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SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

GEOTECHNICAL ENGINEERING SREAM

Comparing the Engineering Performance of Cement and Lime Treated Expansive Soil Located in Asen-Dabo town

A Final Thesis Submitted to the School Of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Civil Engineering (Geotechnical Engineering)

By: Hayelom Tsegay Mehari

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Declaration

This Thesis is my original work and has not been presented for degree in any other University

Hayelom Tsegay	_______________	______________
Name	Signature	Date

Acknowledgment

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Symbols and Abbreviations

UCS	-	unconfined compression strength
MDD	-	maximum dry density
PI	-	plasticity index
LI	-	liquidly index
LL	-	liquid limit
FSI	-	Free swell index
EI	-	expansion Index
AASHTO		-American association of highway and transportation officials
ASTM	-	American society for testing materials
BS	-	Britain standard for material testing
DTA	-	differential thermal analysis
CEC	-	Cation exchange capacity
SSA	-	specific surface area
TP	-	total potassium
NLA	-	National lime association
σ_{max}	-	maximum axial stress
ϵ	-	Strain
GI	-	group index
Gs	-	Specific gravity
Cc	-	centimeter cube
g	-	Gram
Kg	-	kilogram

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Abstract

Expansive soils have the tendency to behave volume change during wetting and drying scenario. This behavior of the soil cause large uplift pressure, differential settlement, and upheaval of structures built on them. Avoiding these types of unsuitable soils is mostly impractical, hence stabilization practices are considered to reduce the pre-specified behavior of the soils.

The aims of the study are (a) characterize the engineering properties of expansive soils in Asen-Dabo town, (b) evaluate the effect of lime and cement addition on some engineering property of expansive soil, and (c) evaluate the effect of curing duration on treatment of expansive soil with lime and cement.

Among the known mechanisms to treat the expansive soils, chemical stabilization is one. In this study soil from expansive site, expansive soil of Asen-Dabo was treated chemically with hydrated lime and Portland cement to understand and compare the engineering performance of the soil. Lime and cement content added in variation of percentages respectively 2%, 4%, 6%, and 8%. Unconfined compressive strength of the expansive soil was done for both immediate and seven day cured specimens and the resulting values were compared.

Both hydrated lime and Portland cement addition brought the same positive effect towards treating the soil. Both chemical stabilizers decreased the liquid limit, plastic index, free swell percentage, linear shrinkage, maximum dry density (MDD), and strain. The plastic limit, unconfined compressive strength, optimum moisture content (OMC), and California bearing ratio of the expansive soil was observed to increase with lime and cement percentage increment. Finally those who are involving in construction industries in the study area can use lime and cement to reduce the effect of expansiveness of the soil.

Key words: **cement stabilization, curing duration, expansion potential, expansive soil, lime stabilization**

1 Introduction

1.1 General

Construction on expansive soil always creates a problem for civil engineers because of its peculiar cyclic swell shrink behavior. This types of soil swells when it comes in contact with water and shrinks when the water evaporates out. Because of this movement lightly loaded structures such as foundations, pavements, canal beds, and linings and residential buildings founded on them are seriously damaged (Chen, 1988).

In the field of Civil Engineering nearly all projects are built in to the ground. Thus any construction on soil such as embankment, tunnel, and dam earth retaining structures, residential or any industrial and commercial buildings, road, airport pavements. needs a foundation with sufficient bearing capacity. In addition this sufficient bearing capacity of the soil upon which civil engineering structures lie, needs not to vary conditionally. Infrastructures and constructions which are founded on soils in which their strength changes conditionally are mostly susceptible to failure which may rang to different extent (John et al., 2015).

Such type of soil needs either replacement with other suitable soil, or treatment with suitable mechanism to acquire enough bearing capacity and strength to support the load imposed upon it. Soil strength generally refers the soil ability to support the load imposed by building or structures perfectly without failure. Treated expansive soil behave differently with different load resulting in varying degrees of initial strength gain and final strength development to support foundation for building purposes (Chen, 1975).

Interested costs associated with the use of high quality materials have led to the need for local soils to be used in geotechnical engineering structures construction, often however, higher water content and low workability of these soils pose difficulties for construction. Frequently additives such as lime, cement, flay ash, lime- cement-fly ash admixtures, geofiber, and polymer stabilizers are used to improve engineering properties. The choice and effectiveness of an additive depends on the type of soil and its field conditions. Hence knowledge of mechanistic behavior of the soil to be treated is equally important as selecting the stabilizer.

1.2 Statement of the Problem

The expansive soils problem appears as cracking and demolition of different infrastructures which are founded upon the soil. This can be seen visually on different pavements, highway embankments, buildings, tunnels, and dam (Musema, 2014). Swelling, shrinkage, low bearing capacity, and volumetric change with respect to water content are Particular problems associated with expansive soil. Upon expansion and swelling the soil exerts destructive pressure on foundations and structures founded on it. If this pressure is greater than the foundation pressure, then uplift or differential uplift can occur causing walls, beams, and columns to crack (Basma et al., 1995).

. One of the site which have faced the expansive soil problem is the study area (Asen-Dabo). In this town different pavements including the trunk asphalt road from Addis Ababa through the town to Jimma city has cracked. There was significant and visually identifiable differential settlement on the asphalt road which have been rehabilitated recently. Different buildings including the recently built youth recreation center which is located at kebele 03 has already cracked and is needing rehabilitation.

It is common to see residential and commercial buildings with problems more than cracking which are reached almost at desuetude stage, even though they are not high rise buildings. The prescribed problem on the town occurs due to not mitigating the soil to control the volume change. To reduce the negative impact of expansive soil, improving its engineering property is needed. Among the mechanisms used to improve the engineering performance of the expansive soil, chemical stabilization is the one. This research is intended to evaluate and compare some of the improved engineering properties of the town’s expansive soil, such as index properties, density-moisture content relation and strength at its natural and stabilized state.

1.3 Research Questions

Questions that would be answered though the research findings are;

- ✓ What is the engineering properties of soils in Asen-Dabo town?
- ✓ How the engineering performance of expansive soil behave, as we use lime and cement as stabilization agent?
- ✓ What is the effect of curing duration on cement and lime treated expansive soil?

1.4 Objectives

1.4.1 General Objective

The general objective of this research is to compare the engineering performance of cement and lime treated expansive soil located in Asen-Dabo town.

1.4.2 Specific Objectives

To characterize some of the engineering properties of expansive soil in Asen-Dabo.

To evaluate the effect of lime and cement addition on some engineering properties of expansive soil

To evaluate the effect of curing duration on treatment of expansive soil with lime and cement.

1.5 Scope of the Study

This research is restricted to the evaluating and comparing the index properties, free swell, unconfined compression strength, density moisture relation and CBR value of lime or cement treated expansive soil sample from the study area. Due to time limitation and lack of laboratory materials it is difficult to study full engineering property of the soil treated with the stabilizers. Soil sample from the study area was taken from eight pits which had a depth ranging from 1.5m to 2m to identify the location with higher expansion potential and detail investigation have done for the two pits. Therefore, findings of this research should be considered as indicative rather than definitive during practical practice on civil engineering construction.

1.6 Research Organization

This research contains six chapters. The first chapter explains the buck ground of the research and explains why the research have been conducted. The second chapter introduces detail literature review about expansive soil and its stabilization mechanisms. Chapter three explains study area, research methodology, material and type of testes proposed to be conducted. Detail laboratory test results and findings are briefly tabulated in chapter four. Detail analysis of test results, discussion and evaluation of the findings were done in chapter five. Chapter six contains both conclusion and recommendations. Appendix at the buck presents detail laboratory readings, calculations with detail tabular and graphical interpretations.

2 Literature Review

2.1 Introduction

Expansive soils have a complicated behavior and are generally characterized by detrimental volume changes when subjected to moisture fluctuations (Khemissa and Mahamedi, 2014). At dry state, the expansive soils are very difficult to compact since their consistency varies from hard to very hard. At wet state, they are very sticking (Mahamedi, 2014).

Expansive clays are clay soils with high plasticity. In dry stat the soil exhibit high bearing capacity which is gradually lost with increase in moisture content. If such type of soil is protected from swelling following exposure to moisture, it exert high swelling pressure. The pressure build up is usually responsible for cracking of buildings, distortion of pavement surfaces and damage to other structures (Tefera, and Leikun, 1999).

On drying the soil crack very badly. In some cases the crack are seen to as deeper 1.5m. Excavated vertical banks in those soils stand so long as the moisture content doesn’t change. Expansive drying makes the soil crumble along a crack lines and fall in to excavated area. Problems due to expansive soil in Ethiopia were not well recognized for many years. This was for a reason that the modern small masonry or brick houses were built on a location that doesn’t Couse foundation problem. But in the last 30 years, however, residential buildings were start to build in a location which covers with highly expansive soil (Tefera, and Leikun, 1999).

In practice the problem associated with swelling (expansive) soils contribute to the establishment and development of various techniques for improving their low engineering performance (celik and nalbantoglu, 2013; turkoz et al., 2014; Phanikumar et al., 2015). Among the potential methods likely to improve the engineering performance of such soils, the chemical treatment is an effective technique introduced many years ago. Different pervious works indicates that using lime and cement can be utilized to overcoming deficiencies in the performance of expansive soils (seco et al., 2011; mahmedi and khemissa, 2015).

Lime or cement stabilization of soil is widely used to reduce soil plasticity, mitigate heave, and increase subgrade stiffness and strength. During chemical treatment the change in engineering performance of the soil is changed due to complex chemical reactions which takes place between the soil, water and admixtures (Charles Lucian, 2012).

2.2 Distribution of Expansive Soil

Swelling clays are found in many parts of the world, particularly in semi-arid areas. Swelling clays are detected in Australia, Canada, China, Israel, Jordan, Saudi Arabia, India, South Africa, Sudan, Ethiopia, Spain, and the United States. This is not to say that such soils do not exist elsewhere, for, indeed, they can be found almost everywhere (Chen, 1988).

2.3 Origin of Expansive Soils

Parent materials that can be associated with expansive soils are either igneous rocks or sedimentary rocks. The basic igneous rocks comprises basalt, dolerite, sills, dykes, and gabbros. The second includes the sedimentary rocks that contain Montmorillonite as a constituent which breaks down physically to form expansive soils (Donaldson, G. W., 1969)

The Montmorillonite was probably formed from two separate origins. The product of weathering and erosion of rocks in the highlands were carried to the streams by coastal planes. The fine grained soil eventually accumulating in the ocean basin. The second case in which Montmorillonite is formed could be, volcanic eruption, sending up clouds of ash, fall on the plains and sees and through process thus ashes were altered to Montmorillonite (Chen, 1975).

2.4 Clay Mineralogy

Most soil classification system defines a clay particles as having an effective diameter of 0.002mm.But particle diameter alone does not determine clay minerals (Peck et al., 1974). Absorption of water by clays leads to expansion. The water observation capacity of the clay depends up on the kind and amount of clay mineral present, their exchangeable ions, electrolyte content of aqueous, and the internal structure. Hence the most important grain property of fine grained soil is its mineralogical composition (Chen, 1975).

Clay minerals have sheet or layers structure and can have various shapes. Atypical clay particle of expansive soil consists microscopic platlate having negative electrical charge on its flat surface and positive electrical charges on its edges (Grim, 1959).

The two basic elemental units of the building blocks are the silicon tetrahedron and the alumino-magnesium octahedron. Silicon tetrahedron is made up of silicon and oxygen atoms. Because of the valence of silicon is +4, it can bond with negatively charged ions or hydroxyl to form the shape of tetrahedron. In silicon tetrahedron the oxygen atoms each have an unsatisfied chemical bond.

The oxygen atoms at the base of tetrahedron are shared with adjacent tetrahedra and the resulting arrangement of tetrahedra forms sheet like structure as shown on the fig blew. The alumino magnesium octahedron consists of aluminum or magnesium atoms surrounded by hydroxyl. The atoms are arranged to form octahedral shape. On octahedral units the sheet structure does not have unsatisfied chemical bond and it looks like the fig below (Grim, 1959).

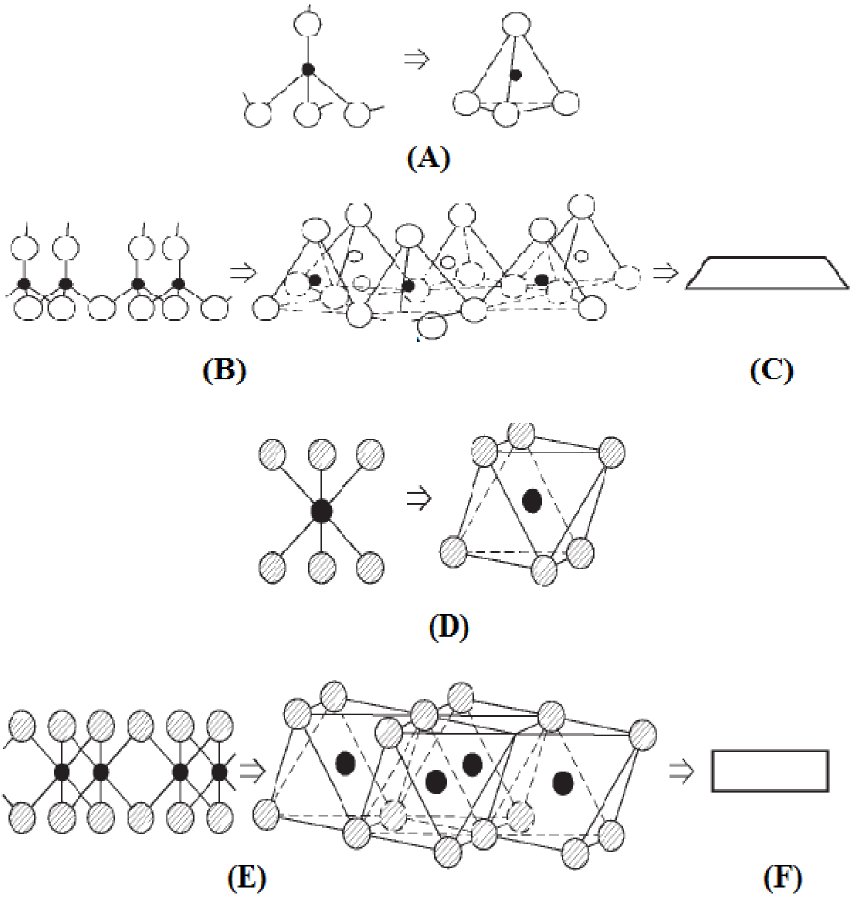


Figure 2.1 Atomic structure of silicon tetrahedra and alumino-magnesium octahedra: (a) silicon tetrahedron; (b) silica sheet; (c) symbolic structure for silica sheet; (d) alumino-magnesium octahedron; (e) octahedral sheet; (f) symbolic structure for octahedral sheet (after lambe and whitman, 1969; mitchell and soga, 2005).

There are three important groups of clay minerals, which are named as Montmorillonite, Illite, and kaolinite. Montmorillonite is the clay mineral which causes most of the expansive soil problems. The name Montmorillonite is uses currently both as a group name for all clay minerals with high expansiveness potential (Mielenz and King, 1995)

A. Kaolinite

Kaolinite is a typical two layer mineral having a single tetrahedral sheet joined by a single octahedral sheet to form what is called a 2 to 1 lattice structure (Chen, 1975). The bonding combination of hydrogen and van der Waals forces results in considerable strength and stability with little tendency for interlayers to swell. The bonding is highly strong that there is no interlayer swelling in the presence of water.it is the list active of clay minerals (Muni Budhu, 2007).

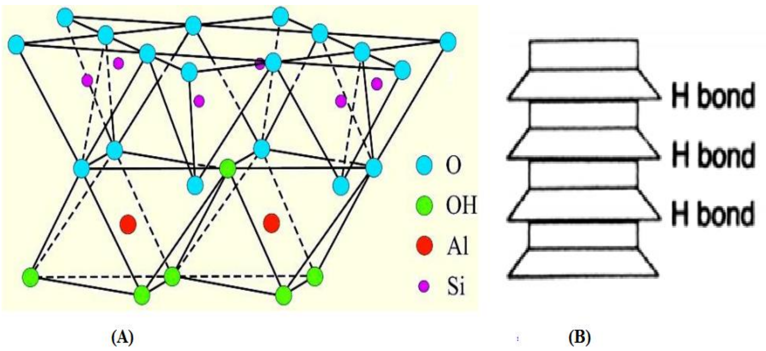


Figure 2.2 Diagrammatic and schematic representation of kaolinite (after Baser, 2009 and Craig, 1997)

B. Montmorillonite

Montmorillonite is a three-layer mineral having a single octahedral sheet sandwiched between two tetrahedral sheets (Chen, 1975). The space between the combined sheets is occupied by water molecules and exchangeable cations. There is very weak bond between the combined sheets due to these ions. Considerable swelling of Montmorillonite being can occur due to additional water observed between the combined sheets (Muni budhu, 2007).

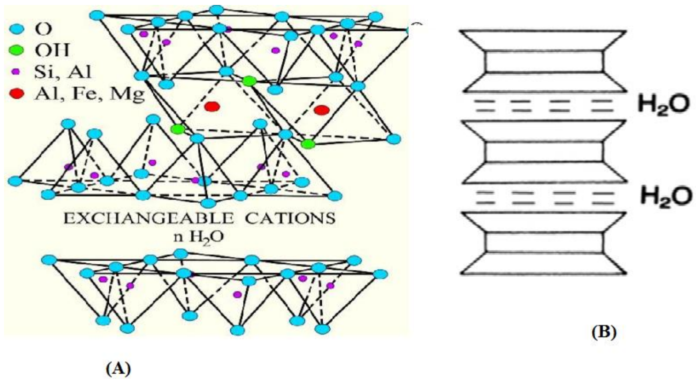


Figure 2.3 Diagrammatic and schematic representation of Montmorillonite (after Baser, 2009 and Craig, 1997)

C. Illite

Illite has similar structure with that of Montmorillonite, but some of the silicon atoms are replaced by aluminum and in addition potassium ions are present between the tetrahedral sheet and adjacent crystals (chen, 1975). The layers of Illite clay minerals are more strongly bonded together than the Montmorillonites. In terms of Cation exchange capacity, inability to absorb and retain water and in physical characteristics Illite is intermediate in activity between clays of kaolinite and Montmorillonite. The layers of Illite clay minerals are linked together by fairly weak bonding to potassium ions held between them (Muni Budhu, 2007).

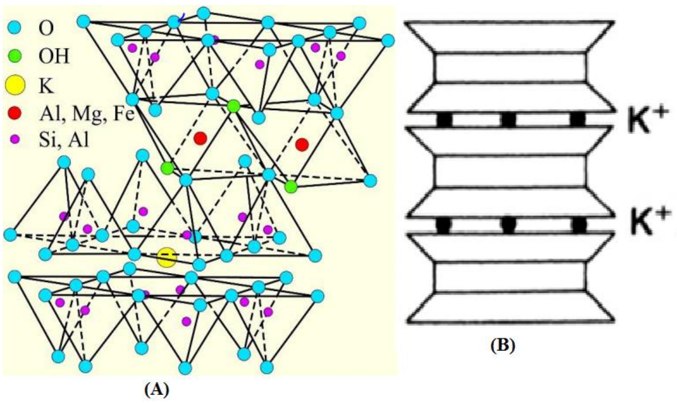


Figure 2.4 Diagrammatic and schematic representation of Illite (after Baser, 2009 and Craig, 1997)

2.5 Factors Affecting Expansive Soil Swelling and Shrinkage

According nelson and miller, 1992 the expansive soil's swelling and shrinkage affecting factors are shown in the table below:

Table 2.1 Factors affecting expansive soil swelling

Factor	Description
Initial water content	Small amount of initial water content on the other hand indicates small degree of saturation. The tendency of soil to observe water will increase and this condition increases swelling potential.
Clay mineralogy	Clay soils which have clay minerals with higher swelling potential like Montmorillonite have hinge swelling potential.as the amount of clay mineral with high swell potential increases the swelling potential of the soil increases
Dry density	The higher the value of initial dry density implies, a closer particle spacing have large swelling potential
Particle size	Fine particles in a soil exists densely, and the finer the particle the higher will be its expansion potential
Concentration of pore fluid salts	Higher concentration of Cation in the pore field decreases expansion potential
Pore field composition	Prevalence of monovalent Cation increases swelling potential while divalent inhabit shrinkage.
Climate	Arid climate courses desiccation of water content. This reduction of water content may lead to increase swelling potential of clay.
Location of water table	Fluctuating the location of water table causes variation of water content along the depth of the clay stratum, and the water content variation affects the soil swell-shrinkage property.
Thickness of clay stratum and confining pressure	High thickness of soil strata and large confining pressure reduces the soils swelling potential.
Field permeability	Joints and fissures in a soil allows to pass water through, and significantly affects swelling capacity

2.6 Physical Properties of Expansive Soil

It is well known to soil engineers that Montmorillonite clays swell when the moisture content is increased, while swelling is absent or limited in Illite and kaolinite. Most important expansive soils physical properties are fatigue of swelling, index properties, dry density, and moisture content.

A. Moisture Content

Irrespective of high swelling potential, if the moisture content of the clay remains unchanged, there will be no volume change; and structures founded on clays with constant moisture content will not be subject to movement caused by heaving. When the moisture content of the clay is changed, volume expansion, both in the vertical and horizontal direction, will take place. Complete saturation is not necessary to accomplish swelling. Slight changes of moisture content, in the magnitude of only 1 to 2 percent, are sufficient to cause detrimental swelling. In the laboratory, clay samples swell in the consolidometer with slight increase of humidity. It is known that floor slabs founded on expansive soils cracked most severely when the moisture content increased slightly due to local wetting. If the floor slab is flooded, as in the case of a rising water table, the floor will heave but the extent of cracking will not be severe (Chen, 1975).

Very dry clays with natural moisture content below 15 percent usually indicate danger. Such clays will easily absorb moisture to as high as 35 percent with resultant damaging expansion to structures. Conversely, clays with moisture contents above 30 percent indicate that most of the expansion has already taken place and further expansion will be small. However, moist clays may desiccate due to lowering of water table or other changes in physical conditions and up on subsequent wetting will again exhibit swelling potential (Chen, 1975).

B. Dry Density

The dry density of the clay is another index of expansion. Soils with dry densities in excess of 110 pcf generally exhibit high swelling potential. Remarks made by excavators complaining that the soils are as hard as a rock is an indication that soils inevitably will present expansion problems. The dry density of the clays is also reflected by the standard penetration resistance test results. Clays with penetration resistance in excess of 15 usually possess some swelling potential. In the highly expansive clay areas of Denver, penetration resistances as high as 30 are not uncommon (Chen, 1975).

C. Index Properties

Chen (1975) founds that it is more convenient to correlate the expansive properties with percentage of silt and clay, liquid limit, and field penetration resistance. The simplified classification of the expansive properties can be conveniently used by engineers as a guide for the choice of type of foundation on expansive soils. Table 2.2 is a guide for estimating volume change of expansive soil (Chen, 1975).

Table 2.2 Data for making estimates of probable volume changes for expansive soils (after Chen, 1975)

Laboratory and field data			Probable expansion percent total volume change	Swelling pressure (ksf)	Degree of expansion
Percentage passing No 200 sieve	Liquid limit percent	Standard penetration resistance (blows/ft.)			
>35	>60	>30	>10	>20	Very high
60-95	40-60	20-30	3-10	5-20	High
30-60	30-40	10-20	1-5	3-5	Medium
<30	<30	<30	<1	1	low

D. Fatigue of Swelling

A clay sample is subjected to full swelling in the consolidometer, allowed to desiccate to its initial moisture content, then is saturated again. This is repeated for a number of cycles. It was observed that the soil showed signs of fatigue after each cycle of drying and wetting (Chen, 1965).

This phenomenon has not been under full investigation. It has been noted that pavements founded on expansive clays which have undergone seasonal movement due to wetting and drying have a tendency to reach a point of stabilization after a number of years. The fatigue of swelling probably can furnish the answer (Chen, 1975). If drying and wetting cycles are repeated, the swelling during the first cycle would be appreciably higher than that in subsequent cycles (Chu, 1973).

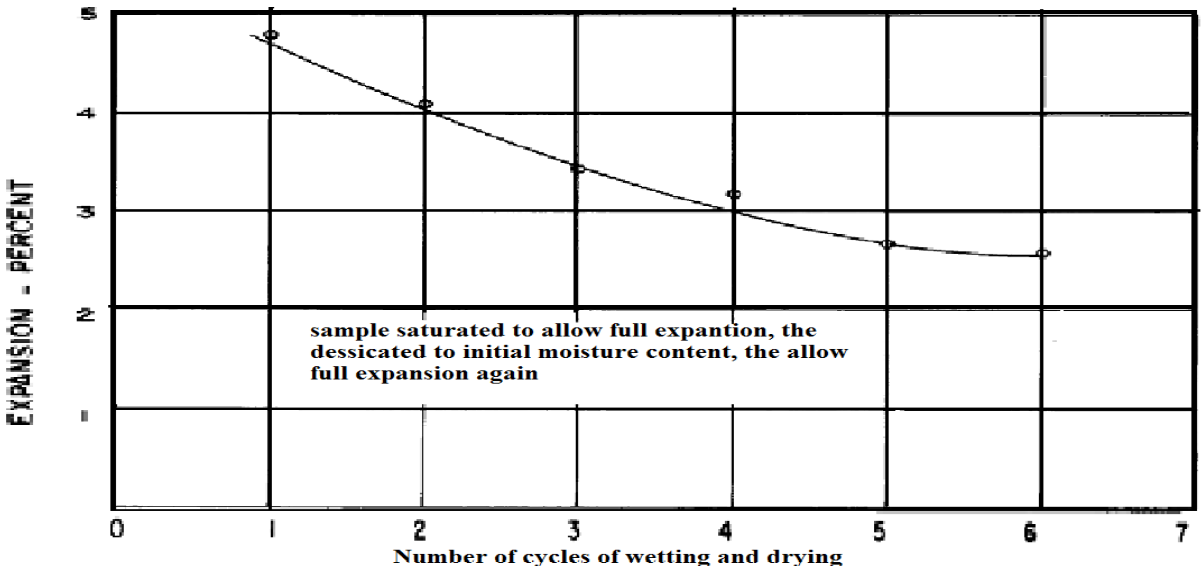


Figure 2.5 Fatigue of swelling (after Chen, 1965)

2.7 Identification of Expansive Soil

Different standards uses different method to classify a soil whether it has the potential to be expansive or not. The identification methods used to identify the swell potential of expansive soils can generally be grouped into two categories. The first category mainly involves field identification methods. Measurement of physical properties of soils, such as Atterberg limits, free swell, and potential volume change are the second category to identify expansive soils. Third category involves measurement of mineralogical and chemical properties of soils, such as clay content, Cation exchange capacity, and specific surface area (John et al., 2015).

2.7.1 Field Identification

Soils that can exhibit high swelling potential can be identified by field observations, mainly during reconnaissance and preliminary investigation stages. Important observations include (Chen, 1988; Nelson, and Miller, 1992):

- ✓ Usually have a color of black or grey.
- ✓ Wide or deep shrinkage cracks.
- ✓ High dry strength and low wet strength.
- ✓ Stickiness and low traffic ability when wet.
- ✓ Cut surfaces have a shiny appearance.
- ✓ Appearance of cracks in nearby structures.

2.7.2 Methods on Physical Properties (Indirect Method)

This method is a valuable tool in evaluating the swelling potential of the soils. It is advisable not to use the indirect tests directly, instead direct tests are also important to avoid an error in conclusions. These methods are related to laboratory soil identification and are vital for the intended purposes (Chen, 1998).

A. Based On Plasticity Index Result

Atterberg limit is commonly used index to classify soils and are used to identify expansiveness. It is common to use two indices on the basis of Atterberg limits which are called plasticity index, PI, and the liquidity index, LI. the more expansive soil directly relates with higher plasticity. While it is true that, high swelling soil will manifest high index property the converse is not true (John et al., 2015). Chen, 1975 develops the swelling potential and plasticity index relationship equation 2.1;

$$S = Be^{PI} \quad \text{..........} 2.1$$

Where: S=swell potential, “B” and “e” are constants with a value of 0.0838 and 0.2558 respectively Peck, Hanson, and Thornburn (1974) correlates a relationship between plastic index of a soil and its potential for expansiveness as shown above on table 2.2.

B. Based On Liquid Limit Result

This method is one of the tools in evaluating the swelling potential of expansive soils. It is also easy to perform and can be executed as routine work in the investigation of other soil properties (Chen, 1998). According Chen (1975) the liquid limit test result is related to expansion potential as shown in table 2.3 below.

Table 2.3 Liquid limit, plasticity index and, and expansion potential relation of clay soil (after Peck et al., 1974 and Chen, 1998)

Plasticity index (%)	Liquid limit (%)	Swelling potential
0-15	< 30	Low
10-35	30-40	Medium
20-55	40-60	high
>55	>60	Very high

Holtze (2011) has correlated plasticity of soil its liquid limit and mineral composition as shown in figure 2.6 below.

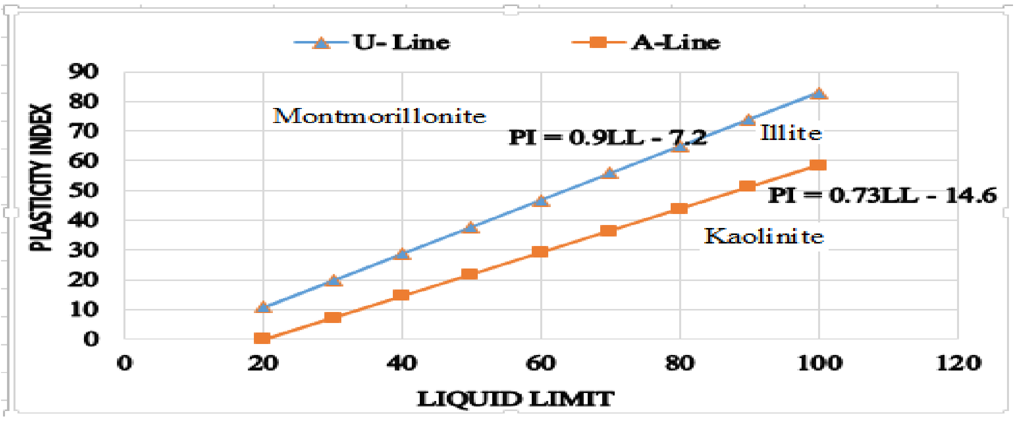


Fig 2.6 Plasticity characteristics of clay mineral (after holtz et al., 2011)

C. Free Swell

The free swell test consists of placing a known volume of dry soil passing through sieve No 40 pour in to a graduated cylinder filled with water and measuring the swelled volume after it has completely settled. The free swell is found as a ratio of change in volume from the dry state to the wet state over the dry volume expressed as percentage (John et al., 2015). The relation between free swell value and expansion potential is as shown in table 2.4.

$$\text{Free swell} = \frac{(\text{final volume} - \text{initial volume}) * 100}{\text{initial volume}} \% \quad \ldots\ldots 2.2$$

Table 2.4 Free swell and degree of expansion relationship (ASTM D 2216)

Free swell value	Degree of expansiveness
< 50%	Not expansive
50%-100%	Marginal
>100%	Expansive

D. Linear Shrinkage

The swell potential is presumed to be related to the opposite property of linear shrinkage measured in a very simple test. Theoretically it appears that the shrinkage characteristics of the clay should be a consistent and reliable index to the swelling potential (Chen, 1975). Altmeyer in 1955 suggests a relationship between linear shrinkage, shrinkage limit and the potential of expansiveness as shown in the table below;

Table 2.5 Shrinkage limit, linear shrinkage, and degree of expansion relationship (after Altmeyer, 1955)

Shrinkage limit (%)	Linear shrinkage (%)	Degree of expansion
< 10	> 8	Critical
10-12	5-8	Marginal
> 12	0-5	Non-critical

E. The Activity Method

According seed et al. (1962) the activity method is prepared based on remolded, artificially prepared soils compacted of 23 mixtures of bentonite, Illite, kaolinite, and fine sand. The expansion was measured as percent swell on soaking from 100% maximum density and optimum moisture content in standard AASHTO compaction test. The activity for the artificially prepared sample is defined as equation 2.3 and grain size activity relationship is shown in figure 2.7.

Activity $=\frac{PI}{C-10}$……………………………………………………………………2.3

Where C is percentage particle size finer than 0.002 mm, and PI is the plasticity index.

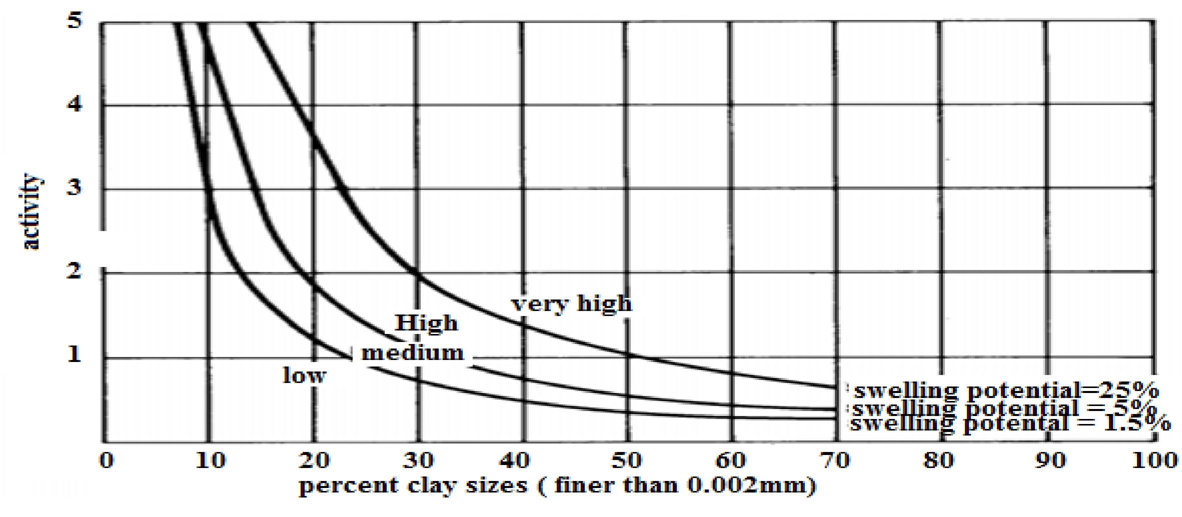


Figure 2.7 Swelling potential and percent clay size relationship (Seed et al, 1962)

Skempton (1953) has developed mathematical relationship between Activity, plasticity Index, and the percentage clay size as equation 2.4 and he suggested three classes of clay according their activity as shown in the table 2.6 below;

$$Activity\ (A_c) = \frac{Plasticity\ Index}{\%\ by\ weight\ finer\ than\ 2\mu m} \quad \text{.......} 2.4$$

Table 2.6 Activity clay type relationship (after Skempton, 1953)

Activity	Clay type
< 0.75	Inactive
0.75-1.25	Normal
> 1.25	Active

2.7.3 Mineralogical Methods

Identifying the existence of Montmorillonite in a soil is one means to notify whether the soil is potentially expansive or not. The mineralogy of clay can be identified on the basis of crystal structure or by means of chemical analysis (John et al., 2015).

A. X-Ray Diffraction

X-ray diffraction method is used to determine the proportion of various minerals that could present in a colonial clay. It consists essentially of comparing the ratio of the intensities of diffraction lines from the different minerals with the intensities of line with the standard substances (Chen, 1975). The use of self-recording counter spectrometer in lieu of photographic techniques increases both the accuracy and the conveyance of the X-ray method (Brindley, 1955).

B. Differential Thermal Analysis (DTA)

DTA is operated by heating a sample of clay and an inert substances and the resulting thermograms which are the plots of temperature difference versus applied heat are compared to those pure material. Each material have characteristic endothermic and exothermic reactions on the termogram (John et al., 2015).

C. Dye Adsorption

Dyestuffs and other reagents which exhibit characteristic color when adsorbed by colloidal clay particles have been used to identify clay soils. When a clay sample has been pretreated with acid, the color assumed by the adsorbed dye depends on the Base Exchange capacity of the various clay mineral present. Hence the existence of Montmorillonite can be detected if its amount is greater than about 5%-10% (Chen, 1975).

D. Electron Microscopy

Electron microscopy provides a means of directly observing the clay particles. It is possible to identify the clay qualitatively based on size and shape of its particles (John et al., 2015). It is possible to obtain the same X-ray pattern and the same differential thermal curve but with different morphological characteristics under electro-microscopic resolution. The main aim of using electro-microscopic examination is to identify the mineralogical composition, internal structure and texture (Chen, 1975). During electro microscopic inspection none swelling clays appears as flat, relatively thick plates, while Montmorillonite have crinkly, honeycomb-like structure. It might be possible to evaluate some properties of expansive soil by observing the degree of crinkling and Interparticle bonding from scanning an electro-microscope (Ravina, 1973).

E. Chemical Methods

Measurement of Cation exchange capacity (CEC), specific surface area (SSA), and the total potassium (TP) are the most common chemical methods that are used to identify clay minerals (John et al., 2015).

I. Cation Exchange Capacity (CEC)

The total number of cations required to balance the negative charge on surface of clay particles is called Cation exchange capacity. It is measured in mill equivalents per 100 gram of clay particles. The amount of a known Cation needed to saturate the exchange size is greater for the mineral with greater unbalanced surface charge. Cation exchange capacity is related to clay mineralogy. High Cation exchange capacity value indicates the presence of more active clay mineral such as Montmorillonite, whereas low Cation exchange capacity indicates the presence of non-expansive clay such as kaolinite (John et al., 2015). The Cation exchange capacity and expansion potential relationship according Mitchell and soga (2005) is as shown in table 2.8;

II. Specific Surface Area

The specific surface area of a soil is defined as the total surface area of a soil particle in a unit mass of soil. Montmorillonite has greater specific surface area than kaolinite and Illite. A clay with high specific surface area will have higher water holding capacity and greater expansion potential (Chittoori and puppala, 2011). But a soil having high specific surface area doesn't necessarily indicate an expansive soil. A soil having high organic fraction may behave as highly reactive surface similarly as that of the material with high specific surface area (jury et al., 1991). Adsorption of polar molecules, such as ethylene glycol, on the surface of the clay material is the most common method to determine SSA (John et al., 2015). The specific surface area and expansion potential relationship according Mitchell and soga (2005) is as shown in table 2.8.

III. Total Potassium

Illite is the only mineral which contains potassium in its structure, and hence the existence of potassium ion in a clay directly depicts the presence of Illite. High potassium content indicates low expansion potential (chittooti and pupplala, 2011). The Total potassium and expansion potential relationship according Mitchell and soga (2005) is as shown in table 2.7;

Table 2.7 Total potassium, specific surface area (ssa), cation exchange capacity, and clay mineral relationship (Mitchell And Soga, 2005)

Clay mineral	Specific surface area (m^2/g)	Cation exchange potential (meq/100mg)	Total potassium (%)
Kaolinite	5-55	1-6	0
Illite	80-120	15-50	6
Montmorillonite	600-800	80-150	0

2.8 Expansive Soil Stabilization

In geotechnical engineering practice the soils at a given site are often less than ideal for the intended purpose. To overcome the problems from expansive soils, it would seem reasonable in such instances to simply relocate the structure or facility. However, considerations other than geotechnical often govern the location of a structure, and the engineer is forced to design for the site at hand. One possibility is to adapt the foundation to the geotechnical conditions at the site. Another possibility is to try to stabilize or improve the engineering properties of the soils at the site. Depending on the circumstances, this second approach may be the most economical solution for the problem (Craig, 1994).

Some methods may be used alone or in conjunction with specific foundation or pavement alternatives. Depending on the situation a particular method may be applied before or after construction. Choice of the method to be used will rely on experience and sound engineering judgment. A program of site investigation, laboratory testing, and consideration of construction techniques is necessary to evaluate the possible alternatives (John et al., 2015).

2.8.1 Mechanical Soil Stabilization

Mechanical stabilization can be defined as a process of improving the stability and shear strength Characteristics of the soil without altering the chemical properties of the soil (Guyer, 2011; Makusa, 2012). The main methods of mechanical stabilization can be categorized;

A. Pre Wetting

The pre wetting theory is based on the assumption that if soil is allowed to swell by wetting prior to construction and if the high soil moisture content is maintained, the soil volume will remain essentially constant, achieving a non-heave state and hence structural damage will not occur (Chen, 1975). Bara (1969), develops the water content-liquid limit relationship for soil liquid limit ranging between 40 and 100 as shown in the figure below. Moisture content above the reference line in figure below would assure that the densities were on the noncritical side of the reference line.

