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ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

Stabilization of expansive soils with lime

(A Case Study on the Adura-Burbey DS6 Road Segment)

A Thesis submitted to

The School of Civil and Environmental Engineering

In partial fulfillment of

The requirements for degree of

Master of Science in Civil Engineering (Road & Transport Engineering)

By
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Advisor

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M.Sc. Thesis on

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Approved by board of examiners

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Chairman	Signature	Date

DECLARATION

I certify that this research work titled "Stablization of expansive soils with lime (A Case Study on the Adura-Burbey DS6 Road Segment)" is my own work. The work has not been presented elsewhere for assessment and award of any degree or diploma. Where a material has been used from other sources, it has been properly acknowledged/ referred.

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Name	Signature	Date

ABSTRACT

Expansive soils occurring in arid and semi-arid climate regions of the world cause serious problems on civil engineering structures. Such soils swell when given an access to water and shrink when they dry out. Several attempts are being made to control the swell-shrink behavior of these soils.

Soil stabilization using chemical admixtures is the oldest and most widespread method of ground improvement. In this study, hydrated lime, were used for stabilization of expansive soils.

Hydrated lime is one of the chemical stabilizers used in expansive soils. Hydrated lime of percent varying from 2 to 12 percent (by dry weight of the soil) is used to investigate the physical and chemical properties of the expansive soils.

The following tests are conducted to evaluate the properties of the expansive soils before and after the addition of lime.

- Atterberg limits
- Shrinkage limits
- Linear shrinkage
- Specific gravity
- Free swell
- Moisture density relation ships
- California bearing ratio (Soaked CBR) CBR Swell
- Unconfined Compressive strength(immediate and 7 days cured)

The liquid limit has decreased, with the increase of plastic limit. As a result of this the plasticity index of the stabilized soil has improved to the required level. The shrinkage limit, specific gravity, CBR and Unconfined compressive strength of the stabilized sample have increased with the addition of lime. Swelling percentage (CBR Swell) decreased with increasing stabilizer percentage.

The lime content of which the CBR values improved for the use of improved subgrade is found out to be 12% by dry weight of the soil. But further investigations are needed for its cost and other stabilization techniques and chemical stabilizer.

ACKNOWLEDGEMENT

I would like to express my sincere gratitude to Ethiopian roads authority for creating this opportunity to have further studies in road and transport stream. I would also like to thank Doctor Bikila Teklu for all his assistance and understanding throughout my post graduate career in this faculty

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ABBREVIATIONS

SG	Specific Gravity
CBR	California bearing ratio
SL	Shrinkage Limit
LS	Linear Shrinkage
UCS	Unconfined Compressive strength
PL	Plastic Limit
LL	Liquid limit
PI	Plasticity index
OMC	Optimum moisture content
KPA	Kilo paskal
cc	cm^3
Cao	Calcium oxide

1. INTRODUCTION

1.1. Background

Expansive soils are generally found in the Highlands and low lands of the Ethiopia. These soils undergo volumetric changes upon wetting and drying, thereby causing ground heave and settlement problems. This characteristic causes considerable construction defects if not adequately taken care of.

Expansive soils are a worldwide problem that has several challenges for civil engineers. Such soils swell when given an access to water and shrink when they dry out. The most common and economical method for stabilizing these soils is using mechanical stabilization that prevent volume changes.

The presence of montmorillonite clay mineral in expansive soils imparts them high swell–shrink potentials. Low rainfall has hindered the weathering of the active montmorillonite mineral into low active clay types such as illite and kaolinite. Further, the rainfall has not been sufficient to leach the clay particles far enough so that the overburden pressure can control the swell (Amer Ali-Rawas and matteus,2006).

Some partially saturated clayey soils are very sensitive to variations in water content and show excessive volume changes. Such soils, when they increase in volume because of an increase in their water contents, are classified as expansive soils (Brojan Das,2010).

Large areas of our country are covered with expansive soils such as dark and light grey clay soils. These clays have caused constant difficulties in road construction and are common occurrences in Ethiopia.

Rather than their low bearing strength expansive soils at equilibrium exhibits constant Strength. Expansive soils shrink and crack when they dry out and swell when they get wet. The cracks allow water to penetrate deep into the soil, hence causing considerable expansion and permanent deformation of the pavement structure. Furthermore, these volume changes may produce lateral displacements (“creep”) of the expansive clay soils.

The term ‘stabilization’ is the process whereby the natural strength and durability of a soil or granular material is increased by the addition of a stabilizing agent. In addition, it may provide a greater resistance to the ingress of water. There are many types of stabilizer that can be used, each with their own advantages and disadvantages.

Chemical stabilization is one of the methods of soil stabilization. It has been used in many fields in the world to improve the soil characteristics such as, bearing capacity, plasticity, workability and stiffness (Muni Budhu, 2007).

1.2. Problem Statement

Problem of expansive soils has appeared as cracking and break-up of pavements, railways, highway embankments, roadways, building foundations, slab-on-grade members and, channel and reservoir linings, irrigation systems, water lines, sewer lines.

There are a number of routes with expansive soil sections in my little experience of road constructions. One of the routes is the sections of the route in Gedo-Nekempt route. In this routes the constructed asphalt layers has cracked and some settlements are observed. These soils were not properly mitigated to control the volume changes.

Particular problems associated with road construction over expansive soils are;

- The seasonal volumetric change
- Low bearing strength
- Shrinkage and Crack
- Swell and
- Expansions

To reduce the impact of black cotton soils, improvement of their engineering properties is required. Chemical stabilizers are commonly used to improve the performance of soils with high plasticity, poor workability, and low strength and stiffness. To achieve effective soil stabilization, special attention needs to be given to proper type and concentration of the stabilizer. Besides, the effectiveness and efficiency of the stabilizer in terms of strength and durability improvement should be stated and specified.

In the state of current practice, even though ASTM and American Association of State Highway and Transport Officials (AASHTO) are excellent starting points for the selection of the stabilizers, they only have official sets of target strengths for lime, Cement and lime-fly ash specimens. Therefore, this research is proposed to evaluate the index properties, compaction and strength of the black cotton soils and their behavior after stabilization.

1.3. Research Objective

The general objective of this thesis is to study the potential benefits obtained by the use of chemical stabilization to render a typical expansive clay soil intended for road sub grade construction in Ethiopia, by increasing its bearing capacity and decreasing its plasticity and Controlling of volume changes.

The main objectives of this thesis are

- ✓ To improve the performance of a material by increasing its strength, stiffness and durability. The performance should be at least equal to, if not better than that of an acceptable to good natural material.
- ✓ To evaluate the engineering properties of untreated and treated specimens.
- ✓ To determine optimum amount of Stabilizing Agent (Lime) needed to attain the required properties of black cotton soils that can be used as a subgrade material.

1.4. Organization of the Thesis

The report contains six Chapters with five appendices. After introducing the objective of this research in chapter 1, Chapter 2 comprises of a comprehensive literature review regarding the details of the subject. Here, the definitions and mechanisms of stabilization of expansive soils have been thoroughly discussed. The type of tests to be conducted is elaborated. In Chapter 3, the general methodology and test procedures followed during the process of this research have been presented. Chapter 4 includes the laboratory tests and results. In this chapter, the different laboratory test results have been thoroughly conducted and the summary of the results tabulated with the details of the laboratory results attached in appendices. Chapter 5 includes the findings and discussions of the laboratory results. Finally, the conclusions of the findings and recommendations are discussed in chapter 6.

2. LITERATURE REVIEW

2.1. Introduction

The problem of expansive soils was not recognized by soil engineers until the late 1930. Prior to 1920, infrastructures were believed to move depending on their respective self-weight. The light weighted structures show relatively small cracks. Damages were noticed with different stages without the recognition of expansive soils. As a consequence of the structural damage, the potential problems associated with expansive soils have been recognized and preventive measures are being incorporated into new designs and Construction works (Amer Ali-Rawas and matteus, 2006).

Expansive soils Expands when they get wet and cracks when they dry. The swell and volumetric change of expansive soils has increased the interest of Engineers in the area. Expansive clays are different in that near to surface; clay often varies in density and moisture conditions from the wet season to the dry season. For example, near-or at-surface clays often dry out during periods of drought but then expand during the rainy season or when they get wet by irrigation water or water from leaky pipes (Robert W.Day, 1999).

2.2. Source of Expansive Soils

The parent materials for expansive soils can be classified in two groups. The first group comprises of the basic igneous rocks. This group comprises of minerals named Feldspar and Pyroxene. The decomposition of these minerals forms an important mineral called smectite (montmorillonite) and other secondary minerals (Chen, 1998).

The formation of morocntmorillonite was probably the weathering and erosion in the highlands and carried by streams to the coastal plains. And volcanic eruptions sending up clouds of ash felt on the plains and the seas with the ashes to be altered to morocntmorillonite (Chen, 1998).

The Presence of montmorillonite clay in these soils imparts them high swell–shrink potentials. Low rainfall has hindered the weathering of the active Montmorillonite mineral into low active clay types such as Illite and kaolinite. Further, the rainfall has not been sufficient to leach the clay particles far enough so that the overburden pressure can control the swell (Amer Ali-Rawas and matteus, 2006).

The second group comprises of sedimentary rocks. The rock comprises of smectite (montmorillonite) as a constituent and breaks down physically to form expansive soils. Smectite (montmorillonite) is one of the main sources of clay materials that forms expansive soils (Chen, 1998).

2.3. Classifications and identifications of expansive soils

Most of the national codes of practice do not give characterization and classification of expansive soils. A simple user-friendly approach based on the free swell ratio, defined as the ratio of the sediment volume of soil in distilled water to that in carbon tetra-chloride or kerosene, is formulated considering the compatibility of the results with Odometer free swell tests and the soil clay mineralogy. Statistical illustrations are provided which clearly indicate the assessment of soil expensiveness based on index properties is an overestimation. There is a consistency in the classifications based on odometer test results and the proposed approach (Chen, 1998).

There are a number of methods for identifying expansive soils. The techniques of identification and classification of expansive soils can be divided in to three parts namely mineralogical identification, indirect methods and direct methods

2.3.1. Mineralogical identification

Mineralogical identification is important for exploring the basic properties of clays; but it is impractical and uneconomical for practicing engineers. The various methods are important in research laboratory for exploring the basic properties of expansive soils. (5)

The mineralogical composition of expansive soils has an important bearing in the swelling potentials. There are five techniques which can help in identifying the mineralogy of the expansive soils (Chen, 1998).

X-ray Diffraction

This method is used in determining the proportions of the various minerals present in a colloidal clay. It consists especially of comparing the ratios of the intensities of diffraction lines from the different mineral with the intensities of lines from the standard substances (Chen, 1998).

Differential thermal analysis

This method is well established as a technique for the control of materials which undergo characteristics changes on heating. The test by itself is not accurate in identifying the expansive soils instead it is used in conjunction with x-ray diffraction (Chen, 1998).

Dye absorption

Mineral can be identified by characteristic colors formed by dyes that are absorbed by the minerals of the soil sample. When a clay sample is pretreated with acid, the color assumed by the absorbed dye depends on the Base Exchange capacity of the various clay minerals present. The presence of the montmorillonite can be identified if the selected sample contains the mineral which is greater than about 5-10% (Chen, 1998).

Chemical analysis

This method is a valuable addition to other methods such as X-ray Diffraction. In the montmorillonite group of clay minerals, the chemical analysis can be used to determine the nature of isomorphism and to show the origin and location of the charge on the lattice (Chen, 1998).

Electron microscope Revolution

Microscopic examination of clay minerals offers the direct observation of the material. Two clays may give the same x-ray pattern and the same differential thermal curve but will show up distinct morphological characteristics under electron microscope resolution (Chen, 1998).

2.3.2. Indirect methods

The second method is called indirect methods which comprises of the index properties, PVC Method and activity methods. 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).

Single index Method

These methods are one of the tools in evaluating the swelling potential of expansive soils. Such tests are easy to perform and should be included as routine tests in the investigation of the proposed sites. Some of the tests included in this category are (Chen, 1998):

- Atterberg limits tests
- Linear shrinkage tests
- Free swell tests and
- Soil Classifications

Atterberg limits tests

Here measurements of the plasticity index and liquid limit are useful indices for the identification of the swelling of expansive soils. There are a number of correlations to determine the swelling potential of the expansive soils. For example Woodland and Lundgren have established the following simplified relationships $S=60K\,(PI)^{2.44}$

Where,
S=Swell Potential

K=3.6*10^$^{-5}$ and is Constant (Chen, 1998)

The limitation to this equation is that it is only applicable for soils with clay content between 8 and 65%. And the computed value is accurate to within 33% of the laboratory result determined swell potential (Chen, 1998).The Atterberg limits test results and degree of expansion on expansive soils are expressed as follows

Table 2-1Atterberg limit results and Degree of Expansion (Chen 1975)

Swelling potential	**Plasticity Index**	**Liquid limit**
Low	0-15	<30
Medium	10-35	30-40
High	20-55	40-60
Very High	55 and above	>60

Shrinkage Limit

The Shrinkage limit is the water content at which the soil changes from solid state to a semi solid state without further change in volume (11). Measurements of Shrinkage limits are one of the tools in the identifications of expansive soils. Wiesman etal used index properties to identify expansive soils and is tabulated in table 2-2 below (Chen, 1998)

Table 2-2Atterberg limit results and Degree of Expansion (Wiseman etal 1985)

Index tests	Usually non problematic	Almost always problematic
Plasticity index	<20	>32
Shrinkage limit	>13	<10
Free swell	<50	>100

Linear Shrinkage

The swell potential is presumed to be related to the opposite property of linear shrinkage measured in a very simple test. In theory the shrinkage characteristics of the clay should be a consistent and reliable index to the swelling potential.

In 1955 Altmeyer has suggested a guide for the determination of potential expansiveness for various values of shrinkage limits. But the recent researchs has failed to show the conclusive evidence of the correlation between the swelling potential and Shrinkage limit. The correlation is presented below as (Chen, 1998).

Table 2-3 Shrinkage limits and Degree of Expansion (Altmeyer 1955)

Shrinkage limit (%)	**Linear Shrinkage (%)**	**Degree of Expansion**
Less than 10	Greater than 8	Critical
10-12	5-8	Marginal
Greater than 12	0-5	Non Critical

Free swell

A free swell test consists of placing a known volume of dry soil in water and noting the swelled volume after the material settles without any surcharge, to the bottom of a graduated cylinder. The difference between the final and initial volume, expressed as a percentage of the initial volume is a free swell volume.

The test is performed by pouring 10cc of dry soil passing a sieve of size 0.425mm (NO.40) in to a 100cc graduated jar filled with water, notifying the swelled volume of the soil after it comes to rest (Nebro D.,2002).

Soil Classifications

Soil is a broad term used in engineering applications which includes all deposits of loose material on the earth’s crust that are created by weathering and erosion of underlying rocks. Although weathering occurs on a geologic scale, the process is continuous and keeps the soil in constant transition. The physical, chemical, and biological processes that form soils vary widely with time, location and environmental conditions and result in a wide range of soil properties. Physical weathering occurs due to temperature changes, erosion, alternate freezing and thawing and due to plant and animal activities causing disintegration of underlying rock strata whereas chemical weathering decomposes rock minerals by oxidation, reduction, hydrolysis, and carbonation. These weathering processes, individually or in combination, can create residual (Dallas N. Little, 1995).

The AASHTO (M 145) soil classification system differentiates soils, first based on particle size and secondly based on Atterburg limits. If 35 percent or more of the mass of the soil is smaller than 75 μm in diameter, then the soil is considered either a silt or clay and if less than 35 percent of particles are smaller than 75 micron sieve, then the soil is considered to be coarse-grained, either a sand or gravel.

For stabilization purposes, soils can be classified into subgrade and base materials based on fractions passing No. 200 sieve. If 25 percent or more passes through the no. 200 sieve the soil can be considered as a subgrade, and if not, they may be classified as a base material. However, more than simple gradation impacts the definition of a subgrade or base. In order to be termed a base material, the material in question must also be targeted for use as a base layer from a structural perspective. On the other hand, an in situ coarse-grained soil with less than 25 percent fines, may be, by definition a native subgrade even though it may achieve the required classification of a base. For stabilization purposes, the soils may be differentiated into subgrade (soil) stabilization and base stabilization (coarse-grained) on the basis on the fine content index (Dallas N. Little, 1995).

2.3.3. Direct measurements

The third and the last methods are called the direct measurements. These methods are the most useful data for practicing Engineers. The tests are very simple and the costs are very small. A number of tests should be ordered before drawing any conclusions to avoid errors. The direct measurements are the most satisfactory and convenient methods to determine the swelling potential and swelling pressure of expansive clay (Chen, 1998).

Direct measurements of expansive soils can be achieved by the use of the conventional one dimensional consolidometer. The consolidometer can be platform type,Scale type or other arrangement. The soil sample is enclosed between two porous plates and confined in a metal lying. The soil sample can be flooded both from the bottom and from the top (Chen, 1998).

2.4. Physical properties of expansive soils

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. The most important physical properties of expansive soils are:

- Moisture content
- Dry density
- Index properties and
- Fatigue of Swelling (Chen, 1998).

2.4.1. Moisture content

If the moisture content of the clay remains unchanged, there will be no volume change irrespective of the high swelling potential. 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 (Chen, 1998).

The initial moisture content of the expansive soils controls the amount of swelling. The relationship between the initial moisture content and the capability of swelling has been studied by Holtz Seed and Many Others (Chen, 1998).

Very dry clays with natural moisture content below 15 percent usually indicate danger. Such expansive soils easily absorb moisture as high as 35 percent with a 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 condition and up on subsequent wetting will again exhibit swelling potential (Chen, 1998).

2.4.2. Dry Density

The dry density of the clay is another index property of the expansive soils. Soils with dry density in excess of 110pcf generally exhibit high swelling potential. The dry density of the clays is also reflected by standard penetration resistance test results. Clays with penetration resistance in excess of 15 usually possess some swelling potential (Chen, 1998)

2.4.3. Index Properties

The simplified classification of expansive properties can be conventionally used by Engineers as a guide for the choice of structures on expansive soils. Some of the index properties to be identified and used are Soil Classification, Liquid Limit, Standard penetrations and the likes (Chen, 1998).

2.4.4. Fatigue of Swelling

A clay sample is subjected to full swelling in the consolidometer, allowed to desiccate to its initial moisture content and is then saturated again. These steps can be repeated for a number of cycles and observed that the soil has shown a sign of fatigue after each cycle of drying and wetting. 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 the swelling can answer the situation (Chen, 1998).

2.5. Clay minerals

The term clay can refer both to a size and to a class of minerals. As a Size term, it refers to all constituents of a soil smaller than a particular size, usually 0.002 mm in engineering classifications. As a mineral term, it refers to Specific clay minerals that are distinguished by small particle size, a net Electrical charge, plastic when mixed with water and (d) high weathering Resistance (Budhu Muni,2007).

Minerals are crystalline and make up the solids constituent of a soil. The mineral particles of fine grained soils are platy. Minerals are classified according to chemical composition and structure. Most minerals of interest to geotechnical engineers are composed of oxygen and silicon. Silicates are a group of minerals with a structural unit called the Silica Tetrahedral. A central silica cation (positively charged ion) is surrounded by four oxygen anions (negatively charged ions), one at each corner of the tetrahedron. Silicate minerals are formed by addition of cations and interaction of tetrahedrons. Silica tetrahedrons combine to form sheets, called silicate sheets, which are thin layers of silica tetrahedrons in which three oxygen ions are shared between adjacent tetrahedrons. Silicate sheets may contain other structural units such as alumina sheets. Alumina sheets are formed by combination of alumina minerals, which consists of an aluminum ion surrounded by six oxygen or hydroxyl atoms in an octahedral (Fransiscus and Wayne, 1994).

Most soil classification system arbitrarily defines clay particles as having an effective diameter of two microns (0.002mm) or less. Particle size alone does not guarantee the classification of fine grained materials. The most important grain property for fine grained soils is the mineralogical composition (Chen, 1998).

