml	120%
Sample 2	10ml	20ml	100%

4.2.6. Atterberg Limits

Index properties are determined according to ASTM D 4318 - Standard Test Method for liquid limit, plastic limit, and plasticity index of Soils. Results of the test are presented in Table 3.4 below.

Table 4. 5: Liquid limit and plastic limit of the soil

		Liquid Limit			**Plastic Limit**
Trial No	1	2	3	4	1
Container No	95	155	150	98	183
Mass of container, g	33.70	33.60	33.60	33.40	32.80
Mass of container + Wet soil, g	50.80	49.60	51.60	49.00	40.00
Mass of container + Dry soil, g	42.30	41.50	42.40	41.00	38.10
Mass of water, g	8.50	8.10	9.20	8.00	1.90
Mass of dry soil, g	8.60	7.90	8.80	7.60	5.30
Water content, %	98.84	102.53	104.55	105.26	0.36
No of blows	35	27	23	17	------

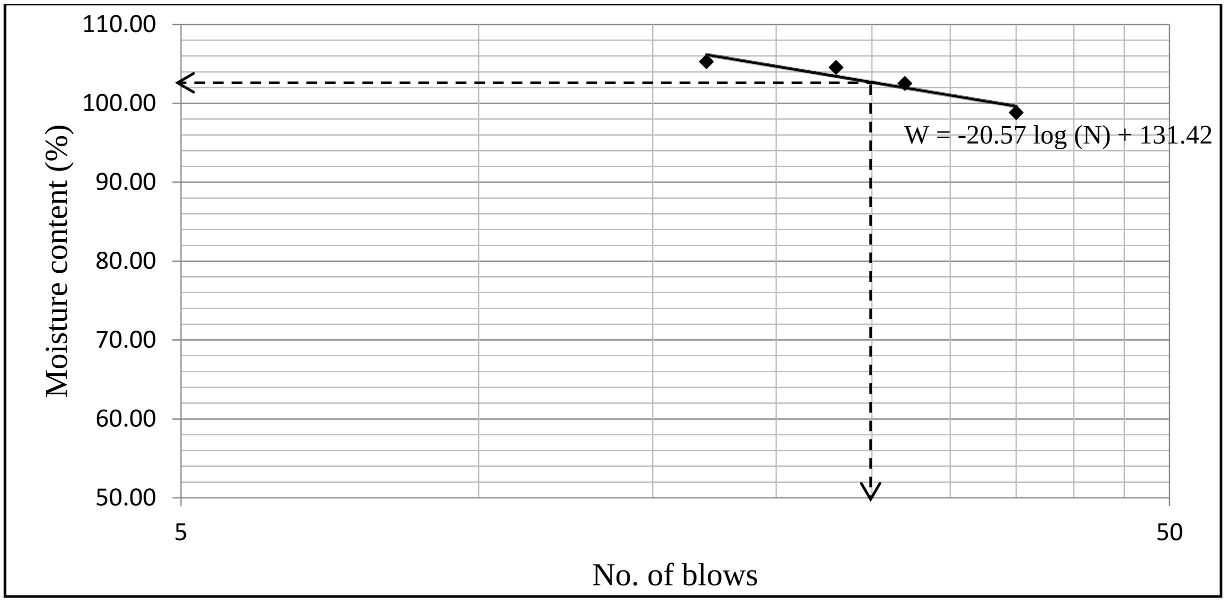

Fig 4. 4: Liquid Limit of the untreated soil

Trial	**No. of blows (N)**	**log (N)**	**W (%)**	$\log \frac{N}{N_{mean}}$	**W- W_{mean}**	$\left[\log \frac{N}{N_{mean}}\right]^2$	$\left(\log \frac{N}{N_{mean}}\right) \times (W - W_{mean})$
1	17	1.23	105.26	-0.16	2.46	0.0261	-0.398
2	23	1.36	104.55	-0.03	1.75	0.0009	-0.053
3	27	1.43	102.53	0.04	-0.27	0.0016	-0.010
4	35	1.54	98.84	0.15	-3.96	0.0232	-0.602
Mean	25.5	1.39	102.80		**Sum**	0.0517	-1.063

W = F_1 Log (N) + b, F_1 = Flow index

$$F_1 = \frac{\sum\left[\left(\log \frac{N_j}{N_{mean}}\right) \times \left(W_j - W_{mean}\right)\right]}{\sum\left[\log \frac{N_j}{N_{mean}}\right]^2}$$

$F_1 = \frac{-1.063}{0.0517} = -20.57$

Insert (25.5, 102.8) in place of W and N in equation W= F_1 Log (N) + b

b = 131.42

The flow curve equation is:

W = -20.57 Log (N) + 131.42

Therefore, the liquid limit is moisture content at 25 blows

L.L = -20.57 Log (25) + 131.43= **102.66%**

Hence, the plastic index (P.I) = L.L – P.L = 102.66 – 36 = **66.66%**

4.2.7. Swelling Pressure

The swelling pressure in the consolidometer test is defined as the pressure which is required to return a swelled specimen back to its original state. The recompacted soil (at O.M.C and M.D.D) specimen is placed in consolidometer and a normal load equal to 1 Psi (7 kNm^2) is applied. Then the sample is inundated and allowed to swell. After the swelling has completed, the sample is loaded. If the sample does not return to its original state, the load is increased until it the original volume is achieved.

Table 4. 6: Oedometer swelling pressure test result of the re-compacted soil specimen

Applied pressure P (kPa)	Final Dial Reading	Change in specimen height (mm)	Final Specimen Height (mm)	Void Height, Hv (mm)	Void Ratio, e
7	13.940	0.02	17.00	7.13	0.71
7	16.581	-2.62	19.62	9.75	0.97
50	16.193	-2.23	19.23	9.36	0.93
100	15.802	-1.84	18.84	8.97	0.89
200	15.192	-1.23	18.23	8.36	0.83
400	14.179	-0.22	17.22	7.35	0.73
800	13.090	0.87	16.13	6.26	0.62

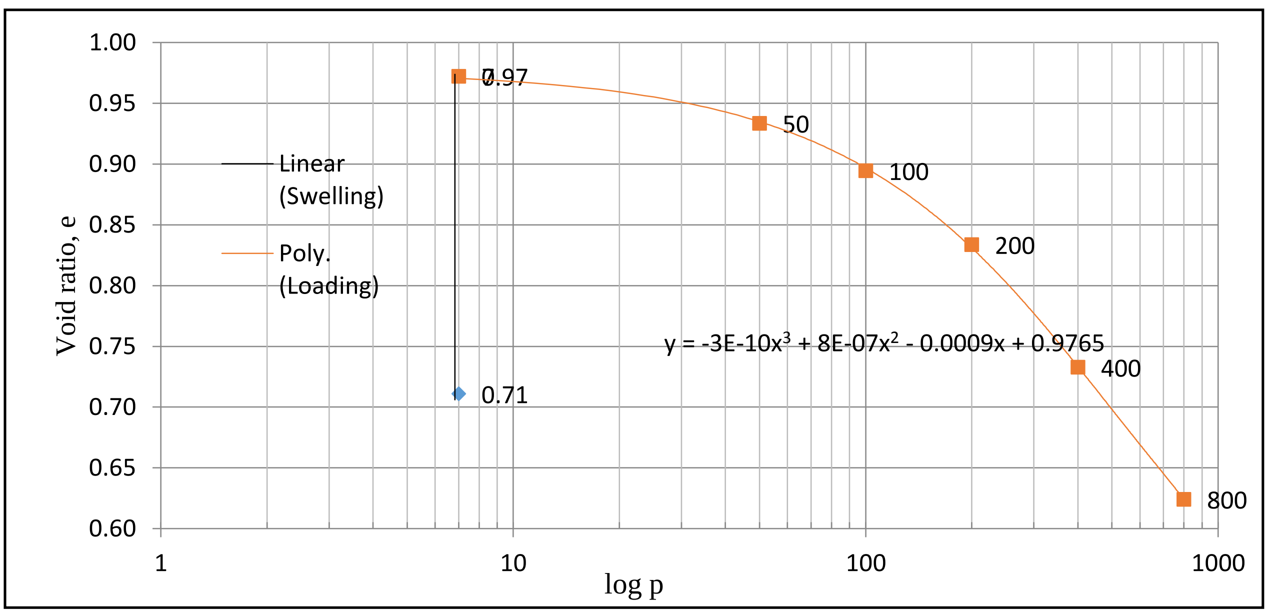

Fig 4. 5: e log p curve of the soil

The e log p curve shown in Fig 4.5 above is used to determine the swelling pressure of the soil.

The swelling pressure is determined by inserting 0.71 in place of Y in the loading curve equation and solving for x (swelling pressure):

$y = -(3*10^{-10})\ x^3 + (8*10^{-7})\ x^2 - 0.0009x + 0.9765$

$0.71 = -(3*10^{-10})\ x^3 + (8*10^{-7})\ x^2 - 0.0009x + 0.9765$

$0 = -(3*10^{-10})\ x^3 + (8*10^{-7})\ x^2 - 0.0009\ x + 0.2665$

By trial and error X is determined to be 437

Therefore, the swelling pressure of the soil is 437 kPa

4.2.8. California Bearing Ratio

It is a measure of resistance of material to penetration of a plunger under controlled density and moisture conditions. An air dried soil sample is thoroughly mixed with water (OMC) as obtained from the compaction test result. Then it is compacted in the mold and immersed in a tank of water allowing free access of water to the top and bottom of specimen for 96 hours.

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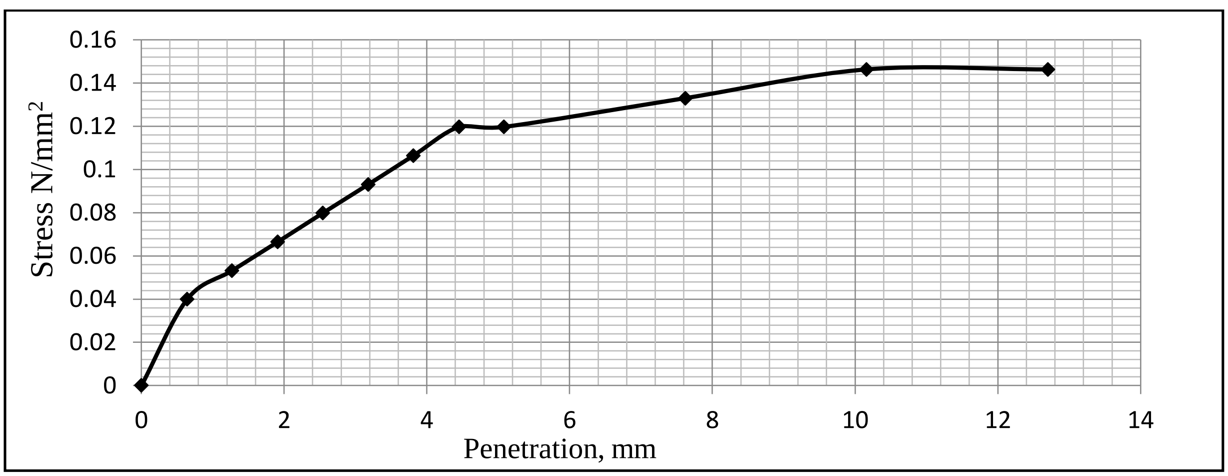

Fig 4. 6: CBR test result, stress - penetration curve

After the soaking is completed the mold is taken out of the water and put on the lower plate of the testing machine with top face exposed. To avoid any upheaval of the soil, a 2.5 kg weighing surcharge. Then the specimen is seat on the penetration plunger. At a rate of 1.275 (mm/min), the load is applied. The load penetration curve result is shown in the figure above. The curve is uniformly convex upwards. Therefore, no correction is required. The load readings at different penetration depths are shown in the table below. The CBR value of the soil is calculated as a percentage of the actual load causing the penetrations of 2.5mm or 5.mm to the standard loads stated in the table below. The greatest value of both penetration depths is recorded as the CBR value. It is shown in the table that at 2.5mm depth the CBR value (1.16) is equal to that of the 5mm depth (1.16). Therefore the CBR value of the untreated soil is 1.16%

Table 4. 7: CBR testing result for untreated soil

Penetration (mm)	**Ring (Div)**	**Load (N)**	**Stress (N/mm^2)**	**Standard stress (N/mm^2)**	**CBR (%)**
0	0.0	0	0		
0.64	3.0	77	0.04		
1.27	4.0	103	0.05		
1.91	5.0	129	0.07		
2.54	6.0	154	0.08	6.9	1
3.18	7.0	180	0.09		
3.81	8.0	206	0.11		
4.45	9.0	232	0.12		
5.08	9.0	232	0.12	10.3	1
7.62	10.0	257	0.13		
10.16	11.0	283	0.15		
12.70	11.0	283	0.15		

4.2.9. Classification of the soil

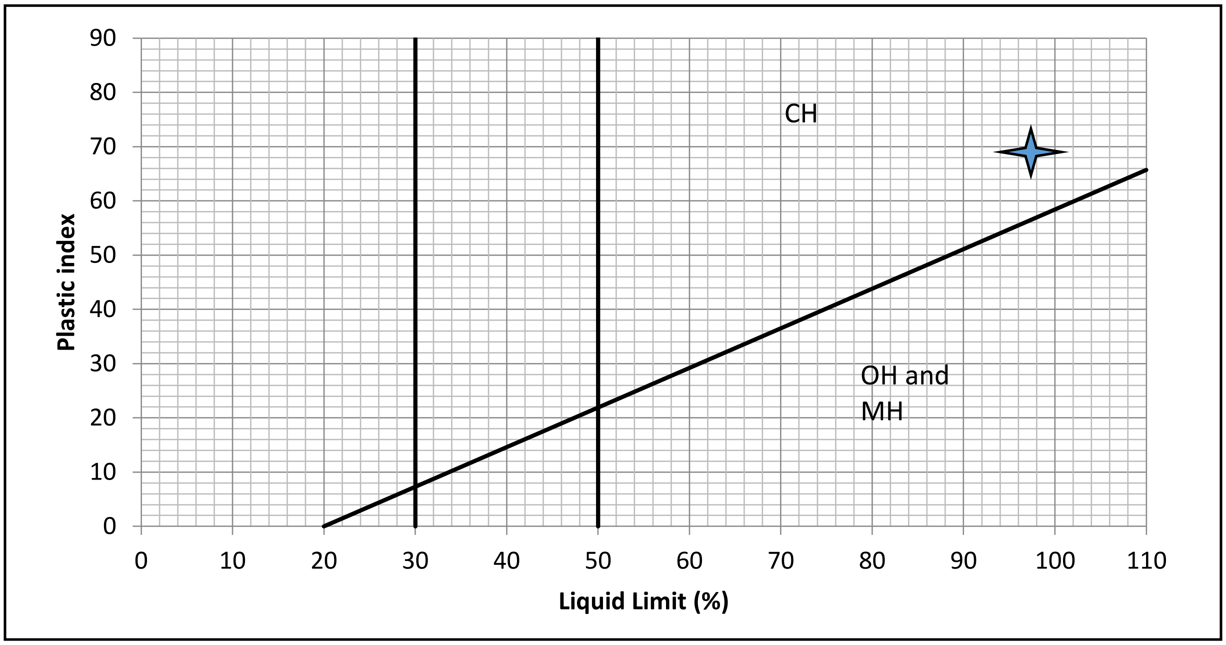

Fig 4. 7: Plasticity Chart

Activity, Ac= $\frac{\text{Plasticity Index}}{\text{\% by weight finer than } 2\mu\text{m}} = \frac{66.8}{30.5}$ = 2.3

Activity	Skempton's category
Ac < 0.75	Inactive
0.75< Ac<1.25	Normal
Ac> 1.25	**Active**

As shown in Fig 4.6, the star indicates where the relation between soil's plastic index and liquid limit lies on the graph. It lies above the A-line and has USC system's label of CH (**Inorganic clay of high plasticity**). The activity of the soil is 2.3 which is ***active*** according to skempton's category.

The high swelling pressure and free swell results shown in the previous sections, 437 kPa and 110% respectively, suggest that soil is ***Expansive soil***.

4.3. Lime-soil mix Properties

A series of tests were performed on compacted soil specimens in which lime was added in the varying percentages, in order to evaluate the changes in engineering properties of the soil

4.3.1. Compaction test for soil – lime mix

The soil sample was air-dried and passes through a No. 4 sieve, Next 3 kg of the soil passing the sieve was thoroughly mixed with lime and water; and then the mix was put in airtight plastic bag and immersed in to a water tanker for 96 hours. The hydrostatic pressure of the water in the tanker keeps the moisture of the sample intact and avoids carbonation of the lime due to air exposure. Hence, the water could migrate through the soil-lime mixture and allow pozzolanic reactions between the lime and soil minerals to take place.

After completing the specified time in the plastic bag, the sample is taken out of the bag and compaction was conducted in 3 layers, each being given 25 blows. Then, the top of the soil is trimmed level with the mould. The base is removed and the soil and mold is weighed. Moisture content samples are taken from the top, middle and base of the soil.

The above procedure is repeated using a fresh batch of soil and lime mixed to higher moisture content. The procedure was repeated until the weight of the soil in the mould passes the maximum value and begins to decrease.

Compaction test for soil + 2% lime

Table 4. 8: Compaction test result for soil +2% lime mixture

Trial		1	2	3	4	5
Wt of mold + wet soil	gram	5940.7	6017.6	6092.4	6136.5	6065
Wt of mold	gram	4573.2	4573.2	4573.2	4573.2	4573.2
Wt. Wet soil	gram	1367.5	1444.4	1519.2	1563.3	1491.8
Vol. of Comp mold	cm^3	944	944	944	944	944
Wet density	g/cm^3	1.45	1.53	1.61	1.66	1.58
Wt. Cont + Dry soil	grams	96.5	101.9	90.6	94.1	99.5
Weight of water	grams	18	23.4	20.1	25.2	30.1
Weight of container	grams	33.3	32.6	33.2	32.8	33.1
Weight of Dry soil	grams	63.2	69.3	57.4	61.3	66.4
Moisture content	%	28.48	33.64	35.02	41.11	45.33
Dry Density	g/cm^3	1.1275	1.1449	1.1919	1.1736	1.0874

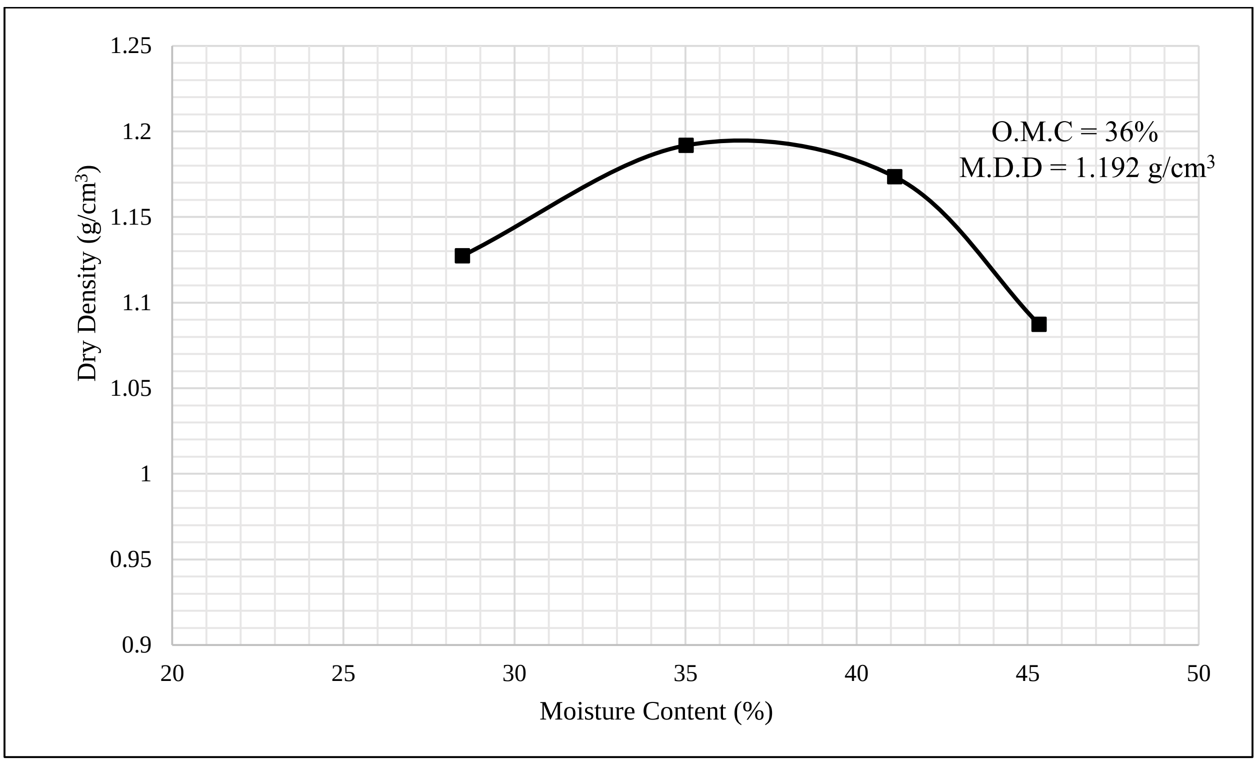

Fig 4. 8: Compaction test result for soil + 2% lime mixture

As shown in Fig 4.7 above, the O.M.C of the untreated soil has increased from 32.7% to 36% while the M.D.D has reduced from 1.27g/cm^3 to 1.192g/cm^3 at 2 % lime addition to the soil.