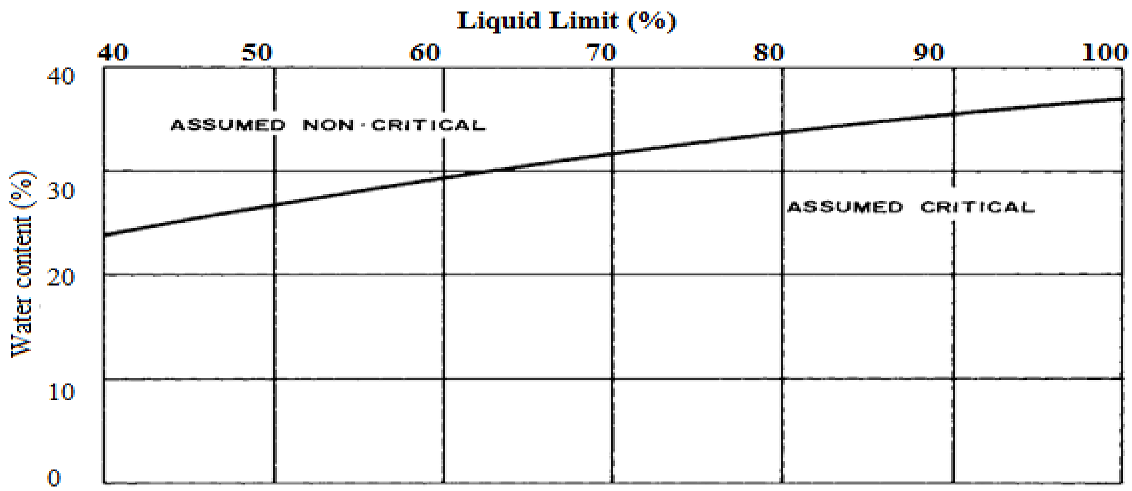


Figure 2.8 Minimum water content required for soil liquid limit (after Bora, 1969)

Pre wetting have the following disadvantages.

- ✓ Moisture migration can continue for many years causing new swelling on its way.
- ✓ It is highly questionable if a uniform moisture content can be obtained in pre wetted areas.
- ✓ Experiments include that ponding water can effectively penetrate the soil to a depth of four feet with in reasonable time. But such depth is insufficient to provide balance moisture zone for the construction of important strictures.

B. Compaction Control

The amount of swelling that the structural fill is exposed to additional moisture depends up on, the moisture content, compacted dry density, surcharge load, and method of compaction (Chen, 1975). Highly expansive soils have to be compacted to some minimum density than the maximum dry density value. Expansive clays expand very little when compacted at law densities and high moisture but expand greatly when compacted at high density and low moisture (Dawson, 1959). Moisture content is important for swelling control. Moisture content by itself does not reduce swelling, but it can result in low density fill (Gizienski and Lee, 1965).

C. Moisture Control Alternatives

Because the root cause of soil expansion is an increase in water content of the foundation soils, it would appear that in order to eliminate problems of heave one would only have to control the water content of the soil. However, it is virtually impossible to prevent an increase in water content of the foundation soils after site development. Nevertheless, it is possible to exercise some control over the rate of increase and the magnitude of seasonal fluctuations.

Water content fluctuations beneath slabs can be reduced by means of horizontal and vertical barriers (John et al., 2015).

D. Excavation and Replacement

Over excavation of expansive soils and replacement with non-expansive or treated soils has been used to mitigate soil heave under a foundation or subgrade. In this method, the expansive soil is excavated to an appropriate depth to minimize heave to an appropriate amount, and then appropriately treated and compacted fill is placed to bring the soil up to grade. The necessary depth of soil that is required to be removed will depend on the overall soil profile, the nature of the soil that will be used for fill, and the allowable heave. An additional advantage provided by a stiff layer of compacted low to non-expansive fill is that it will tend to even out the differences in heave of the underlying native soil, thereby reducing the differential heave (John et al., 2015). According Chen (1975), the important requirements concerning expansive soil replacement are type and quality of material to be replaced, depth of replacement, extent of replacement, and economic issues.

2.8.2 Chemical Stabilization

Chemical stabilization involves mixing or injecting the soil with chemically active compounds such as Portland cement, lime, fly ash, calcium or sodium chloride or with viscoelastic materials such as bitumen. These mechanical stabilizers relay on calcium exchange, Flocculation and agglomeration, and pozzolanic reactions (Chen, 1975).

I. Cation Exchange

In Cation exchange the monovalent cations generally associated with clays are replaced by the divalent ions. The cations can be arranged in a series based on their affinity for exchange and any Cation can replace the ions at its right. (M.Das, 2007)

$Al^{3+}>Ca^{2+}>Mg^{2+}>NH4^{+}>K^{+}>Na^{+}>Li^{+}$

Cation exchange includes an immediate reaction of the clay with the stabilizer within few minutes of mixing, resulting in a soil with improved texture. The tetrahedral (T) and octahedral (O) combination of clay minerals have charge deficiency that results in the attraction of the cations or water molecule. Generally, sodium or potassium (Na^{+} or K^{+}) are prevalent in clay minerals along with water. However, these cations can be replaced by the higher valance cations like Al^{+3}, Ca^{+2}, Mg^{+2} etc. so called Cation exchange. During this process calcium rich chemical stabilizer provides enough cations to replace the monovalent cations resulting in a reduced thickness diffused double layer (Geiman et al., 2014). The calcium is released in suspension of stabilizer-soil water and will be available for the stabilization of soil. The general reaction of the cement with water that yields calcium is presented in relations 2.5 and 2.6.

$$C3S + H \longrightarrow C\text{-}S\text{-}H + Ca\,(OH)\,2 \quad \ldots\ldots 2.5$$

$$C2S + H \longrightarrow C\text{-}S\text{-}H + Ca\,(OH)\,2 \quad \ldots\ldots 2.6$$

Where, $H=H_2O$, $C=Ca$, $S=SiO_2$, C3S=tri calcium silicate, C2S=di-calcium silicate and C-S-H = $C_3S_2H_3$

II. Flocculation and Agglomeration

As Cation exchange, flocculation and agglomeration is also a short-term process, which takes place within few hours of mixing the stabilizer and water with subgrade soil. Flocculation and agglomeration produces a change in the texture of the clay soil. The clay particles tend to clump to form large particles, thereby decreasing liquid limit, increasing plastic limit, reducing plastic index, increasing workability and improving strength and deformation properties of soil (M.Das, 2007).

III. Pozzolanic Reaction

Pozzolanic reaction between soil and the stabilizer involves a reaction between the chemical stabilizer and silica-aluminum of the soil to form cementing material (M.Das, 2007). The PH environment in the system initiates further reaction of the silica and alumina with the clay particles, hence proving extra strength to the stabilized soils (Harty, 1970). The minimum PH of 12.4 is necessary in order to maintain the pozzolanic reaction (Eades and Grim, 1960). According M.Das (2007) pooolanic reaction between lime and soil is indicated on the following reaction equation;

$Ca(OH)_2 + SiO_2$…………………>> CSH (clay silica)...…………………………………….2.7

Where, C=CaO, S=SiO_2, and H= H_2O

2.8.2.1 Lime Stabilization

Lime is among the common admixtures used to stabilize soil in the field particularly expansive soils. Addition of lime significantly reduces the swelling potential, liquid limit, plasticity index and maximum dry density of the soil, and increases its optimum water content, shrinkage limit and strength (Crof, 1967).

Many state highway departments have researched lime treatment and frequently use this method. Generally, from 3 to 8 percent by weight of hydrated lime is added to the soil. It is also used as a follow-up treatment over ponded areas to add strength to the surface and to provide a working surface for equipment (McDowell 1965; Teng et al., 1973). As previously discussed, the primary reactions in the lime reaction include Cation exchange, flocculation-agglomeration, lime carbonation, and pozzolanic reaction (Thompson 1966 and 1968; Little 1995; NLA 2004).The strength characteristics of a lime-treated soil depend primarily on soil type, lime type, lime percentage, and curing conditions such as time and temperature (John et al., 2015).

Lime is not an effective treatment for all types of soils. In general, clay soils with a minimum of 25 percent passing the No. 200 sieve and a plasticity index greater than 10 percent are considered to be good candidates for soil treatment (NLA, 2004). Caution must be exercised, however, with the use of lime. Some soil components such as sulfates, organics, and phosphates can cause reactions that can have serious adverse effects. It is possible to use several types of limes as a stabilizer such as; quick lime, hydrated lime, dolomitic lime, normal hydrated dolomitic lime, and pressure hydrated dolomitic lime, but the most usual lime stabilizers the first two. Quicklime is manufactured by chemically transforming calcium carbonate ($CaCO3$) into calcium oxide (CaO) by heating. Quicklime will react with water to form hydrated lime.

Either quicklime or hydrated lime can be used as an agent for soil treatment. If quicklime is used, the first water that is introduced will be used in the chemical reaction to form hydrated lime, which then reacts with the soil. Caution must be exercised when using quicklime. It can cause serious burns to skin and eyes if personnel come into contact with it. Modern spreading equipment can reduce the potential safety hazards associated with using quicklime. Most lime used for soil treatment is “high calcium” lime, which contains 5 percent or less magnesium oxide or hydroxide (NLA 2004; John et al., 2015).

The production of strong cementing agents can occur from reactions between lime, water, and aluminous or siliceous substances. The high pH environment created by the addition of lime increases the solubility of silica in the soils. The lime supplies a divalent calcium Cation that can form calcium silicates and calcium aluminum hydrates, which can form physical bonds between particles to increase soil strength (John et al. 2015). The researcher have used hydrated high calcium lime ($Ca\ (OH)_2$ as a stabilizing agent.

2.8.2.2 Type of Lime

Table 2.8 lists several types of lime used as additives. Care must be taken to assure that industrial lime is used. Quicklime is manufactured by chemically transforming calcium carbonate ($CaCO3$) into calcium oxide (CaO) by heating. Quicklime will react with water to form hydrated lime. Either quicklime or hydrated lime can be used as an agent for soil treatment. If quicklime is used, the first water that is introduced will be used in the chemical reaction to form hydrated lime, which then reacts with the soil. Caution must be exercised when using quicklime. It can cause serious burns to skin and eyes if personnel come into contact with it. Modern spreading equipment can reduce the potential safety hazards associated with using quicklime. Most lime used for soil treatment is “high calcium” lime, which contains 5 percent or less magnesium oxide or hydroxide (NLA, 2004).

However, sometimes dolomitic lime, which contains 35 to 46 percent magnesium oxide or hydroxide can be used. Dolomitic lime can also perform well when used for soil treatment, but the magnesium fraction of the lime requires more time to react than does calcium. The type of lime that is used can influence the strength of the treated soil. Dolomitic lime generally will be more effective in increasing strength.

Table 2.8 Lime materials used in soil treatment

Lime type	Formula
Quicklime	CaO
Hydrated lime	Ca(OH)2
Dolomitic lime	CaO • MgO
Normal hydrated or monohydrated dolomitic lime	Ca(OH)2 • MgO
Pressure hydrated or dehydrated dolomitic lime	Ca(OH)2 • Mg(OH)2

2.8.2.3 Soil Factors Affecting Lime Reactivity of Soil

Factors influencing the lime reactivity of a soil include the following:

- ✓ A soil pH greater than about 7 indicates good reactivity.
- ✓ Organic carbon greatly retards lime-soil reactions.
- ✓ Poorly drained soils tend to have higher lime reactivity than well drained soils.
- ✓ Calcareous soils have good reactivity.
- ✓ The presence of soluble sulfate salts in the soil can react with lime to Cause ettringite-induced heave.

Chemical composition of hydrated lime is as table shown below;

Table 2.9 Chemical composition of ordinary Portland cement *(after Adelaide Brighton Cement ltd, 2005)*

Common name	Oxide	Percentage composition
Lime	CaO	72.2
Silica	SiO_2	1.8
Alumina	Al_2O_3	0.5
Iron oxide	Fe_2O_3	0.6
Magnesia	MgO	1
Loss on ignition	-----	24
CaO_2	-----	1.1

2.8.2.4 Calculation of Lime Modification Optimum

Eades and Grim (1966) developed an easily applied test to determine if a soil is lime reactive and how much lime, in percent by weight, is necessary to achieve a desired volume change reduction. The test considers that when a calcium-based compound such as lime is added to clay soil, a reaction occurs that is based on soil-silica and soil-alumina solubility at a high pH. The pH of the lime/soil mixture can be used to identify the optimum lime content. The procedure is simple and can be completed within 1 hour. The results of the test are plotted on a graph showing percent lime versus pH. As the lime content is increased, the PI of the lime-treated soil will decrease. The point where the addition of more lime produces little, or no, decrease in PI is the point of maximum effectiveness. Also, as the lime content is increased, the pH of the soil increases. The higher pH is also associated with a lower PI. In the Eades and Grim test the lowest percent lime content to produce a pH of 12.4 is termed the lime modifiation optimum (or LMO) and is the approximate lime content to be used for treating the soil. A drawback of the Eades and Grim method is that a pH of 12.4 does not always ensure lime-soil reactivity (Currin, Allen, and Little 1976). Other factors must be considered to assess the effectiveness of lime in reducing the expansion potential. ASTM D6276 and ASTM C977 provide more detail on the determination of the LMO.

2.8.3 Cement Stabilization

The hydration products of Portland cement include calcium silicate hydrates, calcium aluminate hydrates, and hydrated lime. During hydration Portland cement releases a large amount of lime.it is believed that the Base Exchange and cementing ratio of Portland cement is similar with that of lime (Chen, 1975). In addition to the above actions, the incorporation of Portland cement in clay increases the strength of the mixture. Cement stabilization is similar to that of lime and produces similar result. Cement stabilization develops from the cementious links between the calcium silicate and aluminum hydration products and the soil products (Crof, 1967).

Addition of cement to clay soil reduces the liquid limit, plasticity index and swelling potential and increases the shrinkage limit and strength. The result of mixing cement with clay soil is similar to that of lime. But it is not as effective as lime in treating highly plastic clays. Some clay soils have such a high affinity for water that the cement may not hydrate sufficiently to produce a complete pozzolanic reaction (Mitchell and Raad, 1973).

For clays in which Portland cement treatment is effective, mixing procedures similar to those used for lime treatment can be used. One difference in technique is that the time between cement addition and final mixing should be shorter than that used for lime treatment. Portland cement has a shorter hydration and setting time. Because of the strength increase that can be generated by the use of cement, the soil-cement mixture can increase pavement and slab strength significantly (John et al., 2015).

Generally, the amount of cement required to treat expansive soils ranges from 2 to 6 percent by weight (Chen 1988). A cement content of 2 to 6 percent can produce a soil that acts as a semi rigid slab. This will aid in reducing differential heave throughout the slab. Little et al. (2000) indicated that cement treated materials may be prone to cracking as a result of hydration and moisture loss. Shrinkage cracks can compromise performance if they become wide and admit significant water. However, if proper construction procedures are followed, the effects of shrinkage cracks can be minimized. Techniques that have been used to minimize cracking problems include the following (Petry and Little, 2002).

Both cement and lime have been used in highway construction for modifying the swelling property of subgrade soil, but the use of cement and lime to stabilize under slab soil in buildings is seldom reported.

There appears to be a great potential using cement to modify the under slab soils. With 2% to 6% cement incorporated in the clay, the resulting soil-cement mixture acts as a semi-rigid slab. If the deep seated soil expands, the swelling effect tends to distribute uniformly and reducing damages caused by differential heaving. Such construction is particularly favorable for treatment of large warehouse floor where crack free level floor is essential and the use of structural floor slab is economically prohibitive. Due to this lack of strength the use of lime cannot be provide a semi rigid element beneath the slab (Chen, 1975).Chemical composition of Portland cement are as shown in table shown below.

Table 2.10 Chemical composition of ordinary Portland cement (after joseph Onur baser, 2009)

Common name	Oxide	Percentage composition
Lime	CaO	63.6
Silica	SiO_2	20.7
Alumina	Al_2O_3	6
Iron oxide	Fe_2O_3	2.4
Magnesia	MgO_3	2.4
Soda	Na_2O	0.1
Potassa	K_2O	0.7
Sulfur tri oxide	SO_3	1.4
Losson ignition	---------	1.2
Insoluble residue	--------	0.3
Free CaO	-----	1.1

2.8.3.1 Calculation of Cement Modification Optimum

General guidelines for stabilization are that the plasticity index should be less than 30 for sandy materials. For fine-grained soils, soils with more than 50 percent by weight passing 75µm sieve, the general consistency guidelines are that the plasticity index should be less than 20. In order to ensure proper mixing. A more specific general guideline based on the fines content is given in the equation below which defines the upper limit of PI for selecting soil for cement stabilization (AASHTO, 2008).

$$PI \leq 20 + \frac{50-(\%\ smaller\ than\ 0.075mm}{4} \qquad \text{2.8}$$

2.9 Application Methods of Lime and Cement Stabilization

Commonly used method of lime and cement application is to mix it mechanically, even though this method it is difficult to mix deeper than about 300 mm. therefore it is used mainly for treatment of subgrades below pavements or slabs (John et al., 2015). Lime and cement can also be injected in slurry form through holes drilled into the soil. This method has been used both for remedial measures and for preconstruction purposes. Results of the drill hole technique are erratic, and its use is not encouraged. One factor that limits the effectiveness of the method is the inability to uniformly distribute the lime and cement in the soil mass (Thompson and Robnett, 1976).

The diffusion of the slurry occurs very slowly unless the soil has an extensive network of fissures. Differences in various factors such as soil type and texture or quality control during construction undoubtedly account for the disparity in results using the drill-hole method. In general, one must expect a lower degree of confidence for success of the drill-hole technique than for other techniques (John et al., 2015). For lime stablizations the drill hole or lime columone eechnique can be used during expansive soil stablization.the lime columons can be constructed using the quick lime of poweder form (nelson and miller, 1992). The researcher have used portland cemnet and hydrated lime as a stablizing agent.

3 Study Area and Research Methodology

3.1 Description and Location of the Study Area

Asen-Dabo is a town found in south western part of Ethiopia, Jimma zone. It is located at an elevation of 1,792m above mean see level and is found at a coordinate of 7^{o} 46’25’’N and 37^{o}13’39’’E. Temperatures of the town is in a comfortable range, with the daily mean staying between 20 °C and 25 °C year-round. The town founds at distance of 299.9 km south west of Addis-Ababa (the capital city of the country). It has flat terrain and is covered with red, black and gray soils. Specific location of the study area is as shown in the fig. below;

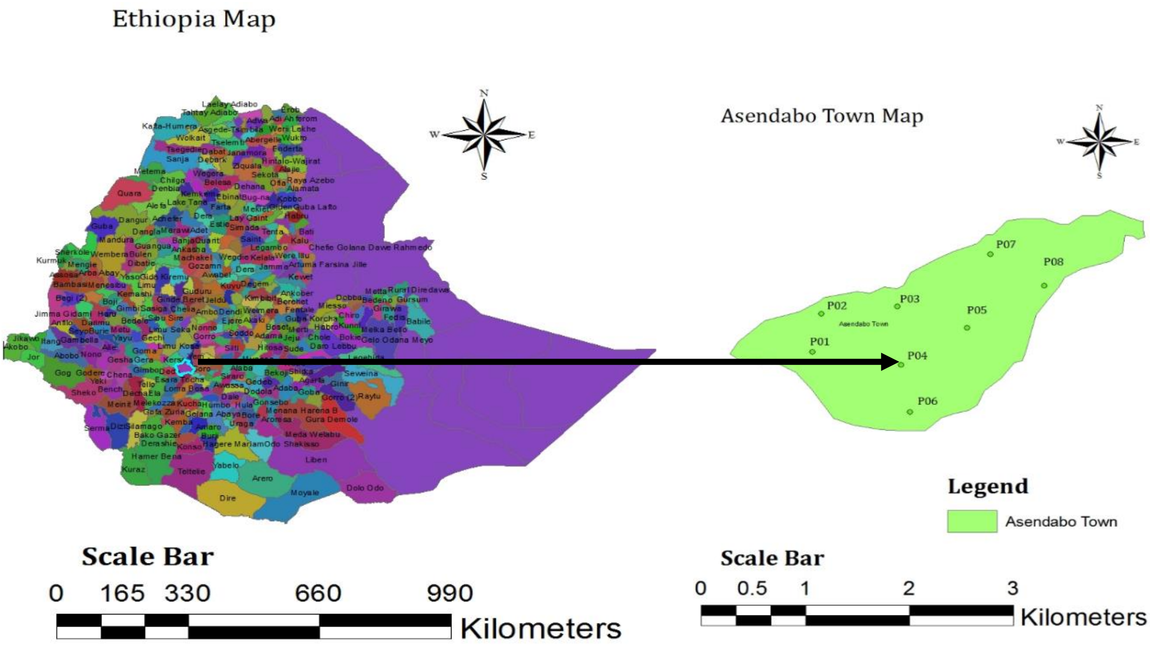


Figure 3.1 Specific location of study area

3.2 Geology of the Area

The main geologic formation of Asen-Dabo town is the Cenozoic tertiary volcanic rock of Nazareth Series and Jimma volcanic that were formed by lava and debris ejected from fissure eruptions. Basalts, Trachyte, Rhyolite, and Ignimbrite are the major rock types that belong to the Trap series formation. Tuft and Alluvial are found in few amounts at different localities (Mengesha, et al, 1996).

3.3 Research Design

The main activities which have conducted in this research after specifying the study area can be grouped as;

I. Secondary Data Collection

Secondary data collection is research activity which is done prior starting detail investigation. In pre investigation stage tasks like searching and gathering related resources, and secondary data collection have been done seriously. Analyzing and organizing the secondary data to form literature review were conducted.

II. **Field Work and Sampling**

Field work is an activity that was takes placed or executed at the research area. Activities that are categorized in this group includes field visiting to observe the color of soil, to visualize the symptom of expansive soil like deep cracks, to identify specific location from which disturbed and undisturbed sample were taken, transportation or shipping of these samples to the laboratory, and sampling.

Generally the soil used in this research is a black cotton expansive soil obtained from the study area. Eight pits were taken as a standing point, and from which detail research were done on two pits after proving as they have the worst expansiveness potential as shown on the appendix one. Disturbed and undisturbed soil samples was collected from all test pits at a depth below 1.5m in order for avoiding organic maters. Gathering and shipping stabilization agent (lime and cement) is also part of field work activity.

III. **Laboratory Work**

Is a stage were all tasks and activities that have been done at the laboratory to examine the effect of using lime and cement as expansive soil stabilizing material was conducted. This stage have been categorized in to two phases;

Phase One; all activities of preparing sample to be tested such as air drying, proportioning or mixing of sample with stabilizers, and Molding sample in order for making it suitable for laboratory testing machine belongs to phase one sub category.

Phase Two; includes laboratory works and procedures that are essential in understanding the properties and behaviors of untreated and treated expansive soils. The following laboratory tests were conducted on this research;

A. Grain Size Analysis

Grain size analyses is performed to determine the different percentage of particles found within a given soil sample. Both wet sieve and haydrometer testes were conducted. acording Rutajama, and Overby,.2000 wet sieve is used to determine whether the soil consistes of predominantly grave, sand, silt and or clay sizes.The hydrometer method covers the quantitative determination of the particle size distribution in a soil from corse sand to the clay size by means of sedimentation.

B. Atterberg Limits

Atterberg limits are water contents which defines the limits of various stages of consistency for fine grain/clay soils. Determining Atterberg limit uses to classify soils and developing correlations with different engineering properties of soil. The upper and lower plastic limit ranges are defined as liquid limit and plastic limit respectively. Plasticity index is the numerical difference between these two limits (Hale, 1980). On this research Atterberg limit tests was conducted accordance to the procedures stated in AASHTO material testing manual.

C. Compaction Test

The objective of this study is to obtain the relationship between the compacted dry density and soil moisture content. The laboratory result of this study gives a specification guide for real field compaction. The dry density which can be achieved by a soil mainly depends on the compaction effort and moisture content and the moisture content which corresponds to the highest density is called optimum moister content (OMC) (Rutajama and Overby,2000). Compaction test on this research is done based on Britain standard (BS) material testing code.

D. Unconfined Compressive Strength

According AASHTO material testing manual, the main aim of performing unconfined compressive strength test is to determine the approximate compressive strength of soils with sufficient cohesion to permit testing in the unconfined state. The unconfined compressive strength test in this research was conducting in accordance to the procedures stated in AASHTO material testing manual. The researcher have conducted both uncured and seven day cured UCS testes.

E. Linear Shrinkage

If the volume of soil which is under continuing drying process is continued to decrease after the plastic limit has been reached, then the soil is experiencing linear shrinkage. Linear shrinkage determines the total linear shrinkage from linear measurements (Rutajama and Overby,2000).The researcher determines the value of linear shrinkage based on BS material testing manual.

F. Specific Gravity

As AASHTO standard specification for specific gravity states, specific gravity is the ratio of mass in air of the given volume of material to the mass in air of gas free distilled water at a given temperature. In this research specific gravity of natural soil were conducted according AASHTO material testing manual.

G. Free Swell

Free swell test gives the approximate value of expansiveness. The researcher were conducted free swell test based on ASTM standard.

H. **California Bearing Ratio**

The resistance to penetration of 2.5mm cylindrical plunger of 50 mm diameter, expressed as a percentage of the known resistance of plunger to 2.5mm in penetration of crushed aggregate is known as California bearing ratio (CBR). It have valuable importance in design of pavement material for natural gravel. The strength of subgrade, sub base and base course material are expressed in CBR (Tanzania material testing manual, 2000). The researcher have conducted one point CBR test based on Britain material testing manual.

IV. Work Organization

This stage is the final stage that the researcher faced to complete the research and it includes activities like

1. Laboratory work out put organizing and sorting in accordance of its necessity.
2. Data analysis and evaluation
3. Thesis writing.
4. Final work dissemination etc.

3.4 Material

Materials needed to carry out the research are;

- ✓ **Portland cement;** It is one of a chemical additive that can be used to stabilize the expansive soils to improve soil engineering properties as well as the mechanical characteristics of the soil like degree of compaction. Generally Cement Stabilization is ideally suited for well graded aggregates with a sufficient amount of fines to effectively fill the available voids space and float the coarse aggregate particles (US army, 1994). The researcher has found Portland cement from the market.
- ✓ **Hydrated lime;** Lime is a chemical additive that has been utilized as a stabilization agent in soils for centuries. Lime will react well with medium, moderately fine, and fine-grained clay soils. In clay soils the main benefit from lime stabilization is the reduction of soil‘s plasticity and improvement of the strength by reducing the soil‘s swell and increasing its degree of Compaction. It also increases the strength and workability of the soil (US army, 1994). The researcher has found hydrated lime for free from Derba Cement factory.
- ✓ boring material
- ✓ molding material
- ✓ Digital camera, pencil, ruler, laptop, paper pen etc.
- ✓ Geotechnical engineering laboratory materials such as unconfined compression test machine, hydrometer testing machine, compaction material, sieves etc.
- ✓ Printer and scanner
- ✓ Stationeries (pencil, marker, pen, notebook, log sheet, eraser, mm paper, stapler
- ✓ Meter Tapes etc.

4 Laboratory Test Results

4.1 Introduction

All tests were conducted in accordance to the methodologies described in chapter three. In this chapter laboratory test out comes are presented for both natural, cement and lime treated expansive soils. Atterberg limits, linear shrinkage, compaction, unconfined compression for immediate and seven day curing and California bring ratio of three day soaking are types of testes performed. The percentage mix was 2%, 4%, 6%, and 8% for both hydrated lime and Portland cement. As it is stated in table 4.1 below eight pits have been considered in order for focusing the thesis with the location of higher expansiveness potential. The initial test result is as shown in table 4.1 and detail investigation have been conducted from pits one and two as a representative.

4.2 Laboratory Test Results of Untreated Soil Samples

4.2.1 Grain Size Analysis of Untreated Soil

The detailed procedures to conduct grain size analysis is out lined in ASTM material testing manual. For course grained materials, the grain size distribution is determined by passing about 500g soil sample either by wet or dry shaken through a series of sieves placed in the order of decreasing standard opening sizes and a pan at the bottom of the stock. If considerable amount of soil with Silt and clay is retained on the No.200 sieve, it has be washed. Washing is done by taking the No.200 sieve with the soil retained on it and pouring watering through the sieve from a top in the laboratory.

According to ASTM, the distribution of different soil particles in a given soil is determined by a sedimentation process using hydrometer test for soil passing No.200 sieve size. About 50g of oven dryad well pulverized soil was taken and dispersing agent of sodiumhexmetaphoshpate and water was added and sucked for about 18 hours. The soil mixture was poured in to the standard measuring flask and filled it up to one litter reading. Lastly the hydrometer reading was taken at different time intervals, reading temperatures and finally analyzed it to determine the size of clay particles. Detail laboratory result is as an appendix one.

4.2.2 Atterberg Limits of Untreated Soil

The liquid limit value for untreated soil was determined in accordance to AASHTO T89-96. Plastic limit and plasticity index of the soil were determined according to the procedure stated on AASHTO T90-96. This laboratory is performed to determine the plastic and liquid limits of natural soils from the study area. In addition of plastic limit and liquid limit value, the natural soil's linear shrinkage value have also determined for all test pits using BS 1377:part2:1990. Laboratory test results of liquid limit, plastic index and linear shrinkage are shown blew in table 4.1.

4.2.3 Free Swell Values of Untreated Soil

The test is executed by pouring 10 cc. air dried soil passing through sieve No 40 into a graduated cylinder. Then add clean water to the graduated cylinder up to 100 cc calibration point. Allow the soil particles to settle by setting the cylinder aside. The free swell value in percent is then calculated using the following relation (Teferra and Leikun, 1999).

Free swell (%) = $\frac{\mathrm{Vf}-\mathrm{Vo}}{\mathrm{Vo}} * 100\%$ ……………….…...……………………………….….…….4.1

Where V_f and V_o are symbols to notify final and initial volume respectively.

The laboratory test result of untreated sample for all eight pits is shown on the table4.4 below.

4.2.4 Specific Gravity of Untreated Soil

AASHTO: 100-95(1995) manual is followed to determine the specific gravity. 10g oven dried test sample is taken as the stoppered bottle is used in the test procedure. The test were performed for both natural soil samples one and two, and the respective values of specific gravities are as shown in the table 4.1.

4.2.5 Moisture Content of Untreated Soil

To determine the amount of water percent in a soil expressed as percentage of mass of dry soil. This is termed the moisture content of the soil. Moisture content was performed based on BS 1377: part 2:1991 and the laboratory outcome is as shown below in table 4.1.

Table 4.1 Atterberg limit and free swell value of different pit locations in Asen-Dabo town

NO	Pit name	Location (kebele)	Natural Water Content (%)	Specific Gravity (Gs)	Free swell (%)	LL (%)	PI (%)	Linear Shrinkage (%)	Clay content (%)
1	ASP1	03	34.873	2.77	104.6	96.8	60.7	22.14	45
2	Asp2	03	33.121	2.72	102.8	93.6	56	20.5	43.65
3	Asp3	03	35.482	2.71	96.4	90.5	54	16.5	49.547
4	ASP4	02	38.265	2.7	97.67	87.9	52.5	13.5	37.65
5	ASP5	01	42.331	2.68	60	78	48	10.5	40.13
6	ASP6	01	42.521	2.66	55	70	41.2	9.5	34.8
7	ASP7	02	39.854	2.69	87	86.8	50.5	11.56	43.9
8	ASP8	02	40.021	2.71	89.3	88.9	51.6	12.66	48

4.3 Laboratory Test Results of Treated Soil Samples

Soil samples from pit one and two are soils with high expansion potential as shown in chapter five soil classification part. Samples from pit one and two were selected as a representative samples from the above eight pits. The samples were treated with lime and cement separately and their laboratory test result for different engineering property is described below.

4.3.1 Atterberg Limit of Treated Soil

The liquid limit value was determined in accordance to AASHTO T89-96. Plastic limit and plasticity index of the soil were determined according to the procedure stated on AASHTO T90-96. A 250g of air dry and pulverized soil sample was taken and mixed with 2%, 4%, 6%, and 8% of hydrated lime and Portland cement. Soil sample and stabilizer are mixed using hand mixing procedure. Table 4.2 as shown blow summarizes the Atterberg limit test out comes of treated soil. Detailed laboratory result is attached in appendix two.

Table 4.2 Atterberg limit laboratory test results for sample one and two

No	Sample type	Soil sample one			Soil sample two		
		LL (%)	PL (%)	PI (%)	LL (%)	PL (%)	PI (%)
1	Natural soil	96.8	36.1	60.7	93.6	37.1	56.5
2	2% cement + 98% soil	96.1	39.89	56.2	91.9	45.89	46.5
3	4% cement + 96% soil	92.4	50.9	37.4	88.2	51.2	32
4	6% cement + 94% soil	85	55.2	29.8	80.8	56.2	24.6
5	8% cement + 92% soil	80.5	59.2	21.3	76.3	60.2	16.1
6	2% Lime + 98% soil	93	60.1	39.9	89.6	61.1	34.7
7	4% Lime + 96% soil	81.2	60.6	20.6	77	61.6	15.4
8	6% Lime + 94% soil	72.4	61.6	10.8	68.2	62.6	5.6
9	8% Lime + 92% soil	69.5	59.6	9.9	66.5	61.5	4.8

4.3.2 Linear Shrinkage of Treated Soil

The shrinkage limit value is determined in accordance to BS 1377:part2:1990. According to BS, this test is commonly performed as a continuance of liquid limit and plastic limit testes, hence material for test could therefore conveniently be prepared as part of liquid limit tests. Summary of laboratory test result is shown in table 4.3 for samples one and two.

Table 4.3 Laboratory test result of linear shrinkage for sample one and two

No	Sample type	Soil sample one		Soil sample two	
		Oven dry length(mm)	Linear shrinkage (%)	oven dry length(mm)	Linear shrinkage (%)
1	Natural soil	109	22.14	111.3	20.5
2	2% cement + 98% soil	114	18.57	116.2	17
3	4% cement + 96% soil	120	14.3	122.3	12.66
4	6% cement + 94% soil	126.7	9.5	131.6	6
5	8% cement + 92% soil	129.5	7.5	133.4	4.76
6	2% Lime + 98% soil	116.9	16.5	116.5	16.8
7	4% Lime + 96% soil	123.9	11.5	127.4	9
8	6% Lime + 94% soil	129.1	7.8	134.4	4
9	8% Lime + 92% soil	133.3	4.8	136.6	2.4

4.3.3 Free Swell of Treated Soil

This laboratory is performed to determine the free swell value of the treated soils. Free swell laboratory test results are shown below in table 4.4. The test procedure is as stated in part 4.2.2.

Table 4.4 Summary of laboratory test results for free swell of sample one and two

No	Sample type	Soil sample one			Soil sample two		
		V_o	V_f	Free swell	V_o	V_f	Free swell
1	Natural soil	10	21	110	10	20.6	105.5
2	2% cement + 98% soil	10	20	100	10	18.3	83
3	4% cement + 96% soil	10	17	70	10	16.4	64
4	6% cement + 94% soil	10	16.5	65	10	15.8	58
5	8% cement + 92% soil	10	14.5	45	10	13.7	37
6	2% Lime + 98% soil	10	18	80	10.5	18.4	75.5
7	4% Lime + 96% soil	11	17.6	60	11	16.2	47
8	6% Lime + 94% soil	11	16.5	50	11	15.3	39
9	8% Lime + 92% soil	12	16.2	35	12	15.2	27

4.3.4 Compaction Test of Treated Soil

The test was conducted by preparing five specimens for each trial and the method of compaction was light compaction or the standard Procter method. Each compaction trial needs 3Kg sample. The soil sample and percentage stabilizer has mixed using hand mixing technique. Compaction which is also called as moisture density relation was done based on BS 1377: part 4:1990. The summarized maximum dry density and corresponding optimum moisture content laboratory results are shown in table 4.6.

Table 4.5 Summary of laboratory test results for MDD and OMC of sample one and two

No	Sample type	Soil sample one		Soil sample two	
		OMC (%)	MDD (g/cm^3)	OMC (%)	MDD (g/cm^3)
1	Natural soil	36.5	1.236	37	1.17
2	2% cement + 98%soil	38.5	1.224	38.5	1.158
3	4% cement + 96%soil	42	1.216	41	1.145
4	6% cement + 94%soil	45	1.204	44.5	1.14
5	8% cement + 92%soil	46.5	1.196	46	1.13
6	2% Lime + 98% soil	37.5	1.204	40.5	1.15
7	4% Lime + 96% soil	41	1.185	44	1.11
8	6% Lime + 94% soil	46	1.176	47.5	1.1
9	8% Lime + 92% soil	50	1.168	48.5	1.08

4.3.5 Unconfined Compression Strength of Treated Soil

Unconfined compression tests were conducted based on AASHTO T 208-2. The soil sample is mixed with 2%, 4%, 6% and 8% by mass of lime and cement separately. The treated soil was compacted using the standard compaction effort. Samples were extruded from the compaction mold by Shelby tube or cylindrical soil sampler. The extruded soil is sealed with impermeable plastic bug to be cured at room temperature for seven days. Both immediate and seven day cured UCS test were conducted for sample one. The summarized UCS test result is shown as table 4.7. Detailed UCS laboratory result is attached in appendix 4.

Table 4.6 Summary of laboratory test results for UCS for sample one (immediate and seven day curing duration)

No	Sample type	Soil sample one (immediate curing duration)		Soil sample one (7day curing duration)	
		Maximum stress (σ_{max}) Kpa	Corresponding strain (ϵ %)	Maximum stress (σ_{max}) Kpa	Corresponding strain (ϵ %)
1	Natural soil	215.55	5	215.55	5
2	2% cement + 98% soil	299.6	4.5	412	3.2
3	4% cement + 96% soil	379.4	4.2	517	2.93
4	6% cement + 94% soil	449.1	3.7	600	2.6
5	8% cement + 92% soil	500	3.2	763	2.1
6	2% Lime + 98% soil	274.74	3.2	365	3.2
7	4% Lime + 96% soil	361.64	2.7	460.6	2.1
8	6% Lime + 94% soil	387.86	2.1	490.2	1.8
9	8% Lime + 92% soil	434.66	1.6	552	1.3

4.3.6 California Bearing Ratio (CBR) Of Treated Soil

Procedures outlined on Britain standard for testing materials (BS 1377: part 4: 1990) were conducted to determine the California bearing ratio and percent swell values of the natural, lime and cement treated expansive soil. One point CBR test were conducted. For each test trail 6Kg sample passing though sieve size 19mm was taken. The summarized California bearing ratio laboratory results is shown in table 4.8. Detailed laboratory result is attached in appendix 5.

Table 4.7 Summary of laboratory test results for CBR of sample one

No	Sample type	Soil sample one CBR value (%)	Soil sample two CBR value (%)
1	Natural soil	1.15	2.3
2	2% cement + 98% soil	3	4.5
3	4% cement + 96% soil	5.2	7.8
4	6% cement + 94% soil	7	10.5
5	8% cement + 92% soil	17	20
6	2% Lime + 98% soil	2.05	3.1
7	4% Lime + 96% soil	4.05	6
8	6% Lime + 94% soil	5.6	8.4
9	8% Lime + 92% soil	9.75	14.6

5 Results and Discussion

5.1 Soil Classification

5.1.1 Classification of the Expansive Soil Based Plasticity Index

Here the classification is done based on the method by that uses plasticity index as its basis of soil classification. The result of Asen-Dabo expansive soil is presented in table 5.1.

Table 5.1 Classification for degree of expansion using liquid limit and plastic index criteria.

No	Pit name	Liquid limit		Plastic index		Free swell	
		value	Swelling potential	Value	Swelling potential	Value	Expansion percentage
1	Pit-1	96.8	Very high	60.7	Very high	104.6	expansive
2	Pit-2	93.6	Very high	56	Very high	102.8	expansive
3	Pit-3	90.5	Very high	54	Very high	96.4	marginal
4	Pit-4	87.9	Very high	52.5	High	97.67	marginal
5	Pit-5	78	Very high	48	High	60	Marginal
6	Pit-6	75.5	Very high	46.4	High	55	Marginal
7	Pit-7	86.8	Very high	50.5	High	87	marginal
8	Pit-8	88.9	Very high	51.6	High	89.3	marginal

Table 5.1 above refers to the variation of liquid limit from 75.5% to 96.8% while its plasticity index varies from 46.4% to 60.7%, in addition to this its expansion percentage is marginal to expansive. Specific gravity of the soil in the town ranges from 2.77 to 2.66. Natural moisture content is in between 34.873 % and 42.521%.

The researcher have done soil classification based on unified and AASHTO soil classification system and the resulted group of the soil is as shown in table 5.2 and 5.3 below.

