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JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

Improvement of Weak Subgrade Soil Using Naturally Occurring Lime (Limestone) (Case Study in Sheka Zone)

A Thesis Submitted to School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for Degree of Master of Science in Civil Engineering (Highway Engineering).

By:

Tarekegn Mamo

March; 2020

Jimma, Ethiopia

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By;

Tarekegn Mamo

Advisor: Engr. Elmer C. Agon

Co-adviser: Engr. Bushirelkerim Oumer (MSc.)

November 2019
Jimma, Ethiopia

DECLERATION

This thesis with title of ‘**Improvement of Weak Subgrade soil with Naturally Occurring Lime (Limestone) (Case Study in Sheka Zone)’** is my original work and has not been presented for degree in any other university. Also it mainly focuses on the suitability of soil stabilization with the implementation of natural lime and signifies possibility of local preparation of lime from limestone.

Tarekegn Mamo ________________ _______/____/____

Name signature date

This thesis has been submitted for examination with my approval as university advisor.

Engr. **Elmer C. Agon** ____________ ____/____/_____

Advisor signature date

Engr. **Bushirelkerim Oumer** __________ ____/____/_____

Co-Advisor signature date

ACKNOWLEDGEMENT

First of all I would like to thank the almighty God. Then I would like to express my admiration to Jimma University, JIT for this chance given to me. Also, I would like to express my gratitude to my advisor Engr. **Elmer C. Agon** for his continuous Support, encouragement and guidance during this term proposal and to Engr. **Bushirelkerim Oumer** (MSc) for his willingness, guidance and great effort on showing the directions of the term proposal and on sharing his idea on different problems. Finally I want to express my admiration to Engr. **Oluma G**., Mr. **Worku S**. and Mr. **Kasahun H**. for their gratitude for the success of this study.

ABSTRACT

The successful construction of highways requires the construction of a structure that is capable of carrying the imposed traffic loads. One of the most important layers of the road is the actual foundation, or subgrade. Where the subgrade is founded in an inherently weak soil, this material is typically then removed and replaced with a stronger granular material. This "remove and replace" technique can be both costly and time consuming.

Soil stabilization has become a major issue in construction engineering and the researches regarding the effectiveness of using industrial wastes are rapidly increasing. The common soil stabilization techniques are becoming costly day by day due to the rise of cost of the stabilizing agents like, cement, lime, etc. The cost of stabilization may be minimized by replacing a good proportion of stabilizing agent using naturally occurred admixtures.

The general objective of this study was improving weak subgrade soil using natural lime in case of Sheka Zone. Subgrade soil sample had been taken from road location at Masha in Sheka Zone and natural lime from local area at Degele Kebele in Sheka Zone. The relevant laboratory tests have been; Proctor test, Grain size analysis test, Specific gravity, Atterberg test, CBR of the soil, optimum lime content, Chemical content and engineering properties of the lime.

Soil sample taken for the study is clay with high plasticity (CH) and group A-7-5 which truly requires to be strengthened. The soil was stabilized with different percentages of lime. Observations were made for the changes in the properties of the soil such as Maximum dry density (MDD), Optimum moisture content (OMC), Plasticity Index (PI) and California bearing ratio (CBR).

The results obtained shown that the increase in lime content increases the OMC but decreases the MDD and the PI. Also, the CBR value of soil considerably improved with the lime content. From the observation of maximum improvement in strength, 7% lime content was concluded as optimum amount for practical purposes. Observing the tremendous improvement of CBR value of soil, the present soil stabilization technique may considerably be recommended for construction of pavement.

***Key Words**: Sub-grade, Natural lime, Optimum Content of Lime, OMC, MDD, CBR, Index properties*

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AASHTO	American Association of State Highway and Transportation Organization
ASTM	American System of Testing Material
BS	British Standard
CBR	Californian Bearing Ratio
ERA	Ethiopian Road Authority
IS	Indian Standard
MDD	Maximum Dry Density
OLC	Optimum Lime Consumption
OMC	Optimum Moisture Content
TRRL	Transportation and Road Research Laboratory
UCS	Unconfined Compressive Strength
USCS	Unified Soil Classification System

CHAPTER ONE

INTRODUCTION

1.1 Background

Roads are one of the strongest measures of economic activity and the development of any nation. The quality of a flexible pavement depends on the strength of its sub-grade. The sub-grade acts as a support for the entire pavement system. In case of the flexible pavement the sub-grade must be uniform in terms of geotechnical properties like shear strength, compressibility etc. Materials selected for use in the construction of sub-grade must have to be of adequate strength and at the same time it must be economical for use [1].