Compaction test for soil + 4% lime

Table 4. 9: Compaction test result for 4% lime soil mix

Trial	Unit	1	2	3	4
Wt of mold + wet soil	gram	5931.8	6050.6	6166	6154.6
Wt of mold	gram	4573.2	4573.2	4573.2	4573.2
Wt. Wet soil	gram	1358.6	1477.4	1592.8	1581.4
Vol. of Mold	cm^3	944	944	944	944
Wet density	g/cm^3	1.44	1.57	1.69	1.68
Wt. Cont + wet soil	grams	156	162.7	153	138.3
Wt. Cont + Dry soil	grams	130.6	134.4	116.9	104.7
Weight of water	grams	25.4	28.3	36.1	33.6
Weight of container	grams	33.3	33.2	33.1	33.6
Weight of Dry soil	grams	97.3	101.2	83.8	71.1
Moisture content	%	26.10	33.64	43.08	47.26
Dry Density	g/cm^3	1.1413	1.1711	1.1793	1.1376

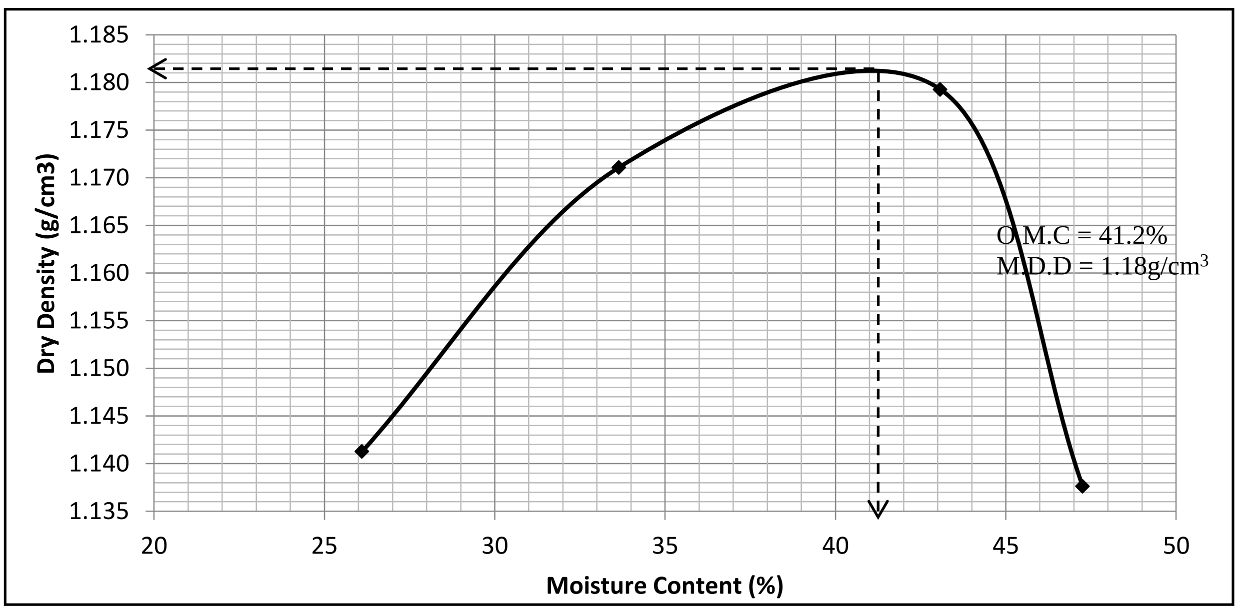

Fig 4. 9: Compaction test for soil +4% lime specimen

The figure above provides the moisture content – dry density relation of the soil + 4% lime mix. Comparison of this result to that of the soil and 2% lime soil mix indicates that the O.M.C. of the soil increased from 33.9% to 41.2 % with 4% lime addition while the M.D.D reduced from 1.27g/cm^3 to 1.182 g/cm^3. Compared to the 2 % lime addition, the O.M.C of the 4% lime soil mix increased from 36% to 41.2% and M.D.D decreased from 1.192 g/cm^3 to 1.182 g/cm^3

Compaction test for soil + 6% lime

Table 4. 10: Compaction test result for soil + 6% lime specimen

Trial		**1**	**2**	**3**	**4**	**5**
Wt of mold + wet soil	gram	5903.2	6003.5	6071	6098	6096
Wt of mold	gram	4573.2	4573.2	4573.2	4573.2	4574.2
Wt. Wet soil	gram	1330	1430.3	1497.8	1524.8	1521.8
Mold Vol.	cm^3	944	944	944	944	945
Wet density	g/cm^3.	1.41	1.52	1.59	1.62	1.61
Wt. Cont + wet soil	grams	107.3	121.8	153	142	136
Wt. Cont + Dry soil	grams	85.8	92.8	109.9	101.1	97
Weight of water	grams	21.5	29	43.1	40.9	39
Weight of container	grams	32.6	33.9	33.1	33.6	34.6
Weight of Dry soil	grams	53.2	58.9	76.8	67.5	62.4
Moisture content	%	40.41	49.24	56.12	60.59	62.50
Dry Density	g/cm^3	1.0034	1.0153	1.0163	1.0058	0.9910

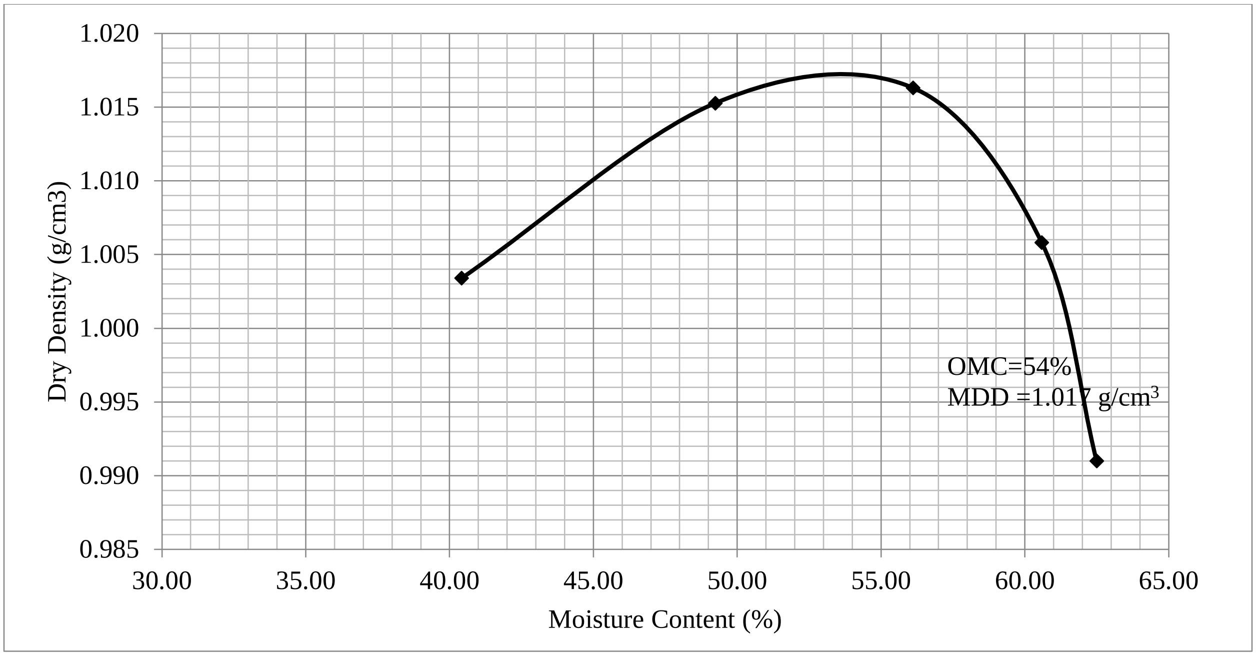

Fig 4. 10: Compaction test result for soil + 6% lime specimen

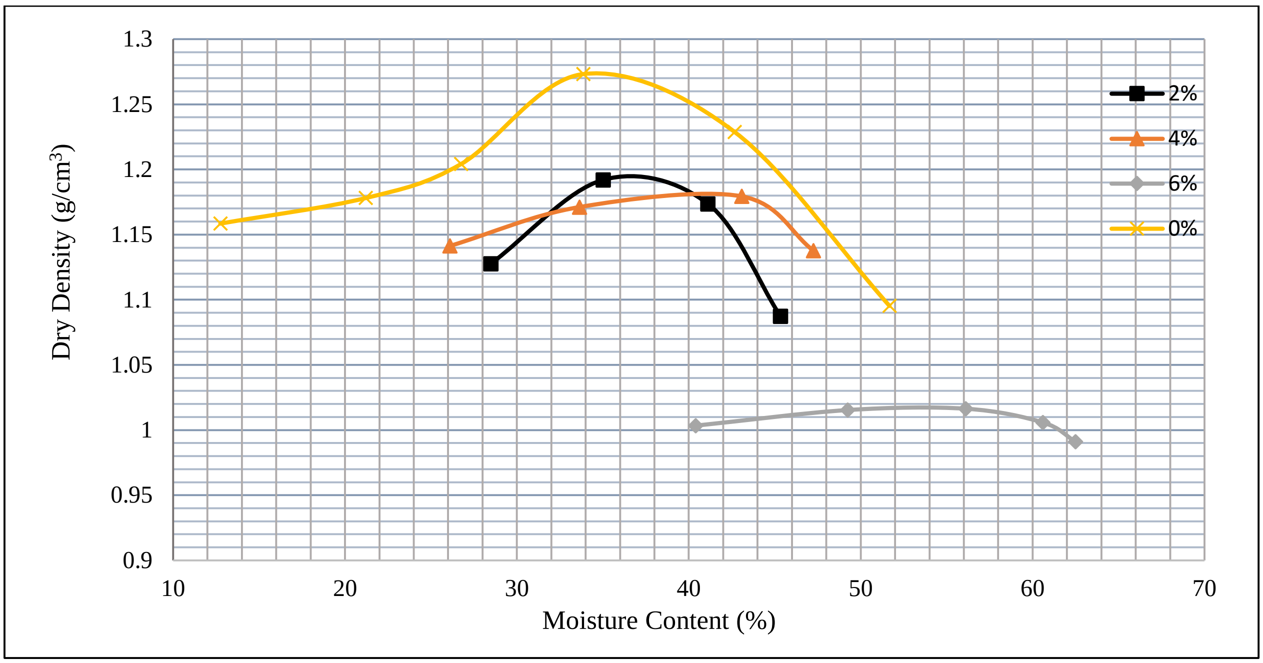

Fig 4. 11: Summary of compaction test results for addition of 2%, 4% and 6% lime

The figure above shows that as the amount of lime added to the soil increase, OMC increases and MDD decreases. The literature review in chapter two identified the reason for the reduction in dry density could be due to an immediate formation of cementitious products which reduce compactibility.

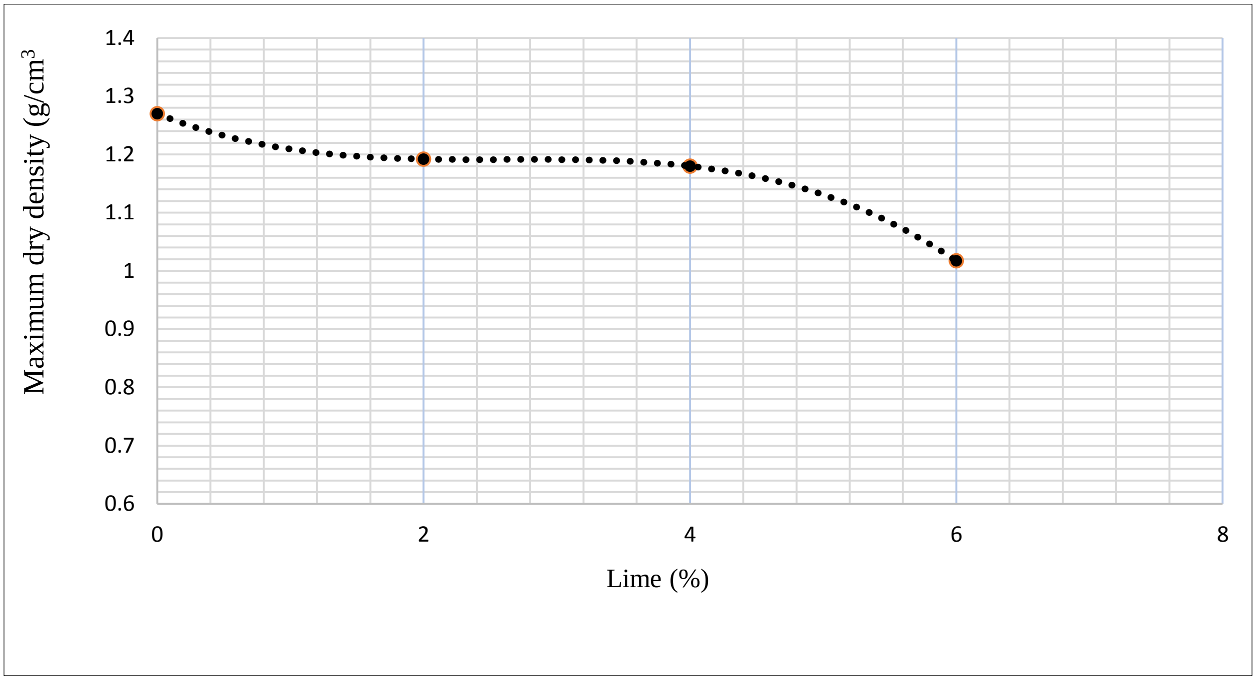

Fig: Lime percentage vs maximum dry density

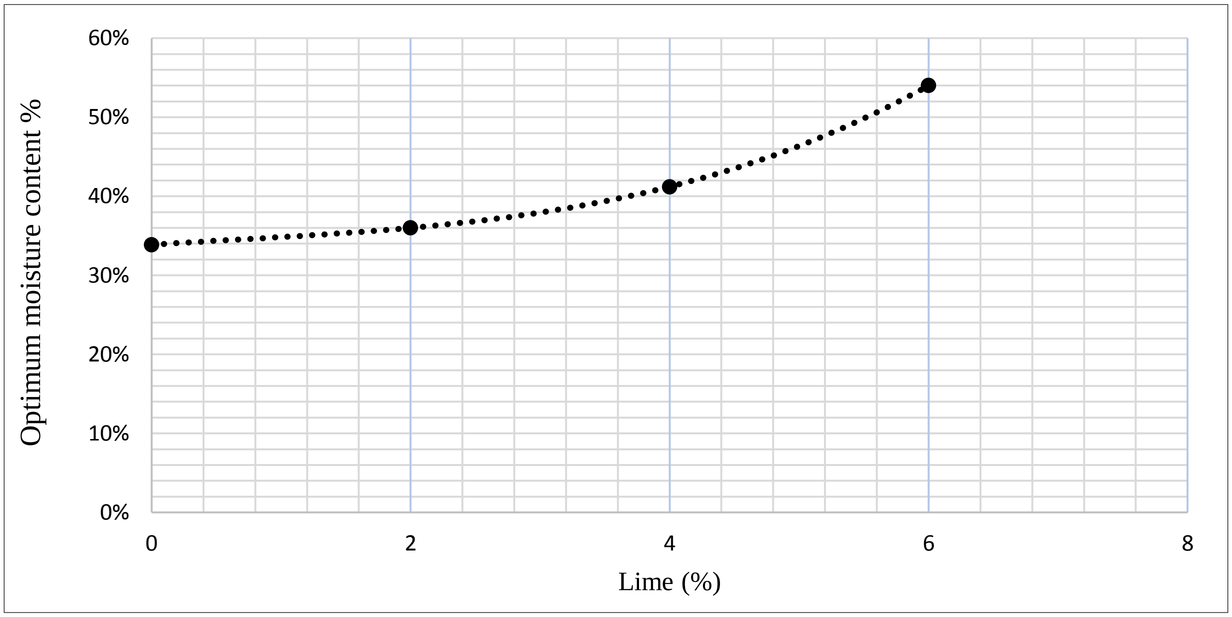

Fig: Lime percentage vs optimum moisture content

4.3.2. Unconfined Compressive Strength of soil – lime mix

A UCS specimen was prepared by mixing the expansive soil, lime and water. The mixed sample was aged for seven days in a water tight plastic bag in a water container. This allows for the pozzolanic reactions to take place without losing any moisture. Then after seven days the sample was re-compacted to the maximum dry density and optimum moisture content as determined from the compaction test conducted for each lime and soil mix.

Unconfined Compressive strength of soil +2% lime specimen

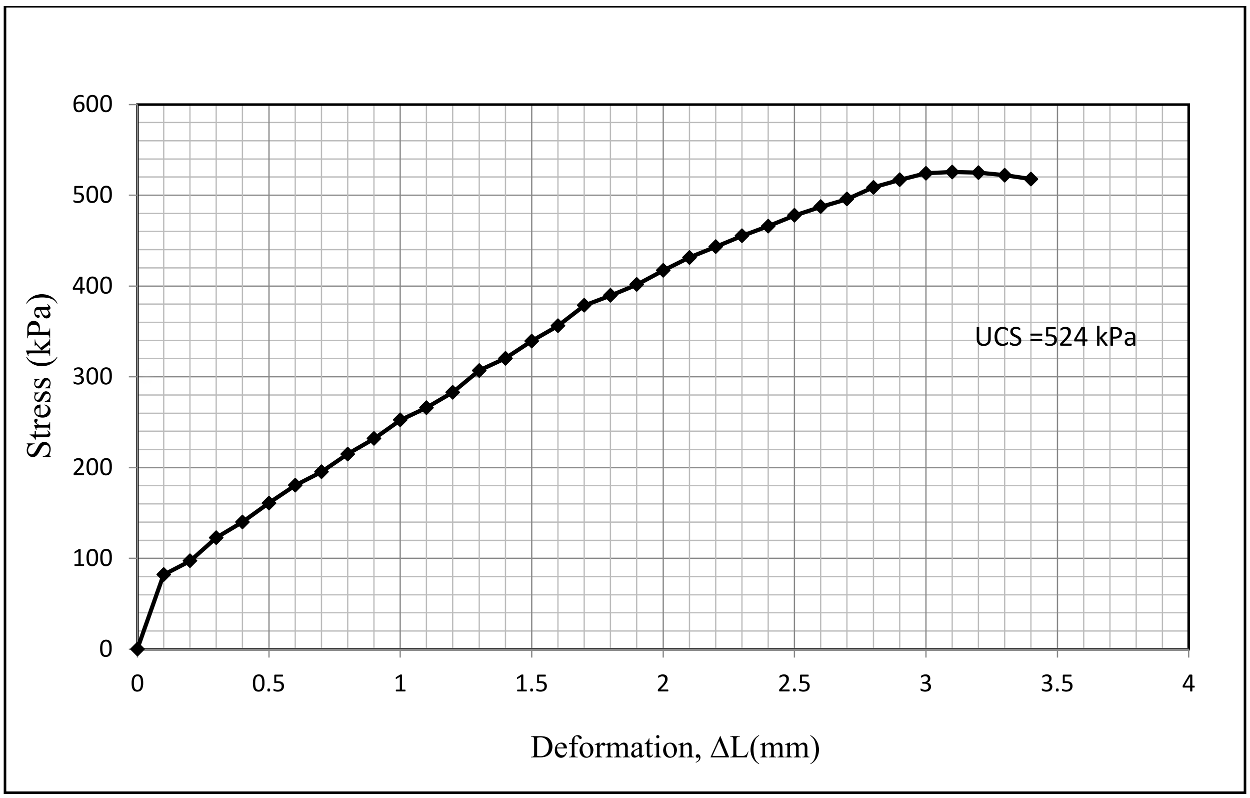

Fig 4. 12: Unconfined Compressive strength of soil +2% lime specimen

The Unconfined compressive strength of the soil + 2% lime is 524 kpa as shown in Fig 4.8 above. It is a significant change compared to that of the untreated soil. The re-compacted soil specimen was tested for moisture content to check the optimum moisture content was achieved.