Table 5.2 AASHTO soil classification for soil sample one and two

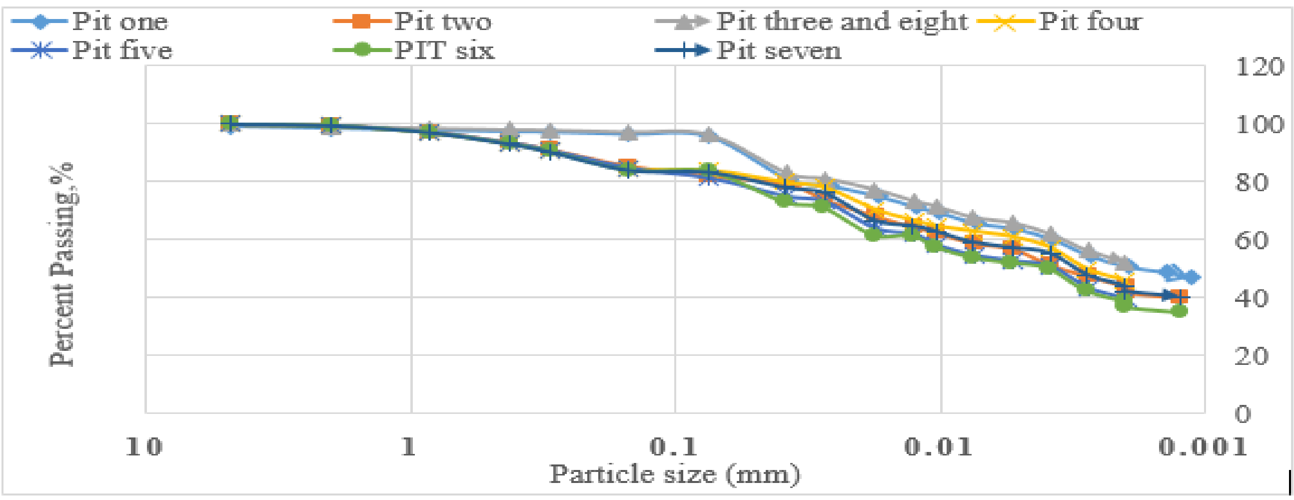
No	Soil type	LL	PL	PI	LL-30	Soil group	GI	Material type
1	Natural soil one	96.8	36.05	60.74	66.8	A-7-5	70	Clay soil
2	Natural soil two	93.6	37.1	56.5	63.6	A-7-5	52	

Table 5.3 Unified classification system for soil sample one and two

No	Soil type	LL	PL	Group symbol	R_{200}	Sf	GF	$\frac{Sf}{Gf}$	Group name
1	Natural soil one	96.8	60.74	CH	4.3928	4.3928	0	>1	Fat clay
2	Natural soil two	93.6	56.5	CH	18.09	18.91	0		

5.1.2 Classification Based on Grain Size Analysis

Particle size and percentage finer relationship for both samples looks like fig 5.1.haydrated Lime and Portland cement addition have not significant effect on grain size characteristics, because they are finer than the natural soil. Hence the researcher have focused only the natural soils grain experiment. Detail tabular analysis of grain analysis is included on appendix 1. According to ASTM the following grain size boundaries are applied.



Gravel	Sand	Silt	Clay
75mm – 4.25mm	4.25mm – 0.075mm	0.075mm – 0.005mm	0.005mm – 0.001mm

Figure 5.1 Percent finer versus particle size relationship for natural soil

5.2 Comparing the Effect of Cement and Lime Addition on Liquid Limit of Expansive Soil

There had been significant decrease on liquid limit value of the sample with addition of lime and cement. Liquid limit value decreased as the amount of lime and cement addition increased. Addition of 2% lime on soil sample one and two decreased the liquid limit value of the expansive soil by 3.8% and 4%, and corresponding cement addition decreased by 0.7 % & 1.7% respectively.

The liquid limit value and degree of expansion relationship is stated on the literature review. The initial liquid limit value of samples one and two has been determined as 96.8% and 93.6% respectively. By adding only 8% lime to sample one and two, the respective liquid limit value has dropped to 69.5% and 66.3%. Adding the same amount of cement to the same sample has dropped the initial liquid limit to 80.5% and 76.3%. Both lime and cement have significant effect on expansion potential, but lime has resulted more effect. Lime and cement content versus liquid limit value of sample one and two are as illustrated in the fig 5.2 and 5.3 shown below.

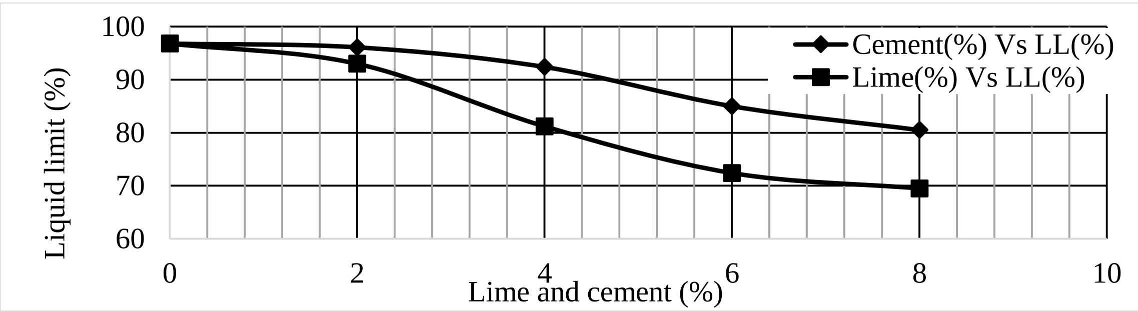


Figure 5.2 Effect of lime and cement addition on liquid limit value of sample one

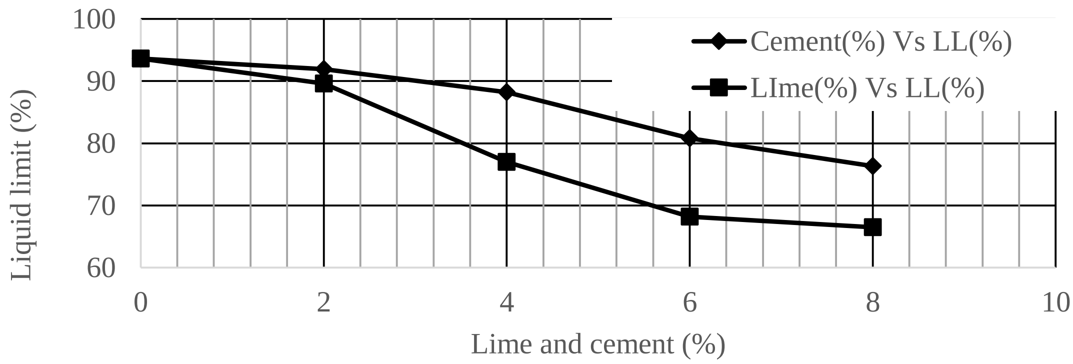


Figure 5.3 Effect of lime and cement addition on liquid limit value of sample two

Fig 2.6 states both soil samples one and two lies into Illite type of expansive soils. Illite clay is one type of expansive clay which can expand highly when get water and shrink when loose its moisture.

5.3 Comparing the Effect of Cement and Lime Addition on Plastic Limit of Expansive Soil

Plastic limit for lime added sample varies in narrow range when we compare cement added samples. Initially lime addition have high effect on plastic limit value on both samples. Addition of 2%, 4%, and 6% lime increases the plastic limit value of sample one, while addition of 8% lime decreased it. Addition of 2% to 8% lime to sample two increased the plastic limit value.

2% to 8% cement addition has increased the plastic limit of both samples one and two. 2% cement addition increased the plastic limit of sample one from its initial value 36.1% to 39.89%, and for sample two from 37.1% to 41.89%. Same percentage of lime addition causes to increase plastic limit value to 60.1% and 60.6% respectively. Lime and cement addition effect on plastic limit value of sample one and two is depicted in the figure 5.4 and 5.5 shown below;

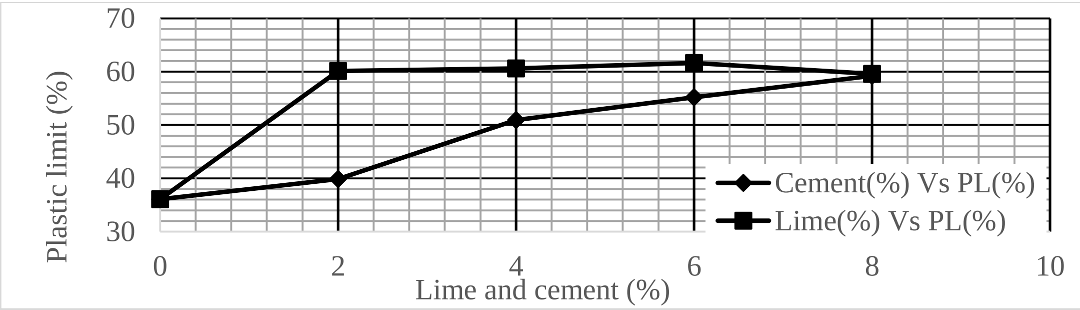


Figure 5.4 Effect of lime and cement addition on plastic limit value of sample one

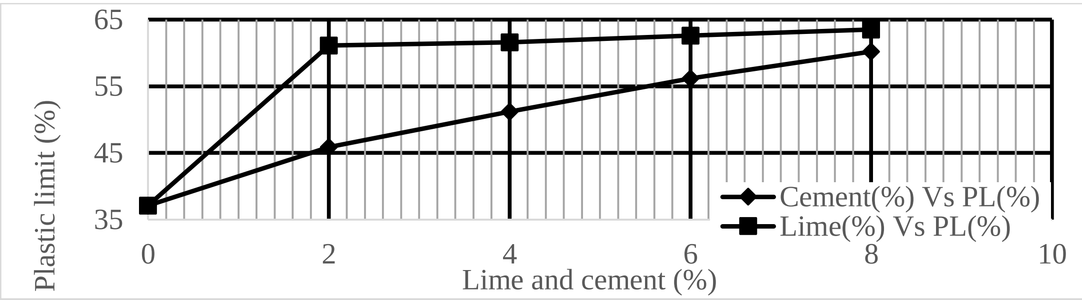


Figure 5.5 Effect of lime and cement addition on plastic limit value of sample two

5.4 Comparing the Effect of Cement and Lime Addition on Plastic Index of Expansive Soil

Addition of cement and lime decreased the plasticity index of both samples significantly. 8% lime addition reduced the plastic index of sample one and two by 50.8% and 51.7% respectively. 8% cement addition dropped the PI value of sample one by 39.4% and sample two by 36.9%.

The plasticity index swelling potential relationship is stated on the literature review. Correlating table 4.2 and 2.3 the initial plasticity index of sample one and two (60.7% and 56.5%) lies on the very high swelling potential category. With only 8% lime addition PI became 9.9 for sample one and 3.8% for sample two. Using equal amount of cement stabilizer has decreased the PI value to 21.3% for sample one and 19.6% for sample two.

8% lime has advanced the expansive soil from very high swelling potential to low swelling potential. Same amount of cement addition advanced swelling potential of the expansive soil from very high swelling potential to medium swelling potential rang. Even though both lime and cement have the capacity to decrease to expression potential of expansive soils, but using lime has resulted high advancement than cement. Lime and cement addition effect on plastic index value of sample one and two is depicted in the figure below;

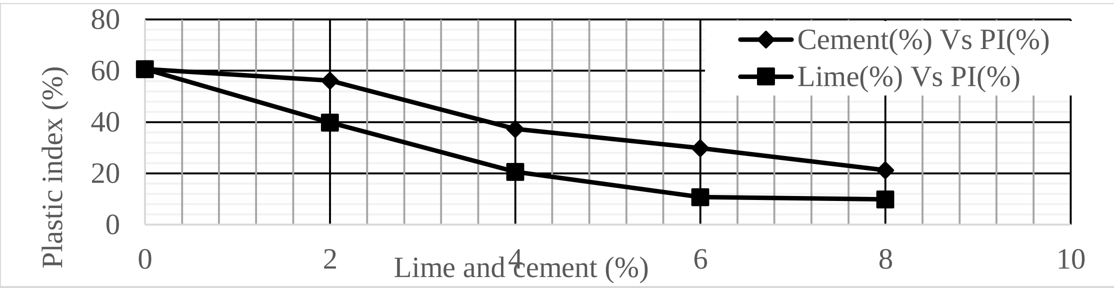


Figure 5.6 Effect of lime and cement addition on plastic index value of sample one

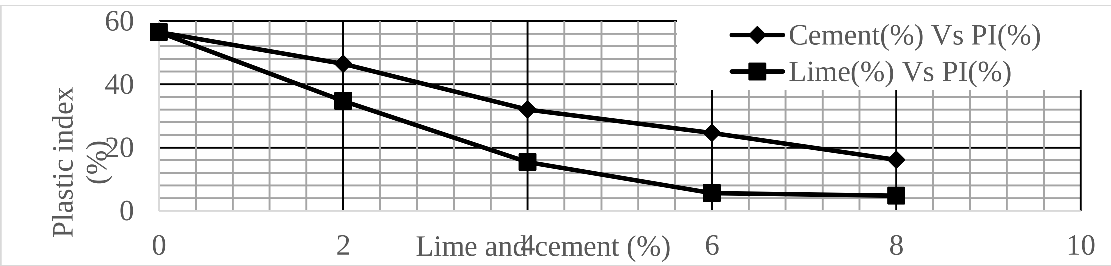


Figure 5.7 Effect of lime and cement addition on plastic index value of sample two

5.5 Comparing the Effect of Cement and Lime Addition on Free Swell of Expansive Soil

Fig 5.8 and 5.9 shown below depicts the effect of Lime and Cement addition on expansive soil free swell value. Free swell value have inversely relation with both lime and cement addition. As the Lime and cement added increases the free swell value of the expansive soil significantly decreased. The free swell reduction effect of lime addition is higher than cement addition.

When the expansive soil was treated with 2% lime, the free swell reduced from its initial value 110% to 80% for sample one and 105.5% to 75.5% for sample two. The same amount of cement addition has reduced the free swell value of sample one and sample two from its initial state to 95% and 85% respectively. The free swell and degree of expansion relationship is stated on literature review. According to table 4.4 and 2.4 the initial free swell value of sample one and sample two lies on problematic expansion category. 8 % lime addition has resulted the free swell of sample one and two to be 38% and 27% respectively while, 8% cement addition resulted in 45% free swell value of sample one and 37% free swell value for sample two. Hence the degree of expansion category of both samples after treated by 8% lime and cement has dropped to non-problematic range.

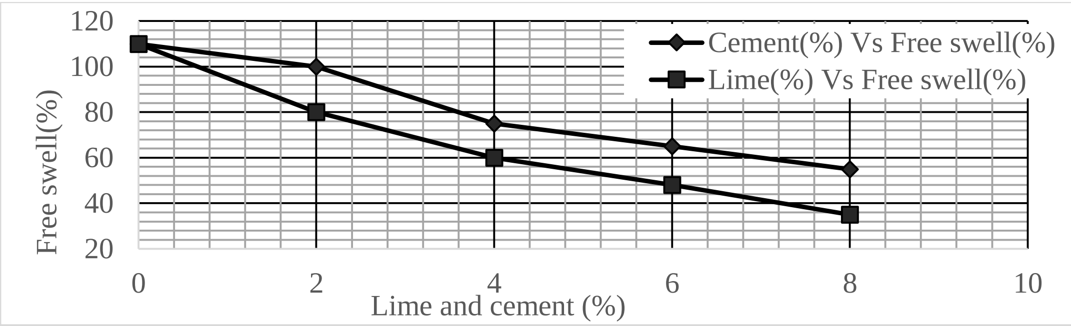


Figure 5.8 Effect of lime and cement addition on free swell value of sample one

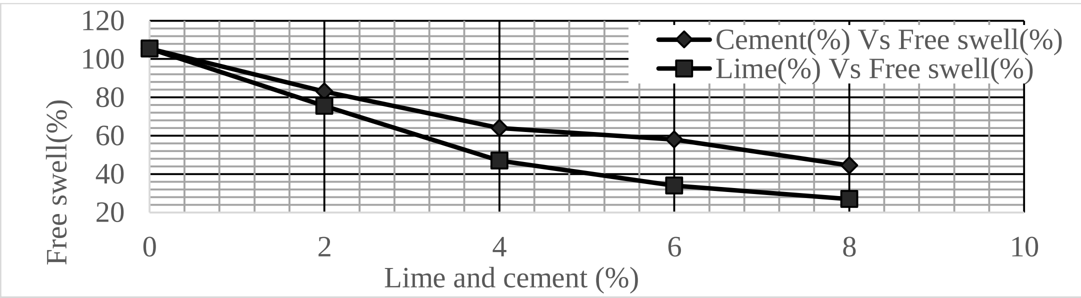


Figure 5.9 Effect of lime and cement addition on free swell value of sample two

5.6 Comparing the Effect of Cement and Lime Addition on Linear Shrinkage of Expansive Soil

The effect of lime and cement addition on linear shrinkage of both sample one and two is shown on fig 5.10 and 5.11 below. The application of lime and cement has resulted significant reduction of linear shrinkage value. The linear shrinkage value of both samples has shown inverse relation with lime and cement addition. Lime addition has resulted in higher linear shrinkage reduction than cement addition. The linear shrinkage value of sample one gradually decreased from 22.14% to 4.8% for 8% lime addition and from 22.14% to 7.5% for the same amount of cement addition. After 8% Lime had added the reduced linear shrinkage value of sample two has become 2.4% and for the same amount of cement addition, linear shrinkage of sample two has reduced to 4.76% from its 20.5% initial value.

The linear shrinkage and degree of expansion relationship is stated in the literature review. Accordingly the initial linear shrinkage value of both sample one and sample two lies on critical expansion range. Sample one has changed its critical expansion behavior to none critical range by addition of 8% lime and to marginal range by adding the same amount of cement. Only 8% lime and cement addition has been enough to change the critical expansion category of sample two to none critical range.

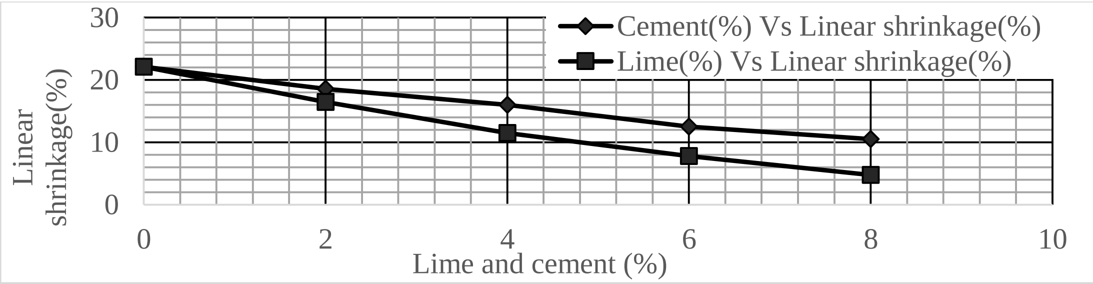


Figure 5.10 Effect of lime and cement addition on linear shrinkage value of sample one

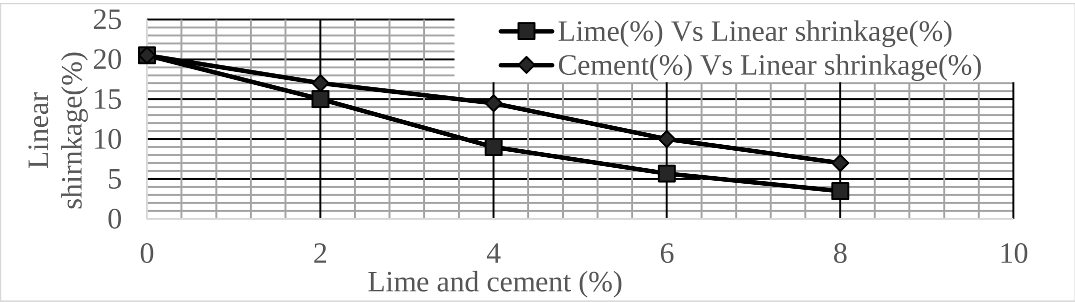
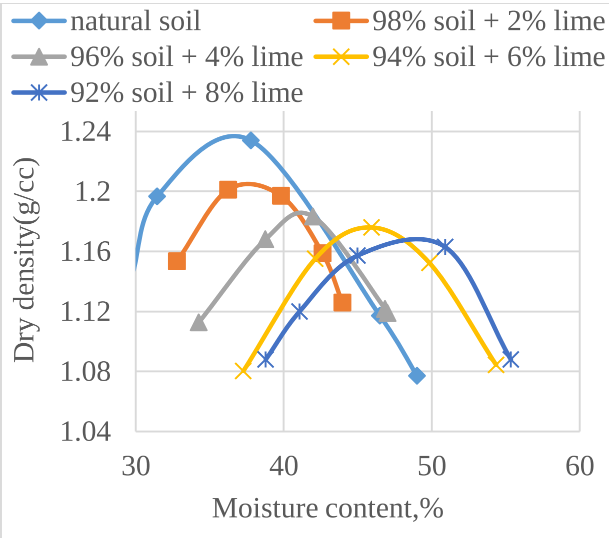


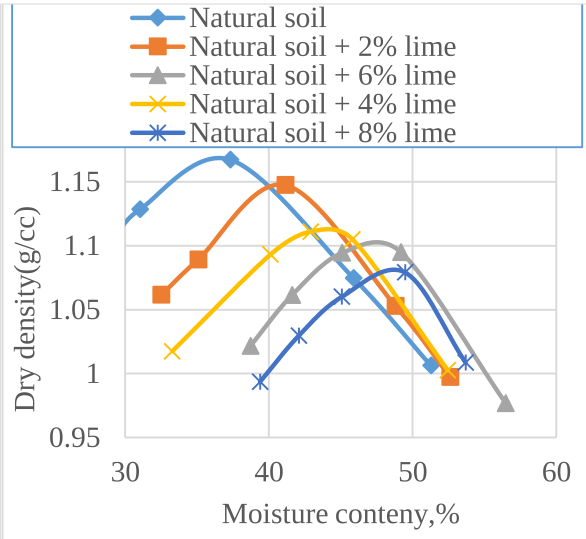
Figure 5.11 Effect of lime and cement addition on linear shrinkage value of sample two

5.7 Comparing the Effect of Cement and Lime Addition on Maximum Dry Density and Optimum Moisture Content Relationship of Expansive Soil

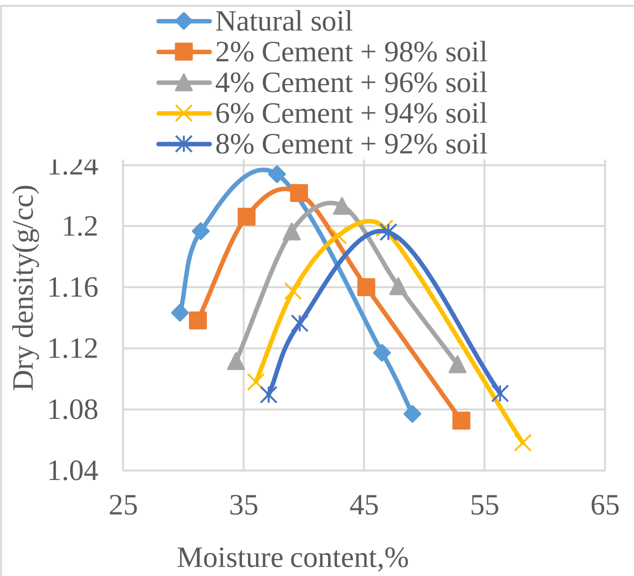
Fig 5.12 shown below depicts the compaction curves for varying lime and cement content of both samples. Detailed laboratory test results are attached in appendix 3.



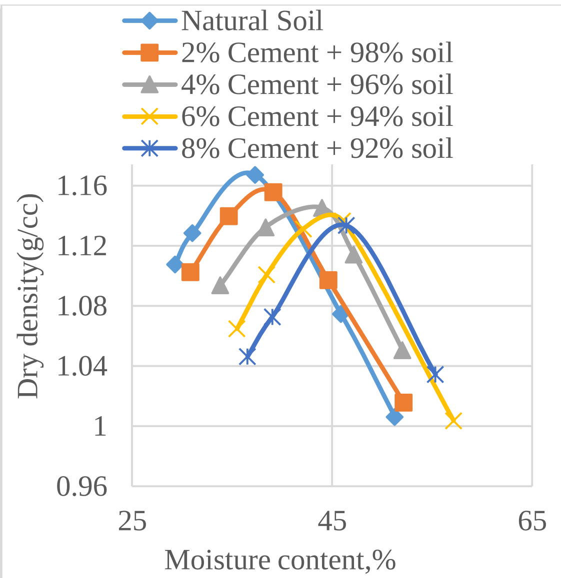
A)



(B)



(C)



(D)

Figure 5.12 Lime and cement addition effect on moisture density relationship A) soil sample one and lime addition B) soil sample two and lime addition C) soil sample one and cement addition and E) soil sample two and cement addition.

5.7.1 Effect of Lime and Cement Addition on Maximum Dry Density (MDD)

Fig 5.13 and 5.14 shows the effect of cement and lime addition on maximum dry density of both samples. The maximum dry density has decreased with addition of lime and cement stabilizers. According Didier (2000), Lime and Cement addition decreases maximum dry density of expansive soils due to Cation exchange and short time pozzolanic reaction takes placed between the stabilizers and the soil.

Lime addition has resulted in higher MDD reduction than cement addition. As it is shown in figure below, MDD value of sample one has gradually decreased from 1.236 g/cc to 1.168 g/cc for 8% Lime addition and from 1.236 g/cc to 1.196 g/cc for the same amount of cement addition. 8% lime addition has reduced MDD value of sample two to 1.08 g/cc, and for the same amount of cement addition MDD of sample two has reduced to 1.13 g/cc from its initial value of 1.17 g/cc. The following figure clearly compares lime and cement addition effect on MDD of expansive soil;

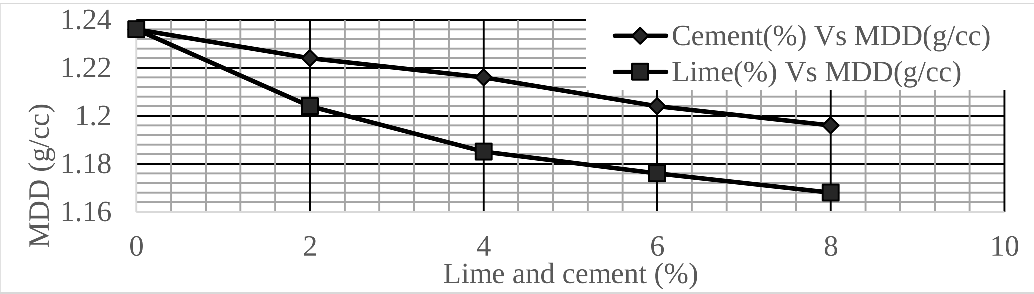


Figure 5.13 Effect of lime and cement addition on maximum dry-density value of sample one

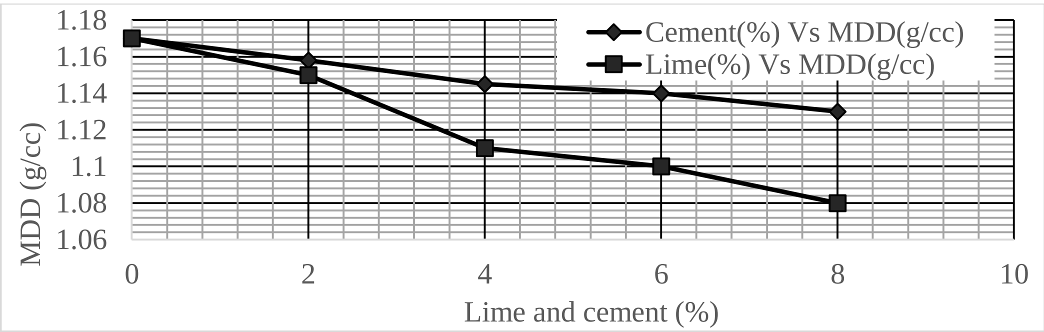


Figure 5.14 Effect of lime and cement addition on maximum dry-density value of sample two

5.7.2 Effect of Lime and Cement Addition on Optimum Moisture Content (OMC)

As it is observed from fig 5.15 and 5.16 shown below, when lime and cement content is increased optimum water content has been decreased. From table 4.6 the initial optimum water content of sample one is 36.5% and of sample two is 37%. By addition of 8% lime to sample one and two the initial optimum moisture content has increased to 50% and 48.5% respectively. The same amount of cement percentage increment has resulted 46.5 OMC of sample one and OMC of sample two is increased to 46%. 8% Lime addition has resulted higher OMC reduction than the same percentage of cement addition. The following fig clearly compares lime and cement addition effect on OMC of expansive soil;

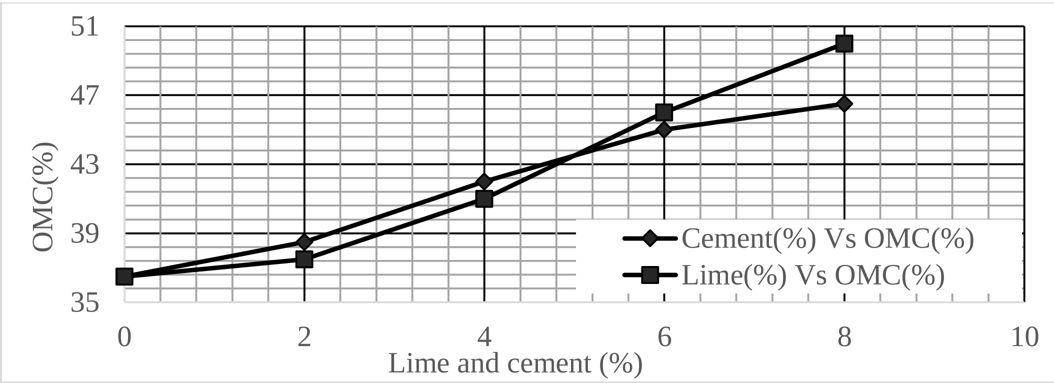


Figure 5.15 Effect of lime and cement addition on optimum moisture content value of sample one

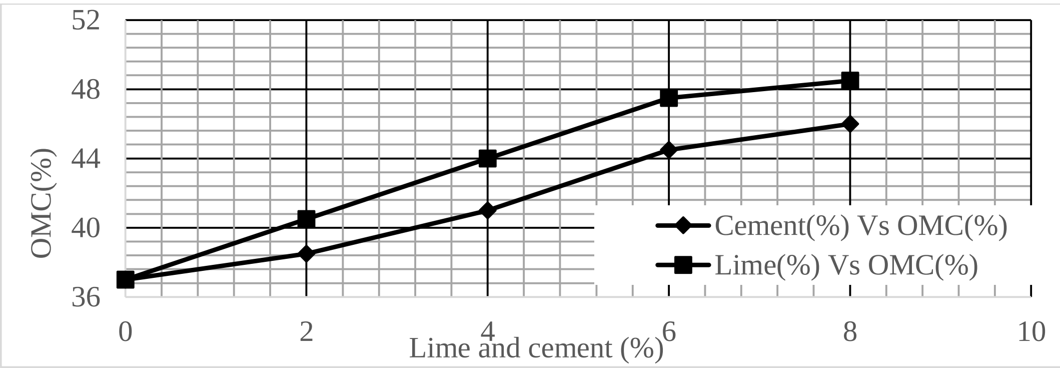


Figure 5.16 Effect of lime and cement addition on optimum moisture content value of sample two

5.8 Comparing the Effect of Cement and Lime Addition on Unconfined Compression Strength of Effective Soil

In this research the stress-strain behavior of soil sample one specimen treated with lime and cement admixtures, different proportions and curing times has been investigated based on unconfined compression test. For comparison purpose the stress-strain behavior of stabilized expansive soil sample one specimen at its seven day and immediate curing time duration are represented in Fig 5.17 and 5.18.

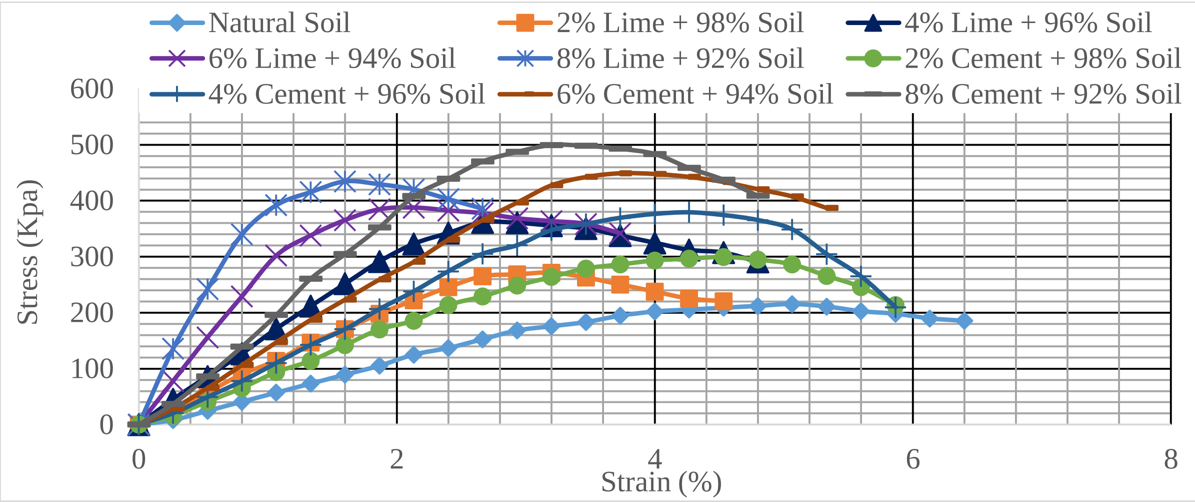


Figure 5.17 Effect of lime and cement addition on stress and strain value of sample one (immediate curing duration)

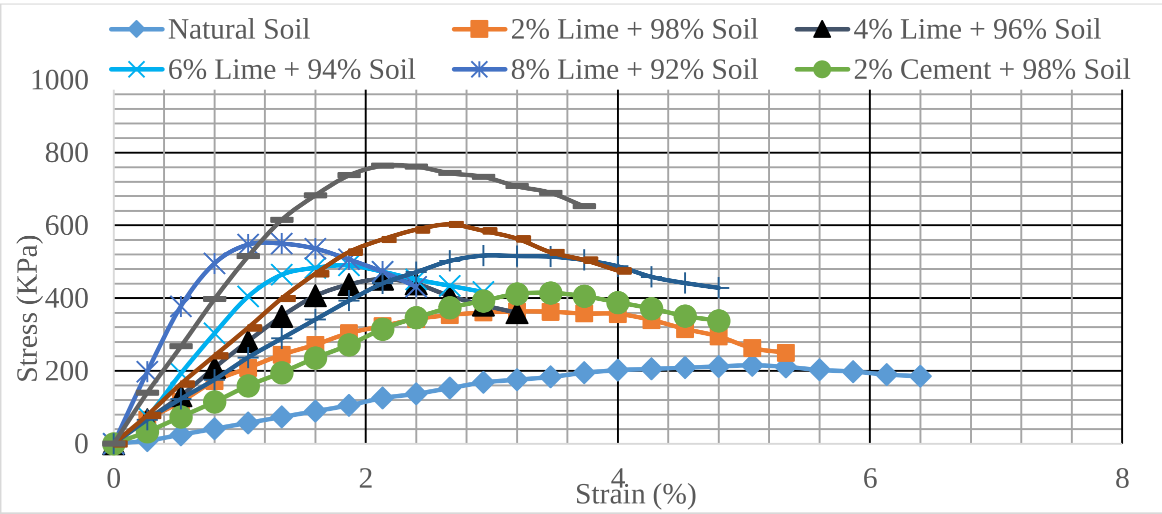


Figure 5.18 Effect of lime and cement addition on stress and strain value of sample one (seven day curing duration)

At its natural state the maximum compressive strength of untreated expansive soil has reached 215.6 Kpa with a strain rate of 5%. Upon treatment with considerable amount of 8% lime the unconfined compressive strength has increased to 434 Kpa with corresponding strain rate of 1.6%. The shear failure mode of the specimen was close to brittle failure i.e. no true failure plane is observed. This indicates lime stabilization increased stiffness, peak strength and brittleness. The same percentage of cement addition has increased the immediate maximum unconfined compressive strength of the specimen to 500 Kpa at a strain rate of 3.2%.

Furthermore unconfined compression strength has been increased with increased curing period. Unconfined compression strength for seven day curing period have been investigated in this research, irrespective of kind of admixtures all treated specimens has gained significant strength and linear reduction in strain. For seven day curing time and 8% lime addition the initial unconfined compression strength of sample one has increased to 549.8 Kpa and its corresponding axial strain has decreased to 1.3%. For the same curing time duration and sample, 8% cement addition produces 763.6 Kpa unconfined compressive strength value corresponding to 2.1% strain. Higher percentage of lime and cement addition produced higher unconfined compressive strength and lower strain value, but cement addition improves more the strength of the expansive soil. Fig 5.19 and 5.20 shown below depicts lime and cement addition effect on stress and strain value of expansive soil;

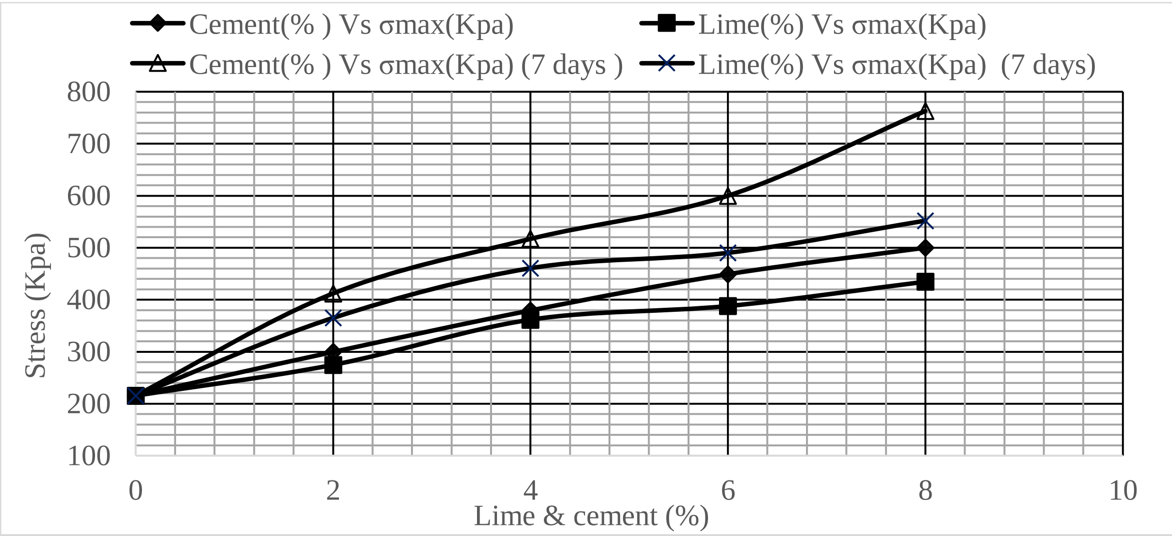


Figure 5.19 Effect of lime and cement addition on stress behavior of sample one and curing time effect

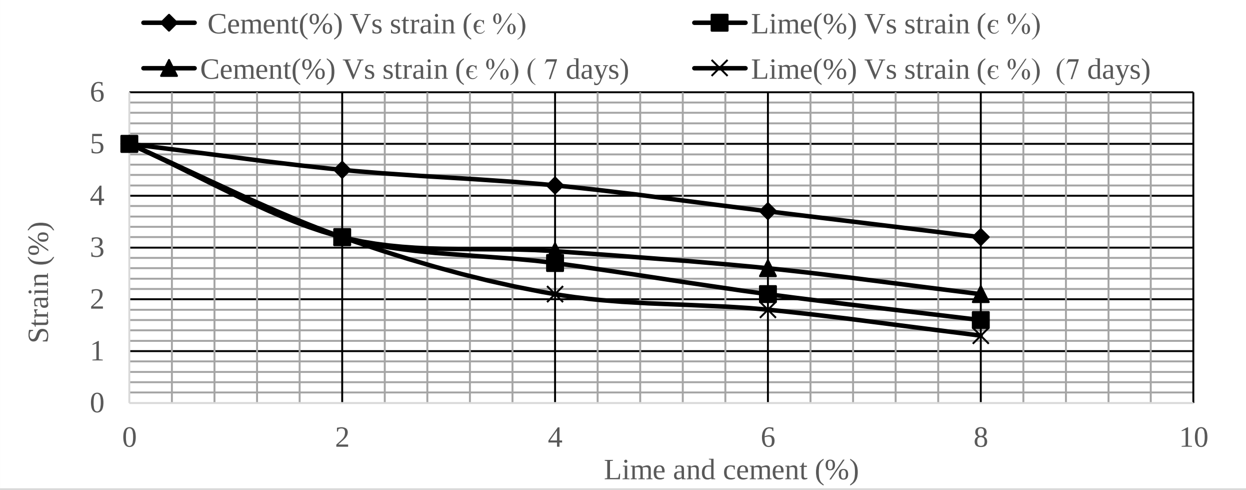


Figure 5.20 Effect of lime and cement addition on axial strain behavior of sample one and curing time effect

5.9 Comparing the Effect of Cement and Lime Addition on California Bearing Ratio (CBR) of Expansive Soil

California bearing ratio for both sample one and two with different lime and cement addition is shown in table 4.8. Cement and lime addition percentage has directly proportional with California bearing ratio value of the sample. California bearing ratio value of the sample specimen has gradually increased with increasing cement and lime percentage as shown in the fig.5.21 and 5.22. The maximum value of California bearing ratio 9.75% for sample one 14.6% for sample two was obtained for 8% of lime addition. The same amount of cement content has been resulted California bearing ratio value of 17% and 20% for samples one and two respectively. Detail graphical representation of lime and cement addition effect is illustrated in appendix 5.