2.5.1. Clay Structures

Clay minerals are predominantly silicates of aluminum and/or iron and magnesium. Some of them also contain alkalies and/or alkaline earths as essential components. These minerals are predominantly crystalline in such a way that the atoms composing them are arranged in definite geometric patterns. Most of the clay minerals have sheet or layered structures. A few have elongated tubular or fibrous structures (NG Pulling, 2005).

The basic idealized crystalline structural unit of a clay mineral is composed of a silica tetrahedron block and an aluminum octahedron block. Aluminum octahedron block may have Aluminum (Al^{3+}) or magnesium (Mg^{2+}). If only aluminum is present, it is called gibbsite [$Al_2(OH)_6$]; if only magnesium is present, it is called brucite [$Mg_3(OH)_6$].

Various clay minerals are formed as these sheets stack on top of each other with different ions bonding them together. A silica tetrahedron and a silica sheet, also an octahedron and an octahedron sheet are presented in figure 2.1 and Figure 2.2, respectively. Also, these figures consist of schematic representations of silica and octahedron sheets (Muni Budhu, 2007).

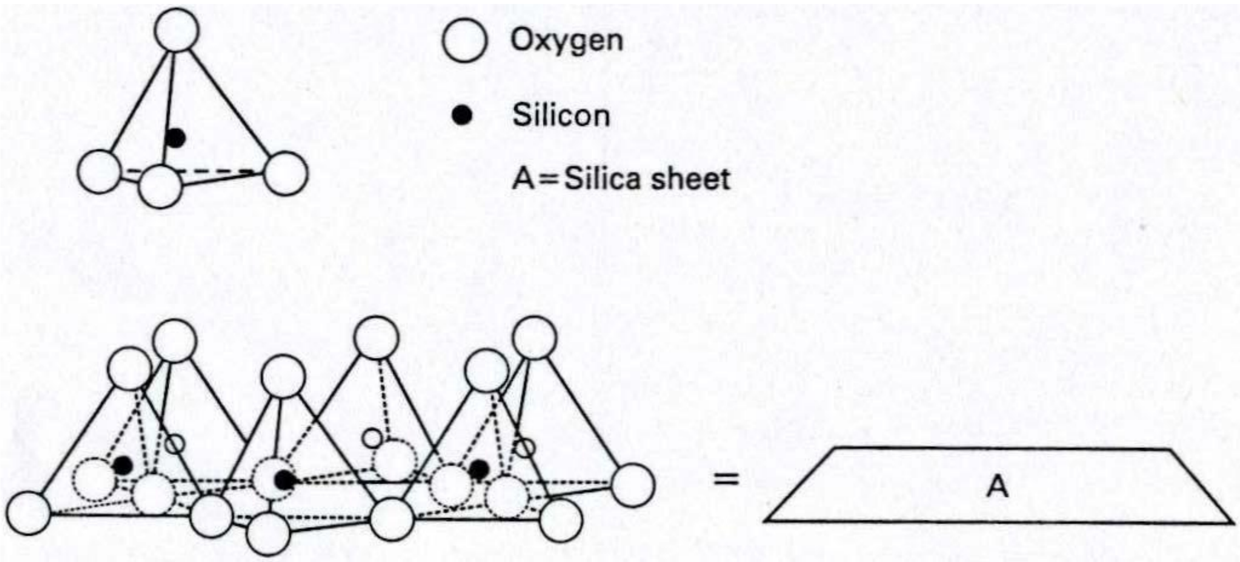


Figure 2-1 a Silica Tetrahedron and a Silica Sheet (Oweis and Khera, 1998)

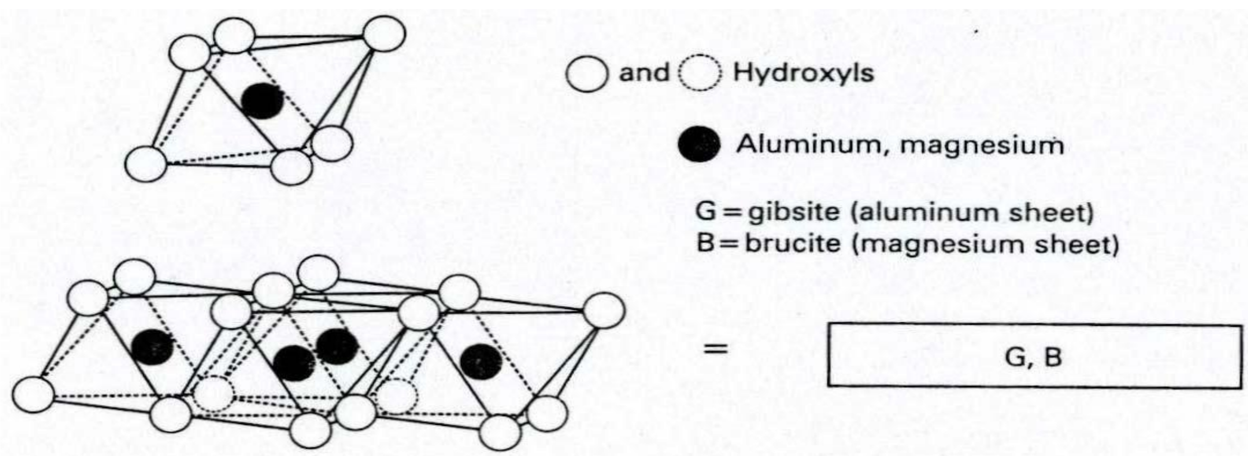


Figure 2-2 an octahedron and an octahedron Sheet (Oweis and Khera, 1998)

There are three structural groups of clay minerals that are important for engineering purposes. These three groups are

- Kaolinite group usually non expansive
- Mika like group- includes illites and vermiculites, which can be expansive but generally do not pose significant problems.

- Smectite group - includes montmorillonites, which are highly expansive and are the most troublesome clay minerals

Kaolinite Group

The kaolin minerals are a group of clay minerals consisting of hydrous aluminum silicates. A common kaolin mineral is kaolinite. Kaolite 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. The bonding combination of hydrogen and van der Waals forces results in considerable strength and stability with little tendency for inter-layers to take on water and swell. The bonding is sufficiently strong that there is no interlayer swelling in the presence of water. Kaolinite is the least active of the clay minerals. Kaolinite can be produced by weathering of certain of the more active clay minerals as well as being directly formed as a byproduct of rock weathering. Kaolinite tends to be found in regions of heavier rainfall.

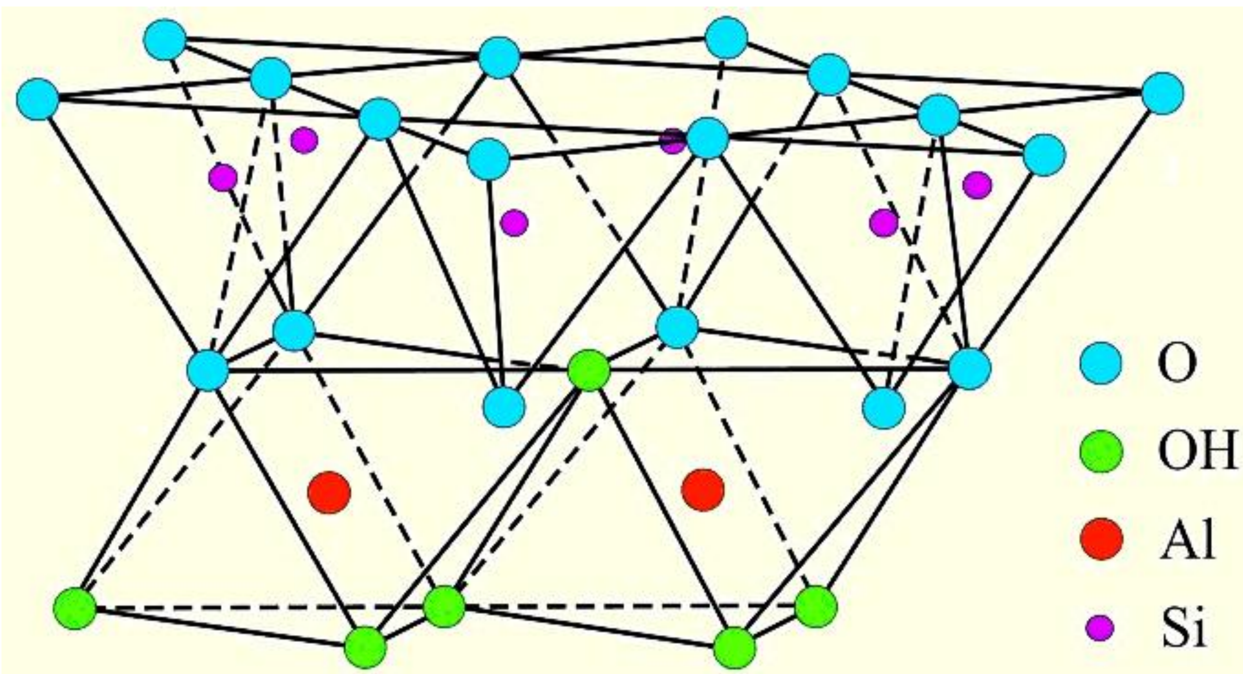


Figure 2-3 Diagrammatic Skectch of the Kaolonite (Onur Baser, 2009)

Mika like Group

Illite has a basic structure consisting of a sheet of alumina octahedrons between and combined with two sheets of silica tetrahedrons. This clay mineral has a structure similar to montmorillonite, but the layers are more strongly bonded together. In terms of cation exchange capacity,inability to absorb and retain water and in physical characterstics illite is intermediate in activity between clays of kaolin and montmorillonite groups. In the octahedral sheet there is partial substitution of aluminum by magnesium and iron. And in the tetrahedral sheet there is partial substitution of silicon by aluminum. The combined sheets are linked together by fairly weak bonding due to (non - exchangeable) potassium ions held between them (Muni Budhu, 2007).

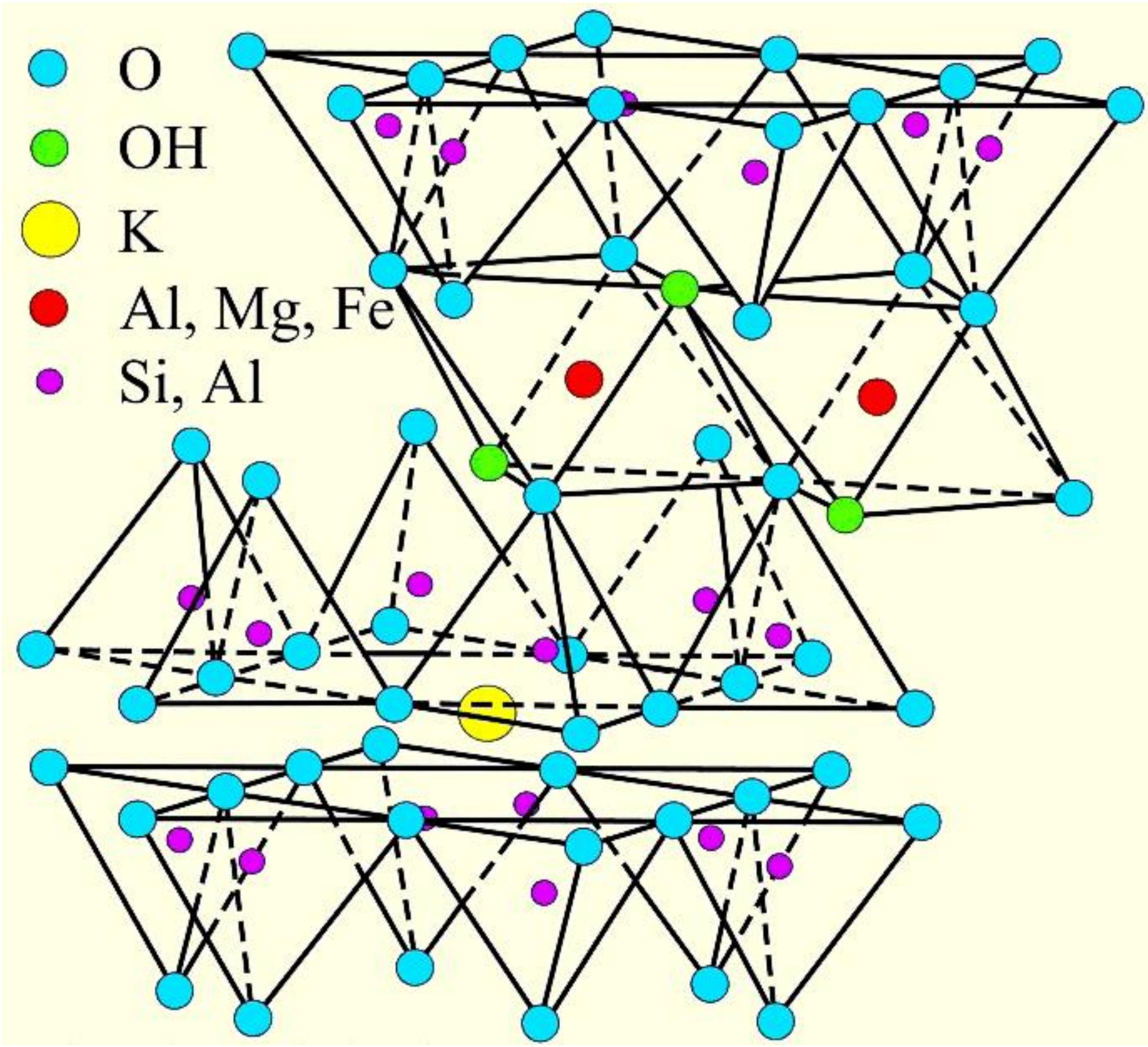


Figure 2-4 Diagrammatic Skectch of the Illite (Onur, Baser, 2009)

Smectite Group

Mont morillonite are a group of clay minerals that are characterized by weekly bonded layers. It is formed from weathering of volcanic ash under poor drainage conditions or in marine waters. Each layers consists of two silica sheets with an aluminium(gibbsite) sheet in the middle. Water and exchangeable cations can enter and separate the layers, creating a very small crystal that has a strong attraction to water. Montmorillnite has the highest activity and it can have the highest water content, greatest compressibility and lowest shear strength of all clay minerals (Muni Budhu, 2007)

The space between the combined sheets is occupied by water molecules and exchangeable cations. There is a very weak bond between the combined sheets due to these ions. Considerable swelling of montmorillonite being can occur due to additional water absorbed between the combined sheets (Muni Budhu, 2007).

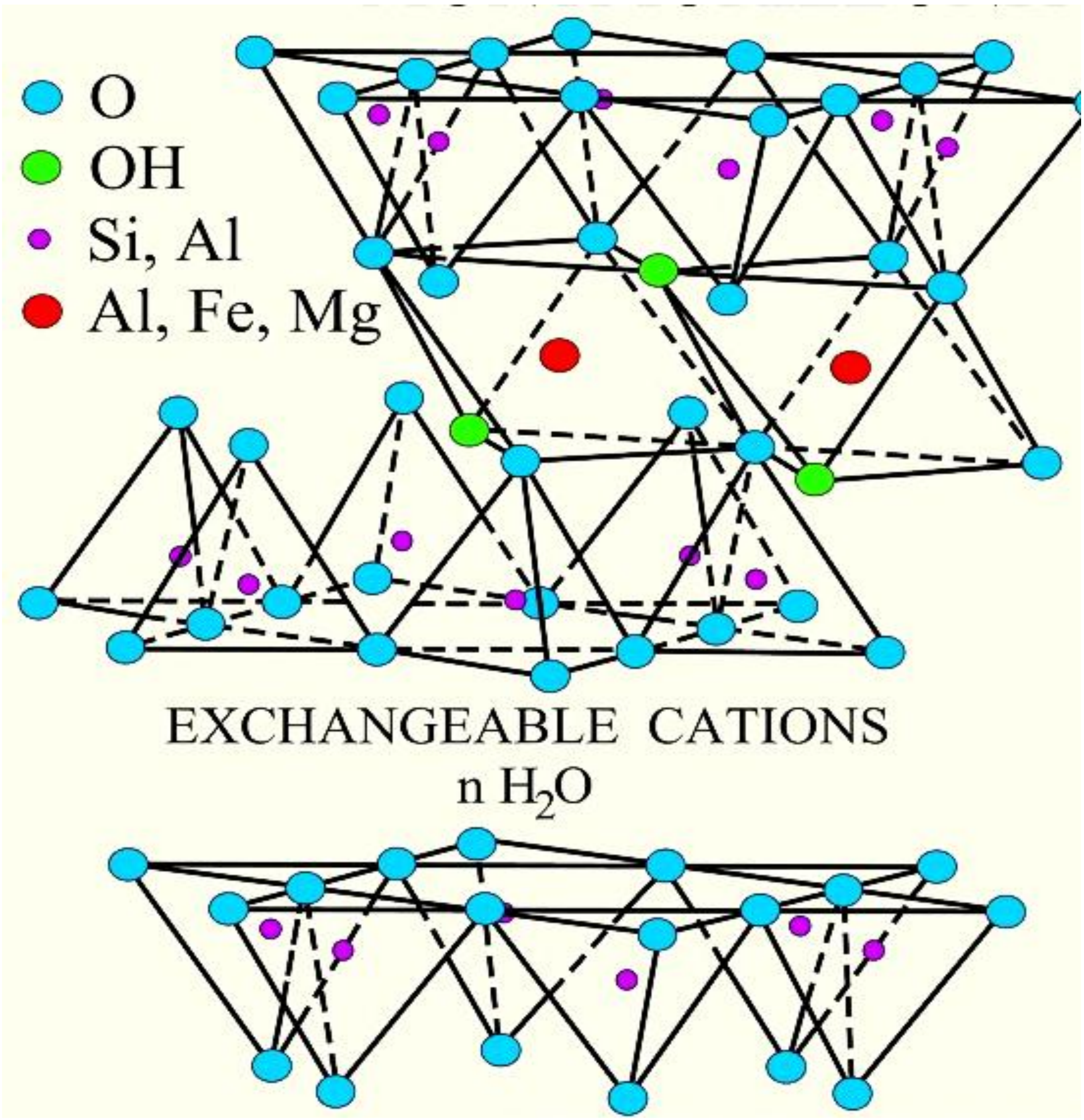
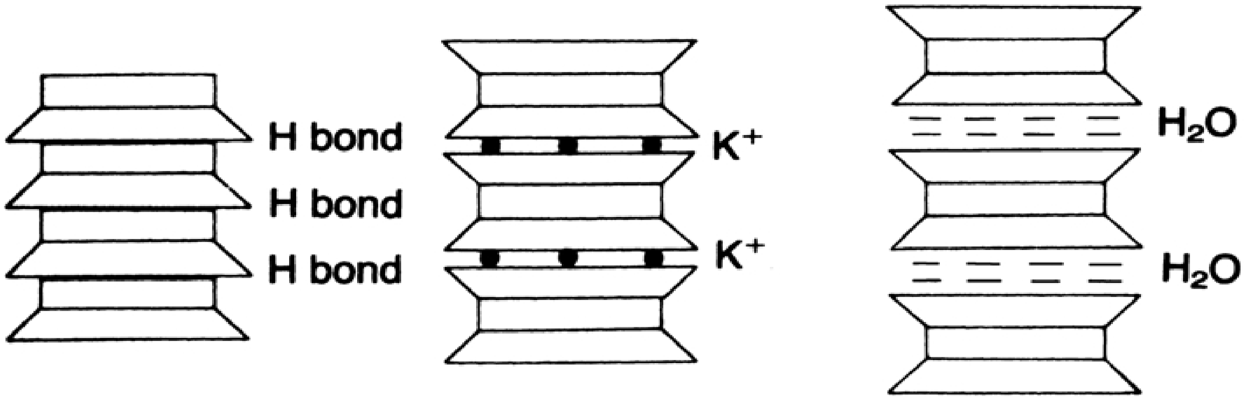


Figure 2-5 Diagrammatic Skectch of the Montmorillonite (Onur,Baser, 2009)



Kaolinite **Illite** **Montmorillonite**

Figure 2-6 Schematic Representations of Clay Minerals (Craig, 1997)

Black Cotton soils

Black cotton soils are inorganic clays of medium to high compressibility. They are characterized by high shrinkage and swelling properties. Because of their high swelling and shrinkage characteristics, black cotton soils have been a challenge to the highway engineers. The Black cotton soil is very hard when dry, but loses its strength completely when in wet condition (NG.Pulling, 2005).