Table 4. 11: Moisture content of soil +2 % lime UCS specimen

Sample No.	Weight
M_C = Mass of empty, clean can + lid (grams)	33.5
M_{CMS} = Mass of can, lid, and moist soil (grams)	142
M_{CDS} = Mass of can, lid, and dry soil (grams)	102.94
M_S = Mass of soil solids (grams)	69.44
W = Water content, w%	36%

Unconfined compressive strength of soil + 4 % lime

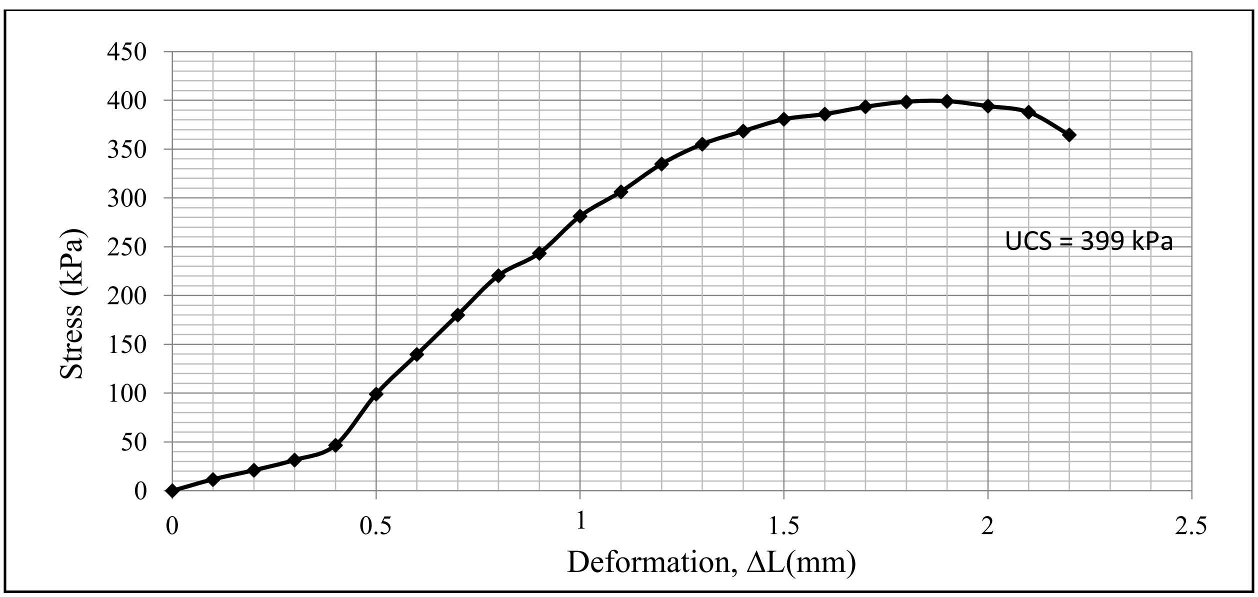

Fig 4. 13: Unconfined Compressive Strength of soil + 4 % lime specimen

The Unconfined compressive strength of the soil +4 % lime specimen is 399 kPa as shown in Fig 4.13 above. It is a significant change compared to that of the untreated soil. The re-compacted soil specimen was tested for moisture content to validate whether the OMC was achieved. The moisture content is determined as shown in the table below.

Table 4. 12: Moisture content of soil +4 % lime UCS specimen

Sample No.	Weight
M_C = Mass of empty, clean can + lid (grams)	33.5
M_{CMS} = Mass of can, lid, and moist soil (grams)	153
M_{CDS} = Mass of can, lid, and dry soil (grams)	82.5
M_S = Mass of soil solids (grams)	50
W = Water content, w%	41%

Unconfined Compression strength test of soil +6% lime

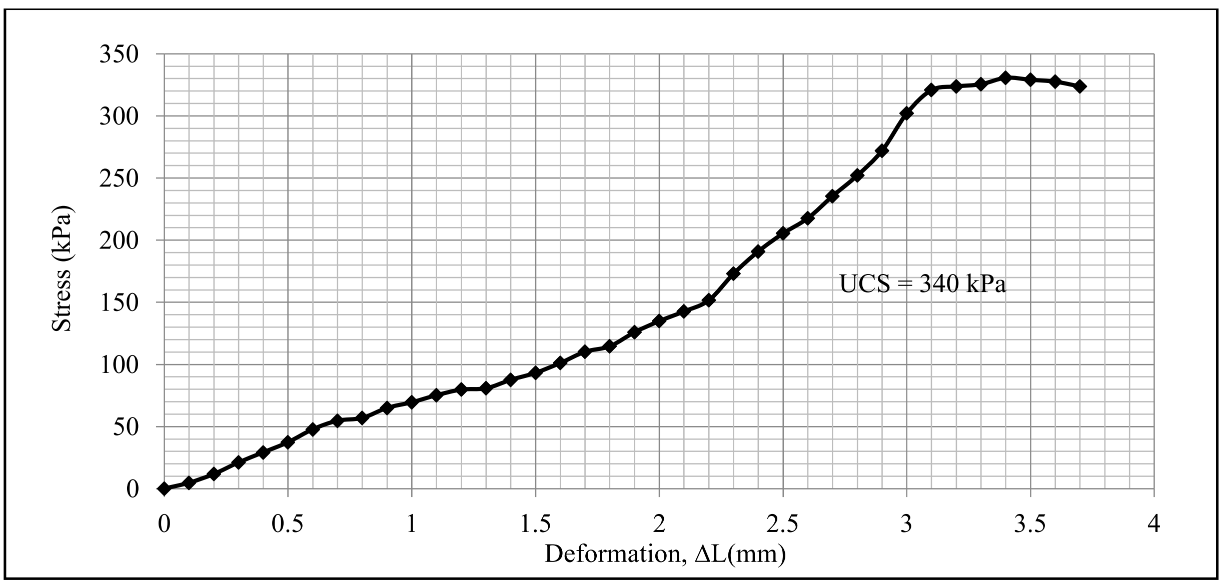

Fig 4. 14: Unconfined Compressive Strength test results soil +6% lime specimen

The Unconfined compressive strength of the soil +6 % lime specimen is 340 kpa as shown in Fig 4.16 above. It is a significant change compared to that of the untreated soil. The re-compacted soil specimen was tested for moisture content to validate whether the Optimum Moisture content was achieved. The moisture content is determined as shown in the table below.

Table 4. 13: Moisture content of soil +6 % lime UCS specimen

Sample No.	**Weight**
M_C = Mass of empty, clean can + lid (grams)	33.5
M_{CMS} = Mass of can, lid, and moist soil (grams)	152
M_{CDS} = Mass of can, lid, and dry soil (grams)	89.195
M_S = Mass of soil solids (grams)	55.695
W = Water content, w%	53%

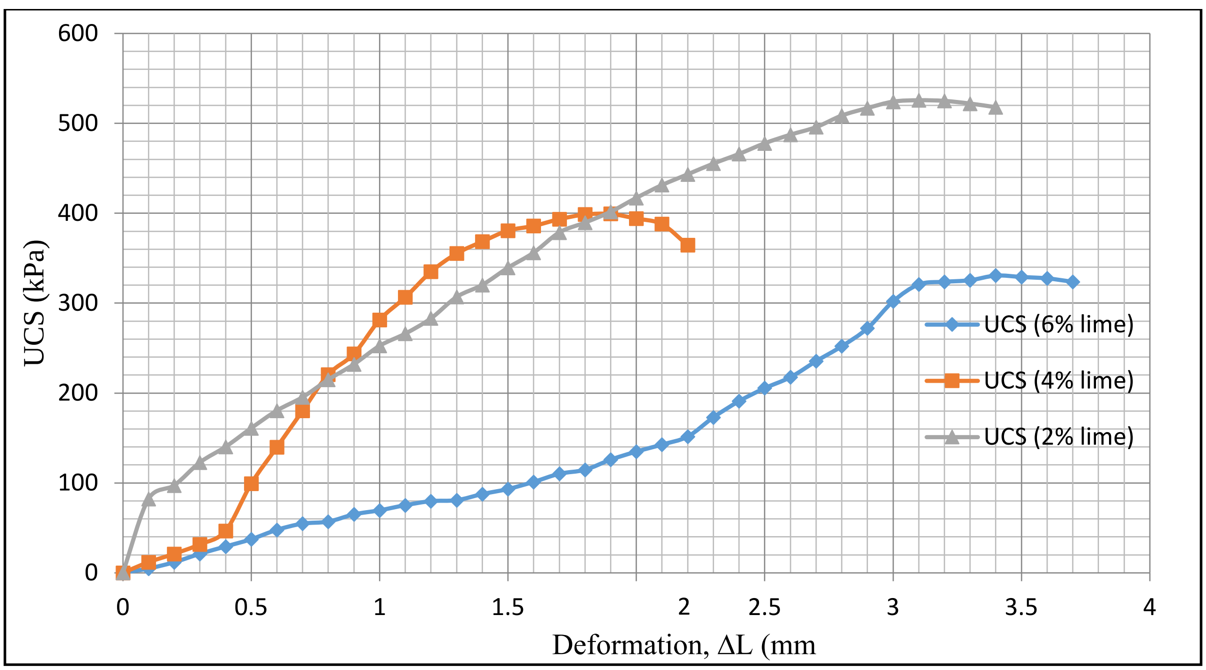

Fig 4. 15: Summary of UCS test results for addition of 2%, 4% and 6% lime

The figure above illustrates that the expansive soil has the highest unconfined compressive strength at the addition of 2% lime. Then the strength starts to decline at higher percentages of lime contents. The reason for the decline could be, the molding water used in sample preparation was optimum moisture content. Since the optimum moisture content is related to the dry density of the soil rather than compressive strength of lime treated soil. The optimum moisture content used as molding water could have been insufficient to react with the lime OR it could have been excessive thereby softening the sample.

4.3.3 Swelling Pressure of soil – lime mix

The swelling pressure in the consolidometer test is defined as the pressure which is required to return a swelled specimen back to its original state. The re-compacted mixure of soil lime (at O.M.C and M.D.D) specimen is placed in consolidometer and a normal load equal to 1 Psi (7 kNm^2) is applied. Then the sample is inundated and allowed to swell. After the swelling has completed, the sample is loaded. If the sample does not return to its original state, the load is increased until it reaches the original volume.

Swelling pressure test for soil + 2% lime

Table 4. 1: Oedometer swelling pressure test result for soil +2% lime

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height, H_v (mm)	Void Ratio, e
7	12.800	0.02	17.00	7.32	0.73
7	13.110	-0.29	17.29	7.61	0.76
50	12.850	-0.03	17.03	7.35	0.73
100	12.580	0.24	16.76	7.08	0.71

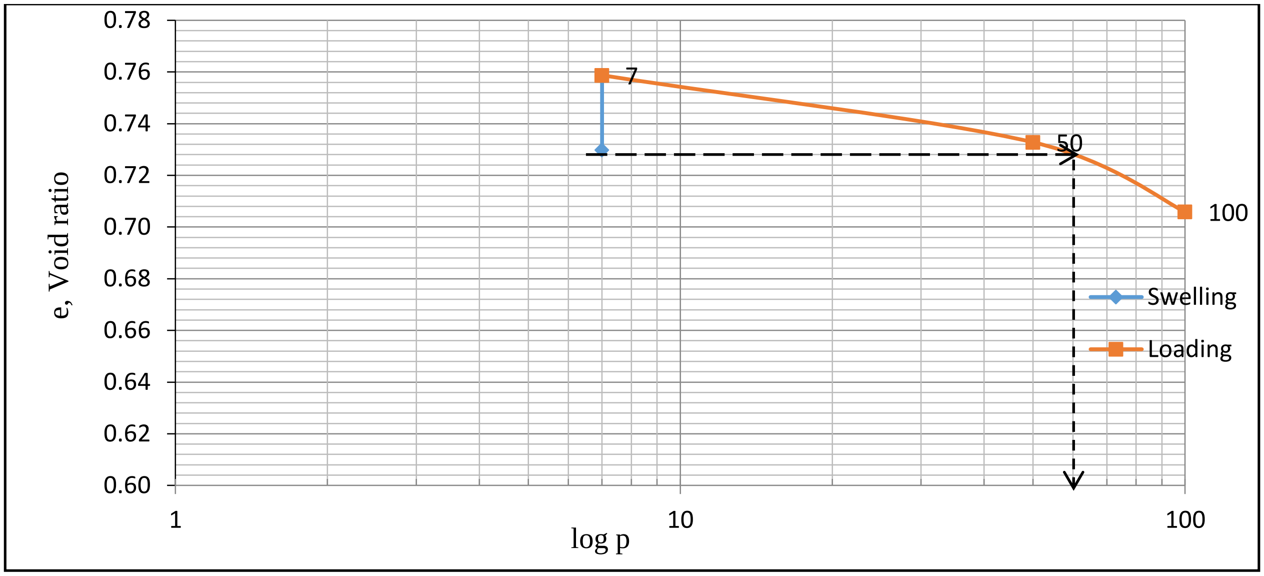

Fig 4. 16: e – Log p curve for soil + 2% lime specimen

The above e- log p curve was used to determine the swelling pressure of the 2% lime-soil mix. When compared to that of the untreated soil, the swelling pressure has reduced from 450 kpa to 60 kpa.

Swelling pressure of soil + 4% lime

Table 4. 15: Oedometer swelling pressure test result of soil + 4 % lime

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, e
7	13.800	0.02	17.00	7.32	0.73
7	13.875	-0.05	17.06	7.38	0.74
50	13.761	0.06	16.94	7.26	0.72

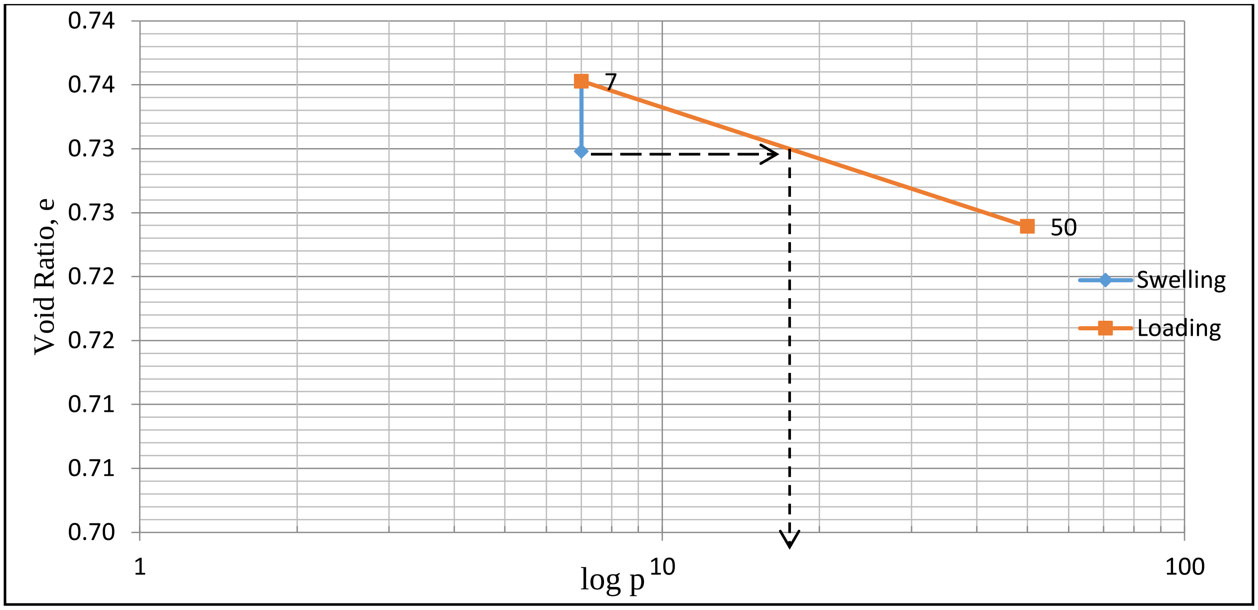

Fig 4. 17: e-log p curve of soil + 4% lime specimen

The above e- log p curve was used to determine the swelling pressure of the 2% lime-soil mix. When compared to that of the untreated soil, the swelling pressure has reduced from 450 kPa to 18 kPa.

Swelling pressure of soil + 6% lime

The 6% lime-soil mix did not swell at setting load of 7 kPa. The Specimen was inundated with water for 12 days but did not present any volume change. Therefore the swelling pressure of the specimen is **0 kPa.**

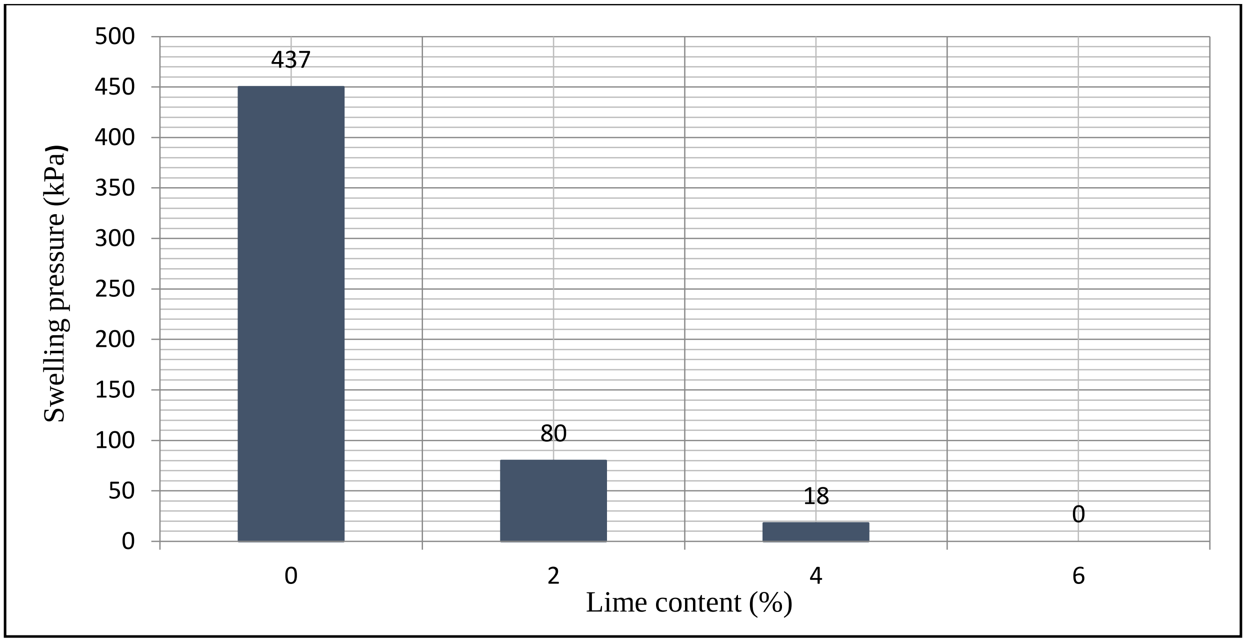

Fig 4. 18: Summary of swelling pressure test results for addition of 2%, 4% and 6% lime

As shown in the figure above the addition of lime significantly reduced the swelling pressure of the soil. At 6 % amount of lime addition the swelling pressure of the soil reduced to value of zero. Based on this result, it can be concluded that 6% lime is the optimum amount required to stabilize the swelling property of the expansive soil.

4.3.4. Atterberg limits of soil and lime

For soil + 2% lime

Table 4. 16: Atterberg limit results for soil + 2% lime

	Liquid Limit				Plastic Limit
Trial No	1	2	3	4	1
Container No	96	147	14	173	62
Mass of container, g	33.60	33.60	33.60	33.40	33.10
Mass of container + Wet soil, g	74.00	68.00	59.20	63.00	43.50
Mass of container + Dry soil, g	54.80	51.30	46.60	48.10	40.50
Mass of water, g	19.20	16.70	12.60	14.90	3.00
Mass of dry soil, g	21.10	17.70	13.00	14.70	7.70
Water content, %	91.00	94.35	96.92	101.36	**38.96%**
No of blows	37	28	23	16	------

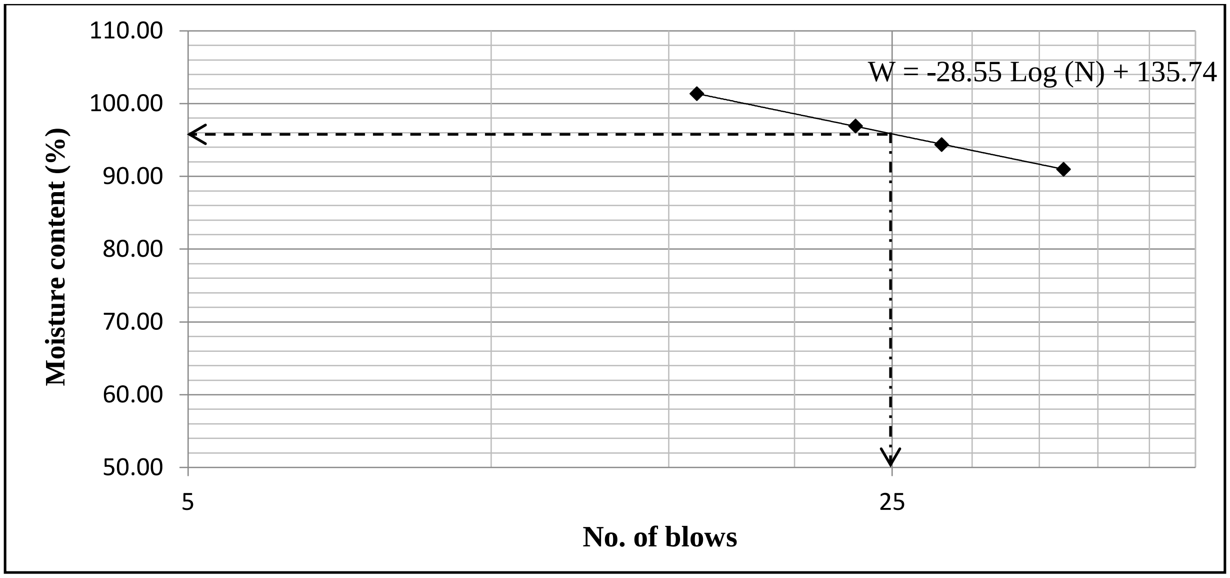

Fig 4. 19: liquid limit of soil + 2% lime

Trial	No. of blows (N)	log (N)	W (%)	$\log\frac{N}{N_{mean}}$	W- W_{mean}	$\left[\log\frac{N}{N_{mean}}\right]^2$	$\left(\log\frac{N}{N_{mean}}\right) \times (W - W_{mean})$
1	16	1.204	101.36	-0.191	5.4525	0.0365	-1.0424
2	23	1.361	96.92	-0.034	1.0125	0.0011	-0.0339
3	28	1.447	94.35	0.052	-1.5575	0.0026	-0.0807
4	37	1.568	91	0.173	-4.9075	0.0298	-0.8485
Mean	26	1.395	95.9075		∑	0.0702	-2.0056