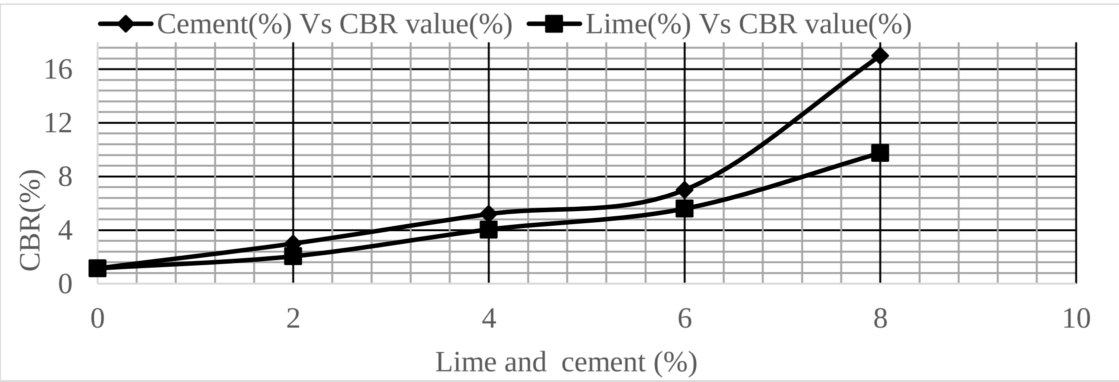


Figure 5.21 Effect of lime and cement addition on CBR value of sample one

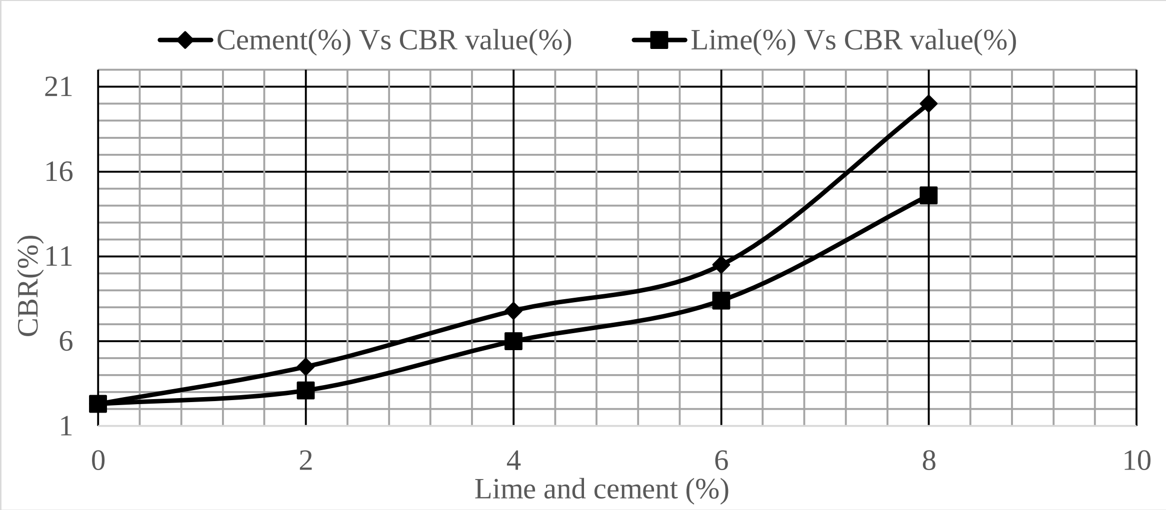


Figure 5.22 Effect of lime and cement addition on CBR value of sample two

6 Conclusion and Recommendation

6.1 Conclusion

1 Lime and cement treated expansive soil exhibits reduction in liquid limit and plastic index values. Higher lime and cement addition results in larger liquid limit and plastic index value reduction. Comparing both additives, lime addition has resulted more reduction in liquid limit and plastic limit than cement addition. Negligible swelling potential was exhibited by lime and cement treated soil even though lime is more effective in expansive swelling potential reduction.

2 There is no clear relation between lime addition and plastic limit value of expansive soil, as 2% to 6% lime addition in sample one increases plastic limit value of the soil and 8% lime addition on the same soil decreases its plastic limit. Lime Addition is effective at the first 2% addition, while cement is effective from 2% to 8% addition. Farther lime addition on the expansive soil have small effect. Plastic limit of both samples has increased with increasing cement percentage addition.

3 The test result of treated specimen indicates that linear shrinkage value decreases with increasing the quantity of lime and cement.it is possible to neglect the expansion potential of the expansive soil through 8% lime or cement addition. Lime addition is effective in linear shrinkage reduction than cement addition.

4 Lime addition has resulted higher free swell reduction than the same amount of cement addition. 8% lime or cement addition had dropped the degree of expansion from problematic to non-problematic category

5 Compaction characteristics of both samples under treatment of lime or cement had showed a continuous decrease in maximum dry density and increase in optimum moisture content. Lime addition has resulted higher decrement in maximum dry density and increment in optimum moisture content when we compare with the same amount of cement addition on the same sample.

6 The increase in hydrated Lime or Portland cement percentage induced greater initial strength development at lower strain to peak strength. Comparing hydrated lime and Portland cement addition effect on unconfined compression strength value of the soil indicates cement addition resulted more unconfined compression strength increment than lime addition and Lime addition had resulted greater strain reduction than cement addition.

7 The length of curing duration had significant effect on stress-strain behavior of both lime or cement treated expansive soil. 7 day curing duration for 8% cement treated soil has resulted 52.6% maximum stress increment from uncured treated soil. The same percentage of lime addition with seven day curing duration resulted only 26.99% increment in maximum stress from the uncured treated soil

8 CBR value of expansive soil increases almost linearly with addition of lime or cement. Example, 8% Lime addition on sample one exhibits 8.6% CBR increment while equal amount of cement addition has resulted 15.85% CBR increment. Cement addition resulted in higher CBR advancement than lime addition.

6.2 Recommendation

1. Those who are involving in construction industries in the study area can use lime and cement to reduce the effect of expansiveness of the soil.
2. In order to know the adverse negative effects on the environment due to the stabilization of soil with both lime and cement detailed investigation should be carried out.
3. Effect of using cement or lime as a stabilizer on consolidation characteristics should be focused on farther studies. In addition economic feasibility study of using lime or cement as a stabilizer needs to be researched.

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Detail Laboratory Results

Appendix 1: Grain Size Analysis

Table 1.1 Grain size (wet sieve) analysis test result for sample one

sieve size (mm)	soil retained (g)	percent retained	percent passing
4.75	4.245	0.849	99.151
2	3.312	0.6624	98.4886
0.85	3.399	0.6798	97.8088
0.425	2.539	0.5078	97.301
0.3	0.909	0.1818	97.1192
0.15	3.501	0.7002	96.419
0.075	4.059	0.8118	95.6072
Pan	1.805	0.361	0

Table 1.2 Hydrometer test result for sample two

Hydrometer number	152H	Weight of sample	50g
Specific gravity	2.77	Zero correction	+6
Dispersing agent	sodium hexametaphosphate	Meniscus of correction	+1

elapsed Time (mn)	Actual hydrometer reading	Temperature (O_c)	Hydrometer reading corrected for meniscus	Effective hydrometer depth(L)	k	D (mm)	CT	a	Corrected hydrometer reading.	% finer p	% adjusted finer (PA)
1	49	21	50	8.3	0.01291	0.037181	0.2	0.975	43.2	84.24	80.539
2	48	21	49	8.4	0.01291	0.026466	0.2	0.975	42.2	82.29	78.675
5	46	21	47	8.8	0.01291	0.01717	0.2	0.975	40.2	78.39	74.946
10	44	21	45	9.1	0.01291	0.012316	0.2	0.975	38.2	74.49	71.217
15	43	21	44	9.2	0.01291	0.010109	0.2	0.975	37.2	72.54	69.353
30	41	21	42	9.6	0.01291	0.007307	0.2	0.975	35.2	68.64	65.624
60	40	21	41	9.7	0.01291	0.00519	0.2	0.975	34.2	66.69	63.744
12	38	21	39	10.1	0.01291	0.003744	0.2	0.975	32.2	62.79	60.076
240	35	21	36	10.6	0.01291	0.002711	0.2	0.975	29.2	56.94	54.438
480	33	21	34	10.9	0.01291	0.001949	0.2	0.975	27.2	53.04	50.710
960	32	21	33	11.1	0.01291	0.001394	0.2	0.975	26.2	51.09	48.845
1440	31	21	32	11.2	0.01291	0.001139	0.2	0.975	25.2	49.14	46.981

Table 1.3 Grain size (wet sieve) analysis test result for sample Two

sieve size	soil retained	percent retained	percent passing
4.75	0	0	100
2	2.403	0.4806	99.5194
0.85	11.049	2.2098	97.3096
0.425	18.435	3.687	93.6226
0.3	11.939	2.3878	91.2348
0.15	29.127	5.8254	85.4094
0.075	17.476	3.4952	81.9142
pan	3587	717.4	0

Table 1.4 Hydrometer test result for sample two

Hydrometer number	152H	Weight of sample	2.61 50g
Specific gravity 2.54	2.72	Zero correction	+6
Dispersing agent	sodium hexametaphosphate	Meniscus of correction	+1

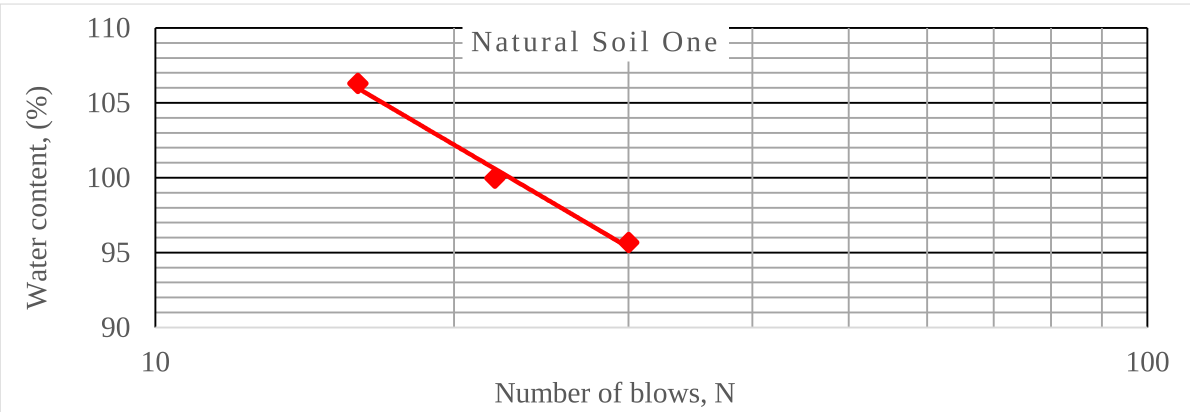
elapsed Time (mn)	Actual hydrometer reading	Temperature (Oc)	Hydrometer reading corrected for meniscus	Effective hydrometer depth(L)	k	D (mm)	CT	a	Corrected hydrometer reading.	% finer p	% adjusted finer (PA)
1	48	21	49	8.3	0.014002	0.040326	0.2	1.02	42.2	86.088	82.306
2	45	21	46	8.8	0.014002	0.028704	0.2	1.02	39.2	79.968	76.455
5	42	21	43	9.2	0.014002	0.018623	0.2	1.02	36.2	73.848	70.604
10	40	21	41	9.6	0.014002	0.013358	0.2	1.02	34.2	69.768	66.703
15	39	21	40	9.7	0.014002	0.010964	0.2	1.02	33.2	67.728	64.752
30	37	21	38	10.1	0.014002	0.007925	0.2	1.02	31.2	63.648	60.852
60	36	21	37	10.2	0.014002	0.005629	0.2	1.02	30.2	61.608	58.901
12	33	21	34	10.7	0.014002	0.004061	0.2	1.02	27.2	55.488	53.050
240	31	21	32	11.1	0.014002	0.00294	0.2	1.02	25.2	51.408	49.149
480	29	21	30	11.4	0.014002	0.002114	0.2	1.02	23.2	47.328	45.248
960	28	21	29	11.5	0.014002	0.002114	0.2	1.02	22.2	45.288	43.298
1440	27	21	28	11.7	0.014002	0.001247	0.2	1.02	21.2	43.248	41.348

Appendix 2:- Atterberg Limits For Different Lime and Cement Addition to the Expansive Soil

2.1 Sample One

Table 2.1.1 Atterberg limits determination for natural soil of sample one

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	A	452-2	HCS1	4	3rd A	6
Mass of can, W1 (g)	18.573	17.383	17.577	6.658	5.566	6.326
Mass of can + moist soil, W2 (g)	43.029	42.024	41.468	12.579	11.341	12.531
Mass of can + dry soil, W3 (g)	31.072	29.706	29.158	10.994	9.828	10.885
Moisture content, W (%)	95.663	99.95	106.294	36.571	35.499	36.104
Number of blows, N	30	22	16			

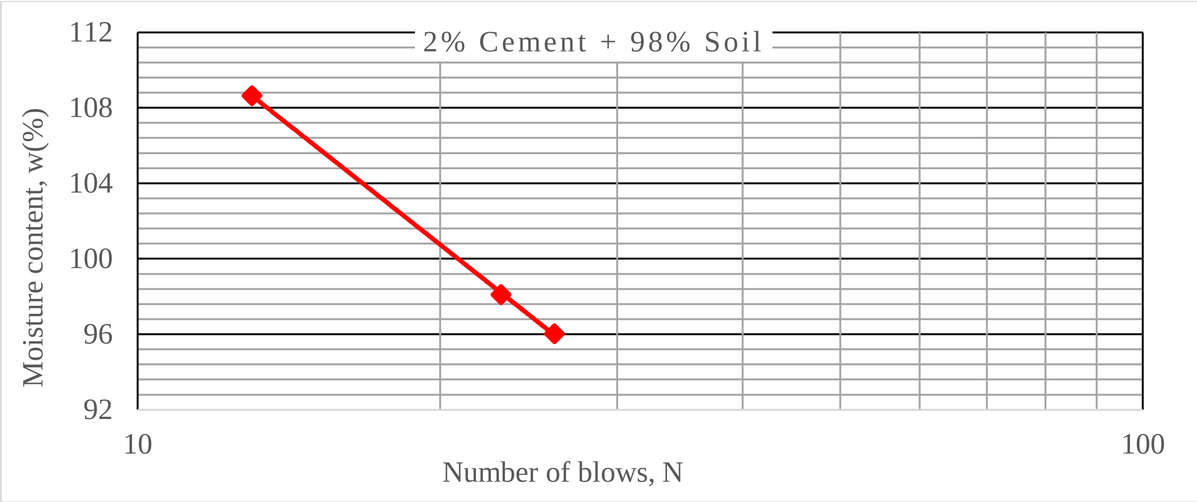


LL=96.1% PL=36.1% PI=56.21%

Fig 2.1.1 liquid limit for natural soil

Table 2.1.2 Atterberg limits determination for 2% cement + 98% soil of sample one

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	sta 25	NC22	ATR 2-2	F1	Tc2	sta 0+15
Mass of can, W1 (g)	17.068	17.595	17.157	6.1	6.338	6.535
Mass of can + moist soil, W2 (g)	37.627	38.397	39.822	11.972	12.562	12.863
Mass of can + dry soil, W3 (g)	27.556	28.096	28.02	10.297	10.719	11.13
Moisture content, W (%)	96.024	98.096	108.644	39.909	42.068	37.714
Number of blows, N	26	23	13			

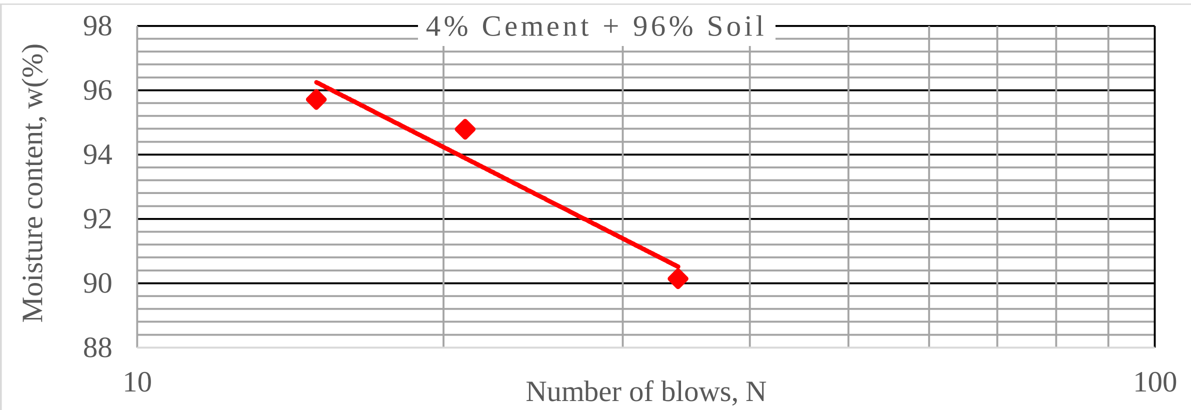


LL=96.1% PL=39.89% PI=56.21%

Fig 2.1.2 Liquid limit for 2% cement + 98% soil of sample one

Table 2.1.3 Atterberg limits determination for 4% cement + 96% soil of sample one

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	G73	T41	HC43	c	D2	T6
Mass of can, W1 (g)	17.685	17.696	17.697	5.821	5.623	6.1
Mass of can + moist soil, W2 (g)	41.997	40.451	40.966	12.235	12.523	12.466
Mass of can + dry soil, W3 (g)	30.471	29.378	29.586	9.831	10.382	10.408
Moisture content, W (%)	90.145	94.786	95.718	59.950	44.988	47.771
Number of blows, N	34	21	15			

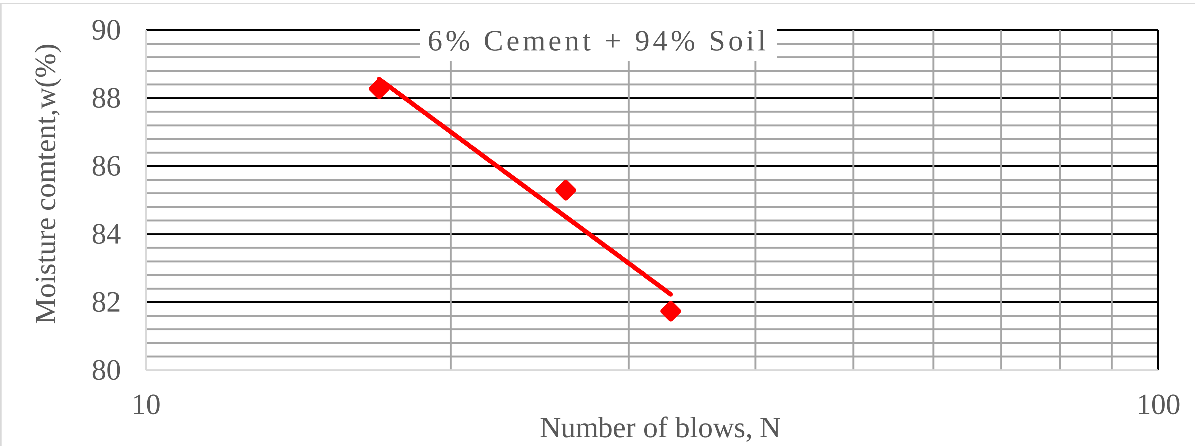


LL=92.4% PL=50.1% PI=42.3%

Fig 2.1.3 liquid limit for 4% cement + 96% soil of sample one

Table 2.1.4 Atterberg limits determination for 6% cement + 94% soil of sample one

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	63	T1	AFETsC1	13	S5	17
Mass of can, W1 (g)	17.336	17.518	17.363	6.365	5.995	5.895
Mass of can + moist soil, W2 (g)	46.933	41.943	48.977	13.385	12.047	11.935
Mass of can + dry soil, W3 (g)	33.622	30.7	34.154	10.902	9.923	9.75
Moisture content, W (%)	81.732	85.290	88.279	54.727	54.0733	56.679
Number of blows, N	33	26	17			

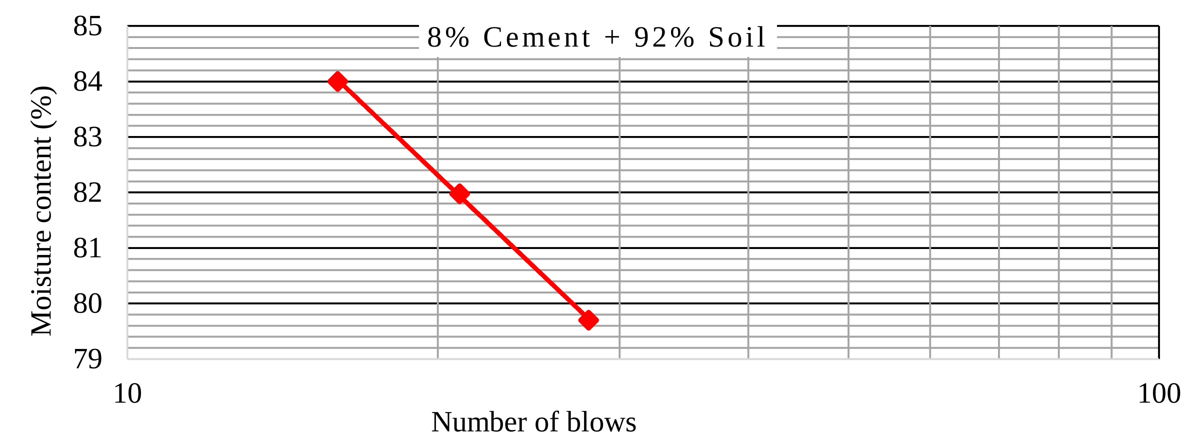


LL=85% PL=55.2% PI=29.8%

Fig 2.1.4 liquid limit for 6% cement + 94% soil of sample one

Table 2.1.5 Atterberg limits determination for 8% cement + 92% soil of sample one

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	T3	17	1A	A2	DB1	N2
Mass of can, W1 (g)	17.874	16.958	17.621	5.773	6.404	6.17
Mass of can + moist soil, W2 (g)	44.335	40.648	39.878	14.506	13.805	14.509
Mass of can + dry soil, W3 (g)	32.599	29.976	29.717	11.251	11.055	11.409
Moisture content, W (%)	79.701	81.978	84.003	59.4195	59.127	59.171
Number of blows, N	28	21	16			

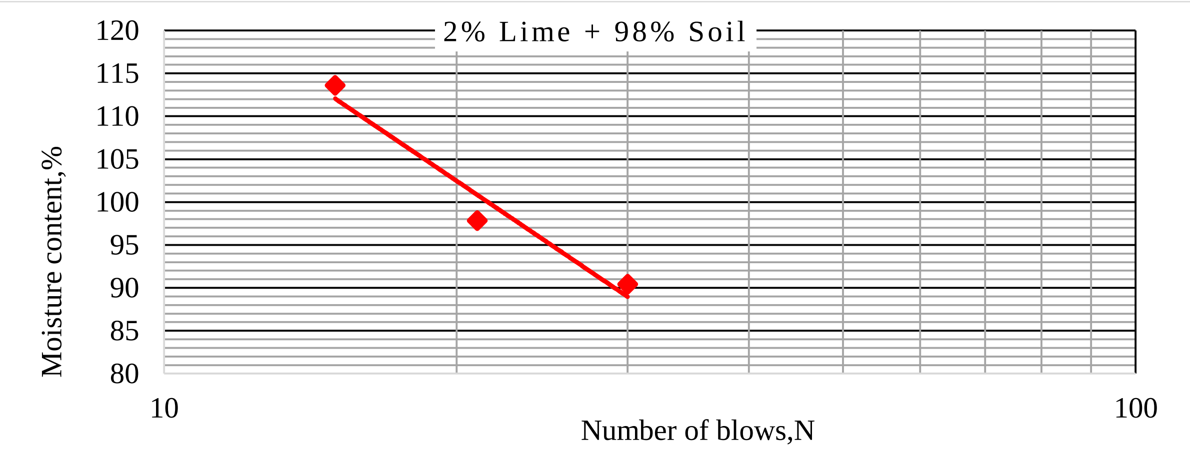


LL=80.5% PL=59.2% PI=21.3%

Fig 2.1.5 Liquid limit for 8% cement + 92% soil of sample one

Table 2.1.6 Atterberg limits determination for 2% Lime + 98% soil of sample one

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	13	T3C1	G53	T6	K2	N4
Mass of can, W1 (g)	18.382	17.4	17.437	6.102	7.888	5.99
Mass of can + moist soil, W2 (g)	43.545	42.63	45.516	12.888	13.897	13.439
Mass of can + dry soil, W3 (g)	31.596	30.152	30.583	10.407	11.6	10.62
Moisture content, W (%)	90.42682	97.851	113.593	57.630	61.880	60.885
Number of blows, N	30	21	15			

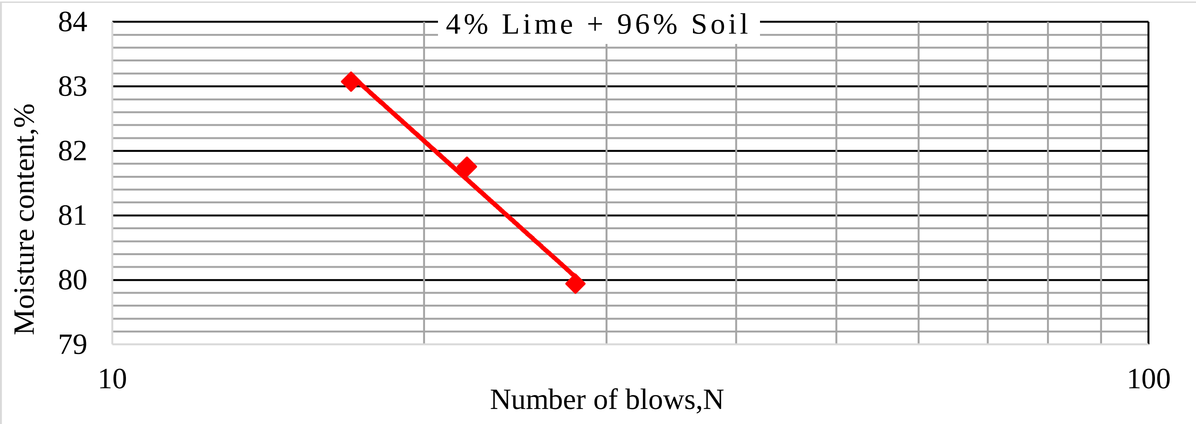


LL=93% PL=60.1% PI=39.9%

Fig 2.1.6 Liquid limit for 2% Lime + 98% soil of sample one

Table 2.1.7 Atterberg limits determination for 4% Lime + 96% soil of sample one

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	T3D2	F3	G-3-2	R3	D2	DB
Mass of can, W1 (g)	19.266	17.28	24.4727	6.44	5.628	6.407
Mass of can + moist soil, W2 (g)	43.526	41.101	53.057	13.805	11.844	12.703
Mass of can + dry soil, W3 (g)	32.748	30.386	40.086	11.05	9.489	10.32
Moisture content, W (%)	79.943	81.756	83.076	59.761	60.994	60.899
Number of blows, N	28	22	17			

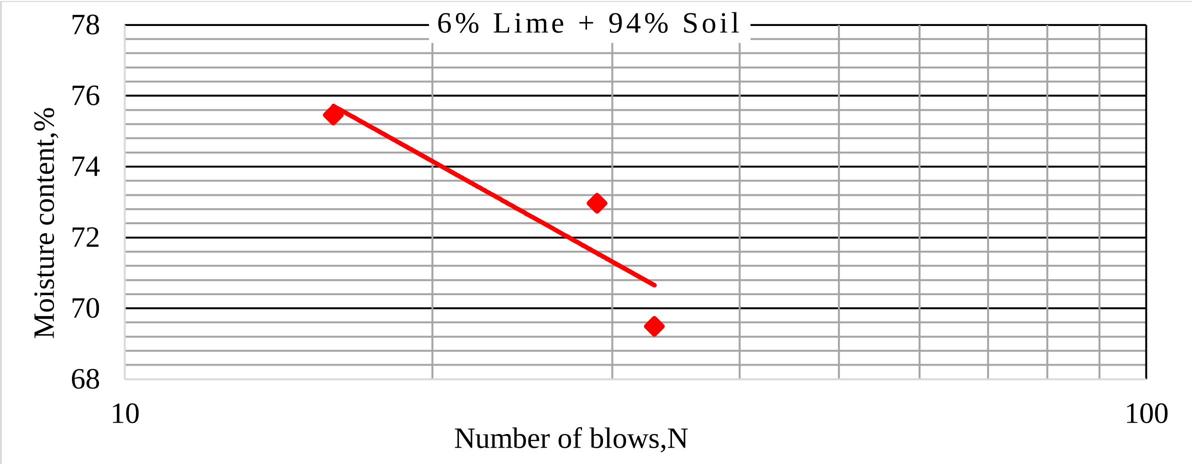


LL=81.2% PL=60.6% PI=20.6%

Fig 2.1.7 Liquid limit for 4% Lime + 96% soil of sample one

Table 2.1.8 Atterberg limits determination for 6% Lime + 94% soil of sample one

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	STA-4	D2	A1	LLB	3	6
Mass of can, W1 (g)	17.573	18.395	16.708	5.519	5.899	5.262
Mass of can + moist soil, W2 (g)	51.652	45.923	42.77	10.164	12.555	13.25
Mass of can + dry soil, W3 (g)	37.68	34.31	31.56	8.44	9.98	10.17
Moisture content, W (%)	69.48824	72.968	75.46	59.020	63.0972	62.56
Number of blows, N	33	29	16			

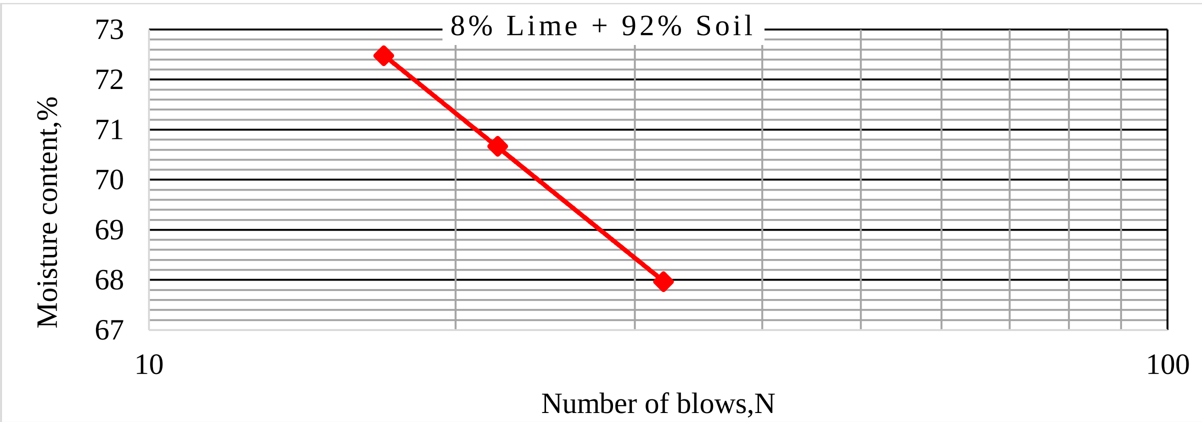


LL=72.4% PL=61.6% PI=10.8%

Fig 2.1.8 Liquid limit for 6% Lime + 94% soil of sample one

Table 2.1.9 Atterberg limits determination for 8% Lime + 92% soil of sample one

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	G71	LC22	HC42	L3	A1	T
Mass of can, W1 (g)	17.959	17.223	17.522	6.161	6.065	6.454
Mass of can + moist soil, W2 (g)	45.637	43.994	45.8052	13.162	12.322	13.756
Mass of can + dry soil, W3 (g)	34.437	32.909	33.92	10.544	9.986	11.035
Moisture content, W (%)	67.969	70.668	72.4795	59.730	59.576	59.397
Number of blows, N	32	22	17			



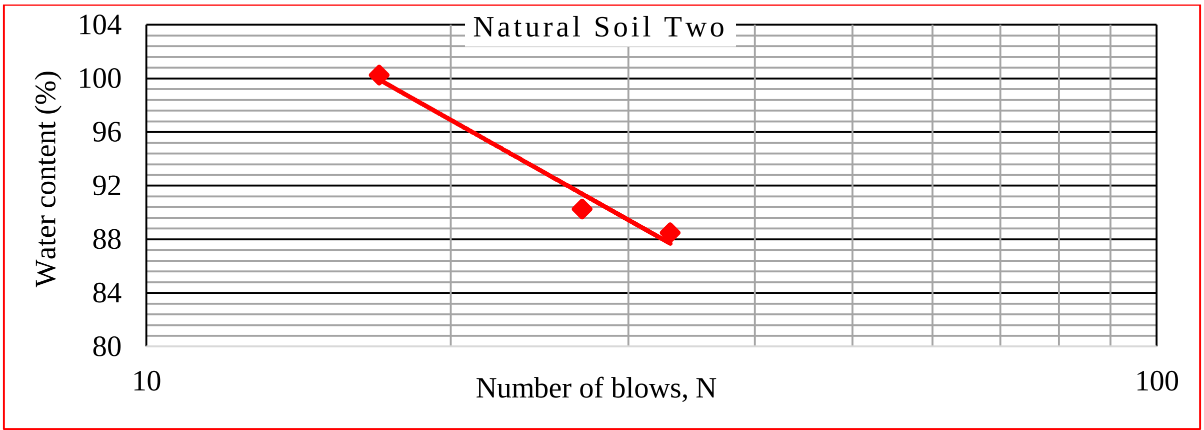
LL=69.5% PL=59.6% PI=9.9%

Fig 2.1.9 Liquid limit for 8% Lime + 92% soil of sample one

2.2 Sample Two

Table 2.2.1 Atterberg limits determination for natural soil of sample Two

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	s2-l0	H	C	K	G-5	G7
mass of can, W1 (g)	17.671	17.67	18.165	6.012	5.997	5.908
mass of can + moist soil, W2 (g)	43.954	42.751	44.669	12.78	12.673	12.531
Mass of can + dry soil, W3 (g)	31.616	30.853	31.401	10.997	10.678	10.896
Moisture content, W (%)	88.476	90.252	100.245	35.767	42.619	32.778
Number of blows, N	33	27	17			

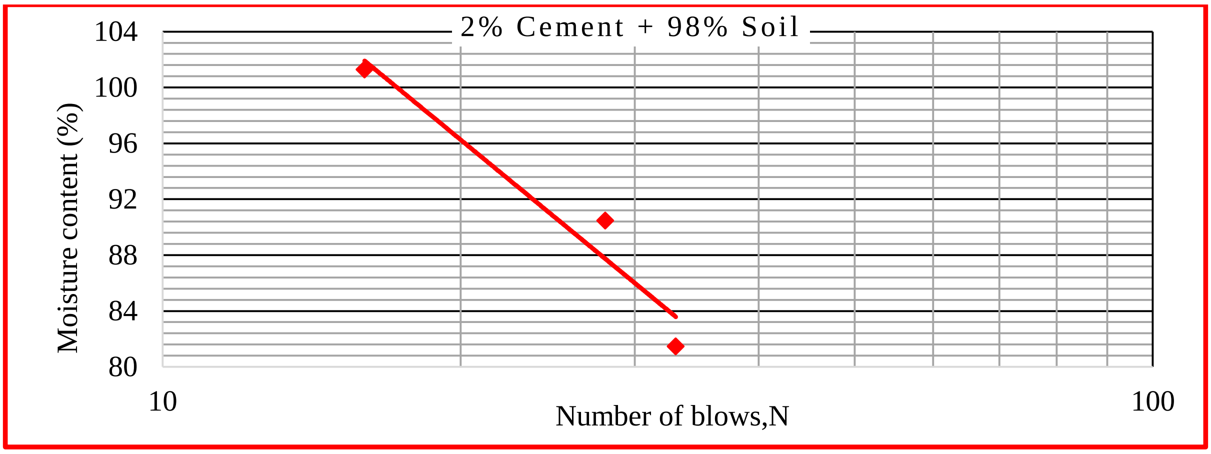


LL=93.6% PL=37.1% PI=56.5%

Fig 2.2.1 Liquid limit for natural soil sample two

Table 2.2.2 Atterberg limits determination for 2% cement + 98% soil of sample two

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	R1	F-8	TR 2-2	F1	M2	A 0+11
mass of can, W1 (g)	17.153	17.079	18.256	6.109	6.446	6.025
mass of can + moist soil, W2 (g)	36.938	37.702	39.987	12.972	11.267	11.863
Mass of can + dry soil, W3 (g)	28.056	27.906	29.052	10.831	9.887	10.188
Moisture content, W (%)	81.46382	90.477	101.287	45.340	40.104	40.235
Number of blows, N	33	28	16			

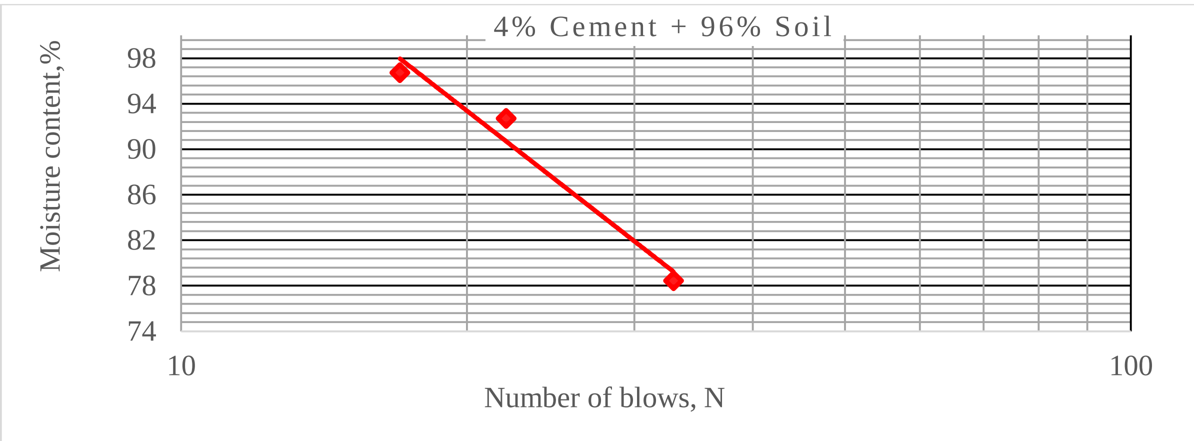


LL=91.9% PL=41.89% PI=50.01%

Fig 2.2.2 Liquid limit for 2% cement + 98% soil of sample Two

Table 2.2.3 Atterberg limits determination for 4% cement + 96% soil of sample two

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	MT-G	HO	B	Z	Z6	R5
mass of can, W1 (g)	17.597	17.696	17.697	6.021	6.203	5.912
mass of can + moist soil, W2 (g)	42.997	42.52	42.856	11.235	11.05	11.466
Mass of can + dry soil, W3 (g)	31.831	30.578	30.486	9.631	9.408	9.699
Moisture content, W (%)	78.445	92.703	96.723	44.432	51.232	46.659
Number of blows, N	33	22	17			

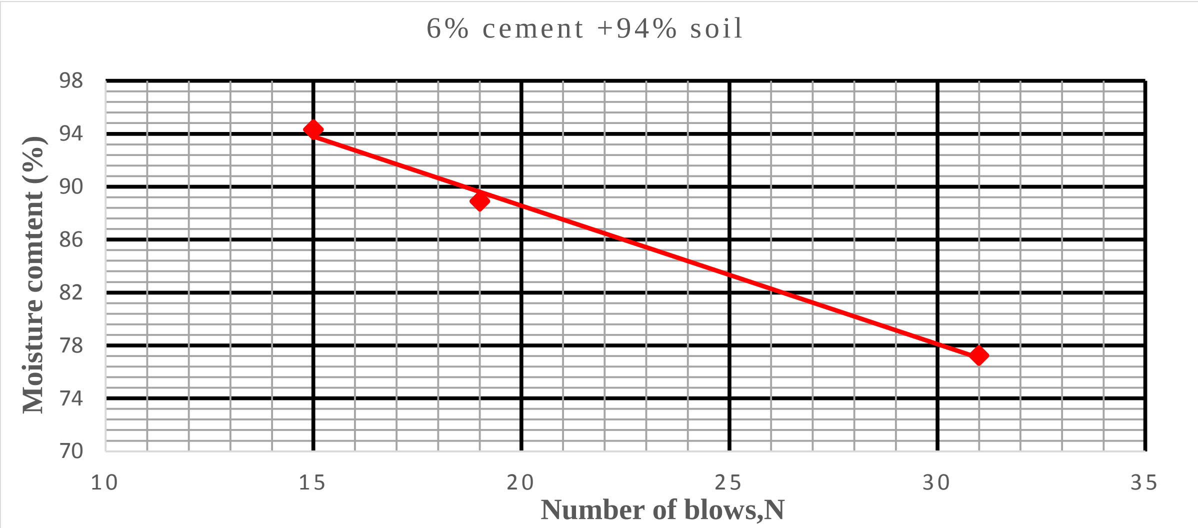


LL=88.2% PL=47.4% PI=40.8%

Fig 2.2.3 Liquid limit for 4% cement + 96% soil of sample Two

Table 2.2.4 Atterberg limits determination for 6% cement + 94% soil of sample two