Black cotton soils owe their specific properties to the presence of swelling clay minerals, mainly montmorillonite. As a result of the wetting and drying, massive expansion and contraction of the clay minerals takes place. Contraction leads to the formation of the wide and deep cracks. These cracks can be wide enough to make the terrain treacherous for animals (NG.Pulling, 2005).

The cracks close after rain when the clay minerals swell. During expansion of the clay minerals high pressures are developed within these soils, causing a characteristic soil structure with wedge shaped aggregates in the surface soil and 'planar' soil blocks in the subsoil. The slippage of one soil block over the other leads to the formation of typical polished surfaces known as "slickensides' on the blocks'

Black cotton soils are difficult to work, they are of very hard consistence when dry and very plastic and sticky when wet. Therefore the workability of the soil is often limited to very short periods of medium (optimal) water status.

Black cotton soils are imperfectly to poorly drained, leaching of soluble weathering products is limited, the contents of available calcium and magnesium are high and the pH is usually above 7. Once they have reached their field capacity, practically no water movement occurs; this is due to the very low hydraulic conductivity of the soil. Flooding leading to crop damage can be a major problem in areas with higher rainfall. Surface water may be drained by open drains, but 'mole' drainage is virtually impossible.

Often, black cotton soils exhibit variations in properties, particularly strength and volume change properties following variation in their in service moisture contents. These properties limit their performance as a support element for pavements. Typical problems include shrink-swell, settlement, collapse, erosion or simply insufficient strength (Chen, 1998).

Particular problems associated with road construction over black cotton soils are commonly the seasonal volumetric changes in these soils. Typical distress is from seasonal wetting and drying whereby soils at the edge of the road wet up and dry out at a different rate than those under a bituminous surfacing. This mechanism causes permanent deformation (rutting) over the cross section of the road and associated crack developments, first occurring in the shoulder area, subsequently developing in the carriageway (Nebro D.,2002)

2.6. Soil stabilization

Stabilization is the process of blending and mixing materials with a soil to improve certain properties of the soil. The process may include the blending of soils to achieve a desired gradation or the mixing of commercially available additives that may alter the gradation, texture or plasticity, or act as a binder for cementation of the soil *(Terzaghi etal, 1984).*

In theory the swelling potential of Expansive Clay can be minimized or completely eliminated by different remedial measures. Some of the remedial measures are listed below.

I. Prewetting

II. Remolding and Compaction

III. Surcharge Loading

IV. Removal and Replacement

V. Chemical Admixtures

2.6.1. Prewetting

The prewetting theory is based on the assumption that if the soil is allowed to swell by wetting prior to construction and if the high soil moisture content is maintained, the soil volume will remain constant, achieving a no-heave state and therefore structural damage will not occur.Moreover, exposing the soil for further moisture will not expose for absorption of additional water. But the moisture content should be maintained at constant level (Chen, 1998).

2.6.2. Remolding and Compaction

The method of compaction is generally limited by available equipments, method of compaction and by varying the molding water content for a given density.

The amount of swelling that occurs when a structural fill is exposed to additional moisture depends up on the following (Chen, 1998).

- The compacted dry density
- The moisture content
- Method of compaction and comp active effort
- Surcharge load

Highly expansive soils can be compacted to some minimum density rather than to a maximum density. It can be seen that expansive clays expand very little when compacted at low densities and High moisture but greatly expand when compacted at high densities and low moistures (Chen, 1998).

2.6.3. Surcharge Loading

Swell can be prevented if expansive clays can be loaded with a surcharge large enough to counteract the expected swell pressures. This method is most effective when swell pressures are low and some heaving can be tolerated in the construction project, such as in secondary highway system. However, many soils exhibit swell pressure too high to be controlled by normal surcharge loads alone.

2.6.4. Removal and Replacement

Removal of expansive soils and replacement with non-expansive soils is one method to provide stable foundation material. In some cases the expansive strata may be entirely removed. Generally, the expansive layer extends to a depth too great to economically allow complete removal and replacement. One mechanism by which the removal and replacement method mitigates expansive potential is by the control of the moisture content in the underlying clay layer (US army, 1984).

Removal and replacement of the expansive soils with selected materials is one of the most adapted practices in our country. In most road construction works a replacement depth of 60cm is specified in almost all projects to replace the expansive material with selected borrow or suitable materials from the construction areas satisfying the required standards and specifications.

2.6.5. Chemical Admixtures

One method to improve expansive soils is chemical stabilization. Chemical stabilization includes the mixing or injecting of chemical substances into the soil. Portland cement, lime, asphalt, calcium chloride, sodium chloride, and paper mill wastes are common chemical stabilizing agents. The effectiveness of these additives depends on the soil conditions, stabilizer properties, and type of construction (i.e., houses, roads, etc.). The selection of a particular additive depends on its costs, benefits, availability, and practicality of its application (US army, 1994).

It involves mixing or injecting the soil with chemically active compounds such as Portland cement, lime, fly ash, calcium or sodium chloride or with visco-elastic materials such as bitumen. Chemical stabilizers can be broadly divided in to three groups: Traditional stabilizers such as hydrated lime, Portland cement and Fly ash; Non-traditional stabilizers comprised of sulfonated oils, ammonium chloride, enzymes, polymers, and potassium Compounds; and By-product stabilizers which include cement kiln dust, lime kiln dust etc. Among these, the most widely used chemical additives are lime, Portland cement and fly ash. Although stabilization with fly ash may be more economical when compared to the other two, the composition of fly ash can be highly variable (US army, 1994).

Additives

There are two basic types of additives used during chemical stabilization. These additives are named as mechanical and chemical additives. Mechanical additives includes such as mixing of cement with a soil which alters the soil property by adding a quantity of material that has the engineering characteristics to upgrade the load bearing capacity of the existing soil. Chemical additives includes such as lime alters the soil chemically thereby improving the load bearing capacity of the soil (US army, 1994).

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).

Quick/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).

Fly Ash: a chemical additive consisting mainly of silicon and aluminum compounds. It can be mixed with lime or calcium in order to get the required quality (US army, 1994).

Calcium Chloride: It is a chemical additive that has the ability to absorb moisture from the air until it liquefies into a solution. The presence of this additive helps to lower the moisture of soils and freezing temperature of that moisture. For this reasons, calcium chloride is a proven stabilizing additive for cold-climate applications (US army, 1994).

Bitumen: is a chemical additive that occurs naturally or as a by-product of petroleum distillation. It is the black pitch used to make asphalt. Asphalt Cement, cutback asphalt, tar and asphalt emulsions are all used to achieve bituminous soil stabilization (US army, 1994).

2.7. Guidelines for stabilizer selection

Soil characteristics including mineralogy, gradation and physio-chemical properties of fine grained soils influence the soil-additive interaction. Hence stabilizer selection should be based on the effectiveness of a given stabilizer to improve the physio-chemical properties of the selected soil. The preliminary selection of the appropriate additive(s) for soil stabilization should consider:

- ✓ Soil consistency and gradation
- ✓ Soil mineralogy and composition
- ✓ Desired engineering properties
- ✓ Purpose of treatment
- ✓ Mechanisms of stabilization
- ✓ Environmental conditions and engineering economics

Soil index properties (i.e., sieve analysis, Atterberg limit testing, and moisture density testing) should be determined based on laboratory testing of field samples. Soil samples should be prepared following AASHTO T 87. The initial processing of most soils involves thorough air drying or assisted drying at a temperature not to exceed 60^{o}C. Aggregations of soil particles should be broken down into individual grains to the extent possible. A representative soil fraction should be selected for testing following AASTHO T 248. The required quantity of soil smaller than 0.425 mm (No. 40 sieve) should be used to determine the soil index properties. Liquid limit testing should be performed following AASHTO T 89 and plastic limit and plasticity index testing should be measured following AASHTO T 90 (US army, 1994).

Soil gradation is a classification of a coarse-grained soil that ranks the soil based on the different particle sizes contained in the soil. Soil gradation is an important aspect of soil mechanics and geotechnical engineering because it is an indicator of other engineering properties such as compressibility, shear strength, and hydraulic conductivity. In a design, the gradation of the in situ or on site soil often controls the design and ground water drainage of the site. A poorly graded soil will have better drainage than a well graded soil (US army, 1994).

The process for grading a soil is in accordance with either the Unified Soil Classification System or the AASHTO Soil Classification System. Gradation of a soil is determined by reading the grain size distribution curve produced from the results of laboratory tests on the soil. Gradation of a soil can also be determined by calculating the coefficient of uniformity, C_u, and the coefficient of curvature, C_c, of the soil and comparing the calculated values with published gradation limits (US army, 1994).

2.8. Lime Stabilization

2.8.1. Introductions

Lime is inorganic chemical compound which is usually known as quick lime or unslaked lime Obtained from a naturally occurring compound called limestone. Quick lime is chemically expressed as calcium oxide, a strong caustic ingredient widely used in construction industry in the preparation of mortar and plasters. Lime decreases soil density and increases soil strength.

Lime reacts with medium, moderately fine, and fine-grained soils to produce decreased plasticity, increased workability, and increased strength. Strength gain is primarily due to the chemical reactions that occur between the lime and soil particles (Little, 1995). A successful approach to identify the solubility of the components in the clay has been discovered. In order to provide durability, soils must be stabilized with sufficient lime to develop pozzolanic reactions. Limes should not be Exposed to air through proper handling methods prior to mixing, this Practice avoids premature carbonation of the lime (Fransiscus and Wayne, 1994)

Lime acts with soils in three ways. The first of the reactions is change of water film around the clay minerals. The second process by which lime changes a soil is that of coagulations or floculations of the soil particles. The third process through which lime affects soil is reaction of lime with soil ingredients to form new chemicals. The two main ingredients of soil which act with lime are aluminia and silica. This is a long term reaction which strengthens the soil lime mixture is also known as Pozzolanic action. The degree to which lime will react with soil depends on quantity of lime, soil type and period of curing for soil lime mixture.

Lime has been used for centuries for the stabilization of clays. For example, lime Was used along with alabaster (gypsum) for mortar and plaster to build the huge Limestone pyramids of ancient Egypt (Fransiscus and Wayne, 1994).

In general, all lime treated fine-grained soils exhibit decreased plasticity, improved workability and reduced volume change characteristics and increased strength of the soil. However, not all soils exhibit improved strength characteristics. It should be emphasized that the properties of soil-lime mixtures are dependent on many variables. Soil type, lime type, lime percentage and curing conditions (time, temperature, and moisture) are the most important (J.Paul Guyer, 2011).

Because of the proven success of lime stabilization in the field of highways and air-field pavements, lime stabilization is now being extended for deep in-situ treatment of clayey soils to improve their strength and reduce compressibility.

2.8.2. Types of lime

Various forms of lime have been successfully used as soil stabilizing agents for many years. However, the most commonly used products are hydrated high-calcium lime, monohydrated dolomitic lime, calcitic quicklime, and dolomitic quicklime. Hydrated lime is used most often because it is much less caustic than quicklime; however, the use of quicklime for soil stabilization has increased in recent years mainly with slurry-type applications. The design lime contents determined from the criteria presented herein are for hydrated lime. If quicklime is used, the design lime contents determined herein for hydrated lime should be reduced by 25 percent. Specifications for quicklime and hydrated lime are found in ASTM C 977 (J.Paul Guyer, 2011).

2.8.3. Physical properties of Quick lime and hydrated lime

Quicklime is white of varying degrees of intensity, depending on its chemical purity. The presence of Impurities within the lime results in a grayish or yellowish appearance. Hydrated limes reflect a similar relationship between purity and Whiteness (Dallas N.Little, 1995).

The most important physical properties of limes related to soil stabilization are specific Gravity, Bulk Density, Heat of solution and Solubility (Dallas N.Little, 1995).

Specific gravity and Bulk density: the true specific gravity and bulk density of pure calcium oxide is 3.34, however values are reported as high as 3.4 and as low as 3. (Dallas N.Little, 1995)

The Aparent specific gravity is a more meaningful property because it represents the density of the actual material as it comes from the procedure. The apparent specific gravity for quicklime varies from 1.6 to 2.8 with dolomatic quicklime averaging from 3 to 4 percent higher (Dallas N.Little, 1995)

Heat of Formation: the heat of formation is synonymous with heat of hydration and reaction. For commercial hydrates the heat of hydration for $Ca(OH)_2$ is approximately $6.396*10^7$J/Kg.k). The substancial heat of hydration is important in the production of hydrated lime or quicklime slurries . Solubility of Hydrated lime ($Ca(OH)_2$ is 1.33 gCaO/l of saturated solution at 10 degree in distilled water(Dallas N.Little, 1995).

2.8.4. Chemical properties of Quick lime and hydrated lime

Quicklime and hydrated limes are reasonably stable compounds. However quicklime is vulnerable to water; even the moisture in the air produces a destabilizing effect by air slaking. Hydrated lime is more stable since water does not cause a change in its composition. The primary factor influencing the stability of the hydrated lime is carbon dioxide which reacts with either quicklime or hydrated lime to form calcium carbonate (Dallas N.Little, 1995).

The reactivity of quicklime with water is great practical importance as this reactivity is the basis for the production of hydrated lime from quicklime. The production of hydared lime from quicklime through a slaking process at a construction site produces a very reactive product for soil stabilization (Dallas N.Little, 1995).

The PH of solution at 25^0C rise sharply with the adition of very low concentration of $Ca(OH)_2$. A small concentration of approximately 0.064g/l of hydrated lime raises the PH value from 7 to 11. The high ph of a lime water solution is of great importance in soil and aggregate stabilization. This is because a high PH or basic environment increases the ability of the lime to react with soil minerals, and produces a cementious product which can stabilize by “gluing” them together (Dallas N.Little, 1995).

The current source of supply of lime to the domestic market are Dire Dawa Cement and Lime Factory, Ethio Lime Factory of Senkele, the Caustic Soda lime unit and the Wonji Shoa Sugar Estates supplemented with small quantity of imports. The Caustic Soda and the Showa Sugar Estates lime factories mainly produce for their own use, while the remaining two are primarily producing for the market. Dire Dawa Cement and Lime Factory mainly concentrate on production of cement and allocate only part of its capacity for production of lime. In addition to the above, the Ethiopian Educational Materials Production and Distribution Enterprise (EMPEDA) using its chalk production unit, produces small quantity of masonry lime for the market. As the major function of this unit is chalk production, only small proportion is allocated to masonry lime production (Dallas N.Little, 1995).

The properties of soil-lime mixtures depend on: a) the properties of the soil, b) The properties of the lime (chemical composition and gradation), c) lime content, d) Method of mixing, e) duration of "mellowing" period (time between mixing and Placement, f) curing environment (moisture conditions, chemistry, and temperature), and g) age (US Army,1994).

Monohydrate lime was significantly higher than the strengths of the soil treated with calcitic slaked lime. Unslaked lime was more effective than hydrated lime in improving strength. At lime contents above two percent, the coarser unslaked limes were more effective because the fine limes caused a significant flocculation, resulting in a reduction in the density of the soil (US Army, 1994).

2.8.5. Benefits expected from stabilizing expansive soils with lime

Some of the expected changes in stabilizing the expansive soils are:

- Resilient Modules/Stiffness
- Fracture and Fatigue and
- Durability

Strength: The most obvious improvement in a lime-reactive soil or aggregate is strength gain over time. The various strength parameters impacted by the pozzolanic reactions that occur include unconfined compressive strength, tensile strength, flexural strength, and CBR. The stabilized soil should exhibit the following requirement to check whether the strength gain of the material is within the required range or not which is tabulated in table 2-4 below. Note the strength requirements are 7 days for cement and 28 days for lime. Strength is gained through lime stabilization **(**Terzagi etal, 1996).

Table 2-4Pavement layers and minimum unconfined compressive

Id	Stabilized soil layer	Minimum unconfined Compressive Strength(Kpa)	
		Flexible pavement	Rigid pavement
1	Base coarse	5175	3450
2	Subbase, select material or Subgrade	1725	1380

The addition of lime on expansive soils greatly changes the stress - strain relationship of the material. Dallas little has characterized the material properties of lime-stabilized soils as related to their impact on overall pavement performance can be divided into four categories. These are Strength, Resilient Modules/Stiffness, fracture and Durability *(Dallas N. Little, 2009).*

Resilient modulus/stiffness- Concurrent with the strengthening of a soil brought about by pozzolanic reactions, are changes in the stress–strain relationship of the material. Lime-stabilized soils fail at much higher deviator stresses than their non-stabilized counterparts, and at a much lower strain (typically about 1 percent strain for the stabilized mixture versus about 3 percent for the non-stabilized material). Materials tested in the laboratory (repeated-load tri-axial and indirect tensile tests) and in the field (impulse deflection testing, vibration testing) both confirm significant increases over time in the resilient properties of lime-treated materials.

It was demonstrated that the lime content determined by the pH test was approximately the same as the lime content producing the maximum compressive strength .Lime can treat even highly organic soils provided that enough lime is used to overcome the organic masking effect. The slow process of carbonation and formation of cementitious products can lead to long-term strength increases. Lime stabilization is carried out through combining four mechanisms: namely cation exchange, flocculation and agglomeration, carbonation and pozzolonic reactions *(Dallas N. Little, 2009)*.

Fracture and fatigue- Flexural fatigue strength is related to the number of loads that can be carried by a material at a given stress level, and it is an important consideration in the evaluation of lime–soil and lime–aggregate mixtures. The strength-gain effects produced by pozzolanic reactions are often substantial for reactive soils. It has been found that lime stabilization is more effective for montmorillonitic soils than for kaolinitic soils (*Dallas N. Little, 2009*).

Durability- The ability of lime-stabilized materials to resist the detrimental effects of moisture and freeze-thaw cycling over time has been evaluated in several ways, in both the laboratory (e.g., soaking in conjunction with strength/stiffness tests, cyclic freeze-thaw tests) and the field. The results of these evaluations have often shown only slight detrimental effects of environment on the levels of strength/stiffness produced by the addition of lime. As an example, an Illinois study found that the ratio of soaked to un-soaked compressive strength of lime–soil mixtures is quite high, at approximately 0.7 to 0.85.The soaked specimens seldom achieved 100 percent saturation and, in most cases, the degree of saturation was in the range of 90 to 95 percent. The durability requirement for the stabilized soil is tabulated in table 2-5 below

Table 2-5Durability requirements for stabilized soils

ID	Type of stabilized soil		Max. allowable weight loss after 12 wet-dry or Freeze-thaw cycle percent of initial specimen weight
	Type	PI	
1	Granular	<10	11
2	Granular	>210	8
3	Silt		8
4	Clay		6

2.8.6. Soils suitable for lime stabilization

Lime has been found to react successfully with medium, moderately fine and fine grained soils causing a decrease in plasticity and swell potential of expansive soils, and an increase in their workability and strength properties .A Research showed that lime may be an effective stabilizer in soils with clay content as low as 7 percent and in soils with plasticity indices below 10 (*15*). The National Lime Association recommends a plasticity index of 10 or greater in order for lime to be considered as a potential stabilizer whereas the U.S Army Corps of Engineers recommends a plasticity Index of 12 or greater for successful lime stabilization (*6, 16*). Based on AASHTO classification, soil types A-4, A-5, A-6, A-7 and some of A-2-6 and A-2-7 are suitable for lime stabilization.

A minimum clay content of (<2µ) of approximately 10 percent and a plasticity index greater than 10 are desirable although benefits have been noted for lower PI Silty soil containing less Clay (Nebro D.,2002)

2.9. Mixture design

The use of lime to dry, modify, and stabilize soil is a well-established construction technique, documented in studies dating back to the 1950s and 1960s. A variety of mixture proportioning procedures have evolved, as various agencies have developed criteria and procedures to fit their specific design needs and objectives, often reflecting local conditions and experience (Dallas N.Little,1995).

The primary objective of mixture design is to identify an optimum lime content to be used during construction to modify or stabilize the soil or aggregate. The optimum lime content is a function of the expectation of how the stabilized material will be used. This is because a fairly wide range of lime contents can be used based on the desired engineering properties of the lime-soil aggregate mixtures. The desired objectives may range from modification to Stabilization (Dallas N.Little,1995).