W= F_1 Log (N) + b, F_1 = Flow index (the slope of the regression line or flow curve)

$$F_1 = \frac{\sum\left[\left(\log\frac{N_j}{N_{mean}}\right)\times(W_j - W_{mean})\right]}{\sum\left[\log\frac{N_j}{N_{mean}}\right]^2} = \frac{-2.0056}{0.0702} = -28.55$$

Replace (26, 95.9) in (W, N) of W= F_1 Log (N) + b

Solve for b, b = 26 + 28.55 Log (26) = 135.74

The flow curve equation is:

W = -28.55 Log (N) + 135.74

Therefore, the liquid limit is moisture content at 25 blows

L.L = -28.55 Log (25) +135.74 = **95.83%**

Hence, the plastic index (P.I) = 95.83% – 38.96 = **56.87%**

Atterberg limits of soil + 4% lime

	Liquid Limit				Plastic Limit
Trial No	1	2	3	4	1
Container No	48	24	11	78	44
Mass of container, g	33.70	33.60	33.60	33.40	32.80
Mass of container + Wet soil, g	68.00	65.80	68.40	64.40	43.80
Mass of container + Dry soil, g	52.90	50.70	51.20	48.40	40.60
Mass of water, g	15.10	15.10	17.20	16.00	3.20
Mass of dry soil, g	19.20	17.10	17.60	15.00	7.80
Water content, %	78.65	88.30	97.73	106.67	**41.03%**
No of blows	39	29	23	14	------

Table 4. 17: Atterberg limit of soil + 4% lime

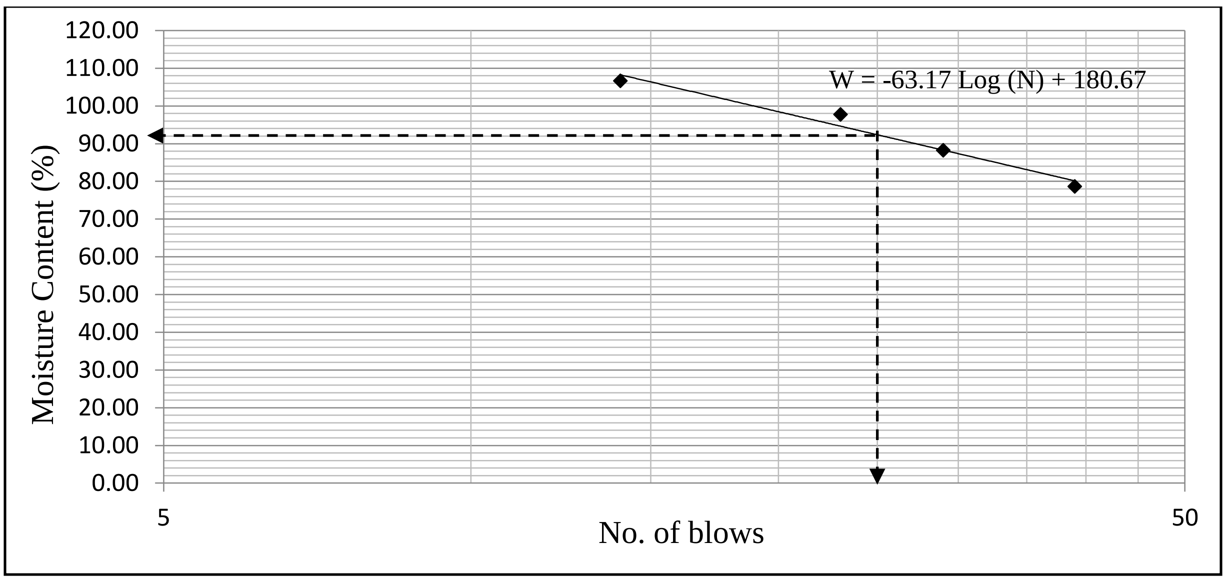

Fig 4. 20: Liquid Limit of the soil + 4% lime

Trial	No. of blows (N)	log (N)	W (%)	$\log\frac{N}{N_{mean}}$	W-W_{mean}	$\left[\log\frac{N}{N_{mean}}\right]^2$	$\left(\log\frac{N}{N_{mean}}\right) \times (W - W_{mean})$
1	14	1.146	106.67	-0.244	13.8325	0.0596	-3.377
2	23	1.361	97.734	-0.0286	4.8925	0.0001	-0.139
3	29	1.462	88.324	0.0720	-4.5375	0.0051	-0.327
4	39	1.591	78.652	0.2007	-14.1875	0.0402	-2.847
Mean	26.25	1.390	92.837		∑	0.105940862	-6.692

W= F_1 Log (N) + b, F_1 = Flow index (the slope of the regression line or the flow curve)

$$F_1 = \frac{\sum\left[\left(\log\frac{N}{N_{mean}}\right)\times(W - W_{mean})\right]}{\sum\left[\log\frac{N}{N_{mean}}\right]^2}$$

F1 = -63.17477896

b = 180.6712664

the flow curve equation is:

W = -63.17 Log (N) + 180.67

Therefore, the liquid limit is moisture content at 25 blows

L.L = -63.17 Log (25) + 180.67 = **92.36%**

Hence, the plastic index (P.I) = 92.36% – 41.03 = **51.33%**

Atterberg limits for soil + 6 % lime

Table 4. 18: Atterberg limit for soil + 6% lime

	Liquid Limit				**Plastic Limit**
Trial No	1	2	3	4	1
Container No	95	155	150	98	183
Mass of container, g	33.20	33.50	33.30	33.40	33.00
Mass of container + Wet soil, g	76.50	65.40	64.50	74.20	42.10
Mass of container + Dry soil, g	56.40	50.40	49.70	54.20	39.20
Mass of water, g	20.10	15.00	14.80	20.00	2.90
Mass of dry soil, g	23.20	16.90	16.40	20.80	6.20
Water content, %	86.64	88.76	90.24	96.15	**46.77**
No of blows	35	26	23	14	------

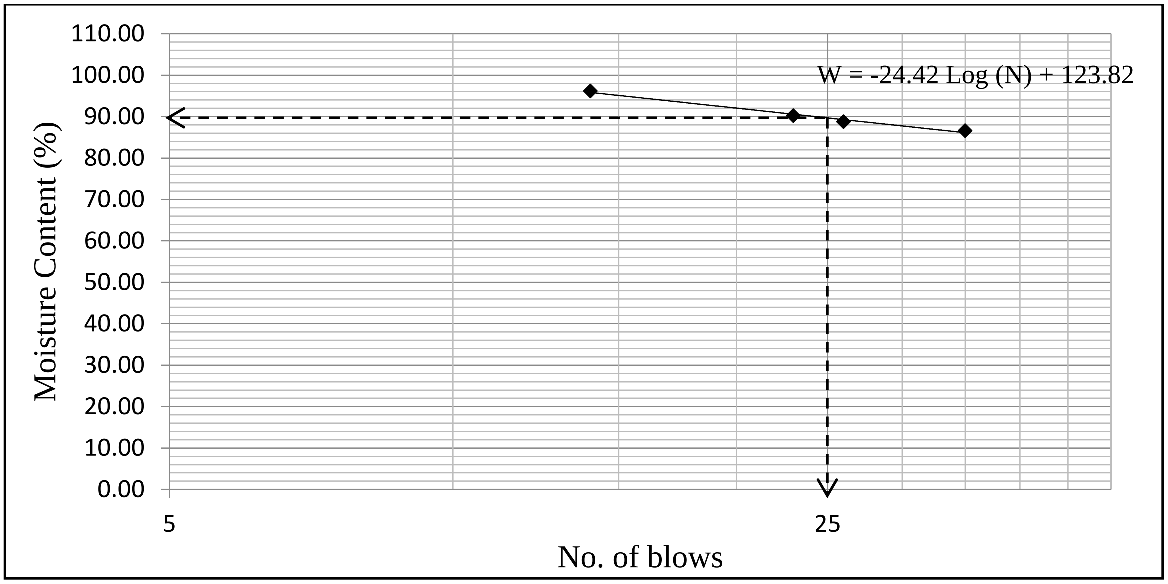

Fig 4. 21: Liquid limit of the soil +6% lime

Trial	**No. of blows (N)**	**log (N)**	**W (%)**	$\log\frac{N}{N_{mean}}$	**W- W_{mean}**	$\left[\log\frac{N}{N_{mean}}\right]^2$	$\left(\log\frac{N}{N_{mean}}\right) \times (W - W_{mean})$
1	14	1.146	96.15	-0.220	5.7025	0.0486	-1.257
2	23	1.361	90.24	-0.0049	-0.20	2.496E-05	0.00103677
3	26	1.414	88.76	0.0482	-1.68	0.0023	-0.0814
4	35	1.544	86.64	0.177	-3.80	0.0314	-0.67523
Mean	24.5	1.366	90.4475		∑	0.0824	-2.013

W= F_1 Log (N) + b, F1 = Flow index (the slope of the regression line or the flow curve)

$$F_1 = \frac{\sum\left[\left(\log\frac{N_j}{N_{mean}}\right) \times \left(W_j - W_{mean}\right)\right]}{\sum\left[\log\frac{N_j}{N_{mean}}\right]^2}$$

F_1= -24.42,

Putting (24.5, 1.366) in place of (W, N) in equation, W= F_1 Log (N) + b and solve for b

b = 24.5 + 24.42 Log (25) = 123.82

The flow curve equation is:

W = -24.42 Log (N) + 123.82

Therefore, the liquid limit is moisture content at 25 blows

L.L = -24.42 Log (25) + 123.82 = **89.68%**

Hence, the plastic index (P.I) = 89.68% – 46.77 = **42.91%**

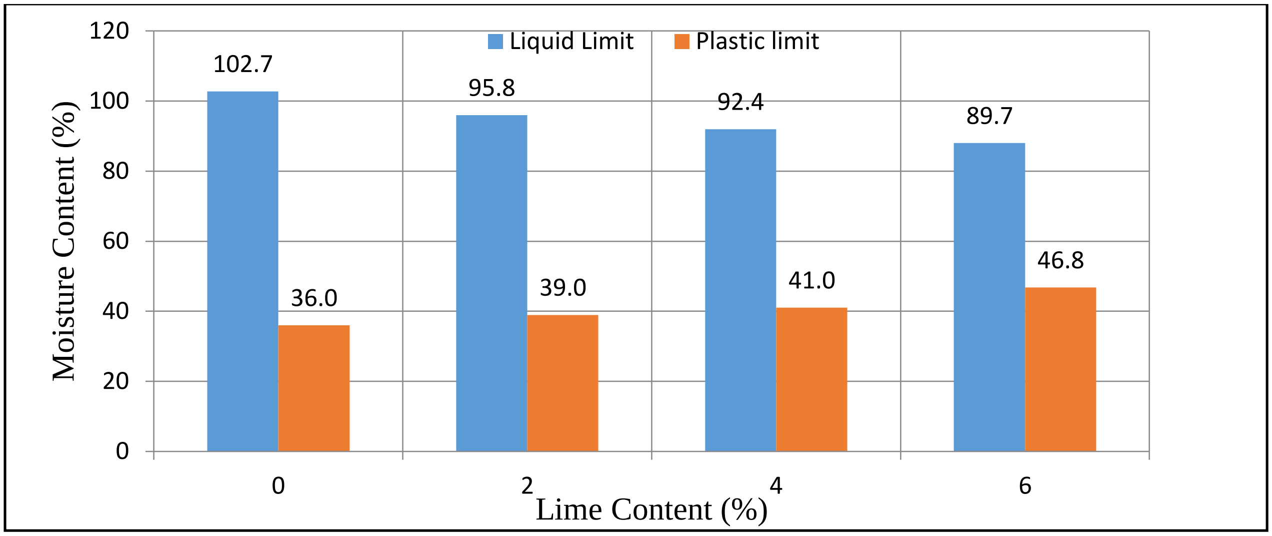

Fig 4. 22: Liquid Limit and plastic limit result at different lime content

The figure above shows the comparison of liquid limit and plastic limit values for different lime contents. As the amount of lime added to the soil increases, significant reduction in the value of liquid limit is observed. On the other hand, as the amount of lime added to the soil increase, the plastic limit also showed some increase.

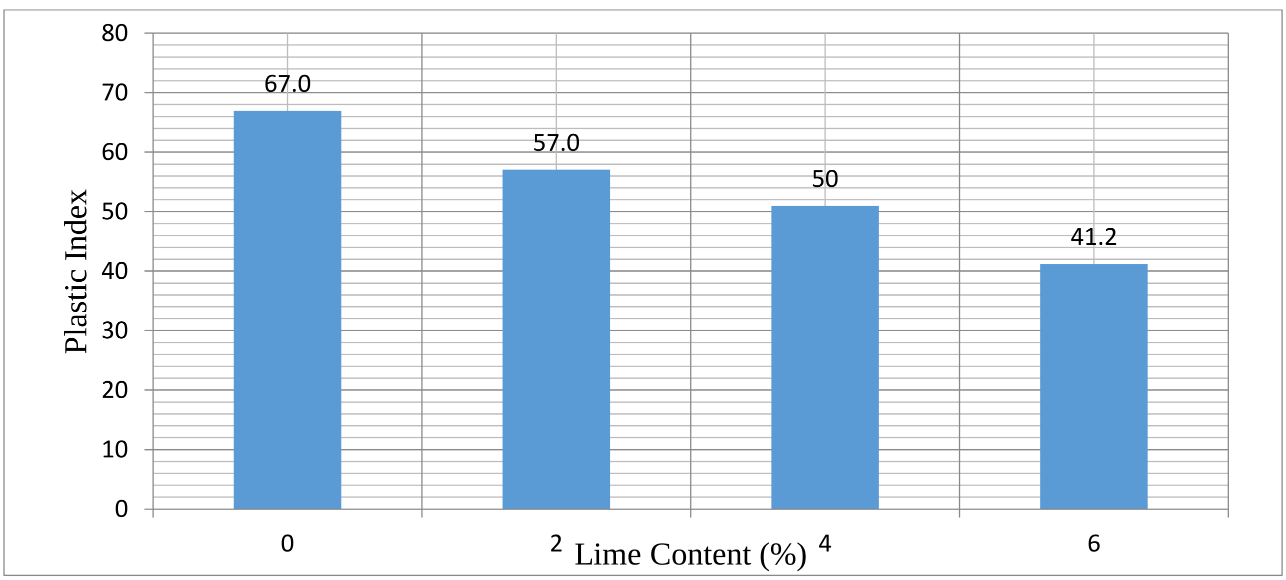

Fig 4. 23: Summary of plastic Index test results for addition of 2%, 4% and 6% lime

The figure above demonstrates that the reduction in liquid limit along with the increase in plastic limit results a significant reduction in the plasticity index of the soil for a series of addition of lime. The reduction in plastic index contributes to the improvement in workability by reducing moisture absorption of the soil.

4.3.5. California Bearing Ratio of soil and lime mix

An air dried soil sample and lime is thoroughly mixed with water (OMC) as obtained from the compaction test result. Then, the mixture is put into water tight plastic bag and immersed in to water tank for seven days. By doing so, there won't be moisture loss and enough time will be provided for curing. Then it is compacted in the mold and immersed in a tank of water allowing free access of water to the top and bottom of specimen for 96 hours. Afterwards, the CBR test is conducted and the load penetration result is shown in the tables below.

California Bearing Ratio of soil + 2% lime

Table 4. 19: CBR test result of soil + 2% lime

Penetration (mm)	**Ring (Div)**	**Load (N)**	**Stress (N/mm^2)**	**Standard stress (N/mm^2)**	**CBR (%)**
0	0.0	0	0		
0.64	4.0	103	0.05		
1.27	7.0	180	0.09		
1.91	8.0	206	0.11		
2.54	10.0	257	0.13	6.9	2
3.18	11.0	283	0.15		
3.81	12.0	309	0.16		
4.45	13.0	334	0.17		
5.08	14.0	360	0.19	10.3	2
7.62	15.0	386	0.20		
10.16	15.0	386	0.20		
12.70	16.0	412	0.21		

It is shown in the table that at 2.5mm depth the CBR value (1.93) is greater than that of the 5mm depth (1.81). Therefore the CBR value of the soil + 2% lime is 2%

California Bearing Ratio test for soil + 4% lime

Table 4. 20: CBR result of soil + 4% lime

Penetration (mm)	Ring (Div)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0	0.0	0	0		
0.64	8.0	206	0.11		
1.27	12.0	309	0.16		
1.91	18.0	463	0.24		
2.54	19.0	489	0.25	6.9	3.7
3.18	21.0	540	0.28		
3.81	22.0	566	0.29		
4.45	23.0	592	0.31		
5.08	24.0	618	0.32	10.3	3.1
7.62	26.0	669	0.35		
10.16	27.0	695	0.36		
12.70	28.0	720	0.37		

It is shown in the table that at 2.5mm depth the CBR value (3.66) is greater than that of the 5mm depth (3.10). Therefore the CBR value of the soil + 4% lime is 4%

California Bearing Ratio test for soil + 6% lime

Table 4. 21: CBR test result for 6% lime

Peneteration (mm)	Ring (Div)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0	0.0	0	0		
0.64	5.0	129	0.07		
1.27	12.0	309	0.16		
1.91	16.0	412	0.21		
2.54	20.0	515	0.27	6.9	3.8
3.18	22.0	566	0.29		
3.81	26.0	669	0.35		
4.45	28.0	720	0.37		
5.08	29.0	746	0.39	10.3	3.7
7.62	33.0	849	0.44		
10.16	35.0	901	0.47		
12.70	36.0	926	0.48		

It is shown in the table that at 2.5mm depth the CBR value (3.85) is greater than that of the 5mm depth (3.74). Therefore the CBR value of the soil + 2% lime is 4%

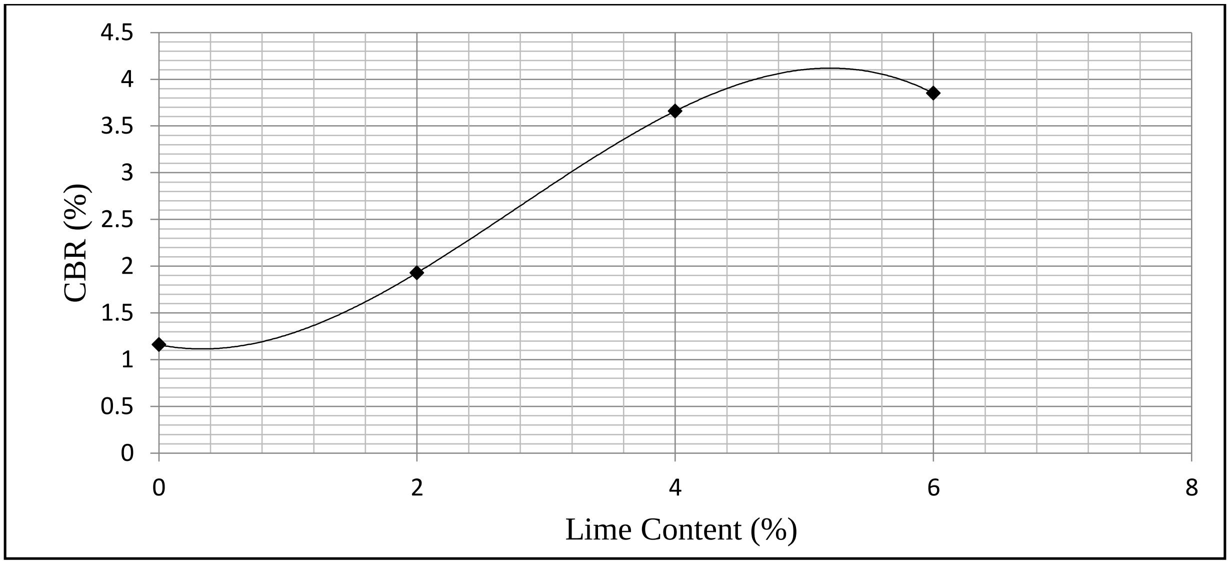

Fig 4. 24: CBR (soaked) values at different lime contents

The figure above shows that as the amount of lime added to the soil increases the CBR value also increases.

4.4. Identification of Lime Fixation Point

It is important to identify the amount of lime added to the soil that results in optimum result in geotechnical properties. This amount of lime is known as Lime fixation point. To determine the lime fixation point, the effect of adding 2%, 4%, and 6% lime on moisture density relation, unconfined compressive strength, consistency limits and swelling pressure of the soil must be compared.

It has been already established that the soil is expansive It is known that the problematic characteristic of expansive soil is its swell shrink property. Hence, the dominant factor to consider while fixing the optimum lime amount is the swelling pressure. It has been shown in the previous section that a lime amount of 6% obtains zero swelling pressure at O.M.C and M.D.D. Therefore, the optimum lime amount that stabilizes the soil is 6%

4.5. Effect of Moisture Variation at lime fixation point

The previous section stated that addition of 6% lime at O.M.C (53%) results in best results. This section shows the results on variations of some properties of the soil at the wet and dry side of this moisture content

Swelling Pressure

Specimens were prepared using 6% lime, soil and different amounts of water. Swelling pressure at 41%, 45%, 50%, 60%, 62% and 67% moisture contents was measured. Samples with moisture content below 41% were difficult to prepare because the sample starts to crumble when extruded from the compaction mold. Preparation of samples with moisture content higher than 67% were also difficult. The samples were highly plastic, muddy and sticky which makes compaction and extruding impossible.

All the samples (41% - 67%) were measured for swelling pressure and the table below shows that moisture content variation at 6% lime addition has no significant effect on the swelling pressure of the soil.