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	J	L	L1	R	L5	S0
mass of can, W1 (g)	17.519	17.612	16.963	6.243	6.097	6.375
mass of can + moist soil, W2 (g)	45.633	42.943	47.977	14.425	13.237	12.788
Mass of can + dry soil, W3 (g)	33.382	31.022	32.923	11.936	10.648	10.397
Moisture content, W (%)	77.230	88.896	94.323	43.720	56.888	59.448
Number of blows, N	31	19	15			

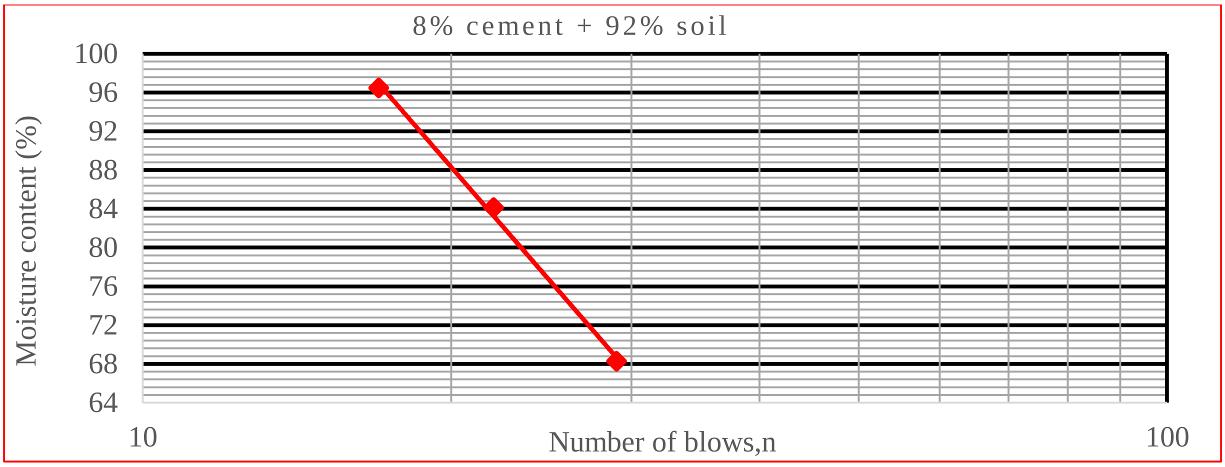


LL=83.5% PL=53.5% PI=30%

Fig 2.2.4 Liquid limit for 6% cement + 94% soil of sample Two

Table 2.2.5 Atterberg limits determination for 8% cement + 92% soil of sample two

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	T12	F-M2	EW	C1	C2	C3
mass of can, W1 (g)	16.874	16.147	16.987	6.081	5.978	6.106
mass of can + moist soil, W2 (g)	43.335	39.648	37.878	13.348	14.726	13.678
Mass of can + dry soil, W3 (g)	32.599	28.911	27.62	10.808	11.402	10.898
Moisture content, W (%)	68.273	84.119	96.473	53.733	61.283	58.013
Number of blows, N	29	22	17			

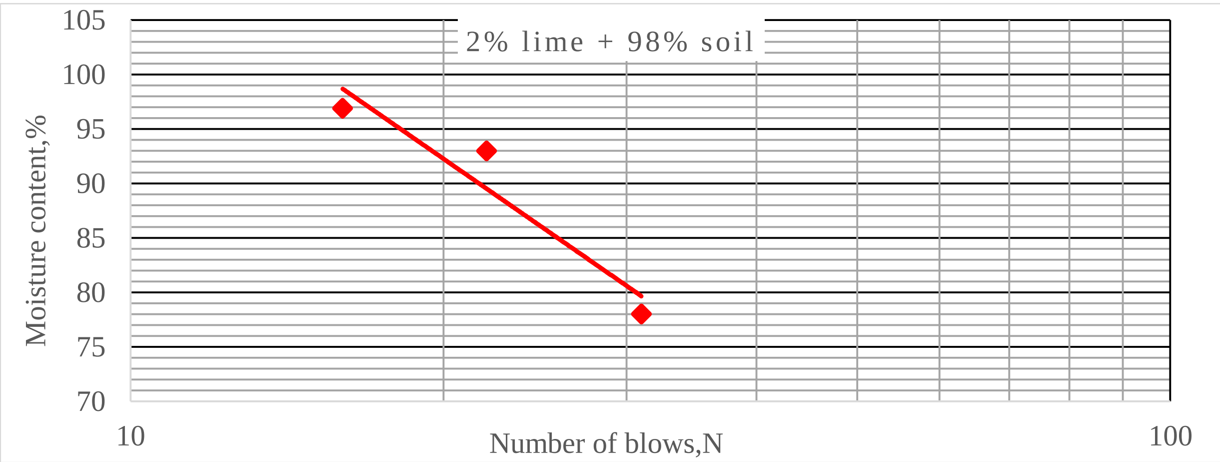


LL=66.5% PL=62.7% PI=3.8%

Fig 2.2.5 Liquid limit for 8% cement + 92% soil of sample Two

Table 2.2.6 Atterberg limits determination for 2% Lime + 98% soil of sample two

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	C11	D8	N-M22	H	PI	D5
mass of can, W1 (g)	17.471	17.389	17.461	6.102	7.888	5.99
mass of can + moist soil, W2 (g)	42.615	43.584	44.387	13.758	12.678	12.879
Mass of can + dry soil, W3 (g)	31.596	30.963	31.136	10.945	10.791	10.331
Moisture content, W (%)	78.010	92.979	96.899	58.083	65.001	58.696
Number of blows, N	31	22	16			

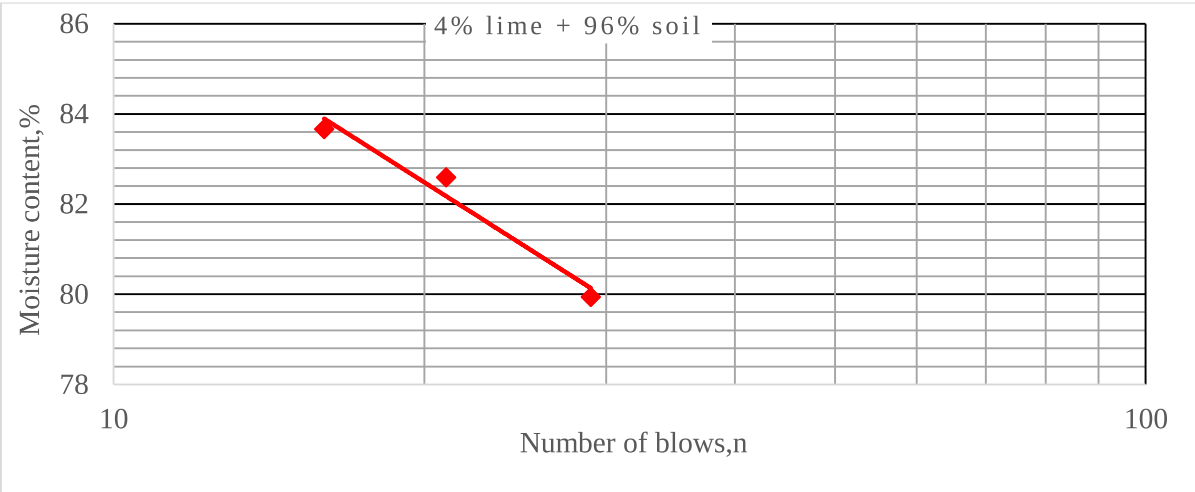


LL=86.7% PL=60.6% PI=26.1%

Fig 2.2.6 Liquid limit for 2% cement + 98% soil of sample Two

Table 2.2.7 Atterberg limits determination for 4% Lime + 96% soil of sample two

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	T3D2	F3	G-3-2	R3	D2	DB
mass of can, W1 (g)	19.266	17.28	24.4727	6.44	5.628	6.407
mass of can + moist soil, W2 (g)	43.526	41.101	53.057	13.805	11.844	12.703
Mass of can + dry soil, W3 (g)	32.748	30.326	40.036	11.05	9.489	10.32
Moisture content, W (%)	79.943	82.592	83.664	59.761	60.994	60.899
Number of blows, N	29	21	16			

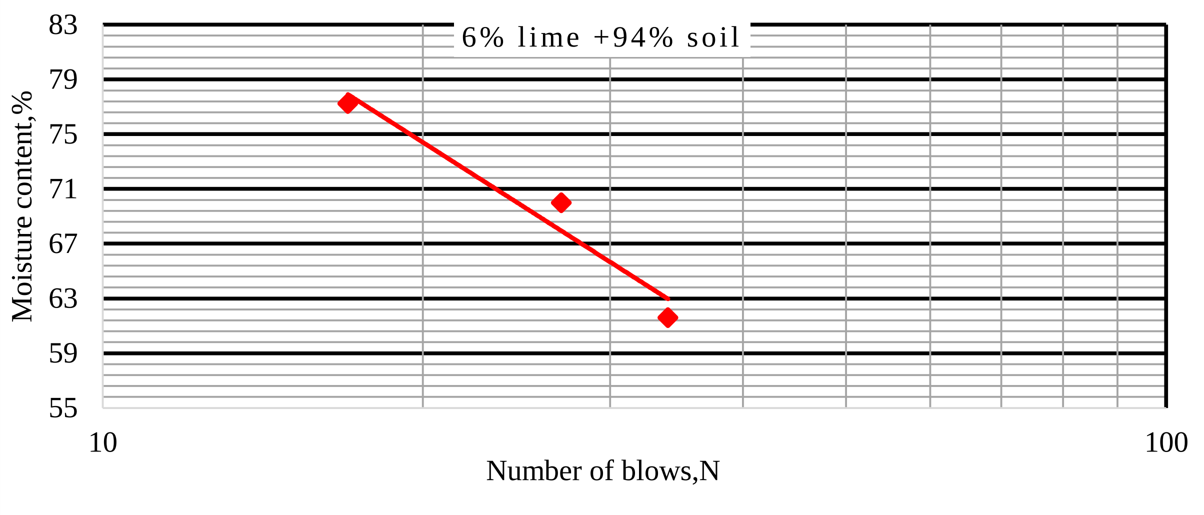


LL=77% PL=66.1% PI=15.4%

Fig 2.2.7 Liquid limit for 4% cement + 96% soil of sample Two

Table 2.2.8 Atterberg limits determination for 6% Lime + 94% soil of sample two

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	CH-1	GE-1K	15	5MB	J	S-TAR
mass of can, W1 (g)	16.852	17.841	17.342	6.01	5.89	6.207
mass of can + moist soil, W2 (g)	48.897	43.587	47.188	10.999	12.758	11.56
Mass of can + dry soil, W3 (g)	36.682	32.987	34.182	9.25	10.163	9.284
Moisture content, W (%)	61.598	69.985	77.232	53.981	60.730	73.968
Number of blows, N	34	27	17			



LL=69.8 PL=62.9 PI=6.9

Fig 2.2.8 Liquid limit for 6% cement + 94% soil of sample Two

Table 2.2.9 Atterberg limits determination for 8% Lime + 92% soil of sample two

description	liquid limit			plastic limit		
test No	1	2	3	1	2	3
Can cod	W2	RT-K4	STG-9	STA-01	LV-3	ND-8HF
Mass of can, W1 (g)	16.856	16.508	16.555	5.981	5.72	6.52
Mass of can + moist soil, W2 (g)	44.799	42.68	46.98	12.88	13.585	12.65
Mass of can + dry soil, W3 (g)	34.17	31.887	34.109	10.298	10.679	10.134
Moisture content, W (%)	61.389	70.180	73.322	59.810	58.600	69.618
Number of blows, N	31	21	18			

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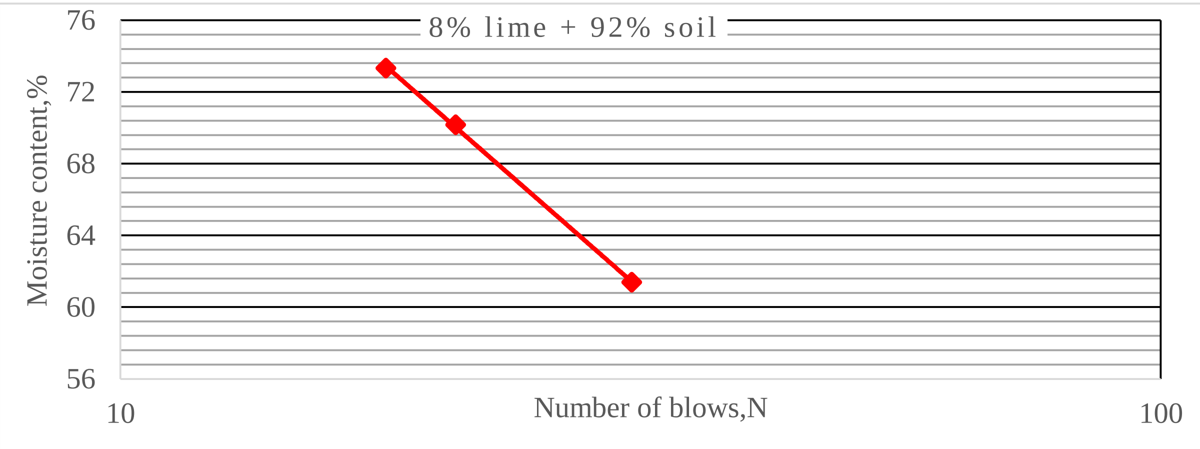


LL=66.5% PL=62.7% PI=3.8%

Fig 2.2.9 liquid limit for 8% cement + 95% soil of sample Two

Appendix 3:- Compaction Test Results for Different Lime and Cement Addition to the Expansive Soil

3.1 Sample One

Table 3.1.1 Natural soil sample one compaction test

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.99	4.39	G	17.24	60.21	52.788	29.728	1.143
			17	16.999	87.692	68.012		
2	2.99	4.475	G12-1	17.935	82.671	65.931	31.451	1.197
			H2-3	17.756	94.871	77.991		
3	2.99	4.595	E	18.796	94.396	73.878	37.784	1.233
			T41	17.691	95.967	74.282		
4	2.99	4.535	DS3	17.678	90.795	65.2	46.495	1.117
			G73	17.668	101.025	77.58		
5	2.99	4.505	SS	17.029	105.477	76.688	49.007	1.077
			Nc71	17.781	106.897	77.287		

Mold volume=944cc

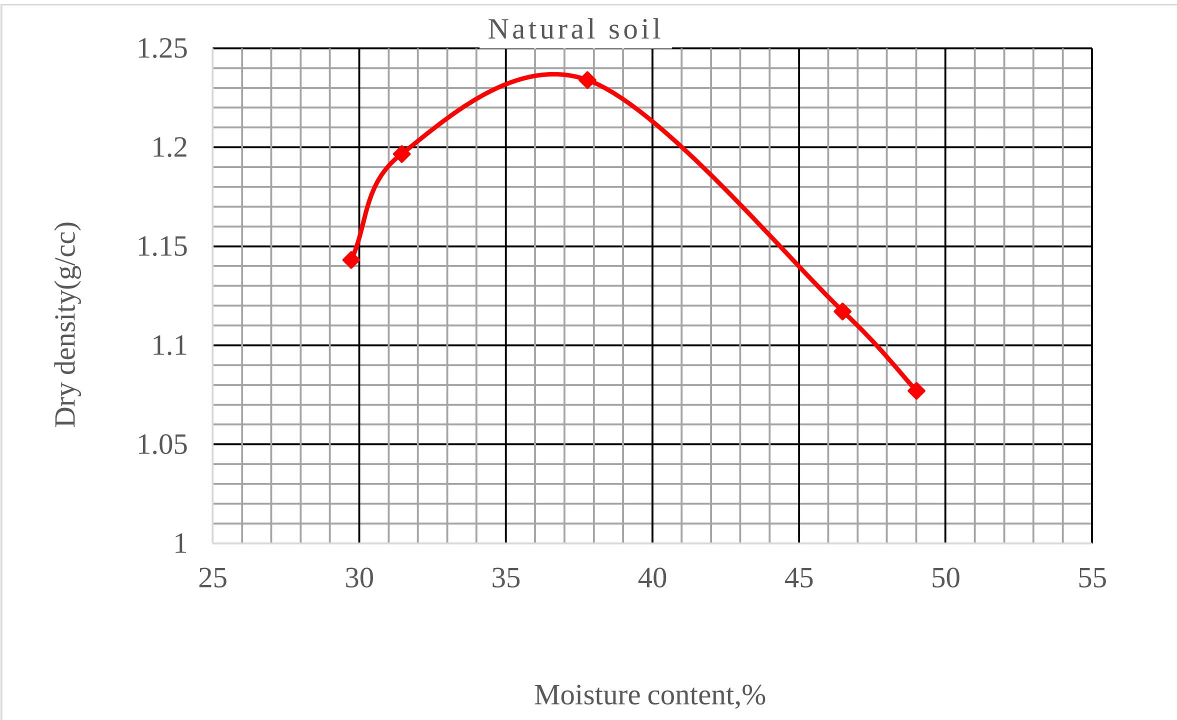


Fig 3.1.1 Dry density moisture content curve for soil sample one

Table 3.1.2 Compaction test for 2% cement + 98% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M/can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.999	4.4	1A	17.618	92.699	74.91	31.208	1.138
			T3	17.875	101.032	81.176		
2	2.999	4.53	17	16.962	95.402	74.666	35.244	1.206
			TSC1	17.39	90.896	72.02		
3	2.999	4.6	63	17.343	94.17	72.56	39.612	1.223
			T1	17.523	98.891	75.607		
4	2.999	4.58	T3C1	17.578	99.144	73.21	45.483	1.16
			Nc2S	17.317	94.515	71.02		
5	2.999	4.54	D1	18.221	76.439	57.62	53.077	1.072
			LC22	17.51	79.9	56.9		

Mold volume=944cc

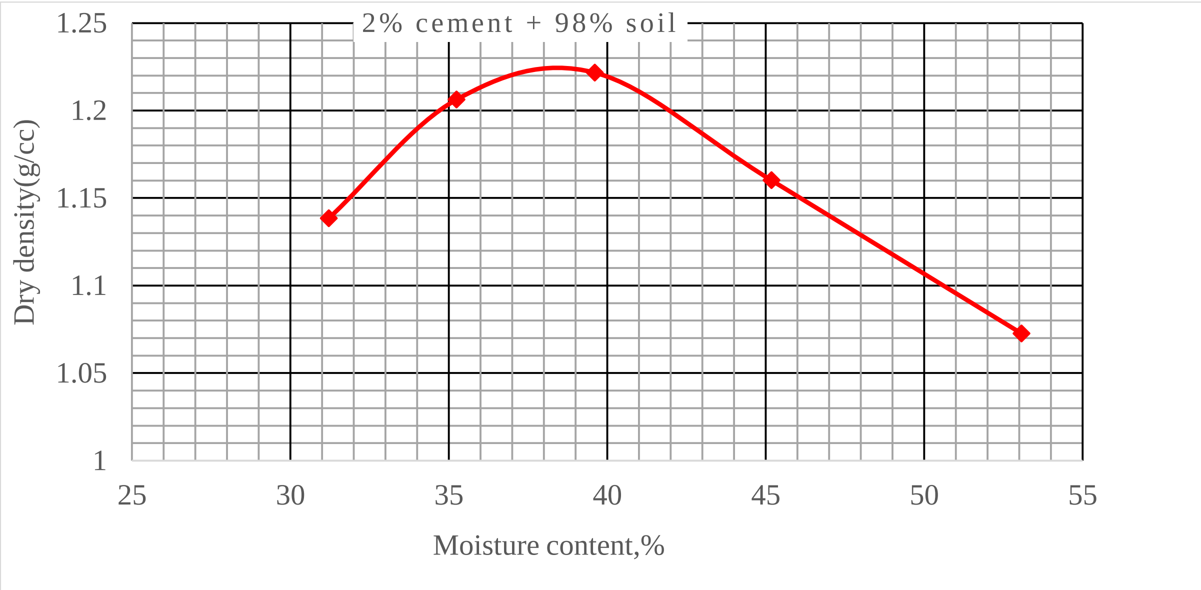


Fig 3.1.2 Dry density moisture content curve for 2% cement + 98% soil

Table 3.1.3 Compaction test for 4% cement + 96% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.99	4.4	82	18.893	77.567	62.063	34.371	1.1115
			5B	17.954	80.458	65.01		
2	2.99	4.56	C	18.439	74.401	58.816	38.991	1.1965
			E	17.076	75.007	58.638		
3	2.99	4.63	STA-2-5	17.087	93.337	70.502	43.176	1.2134
			HC4	17.623	88.659	67.09		
4	2.99	4.61	HC5	18.05	92.864	68.664	47.852	1.1606
			L3	17.76	86.752	64.41		
5	2.99	4.59	7	17.576	85.871	62.46	52.752	1.1095
			A-3	18.006	107.766	76.54		

Mold volume=944cc

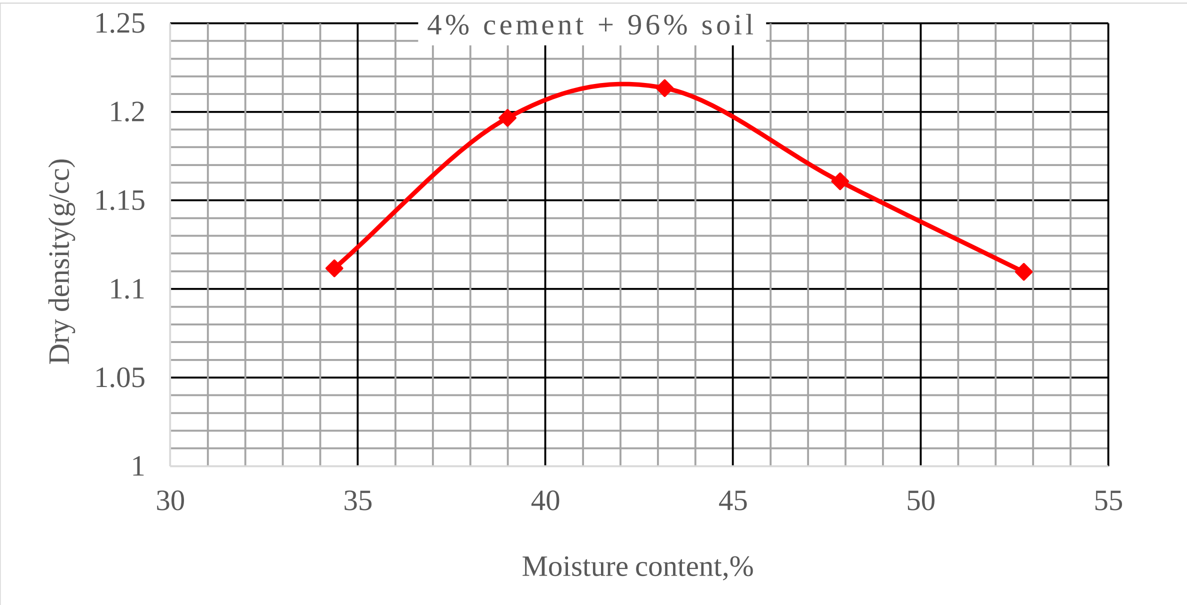


Fig 3.1.3 Dry density moisture content curve for 4% cement + 96% soil

Table 3.1.4 Compaction test for 6% cement + 94% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.999	4.4	LC12	18.164	87.184	68.871	36.022	1.098
			T14	17.693	76.68	61.088		
2	2.999	4.51	Nc51	16.985	66.23	51.857	39.113	1.1574
			13	18.375	84.848	66.892		
3	2.999	4.6	A	18.572	69.596	53.803	42.854	1.1938
			G1	18.186	90.758	69.699		
4	2.999	4.65	G73	17.681	83.726	62.793	46.707	1.19863
			452.2	17.383	92	68.139		
5	2.999	4.57	G2	17.956	69.866	51.074	58.157	1.0582
			HCN	17.892	92.274	64.5		

Mold volume=944cc

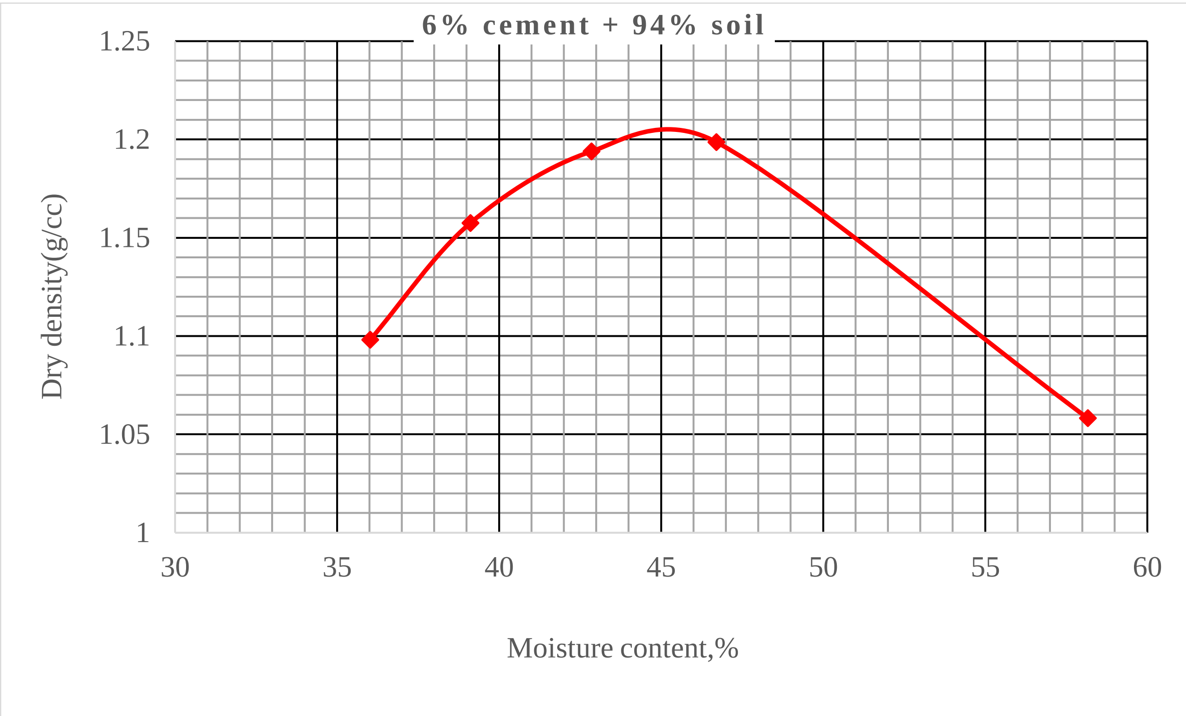


Fig 3.1.4 Dry density moisture content curve for 6% cement + 94% soil

Table 3.1.5 Compaction test for 8% cement + 92% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.99	4.4	HS3	17.395	82.449	64.233	37.078	1.0896
			GS3	17.423	81.11	64.506		
2	2.99	4.488	LC42	17.67	71.584	56.059	39.646	1.1363
			HC51	17.58	84.805	65.995		
3	2.99	4.65	D32	17.543	102.12	74.712	47.024	1.196
			ATR	17.618	92.616	68.949		
4	2.99	4.599	E	18.823	77.318	53.493	56.301	1.0904
			H2	17.682	103.857	77.574		

Mold volume=944cc

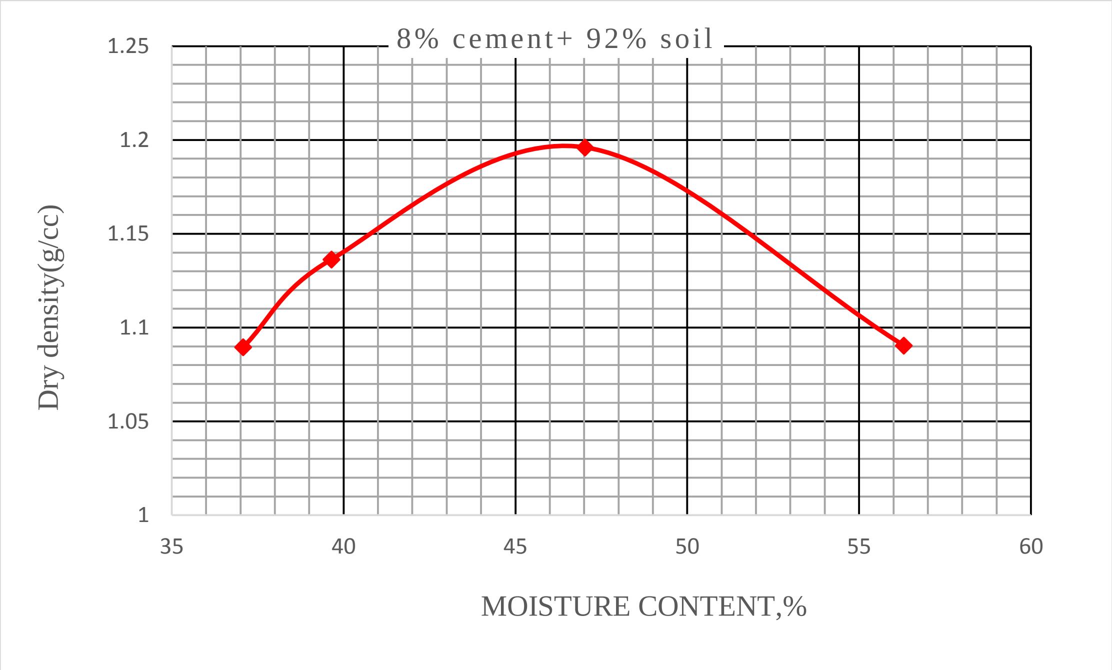


Fig 3.1.5 Dry density moisture content curve for 8% cement + 92% soil sample one

Table 3.1.6 Compaction test for 2% Lime + 98% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.99	4.436	D1	18.207	88.388	71.326	32.795	1.1534
			G71	17.933	92.391	73.719		
2	2.99	4.535	LC22	17.414	97.699	75.972	36.251	1.2011
			DL1-1	17.833	84.642	67.175		
3	2.99	4.57	T1C2	18.222	106.13	80.76	39.810	1.1971
			G74	17.766	97.289	74.955		
4	2.99	4.55	HC42	17.559	79.802	62.058	42.619	1.1587
			D	17.317	76.672	58.149		
5	2.99	4.52	LB1	17.501	94.79	72.589	43.962	1.1258
			LC33	25.923	90.435	69.623		

Mold volume=944cc

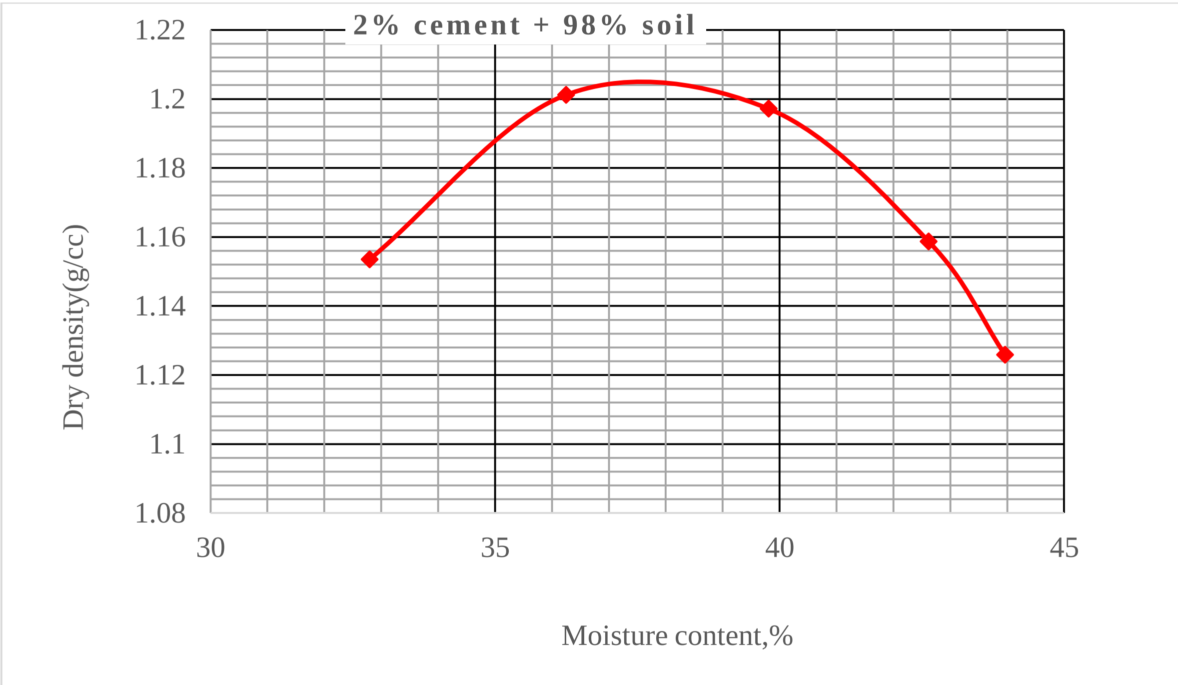


Fig 3.1.6 Dry density moisture content curve for 2% Lime + 98% soil sample two

Table 3.1.7 Compaction test for 4% Lime + 96% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.99	4.4	HC31	18.04	98.067	77.579	34.249	1.112585
			LC-11	17.512	88.269	70.281		
2	2.99	4.52	DS2	17.862	101.369	78.222	38.759	1.168035
			LC52	17.815	102.587	78.727		
3	2.99	4.576	GS-3	17.703	75.685	58.322	41.991	1.183237
			LC31	17.525	78.451	60.663		
4	2.99	4.545	BA	17.663	94.348	69.959	46.854	1.121687
			LC13	17.605	93.378	69.126		
5	2.99	4.543	1B	17.214	85.091	63.577	47.029	1.118909
			G4-2	17.449	76.69	57.57		

Mold volume=944cc

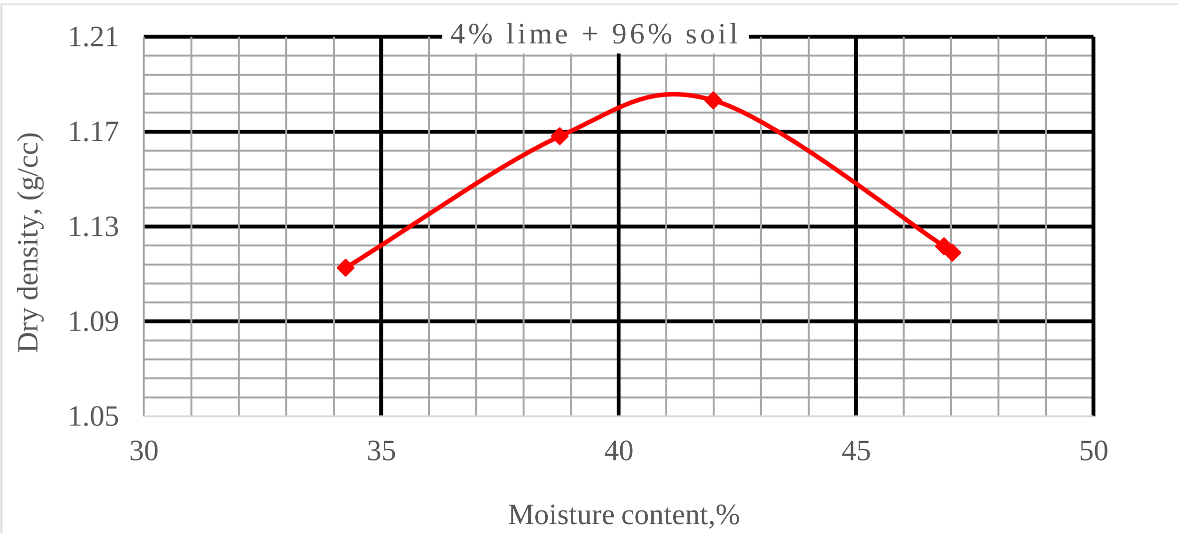


Fig 3.1.7 Dry density moisture content curve for 4% Lime + 96% soil sample two

Table 3.1.8 Compaction test for 6% Lime + 94% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	3	4.4	HC13	18.169	55.202	44.96	37.27436	1.080355
			LC12	18.154	60.518	49.231		
2	3	4.55	A-3	17.988	97.946	74.294	42.13628	1.155194
			TSC2	17.964	96.919	73.462		
3	3	4.62	LA	17.613	85.273	64.087	45.93124	1.175966
			T1C2	17.584	82.684	62.089		
4	3	4.63	17	16.958	78.327	58.087	49.85651	1.152232
			T2C2	18.909	78.571	58.968		
5	3	4.58	T5C1	17.37	85.324	62.439	54.3542	1.084343
			T3	17.865	75.426	54.312		

Mold volume=944cc

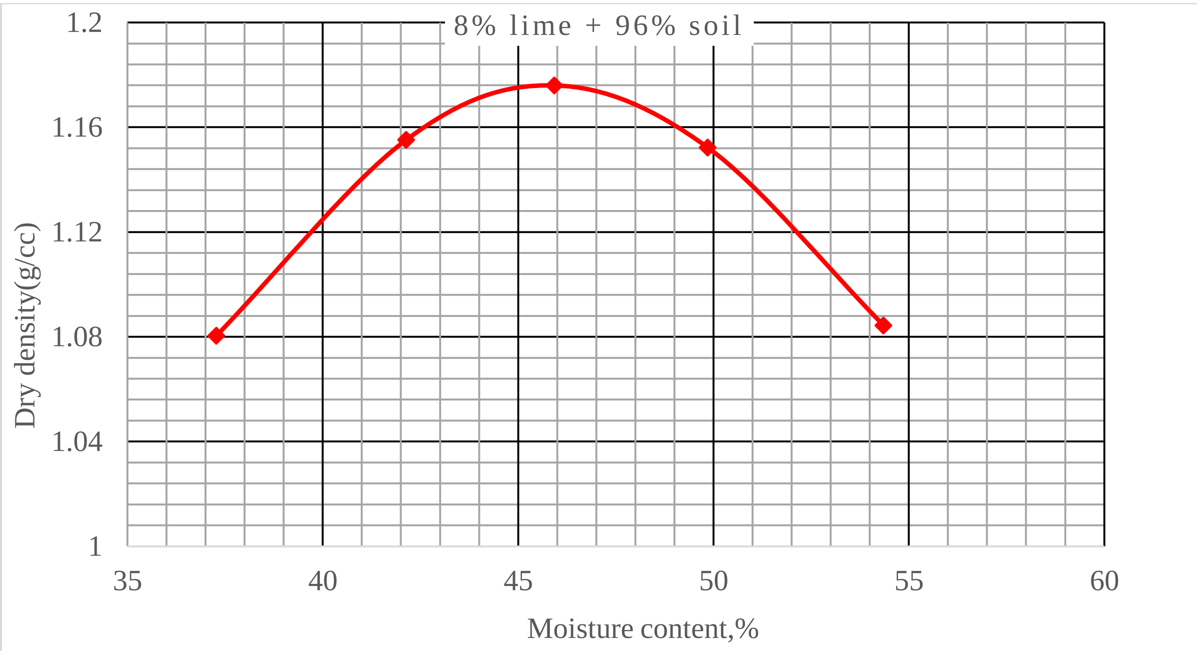


Fig 3.1.8 Dry density moisture content curve for 6% Lime + 94% soil sample two

Table 3.1.9 Compaction test for 8% Lime + 92% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.an & dry soil (g)	w%	ρ_d(Kpa)
1	2.99	4.5	T	18.315	69.23	53.576	38.78735	1.087995
			J	6.654	75.58	58.408		
2	2.99	4.57	LL3	28.277	86.569	69.471	41.07267	1.11999
			MPL	6.17	58.44	43.336		
3	2.99	4.668	S12	27.826	84.269	66.065	44.9962	1.157272
			427	18.11	98.809	74.786		
4	2.99	4.745	D3	18.149	86.35	64.07	50.91174	1.162931
			D6	27.79	80.78	62.355		
5	2.99	4.68	PS7	27.704	79.977	60.659	55.35293	1.087845
			H	17.97	80.4	59.019		