Modification

Mixture design can be divided in to two categories based on our objectives. The first category is defined as modification. In this category the objectives are limited in plasticity reduction, improved workability, immediate shear strength increase and reduced Volume change potential. The mix design criteria for this category include (Dallas N.Little,1995).

- No further decrease in PI with increased percentage of lime
- Acceptable PI reduction for the particular Stabilization Objective
- Acceptable swell potential reduction
- Shear Strength (i.e CBR or R-Value)

Stabilization

This category involves strength related criteria which are primarily related to pozzolanic reaction between lime and the soil minerals. Strength criteria can be rationally related pavement performance and pavement structural design. Additionally the mix can enhance the durability. Strength and durability are objectives of stabilization (Dallas N.Little, 1995).

2.9.1. Current Mix design Procedures

A large number of mix design procedures exist. This is because various states and agencies have developed particular criteria and procedures to fit their specific design needs and lime-soil mixture property objectives.

Several mix design procedures are summarized in a number of publications.Some of the Mixture design procedures are the California procedure, Eades and Grim Procedure, Illinois Procedure, Oklahoma Procedure, South Dakota Procedure, Thompson Procedure and Virginia Procedure. The Thompson procedure is one of the procedures that this thesis adapts. This is because the procedures are in line with the laboratory test procedures followed and analysis will be simple for clarification (Dallas N.Little,1995).

The objective of this research mix design is emphasized in acceptable reduction of the plasticity index, swell potential and shear strength gain sufficient for anticipated uses.

2.10. Engineering properties of lime stabilized soils

In general all lime treated fine grained soils exhibit a reduction in plasticity, decreased shrink-swell and improved workability. However not all soils demonstrate a substantial level of improved strength gain. This strength gain is due to pozzolanic reactivity. The level of improvement in physical properties exhibited in soils is dependent up on soil type, lime type, lime percentage and curing condition, i.e time, temperature and moisture(8).

Lime treatment results in both immediate and long term effects on soil properties. These discussions of the engineering properties of lime treated soils are divided in to two categories. These are immediate (uncured) and long term (cured) strength properties. The curing period refers to a period of time when temperature and moisture are sufficientto provide an adequate environment for pizzolanic strength gain (Dallas N.Little,1995).

2.10.1. Uncured mixtures

The immediate effects of lime treatment on soils which are suitable for lime stabilization are due to the mechanism of cation exchange,$Ca(OH)_2$ adsorption to the clay surfaces and to some extent rapid development of pozzolanic products. The level of physical property changes in the soil which results from lime treatment is quiet soil dependent. However virtually all fine grained soils , regardless of soil lime reactivity, derive some level of physical property consistency improvements through lime treatment as reflected by changes in atterberg limits and changes in volumetric measurements due to moisture fluctuations (Dallas N.Little,1995).

Some of the uncured mixtures engineering properties of lime treated fine grain soils are Plasticity, moisture density relations, swell potential, and strength and deformation properties.

Plasticity

Substantial plasticity reductions are observed by lime treatment and the soil often becomes non plastic. Generally high PI and High clay content soils require greater quantities of lime to achieve the non-plastic condition (Dallas N.Little,1995).

Moisture density relationships

The results of immediate reactions (Cation exchange, flocculation/Agglomeration) between lime and the soil is a substantial change in soil density relationships. For a lower specific compactive effort, lime treated soils have lower density and higher optimum moisture content than the untreated soils (Dallas N.Little,1995).

Swell potential

Soil swell potential and swelling pressures are normally reduced by lime treatment. The swell of the soil is determined during the 96 hour soak period which is part of the CBR test procedure. Although the swell potential of lime treated soils vary, it is common for lime treatment to reduce the swell to less than 0.1percent (Dallas N.Little,1995)

Strength and deformation properties

Lime treatment of fine grained soils produces immediate improvement in strength and deformation properties of uncured soil-lime mixtures. These immediate benefits are evident from CBR, Cone index, R-value Static Compression and resilent modulus testing (8).

2.10.2. Cured mixtures

The most important effects of long term curing is the development of pozzolanic products. The development of more pozzolanic products results in more “glue” to hold the particles of the soil together a mineralogical change favorable to greater strength. One of the most important engineering properties of soils to be considered in cured mixtures is the unconfined compressive strength of soils.

The unconfined compressive strength of soils is one of the most widely used measure of the shear strength of lab fabricated lime stabilized soils. This is because the UCS is a measure of compressive shear strength and is presented in engineering units which can be used in engineering calculations, analysis and design. The unconfined compressive strength of lime treated soils increases with the increase of curing period (Dallas N.Little, 1995).

2.11. Concluding Summary

Expansive soils are the most problematic soils in the construction industry to work with. There are a number of ways to mitigate these problematic soils. One of the methods is chemical stabilization of the problematic soil with different additives. Lime is the chemical stabilizer for this thesis.

Lime is one of the most utilized chemical stabilizers used for fine grained soils especially for clay minerals. In most literatures and reference books, Stabilization of expansive soils with lime shows

- ✓ Reducing the liquid limit, plasticity index, shrinkage limit and linear shrinkage of the problematic soil
- ✓ Increasing the bearing capacity of the soil
- ✓ Increasing the unconfined compressive strength of the soil
- ✓ Improving the workability of the problematic soil
- ✓ Reducing the degree of expansion and swell

Lime is a chemical compound which is usually known as quick lime or unslaked lime. Lime has been used for centuries as a chemical stabilizer for clay minerals. The most usually used forms of lime as chemical stabilizers in clay minerals are quick lime and hydrated lime. Hydrated lime is the most widely used stabilizer than quick lime as quick lime is much caustic than hydrated lime. Hydrated lime is the one utilized in this thesis.

In general all lime treated fine grained soils exhibit a reduction in plasticity, decreased shrink-swell and improved workability. Lime treated soils has also show a reduction in maximum dry density with an increase of the optimum moisture content. Lime treated soils has also improve the strength of the soil through the pozzolanic reactions of the lime soil mixtures.

3. RESEARCH METHODOLOGY

3.1. Description of the study area

The study area is located at the south western part of the country named Gambella regional state. The project road begins at adura village around 150km away from the Gambela town and ends at Burbey village,found 45km west of adura village, at the Ethio-Sudan border, Baro River. There are a number of projects under construction in this region like Gambella Itang and Adura burbey.

The Adura-Burbey is located at the lowland part of the country, which is characterized by Flat terrain. The project has a length of 44km. the whole project length is covered by toll savanna grass and different types of trees and bushes.

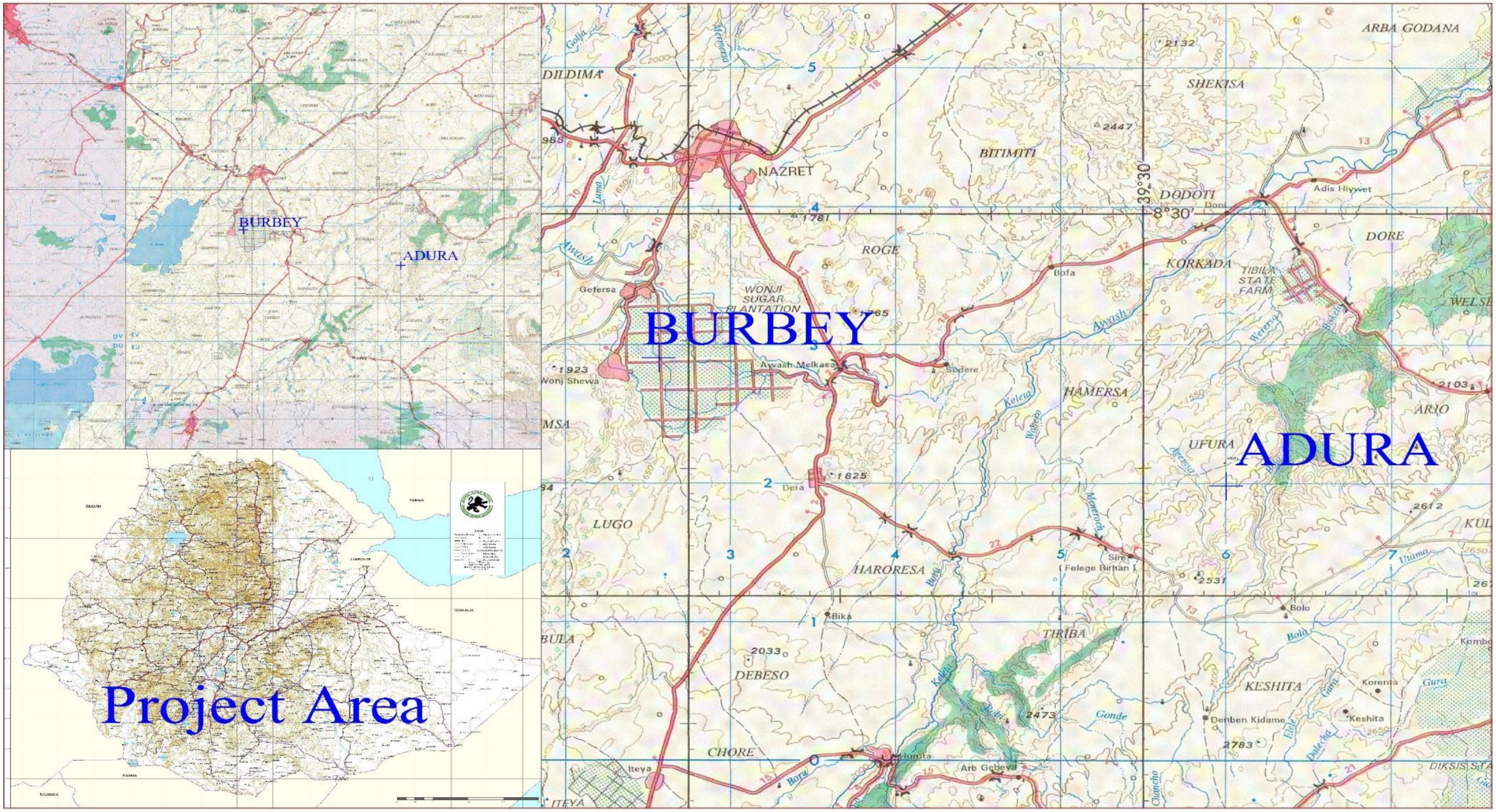


Figure 3-1 Location map of the study area

3.2. Description of the research design

Relevant literatures concerning Expansive soils are reviewed and designed to achieve the goals as follows.

I. Sampling of the material (Black cotton soil)

II. Laboratory testing

III. Laboratory data are analyzed

IV. Mixture Design and

V. Conclusions and recommendations will be made

3.2.1. Sampling of the material

Previous project data are collected from Core Consulting Engineers Plc about the soil and material investigations for Adura— Burbey road project as a Desk Study. Some specific locations which are thought to be expansive are selected from the previous laboratory results which were investigated in 2006 G .C. Some of the previous results are tabulated in table 3.1 below.

Sampling stations are selected from the lists with reference to their CBR, CBR Swell and Economy. Five sampling locations are selected from the previous data for further investigation. The selected stations and their respective previous laboratory results are attached in Table 3.2 below.

Further refinements of the samplings were made. The sample at 30+750 has a CBR value of 2.93 and CBR swell of 3.09. During the site visit and discussions with the representative staffs assigned on the construction site, the location (30+750) can be escaped and concentrate on other stations. Therefore, the sampling stations are refined to four stations namely stations 3+000,23+450,30+750,32+700 and 38+800.

Due to high flood in the area sampling at station 38+800 was not possible. This is because the location was in accessible and totally flooded during sampling. Through visual inspection and consulting the staffs on the specific project, location at 35+000 is also looks like problematic. Because of this samples were collected at 35+000 for further investigations. The final refined and properly selected stations for sampling were 3+000, 23+450, 32+700 and 35+000.

Table 3-1 previous laboratory results for Adura-Burbay Project (Source: Core consulting engineers' plc)

Sample Number	Sample Station	CBR Value	CBR Swell
1	1+000.00	2.9	2.33
2	3+000.00	2.66	6.85
3	5+600.00	7.32	1.08
4	7+500.00	4.03	2.39
5	9+900.00	4.94	2.96
6	12+950.00	4.39	1.61
7	15+050.00	6.59	1.14
8	17+100.00	7.14	2.07
9	19+200.00	3.29	2.93
10	21+200.00	5.57	2.41
11	23+450.00	1.86	5.11
12	26+200.00	3.34	2.45
13	29+000.00	3.9	2.51
14	30+750.00	2.93	3.09
15	32+700.00	1.46	5.91
16	34+300.00	3.48	2.99
17	36+300.00	2.38	2.76
18	38+800.00	2.2	5.43
19	41+000.00	2.93	2.62

Table 3-2 selected laboratory results for Adura-Burbay Project (Source: Core consulting Engineers plc)

Sample Number	Sample Station	CBR Value	CBR Swell	Remark
1	3+000.00	2.66	6.85	Sample collected
2	23+450.00	1.86	5.11	Sample collected
3	30+750.00	2.93	3.09	Escaped
4	32+700.00	1.46	5.91	Sample collected
5	38+800.00	2.2	5.43	Escaped
6	35+000			New Location

Samples were collected by an open excavation from a depth of 0.5 to 1.5m below natural ground level. Tests are conducted for the four stations with out the addition of lime. The results are shown in table 3.3 below.

Table 3-3 Laboratory test results for the selected stations for further studies

Sample Number	Sample Station	Liquid Limit	Plastic Limit	Plasticity Index	Soil classification
1	3+000.00	57.45	28	29.91	A-7-5(34)
2	23+450.00	56.6	27.28	29.32	A-7-5(32)
3	32+700.00	64.14	31	33.11	A-7-5(38)
4	35+000.00	55.47	27	28.19	A-7-5(32)

From the laboratory results, all the samples exhibit high expansive potential. Following the laboratory results conducted on the samples specified above, samples collected from stations 3+000, 23+450 and 35+000 have relatively similar Index properties and have similar classifications. And the sample collected at station 32+700 has properties relatively higher index properties. Because of this the emphasis of the research will focus on two categories.

Category one, the samples collected at 3+000, 23+450 and 35+000 will be blended and further studies with the addition of lime will be made. The following laboratory tests will be conducted for this category

I. Atterberg limits
II. Specific gravity
III. Free swell
IV. Linear shrinkage
V. Shrinkage limits and
VI. Unconfined Compressive Stress (not cured strength)

In category Two, sample collected from station 32+700 was thoroughly tested and investigated for the following tests and analyzed separately.

I. Atterberg limit tests
II. Specific gravity
III. Free swell
IV. Shrinkage limit tests
V. Moisture density relations
VI. California bearing ratio
VII. Unconfined compressive stress(7 days cured)

3.2.2. Laboratory testing

Various laboratory tests are available to identify and study the properties of soils. There are different tests to identify the physical and chemical properties of the soil. These properties are vital to choose the type of stabilizers as well as the property of stabilizers. Before testing, materials were prepared in accordance with AASHTO T 87-86. These methods involve air drying of samples and breaking up of soil aggregations by rubber covered mallet. The following tests were carried out to determine the soil properties.

- Atterberg Limits
- Shrinkage limit and linear shrinkage
- Maximum Dry Density (MDD) and Optimum Moisture Content (OMC)

- California Bearing Ratio (CBR) and CBR Swell
- Unconfined Compressive strength (UCS)
- Specific Gravity
- Free swell

3.2.2.1. Atterberg limits

The Atterberg limit tests consist of Liquid limit Plastic limit and Plasticity index. The test procedures are outlined in AASHTO materials testing Manual. The liquid limit values are determined in accordance with AASHTO T89-96. Similarly the plastic limit and plasticity index of the soil samples are determined in accordance with AASHTO T90-96

3.2.2.2. Shrinkage Limits and Linear Shrinkage

A sample with mass of about 30gm was taken from a thoroughly mixed material passing the 0.425(No.40) sieve prepared in accordance with the standard methods of preparation (T-87). And the tests were conducted in accordance with AASHTO T92-97.

3.2.2.3. Standard Proctor Compaction tests

Soil compaction consists of closing, packing the soil particles together the soil particles, so that increases the dry unit weight. Soil compaction only reduces the air void in the soil. A representative sample of approximately 3kg passing 4.75 (No.4) and tests were conducted in accordance with AASHTO T99-97.

3.2.2.4. California bearing ratio (CBR) and CBR Swell

The CBR test indirectly measures the shearing resistance of a soil under controlled moisture and density conditions. A soil sample retained on No. 19 will be discarded from the test. a soil mass is prepared in accordance with standards and tests were conducted in accordance with AASHTO T193-99.

3.2.2.5. Unconfined Compressive Strength (UCS)

The unconfined compressive strength test is applicable only in cohesive soils. The test is carried out in cylindrical specimen with dimensions of 50mm diameter 100mm in length. In this test the soil goes to failure by axial load only with no confining surrounding stresses. The UCS values are determined in accordance with AASHTO T-208.

The unconfined compressive strength tests were conducted for both blended and sample at station 32+700. 7 days cured UCS test was conducted for sample at 32+700 but immediate (uncured) strength test was made for blended sample (3+000,23+450 &35+000).

The immediate (uncured) and cured unconfined compressive strength tests were not conducted for the same sample. To see the effect of curing it was best to test for only similar samples not different samples. But because of limitations in sample size, time and finance, the tests(cured and uncured) were conducted on different samples.

3.2.2.6. Specific Gravity

Specific gravity of a soil is defined as the ratio of the unit weight of the given material to the unit weight of water. A mass of 70gm was used for each test. The specific gravity of soils was determined in accordance with AASHTO T100-95(2000).

3.2.2.7. Free Swell

Free swell of a soil is defined as the ratio of the difference in volume of the sample to the original volume. The Free swell of soils was determined in accordance with IS: 2720(Part 40) 1977.

3.2.3. Laboratory Data Analysis and Results

Here the laboratory data's for the collected samples are elaborated and analyzed for the natural subgrade soil and the lime treated soil which can help in visualizing and concluding the results. For the performed tests, the results are organized in a meaningful way and each data's are interpreted.

3.2.4. Mixture Design

The mixture design of the sample at 32+700 was determined with the objective of acceptable CBR swell and acceptable California bearing ratio (CBR).

3.2.5. Conclusions and Recommendations

Based on the Literature Reviews, results obtained from laboratory and design analysis the true outputs and recommendations will be made. The conclusion of the thesis will be based on the laboratory results and they will be checked with the previous researches.

The recommendations of the thesis will be based on the literatures and the results of this thesis. Necessary information's will be forwarded for the coming researchers. The design methodology and the work flow to arrive to the required goal are indicated in flow chart below.