Table 4. 22: Swelling Pressure at different moisture contents (left is dry side, right is wet side)

Moisture content	Swelling Pressure	Moisture content	Swelling pressure
41%	0 kPa	60%	0 kPa
45%	0 kPa	62%	0 kPa
50%	0 kPa	67%	0 kPa

Unconfined Compressive Strength

Unconfined compressive strength test is conducted on lime treated soil specimens having moisture contents other than the OMC. The specimens were prepared by mixing air dried soil passing through No.4 sieve with the optimum amount of lime and a series of water amounts. The samples were cured and prepared using the same procedure as the used in previous UCS tests.

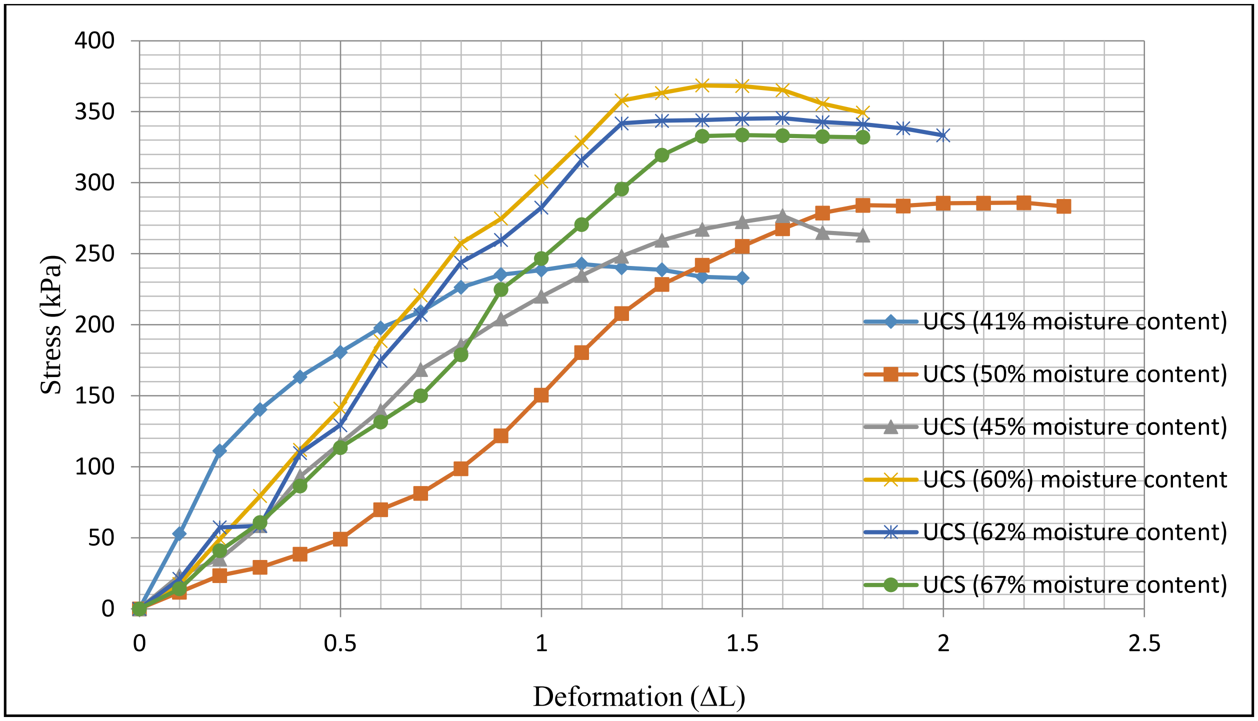

Fig 4. 25: UCS curves for specimens cured 7 days at 41%, 50%, 45%, 60%, 62%, 67% molding water contents

The figure above shows the unconfined compressive strength curve of the soil treated with 6% lime at different moisture contents. It can be observed in the graph that at 50% moisture content, the soil lime mix gained the maximum strength more slowly than the ones prepared by other moisture contents. Furthermore, the fastest strength gain is observed at 41% moisture content. The figure also indicates that the highest strength is observed at 60% moisture content.

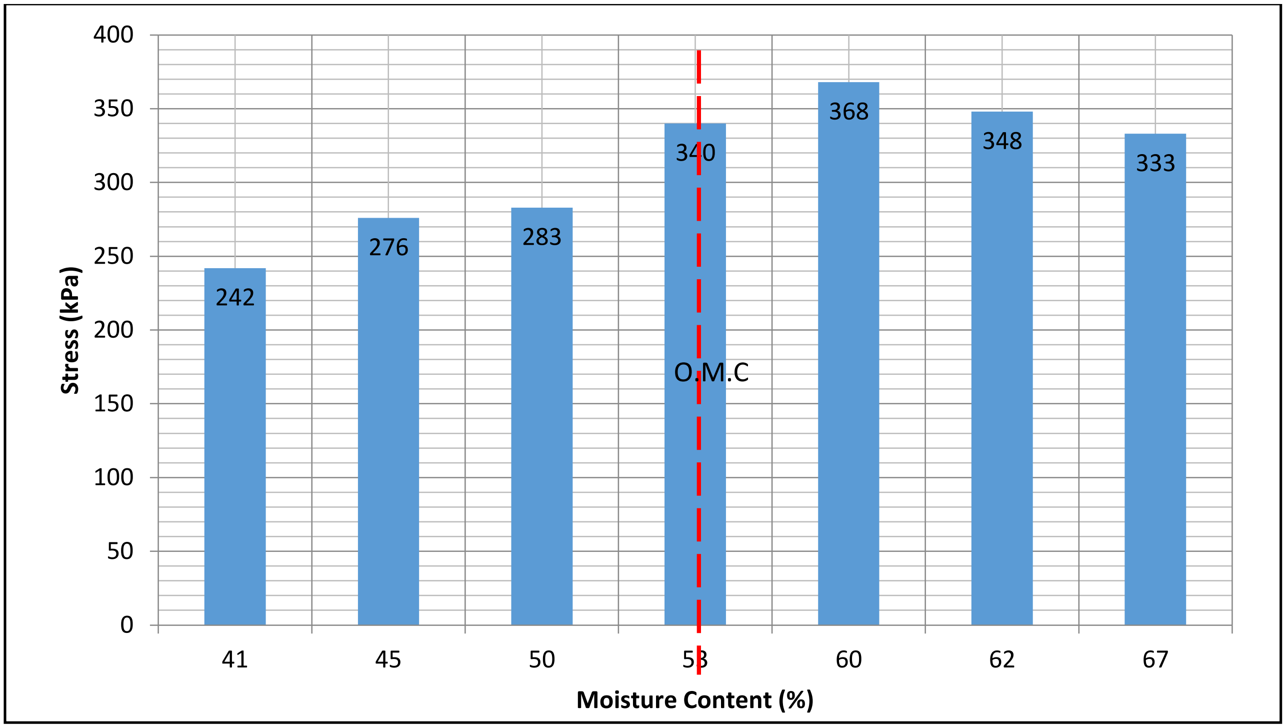

Fig 4. 26: UCS test results on wet and dry side of OMC (7 day curing)

The summary of UCS test results of samples cured for seven days is shown in the figure above. It shows that the samples with moisture content on the wet side of OMC have higher compressive strength than samples on the dry side of the OMC. The sample prepared at the OMC have higher compressive strength than those samples prepared on the dry side of the OMC and lower compressive strength than samples prepared on the wet side of the OMC. Hence the additional water added, after the OMC has been attained, can contribute to the lime stabilization process by providing higher compressive strength.

4.6. Effect of curing time on molding water variation of the lime stabilization

To demonstrate the influence of curing time on the higher strength obtained on the wet side of the OMC, the soil and lime mixture was compacted at the moisture contents previously mentioned (41%, 45%, 50%, 53%, 60%, 62%, 67%) and cured only for 24 hours. Then the UCS test was conducted on soil lime mix specimen so that the results obtained, as in shown in the figures below, can be compared with that of specimens cured for 7 days.

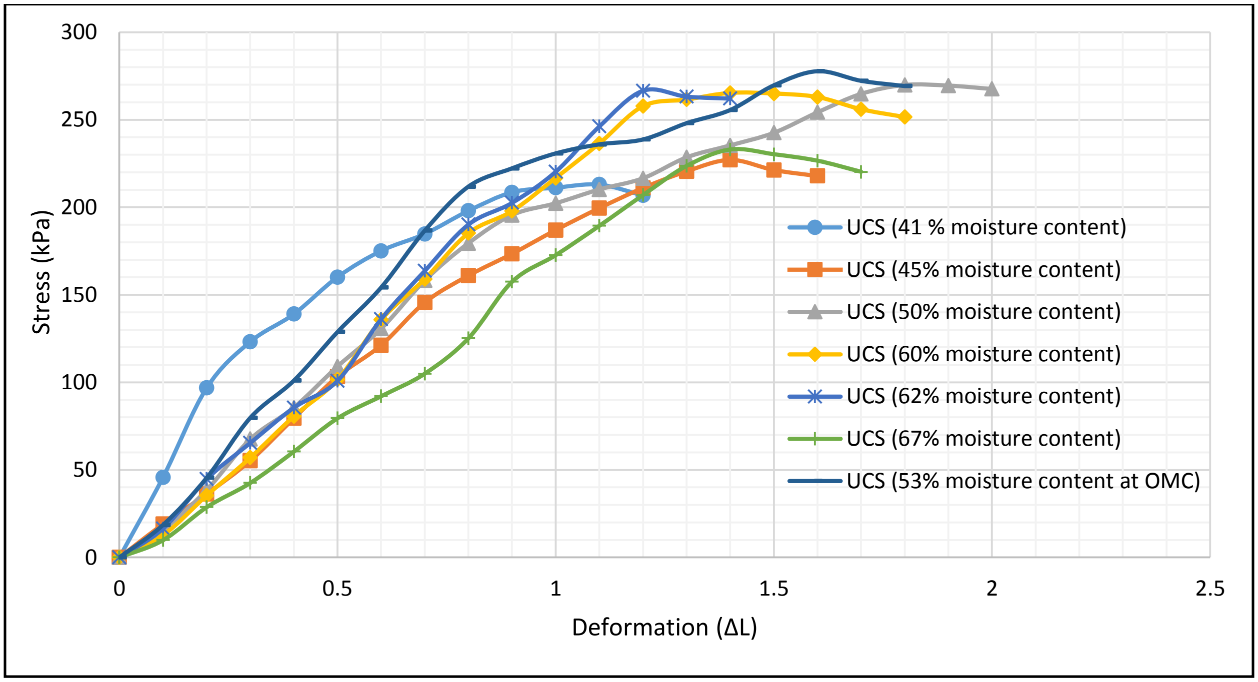

Fig 4.27 UCS curves for specimens cured 24 hours at moisture contents of (41%, 50%, 45%, 60%, 62%, 67%)

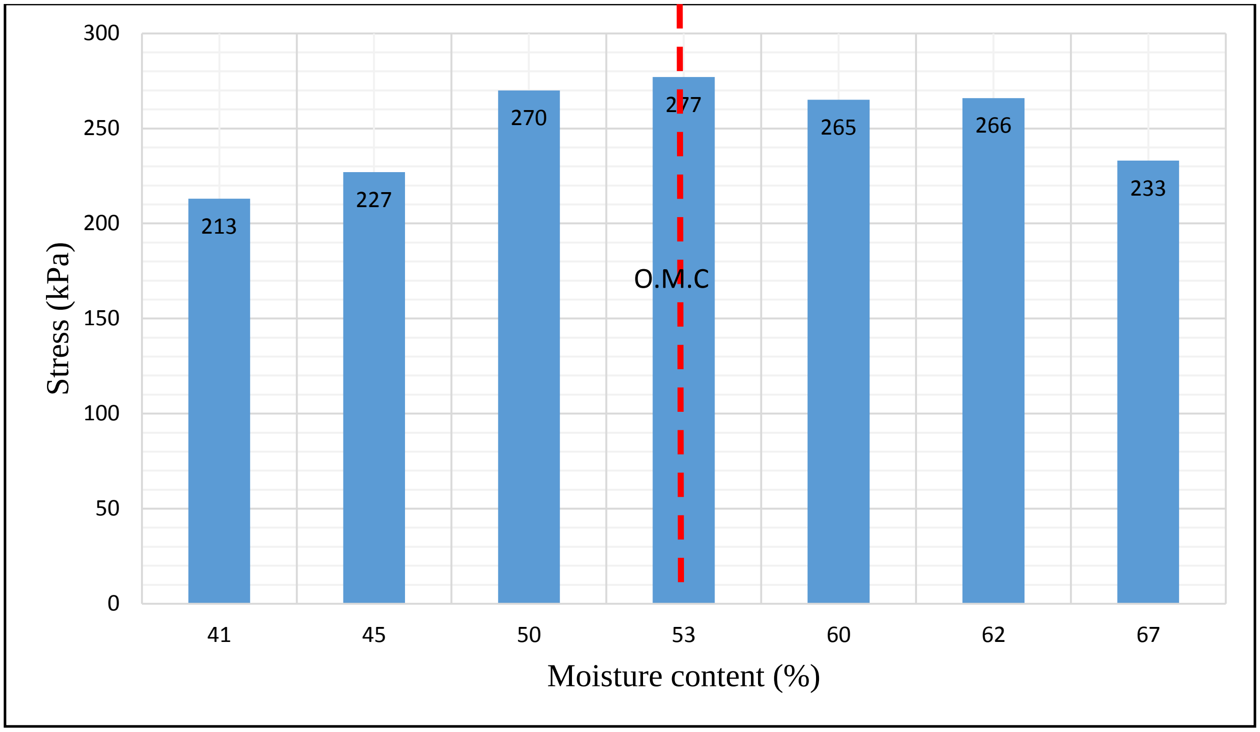

Fig 4.27 UCS results for specimens cured 24 hours at moisture contents of (41%, 50%, 45%, 60%, 62%, 67%)

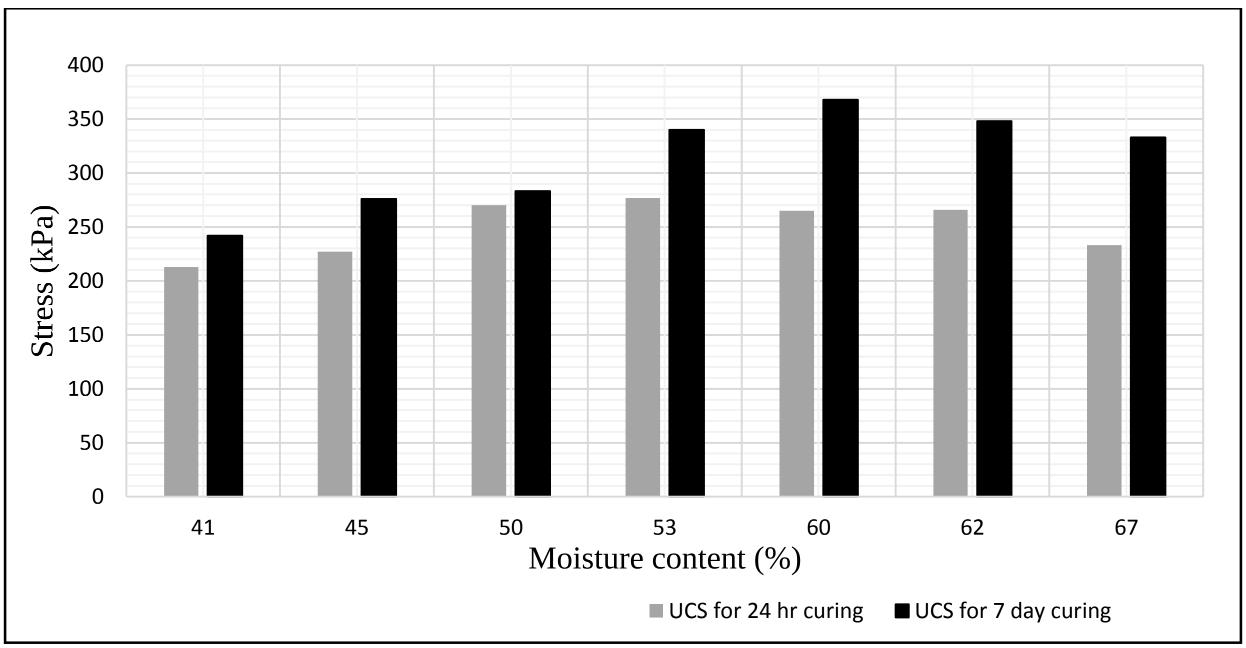

Fig 4.28 UCS result comparison for 24 hrs and 7 days curing

The figures above shows that the specimens prepared on the wet and dry side of OMC provided higher strength for 7 day curing than that of 24 hours curing. As it has been observed in the previous section, higher UCS results are obtained for specimens on the wet side of OMC and cured for 7 days. However, for 24 hour curing, both the wet and dry side of OMC provided lower compressive strength result compared to the specimens prepared at OMC.

In conclusion, molding water higher than OMC and sufficient curing time can contribute to lime stabilization by increasing the unconfined compressive strength.

Chapter Five

Conclusions and Recommendations

This chapter includes conclusions drawn from the laboratory test results discussed in the previous chapter and recommendations that are applicable in projects involving lime stabilization. It also provides recommendations research areas for the future.

5.1. Conclusions

1. Engineering properties of expansive soil can be altered by the addition of small amount of lime. As the amount of lime added to the soil increases, the swelling pressure of expansive soil reduces; the UCS of the soil improves; Liquid limit decreases; Plastic limit increases; Plastic index decreases; OMC increases; MDD decreases.
2. Variation in molding water content has no effect on the swelling pressure, once optimum amount of lime is added to expansive soil.
3. Variation of molding water content has significant effect on unconfined compressive strength of expansive soil. On the wet side of the OMC, the unconfined compressive strength has increased while on the dry it decreased (The best result is obtained at moisture content slightly larger the OMC)

5.2. Recommendation

1. Lime stabilization of expansive soils can be used to solve their swelling property
2. For projects involving lime stabilization, it is important to use moisture content higher than the OMC. This is because even higher compressive strength can be obtained from wet side of OMC than the compressive strength provided at OMC of soil lime mix

5.3. Recommendations for future research

- The research has shown that, as the amount of lime increases, the lower MDD and the higher OMC value gets for constant compactive effort. The MDD can improve at higher compactive effort and the change in the soil property could be studied.
- The influence of molding water content on: lime stabilization of other soil types, stabilization of soil with lime and other stabilizing materials should also be researched
- Effect of molding water content in relation to other factors in lime stabilization, such as type of lime used, curing condition, pulverization quality should be studied.