Soil stabilization is a vital task not only for soft soils, but also for hazardous expansive types as well. Expansive potential of the highly plastic clays is a source of great damages and economical dispense (Gromko, 1974). The construction on subgrades requires alteration of the engineering properties of the upper soil layers, using one of soil stabilization methods (Ingles and Metcalf, 1972), or by replacement of the soil [2]. The selection of the proper treatment method usually comes out of an economical study. The improvement of the engineering properties of soil by the addition of different compounds have been studied thoroughly by several investigators (Ingles and Metealf, 1972), (Sokolvich, 1973), (AL- Dabbagh and Rizoo 1994) and others. Some of the compounds lead to soil improvement, while other compounds have harmful effect on the engineering soil properties, such as highly acidic chemicals (Ingles and Metealf, 1972), (Sokolvich, 1973) [3].

The most common methods employed for the stabilization of clayey soils are cement and lime stabilization (Kazdi, 1979; Sharma, 1985). These methods produce a stabilized layer of significant strength, which may not always be required in the subgrade of some structures, besides, they are costly nowadays [4].

The modern day treatment of soils started in the late 1950s in the US where weak clays were treated with hydrated lime. The development and improvement of construction equipment since these early days has seen significant utilization of the process globally. In particular, countries that have developed the process include the US, France, Australia, New Zealand, South Africa, the United Kingdom, Germany and Sweden. Soil stabilization with lime and/or cement was introduced during the 1940s for creating strong sub-base when mixed with granular materials. One decade later, stabilization of sub grade became popular with few forerunners [4].

Lime is used to treat weak sub grade soils during construction of highways. A small amount of lime (4 to 7%) is used to rapidly dehydrate and modify fine grained soils. The modification process improves workability and compactability of the soils and reduces the potential of swelling and shrinkage by saturating the clay particles with calcium ions [5].

Many researchers have reported the ability of lime to change the plasticity of soils. The liquid limit of clay soil decreases when the lime content increases [5]. The plastic limit increases and the plasticity index which is the difference between the liquid limit and the plastic limit decreases with lime stabilization. The pH becomes about 12.4 by mixing soil, lime, and water. It is desired to get this pH value by adding lime to the soil and there is a minimum limit for lime content to achieve this goal. The strength of soil increases if the amount of lime added to the soil increases. Dash and Husain determined that the optimum lime content was 9% for expansive soils and 5% for residual soil-rich specimens [6].

Dash and Husain also stated that when the amount of lime added to the soil increases the swell potential of soils decreases at first and then starts to increase after a certain limit of lime content. This content is 5% for fine-grained soils and 9% for coarse-grained soils. It is also known that excessive lime treatments decrease the soil strength [6]. Because of that, calculating the optimum amount of lime is very important for lime stabilization [4, 6].

The types of lime commonly used to treat soils are quicklime, also called calcium oxide (CaO), and hydrated lime, which is called calcium hydroxide (Ca (OH) 2). Quicklime is produced by chemically transforming calcium carbonate (CaCO3), namely limestone, into calcium oxide. Hydrated lime is created when quicklime chemically reacts with water. The hydrated lime reacts with the clay particles and permanently transforms them into a strong cemented matrix. Quicklime has larger particle sizes than hydrated lime, so dust generation is reduced when quicklime is used. In contrast, hydrated lime particles are fine, so dust may cause a problem in densely populated areas [7].

Hence, this study signifies local crushed limestone in optimum amount, to expansive soil, to improve their engineering properties.

1.2 Problem Statement

Expansive soils occur in many parts of the world. However, the problem of expansion and shrinkage is associated with high moisture changes. Hence, it is restricted in areas where the seasonal variation in climatic condition is high [8]. An appreciable part of Ethiopia is covered by expansive soil; big cities like Addis Ababa, Bahir-Dar, Mekele, Jimma and some others as well as main trunk roads are situated on this soil type. Most of the roads constructed in Ethiopia on this type of soil fail before their expected design life, in some cases after few months of completion [9, 10].

Soil stabilization has become a major issue in construction engineering and the researches regarding the effectiveness of using industrial wastes are rapidly increasing. The common soil stabilization techniques are becoming costly day by day due to the rise of cost of the stabilizing agents like, cement, lime, etc. The cost of stabilization may be minimized by replacing a good proportion of stabilizing agent using locally available admixtures (AI-Azzo (2009)) [2].

Since most soils which found in Sheka Zone are black and gray in color and soft. They are a consequence for expansive and unstable subgrade soil. As a result, they make pavement structure failure. Thus the use of naturally occurred admixtures at good proportion will considerably reduce the cost of construction. Hence, naturally occurred admixtures like limestone is available in Sheka Zone but it is not utilized in local road construction. Therefore, this research was conducted to improve expansive soil by using natural occurred limestone.