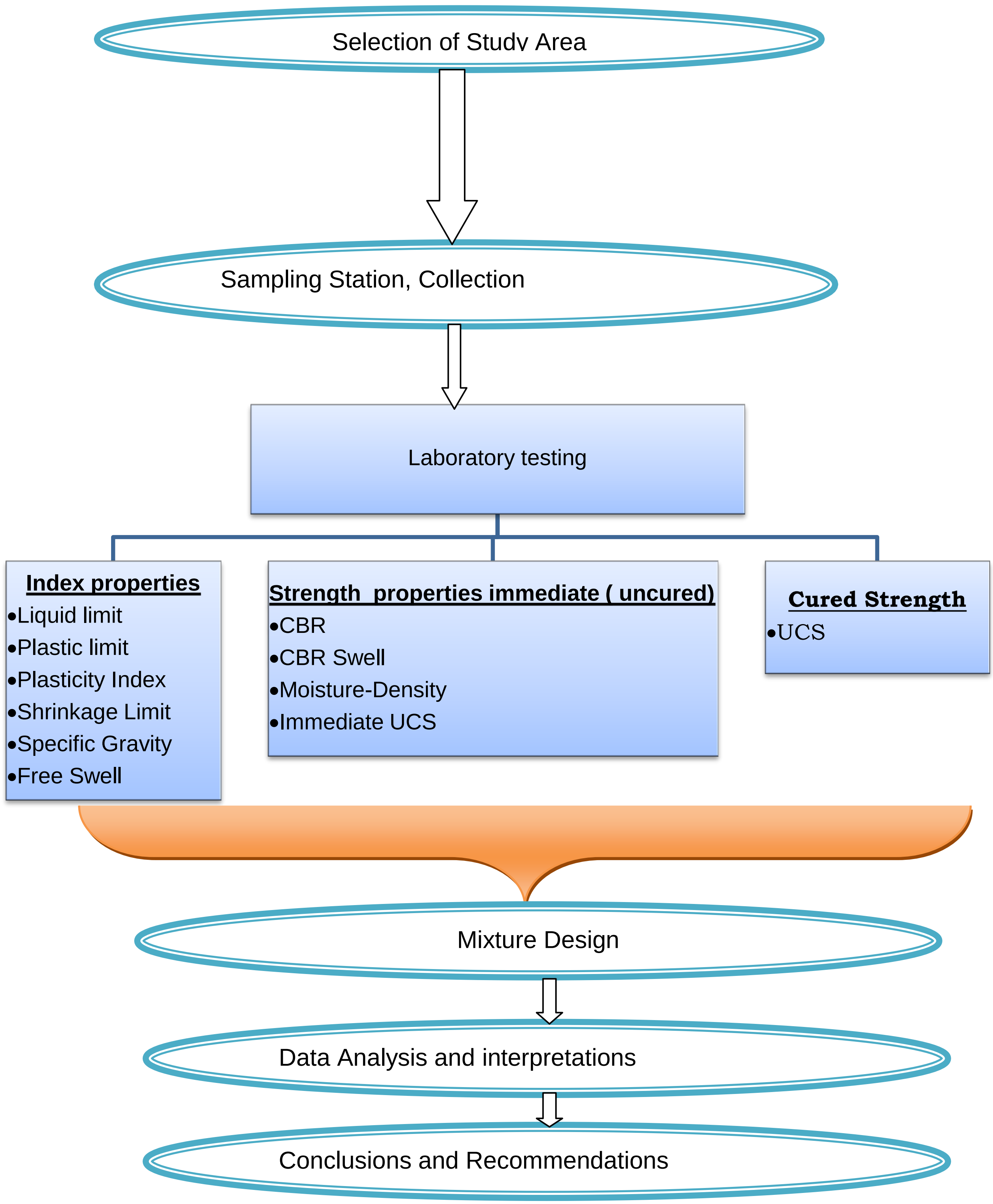


Figure 3-2 Description of the study design

4. LABORATORY TESTS AND RESULTS

4.1. General

Based on the methodology reported in chapter 3, how the samples are collected, laboratory test types and the respective results are discussed. Each test types for the samples will be presented with and without the addition of lime.

4.2. Sample collection and preparations

During sample collection a disturbance of the sample is highly expected. Samples are collected considering the reduction of disturbances and are mixed with lime for further investigation of the soil sample Samples are collected from the region specified in the methodology part and tested for the following laboratory tests to evaluate the specimen.

- Atterberg Limits and Shrinkage Limits
- Linear shrinkage
- Free Swell
- Specific Gravity
- Maximum Dry Density (MDD) and Optimum Moisture Content (OMC)
- California Bearing Ratio (CBR) and CBR Swell
- Unconfined Compressive strength (UCS)

4.2.1. Atterberg Limits

The test procedure adapted for the determination of Liquid limit, Plastic Limit and plasticity index are in accordance with AASHTO T89-96 and T90-00 respectively. Hand mixing in a porcelain pan is the method of mixing. The tests were conducted by mixing lime content of 0, 2, 4,6,8,10,12 percent by oven dry weight. A sample weighting about 10gm was taken from the mixture prepared for liquid limit test for each samples. Two plastic limits tests were conducted for each samples and the average of the two was taken as the Plastic Limit. Summaries of the test results are tabulated in table 4.1 and 4.2 below and the detail laboratory data's are attached as an Appendix-I

4.2.2. Shrinkage limits and linear shrinkage

The shrinkage limit of the soil sample is determined in accordance with AASHTO T92 97(2000). A sample is extracted with a mass of about 30gm taken from a thoroughly mixed material sample in accordance with T87.

The samples collected at station 32+700 and the blended samples (samples at station 3+000, 23+450 and 35+000) are tested with and without the addition of lime. The percent of lime used for the tests are 2, 4, 6,8,10 and 12 percepts by dry weight of the soil samples.

The summary of the laboratory results for the samples at station 32+700 and blended samples are tabulated in table 4.1 and 4.2 below respectively. The laboratory results of the shrinkage limit and linear shrinkage tests and results are attached in Appendix II

Table 4-1 Summary of laboratory results for Atterberg limits,Shrinkage Limit and Linear Shrinkage for sample at Sta. 32+700

Serial Number	Soil Type	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Shrinkage Limit (%)	Linear Srinkage (%)
1	Natural Soil	64.14	31.03	33.11	5.0	22.1
2	Soil+2% lime	60.33	26	34.29	7.6	21.4
3	Soil+4% lime	59.16	26.25	32.91	9.2	20.7
4	Soil+6% lime	56.07	26.27	29.8	9.3	20.1
5	Soil+8% lime	52.48	26.81	25.67	9.4	18.00
6	Soil+10% lime	50.5	28.83	21.67	10.9	17.00
7	Soil+12% lime	45.66	30.56	15.10	11.6	16.00

Table 4-2 Summary of laboratory results for Atterberg limits, Shrinkage Limit and Linear Shrinkage for sample at Sta. Blended material

Serial Number	Soil Type	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Shrinkage Limit (%)	Linear Srinkage (%)
1	Natural Soil	56.6	27.28	29.32	5.60	24.90
2	Soil+2% lime	54.52	34	20.34	7.9	23.60
3	Soil+4% lime	53.02	33	19.98	10.10	22.10
4	Soil+6% lime	50.47	32	18.73	10.4	21.40
5	Soil+8% lime	43.83	28	15.54	10.4	21.00
6	Soil+10% lime	41.39	26	15.07	12.10	19.30
7	Soil+12% lime	41	26	15	12.90	17.10

4.2.3. Specific Gravity

The test procedure followed for the determination of Specific gravity is in accordance with AASHTO T100-95(1999). A sample weighting about 25gm is used in the test on oven dry basis as the volumetric flask is used in our test procedure. Tests were performed with the addition of lime 0, 2, 4,6,8,10,12 percent by weight of oven dry hydrated lime. The test results are tabulated in table 4.2 below. The detail laboratory results of specific gravity are attached in Appendix III

4.2.4. Free swell

The test procedure followed for the determination of free swell is in accordance with IS: 2720(Part 40) 1977.Tests were performed with the addition of lime 0, 2, 4,6,8,10,12 percent by weight of oven dry hydrated lime. The test results are tabulated in table 4.2 below.

Table 4-3 Summary of laboratory results for Specific Gravity and Free Swell

Serial Number	Soil Type	Specific Gravity(Gs)	Free swell	Specific Gravity(Gs)	Free swell
		Sample at Station 32+700		Sample for Blended	
1	Natural Soil	2.485	70	2.387	70
2	Soil+2% lime	2.50	60	2.416	60
3	Soil+4% lime	2.597	45	2.424	50
4	Soil+6% lime	2.597	40	2.538	40
5	Soil+8% lime	2.614	35	2.548	30
6	Soil+10% lime	2.632	30	2.632	20
7	Soil+12% lime	2.649	25	2.662	10

4.2.5. Moisture density relations

The moisture density relations are determined based on AASHTO T99-97. Tests were conducted with the addition of lime 0, 2, 6,8,10 and 12 percent hydrated lime. Summarized results are tabulated in Table 4.3 below. The details of the test results are attached in Appendix IV

Table 4-4 Summary of laboratory results for maximum dry density and optimum water content for sample at station 32+700

Serial Number	Soil Type	OMC (%)	MDD
1	Natural Soil	25	1.51
2	Soil+2% lime	25.1	1.51
3	Soil+4% lime	26	1.49
4	Soil+6% lime	27.3	1.48
5	Soil+8% lime	28.2	1.48
6	Soil+10% lime	28.5	1.47
7	Soil+12% lime	29.30	1.46

4.2.6. California Bearing ratio (CBR) and CBR Swell

The CBR Values are determined based on AASHTO T193-99. Tests were conducted with the addition of lime 0, 2, 6,8,10 and 12 percent hydrated lime. Specimens are molded at respective optimum moisture content as determined in moisture density relationships. The compactive efforts used are in accordance with T99. Moisture content and density before soaking were determined. Swell measurements are taken at the time of soaking and after four days of soaking. Results are tabulated and illustrated in Table 4.5 below. The details of the laboratory results are attached in Appendix IV.

Table 4-5 Summary of laboratory results for CBR and Percent Swell of the Sample at station 32+700

Serial Number	Soil Type	CBR Values in %			CBR SWELL (%)		
		10 Blows	30 Blows	65 Blows	10 Blows	30 blows	65 blows
1	Natural Soil	0.7	1.2	2	3.97	3.5	3.11
2	Soil+2% lime	0.9	1.5	1.7	3.9	3.48	3.08
3	Soil+4% lime	0.9	1.9	2.6	3.68	3.23	2.92
4	Soil+6% lime	1.7	2.6	3.3	3.38	2.91	2.62
5	Soil+8% lime	1.7	3.3	4.5	2.91	2.38	2.13
6	Soil+10% lime	2	4.3	5.9	2.64	2.21	1.97
7	Soil+12% lime	3.5	6.5	8.9	2.3	1.97	1.73

4.2.7. Unconfined Compressive Stress

The UCS Values are determined based on AASHTO T208-96(2000). Tests were conducted with the addition of lime 0, 2, 6,8,10 and 12 percent hydrated lime. Two samples are tested for the UCS. The first sample was blended sample (sample collected at 3+000, 23+450 and 35+000) and the other sample was from station 32+700. The blended sample was tested for the Immediate UCS and the sample collected at station 32+700 is tested for the 7 days cured stress. Specimens are cut out by pushing a cylindrical soil sampler in to a compaction mold using a universal compression machine the samples that were extruded from the cylinder was wrapped in aluminum foil and plastic cover and tested for its strength for both immediate strength gain and cured strength for seven days in accordance with AASHTO T208-92.The summarized lab results are shown in table 4.5 below. But the detail laboratory values are attached in Appendix V.

Table 4-6 Summary of laboratory results for Unconfined Compressive Stress for two samples

Serial Number	Soil Type	Immediate(KPA)for Blended material	Cured for 7 days (KPA) for sample at 32+700
1	Natural Soil		
2	Soil+2% lime	113.12	276.21
3	Soil+4% lime	134.56	374.41
4	Soil+6% lime	138.83	484.59
5	Soil+8% lime	167.33	606.47
6	Soil+10% lime	216.94	696.66
7	Soil+12% lime	254.03	791.73

4.3. Summary of laboratory tests

The laboratory tests conducted in this research are Atterberg limit tests; shrinkage limits linear shrinkage, specific gravity free swell, California bearing ratio, moisture density relations, and unconfined compressive strength. All tests are performed in accordance with AASHTO laboratory test standards.Lime percentages from 2 to 12 percent's by dry weight of the soil sample are used for each test types. Using uniform lime percentage for each test can help in understanding the visible effects of lime at each stage of lime applications.

Samples were collected from four stations namely 3+000, 23+450,32+700 and 35+000 for investigating the properties of the black cotton soil. After testing the samples for index and classification properties, samples collected at stations 3+000, 23+450 and 35+000 shows relatively similar properties. Because of this the research adopts to investigate the properties of the soil by blending the three samples(samples collected at station 3+000,23+450 and 35+000) and separate studies for sample at station 32+700

5. LABORATORY DATA ANALYSIS AND INTERPRETATIONS

5.1. General

The general procedure followed for the analysis of the data, primarily consists of performing mathematical and graphical illustration analysis coupled with subjective evaluation. Tabular and graphical analysis was adopted to determine whether lime stabilization has an effect on Atterberg limits, shrinkage limits, linear shrinkage, moisture density relations, California bearing ratios and unconfined compressive strength of the sample. With refined samples used in the laboratory, clear effects are identified. The graphical and tabular interpretations were performed by means of Microsoft Excel software.

5.2. Effects of lime on Atterberg Limits

5.2.1. Effects of lime on Liquid limit

The laboratory results of the liquid limit of the sample at station 32+700, the blended samples at stations 3+000, 23+450 and 35+000 are illustrated in figure 5-1 and 5-2 below.

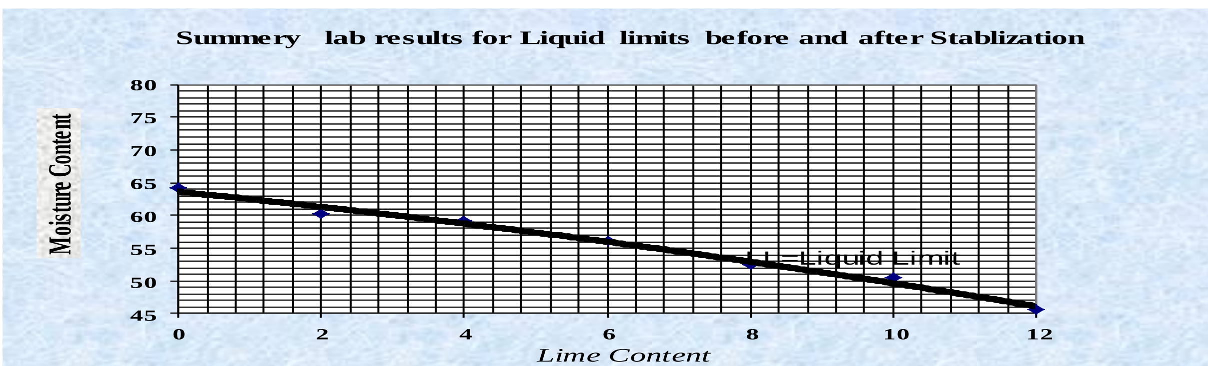


Figure 5-1 Liquid limit Versus Lime Content for sample at 32+700

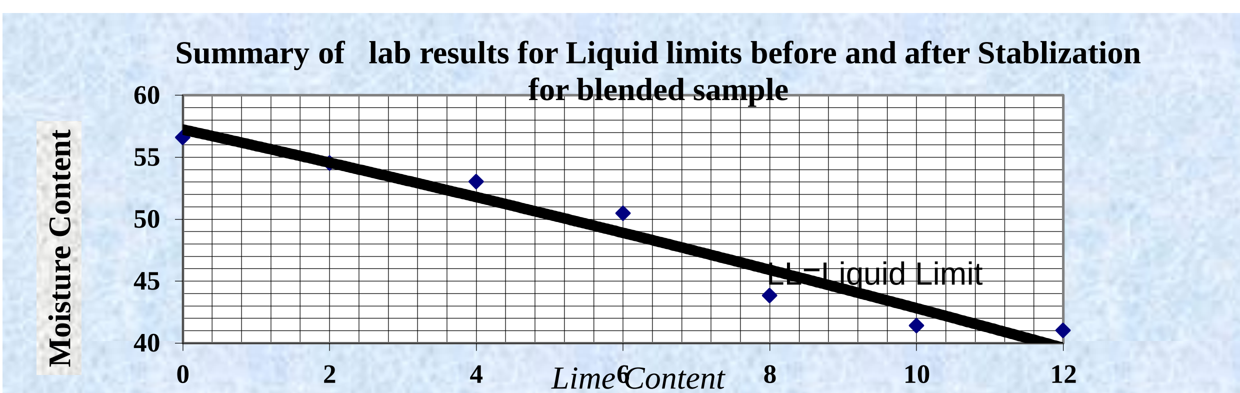


Figure 5-2 Liquid limits versus lime contents for Blended samples

As shown in the figures above, the liquid limit of samples has decreased with the addition of lime. The liquid limits have also decreased with the increase of lime percentage. The decrease of the liquid limit was in the order of 3.81% and 2.08 % by the addition of 2 percent lime for samples at 32+700 and Blended (3+000,23+450 & 35+000) respectively .The maximum reduction in liquid limit was 18.48% and 15.6 % with the addition of 12% lime for samples at 32+700 and blended samples respectively.

Table 5-1Summary of the laboratory results for atterberg limits for both samples

ID	Soil Type	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
		For sample at 32+700			Blended sample		
1	Natural Soil	64.14	31.03	33.11	56.6	27.28	29.32
2	Soil+2% lime	60.33	26	34.29	54.52	34	20.34
3	Soil+4% lime	59.16	26.25	32.91	53.02	33	19.98
4	Soil+6% lime	56.07	26.27	29.8	50.47	32	18.73
5	Soil+8% lime	52.48	26.81	25.67	43.83	28	15.54
6	Soil+10% lime	50.5	28.83	21.67	41.39	26	15.07
7	Soil+12% lime	45.66	30.56	15.10	41	26	15

From table 5.1 above the liquid limit of both samples has improved through stabilization. in the literature review (table 2-1), the liquid limit and degree of expansions have been stated. The untreated soils liquid limit values determined in laboratory are 64.14 and 56.6% for samples at 32+700 and blended samples respectively. The category of degree of expansions falls in **very high** and **high** for samples at 32+700 and blended samples respectively. With 12% lime treatment the sample at 32+700 reduced from swelling potential of very high to high swelling potential. But the blended sample has not changed its category.

5.2.2. Effects of lime on Plastic Limit

The effects of lime treatment on the plastic limit of both samples are shown in the table 5-1 above and figure 5-3 and figures 5-4 below

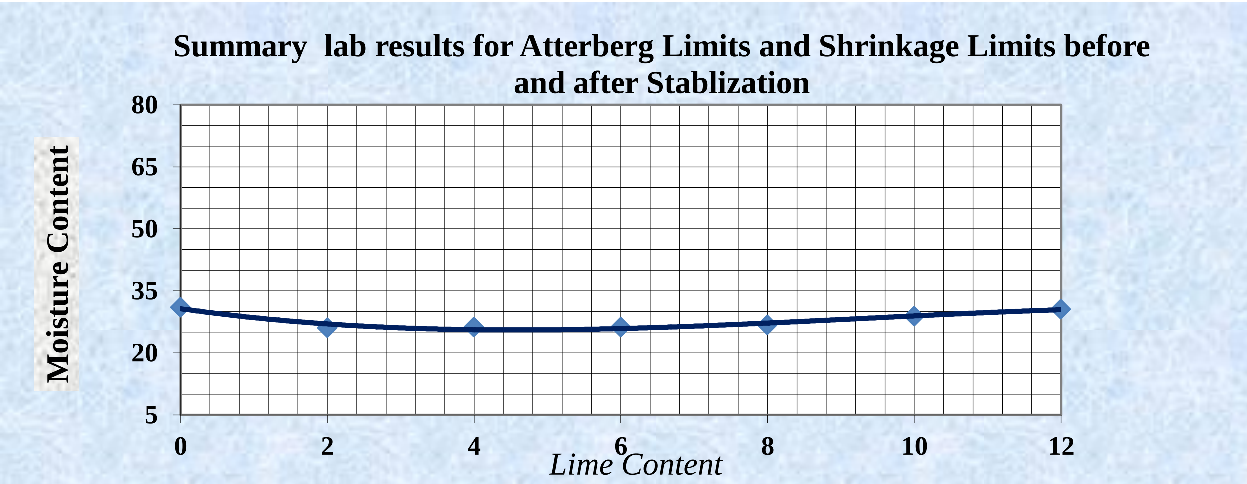


Figure 5-3 Plastic limits versus lime contents for sample at 32+700

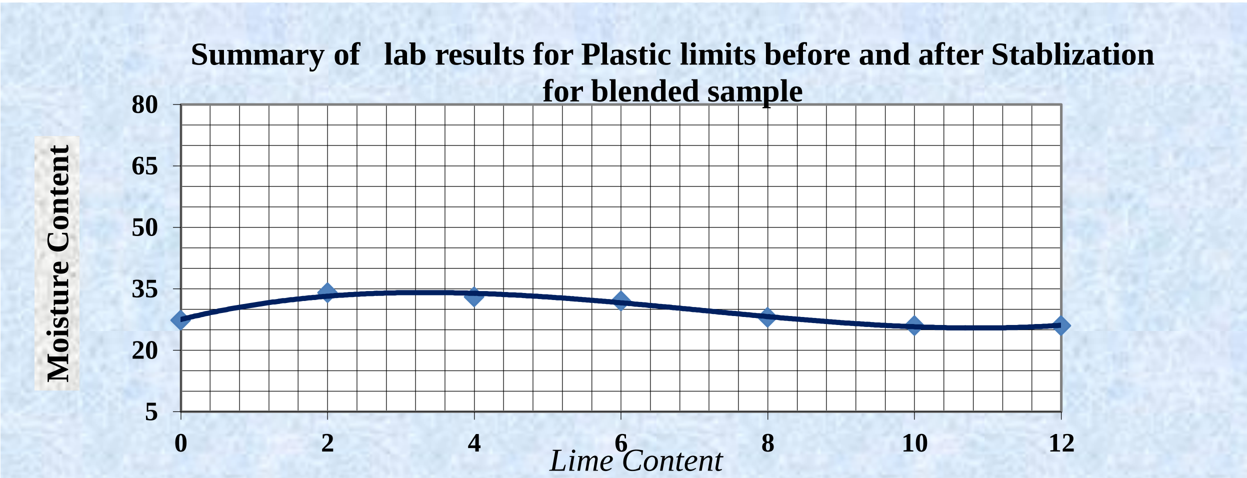


Figure 5-4 Plastic limits versus lime contents for Blended sample

From table 5-1 and figures 5-3 and figures 5-4, the Plastic limit of sample at 32+700 has decreased from 31.03 to 26% by the addition of 2% lime and starts to increase to reach 30.56% with 12% lime treatment. The increase of the Plastic limit was in the order of 4.56 % by the addition of 12 percent lime for sample at 32+700.