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APPENDIX

Unconfined Compressive strength of the Soil

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ἑ)	strain (ἑ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	4	0.1	0.001316	0.13158	1133.54	1135.033	53.2	4.68709
20	6	0.2	0.002632	0.26316	1133.54	1136.531	79.8	7.02137
30	15	0.3	0.003947	0.39474	1133.54	1138.032	199.5	17.5303
40	24	0.4	0.005263	0.52632	1133.54	1139.538	319.2	28.0114
50	33	0.5	0.006579	0.65789	1133.54	1141.047	438.9	38.4647
60	39	0.6	0.007895	0.78947	1133.54	1142.56	518.7	45.398
70	45	0.7	0.009211	0.92105	1133.54	1144.078	598.5	52.3129
80	50	0.8	0.010526	1.05263	1133.54	1145.599	665	58.0482
90	56	0.9	0.011842	1.18421	1133.54	1147.124	744.8	64.9276
100	60	1	0.013158	1.31579	1133.54	1148.654	798	69.4726
110	64	1.1	0.014474	1.44737	1133.54	1150.187	851.2	74.0053
120	68	1.2	0.015789	1.57895	1133.54	1151.725	904.4	78.5257
130	75	1.3	0.017105	1.71053	1133.54	1153.267	997.5	86.4934
140	77	1.4	0.018421	1.84211	1133.54	1154.813	1024.1	88.681
150	80	1.5	0.019737	1.97368	1133.54	1156.363	1064	92.0126
160	84	1.6	0.021053	2.10526	1133.54	1157.917	1117.2	96.4836
170	87	1.7	0.022368	2.23684	1133.54	1159.476	1157.1	99.7951
180	92	1.8	0.023684	2.36842	1133.54	1161.038	1223.6	105.388
190	95	1.9	0.025	2.5	1133.54	1162.605	1263.5	108.678
200	100	2	0.026316	2.63158	1133.54	1164.176	1330	114.244
210	104	2.1	0.027632	2.76316	1133.54	1165.752	1383.2	118.653
220	108	2.2	0.028947	2.89474	1133.54	1167.331	1436.4	123.05
230	110	2.3	0.030263	3.02632	1133.54	1168.915	1463	125.159
240	114	2.4	0.031579	3.15789	1133.54	1170.503	1516.2	129.534
250	117	2.5	0.032895	3.28947	1133.54	1172.096	1556.1	132.762
260	121	2.6	0.034211	3.42105	1133.54	1173.693	1609.3	137.114
270	125	2.7	0.035526	3.55263	1133.54	1175.294	1662.5	141.454
280	127	2.8	0.036842	3.68421	1133.54	1176.899	1689.1	143.521
290	133	2.9	0.038158	3.81579	1133.54	1178.509	1768.9	150.096
300	137	3	0.039474	3.94737	1133.54	1180.124	1822.1	154.399
310	139	3.1	0.040789	4.07895	1133.54	1181.743	1848.7	156.438
320	143	3.2	0.042105	4.21053	1133.54	1183.366	1901.9	160.72
330	146	3.3	0.043421	4.34211	1133.54	1184.994	1941.8	163.866
340	150	3.4	0.044737	4.47368	1133.54	1186.626	1995	168.124
350	153	3.5	0.046053	4.60526	1133.54	1188.263	2034.9	171.25
360	157	3.6	0.047368	4.73684	1133.54	1189.904	2088.1	175.485

370	160	3.7	0.048684	4.86842	1133.54	1191.55	2128	178.591
380	163	3.8	0.05	5	1133.54	1193.2	2167.9	181.688
390	166	3.9	0.051316	5.13158	1133.54	1194.855	2207.8	184.776
400	170	4	0.052632	5.26316	1133.54	1196.514	2261	188.966
410	173	4.1	0.053947	5.39474	1133.54	1198.179	2300.9	192.033
420	175	4.2	0.055263	5.52632	1133.54	1199.847	2327.5	193.983
430	178	4.3	0.056579	5.65789	1133.54	1201.521	2367.4	197.034
440	181	4.4	0.057895	5.78947	1133.54	1203.199	2407.3	200.075
450	183	4.5	0.059211	5.92105	1133.54	1204.882	2433.9	202.003
460	186	4.6	0.060526	6.05263	1133.54	1206.569	2473.8	205.028
470	189	4.7	0.061842	6.18421	1133.54	1208.261	2513.7	208.043
480	192	4.8	0.063158	6.31579	1133.54	1209.958	2553.6	211.049
490	195	4.9	0.064474	6.44737	1133.54	1211.66	2593.5	214.045
500	197	5	0.065789	6.57895	1133.54	1213.367	2620.1	215.936
510	198	5.1	0.067105	6.71053	1133.54	1215.078	2633.4	216.727
520	200	5.2	0.068421	6.84211	1133.54	1216.794	2660	218.607
530	203	5.3	0.069737	6.97368	1133.54	1218.515	2699.9	221.573
540	205	5.4	0.071053	7.10526	1133.54	1220.241	2726.5	223.439
550	208	5.5	0.072368	7.23684	1133.54	1221.972	2766.4	226.388
560	210	5.6	0.073684	7.36842	1133.54	1223.708	2793	228.241
570	211	5.7	0.075	7.5	1133.54	1225.449	2806.3	229.002
580	212	5.8	0.076316	7.63158	1133.54	1227.194	2819.6	229.76
590	215	5.9	0.077632	7.76316	1133.54	1228.945	2859.5	232.679
600	216	6	0.078947	7.89474	1133.54	1230.701	2872.8	233.428
610	217	6.1	0.080263	8.02632	1133.54	1232.461	2886.1	234.174
620	218	6.2	0.081579	8.15789	1133.54	1234.227	2899.4	234.916
630	219	6.3	0.082895	8.28947	1133.54	1235.998	2912.7	235.656
640	219	6.4	0.084211	8.42105	1133.54	1237.774	2912.7	235.318
650	219	6.5	0.085526	8.55263	1133.54	1239.555	2912.7	234.98
660	219	6.6	0.086842	8.68421	1133.54	1241.341	2912.7	234.641
670	220	6.7	0.088158	8.81579	1133.54	1243.132	2926	235.373
680	220	6.8	0.089474	8.94737	1133.54	1244.928	2926	235.034
690	220	6.9	0.090789	9.07895	1133.54	1246.73	2926	234.694
700	220	7	0.092105	9.21053	1133.54	1248.537	2926	234.354
710	219	7.1	0.093421	9.34211	1133.54	1250.349	2912.7	232.951
720	217	7.2	0.094737	9.47368	1133.54	1252.166	2886.1	230.489

Unconfined Compressive strength (2% lime + soil mix)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ὲ)	strain (ὲ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	70	0.1	0.001316	0.1315789	1133.54	1135.033	931	82.024
20	83	0.2	0.002632	0.2631579	1133.54	1136.531	1103.9	97.1289
30	105	0.3	0.003947	0.3947368	1133.54	1138.032	1396.5	122.712
40	120	0.4	0.005263	0.5263158	1133.54	1139.538	1596	140.057
50	138	0.5	0.006579	0.6578947	1133.54	1141.047	1835.4	160.852
60	155	0.6	0.007895	0.7894737	1133.54	1142.56	2061.5	180.428
70	168	0.7	0.009211	0.9210526	1133.54	1144.078	2234.4	195.301
80	185	0.8	0.010526	1.0526316	1133.54	1145.599	2460.5	214.778
90	200	0.9	0.011842	1.1842105	1133.54	1147.124	2660	231.884
100	218	1	0.013158	1.3157895	1133.54	1148.654	2899.4	252.417
110	230	1.1	0.014474	1.4473684	1133.54	1150.187	3059	265.957
120	245	1.2	0.015789	1.5789474	1133.54	1151.725	3258.5	282.923
130	266	1.3	0.017105	1.7105263	1133.54	1153.267	3537.8	306.763
140	278	1.4	0.018421	1.8421053	1133.54	1154.813	3697.4	320.173
150	295	1.5	0.019737	1.9736842	1133.54	1156.363	3923.5	339.297
160	310	1.6	0.021053	2.1052632	1133.54	1157.917	4123	356.07
170	330	1.7	0.022368	2.2368421	1133.54	1159.476	4389	378.533
180	340	1.8	0.023684	2.3684211	1133.54	1161.038	4522	389.479
190	351	1.9	0.025	2.5	1133.54	1162.605	4668.3	401.538
200	365	2	0.026316	2.6315789	1133.54	1164.176	4854.5	416.99
210	378	2.1	0.027632	2.7631579	1133.54	1165.752	5027.4	431.258
220	389	2.2	0.028947	2.8947368	1133.54	1167.331	5173.7	443.208
230	400	2.3	0.030263	3.0263158	1133.54	1168.915	5320	455.123
240	410	2.4	0.031579	3.1578947	1133.54	1170.503	5453	465.868
250	421	2.5	0.032895	3.2894737	1133.54	1172.096	5599.3	477.717
260	430	2.6	0.034211	3.4210526	1133.54	1173.693	5719	487.266
270	438	2.7	0.035526	3.5526316	1133.54	1175.294	5825.4	495.655
280	450	2.8	0.036842	3.6842105	1133.54	1176.899	5985	508.54
290	458	2.9	0.038158	3.8157895	1133.54	1178.509	6091.4	516.873
300	465	3	0.039474	3.9473684	1133.54	1180.124	6184.5	524.055
310	467	3.1	0.040789	4.0789474	1133.54	1181.743	6211.1	525.588
320	467	3.2	0.042105	4.2105263	1133.54	1183.366	6211.1	524.867
330	465	3.3	0.043421	4.3421053	1133.54	1184.994	6184.5	521.902
340	462	3.4	0.044737	4.4736842	1133.54	1186.626	6144.6	517.821

Unconfined Compressive Strength (4% lime + Soil Mix)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ἐ)	strain (ἐ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	10	0.1	0.00132	0.131579	1133.54	1135.033	133	11.718
20	18	0.2	0.00263	0.263158	1133.54	1136.531	239.4	21.064
30	27	0.3	0.00395	0.394737	1133.54	1138.032	359.1	31.554
40	40	0.4	0.00526	0.526316	1133.54	1139.538	532	46.686
50	85	0.5	0.00658	0.657895	1133.54	1141.047	1130.5	99.076
60	120	0.6	0.00789	0.789474	1133.54	1142.56	1596	139.69
70	155	0.7	0.00921	0.921053	1133.54	1144.078	2061.5	180.19
80	190	0.8	0.01053	1.052632	1133.54	1145.599	2527	220.58
90	210	0.9	0.01184	1.184211	1133.54	1147.124	2793	243.48
100	243	1	0.01316	1.315789	1133.54	1148.654	3231.9	281.36
110	265	1.1	0.01447	1.447368	1133.54	1150.187	3524.5	306.43
120	290	1.2	0.01579	1.578947	1133.54	1151.725	3857	334.89
130	308	1.3	0.01711	1.710526	1133.54	1153.267	4096.4	355.2
140	320	1.4	0.01842	1.842105	1133.54	1154.813	4256	368.54
150	331	1.5	0.01974	1.973684	1133.54	1156.363	4402.3	380.7
160	336	1.6	0.02105	2.105263	1133.54	1157.917	4468.8	385.93
170	343	1.7	0.02237	2.236842	1133.54	1159.476	4561.9	393.45
180	348	1.8	0.02368	2.368421	1133.54	1161.038	4628.4	398.64
190	349	1.9	0.025	2.5	1133.54	1162.605	4641.7	399.25
200	345	2	0.02632	2.631579	1133.54	1164.176	4588.5	394.14
210	340	2.1	0.02763	2.763158	1133.54	1165.752	4522	387.9
220	320	2.2	0.02895	2.894737	1133.54	1167.331	4256	364.59

Unconfined Compressive Strength (6% lime + Soil Mix)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ἐ)	strain (ἐ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	4	0.1	0.00132	0.13158	1133.54	1135.033	53.2	4.687086
20	10	0.2	0.00263	0.26316	1133.54	1136.531	133	11.70228
30	18	0.3	0.00395	0.39474	1133.54	1138.032	239.4	21.03631
40	25	0.4	0.00526	0.52632	1133.54	1139.538	332.5	29.1785
50	32	0.5	0.00658	0.65789	1133.54	1141.047	425.6	37.29908
60	41	0.6	0.00789	0.78947	1133.54	1142.56	545.3	47.72615
70	47	0.7	0.00921	0.92105	1133.54	1144.078	625.1	54.6379
80	49	0.8	0.01053	1.05263	1133.54	1145.599	651.7	56.88727
90	56	0.9	0.01184	1.18421	1133.54	1147.124	744.8	64.92757

100	60	1	0.01316	1.31579	1133.54	1148.654	798	69.47263
110	65	1.1	0.01447	1.44737	1133.54	1150.187	864.5	75.16166
120	69	1.2	0.01579	1.57895	1133.54	1151.725	917.7	79.68047
130	70	1.3	0.01711	1.71053	1133.54	1153.267	931	80.72719
140	76	1.4	0.01842	1.84211	1133.54	1154.813	1010.8	87.52933
150	81	1.5	0.01974	1.97368	1133.54	1156.363	1077.3	93.16279
160	88	1.6	0.02105	2.10526	1133.54	1157.917	1170.4	101.078
170	96	1.7	0.02237	2.23684	1133.54	1159.476	1276.8	110.1187
180	100	1.8	0.02368	2.36842	1133.54	1161.038	1330	114.5526
190	110	1.9	0.025	2.5	1133.54	1162.605	1463	125.8381
200	118	2	0.02632	2.63158	1133.54	1164.176	1569.4	134.8078
210	125	2.1	0.02763	2.76316	1133.54	1165.752	1662.5	142.6119
220	133	2.2	0.02895	2.89474	1133.54	1167.331	1768.9	151.5337
230	152	2.3	0.03026	3.02632	1133.54	1168.915	2021.6	172.9467
240	168	2.4	0.03158	3.15789	1133.54	1170.503	2234.4	190.8922
250	181	2.5	0.03289	3.28947	1133.54	1172.096	2407.3	205.3842
260	192	2.6	0.03421	3.42105	1133.54	1173.693	2553.6	217.5697
270	208	2.7	0.03553	3.55263	1133.54	1175.294	2766.4	235.3794
280	223	2.8	0.03684	3.68421	1133.54	1176.899	2965.9	252.0096
290	241	2.9	0.03816	3.81579	1133.54	1178.509	3205.3	271.9792
300	268	3	0.03947	3.94737	1133.54	1180.124	3564.4	302.0361
310	285	3.1	0.04079	4.07895	1133.54	1181.743	3790.5	320.7551
320	288	3.2	0.04211	4.21053	1133.54	1183.366	3830.4	323.6869
330	290	3.3	0.04342	4.34211	1133.54	1184.994	3857	325.487
340	295	3.4	0.04474	4.47368	1133.54	1186.626	3923.5	330.6434
350	294	3.5	0.04605	4.60526	1133.54	1188.263	3910.2	329.0687
360	293	3.6	0.04737	4.73684	1133.54	1189.904	3896.9	327.497
370	290	3.7	0.04868	4.86842	1133.54	1191.55	3857	323.6961

Unconfined Compressive strength of the Soil +6% lime, 7 day cure (41% moisture content)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ἐ)	strain (ἐ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	44	0.1	0.001316	0.13158	1133.54	1135.033	590.2174	52
20	94	0.2	0.002632	0.26316	1133.54	1136.531	1250.184	110
30	120	0.3	0.003947	0.39474	1133.54	1138.032	1593.245	140
40	135	0.4	0.005263	0.52632	1133.54	1139.538	1800.469	158
50	156	0.5	0.006579	0.65789	1133.54	1141.047	2076.705	182
60	171	0.6	0.007895	0.78947	1133.54	1142.56	2273.695	199
70	181	0.7	0.009211	0.92105	1133.54	1144.078	2402.563	210
80	194	0.8	0.010526	1.05263	1133.54	1145.599	2577.598	225

90	204	0.9	0.011842	1.18421	1133.54	1147.124	2718.685	237
100	207	1	0.013158	1.31579	1133.54	1148.654	2756.769	240
110	209	1.1	0.014474	1.44737	1133.54	1150.187	2783.454	242
120	204	1.2	0.015789	1.57895	1133.54	1151.725	2706.554	235
130	199	1.3	0.017105	1.71053	1133.54	1153.267	2652.514	230

Unconfined Compressive strength of the Soil +6% lime 7 day cure (45% moisture content)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ἐ)	strain (ἐ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	18	0.1	0.001316	0.13158	1133.54	1135.033	237.4263	20.918
20	34	0.2	0.002632	0.26316	1133.54	1136.531	452.9303	39.852
30	52	0.3	0.003947	0.39474	1133.54	1138.032	690.9221	60.712
40	75	0.4	0.005263	0.52632	1133.54	1139.538	995.4089	87.352
50	97	0.5	0.006579	0.65789	1133.54	1141.047	1295.396	113.53
60	114	0.6	0.007895	0.78947	1133.54	1142.56	1522.027	133.21
70	145	0.7	0.009211	0.92105	1133.54	1144.078	1928.491	168.56
80	149	0.8	0.010526	1.05263	1133.54	1145.599	1979.457	172.79
90	176	0.9	0.011842	1.18421	1133.54	1147.124	2340.799	204.06
100	190	1	0.013158	1.31579	1133.54	1148.654	2526.993	220
110	203	1.1	0.014474	1.44737	1133.54	1150.187	2699.893	234.74
120	215	1.2	0.015789	1.57895	1133.54	1151.725	2859.492	248.28
130	225	1.3	0.017105	1.71053	1133.54	1153.267	2992.497	259.48
140	232	1.4	0.018421	1.84211	1133.54	1154.813	3085.591	267.19
150	237	1.5	0.019737	1.97368	1133.54	1156.363	3152.095	272.59
160	241	1.6	0.021053	2.10526	1133.54	1157.917	3205.3	276.82
170	231	1.7	0.022368	2.23684	1133.54	1159.476	3072.297	264.97
180	230	1.8	0.023684	2.36842	1133.54	1161.038	3055.342	263.16

Unconfined Compressive strength of the Soil +6% lime, 7 day cure (50% moisture content)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ἐ)	strain (ἐ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	12	0.1	0.001316	0.13158	1133.54	1135.033	159.5857	14.06
20	20	0.2	0.002632	0.26316	1133.54	1136.531	265.9937	23.404
30	25	0.3	0.003947	0.39474	1133.54	1138.032	332.4989	29.217
40	33	0.4	0.005263	0.52632	1133.54	1139.538	438.8929	38.515
50	42	0.5	0.006579	0.65789	1133.54	1141.047	558.5425	48.95
60	60	0.6	0.007895	0.78947	1133.54	1142.56	797.9983	69.843
70	70	0.7	0.009211	0.92105	1133.54	1144.078	930.9931	81.375
80	85	0.8	0.010526	1.05263	1133.54	1145.599	1130.5	98.682

90	105	0.9	0.011842	1.18421	1133.54	1147.124	1396.498	121.74
100	130	1	0.013158	1.31579	1133.54	1148.654	1729	150.52
110	156	1.1	0.014474	1.44737	1133.54	1150.187	2074.789	180.39
120	180	1.2	0.015789	1.57895	1133.54	1151.725	2393.999	207.86
130	198	1.3	0.017105	1.71053	1133.54	1153.267	2633.393	228.34
140	210	1.4	0.018421	1.84211	1133.54	1154.813	2792.996	241.86
150	222	1.5	0.019737	1.97368	1133.54	1156.363	2952.599	255.34
160	233	1.6	0.021053	2.10526	1133.54	1157.917	3098.899	267.63
170	243	1.7	0.022368	2.23684	1133.54	1159.476	3231.899	278.74
180	248	1.8	0.023684	2.36842	1133.54	1161.038	3298.394	284.09
190	248	1.8	0.023684	2.36842	1133.54	1161.038	3293.947	283.71
200	249	1.8	0.023684	2.36842	1133.54	1161.038	3316.03	285.61
210	249	1.8	0.023684	2.36842	1133.54	1161.038	3316.03	285.61
220	249	1.8	0.023684	2.36842	1133.54	1161.038	3317.133	285.7
230	250	1.8	0.023684	2.36842	1133.54	1161.038	3320.302	285.98
240	247	1.8	0.023684	2.36842	1133.54	1161.038	3289.372	283.31

Unconfined Compressive strength of the Soil +6% lime, 7 days curing (60% moisture content)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ὲ)	strain (ὲ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	15	0.1	0.001316	0.13158	1133.54	1135.033	199.4935	17.576
20	42	0.2	0.002632	0.26316	1133.54	1136.531	558.5936	49.149
30	68	0.3	0.003947	0.39474	1133.54	1138.032	900.7184	79.147
40	96	0.4	0.005263	0.52632	1133.54	1139.538	1276.795	112.05
50	121	0.5	0.006579	0.65789	1133.54	1141.047	1609.298	141.04
60	162	0.6	0.007895	0.78947	1133.54	1142.56	2154.594	188.58
70	190	0.7	0.009211	0.92105	1133.54	1144.078	2526.993	220.88
80	222	0.8	0.010526	1.05263	1133.54	1145.599	2947.454	257.29
90	237	0.9	0.011842	1.18421	1133.54	1147.124	3152.091	274.78
100	260	1	0.013158	1.31579	1133.54	1148.654	3457.999	301.05
110	284	1.1	0.014474	1.44737	1133.54	1150.187	3777.193	328.4
120	310	1.2	0.015789	1.57895	1133.54	1151.725	4122.957	357.98
130	315	1.3	0.017105	1.71053	1133.54	1153.267	4189.496	363.27
140	320	1.4	0.018421	1.84211	1133.54	1154.813	4255.994	368.54
150	320	1.5	0.019737	1.97368	1133.54	1156.363	4255.994	368.05
160	318	1.6	0.021053	2.10526	1133.54	1157.917	4229.397	365.26
170	310	1.7	0.022368	2.23684	1133.54	1159.476	4122.991	355.59
180	305	1.8	0.023684	2.36842	1133.54	1161.038	4056.494	349.39

Unconfined Compressive strength of the Soil +6% lime, 7 days curing (62% moisture content)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ἐ)	strain (ἐ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	18	0.1	0.001316	0.13158	1133.54	1135.033	239.399	21.092
20	49	0.2	0.002632	0.26316	1133.54	1136.531	651.6982	57.341
30	50	0.3	0.003947	0.39474	1133.54	1138.032	663.6321	58.314
40	94	0.4	0.005263	0.52632	1133.54	1139.538	1250.198	109.71
50	111	0.5	0.006579	0.65789	1133.54	1141.047	1476.298	129.38
60	150	0.6	0.007895	0.78947	1133.54	1142.56	1994.99	174.61
70	181	0.7	0.009211	0.92105	1133.54	1144.078	2401.716	209.93
80	210	0.8	0.010526	1.05263	1133.54	1145.599	2793.05	243.81
90	224	0.9	0.011842	1.18421	1133.54	1147.124	2979.197	259.71
100	244	1	0.013158	1.31579	1133.54	1148.654	3245.2	282.52
110	273	1.1	0.014474	1.44737	1133.54	1150.187	3630.889	315.68
120	296	1.2	0.015789	1.57895	1133.54	1151.725	3936.792	341.82
130	298	1.3	0.017105	1.71053	1133.54	1153.267	3963.398	343.67
140	299	1.4	0.018421	1.84211	1133.54	1154.813	3972.845	344.03
150	300	1.5	0.019737	1.97368	1133.54	1156.363	3989.683	345.02
160	301	1.6	0.021053	2.10526	1133.54	1157.917	4000.72	345.51
170	299	1.7	0.022368	2.23684	1133.54	1159.476	3974.068	342.75

Unconfined Compressive strength of the Soil +6% lime, 7 days curing (67% moisture content)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ἐ)	strain (ἐ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	12	0.1	0.001316	0.13158	1133.54	1135.033	159.5857	14.06
20	35	0.2	0.002632	0.26316	1133.54	1136.531	465.4094	40.95
30	52	0.3	0.003947	0.39474	1133.54	1138.032	691.5822	60.77
40	74	0.4	0.005263	0.52632	1133.54	1139.538	984.1958	86.368
50	97	0.5	0.006579	0.65789	1133.54	1141.047	1295.51	113.54
60	113	0.6	0.007895	0.78947	1133.54	1142.56	1502.889	131.54
70	129	0.7	0.009211	0.92105	1133.54	1144.078	1715.704	149.96
80	154	0.8	0.010526	1.05263	1133.54	1145.599	2048.193	178.79
90	194	0.9	0.011842	1.18421	1133.54	1147.124	2580.192	224.93
100	213	1	0.013158	1.31579	1133.54	1148.654	2832.891	246.63
110	234	1.1	0.014474	1.44737	1133.54	1150.187	3112.189	270.58
120	256	1.2	0.015789	1.57895	1133.54	1151.725	3404.799	295.63
130	277	1.3	0.017105	1.71053	1133.54	1153.267	3684.1	319.45