Mold volume=944cc

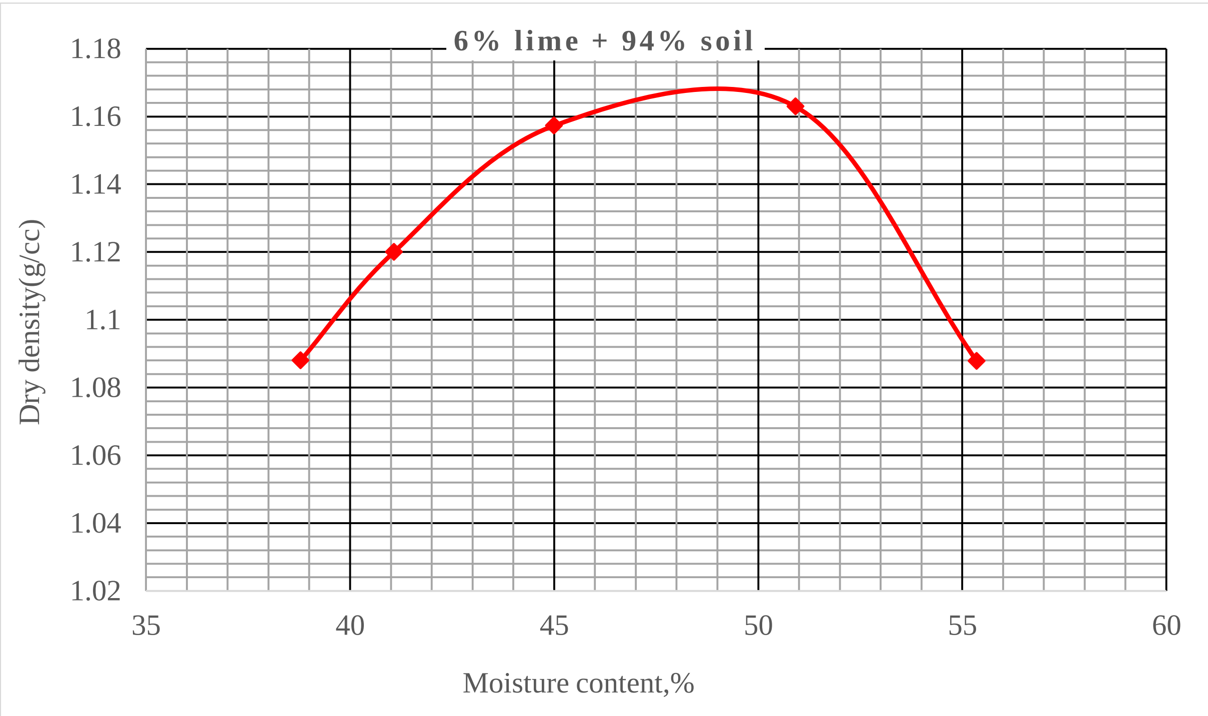


Fig 3.1.9 Dry density moisture content curve for 8% Lime + 92% soil sample two

3.2 Sample Two

Table 3.2.1 Natural soil sample two compaction test

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.99	4.342	G	17.24	62.618	54.899	29.2936	1.1077
			17	16.999	91.199	70.732		
2	2.99	4.386	G12	17.935	85.978	68.568	31.046	1.12846
			H23	17.756	98.665	81.111		
3	2.99	4.503	E	18.796	98.172	76.833	37.316	1.1672
			T41	17.691	99.806	77.253		
4	2.99	4.47	DS3	17.678	94.427	67.808	45.897	1.07459
			G73	17.668	105.066	80.683		
5	2.99	4.427	SS	17.029	109.696	79.456	51.279	1.0062
			Nc7	17.781	111.172	78.378		

Mold volume=944cc

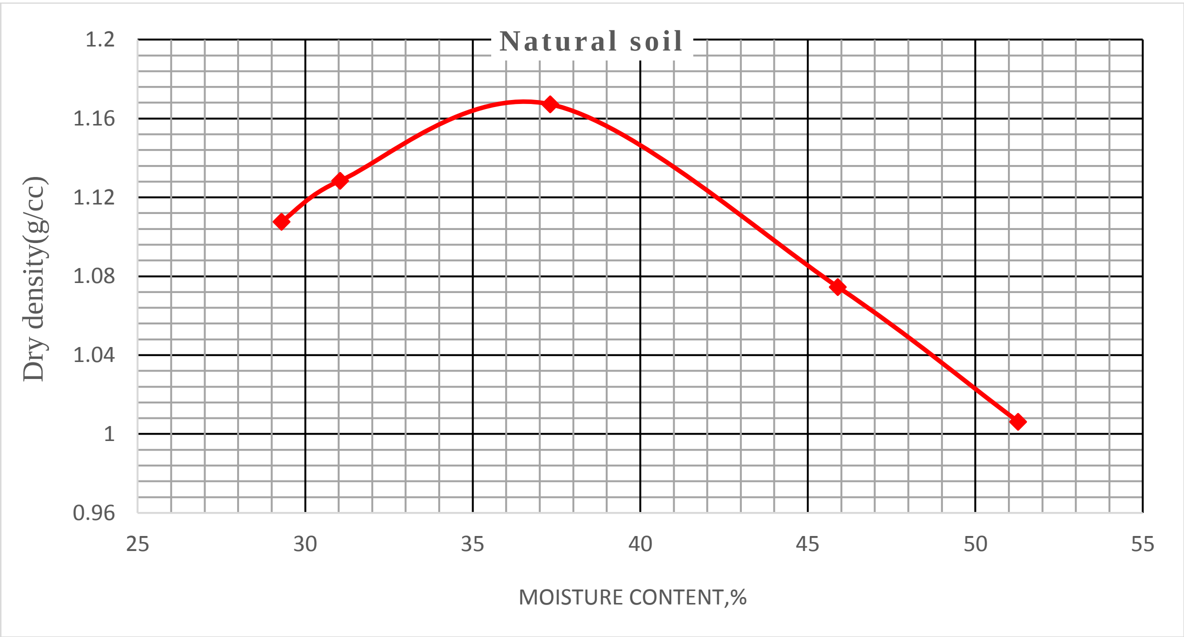


Fig 3.2.1 dry density moisture content curve for soil sample two

Table 3.2.2 Compaction test for 2% cement + 98% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.999	4.352	1A	17.618	96.407	77.906	30.858	1.10255
			T3	17.875	105.073	84.423		
2	2.999	4.439	17	16.962	99.218	77.799	34.671	1.139775
			TSC1	17.39	94.532	74.9		
3	2.999	4.508	63	17.343	97.937	75.462	39.149	1.155629
			T1	17.523	102.847	78.631		
4	2.999	4.488	T3C1	17.578	103.11	76.138	44.636	1.09714
			Nc2S	17.317	98.296	73.861		
5	2.999	4.449	D1	18.221	79.497	59.925	52.169	1.015675
			LC22	17.51	83.096	59.176		

Mold volume=944cc

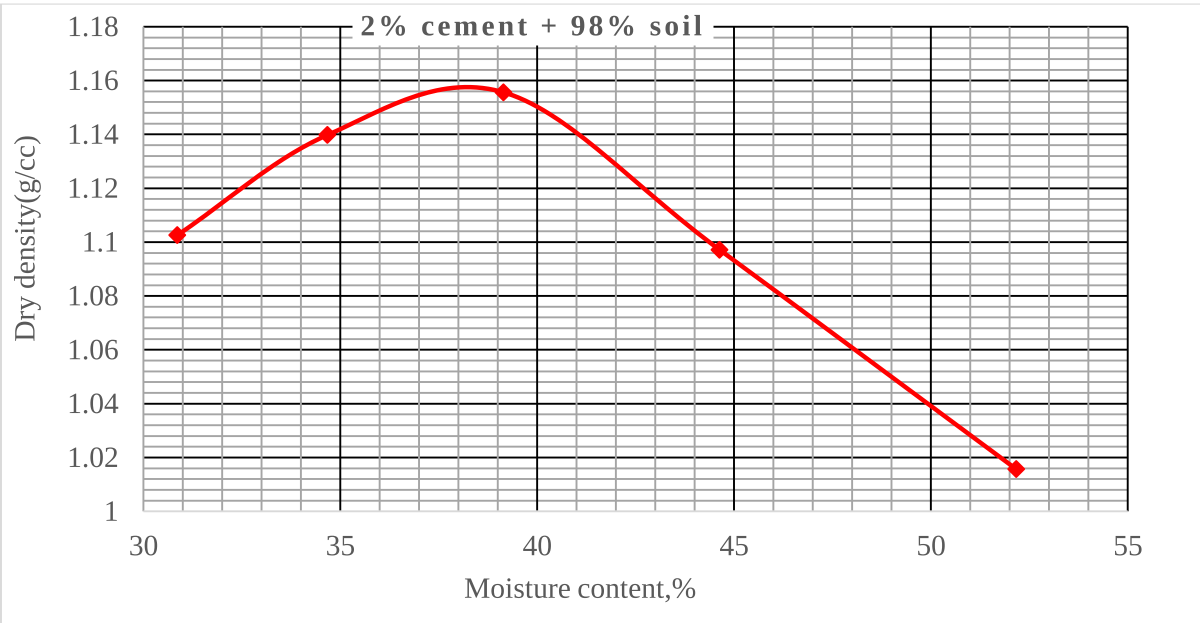


Fig 3.2.2 Dry density moisture content curve for 2% cement + 98% soil sample two

Table 3.2.3 Compaction test for 4% cement + 96% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.99	4.372	82	18.893	80.669	64.545	33.8369	1.093855
			5B	17.954	83.676	67.61		
2	2.99	4.469	C	18.439	77.377	61.168	38.3536	1.132415
			E	17.076	78.007	60.983		
3	2.99	4.547	STA	17.087	97.07	73.322	44.0187	1.14524
			HC4	17.623	92.205	68.774		
4	2.99	4.538	HC5	18.05	96.578	71.411	47.1831	1.114143
			L3	17.76	90.222	66.986		
5	2.99	4.498	7	17.576	89.306	64.958	52.0545	1.050582
			A-3	18.006	112.077	79.602		

Mold volume=944cc

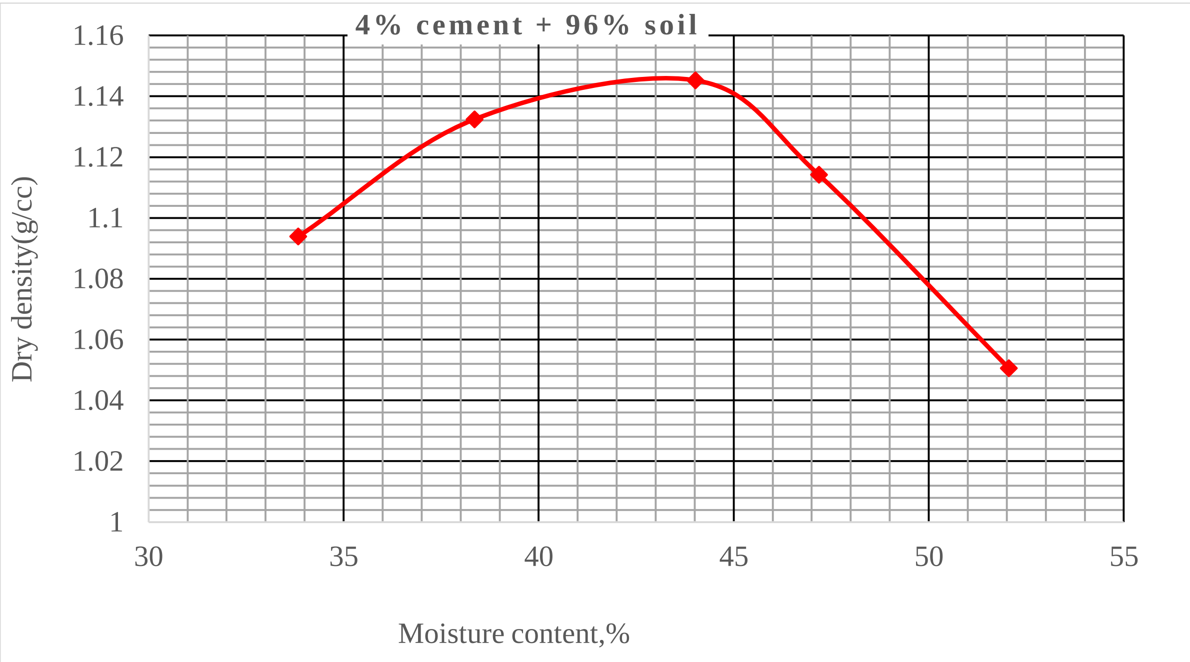


Fig 3.2.3 Dry density moisture content curve for 4% cement + 96% soil sample two

Table 3.2.4 Compaction test for 6% cement + 94% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.999	4.4	LC1	18.164	87.184	68.871	36.022	1.098
			T14	17.693	76.68	61.088		
2	2.999	4.51	Nc51	16.985	66.23	51.857	39.113	1.157
			13	18.375	84.848	66.892		
3	2.999	4.6	A	18.572	69.596	53.803	42.854	1.193
			G1	18.186	90.758	69.699		
4	2.999	4.65	G73	17.681	83.726	62.793	46.707	1.198
			452.	17.383	92	68.139		
5	2.999	4.57	G2	17.956	69.866	51.074	58.15	1.058
			HCN	17.892	92.274	64.5		

Mold volume=944cc

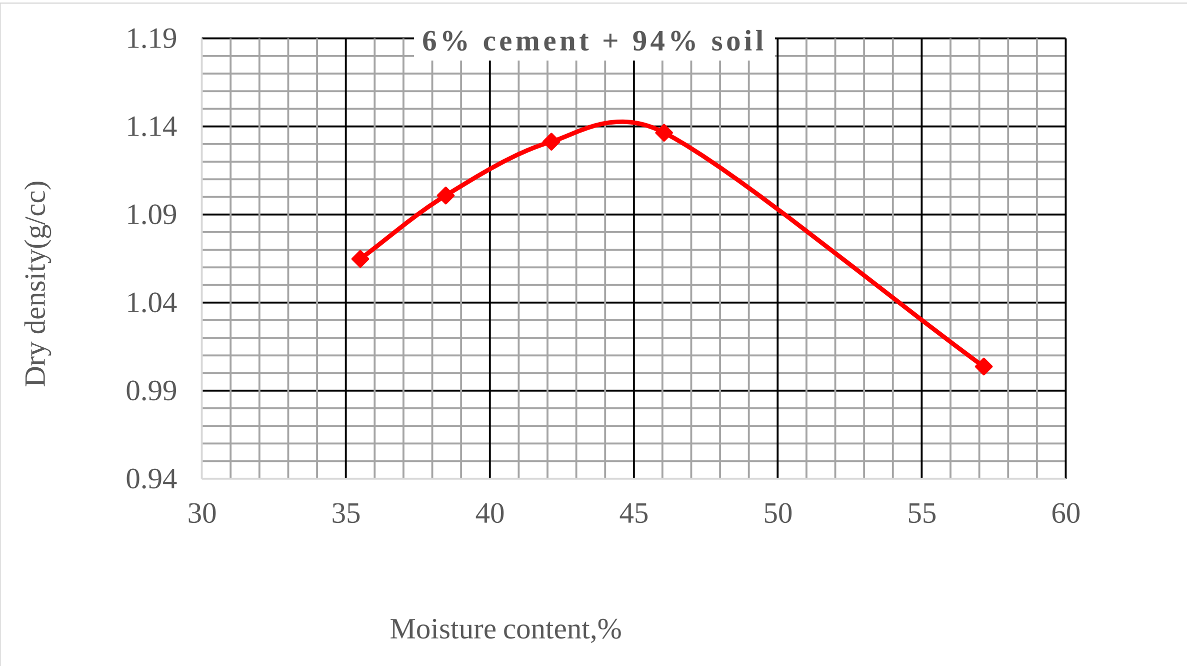


Fig 3.2.4 Dry density moisture content curve for 6% cement + 94% soil sample two

Table 3.2.5 Compaction test for 8% cement + 92% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.99	4.339	HS3	17.395	85.747	66.802	36.55756	1.04664
			GS3	17.423	84.354	67.086		
2	2.99	4.398	LC42	17.67	74.447	58.301	39.02833	1.072833
			HC51	17.58	88.197	68.635		
3	2.99	4.557	D32	17.543	106.205	77.7	46.44541	1.133499
			ATR	17.618	96.321	71.707		
4	2.99	4.507	E	18.823	80.411	55.633	55.35223	1.034418
			H2	17.682	108.0113	80.677		

Mold volume=944cc

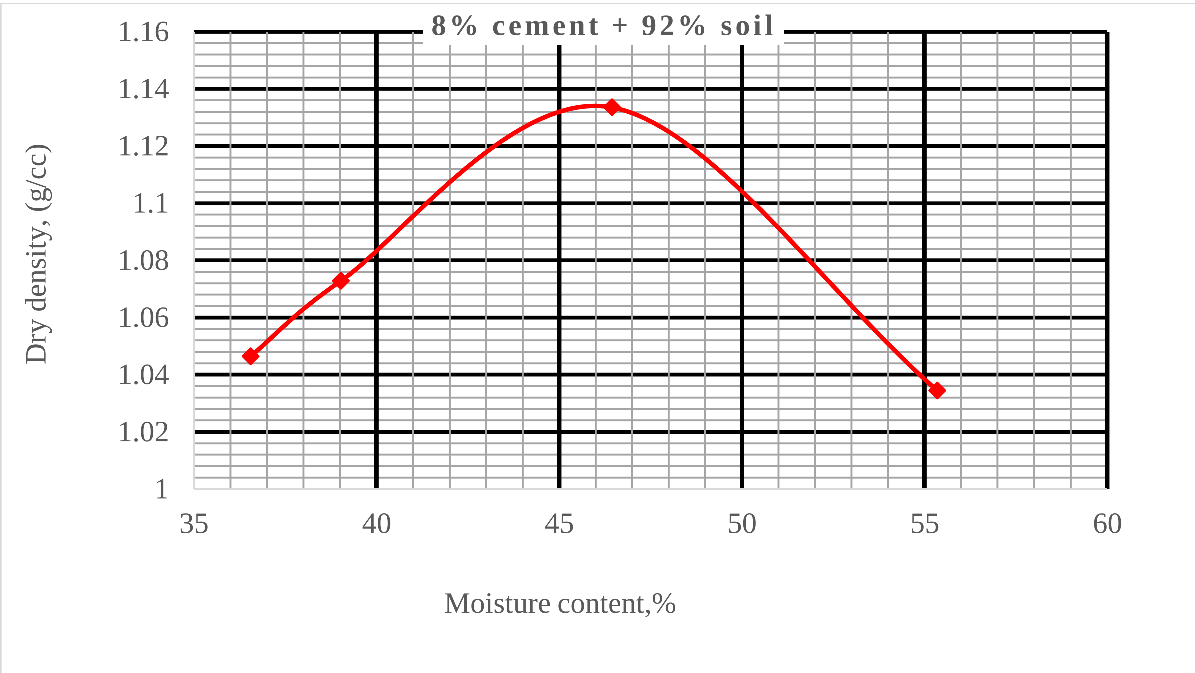


Fig 3.2.5 Dry density moisture content curve for 8% cement + 92% soil sample two

Table 3.2.6 Compaction test for 2%Lime + 98% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.99	4.342	G	17.24	62.618	54.899	29.2936	1.10771
			17	16.999	91.199	70.732		
2	2.99	4.386	G12-1	17.935	85.978	68.568	31.0460	1.12846
			H2-3	17.756	98.665	81.111		
3	2.99	4.503	E	18.796	98.172	76.833	37.3163	1.16719
			T41	17.691	99.806	77.253		
4	2.99	4.47	DS3	17.678	94.427	67.808	46.8969	1.07459
			G73	17.668	105.066	80.683		
5	2.99	4.427	SS	17.029	109.696	79.456	51.2793	1.00624
			Nc71	17.781	111.172	78.378		

Mold volume=944cc

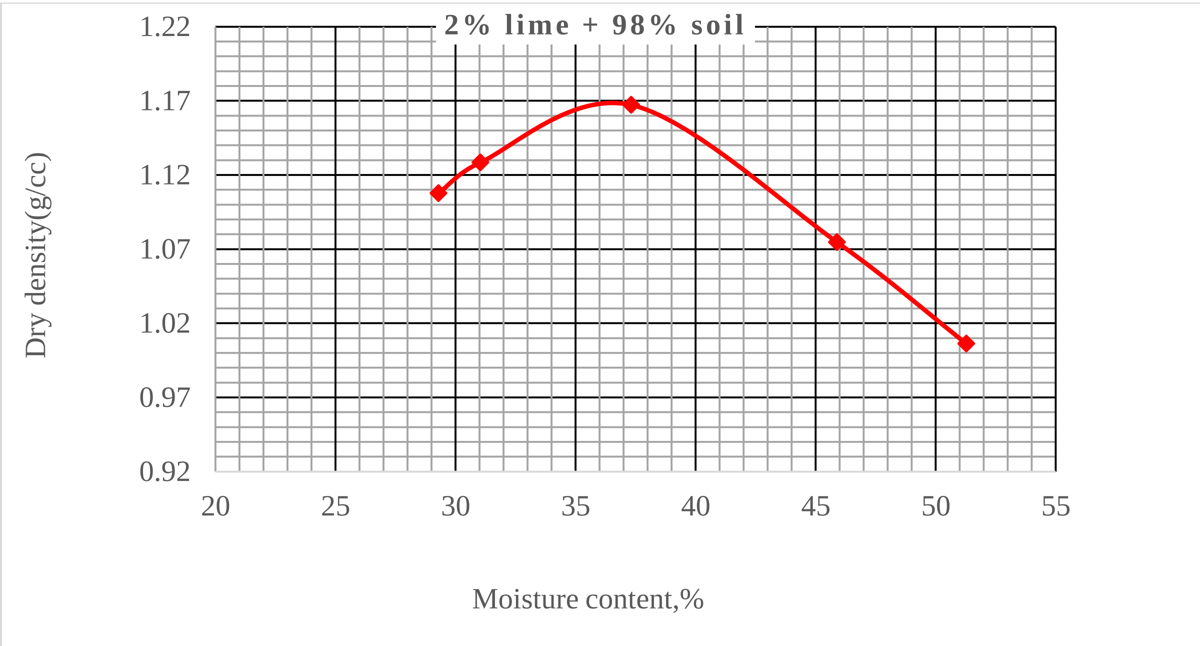


Fig 3.2.6 Dry density moisture content curve for 2% Lime + 98% soil sample two

Table 3.2.7 Compaction test for 4%Lime + 96% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.99	4.338	D1	18.207	92.5432	74.894	33.2824	1.017491
			G71	17.933	80.514	64.142		
2	2.99	4.436	LC22	17.414	70.842	54.449	40.1212	1.093182
			DL1	17.833	89.09	70.236		
3	2.99	4.489	T1C2	18.222	74.076	56.493	42.9219	1.111043
			G74	17.766	95.296	73.184		
4	2.99	4.511	HC4	17.559	87.912	65.933	45.8193	1.104949
			D	17.317	96.6	71.545		
5	2.99	4.433	LB1	17.501	73.359	55.628	52.4532	1.002669
			LC33	25.923	96.89	70.725		

Mold volume=944cc

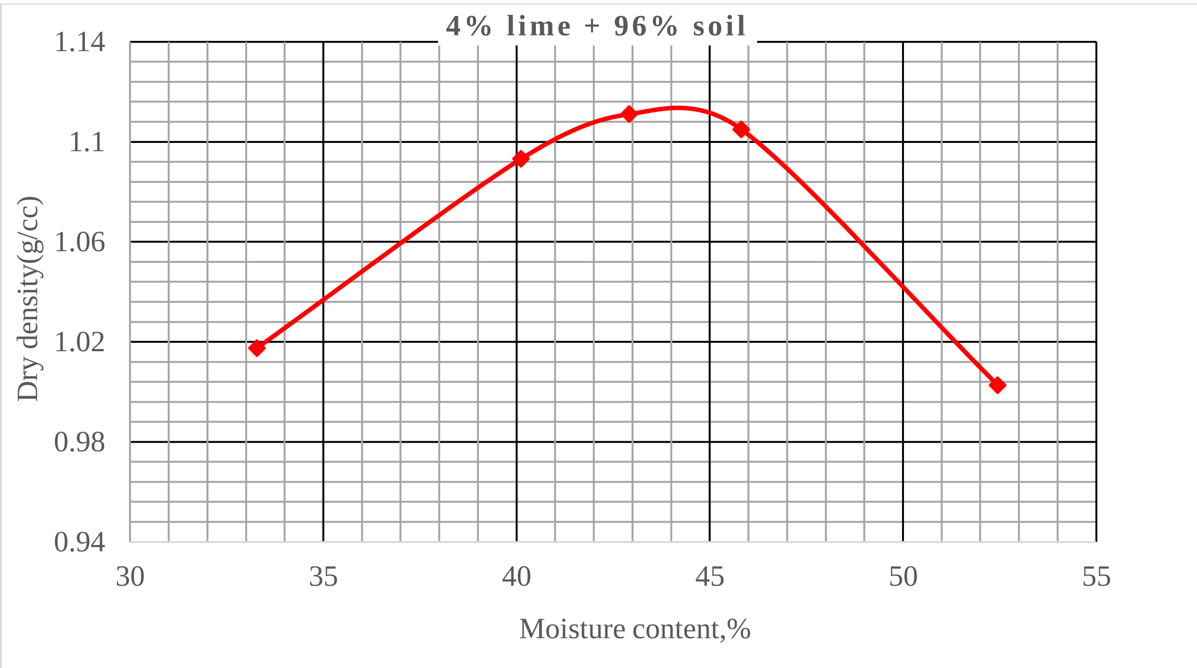


Fig 3.2.7 Dry density moisture content curve for 4% Lime + 96% soil sample two

Table 3.2.8 Compaction test for 6%Lime + 94% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	3	4.338	HC13	18.169	59.962	48.208	38.722	1.021
			LC12	18.154	64.544	51.693		
2	3	4.419	A-3	17.988	102.843	77.909	41.618	1.061
			TSC2	17.964	101.765	77.135		
3	3	4.499	LA	17.613	89.537	67.291	45.101	1.094
			T1C2	17.584	86.818	65.193		
4	3	4.542	17	16.958	83.243	60.991	49.197	1.094
			T2C2	18.909	82.499	61.916		
5	3	4.443	T5C1	17.37	89.59	63.561	56.478	0.976
			T3	17.865	79.197	57.028		

Mold volume=944cc

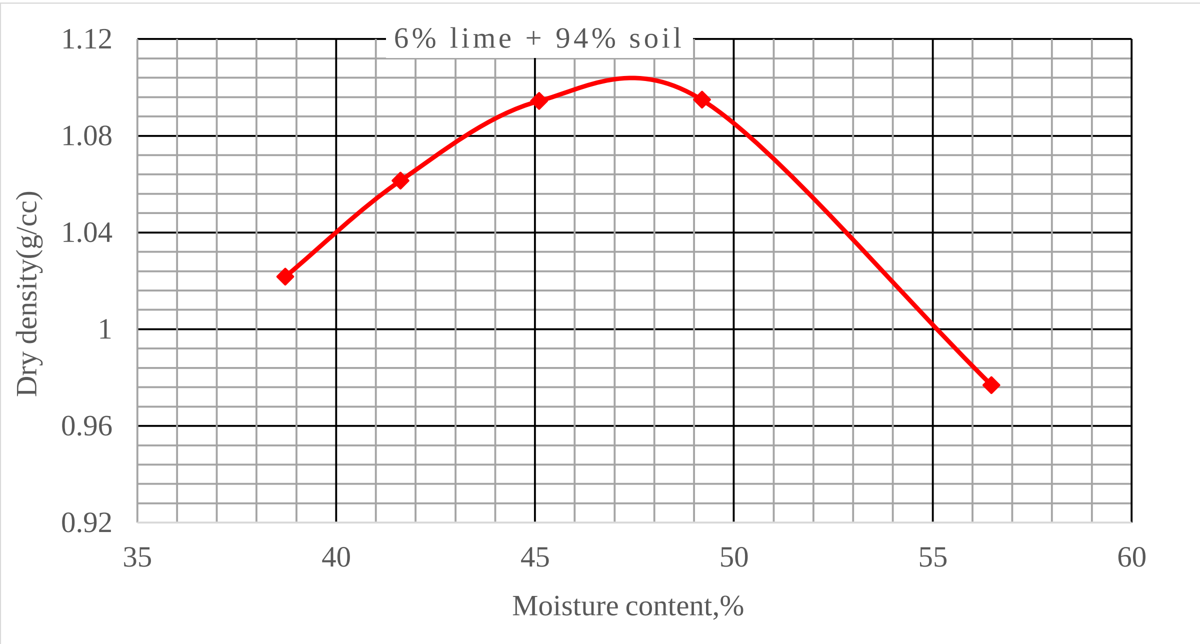


Fig 3.2.8 Dry density moisture content curve for 6% Lime + 94% soil sample two

Table 3.2.9 Compaction test for 8%Lime + 92% soil of sample one

trial	M.mold (Kg)	M.mold & compacted soil (Kg)	cane code	M.can (g)	M.can & wet soil (g)	M.can & dry soil (g)	w%	ρ_d(Kpa)
1	2.99	4.375	T	18.315	72.691	56.255	39.38884	0.993
			J	6.654	79.359	60.328		
2	2.99	4.453	LL3	28.277	90.897	72.945	42.08511	1.029
			MPL	6.17	61.362	44.503		
3	2.99	4.528	S12	27.826	89.482	69.368	45.08484	1.060
			42739	18.11	103.749	78.525		
4	2.99	4.603	D3	18.149	90.667	67.273	49.480	1.079
			D6	27.79	84.819	65.473		
5	2.99	4.54	PS7	27.704	83.976	63.692	53.692	1.008
			H	17.97	84.42	61.97		

Mold volume=944cc

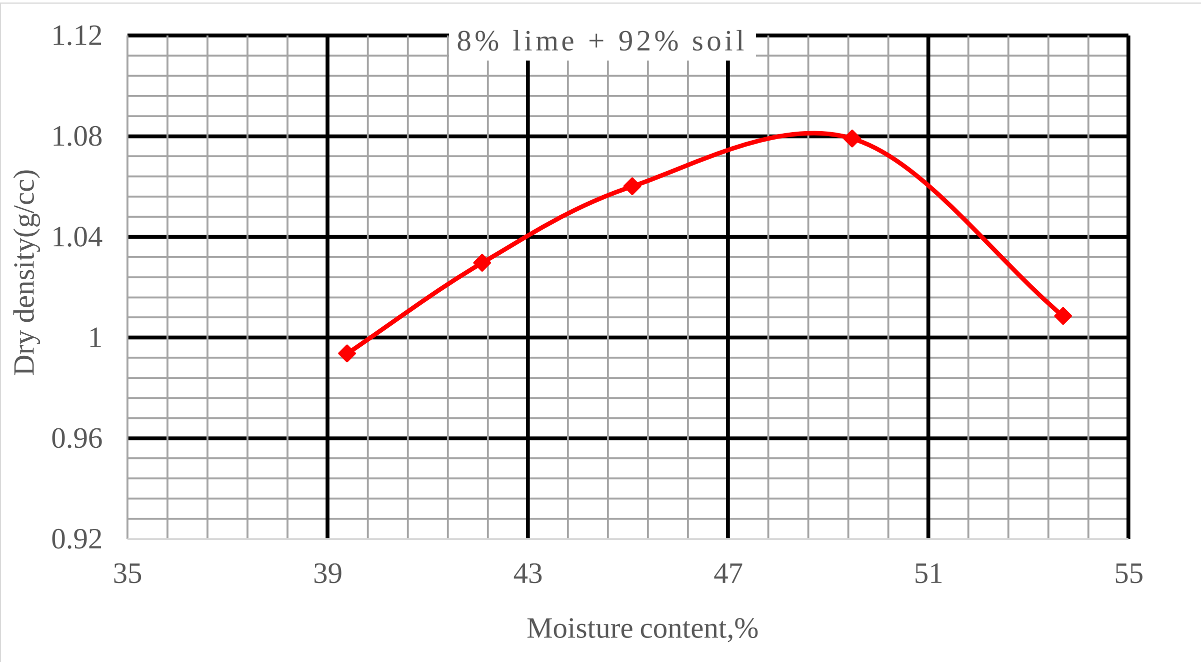


Fig 3.2.9 Dry density moisture content curve for 8% Lime + 92% soil sample two

Appendix 4:-UCS Test Results for Different Lime and Cement Addition to the Expansive Soil Sample One Immediate Curing Duration

Table 4.1.1 UCS test for natural soil sample one

Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation $(\Delta L)=Dr * 0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr * cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	1	8.4	0.00102	8.234647
40	0.4	0.005333	3	25.2	0.001023	24.63789
60	0.6	0.008	5	42	0.001026	40.95305
80	0.8	0.010667	7	58.8	0.001028	57.18015
100	1	0.013333	9	75.6	0.001031	73.31918
120	1.2	0.016	11	92.4	0.001034	89.37013
140	1.4	0.018667	13	109.2	0.001037	105.333
160	1.6	0.021333	15.5	130.2	0.00104	125.2481
180	1.8	0.024	17	142.8	0.001042	136.9946
200	2	0.026667	19	159.6	0.001045	152.6932
220	2.2	0.029333	21	176.4	0.001048	168.3038
240	2.4	0.032	22	184.8	0.001051	175.8339
260	2.6	0.034667	23	193.2	0.001054	183.32
280	2.8	0.037333	24.5	205.8	0.001057	194.7362
300	3	0.04	25.5	214.2	0.00106	202.1231
320	3.2	0.042667	26	218.4	0.001063	205.5139
340	3.4	0.045333	26.5	222.6	0.001066	208.8826
360	3.6	0.048	27	226.8	0.001069	212.2293
380	3.8	0.050667	27.5	231	0.001072	215.554
400	4	0.053333	27	226.8	0.001075	211.0403
420	4.2	0.056	26	218.4	0.001078	202.6516
440	4.4	0.058667	25.5	214.2	0.001081	198.193
460	4.6	0.061333	24.5	205.8	0.001084	189.8813
480	4.8	0.064	24	201.6	0.001087	185.4777

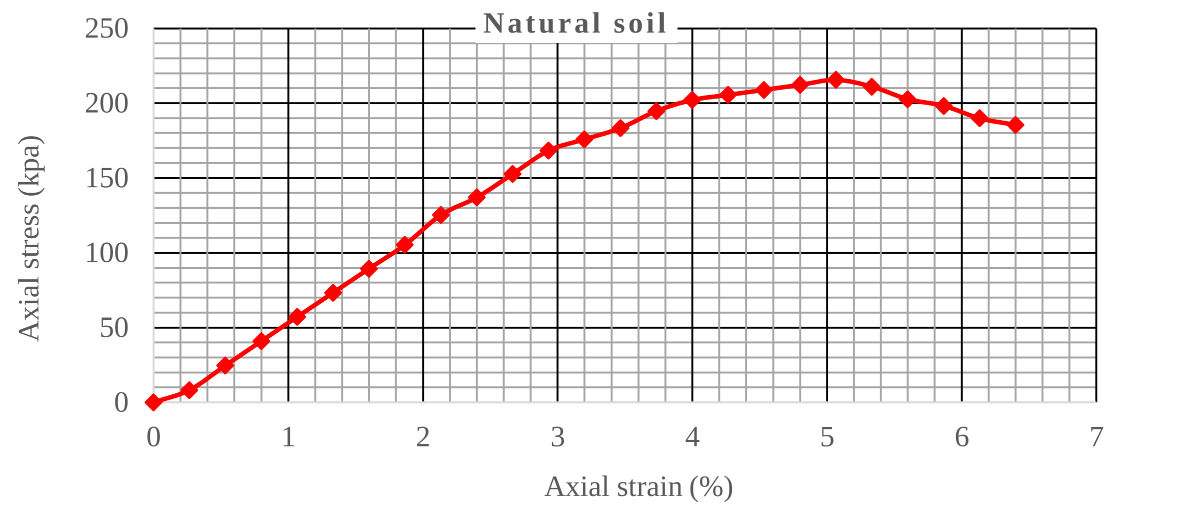


Fig 4.1.1 Stress–strain curves from UCS test for Natural soil sample one

Table 4.1.2 UCS test for 2%Lime + 92% soil of sample one

Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation (ΔL)=$Dr*$ 0.01 (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr*cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	3	25.2	0.00102	24.70394
40	0.4	0.005333	7	58.8	0.001023	57.4884
60	0.6	0.008	11	92.4	0.001026	90.09672
80	0.8	0.010667	14	117.6	0.001028	114.3603
100	1	0.013333	18	151.2	0.001031	146.6384
120	1.2	0.016	21	176.4	0.001034	170.6157
140	1.4	0.018667	24.5	205.8	0.001037	198.5122
160	1.6	0.021333	27.5	231	0.00104	222.2144
180	1.8	0.024	30.5	256.2	0.001042	245.7844
200	2	0.026667	33	277.2	0.001045	265.2041
220	2.2	0.029333	33.5	281.4	0.001048	268.4847
240	2.4	0.032	34	285.6	0.001051	271.7433
260	2.6	0.034667	33	277.2	0.001054	263.0243
280	2.8	0.037333	31.5	264.6	0.001057	250.3751
300	3	0.04	30	252	0.00106	237.7919
320	3.2	0.042667	28.5	239.4	0.001063	225.2748
340	3.4	0.045333	28	235.2	0.001066	220.7061

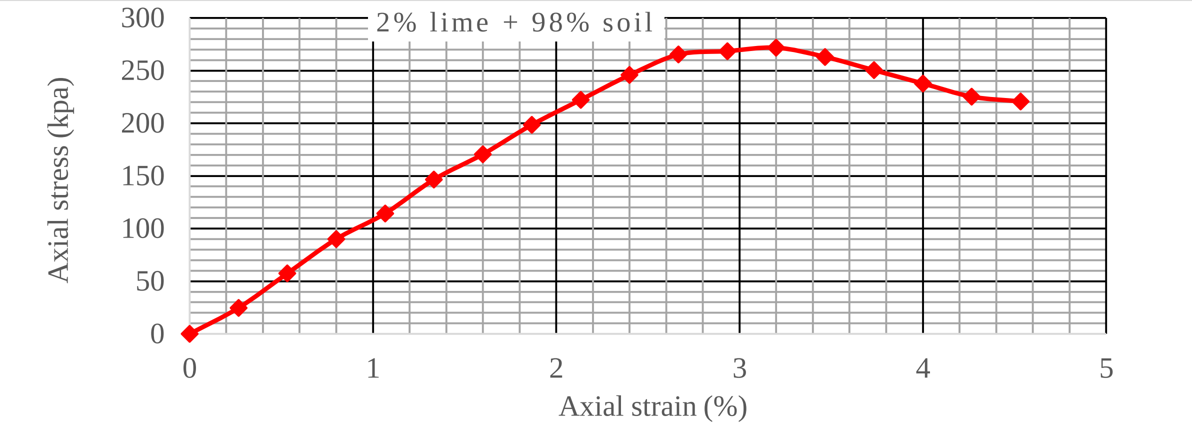


Fig 4.1.2 Stress–strain curves from UCS test for 2% lime + 98% soil sample one

Table 4.1.3 UCS test for 4%Lime + 96% soil of sample one

Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation (ΔL)=$Dr * 0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr * cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	5.5	46.2	0.00102	45.29056
40	0.4	0.005333	10.5	88.2	0.001023	86.2326
60	0.6	0.008	15.5	130.2	0.001026	126.9545
80	0.8	0.010667	21	176.4	0.001028	171.5405
100	1	0.013333	26	218.4	0.001031	211.811
120	1.2	0.016	31	260.4	0.001034	251.8613
140	1.4	0.018667	36	302.4	0.001037	291.6914
160	1.6	0.021333	40	336	0.00104	323.2209
180	1.8	0.024	42.5	357	0.001042	342.4864
200	2	0.026667	45	378	0.001045	361.6419
220	2.2	0.029333	45	378	0.001048	360.6511
240	2.4	0.032	44.5	373.8	0.001051	355.6641
260	2.6	0.034667	44	369.6	0.001054	350.6991
280	2.8	0.037333	42.5	357	0.001057	337.8077
300	3	0.04	41	344.4	0.00106	324.9823
320	3.2	0.042667	39.5	331.8	0.001063	312.223
340	3.4	0.045333	39	327.6	0.001066	307.4121
360	3.6	0.048	37	310.8	0.001069	290.8327

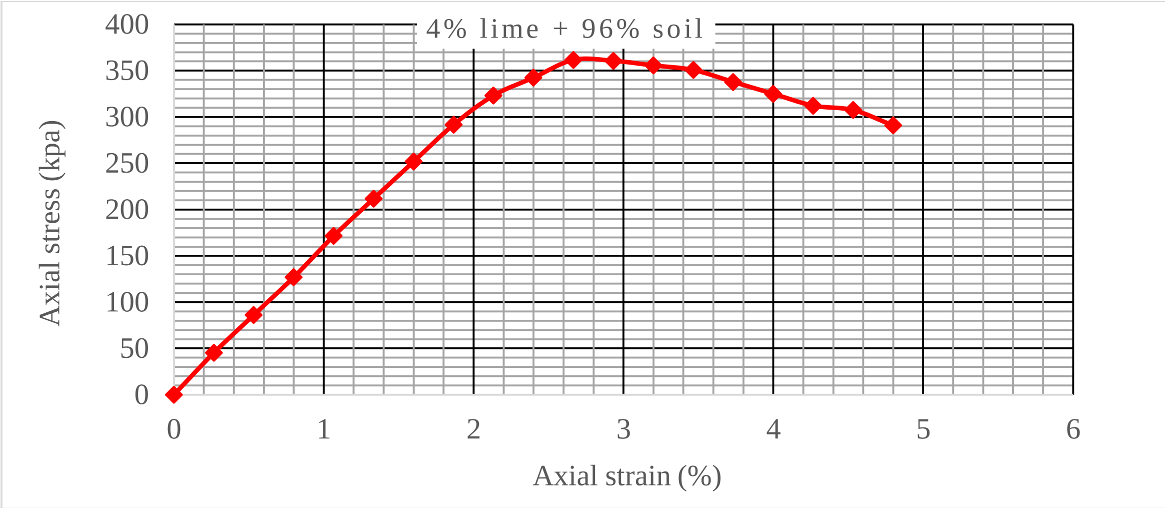


Fig 4.1.3 Stress–strain curves from UCS test for 4% lime + 96% soil sample one

Table 4.1.4 UCS test for 6%Lime + 94% soil of sample one

Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation (ΔL)=$Dr*$ 0.01 (mm)	Axial stain(ϵ)= $\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr*cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress= $\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	9.5	79.8	0.00102	78.22914
40	0.4	0.005333	19	159.6	0.001023	156.0399
60	0.6	0.008	28	235.2	0.001026	229.3371
80	0.8	0.010667	37	310.8	0.001028	302.2379
100	1	0.013333	41.5	348.6	0.001031	338.0829
120	1.2	0.016	45	378	0.001034	365.6051
140	1.4	0.018667	47.5	399	0.001037	384.8706
160	1.6	0.021333	48	403.2	0.00104	387.8651
180	1.8	0.024	47.5	399	0.001042	382.779
200	2	0.026667	47	394.8	0.001045	377.7149
220	2.2	0.029333	46	386.4	0.001048	368.6656
240	2.4	0.032	45.5	382.2	0.001051	363.6565
260	2.6	0.034667	45	378	0.001054	358.6695
280	2.8	0.037333	43	361.2	0.001057	341.7819