But the plastic limit of the blended sample shows opposite result with that of the sample at 32+700. The plastic limit of the blended sample has increased from 27.28% to 34% with the addition of 2% lime and starts decreasing with further increase in lime percentages.

5.2.3. Effects of lime on Plasticity index

The properties of the plasticity index with the addition of lime are tabulated in table 5.1 above are shown in the figure 5-5 and 5-6 below

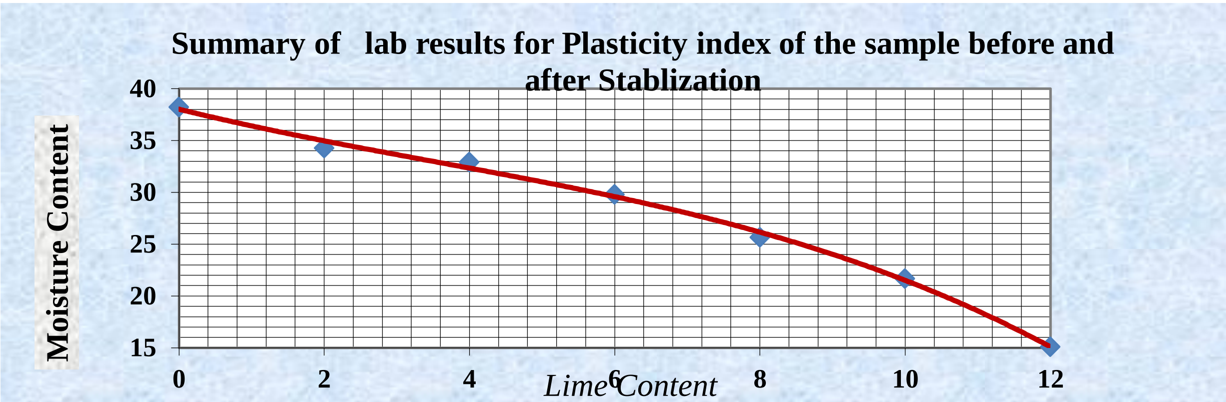


Figure 5-5 Plasticity index versus lime contents for sample at station 32+70

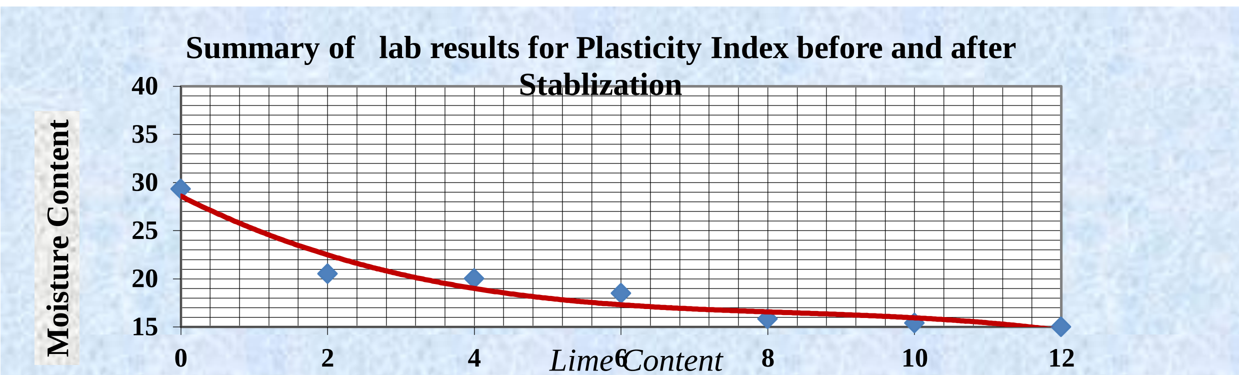


Figure 5-6 Plasticity index versus lime contents for Blended sample

The Plasticity index is the difference between liquid limit and plastic limit. Since liquid limit has decreased with excess amount and plastic limit has also increased with decreased rate, the plasticity index has decreased with the addition of lime. The maximum reduction was 23.14% and 14.32% for samples at 32+700 and blended respectively with the addition of 12% lime.

From table 5.1 above the plasticity index of both samples has improved through stabilization. from table 2-1 in the literature review the plasticity index and degree of expansions have been stated. The untreated plasticity index values determined in laboratory were 33.11% and 29.32% for samples at 32+700 and blended samples respectively. Both samples have plasticity index values which fall in the range of 20-55% which is categorized as high swelling potential soils. With 12% lime treatment, the plasticity index values become 15.1% and 15% for samples at station 32+700 and for blended samples respectively. The results fall in the range of 0-15%, which is classified as low swelling potential. Here it can be noted that the addition of 12% lime for both samples has reduced the swelling potential of the problematic soils from high swelling potential soils to low swelling potential which is a satisfactory and required range.

5.3. Effects of lime on Shrinkage limits

The properties of the shrinkage limit with the addition of lime are tabulated in table 4-1 and table 4-2 for samples at 32+700 and blended samples respectively and are shown in the figure 5-7 and 5-8 below

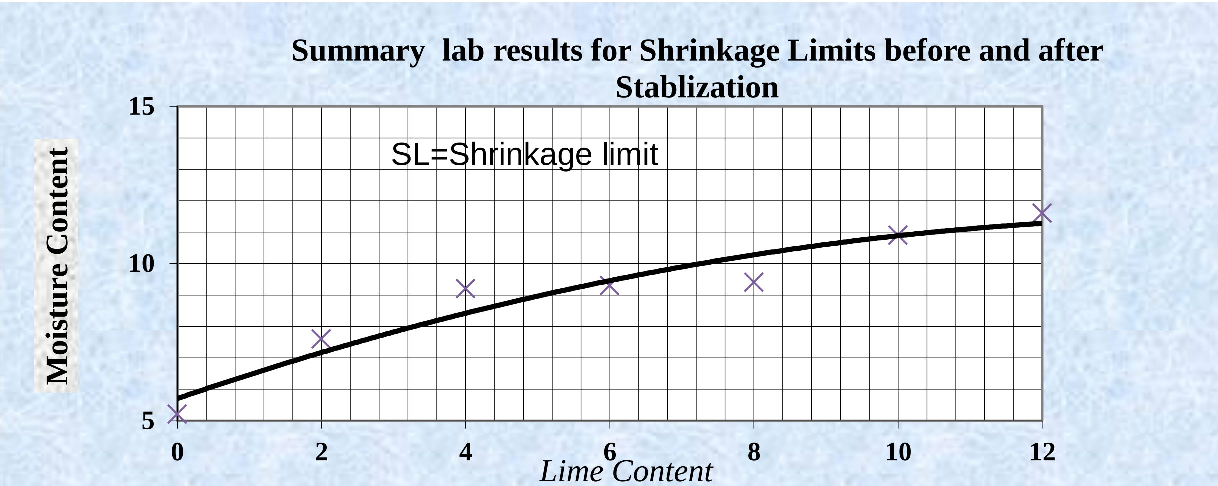


Figure 5-7 Shrinkage limits versus lime contents for the sample at station 32+7000

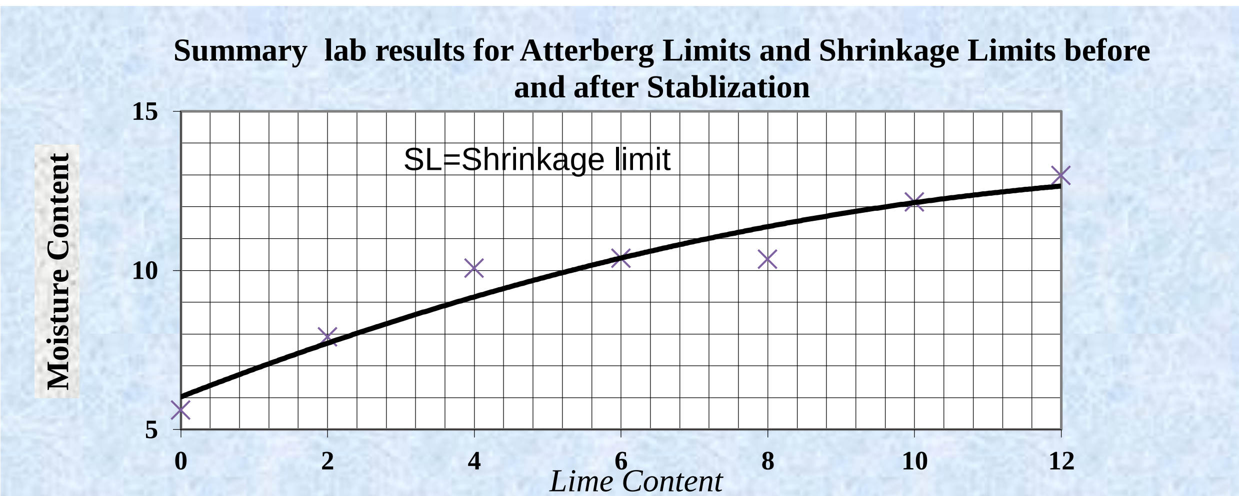


Figure 5-8 Shrinkage limit vs Lime Content for Blended sample

The Shrinkage limit of samples has generally increased with the addition of lime for the sample at station 32+700 and for the blended sample. The Shrinkage limit has also increased with the increase of lime percentage for the sample at station 32+700 and for the blended sample. The maximum increase in Shrinkage limit was 6.6% and 7.4 %

with the addition of 12% lime for the sample at station 32+700 and for blended sample respectively

From tables 4-1 and 4-2, above the shrinkage limit of both samples has improved through stabilization. From table 2-2 in the literature review, the shrinkage limit and degree of expansions have been stated. The untreated shrinkage limit values determined in laboratory were 5% and 5.6% for samples at 32+700 and blended samples respectively. Both samples have shrinkage limit values which are <10% which is categorized as **problematic**. With 12% lime treatment, the shrinkage limit values become 11.6% and 12.9% for samples at station 32+700 and for blended samples respectively. Here it can be seen that the addition of 12% lime for both samples has improved the swelling potential of the problematic soils from a class of **problematic soils** to non-problematic soils which is a satisfactory and required range.

5.4. Effects of lime on Specific gravity

The results of specific gravity with the addition of lime are tabulated in table 4-3 and are illustrated in figure 5-9 and 5-10 below.

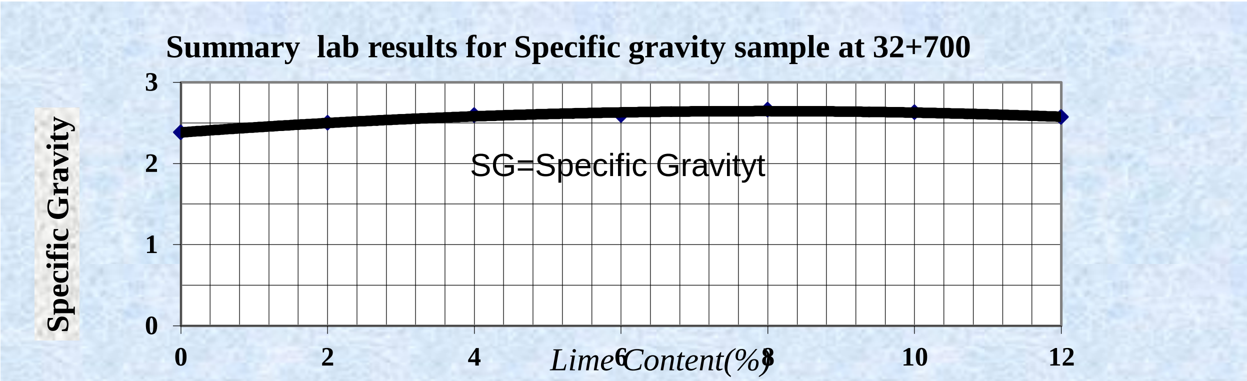


Figure 5-9 Specific Gravity versus lime contents for sample at station 32+700

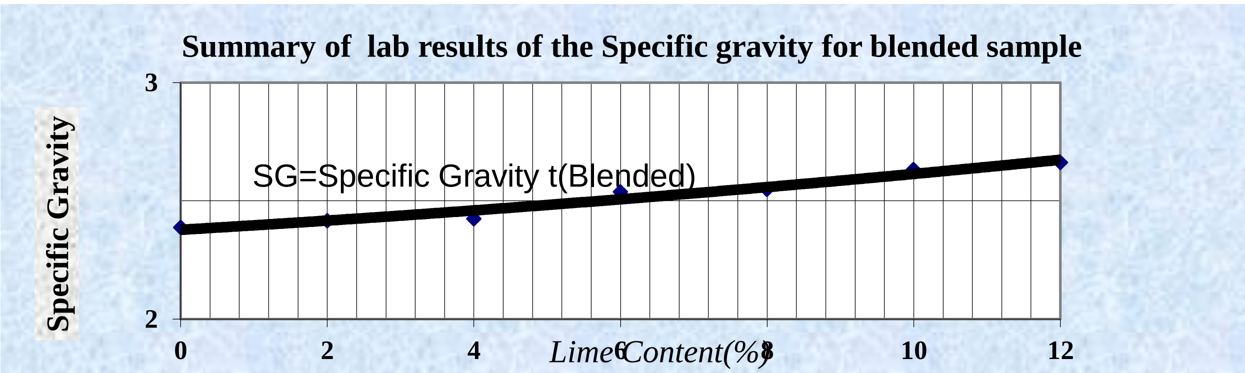


Figure 5-10 Specific gravity Versus Lime Content for the blended sample

Generally the specific gravity of the samples has increased with the addition of lime for both samples. The specific gravity of both samples has also increased with the increase of lime percentage. The increase of the specific gravity was in the order of 0.02 and 0.03 for the sample at 32+700 and Blended sample respectively by the addition of 2 percent lime. The maximum increase in specific gravity was 0.16 and 0.28 for sample at 32+700 and Blended samples respectively with the addition of 12% lime.

5.5. Effects of lime on free swell

The results of Free swell with the addition of lime are tabulated in table 4-3 above and are illustrated in figure 5-11 and 5-12 below.

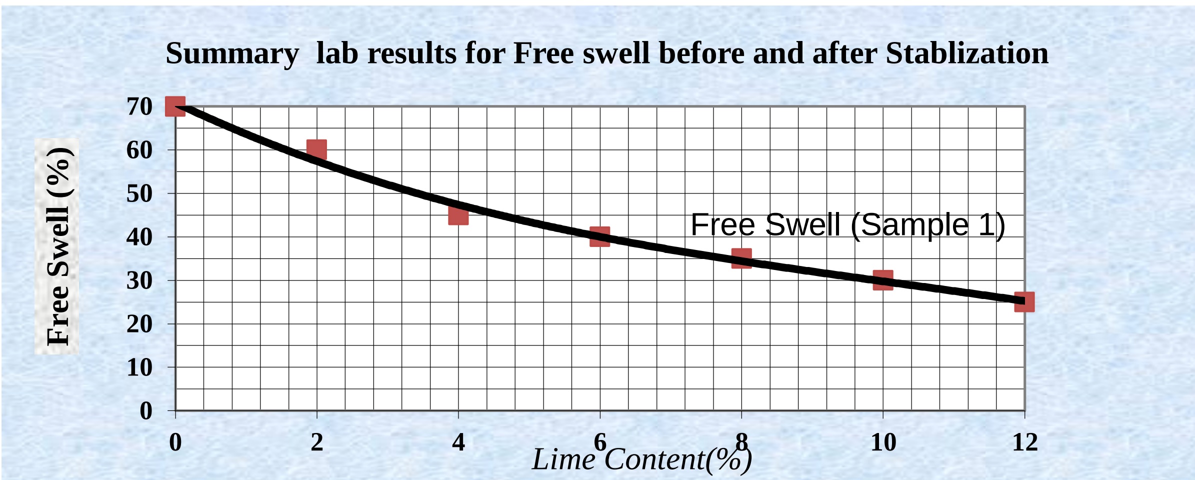


Figure 5-11 Free swell versus lime content for sample at Sta. 32+700

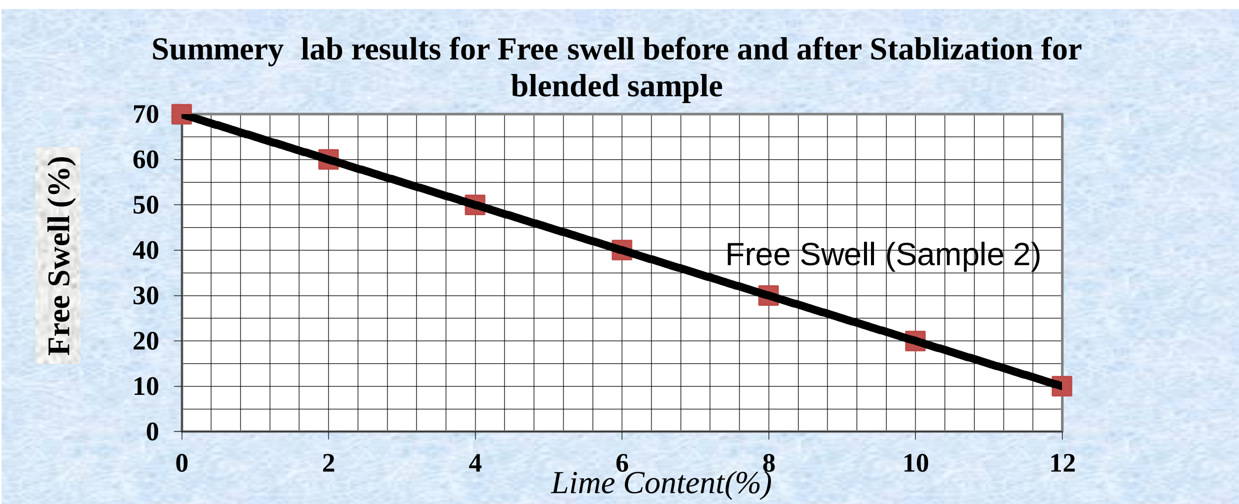


Figure 5-12 Free Swell Versus Lime Content for Blended sample

As shown in the figures above, the free swell of the sample has decreased with the increase in lime for both samples. The free swell has also decreased with the increase of lime percentage for both samples. The decrease of the free swell was in the order of

10% by the addition of 2 percent lime for both samples. The maximum decrease in free swell was 45% and 50% for Samples at 32+700 and blended samples respectively with the addition of 12% lime

In tables 4-3 above the free swell of both samples has improved through stabilization. In table 2-2 in the literature review the free swell and degree of expansions have been stated. The untreated free swell values determined in laboratory were 70% for both samples. Both samples have free swell values >50% which is categorized as **problematic**. With 12% lime treatment, the free swell values become 25% and 10% for samples at station 32+700 and for blended samples respectively. Here it can be seen that the addition of 12% lime for both samples has improved the swelling potential of the problematic soils from a class of **problematic soils** to non-problematic soils which is a satisfactory and required range

5.6. Effects of lime on other results

From the laboratory tests and results of moisture density and CBR Values, some combinations are made and the effects of lime on these different combinations are analyzed below. The summaries' of the laboratory results and the results of the relationships of these combinations are tabulated in table 5-2 below and are illustrated in the following figures below.