140	289	1.4	0.018421	1.84211	1133.54	1154.813	3843.691	332.84
150	290	1.5	0.019737	1.97368	1133.54	1156.363	3856.991	333.55
160	290	1.6	0.021053	2.10526	1133.54	1157.917	3856.999	333.1
170	290	1.7	0.022368	2.23684	1133.54	1159.476	3854.746	332.46
180	290	1.8	0.023684	2.36842	1133.54	1161.038	3854.81	332.01

Unconfined Compressive strength of the Soil +6% lime, 7 days curing (53% moisture content, OMC)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ὲ)	strain (ὲ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	12	0.1	0.001316	0.13158	1133.54	1135.033	160.7063	14.159
20	35	0.2	0.002632	0.26316	1133.54	1136.531	469.2959	41.292
30	53	0.3	0.003947	0.39474	1133.54	1138.032	698.2786	61.358
40	75	0.4	0.005263	0.52632	1133.54	1139.538	995.04	87.32
50	99	0.5	0.006579	0.65789	1133.54	1141.047	1311.52	114.94
60	115	0.6	0.007895	0.78947	1133.54	1142.56	1523.479	133.34
70	131	0.7	0.009211	0.92105	1133.54	1144.078	1741.519	152.22
80	157	0.8	0.010526	1.05263	1133.54	1145.599	2081.776	181.72
90	192	0.9	0.011842	1.18421	1133.54	1147.124	2549.576	222.26
100	198	1	0.013158	1.31579	1133.54	1148.654	2629.491	228.92
110	204	1.1	0.014474	1.44737	1133.54	1150.187	2713.959	235.96
120	218	1.2	0.015789	1.57895	1133.54	1151.725	2894.736	251.34
130	239	1.3	0.017105	1.71053	1133.54	1153.267	3184.389	276.12
140	262	1.4	0.018421	1.84211	1133.54	1154.813	3488.458	302.08
150	284	1.5	0.019737	1.97368	1133.54	1156.363	3779.688	326.86
160	297	1.6	0.021053	2.10526	1133.54	1157.917	3948.72	341.02
170	237	1.7	0.022368	2.23684	1133.54	1159.476	3156.777	272.26
180	235	1.8	0.023684	2.36842	1133.54	1161.038	3126.188	269.26

Unconfined Compressive strength of the Soil +6% lime, 24hrs curing (41% moisture content)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ὲ)	strain (ὲ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	39	0.1	0.001316	0.13158	1133.54	1135.033	519.3913	45.76
20	83	0.2	0.002632	0.26316	1133.54	1136.531	1100.162	96.8
30	105	0.3	0.003947	0.39474	1133.54	1138.032	1402.056	123.2
40	119	0.4	0.005263	0.52632	1133.54	1139.538	1584.413	139.04
50	137	0.5	0.006579	0.65789	1133.54	1141.047	1827.501	160.16
60	150	0.6	0.007895	0.78947	1133.54	1142.56	2000.851	175.12
70	159	0.7	0.009211	0.92105	1133.54	1144.078	2114.255	184.8
80	171	0.8	0.010526	1.05263	1133.54	1145.599	2268.286	198

90	180	0.9	0.011842	1.18421	1133.54	1147.124	2392.443	208.56
100	182	1	0.013158	1.31579	1133.54	1148.654	2425.957	211.2
110	184	1.1	0.014474	1.44737	1133.54	1150.187	2449.439	212.96
120	179	1.2	0.015789	1.57895	1133.54	1151.725	2381.768	206.8

Unconfined compressive strength of the soil +6% lime, 24hrs curing (45% moisture content)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ἐ)	strain (ἐ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	16	0.1	0.001316	0.13158	1133.54	1135.033	216.0579	19.035
20	31	0.2	0.002632	0.26316	1133.54	1136.531	412.1666	36.265
30	47	0.3	0.003947	0.39474	1133.54	1138.032	628.7391	55.248
40	68	0.4	0.005263	0.52632	1133.54	1139.538	905.8221	79.49
50	89	0.5	0.006579	0.65789	1133.54	1141.047	1178.811	103.31
60	104	0.6	0.007895	0.78947	1133.54	1142.56	1385.045	121.22
70	125	0.7	0.009211	0.92105	1133.54	1144.078	1666.097	145.63
80	139	0.8	0.010526	1.05263	1133.54	1145.599	1844.265	160.99
90	150	0.9	0.011842	1.18421	1133.54	1147.124	1989.679	173.45
100	161	1	0.013158	1.31579	1133.54	1148.654	2147.944	187
110	173	1.1	0.014474	1.44737	1133.54	1150.187	2294.909	199.52
120	183	1.2	0.015789	1.57895	1133.54	1151.725	2430.568	211.04
130	191	1.3	0.017105	1.71053	1133.54	1153.267	2543.622	220.56
140	197	1.4	0.018421	1.84211	1133.54	1154.813	2622.752	227.11
150	192	1.5	0.019737	1.97368	1133.54	1156.363	2559.702	221.36
160	190	1.6	0.021053	2.10526	1133.54	1157.917	2523.009	217.89

Unconfined compressive strength of the coil +6% lime, 24 hrs curing (50% moisture content)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ἐ)	strain (ἐ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	13	0.1	0.001316	0.13158	1133.54	1135.033	178.736	15.747
20	33	0.2	0.002632	0.26316	1133.54	1136.531	437.8826	38.528
30	58	0.3	0.003947	0.39474	1133.54	1138.032	769.1869	67.589
40	73	0.4	0.005263	0.52632	1133.54	1139.538	975.8795	85.638
50	94	0.5	0.006579	0.65789	1133.54	1141.047	1245.428	109.15
60	112	0.6	0.007895	0.78947	1133.54	1142.56	1493.181	130.69
70	136	0.7	0.009211	0.92105	1133.54	1144.078	1810.099	158.21
80	155	0.8	0.010526	1.05263	1133.54	1145.599	2056.596	179.52
90	169	0.9	0.011842	1.18421	1133.54	1147.124	2243.631	195.59
100	175	1	0.013158	1.31579	1133.54	1148.654	2323.895	202.31
110	182	1.1	0.014474	1.44737	1133.54	1150.187	2417.864	210.21

120	188	1.2	0.015789	1.57895	1133.54	1151.725	2494.461	216.58
130	198	1.3	0.017105	1.71053	1133.54	1153.267	2635.538	228.53
140	204	1.4	0.018421	1.84211	1133.54	1154.813	2718.081	235.37
150	211	1.5	0.019737	1.97368	1133.54	1156.363	2804.969	242.57
160	221	1.6	0.021053	2.10526	1133.54	1157.917	2943.954	254.25
170	231	1.7	0.022368	2.23684	1133.54	1159.476	3070.304	264.8
180	236	1.8	0.023684	2.36842	1133.54	1161.038	3133.474	269.89
190	236	1.8	0.023684	2.36842	1133.54	1161.038	3140.279	270.47
200	234	1.8	0.023684	2.36842	1133.54	1161.038	3106.762	267.58

Unconfined compressive strength of the soil +6% lime, 24hrs curing (60% moisture content)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ἐ)	strain (ἐ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	11	0.1	0.001316	0.13158	1133.54	1135.033	143.6353	12.655
20	30	0.2	0.002632	0.26316	1133.54	1136.531	402.1874	35.387
30	49	0.3	0.003947	0.39474	1133.54	1138.032	648.5172	56.986
40	69	0.4	0.005263	0.52632	1133.54	1139.538	919.2923	80.672
50	87	0.5	0.006579	0.65789	1133.54	1141.047	1158.695	101.55
60	117	0.6	0.007895	0.78947	1133.54	1142.56	1551.308	135.77
70	137	0.7	0.009211	0.92105	1133.54	1144.078	1819.435	159.03
80	160	0.8	0.010526	1.05263	1133.54	1145.599	2122.167	185.25
90	171	0.9	0.011842	1.18421	1133.54	1147.124	2269.506	197.84
100	187	1	0.013158	1.31579	1133.54	1148.654	2489.76	216.75
110	204	1.1	0.014474	1.44737	1133.54	1150.187	2719.579	236.45
120	223	1.2	0.015789	1.57895	1133.54	1151.725	2968.529	257.75
130	227	1.3	0.017105	1.71053	1133.54	1153.267	3016.437	261.56
140	230	1.4	0.018421	1.84211	1133.54	1154.813	3064.315	265.35
150	230	1.5	0.019737	1.97368	1133.54	1156.363	3064.316	265
160	229	1.6	0.021053	2.10526	1133.54	1157.917	3045.166	262.99
170	223	1.7	0.022368	2.23684	1133.54	1159.476	2968.554	256.03
180	220	1.8	0.023684	2.36842	1133.54	1161.038	2920.675	251.56

Unconfined compressive strength of the soil +6% lime, 24hrs curing (62% moisture content)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ἐ)	strain (ἐ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	14	0.1	0.001316	0.13158	1133.54	1135.033	186.7312	16.452
20	38	0.2	0.002632	0.26316	1133.54	1136.531	508.3246	44.726
30	56	0.3	0.003947	0.39474	1133.54	1138.032	744.4162	65.413

40	73	0.4	0.005263	0.52632	1133.54	1139.538	975.1545	85.575
50	87	0.5	0.006579	0.65789	1133.54	1141.047	1151.512	100.92
60	117	0.6	0.007895	0.78947	1133.54	1142.56	1556.092	136.19
70	141	0.7	0.009211	0.92105	1133.54	1144.078	1873.339	163.74
80	164	0.8	0.010526	1.05263	1133.54	1145.599	2178.579	190.17
90	175	0.9	0.011842	1.18421	1133.54	1147.124	2323.773	202.57
100	190	1	0.013158	1.31579	1133.54	1148.654	2531.256	220.37
110	213	1.1	0.014474	1.44737	1133.54	1150.187	2832.093	246.23
120	231	1.2	0.015789	1.57895	1133.54	1151.725	3070.698	266.62
130	228	1.3	0.017105	1.71053	1133.54	1153.267	3036.071	263.26
140	228	1.4	0.018421	1.84211	1133.54	1154.813	3027.311	262.15

Unconfined compressive strength of the soil +6% lime, 24 hrs curing (67% moisture content)

Deformn dial reading	Load dial reading	sample deformation ΔL	strain (ἐ)	strain (ἐ) (%)	Area (Ao) mm^2	Corrected Area A' mm^2	Load (kN)	Stress (kPa)
0	0	0	0	0	1133.54	1133.54	0	0
10	8	0.1	0.001316	0.13158	1133.54	1135.033	111.71	9.842
20	24	0.2	0.002632	0.26316	1133.54	1136.531	325.7866	28.665
30	36	0.3	0.003947	0.39474	1133.54	1138.032	484.1075	42.539
40	52	0.4	0.005263	0.52632	1133.54	1139.538	688.9371	60.458
50	68	0.5	0.006579	0.65789	1133.54	1141.047	906.8573	79.476
60	79	0.6	0.007895	0.78947	1133.54	1142.56	1052.023	92.076
70	90	0.7	0.009211	0.92105	1133.54	1144.078	1200.993	104.97
80	108	0.8	0.010526	1.05263	1133.54	1145.599	1433.735	125.15
90	136	0.9	0.011842	1.18421	1133.54	1147.124	1806.135	157.45
100	149	1	0.013158	1.31579	1133.54	1148.654	1983.023	172.64
110	164	1.1	0.014474	1.44737	1133.54	1150.187	2178.532	189.41
120	179	1.2	0.015789	1.57895	1133.54	1151.725	2383.359	206.94
130	194	1.3	0.017105	1.71053	1133.54	1153.267	2578.87	223.61
140	202	1.4	0.018421	1.84211	1133.54	1154.813	2690.583	232.99
150	200	1.5	0.019737	1.97368	1133.54	1156.363	2662.977	230.29
160	197	1.6	0.021053	2.10526	1133.54	1157.917	2623.606	226.58
170	192	1.7	0.022368	2.23684	1133.54	1159.476	2553.799	220.25

Compaction test result of the soil

Trial		1	2	3	4	5	6	
Wt of mold + wet soil	g	4087	4202	4295	4463	4509	4422	
Wt of mold	g	2854	2854	2854	2854	2854	2854	
Wt. Wet soil	g	1233	1348	1441	1609	1655	1568	
Volume of mold	cm^3	944	944	944	944	944	944	
Wet density	g/cm^3	1.31	1.43	1.53	1.70	1.75	1.66	
								I.M.C
Container No.		109	A26	123	2	A18	87	A43
Wt. Cont. + wet soil	g	131	136	106	99	133	151	113
Wt. Cont. + Dry soil	g	118	115	87	78	98	105	106
Weight of water	g	13	21	19	21	35	46	7
Weight of container	g	16	16	16	16	16	16	16
Weight of Dry soil	g	102	99	71	62	82	89	90
Moisture content	%	12.75	21.21	26.76	33.87	42.683	51.6854	7.77778
Dry Density	g/cm^3	1.158	1.178	1.204	1.273	1.2287	1.09504	

Compaction test result of soil + 2% lime

Trial		1	2	3	4	5
Wt of mold + wet soil	g	5940.7	6017.6	6092.4	6136.5	6065
Wt of mold	g	4573.2	4573.2	4573.2	4573.2	4573.2
Wt. Wet soil	g	1367.5	1444.4	1519.2	1563.3	1491.8
Volume of mold	cm^3	944	944	944	944	944
Wet density	g/cm^3	1.45	1.53	1.61	1.66	1.58
Container No.		F6	S32	S1	A21	G7
Wt. Cont + wet soil	g	114.5	125.3	110.7	119.3	129.6
Wt. Cont + Dry soil	g	96.5	101.9	90.6	94.1	99.5
Weight of water	g	18	23.4	20.1	25.2	30.1
Weight of container	g	33.3	32.6	33.2	32.8	33.1
Weight of Dry soil	g	63.2	69.3	57.4	61.3	66.4
Moisture content	%	28.48	33.64	35.02	41.11	45.33
Dry Density	g/cm^3	1.1275	1.1449	1.1919	1.1736	1.0874

Compaction test result of soil + 4% lime

Trial		1	2	3	4
Wt of mold + wet soil	g	5931.8	6050.6	6166	6154.6
Wt of mold	g	4573.2	4573.2	4573.2	4573.2
Wt. Wet soil	g	1358.6	1477.4	1592.8	1581.4
Volume of mold	cm^3	944	944	944	944
Wet density	g/cm^3	1.44	1.57	1.69	1.68

Container No.					
Wt. Cont + wet soil	g	156	162.7	153	138.3
Wt. Cont + Dry soil	g	130.6	134.4	116.9	104.7
Weight of water	g	25.4	28.3	36.1	33.6
Weight of container	g	33.3	33.2	33.1	33.6
Weight of Dry soil	g	97.3	101.2	83.8	71.1

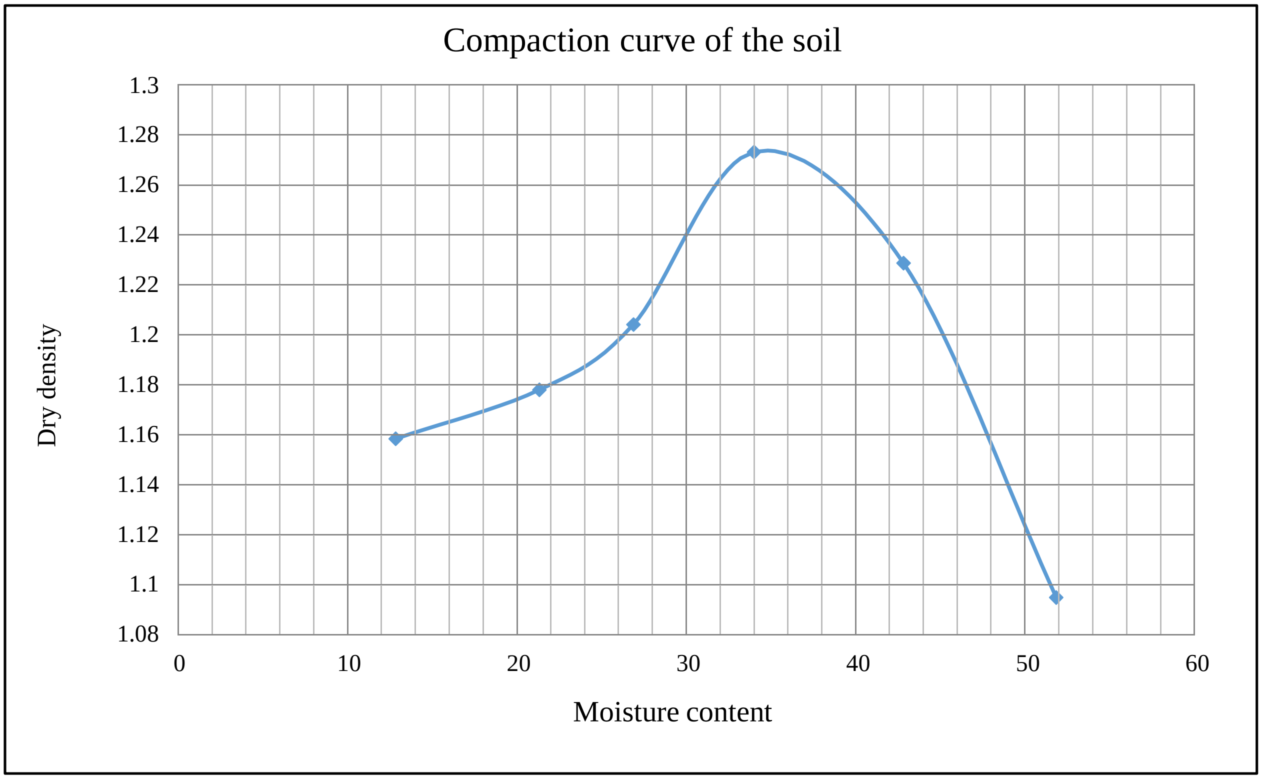
Moisture content	%	26.10	33.64	43.08	47.26
Dry Density	g/cm^3	1.1413	1.1711	1.1793	1.1376

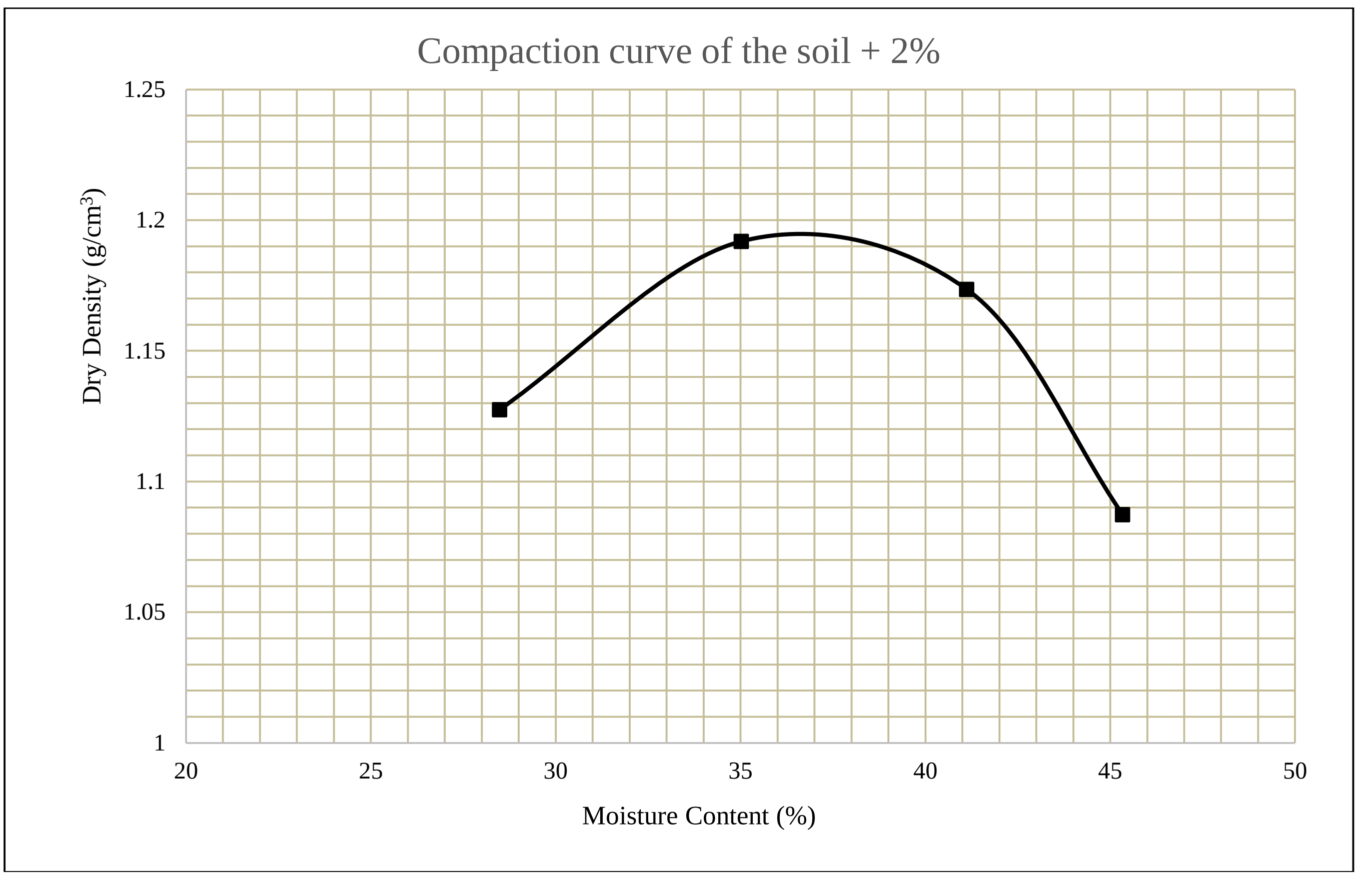
Compaction test result of soil + 6% lime