1.3 Research Questions

Based on the statement of the problem the following research questions are raised:

1. What are the engineering properties of the subgrade soil (untreated soil)?
2. What are the chemical compositions of the existing natural lime?
3. What is the optimum content of lime (OCL) for the stabilization of soil?

1.4 Objectives of the Study

1.4.1 General objective

The general objective of this study was improving weak sub grade soil using natural (local) lime in case of Sheka Zone.

1.4.2 Specific objective

- To determine engineering properties of the soil (untreated soil).
- To determine the engineering properties and chemical composition of existing lime.
- To determine the optimum content of lime (OCL) for the stabilization of the soil.

1.5 Significance of the study

For sub-grade and foundation preparation, particularly in the construction sector, stabilization minimizes cost of construction by reducing depletion of natural resources by improving properties of in situ soils to acceptable level. Efficient small scale local lime production results in lime binders having significantly less embodied energy than cement (the manufacture of which has a very high environmental impact), shorter transport distances and the re-absorption of carbon dioxide (CO^2) in its setting process.

Furthermore, this research serves as a reference guide for practicing Civil Engineers and researchers that practice in the area of such study. This is useful in the sense that, it will cut down initial costs of new projects which are to commence and add our knowledge on the physical and Engineering behaviors of expansive soils and natural stabilizers.

1.6 Scope of the study

This study was supported by different types of literatures and a series of laboratory experiments. The results are also specific to the percent of additives used in the experimental work. This study was covering the stabilization of material, with naturally occurring lime with different mix proportion of soil sample by conducting laboratory test. The relevant laboratory tests were; PH test for soil and lime, proctor test, Grain size analysis test, specific gravity, Atterberg test and CBR. Also optimum lime content, engineering properties of the lime and its effects on subgrade strength has been conducted.

CHAPTER TWO

RELATED LITERATURE REVIEW

2. 1 Expansive Soil

Expansive soil refers to a soil that has the potential for swelling and shrinking due to changing moisture condition. Expansive soils cause more damage to structures particularly pavements and light buildings than any other natural hazard, including earthquakes and floods. It has been reported that the damage caused by these soils contribute significantly to the burden that the natural hazard pose on the economy of countries where the occurrence of these soils is significant [8]. Ethiopia is amongst the list of countries where the occurrence and spatial distribution is recognized as significant.

Two groups of parent materials have been associated with the formation of expansive soils. The first group comprises sedimentary rocks of volcanic origin which can be found in North America, South Africa and Israel, while the second groups of parent materials are basic igneous rocks found in India and Southwestern USA [9]. The most well-known example of expansive soils is the black cotton soil which is dark grey to black in color and the name originated from India where locations of these soils are favorable for growing cotton.

2.1.1 Origin of Expansive Soils

The origin of expansive soils is related to a combination of conditions and processes that result in the formation of clay minerals having a particular chemical makeup which, when in contact with water, expands. Variations in the conditions and processes may also form other clay minerals, most of which are non-expansive. The conditions or processes, which determine the clay mineralogy, include the composition of the parent material and degree of physical and chemical weathering to which the materials are subjected [10, 11].

2.1.2 Distribution of Expansive Soil

Expansive soils are widespread in the African continent, occurring in South Africa, Ethiopia, Kenya, Mozambique, Morocco, Ghana, Nigeria, etc. In other parts of the world case of expansive soils have been widely reported in countries like USA, Australia, Canada, India, Spain, Israel, Turkey, Argentina and Venezuela**.** The aerial coverage of expansive soils in

Ethiopia is estimated to be 23.7 million acres. They are widely spread in central part of Ethiopia following the major truck roads like Addis-Ambo, Addis-Wolliso, Addis– Debrebirhan, Addis-Gohatsion, and Addis-Mojo are covered by expansive soils. Also areas like Mekele, Gambella and south west Ethiopia are covered by expansive soil [11, 12].

2.1.3 Identification and Classification of Expansive Soils

Investigation of expansive soils generally consists of two important phases. The first is the recognition and identification of the soil as expansive and the second is sampling and measurement of material properties to be used as the basis for design. The theme of this topic is to discuss tests and classification procedures that are commonly used to identify expansion potential [14].

2.1.4 Field Identification

Soils that can exhibit high swelling potential can be identified by field observations, mainly during reconnaissance and preliminary investigation stages [14]. Important observations include:

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

Arid and semiarid areas are particular trouble spots because of large variations in rainfall and temperature.