Table 5-2 Summarized laboratory results on moisture density and CBR values

	Moisture Vs DD		DD VS Percent Swell		DD VS CBR		Percent Swell Vs CBR	
Lime (%)	Moisture (%)	DD(g/cc)	DD(g/cc)	% Swell	DD(g/cc)	CBR (%)	% Swell	CBR (%)
0	25.00	1.51	1.51	3.11	1.51	2.04	3.11	2.04
2	25.10	1.51	1.51	3.08	1.51	1.67	3.08	1.67
4	26.00	1.49	1.49	2.92	1.49	2.60	2.92	2.60
6	27.30	1.48	1.48	2.62	1.48	3.34	2.62	3.34
8	28.20	1.48	1.48	2.13	1.48	4.46	2.13	4.46
10	28.50	1.47	1.47	1.97	1.47	5.94	1.97	5.94
12	29.30	1.46	1.46	1.73	1.46	8.92	1.73	8.92

5.6.1. Effects of lime on Moisture density relations

Moisture density relations are performed for the sample at station 32+700. The moisture density relationships are performed in accordance with AASHTO T-208. The results of laboratory tests are attached in Appendix IV. The summarized results for comparisons are tabulated in table 5-2 and the proctor curves are illustrated in figure 5-13 below.

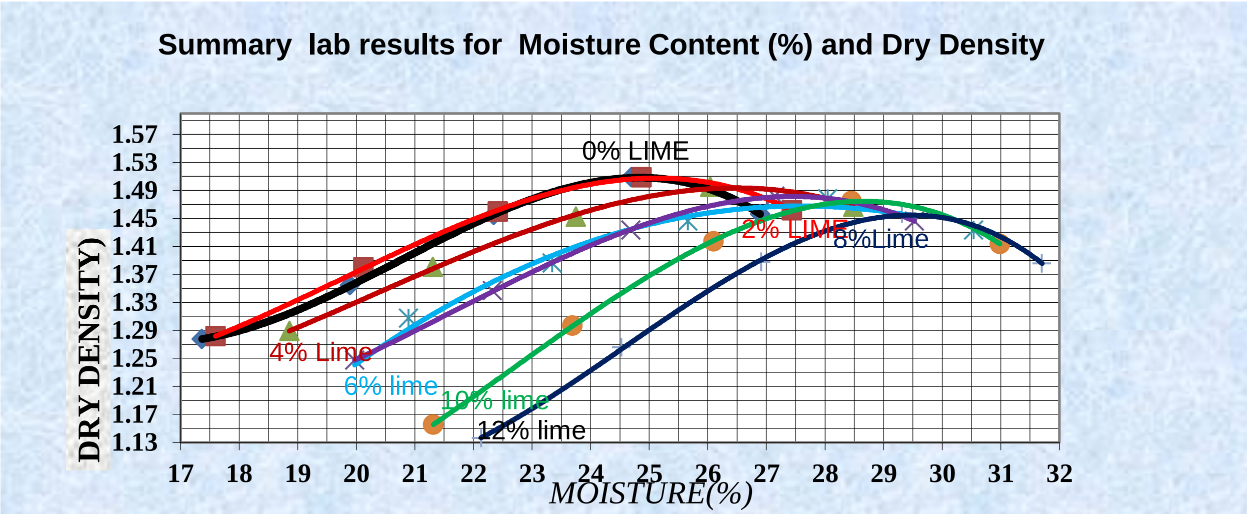


Figure 5-13 Dry density versus Moisture Content for the sample at station 32+700

As shown in figure 5-13, the dry density and moisture content up on lime treatment has opposite effect with the increase of optimum moisture content and a decrease of MDD. This scenario is generally true for all lime treated expansive soils.

5.6.2. Effects of lime on OMC and MDD

The results MDD and OMC with the addition of lime are tabulated in table 5-2 above and are illustrated in figure 5-14 below for sample at 32+700.

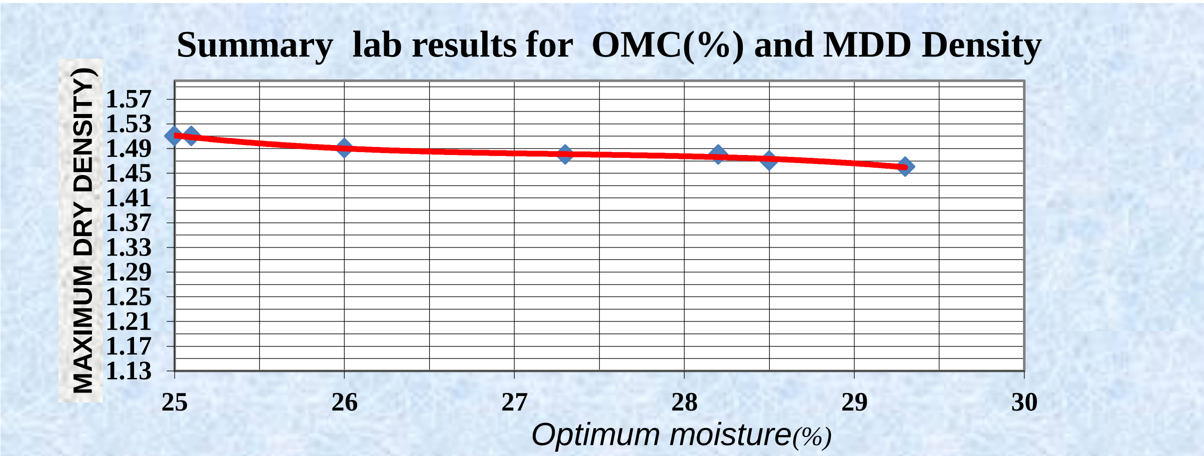


Figure 5-14 Maximum dry density Vs Optimum moisture content for sample at 32+700

The dry density of the sample has decreased with the increase in lime. But The moisture content has increased with the increase of lime. The increase of lime percentage has decreased the dry density at some degree at the expense of the increase in moisture content

The amount of lime added did not bring a significant change in the optimum moisture content and maximum dry density. The optimum moisture content is very important for the preparation of fully saturated lime stabilized soil.

The addition of lime changes the optimum moisture content (OMC) and maximum dry density (MDD) of soils because the effects of cation exchange and short-term pozzolanic reactions between lime and the soil results in flocculation and agglomeration of clay particles leading to textural changes. These results are in consistent obtained by Afes and Didier (2000). These results reveal the soils in wetter than original condition to be compacted satisfactorily

5.6.3. Effects of lime on MDD and Percent swell

The maximum dry density and present swell of the samples are indicated in the chart below. From the chart it is indicated that as the dry density of the treated soil increases, the percent swell decreases. And the dry density of the sample increases with the increase of lime percentage with the reduction of percent swell as well

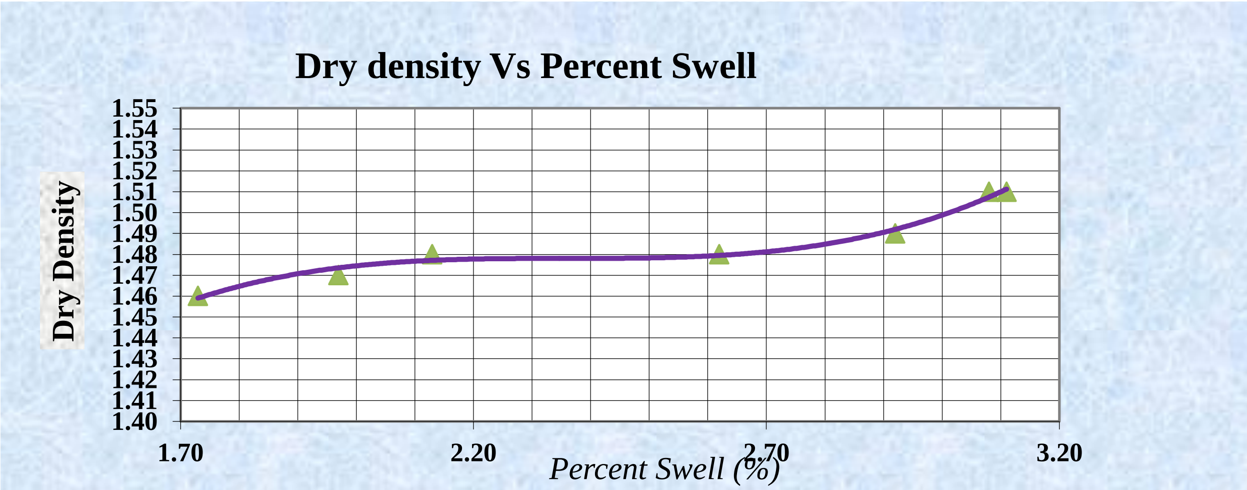


Figure 5-15 Dry density Vs Percent Swell for sample at 32+700

Percent swell is one of parameters to be controlled in the construction of black cotton soils. In most Ethiopian construction practices, the swell parameter should not exceed 3 %. In the Adura- Burbey DS6 road project, the swell of the soils was not allowed to exceed 2%. Lime treatment with 12% for the sample has improved from 5.49% to 1.73% which is by far the most improved criteria of the required construction specification to be used.

5.6.4. Effects of lime on Percent swell and CBR

The CBR Value increases with the increase of lime with the opposite effect on the CBR Swell. The CBR Value has also increased with the increase in lime Percentage. The effect of lime on CBR and Swell are opposite but with positive outcome after stabilization

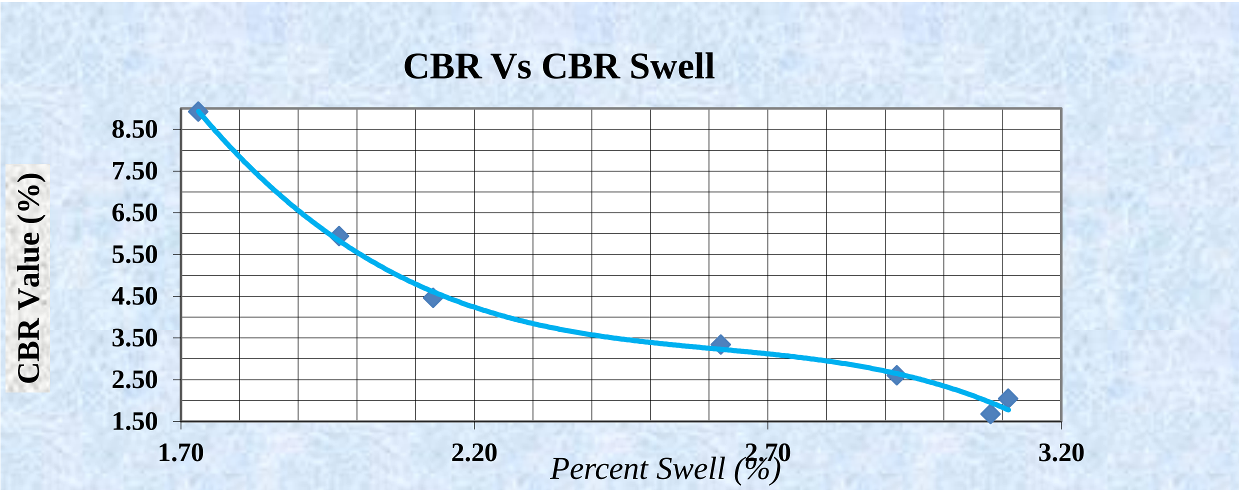


Figure 5-16 CBR VS Percent Swell for sample at 32+700

The previous CBR Value was 1.46% which is considered as poor subgrade in accordance with ERA Pavement design manual. But through lime treatment the CBR of the soil has improved but, the improvement was not to the required level. The addition of 12% lime has changed the CBR Value to 8.92% which is still a fair subgrade to be used on the specific project. CBR classes of 20-50% for any subgrade are specified as Good CBR values in most literatures.

5.6.5. Effects of lime on Dry density and CBR

The results MDD and OMC with the addition of lime are tabulated in table 5-2 above and are illustrated in figure 5-17 below for sample at 32+700.

The CBR Value increases with the increase of lime with the opposite effect on the dry density. The CBR Value has also increased with the increase in lime Percentage. The effect of lime on CBR and dry density are opposite but with positive outcome after stabilizations.

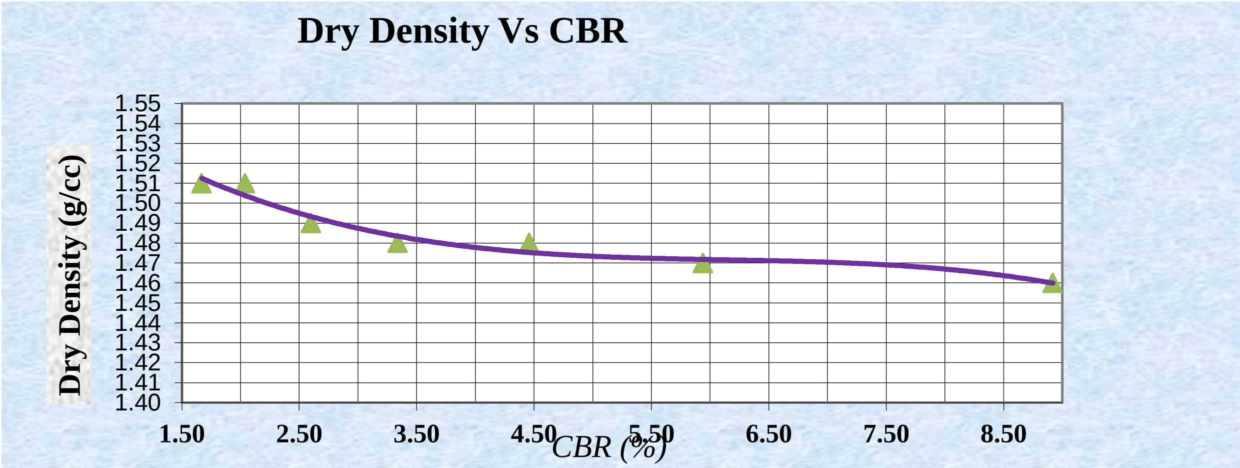


Figure 5-17 Dry Density Vs CBR for sample at 32+700

The dry density of the samples should be controlled as it is one of the parameter to be controlled in order to control the Swell of the expansive soils. The expansive nature of problematic soils can be controlled through compaction. In CBR determinations, the blows was increased to 65 because the CBR value will be determined on nearly or exactly at the maximum dry density and optimum moisture content.

5.7. Effect of Lime on CBR at 2.54mm and 5.08mm Readings

The results CBR with the addition of lime are tabulated in table 4-5 above and are illustrated in figure 5-18 and 5-19 below for sample at 32+700.

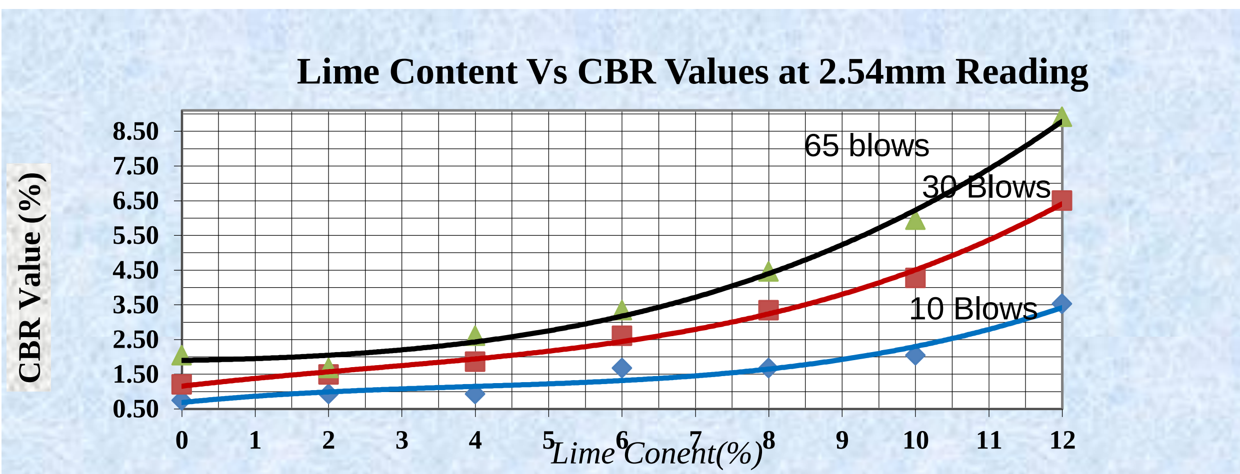


Figure 5-18 Lime Content Vs CBR Values at 2.54mm reading for sample at 32+700

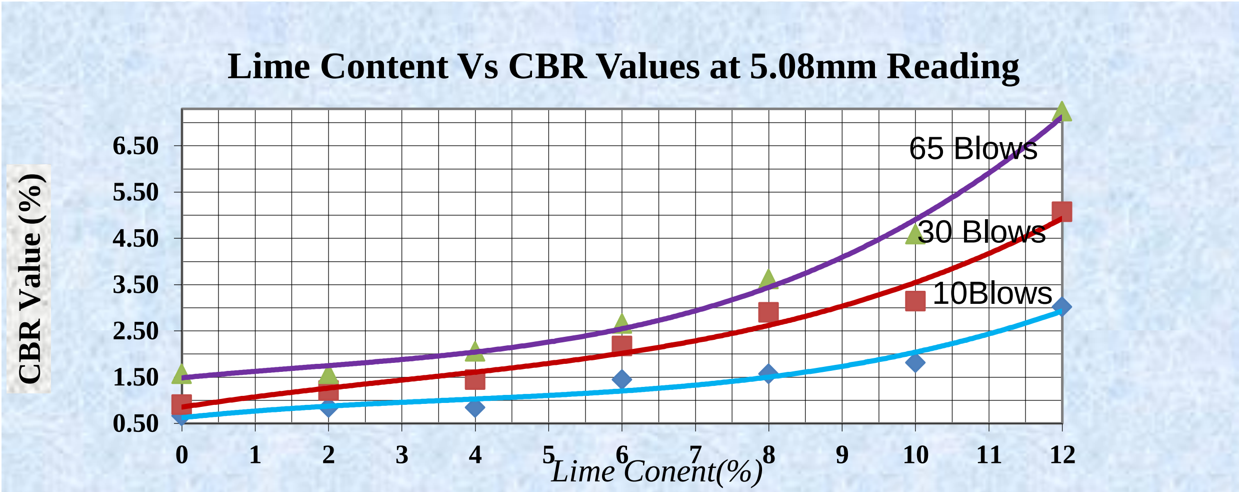


Figure 5-19 Lime content Vs CBR Values at 5.08mm for sample at 32+700

The effect of lime on CBR has a positive effect up on lime treatment with the graphs illustrated in figure 5-19 and 5-20 below. The CBR values of the treated soils have increased with the addition of lime. The CBR Values of the treated soil has also increased with increase in lime percentages. The CBR values for natural soil have improved with values of 1.3% at 2.54mm and 0.91 at 5.08mm readings respectively. These results show that there is a little improvement in CBR with an increase of comp active efforts.

The laboratory results obtained at 2.54mm reading has a higher value than the CBR results obtained at 5.08mm reading. These results comply with most of the researches conducted. In practice the CBR value obtained at 2.54mm are considered as the design CBR value. The values obtained at 2.54mm reading are considered as the Design CBR value for this research.

5.8. Effect of lime on Unconfined Compressive strength

The summary of laboratory results of the UCS with the addition of lime are tabulated in table 4-6 above and are illustrated in figure 5-20 and 5-21 below for sample at 32+700 and Blended samples respectively. The tests and results of the unconfined compressive strength before and after the addition of lime are attached in Appendix VI.