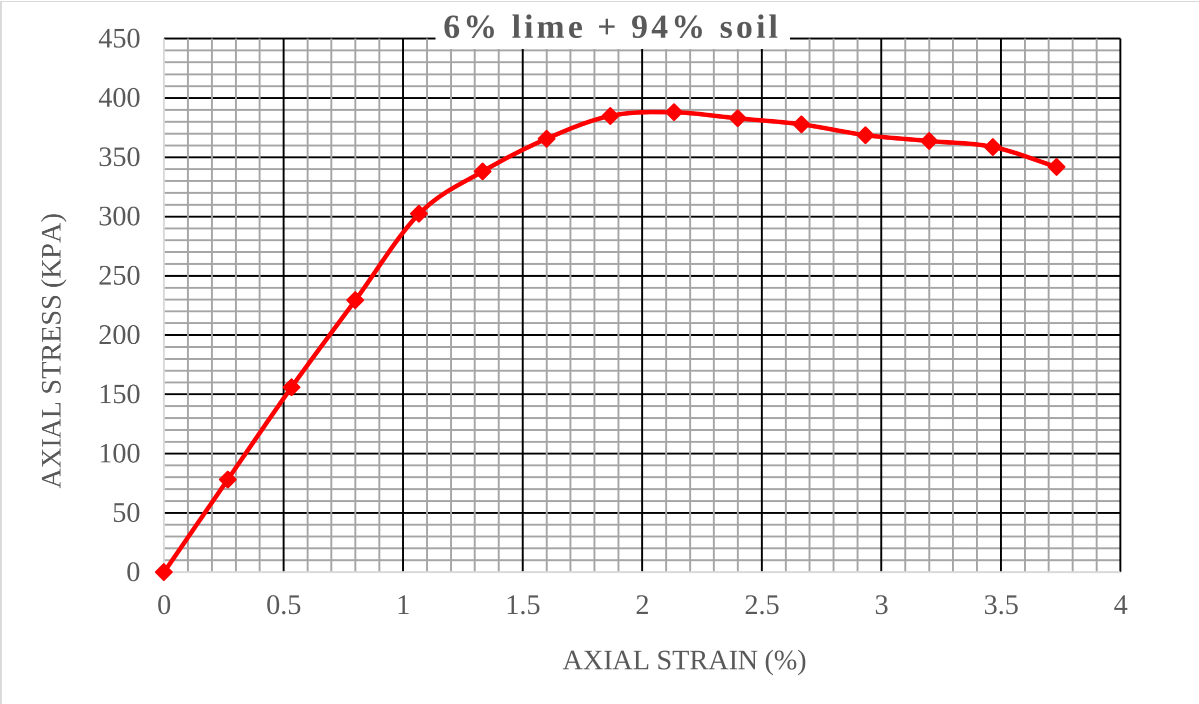


Fig 4.1.4 Stress–strain curves from UCS test for 6% lime + 94% soil sample one

Table 4.1.5 UCS test for 8%Lime + 92% soil of sample one

Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation (ΔL)=$Dr * 0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr * cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	16.5	138.6	0.00102	135.8717
40	0.4	0.005333	29.5	247.8	0.001023	242.2725
60	0.6	0.008	41.5	348.6	0.001026	339.9104
80	0.8	0.010667	48	403.2	0.001028	392.0925
100	1	0.013333	51	428.4	0.001031	415.4753
120	1.2	0.016	53.5	449.4	0.001034	434.6638
140	1.4	0.018667	53	445.2	0.001037	429.4346
160	1.6	0.021333	52	436.8	0.00104	420.1872
180	1.8	0.024	50	420	0.001042	402.9252
200	2	0.026667	48	403.2	0.001045	385.7514

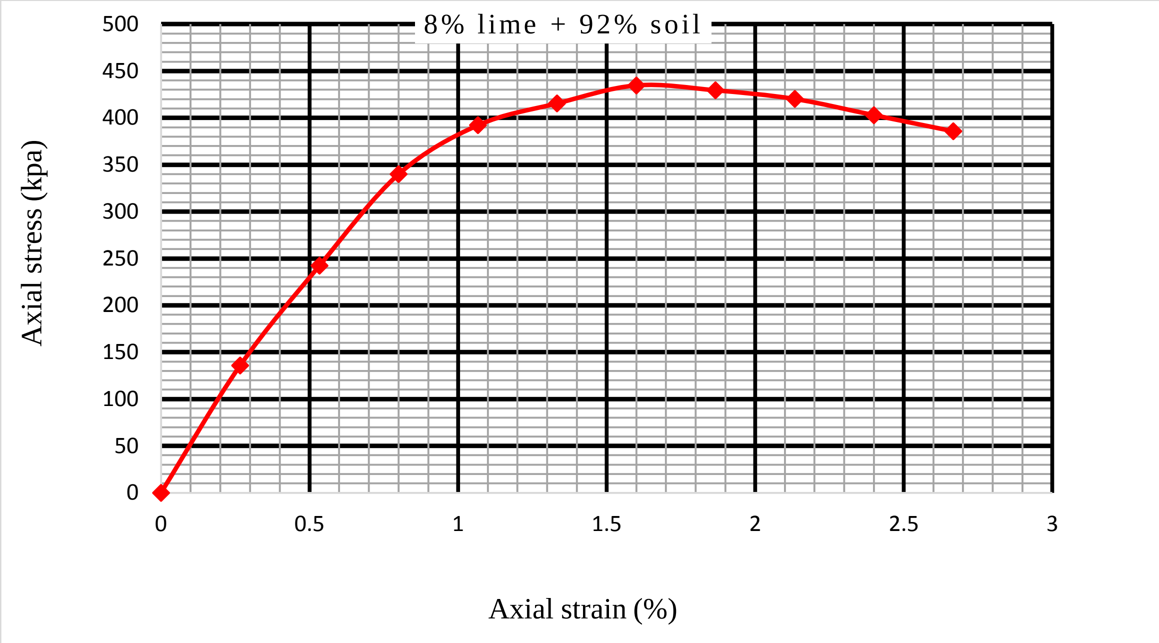


Fig 4.1.5 Stress–strain curves from UCS test for 8% lime + 92% soil sample one

Table 4.1.6 UCS test for 2% cement + 98% soil of sample one

Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation (ΔL)=$Dr * 0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr * cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	2	16.8	0.00102	16.47512
40	0.4	0.005333	5	42	0.001022	41.07768
60	0.6	0.008	8	67.2	0.001025	65.54808
80	0.8	0.010667	11.5	96.6	0.001028	93.97207
100	1	0.013333	14	117.6	0.001031	114.0924
120	1.2	0.016	17.5	147	0.001034	142.2301
140	1.4	0.018667	21	176.4	0.001036	170.2136
160	1.6	0.021333	23	193.2	0.001039	185.9178
180	1.8	0.024	26.5	222.6	0.001042	213.626
200	2	0.026667	28.5	239.4	0.001045	229.1209
220	2.2	0.029333	31	260.4	0.001048	248.5365
240	2.4	0.032	33	277.2	0.001051	263.8442
260	2.6	0.034667	35	294	0.001054	279.0639
280	2.8	0.037333	36	302.4	0.001056	286.2442
300	3	0.04	37	310.8	0.001059	293.3805
320	3.2	0.042667	37.5	315	0.001062	296.5192
340	3.4	0.045333	38	319.2	0.001065	299.6358
360	3.6	0.048	37.5	315	0.001068	294.8673
380	3.8	0.050667	36.5	306.6	0.001071	286.2002
400	4	0.053333	34	285.6	0.001074	265.8486
420	4.2	0.056	31.5	264.6	0.001077	245.6071
440	4.4	0.058667	27.5	231	0.00108	213.8132

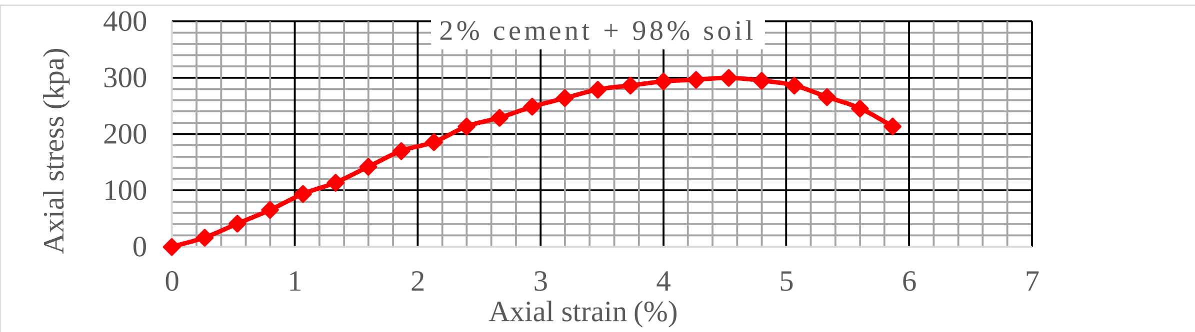
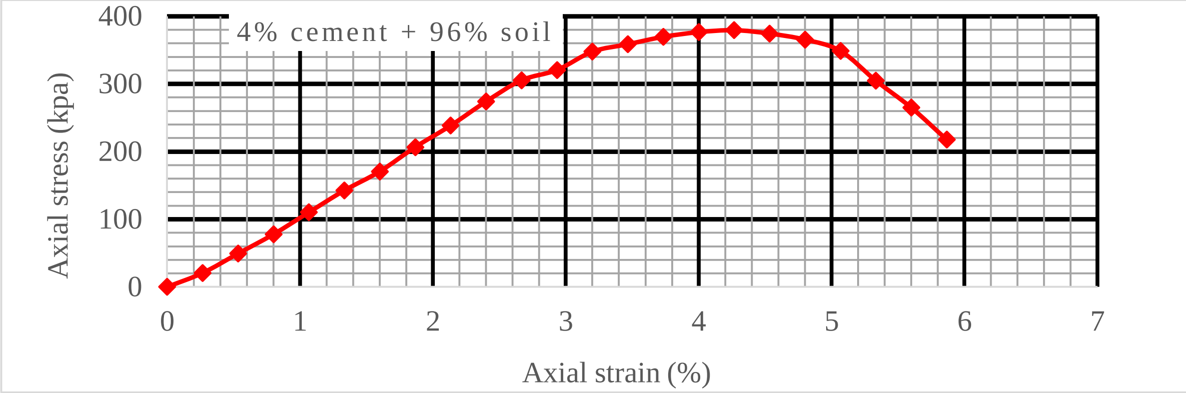


Fig 4.1.6 Stress–strain curves from UCS test for 2% Cement + 92% soil sample one

Table 4.1.7 UCS test for 4% cement + 96% soil of sample one

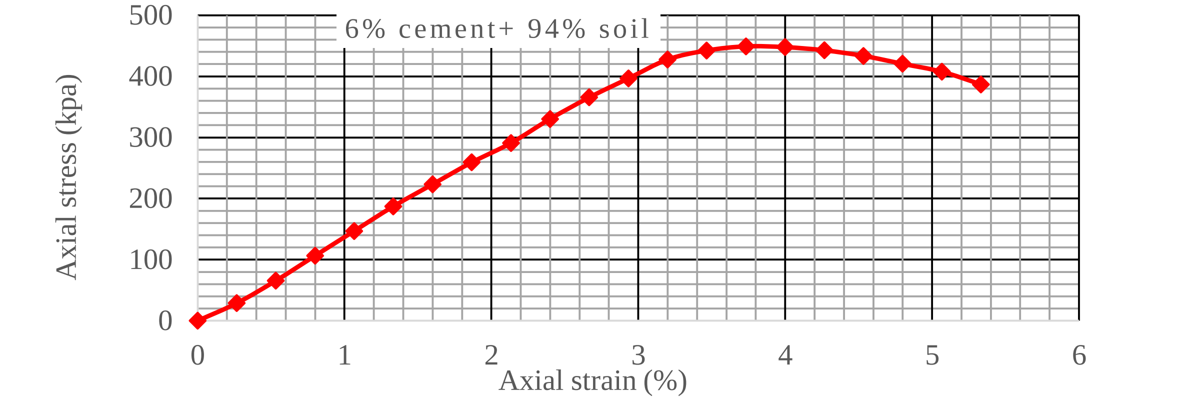
Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation $(\Delta L)=Dr * 0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr * cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	2.5	21	0.00102	20.58662
40	0.4	0.005333	6	50.4	0.001023	49.27577
60	0.6	0.008	9.5	79.8	0.001026	77.8108
80	0.8	0.010667	13.5	113.4	0.001028	110.276
100	1	0.013333	17.5	147	0.001031	142.5651
120	1.2	0.016	21	176.4	0.001034	170.6157
140	1.4	0.018667	25.5	214.2	0.001037	206.6148
160	1.6	0.021333	29.5	247.8	0.00104	238.3754
180	1.8	0.024	34	285.6	0.001042	273.9891
200	2	0.026667	38	319.2	0.001045	305.3865
220	2.2	0.029333	40	336	0.001048	320.5788
240	2.4	0.032	43.5	365.4	0.001051	347.6716
260	2.6	0.034667	45	378	0.001054	358.6695
280	2.8	0.037333	46.5	390.6	0.001057	369.6013
300	3	0.04	47.5	399	0.00106	376.5039
320	3.2	0.042667	48	403.2	0.001063	379.4102
340	3.4	0.045333	47.5	399	0.001066	374.4122
360	3.6	0.048	46.5	390.6	0.001069	365.506
380	3.8	0.050667	44.5	373.8	0.001072	348.8055
400	4	0.053333	39	327.6	0.001075	304.836
420	4.2	0.056	34	285.6	0.001078	265.0059
440	4.4	0.058667	28	235.2	0.001081	217.6237



4.1.7 Stress–strain curves from UCS test for 4% Cement + 96% soil sample one

Table 4.1.8 UCS test for 6% cement + 94% soil of sample one

Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation $(\Delta L)=Dr*0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr*cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	3.5	29.4	0.00102	28.82126
40	0.4	0.005333	8	67.2	0.001023	65.70103
60	0.6	0.008	13	109.2	0.001026	106.4779
80	0.8	0.010667	18	151.2	0.001028	147.0347
100	1	0.013333	23	193.2	0.001031	187.3712
120	1.2	0.016	27.5	231	0.001034	223.4253
140	1.4	0.018667	32	268.8	0.001037	259.2813
160	1.6	0.021333	36	302.4	0.00104	290.8988
180	1.8	0.024	41	344.4	0.001042	330.3987
200	2	0.026667	45.5	382.2	0.001045	365.6601
220	2.2	0.029333	49.5	415.8	0.001048	396.7162
240	2.4	0.032	53.5	449.4	0.001051	427.5961
260	2.6	0.034667	55.5	466.2	0.001054	442.359
280	2.8	0.037333	56.5	474.6	0.001057	449.0855
300	3	0.04	56.5	474.6	0.00106	447.8415
320	3.2	0.042667	56	470.4	0.001063	442.6453
340	3.4	0.045333	55	462	0.001066	433.5299
360	3.6	0.048	53.5	449.4	0.001069	420.5284
380	3.8	0.050667	52	436.8	0.001072	407.593
400	4	0.053333	49.5	415.8	0.001075	386.9073



4.1.8 Stress–strain curves from UCS test for 6% Cement + 94% soil sample one

Table 4.1.9 UCS test for 8% cement + 92% soil of sample one

Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation (ΔL)=$Dr*0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr*cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	4.5	37.8	0.00102	37.05591
40	0.4	0.005333	10.5	88.2	0.001023	86.2326
60	0.6	0.008	17	142.8	0.001026	139.2404
80	0.8	0.010667	24	201.6	0.001028	196.0462
100	1	0.013333	32	268.8	0.001031	260.6904
120	1.2	0.016	37.5	315	0.001034	304.6709
140	1.4	0.018667	43.5	365.4	0.001037	352.4605
160	1.6	0.021333	50.5	424.2	0.00104	408.0664
180	1.8	0.024	54.5	457.8	0.001042	439.1885
200	2	0.026667	58.5	491.4	0.001045	470.1345
220	2.2	0.029333	60.8	510.72	0.001048	487.2797
240	2.4	0.032	62.5	525	0.001051	499.5282
260	2.6	0.034667	62.5	525	0.001054	498.1521
280	2.8	0.037333	62	520.8	0.001057	492.8018
300	3	0.04	61	512.4	0.00106	483.5103
320	3.2	0.042667	58	487.2	0.001063	458.454
340	3.4	0.045333	55.5	466.2	0.001066	437.4711
360	3.6	0.048	52	436.8	0.001069	408.7379

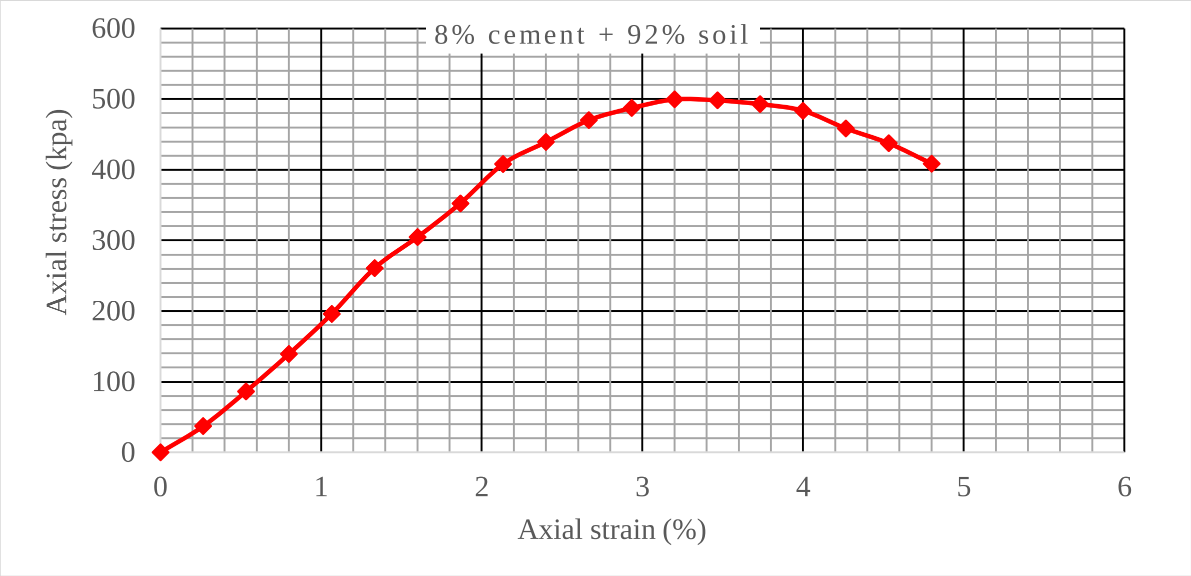


Fig 4.1.9 Stress–strain curves from UCS test for 8% Cement + 92% soil sample one

4.2 Sample one 7 days curing duration

Table 4.2.1 UCS test for 2% Lime + 98% soil of sample one (7 day curing duration)

Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation (ΔL)=$Dr * 0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr * cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	8	67.2	0.00102	65.87717
40	0.4	0.005333	14	117.6	0.001023	114.9768
60	0.6	0.008	21	176.4	0.001026	172.0028
80	0.8	0.010667	25.5	214.2	0.001028	208.2991
100	1	0.013333	30	252	0.001031	244.3973
120	1.2	0.016	33.5	281.4	0.001034	272.1727
140	1.4	0.018667	37.5	315	0.001037	303.8452
160	1.6	0.021333	40	336	0.00104	323.2209
180	1.8	0.024	42.5	357	0.001042	342.4864
200	2	0.026667	44	369.6	0.001045	353.6054
220	2.2	0.029333	45	378	0.001048	360.6511
240	2.4	0.032	45.5	382.2	0.001051	363.6565
260	2.6	0.034667	45.5	382.2	0.001054	362.6547
280	2.8	0.037333	45	378	0.001057	357.6787
300	3	0.04	45	378	0.00106	356.6879
320	3.2	0.042667	43	361.2	0.001063	339.8883
340	3.4	0.045333	40	336	0.001066	315.2945
360	3.6	0.048	37.5	315	0.001069	294.7629
380	3.8	0.050667	33.5	281.4	0.001072	262.5839
400	4	0.053333	32	268.8	0.001075	250.1219

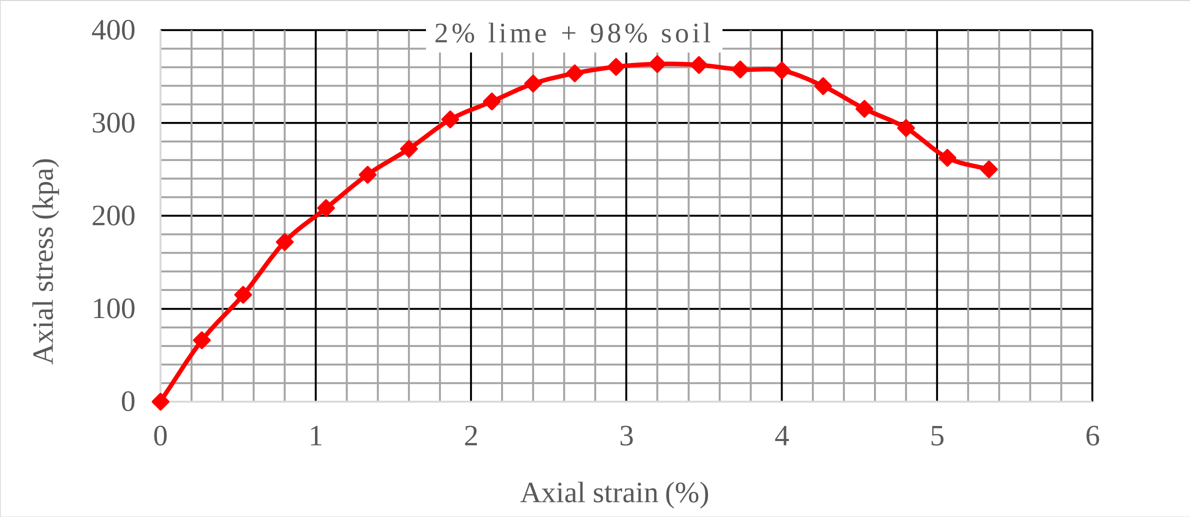


Fig 4.2.1 Stress–strain curves from UCS test for 2% Lime + 98% soil sample one (7 days curing duration)

Table 4.2.2 UCS test for 4% Lime + 96% soil of sample one (7 day curing duration)

Lo=75mm ,Ao=0.00101736, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation (ΔL)=$Dr * 0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr * cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	8	67.2	0.00102	65.87717
40	0.4	0.005333	16	134.4	0.001023	131.4021
60	0.6	0.008	25.5	214.2	0.001026	208.8606
80	0.8	0.010667	34.5	289.8	0.001028	281.8165
100	1	0.013333	43	361.2	0.001031	350.3027
120	1.2	0.016	50	420	0.001034	406.2279
140	1.4	0.018667	54	453.6	0.001037	437.5372
160	1.6	0.021333	56	470.4	0.00104	452.5092
180	1.8	0.024	54.5	457.8	0.001042	439.1885
200	2	0.026667	50.5	424.2	0.001045	405.8426
220	2.2	0.029333	47.5	399	0.001048	380.6873
240	2.4	0.032	45	378	0.001051	359.6603

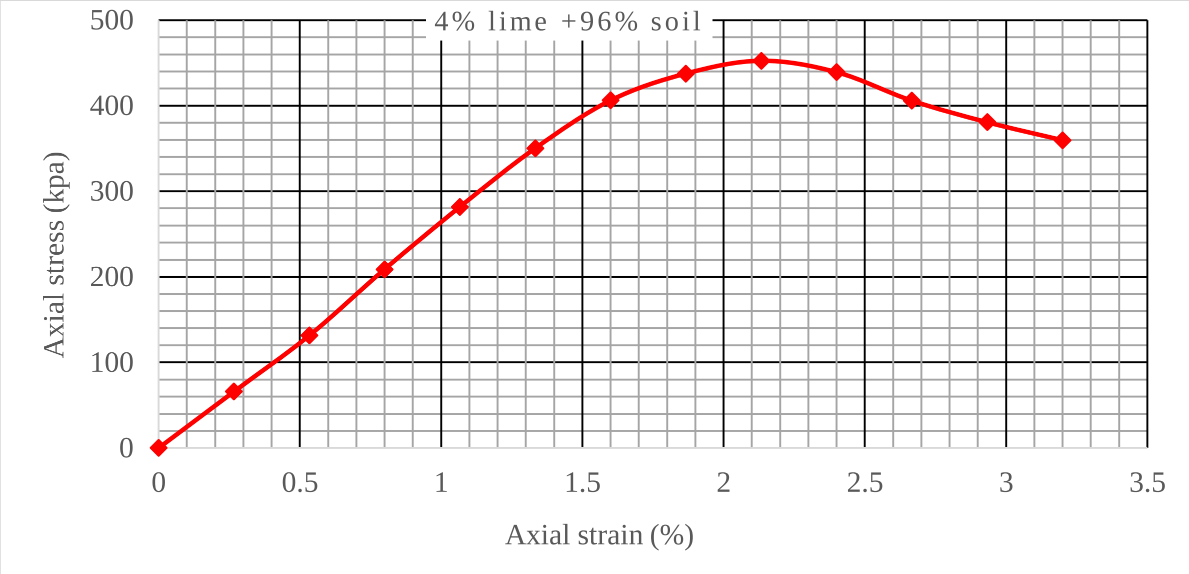


Fig 4.2.2 Stress–strain curves from UCS test for 4% Lime + 96% soil sample one (7 days curing duration)

Table 4.2.3 UCS test for 6% Lime + 94% soil of sample one (7 day curing duration)

Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation (ΔL)=$Dr * 0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr * cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	9	75.6	0.00102	74.11182
40	0.4	0.005333	23.5	197.4	0.001023	192.9968
60	0.6	0.008	37	310.8	0.001026	303.0526
80	0.8	0.010667	49.5	415.8	0.001028	404.3454
100	1	0.013333	57	478.8	0.001031	464.3548
120	1.2	0.016	59.5	499.8	0.001034	483.4112
140	1.4	0.018667	60.5	508.2	0.001037	490.2037
160	1.6	0.021333	58.5	491.4	0.00104	472.7105
180	1.8	0.024	56	470.4	0.001042	451.2762
200	2	0.026667	54	453.6	0.001045	433.9703
220	2.2	0.029333	52	436.8	0.001048	416.7524

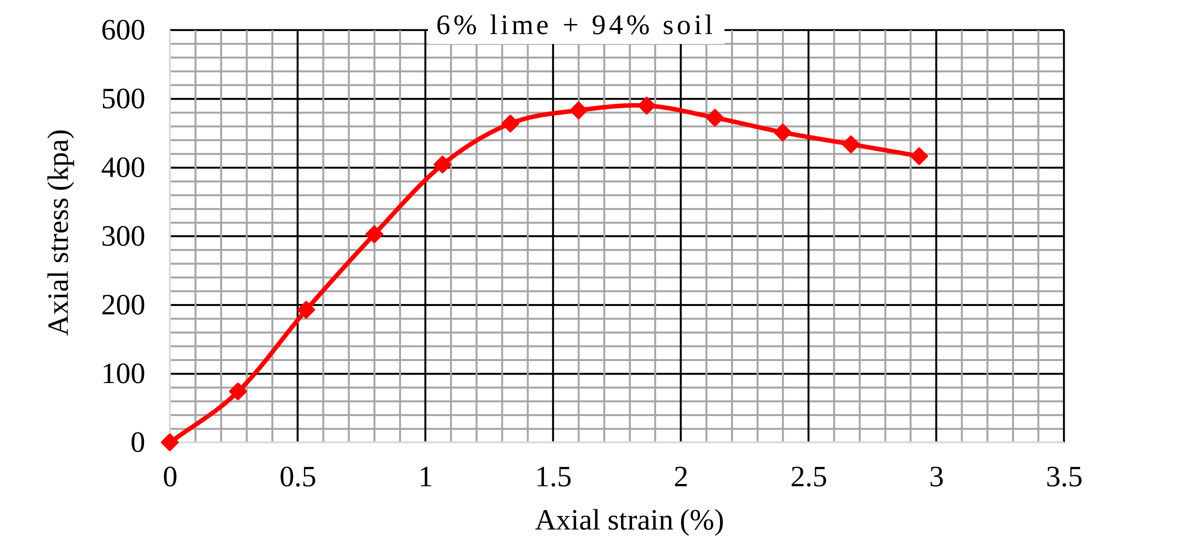


Fig 4.2.3 Stress–strain curves from UCS test for 6% Lime + 94% soil sample one (7 days curing duration)

Table 4.2.4 UCS test for 8% Lime + 92% soil of sample one (7 day curing duration)

Lo=75mm ,Ao=0.00101736, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation (ΔL)=$Dr * 0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr * cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	24	201.6	0.00102	197.6315
40	0.4	0.005333	46	386.4	0.001023	377.7809
60	0.6	0.008	60.5	508.2	0.001026	495.532
80	0.8	0.010667	67	562.8	0.001028	547.2957
100	1	0.013333	67.5	567	0.001031	549.8938
120	1.2	0.016	66	554.4	0.001034	536.2208
140	1.4	0.018667	62.5	525	0.001037	506.4087
160	1.6	0.021333	58.5	491.4	0.00104	472.7105
180	1.8	0.024	53.5	449.4	0.001042	431.13

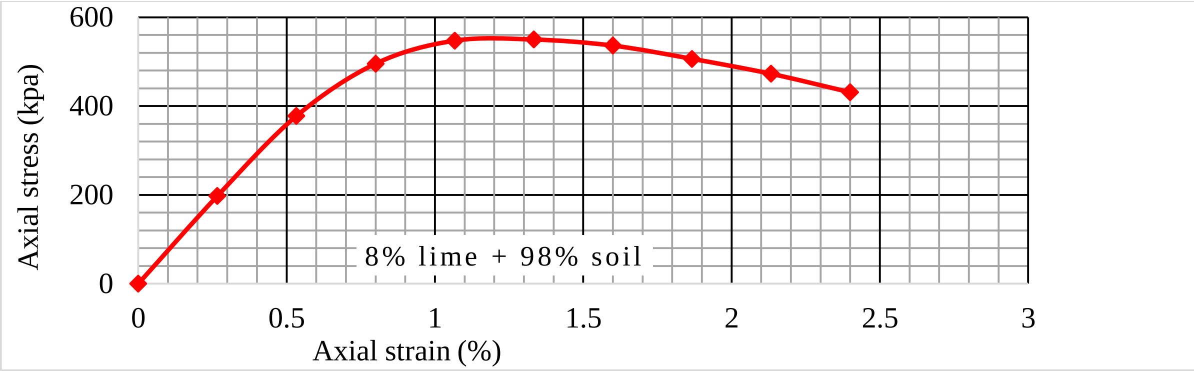


Fig 4.2.4 Stress–strain curves from UCS test for 8% Lime + 92% soil sample one (7 days curing duration)

Table 4.2.5 UCS test for 2% Cement+ 98% soil of sample one (7 day curing duration)

Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation (ΔL)=$Dr * 0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr * cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	4	33.6	0.00102	32.93859
40	0.4	0.005333	9	75.6	0.001023	73.91366
60	0.6	0.008	14	117.6	0.001026	114.6686
80	0.8	0.010667	19.5	163.8	0.001028	159.2876
100	1	0.013333	24	201.6	0.001031	195.5178
120	1.2	0.016	29	243.6	0.001034	235.6122
140	1.4	0.018667	33.5	281.4	0.001037	271.4351
160	1.6	0.021333	39	327.6	0.00104	315.1404
180	1.8	0.024	43	361.2	0.001042	346.5157
200	2	0.026667	46.5	390.6	0.001045	373.6966
220	2.2	0.029333	49	411.6	0.001048	392.709
240	2.4	0.032	51.5	432.6	0.001051	411.6112
260	2.6	0.034667	52	436.8	0.001054	414.4625
280	2.8	0.037333	51	428.4	0.001057	405.3692
300	3	0.04	49	411.6	0.00106	388.3935
320	3.2	0.042667	47	394.8	0.001063	371.5059
340	3.4	0.045333	44.5	373.8	0.001066	350.7651
360	3.6	0.048	43	361.2	0.001069	337.9948

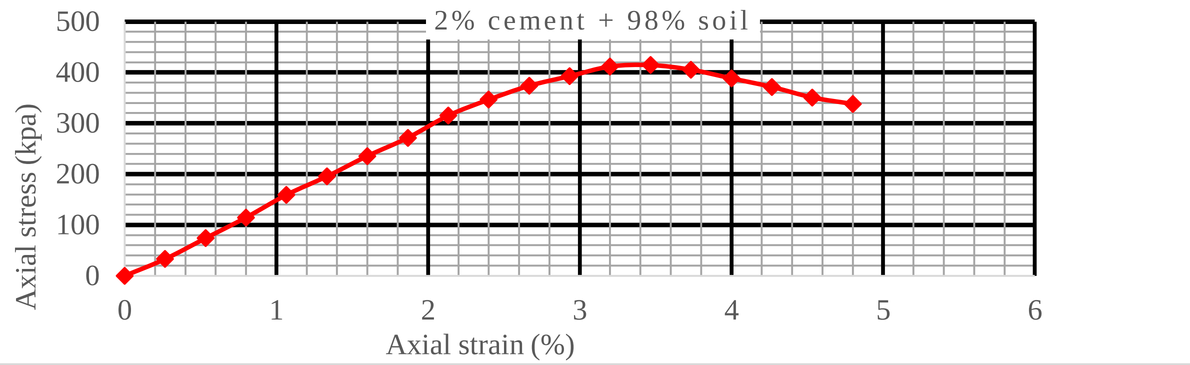


Fig 4.2.5 Stress–strain curves from UCS test for 2% cement + 98% soil sample one (7 days curing duration)

Table 4.2.6 UCS test for 2% Cement+ 98% soil of sample one (7 day curing duration)

Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation (ΔL)=$Dr * 0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr * cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	8	67.2	0.00102	65.87717
40	0.4	0.005333	15	126	0.001023	123.1894
60	0.6	0.008	21.5	180.6	0.001026	176.0981
80	0.8	0.010667	29	243.6	0.001028	236.8892
100	1	0.013333	35.5	298.2	0.001031	289.2034
120	1.2	0.016	42	352.8	0.001034	341.2314
140	1.4	0.018667	48.5	407.4	0.001037	392.9732
160	1.6	0.021333	54.5	457.8	0.00104	440.3885
180	1.8	0.024	58.5	491.4	0.001042	471.4225
200	2	0.026667	62.5	525	0.001045	502.2804
220	2.2	0.029333	64.5	541.8	0.001048	516.9332
240	2.4	0.032	64.5	541.8	0.001051	515.5131
260	2.6	0.034667	64.5	541.8	0.001054	514.0929
280	2.8	0.037333	63.5	533.4	0.001057	504.7244
300	3	0.04	61.5	516.6	0.00106	487.4735
320	3.2	0.042667	58	487.2	0.001063	458.454
340	3.4	0.045333	56	470.4	0.001066	441.4123
360	3.6	0.048	54.5	457.8	0.001069	428.3888

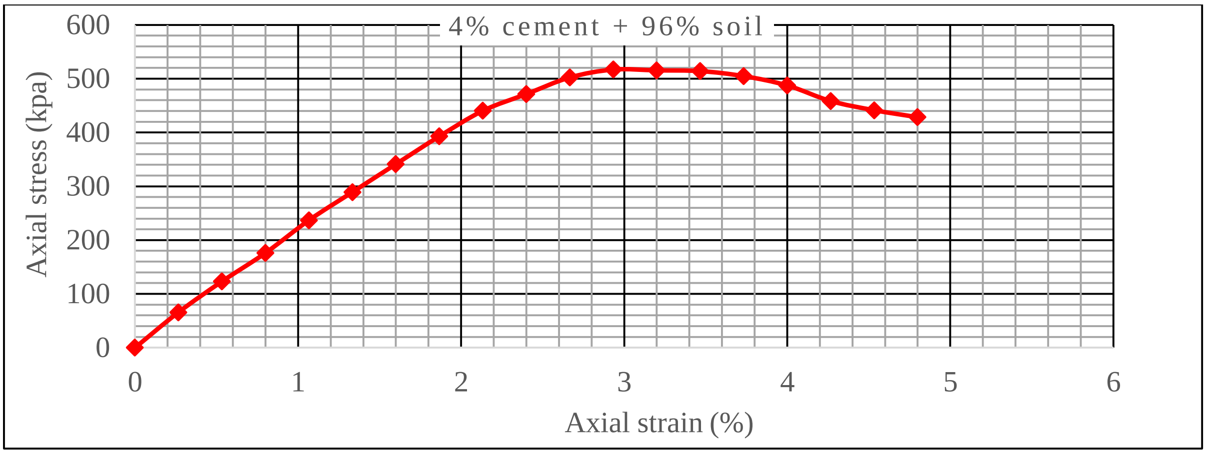


Fig 4.2.6 Stress–strain curves from UCS test for 4% cement + 96% soil sample one (7 days curing duration)

Table 4.2.7 UCS test for 6% Cement+ 94% soil of sample one (7 day curing duration)

Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation (ΔL)=$Dr * 0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr * cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	9.5	79.8	0.00102	78.22914
40	0.4	0.005333	20	168	0.001023	164.2526
60	0.6	0.008	29.5	247.8	0.001026	241.623
80	0.8	0.010667	39	327.6	0.001028	318.5751
100	1	0.013333	49	411.6	0.001031	399.1822
120	1.2	0.016	57.5	483	0.001034	467.1621
140	1.4	0.018667	65	546	0.001037	526.6651
160	1.6	0.021333	69.5	583.8	0.00104	561.5963
180	1.8	0.024	73	613.2	0.001042	588.2708
200	2	0.026667	75	630	0.001045	602.7365
220	2.2	0.029333	73	613.2	0.001048	585.0562
240	2.4	0.032	70.5	592.2	0.001051	563.4678
260	2.6	0.034667	66	554.4	0.001054	526.0486
280	2.8	0.037333	63.5	533.4	0.001057	504.7244
300	3	0.04	60	504	0.00106	475.5839

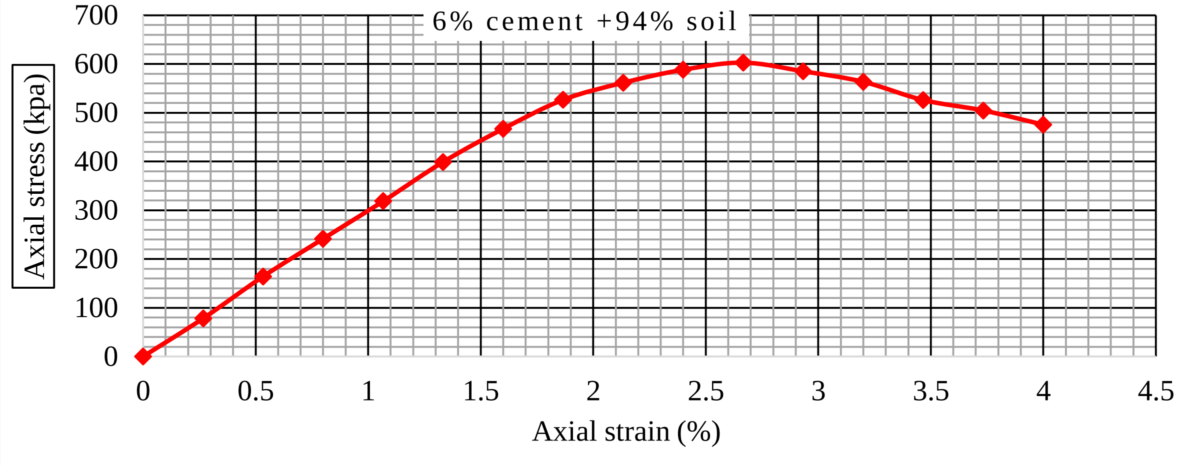


Fig 4.2.7 Stress–strain curves from UCS test for 6% cement + 94% soil sample one (7 days curing duration)