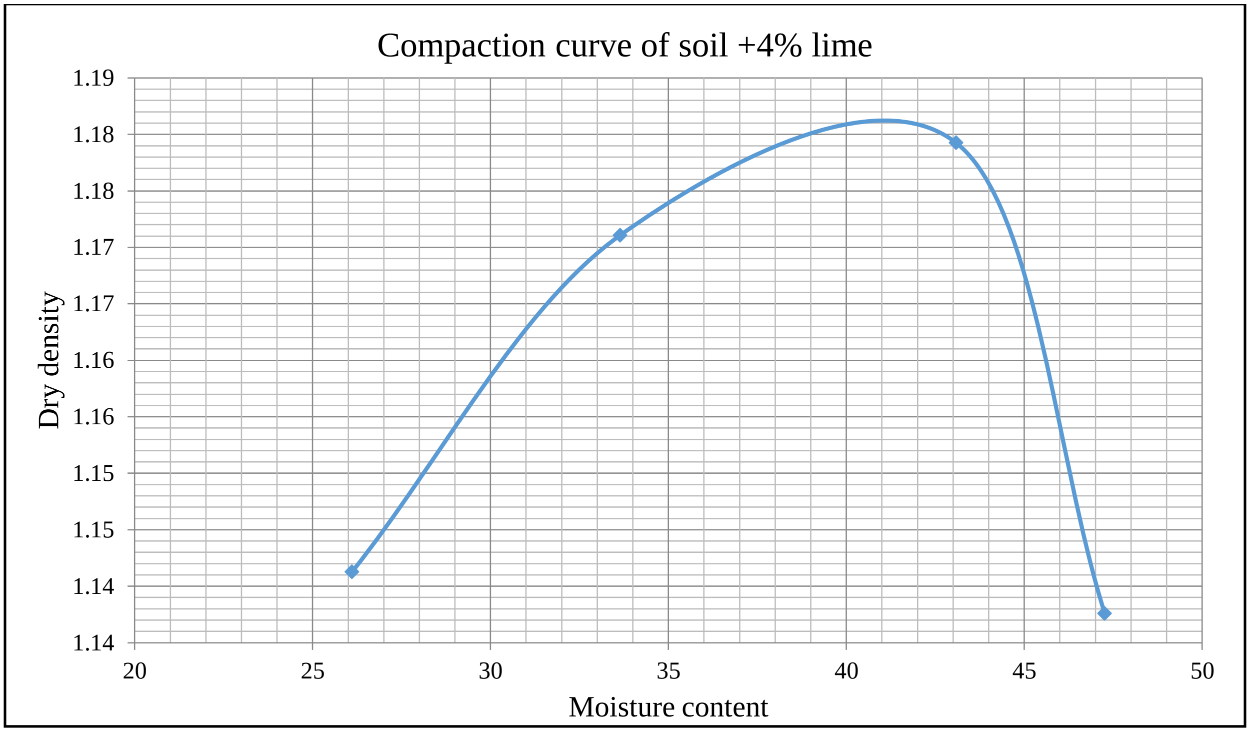
Trial		1	2	3	4	5
Wt of mold + wet soil	g	5903.2	6003.5	6071	6098	6096
Wt of mold	g	4573.2	4573.2	4573.2	4573.2	4574.2
Wt. Wet soil	g	1330	1430.3	1497.8	1524.8	1521.8
Mold Vol.	cm^3	944	944	944	944	945
Wet density	g/cm^3	1.41	1.52	1.59	1.62	1.61

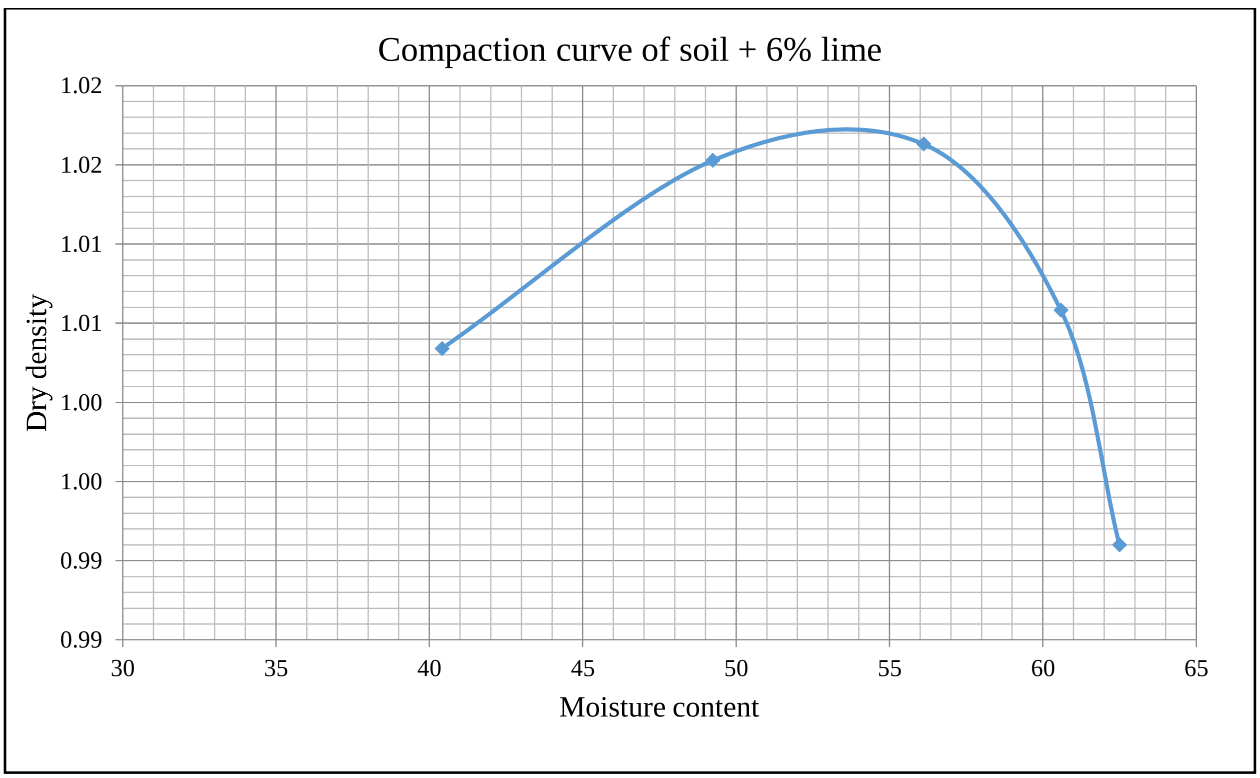
Container No.		B12	H8	A23	T6	D3
Wt. Cont + wet soil	g	107.3	121.8	153	142	136
Wt. Cont + Dry soil	g	85.8	92.8	109.9	101.1	97
Weight of water	g	21.5	29	43.1	40.9	39
Weight of container	g	32.6	33.9	33.1	33.6	34.6
Weight of Dry soil	g	53.2	58.9	76.8	67.5	62.4

Moisture content	%	40.41	49.24	56.12	60.59	62.50
Dry Density	g/cm^3	1.0034	1.0153	1.0163	1.0058	0.9910


Compaction curve of the soil
Dry density
Moisture content
1.3
1.28
1.26
1.24
1.22
1.2
1.18
1.16
1.14
1.12
1.1
1.08
0
10
20
30
40
50
60


Compaction curve of the soil + 2%
Dry Density (g/cm3)
Moisture Content (%)
1.25
1.2
1.15
1.1
1.05
1
20
25
30
35
40
45
50


Compaction curve of soil +4% lime
Dry density
1.19
1.18
1.18
1.17
1.17
1.16
1.16
1.15
1.15
1.14
1.14
20
25
30
35
40
45
50
Moisture content


Compaction curve of soil + 6% lime
Dry density
1.02
1.02
1.01
1.01
1.00
1.00
0.99
0.99
30
35
40
45
50
55
60
65
Moisture content

Swelling pressure of the soil

A) In the beginning of the test

Sample Type = Remolded

Ring area, cm^2= 19.625

Height of sample = 17

setting load, kPa = 7

Initial Void ratio =

Initial moisture content (%).= 33

Specific gravity= 2.71

wet density(g/cm^3) = 1.7

B) In the end of the test

Wt of wet soil+ ring= 126.5

Weight of ring = 54.2

Wet specimen Wt = 72.3

Dry specimen wt= 52.5

Dry specimen density= 1.23

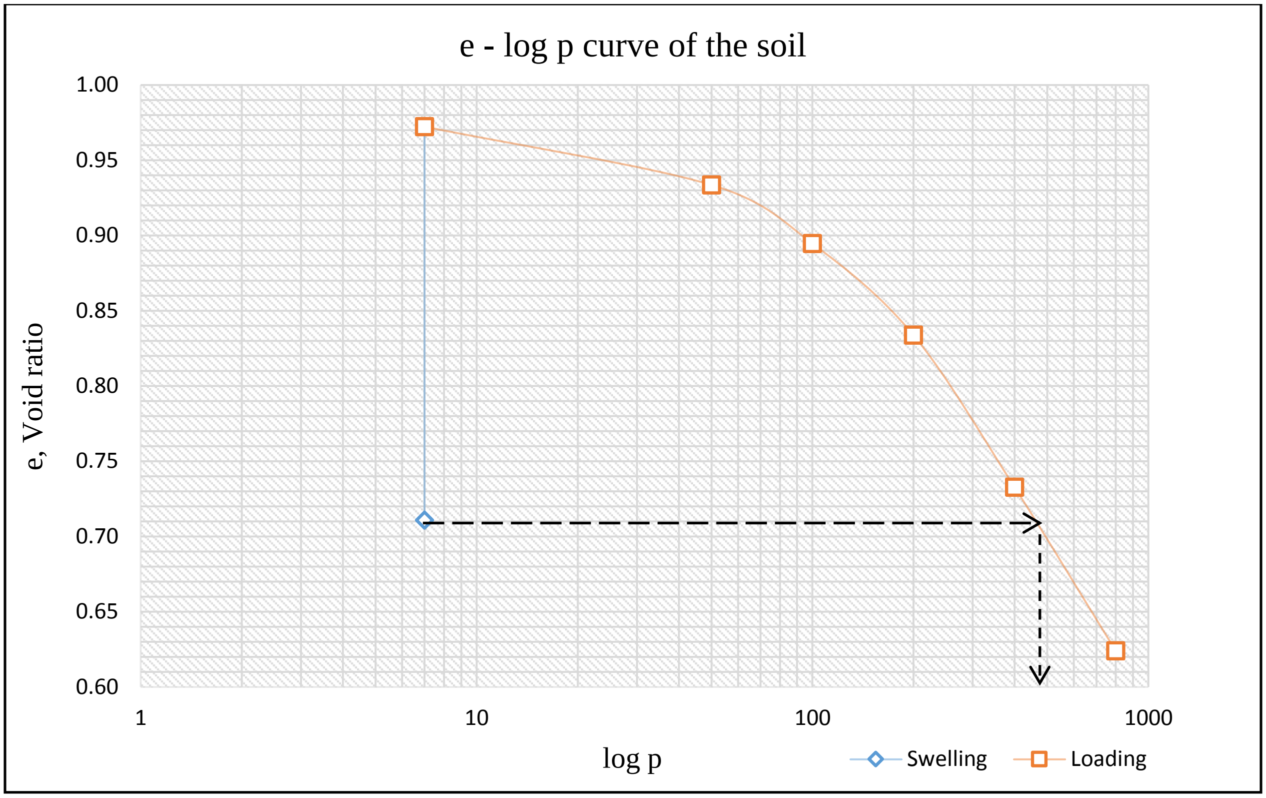
Hs (mm)= 9.87

Final Void ratio= 0.61

$$H_s = \frac{M_s}{G*d_w*A}$$

$$e = \frac{H - H_s}{H_s}$$

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	13.940	0.02	17.00	7.13	0.71
7	16.581	-2.62	19.62	9.75	0.97
50	16.193	-2.23	19.23	9.36	0.93
100	15.802	-1.84	18.84	8.97	0.89
200	15.192	-1.23	18.23	8.36	0.83
400	14.179	-0.22	17.22	7.35	0.73
800	13.090	0.87	16.13	6.26	0.62


e - log p curve of the soil
e, Void ratio
1.00
0.95
0.90
0.85
0.80
0.75
0.70
0.65
0.60
1
10
100
1000
log p
Swelling
Loading

Swelling pressure of the soil + 2% lime

A) In the beginning of the test

Sample Type = Remolded

Ring area, cm^2= 19.625

Height of sample = 17

setting load, kPa = 7

Initial Void ratio =

Initial moisture content (%).= 35

Specific gravity= 2.71

wet density(g/cm^3) = 1.61

B) In the end of the test

Wt of wet soil+ ring= 128.5

Weight of ring = 54.2

Wet specimen Wt = 72.3

Dry specimen wt= 51.5

Dry specimen density= 1.19

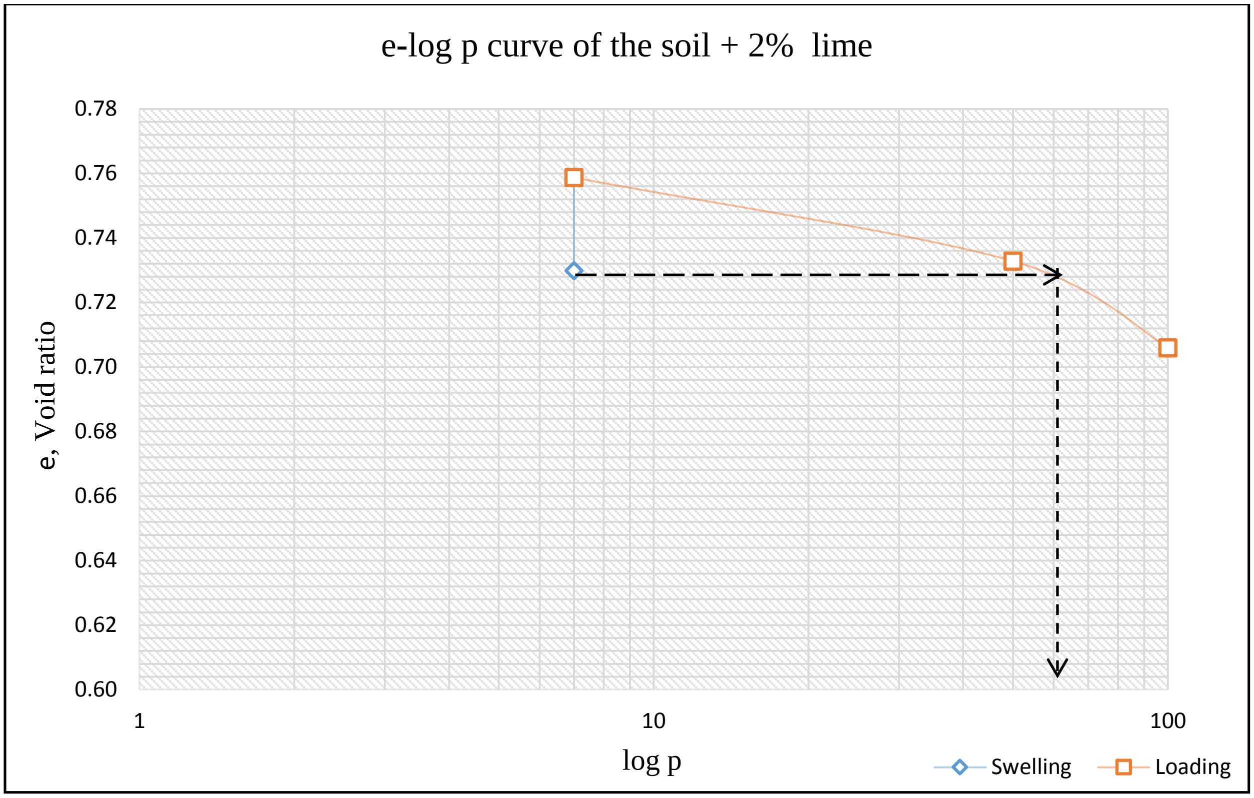
Hs (mm)= 9.68

Final Void ratio= 0.61

$$H_s = \frac{M_s}{G*d_w*A}$$

$$e = \frac{H - H_s}{H_s}$$

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	12.800	0.02	17.00	7.32	0.73
7	13.110	-0.29	17.29	7.61	0.76
50	12.850	-0.03	17.03	7.35	0.73
100	12.580	0.24	16.76	7.08	0.71


e-log p curve of the soil + 2% lime
e, Void ratio
0.78
0.76
0.74
0.72
0.70
0.68
0.66
0.64
0.62
0.60
1
10
100
log p
Swelling
Loading

Swelling Pressure of soil +4% lime

A) In the beginning of the test

Sample Type = Remolded

Ring area, cm^2= 19.625

Height of sample = 1.8

setting load, kPa = 7

Initial Void ratio =

Initial moisture content (%).= 34.5

Specific gravity= 2.71

Weight of sample and ring= 111

wet density(g/cm^3) = 1.59943383

Dry Density (g/cm^3)= 1.18917013

B) In the end of the test

Wt of wet soil+ ring= 119

Weight of ring = 54.5

Wet specimen Wt = 72.3

Dry specimen wt= 45.7

Dry specimen density= 1.17

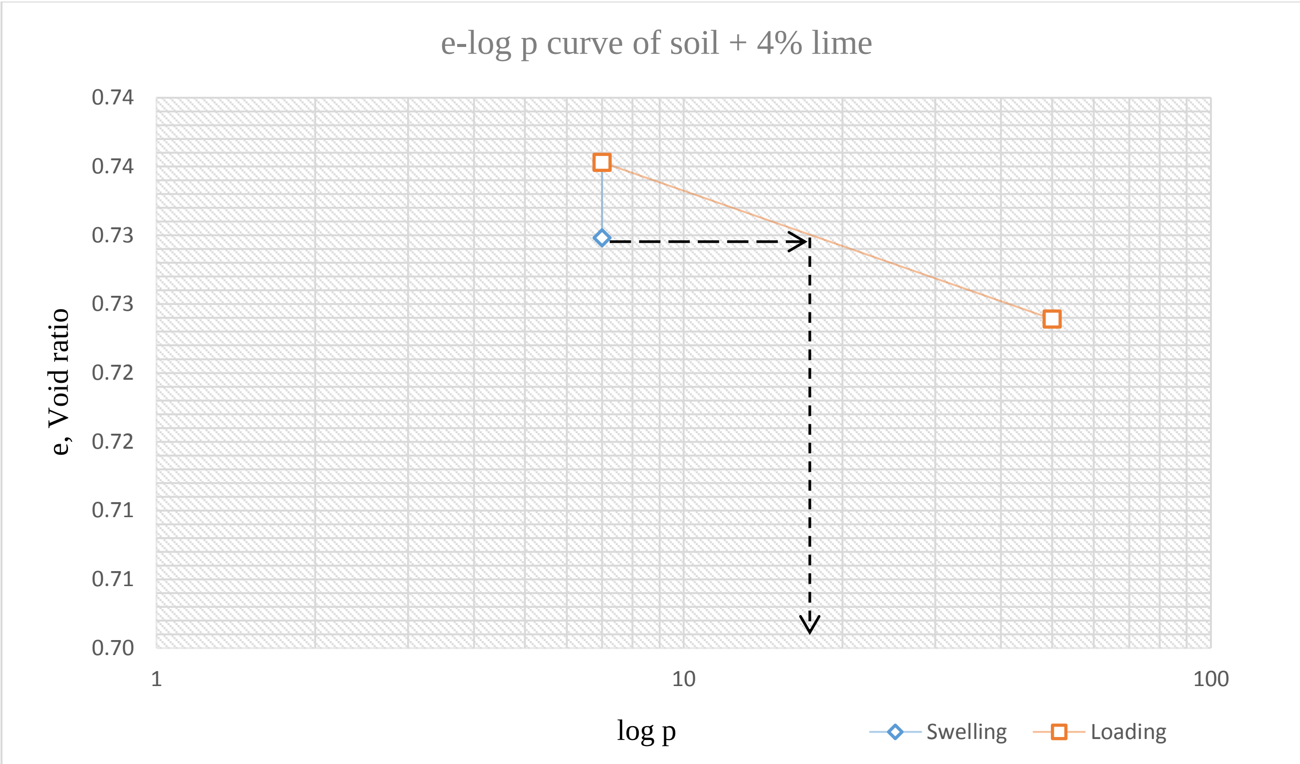
Hs (mm)= 8.59

Final Void ratio= 0.61

$$H_s = \frac{M_s}{G*d_w*A}$$

$$e = \frac{H - H_s}{H_s}$$

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height, H_v (mm)	Void Ratio, E
Loading					
7	13.800	0.02	17.00	7.32	0.73
7	13.875	-0.05	17.06	7.38	0.74
50	13.761	0.06	16.94	7.26	0.72


e-log p curve of soil + 4% lime
0.74
0.74
0.73
0.73
0.72
0.72
0.71
0.71
0.70
e, Void ratio
1
10
100
log p
Swelling
Loading