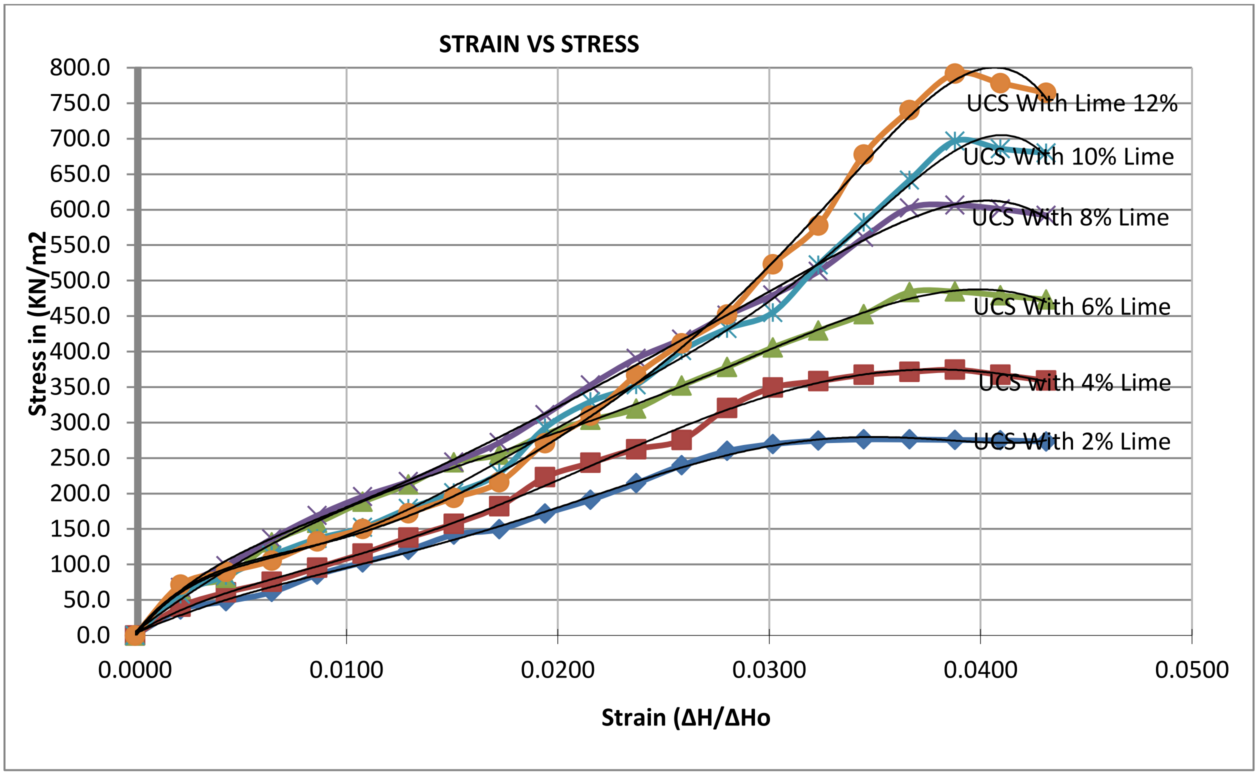


Figure 5-20 Combined Strain Vs Stress for samples at station 32+700 (cured for 7 days)

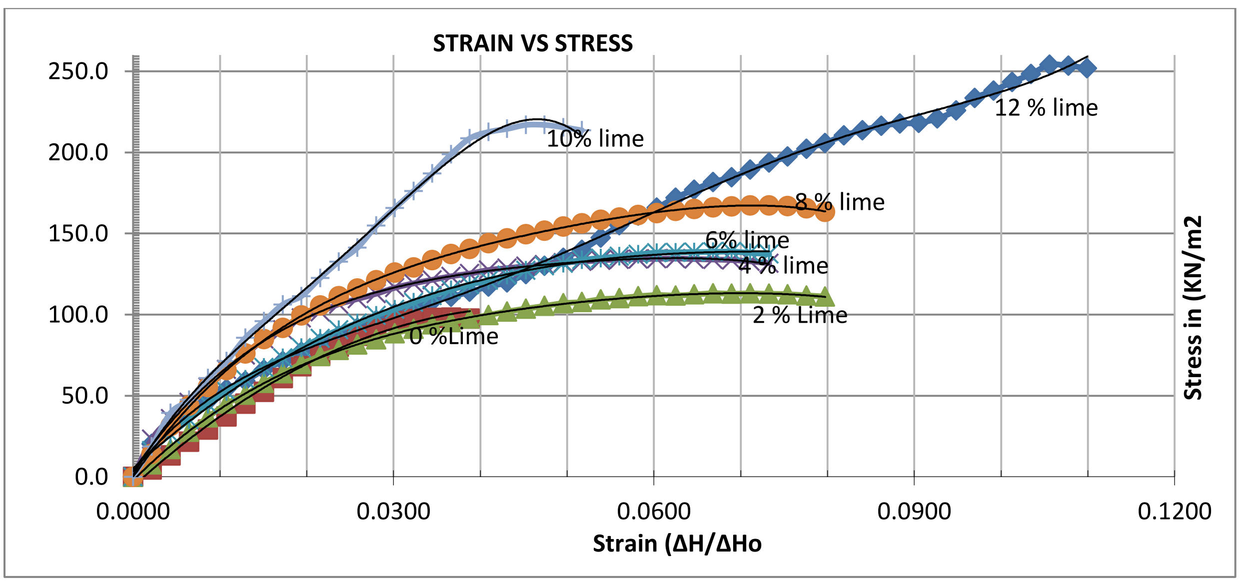


Figure 5-21 Combined chart for strain Vs Stress for Blended sample for uncured condition

From figures 5-20 and figures 5-21, the effect of lime on the unconfined compressive strength has a positive impact up on addition of lime for the immediate and 7 days cured samples. As can be seen clearly the UCS value has increased up on the treatment of lime. The UCS values have also increased with the increase of lime percentage.

From different literatures curing of the lime stabilized samples will increase the unconfined compressive strength of expansive soils. Different samples collected at 32+700 and the blended samples were tested for 7 days cured and immediately tested respectively.

It is difficult to compare different samples with one criterion with different test procedures. It was not possible to conduct similar tests on the same sample because of the limitations in sample size and economy as well.

The cured sample has a UCS value of 276.21kpa with 2% lime treatment. Similarly the UCS value for blended sample was 113.12kpa with 2% lime. The maximum values of UCS for sample at 32+700 and Blended samples with the addition of 12% lime were 791.73% and 254.03% respectively.

From table 2-4 the minimum unconfined compressive strength and pavement layers are related. From the table, the minimum UCS value for stabilized subgrade is specified to be 1725kpa. The values obtained for samples at 32+700 and blended samples are below the requirements and can be said that strength was not achieved for these samples (locations) up to 12% lime.

5.9. Mixture Design

Steps for mixture design and testing

Most fine grained soils can be effectively stabilized with 3 to 10 percent lime on (dry unit basis of soil).

Step1: State the method of preparing the materials and Laboratory tests procedures

The samples from station 3+000, 23+450, 30+750, 32+700 and 35+000 are collected in and are prepared for treatment in accordance with AASHTO T 87-86 (2000). As explained in methodology the tests conducted for samples at 32+700 and blended samples were outlined. To apply the mixture design criteria’s, sample at 32+700 will be used because most important tests related to my objectives are conducted to this specific sample. The results of the laboratory tests are tabulated in table 4-1 table 4-4 and 4-5 in chapter four and are refined in table 5-3 below.

Table 5-3 Summary of laboratory results for sample at 32+700

Serial Number	Soil Type	Plasticity index (%)	CBR Value (%)	CBR Swell (%)	OMC (%)	MDD
1	Natural Soil	38.24	1.48	3.11	25.00	1.51
2	Soil+2% lime	34.29	1.54	3.08	25.10	1.51
3	Soil+4% lime	32.91	2.08	2.92	26.00	1.49
4	Soil+6% lime	29.8	2.82	2.62	27.30	1.48
5	Soil+8% lime	25.67	3.63	2.13	28.20	1.48
6	Soil+10% lime	21.67	4.11	1.97	28.50	1.47
7	Soil+12% lime	15.1	6.44	1.73	29.30	1.46

Step 2: Set the mixture design Criteria.

From soils and materials investigation report reported on October 2006, subgrade samples were collected almost at every one kilometers of the route and different tests were conducted with summaries of the results as explained below:

- 57% percent of the samples along the route fall in A-7-5 and 43% as A-7-6 material classification class
- The CBR values of the materials in A-7-5 fall in the range of 2.2 to 4.94 percent
- The swelling of the CBR Tested samples varies between 2.33% to 5.44%.
- The shrinkage limit of the samples varies from 9 to 17 percent.
- The A-7-5 materials are poor for subgrade construction as it covers 57% of the whole project length.
- The use of borrow and blanketing thicker materials are recommended as a solution for this poor subgrade
- There is also a scarcity of borrow material on this alignment

From the soil and material investigation reports and laboratory test results, the thesis will concentrate on improving the CBR Strength, reduction of CBR Swell and or the reduction of the plasticity index to achieve the goal.

Therefore the criteria for this thesis are

- Reduction of the plasticity index
- Improvement of the CBR and CBR Swell.

Step 3: Determine the Design Lime Content for the specific Design Criteria's.

The results of Atterberg limits and CBR Values before and after stabilization are summarized in table 5-4 below

Table 5-4 Summary of laboratory results for sample at 32+700

ID	Soil Type	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	OMC (%)	MDD	CBR Values (%)	CBR Swell (%)
1	Natural Soil	64.14	31.03	33.11	25	1.51	2.04	3.11
2	Soil+2% lime	60.33	26.0	34.29	25.1	1.51	1.67	3.08
3	Soil+4% lime	59.16	26.25	32.91	26.0	1.49	2.60	2.92
4	Soil+6% lime	56.07	26.27	29.8	27.30	1.48	3.34	2.62
5	Soil+8% lime	52.48	26.81	25.67	28.20	1.48	4.46	2.13
6	Soil+10% lime	50.50	28.83	21.67	28.50	1.47	5.94	1.97
7	Soil+12% lime	45.66	30.56	15.10	29.30	1.46	8.92	1.73

From table 5-4, it can be seen that the plasticity index of the sample at 32+700 was 33.11. With the addition of 12% lime the plasticity index value becomes 15.10%. The plasticity nature of the sample (32+700) has improved greatly from its nature of higher swelling potential range (20-55%) to a lower swelling potential range (0-15%)

From the soil and materials investigation report, it is stated that the A-7-5 materials are poor for road bed that needs thicker embankment material to avoid the detrimental effects of the volume change.

According to ERA pavement design manual volume 1 (Flexible pavements and gravel roads) chapter three explains details concerning subgrade materials. According to the manual the strength of the Subgrade soil is assessed by the type of soil, its density and moisture content.

For subgrades with CBR value less than 2 needs special treatment. In our case stabilization is one of the options being used. According to ERA manual subgrades are classified from S1 to S6 based on the California bearing ratio (CBR), and are illustrated in table below.

Table 5-5 CBR range subgrade class (ERA 2002 Pavement manual)

Serial No.	Classs	CBR% Range
1	S1	2
2	S2	3-4
3	S3	5-7
4	S4	8-14
5	S5	15-29
6	S6	30+

According to the soil and materials investigation report, sections of the route with CBR>3.5% and swell of about 2% can be used for Embankment construction which needs to be covered with blanketing material.

Before stabilization of the sample at station 32+700, the CBR value was 2.04% and Swell of 3.11 which is an indication of how poor this subgrade material is for subgrade construction. But by stabilizing the sample its CBR and CBR swell are improved slowly depending on the amount of lime used. For example with the addition of 4% lime the CBR Value becomes 2.6%, and swell of 2.92 which falls in S1. The value of CBR with lime stabilization of 12% was 8.92% which is a good improvement for subgrade class lying in S4 (8-14).

From Bowls, 1992 CBR values and the quality of subgrades in pavement design are explained in the table 5-6 below.

Table 5-6 CBR range Subgrade quality (Bowls, 1992)

Serial Number	CBR (%) Range	Subgrade Quality
1	0-3	Very poor subgrade
2	3-7	Poor to fair subgrade
3	7-20	Fair subgrade
4	20-50	Good subgrade
5	50+	Excellent subgrde

From table 5-6 above,subgrades having a CBR values in the range of 0-7% are very poor and poor to fair. They are considered as unsuitable subgrades and needs to be treated especially in terms of pavement applications. For our case since the route is gravel wearing coarse it can be dependent for the ranges depicted in fair subgrade to avoid high embankments and sealing blankets.

The other criteria to be investigated were the CBR swell. The Maximum allowable CBR swell in the project was 2%. Before lime treatment the swell was 3.11%.with the addition of 12% lime, the swell of the soil becomes 1.73% which is in acceptable range and specification for the project.

Therefore the design lime content for this section of the route with all the three criteria(Acceptable CBR strength and swell with required plasticity index reductions) is estimated to be 12% lime, which is reasonably good and safe for the roadbed.

Advantages of stabilizing the Subgrade for the pavement design

From pavement design report made by core consulting engineers plc. (2006G.C), the following main points were explained

a) Sub grade material to have a CBR strength of at least 3.5% and swell of 2%
b) The project road is going to be constructed on RR-50 standard
c) Kenya design manual is followed because the ERA manual does not consider Black cotton soils.
d) For RR-50 the AADT will be in the range of 15-50 veh/day
e) They have used an improved subgrade material with minimum CBR value of 8% and swell of <1 to use it as an improved subgrade

From pavement design report (Core Consulting Engineers plc-2006) again, the design CBR of 8% and AADT of 15-50 Veh/day, the thickness of the required minimum gravel wearing thickness was determined to be 225mm.Expecting the annual gravel loss of 10mm/annum and assuming re-gravelling every two years, the total gravel loss is expected to be 20mm. then the total gravel wearing thickness becomes 245mm which is almost 250mm.

It was mentioned in the report that the CBR of most of the project length were below 2%, and, the designer has specified to use an improved subgrade with minimum CBR value of 8% and swell of <1%.In order to achieve this criteria (CBR>8% and Swell<1%), the designer has proposed an improved subgrade material from borrows to design the pavement layer thickness..

The use of lime stabilization using 12% lime has improved the CBR value from 1.46% to 8.92%, which was one of the required specifications in the project for improved subgrade. This can imply that the stabilized soil can totally avoids the use of improved subgrade material.

From the previous conditions of the project (Adura-Burbey DS6 road), the problems of the expansive subgrades were mitigated by using Borrow materials of CBR> 3.5% and blanketing material of CBR value of> 8%. These specifications were additional costs for the project. For example they have specified to use a blanketing material of 60cm and an embankment material of 1m to 2.4m which is too costly and can took prolonged construction period.

From this thesis it is evident that stabilization of the problematic soil with 12% lime has

- Greatly improved the plasticity nature from 33.11% to 15.1%
- Improved the CBR strength from 1.46% to 8.92%
- Has improved the workability
- Has improved the CBR swell from 5.49% to 1.73%

Though mathematical calculations related to costs were not made to clearly identify the effects, one can easily identify the use of lime stabilization can reduce the unnecessary costs related to embankment and blanketing materials to mitigate the problematic soils. Therefore it is advantageous to use lime stabilization rather than utilizing thicker embankments and blanketing material, which are by far costly and time consuming.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1. Conclusions

In this study, the suitability of lime as stabilizers for swelling potential of an expansive soil was studied. Lime was added to Samples to indent the improvements. Based on the interpretations of the results the following conclusions are made

1. The addition of lime to the tested samples led to the reduction in liquid limit, plasticity index and an increase in plastic limit of the soil sample at 32+700.
2. The plastic limit of the blended samples has not shown a clear trend as it increases from 0 to 2% and starts decreasing from 2% to 12%.
3. Both samples have shown a reduction in plasticity index with the addition of lime. And the plasticity index of both samples has improved from its high swelling potentials to low swelling potentials with the addition of 12% lime.
4. The reduction in plasticity limit is an indication of a marked increase in workability which in turn expedites manipulation and placement of the treated soil. Workability is improved because flocculation makes the clay more friable; this assists combination for effective mixing and compaction
5. The addition of lime has resulted in an increase of the shrinkage limit of the soil
6. The addition of lime has resulted in a reduction of the free swell of the soil. As the percentage of stabilizer increased, free swell ratio decreased.
7. The addition of lime has a positive effect in increasing the specific gravity of the soil samples
8. The addition of lime for the studied soil has resulted in an increase in optimum moisture content and reduction of maximum dry density for the same compaction effort.
9. The optimum lime content in improving the CBR of the soil from its poor subgrade quality to poor to fair class is found out as 12 percent.
10. The addition of lime has resulted in an increase of the unconfined compressive strength of the tested samples
11. The addition of optimum lime content has resulted in an improvement of the overall performance of the subgrade by increasing its strength and other workability criteria's for the project.

12. By addition of lime, the swelling percentage decreased considerably. The reduction was higher for lime added samples having more lime content.

13. The addition of lime up to 12% does not bring a significant improvement in California bearing ratio which falls in the range of 7-20 percent. But the achievement in Improving the subgrade quality is cost effective because this will reduce the use of borrow materials on the project

6.2. Recommendations

From the findings of this research, the use of lime as a stabilizer for the area is one of the best options to be implemented in the road construction. From soil and material investigation report of the Adura-Burbey project, the construction works are specified to use a borrow sources as the subgrade material. It is also mentioned that most of the borrow materials are marginal to use as a subgrade material. Stabilizing the soil sample has increased the subgrade CBR from very poor subgrade to fair subgrade. Stabilization of the subgrade soil with lime has improved the swelling potential and decreased the plasticity nature of the soil sample. It is therefore, highly recommended to use lime stabilization for the effective construction and cost minimization of the project.

In addition the following further research topics are recommended

1. **The optimum lime content on actual field conditions based on the design lime content in the laboratory should be investigated.**

The use of lime in the laboratory and actual field conditions are not same. There are different factors which are different from the laboratory conditions like atmospheric conditions and ineffective usage of the specified lime percentages. Thus the actual optimum lime content on site should be investigated and analyzed.

2. **The method of curing on site should be studied.**

Curing methods in the laboratory are quite different from the curing methods on site. These phenomena's should thoroughly investigated and recommended.

3. **Effects lime treated subgrades in pavement thickness reduction in flexible pavement design should thoroughly be assed.**

In a flexible pavement design, the subgrade type and class is one of the parameters which contribute for the pavement thickness. As the subgrade strength increases the thickness required for pavement will be reduced. Therefore, the effects of strength increase due to lime stabilization on pavement thickness should thoroughly be studied.

4. **The uncured and cured strengths of the unconfined compressive strength of lime treated subgrades at 3 days,7 days, 14 days and 28 days should be investigated for continuous road alignment .**

The unconfined compressive strength of the subgrade soils shows a wide variation before and after curing of the sample. As the construction of the road time taking and the environmental conditions are variable, the subgrade soil should be studied both the cured and uncured conditions of real scenarios.

5. **The effects of lime treated subgrade soils in rigid pavement design should be considered.**

There are two types of pavement design approaches. Rigid pavement is one of the design approaches in pavement structures. As it is already found out lime treatment has increased the strength of the treated soils. Thus the effects of lime treated subgrade soils in flexible pavement design should be studied.

6. **The durability of lime treated mixtures should be studied.**

7. **The mixture design of lime treated subgrades should thoroughly be investigated to come up with a results and guidance in the country**

Lime stabilization of expansive subgrades throughout the country should be conducted and the mixture designs should be formulated. With the necessary investigations and results guidance can be developed for the country to mitigate serious hazards due to highly expansive soils.

8. **Effects of curing on moisture density relationships**

if a mixture of lime and soil allowed to cure and gain strength prior to compaction, further reduction in dry density and an increase in optimum moisture content may be observed as compared with the immediate moisture density relationships. Thus the effects of curing on moisture density relationships should be studied.

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ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

M.Sc. Thesis on

Stabilization of expansive soils with lime

(A Case Study on the Adura-Burbey DS6 Road Segment)

By

Reshid Musema Umer

Addis Ababa Institute of Technology

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