Table 4.2.8 UCS test for 8% Cement+ 92% soil of sample one (7 day curing duration)

Lo=75mm , Ao=0.00101736 m^2, and ring factor (cf)=8.4N/division						
Division reading (Dr)	Axial deformation (ΔL)=$Dr * 0.01$ (mm)	Axial stain(ϵ)=$\frac{\Delta L}{Lo}$	proving ring dial reading (Pr)	$Load = Pr * cf$ (N)	Corrected area (m^2) $Ac = \frac{Ao}{1-\epsilon}$	Stress=$\frac{Load}{1000*Ac}$ (Kpa)
0	0	0	0	0	0.001017	0
20	0.2	0.002667	17	142.8	0.00102	139.989
40	0.4	0.005333	32.5	273	0.001023	266.9104
60	0.6	0.008	48.5	407.4	0.001026	397.2446
80	0.8	0.010667	63	529.2	0.001028	514.6214
100	1	0.013333	75.5	634.2	0.001031	615.0664
120	1.2	0.016	84	705.6	0.001034	682.4628
140	1.4	0.018667	91	764.4	0.001037	737.3311
160	1.6	0.021333	94.5	793.8	0.00104	763.6093
180	1.8	0.024	94.5	793.8	0.001042	761.5287
200	2	0.026667	92.5	777	0.001045	743.375
220	2.2	0.029333	91.5	768.6	0.001048	733.3239
240	2.4	0.032	88.5	743.4	0.001051	707.3319
260	2.6	0.034667	86.5	726.6	0.001054	689.4425
280	2.8	0.037333	82	688.8	0.001057	651.7701

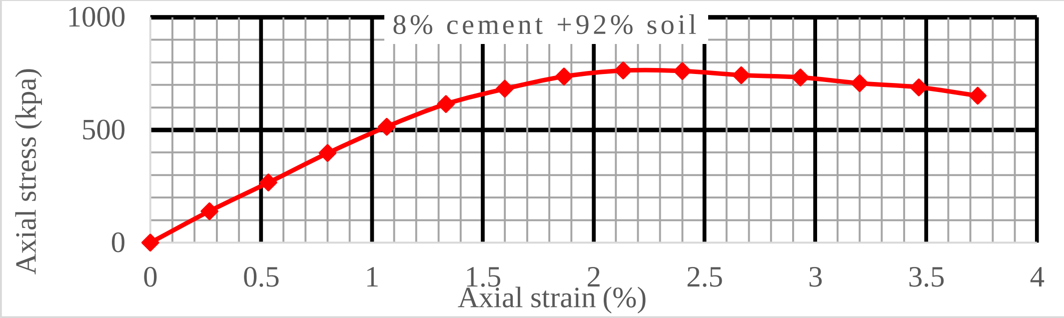


Fig 4.2.8 Stress–strain curves from UCS test for 8% cement + 92% soil sample one (7 days curing duration)

Appendix 5:-CBR Test Results for Different Lime and Cement Addition to the Expansive Soil

5.1Sample one

Table 5.1.1 Natural and Lime stabilized soil CBR test result (soil sample one)

Penetration (mm)	natural soil		2% lime & 98% soil		4% lime & 96% soil		6% lime & 94% soil		8% lime & 92% soil	
	Dial reading	Load (KN)	dial reading	Load	dial reading	Load	dial reading	Load	dial reading	Load
0	0	0	0	0	0	0	0	0	0	0
0.64	6	0.076	6	0.076	0	0	14	0.179	30	0.38
1.27	10	0.127	12	0.153	18	0.230	28	0.358	51	0.652
1.96	14	0.179	17	0.217	26	0.332	41	0.524	70	0.895
2.54	16	0.204	21	0.268	34	0.434	50	0.639	88	1.125
3.18	17	0.217	23	0.294	42	0.537	61	0.780	105	1.342
3.81	18	0.230	27	0.34533	49	0.626	72	0.920	119	1.522
4.45	19.5	0.249	30	0.383	56	0.716	80	1.023	137	1.752
5.08	21	0.268	33	0.422	65	0.831	89	1.138	154	1.969

calculation of CBR										
material type	natural soil		2% lime & 98% soil		4% lime & 96% soil		6% lime & 94% soil		8% lime & 91% soil	
penetration	2.5	5	2.5	5	2.5	5	2.5	5	2.5	5
load (KN)	0.205	0.267	0.26	0.41	0.41	0.81	0.62	1.12	1.1	1.95
Standard load	13.2	20	13.2	20	13.2	20	13.2	20	13.2	20
Corresp.CBR	1.553	1.338	1.969	2.05	3.106	4.05	4.696	5.6	8.333	9.7
CBR	**1.55**		**2.05**		**4.05**		**5.6**		**9.75**	

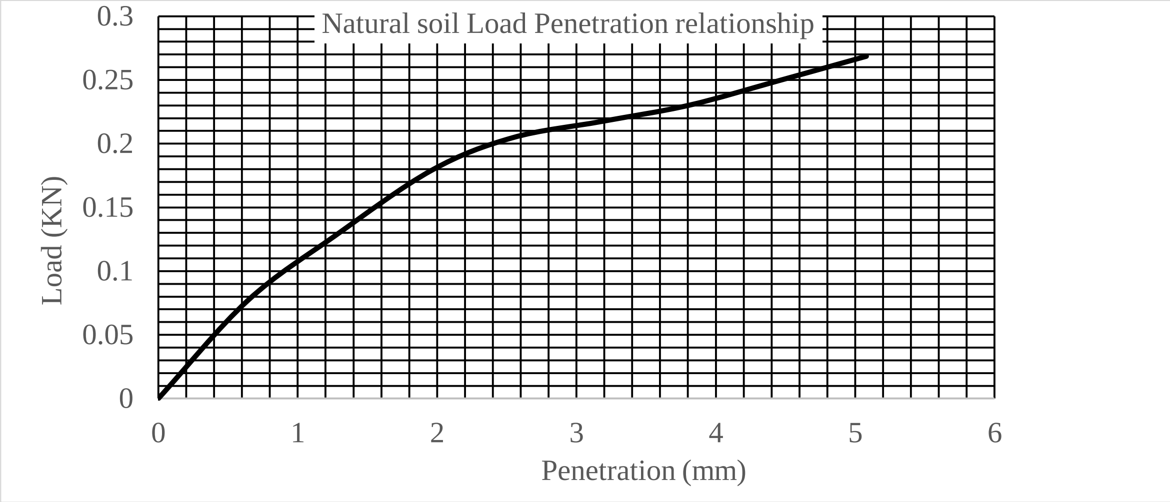


Fig 5.1.1 Loads vs. Penetration of natural Soil Sample one

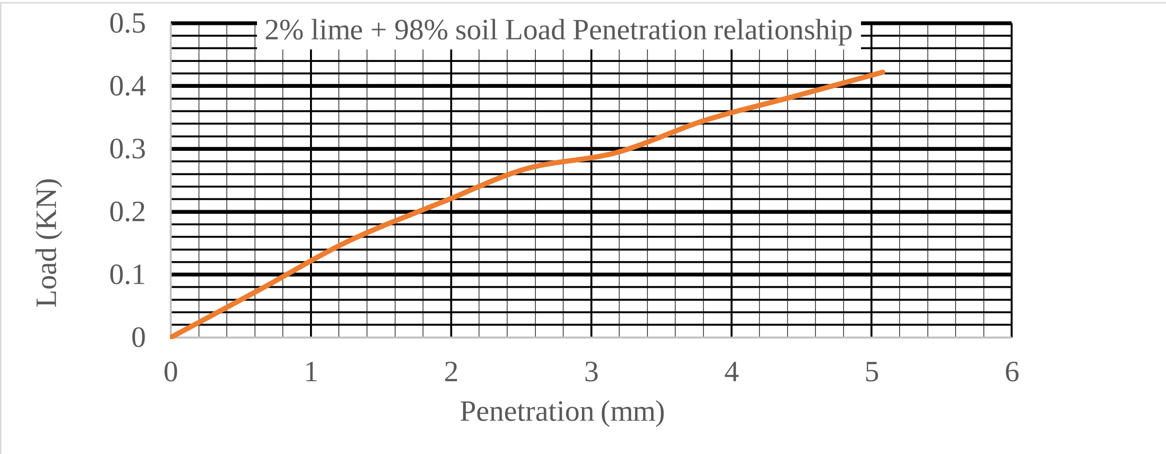


Fig 5.1.2 Loads vs. Penetration of natural Soil Sample one + 2% Lime

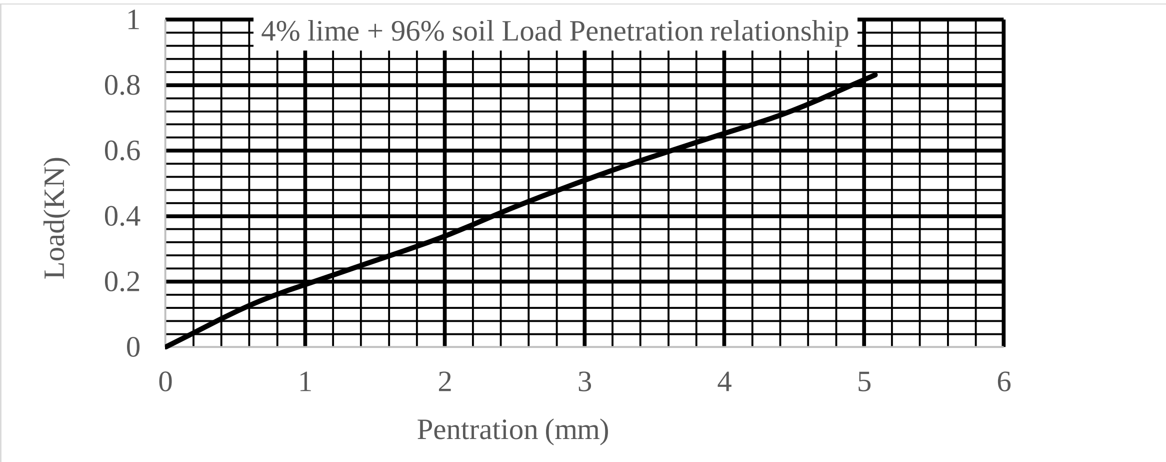


Fig 5.1.3 Loads vs. Penetration of natural Soil Sample one + 4% Lime

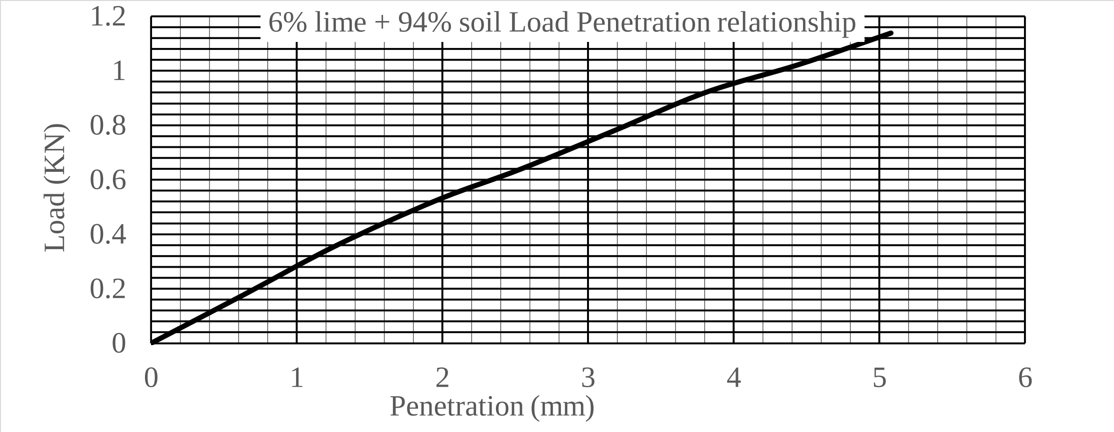


Fig 5.1.4 Loads vs. Penetration of natural Soil Sample one + 6% Lime

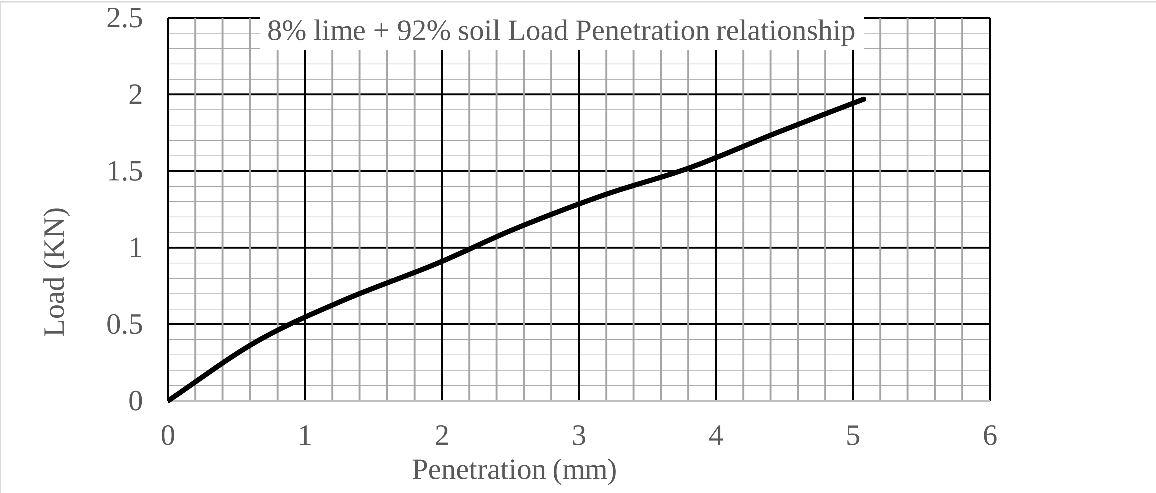


Fig 5.1.5 Loads vs. Penetration of natural Soil Sample one + 8% Lime

Table 5.1.1 Cement stabilized soil CBR test result (soil sample one)

No	Penetration (mm	2% Cement & 98% soil		4% Cement & 96% soil		6% Cement & 94% soil		8% cement & 91% soil	
		dial reading	Load (KN)	dial reading	Load (KN)	dial reading	Load (KN)	dial reading	Load (KN)
1	0	0	0	0	0	0	0	0	0
2	0.64	9	0.115	13	0.166	19	0.243	28	0.358
3	1.27	19	0.243	23	0.294	29	0.370	55	0.703
4	1.96	26	0.332	38	0.486	42	0.537	87	1.112
5	2.54	31	0.396	48	0.613	56	0.716	118	1.509
6	3.18	33.5	0.4284	57	0.729	68	0.869	158	2.020
7	3.81	36	0.460	66	0.844	86	1.099	199	2.545
8	4.45	38	0.486	77	0.984	99	1.266	235	3.005
9	5.08	41	0.524	82	1.048	112	1.432	271	3.466
calculation of CBR									
material type	Natural soil	2% lime & 98% soil		4% lime & 96% soil		6% lime & 94% soil		8% lime & 91% soil	
penetration		2.5	5	2.5	5	2.5	5	2.5	5
load (KN)		0.4	0.52	0.6	1.04	0.7	1.4	1.5	3.4
Standard load		13.2	20	13.2	20	13.2	20	13.2	20
corresponding CBR		3.030	2.6	4.545	5.2	5.303	7	11.363	17
CBR		3.03		5.2		7		17	

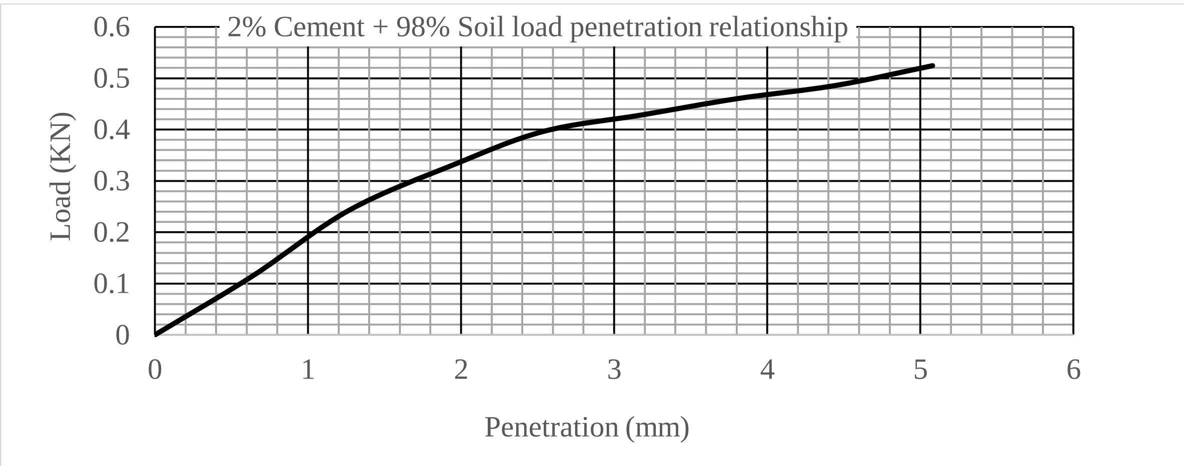


Fig 5.1.6 Loads vs. Penetration of natural Soil Sample one + 2% cement

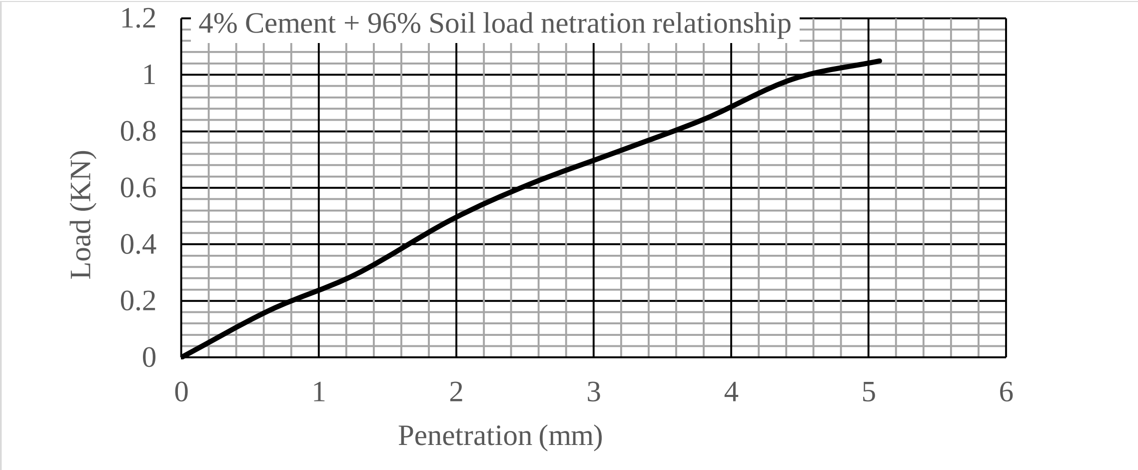


Fig 5.1.7 Loads vs. Penetration of natural Soil Sample one + 4% cement

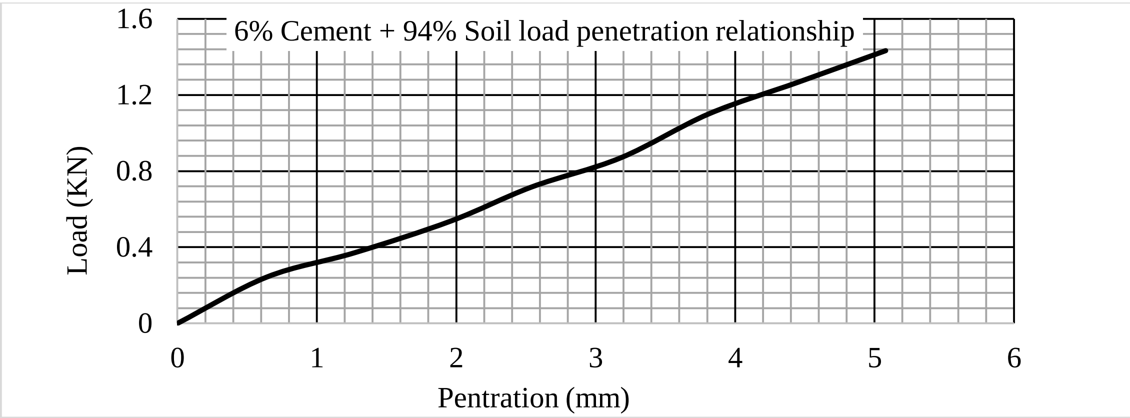


Fig 5.1.8 Loads vs. Penetration of natural Soil Sample one + 6% cement

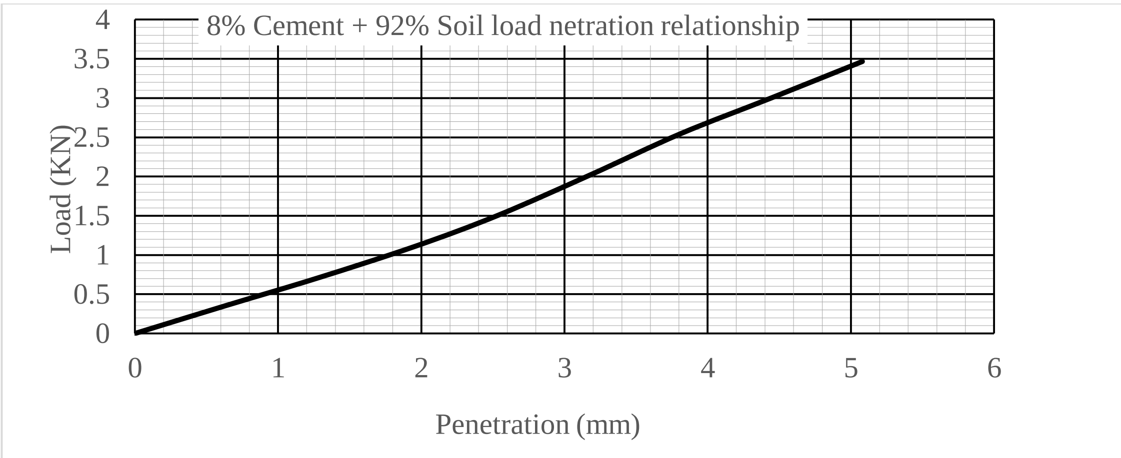


Fig 5.1.9 Loads vs. Penetration of natural Soil Sample one 8% cement

5.2Sample Two

Table 5.2.1 Natural and Lime stabilized soil CBR test result (soil sample Two)

Penetration (mm)	natural soil		2% lime & 98% soil		4% lime & 96% soil		6% lime & 94% soil		8% lime & 92% soil	
	Dial reading	Load (KN)	dial reading	Load	dial reading	Load	dial reading	Load	dial reading	Load
0	0	0	0	0	0	0	0	0	0	0
0.64	9	0.115	9	0.115	15	0.191	20.5	0.262	20.5	0.262
1.27	15	0.191	18	0.230	26	0.332	41.5	0.530	41.5	0.530
1.96	21	0.268	25	0.319	38	0.486	61	0.780	61	0.780
2.54	24	0.306	31	0.396	50	0.639	74	0.946	74	0.946
3.18	25	0.319	35	0.447	62	0.792	90.5	1.157	90.5	1.157
3.81	27	0.345	40	0.511	72	0.920	107	1.368	107	1.368
4.45	28	0.358	44.5	0.569	83	1.061	118.5	1.515	118.5	1.515
5.08	30	0.383	49	0.626	96	1.227	132	1.688	132	1.688
calculation of CBR										
material type	natural soil		2% lime & 98% soil		4% lime & 96% soil		6% lime & 94% soil		8% lime & 92% soil	
penetration	2.5	5	2.5	5	2.5	5	2.5	5	2.5	5
load (KN)	0.305	0.38	0.39	0.62	0.62	1.2	0.94	1.68	1.66	2.92
Standard load	13.2	20	13.2	20	13.2	20	13.2	20	13.2	20
corresponding CBR	2.310	1.9	2.954	3.1	4.697	6	7.121	8.4	12.575	14.6
CBR	**2.3**		**3.1**		**6**		**8.4**		**14.6**	

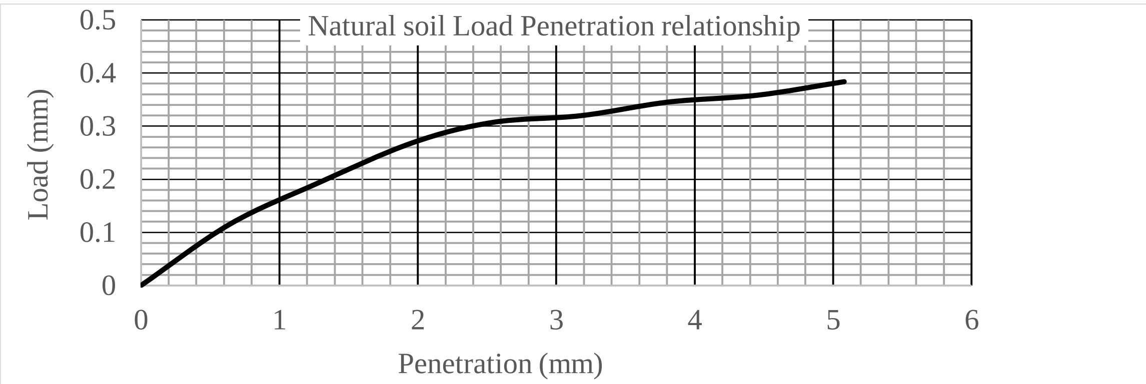


Fig 5.2.1 Loads vs. Penetration of natural Soil Sample two

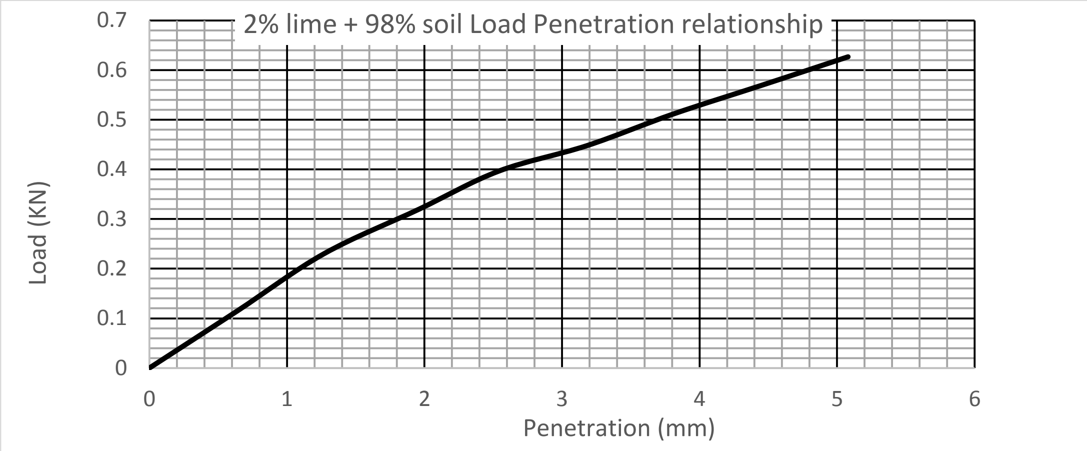


Fig 5.2.2 Loads vs. Penetration of natural Soil Sample one + 2% Lime

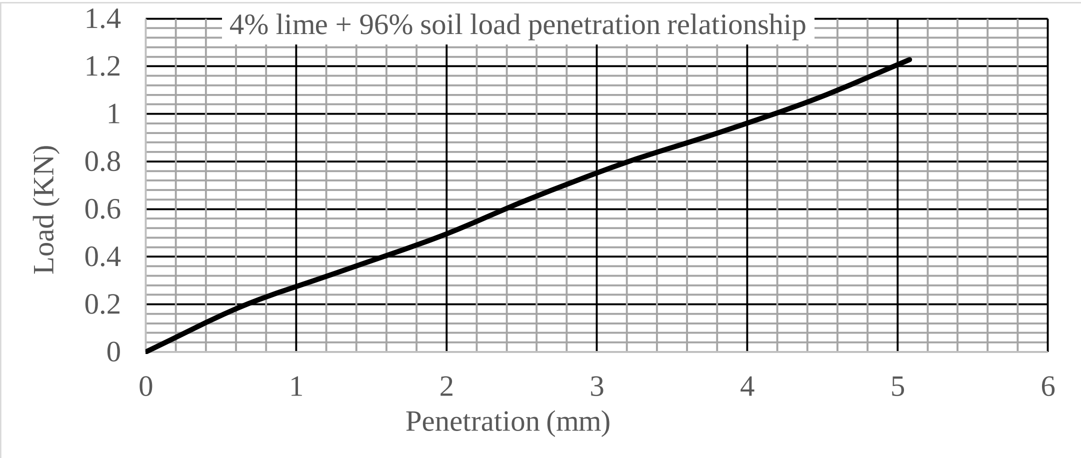


Fig 5.2.3 Loads vs. Penetration of natural Soil Sample one + 4% Lime

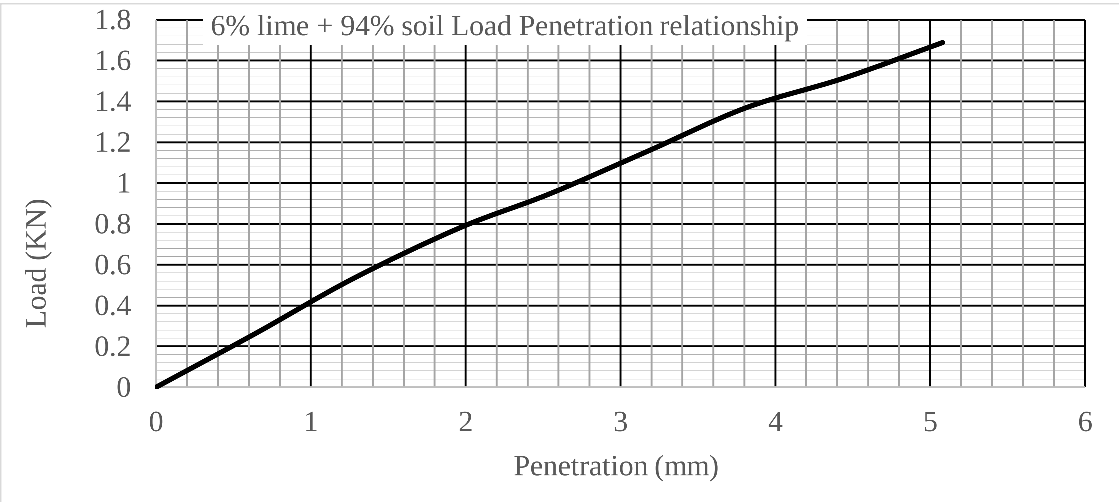
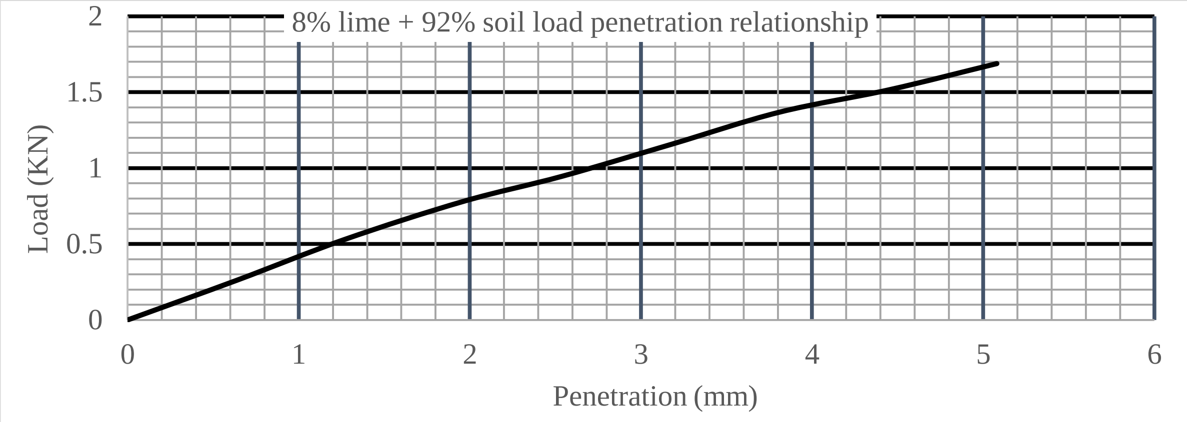


Fig 5.2.4 Loads vs. Penetration of natural Soil Sample one + 6% Lime



5.2.5 Loads vs. Penetration of natural Soil Sample one + 8% Lime

Table 4.2.2 cement stabilized soil CBR test result (soil sample one)

No	Penetration (mm	2% Cement & 98% soil		4% Cement & 96% soil		6% Cement & 94% soil		8% cement & 92% soil	
		dial reading	Load (KN)	dial reading	Load (KN)	dial reading	Load (KN)	dial reading	Load (KN)
1	0	0	0	0	0	0	0	0	0
2	0.64	13.5	0.172	19	0.24301	23	0.294	41.5	0.530
3	1.27	29	0.370	35	0.44765	43	0.549	81.5	1.042
4	1.96	39.5	0.505	56	0.71624	62	0.792	134	1.713
5	2.54	47	0.601	71	0.90809	83	1.061	185	2.366
6	3.18	51	0.652	81	1.03599	101	1.291	234.5	2.999
7	3.81	54.5	0.697	98	1.25342	127.5	1.630	295	3.773
8	4.45	57.5	0.735	114	1.45806	147	1.880	348.5	4.457
9	5.08	62	0.792	122	1.56038	166	2.123	402	5.141
calculation of CBR									
material type	natural soil	2% lime & 98% soil		4% lime & 96% soil		6% lime & 94% soil		8% lime & 92% soil	
penetration		2.5	5	2.5	5	2.5	5	2.5	5
load (KN)		0.6	0.78	0.88	1.56	1	2.1	2.3	5
Standard load		13.2	20	13.2	20	13.2	20	13.2	20
corresponding CBR		4.545	3.9	6.666	7.8	7.575	10.5	17.424	25
CBR		**4.5**		**7.8**		**10.5**		**25**	

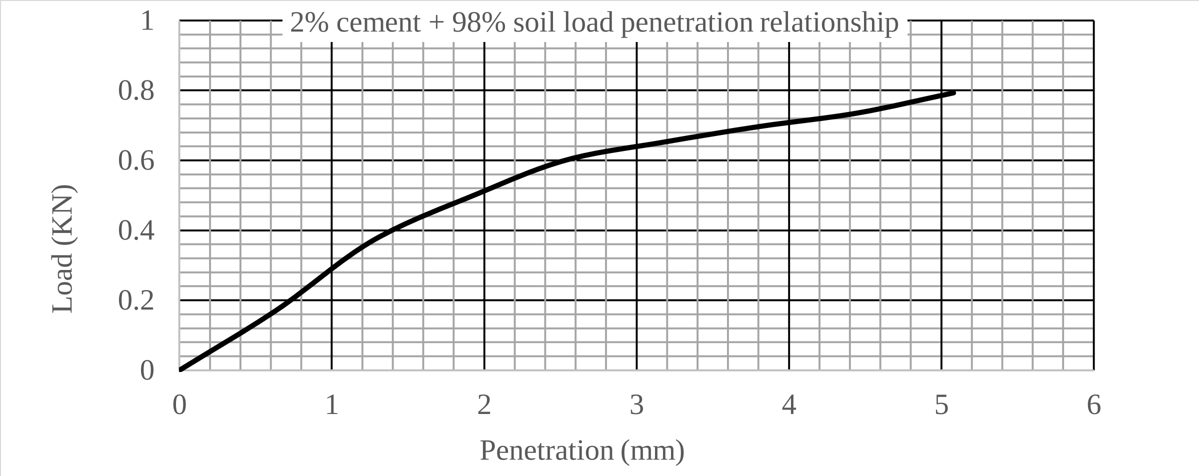


Fig 5.2.6 Loads vs. Penetration of natural Soil Sample one + 2% cement

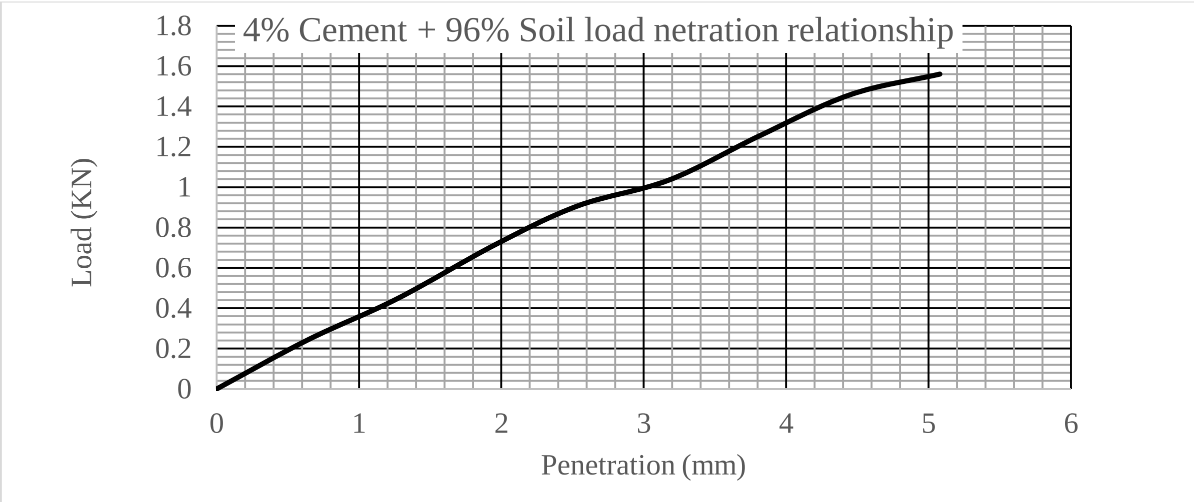


Fig 5.2.7 Loads vs. Penetration of natural Soil Sample one + 4% cement

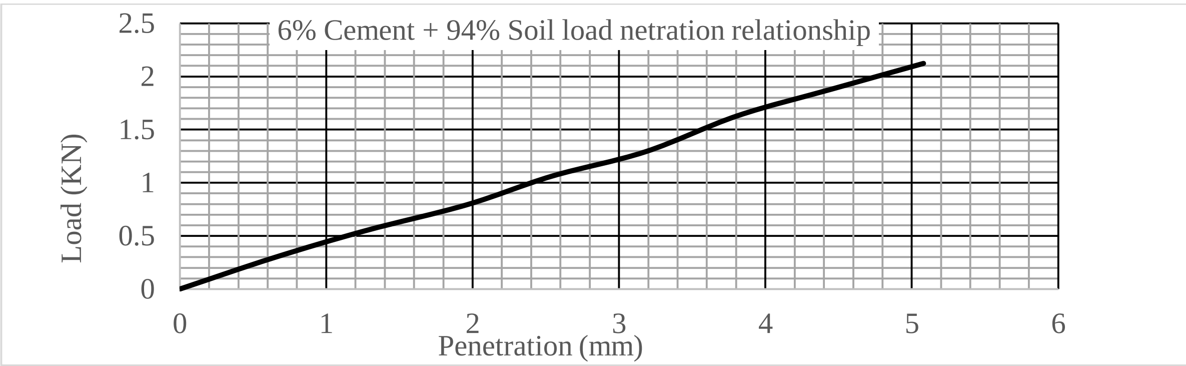


Fig 5.2.8 Loads vs. Penetration of natural Soil Sample one 6% cement

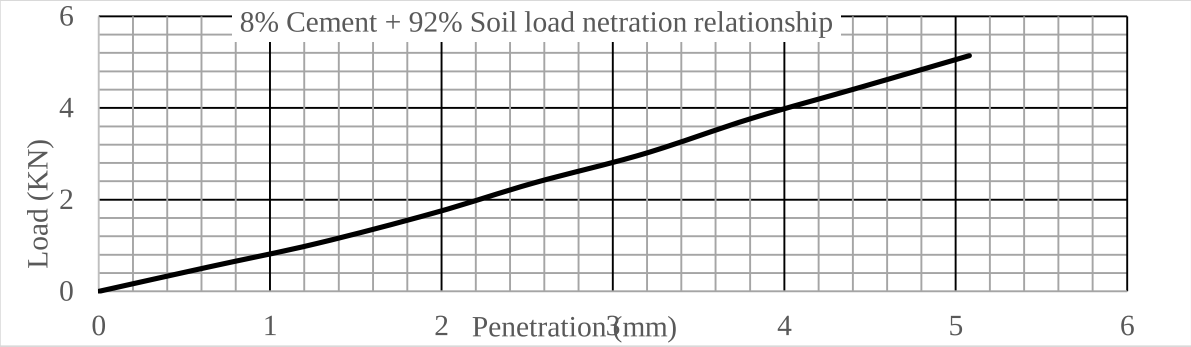


Fig 5.2.9 Loads vs. Penetration of natural Soil Sample one + 8% Lime