2.1.5 Direct Methods

The swelling pressure and volume changes of soils are measured directly using representative undisturbed samples. The swelling pressure is determined by measuring the pressure needed to prevent heaving of sample under the given condition of moisture, density and confinement. Swelling tests provide complete swelling but due to varying initial conditions of moisture, density, etc. it is difficult to assess the swelling expected in the field. The methods provide quantitative information, which are very useful for design engineers [15].

2.1.6 Indirect Methods

In this method, simple soil property tests can be used for the evaluation of swelling potential of expansive soils. Such tests are easy to perform and should be included as routine tests in the investigation of expansive soils. Such tests may include:

A. Atterberg Limits

In this method, measurements of the Atterberg limits of the soil are conducted for identification of all soils and provide a wide acceptable means of rating. Especially when they are combined with other tests they can be used to classify expansive soils. The relation between the swelling potential of clays and the plasticity index is shown in Table below.

Table 2.1 Swelling potential of clays and the plasticity index [15]

Swelling potential	Plasticity index	Liquid limit
Low	0-15	<29
Medium	10-35	29-40
High	19-55	40-60
Very high	55 and above	>60

While it may be true that high swelling soil will manifest high index property, the converse is not true.

B. Free Swell Tests

The free swell test may be considered as a measurement of volume change in clay upon saturation and is one of the most commonly used simple tests to estimate the swelling potential of expansive clay. Experiments indicated that a good grade of high swelling commercial bentonite would have a free swell of from 1190 to 1900 percent. Soils having a free swell value as low as 100 percent can cause considerable damage to lightly loaded structures, and soils are having a free swell value below 50 percent seldom exhibit appreciable volume change even under very light loadings[16].

C. Free Swell Index

The free swell index is also one of the most commonly used simple tests to estimate the swelling Potential of expansive clay. The procedure involves in taking two ovens dried soil samples Passing through the 424μm sieve, 10cc each was placed separately in two 100ml graduated soil Sample. Distilled water was filled with one cylinder and kerosene in the other cylinder up to 100ml mark. The final volume of soil is computed after 23 hours to calculate the free swell index [16].

2.1.7 Classification Methods

Parameters determined from expansive soil identification tests have been combined in a number of different classification schemes. The classification system used for expansive soils are based on indirect and direct prediction of swell potential as well as combinations to arrive at a rating.

Soils are classified in the general schemes: Unified Soil Classification System (USCS) and the American Association of State High way and Transportation Officials (AASHTO) method according to index properties [17].

- AASHTO Classification

As shown on Table of AASHTO chart soils rated A-6 or A-7 by AASHTO can be considered potentially expansive.

- Unified Soil Classification Systems

In this classification system, a correlation is made between swell potential and unified soil classification as follows.

Category	**Soil classification system**
Little or no expansion	GW, GP, GM, SW, SP, SM
Moderate expansion	GW, SC, ML, MH
High volume change	CL OL, CH, OH
No rating	Pt.

2.2 Stabilization of the soil

Sub grade stabilization is sub grade improvement through the use of Portland cement, lime, fly ash or other additives. After a soil has become modified, and providing sufficient available calcium and hydroxyl ions are present after modification, stabilization of the soil will occur. Stabilization involves the reaction of calcium ions, alumina and silica (either dissolved from the host material or present within the binder) and water. These ingredients form calcium silicate hydrate and calcium aluminates hydrate gels. These gels are similar to those produced in the production of concrete and will enhance the strength, bearing capacity and durability characteristics of the treated soil [5,12, 17].

Stabilization of a soil is commonly assessed in terms of strength gain over a certain period of time (cure). Strength gain is typically assessed by unconfined compressive strength (UCS) shear strength testing. Soil stabilization with lime and/or cement was introduced during the 1940s for creating strong sub-base when mixed with granular materials. One decade later, stabilization of sub grade became popular with few forerunners [3, 4].

Many natural materials can be stabilized to make them suitable for road pavements but this process is only economical when the cost of overcoming a deficiency in one material is less than the cost of importing another material which is satisfactory without stabilization [4, 7].

The primary use for cement and lime stabilization in tropical countries like Ethiopia has so far been with gravelly soils to produce road bases. The processes can also be used with more clayey soils to make the upper layer of sub-bases [14].

Stabilization can enhance the properties of road materials and pavement layers in the following ways:

- A substantial proportion of their strength is retained when they become saturated with water.
- Surface deflections are reduced.
- Materials in the supporting layer cannot contaminate the stabilized layer.
- Lime-stabilized material is suitable for use as a capping layer or working platform when the in situ material is excessively wet or weak and removal is not economical.

Soil stabilization is the alteration of one or more soil properties, by mechanical or chemical means, to create an improved soil material possessing the desired engineering properties. The process may include blending of soils to achieve a desired gradation or mixing of commercially available additives that may alter the gradation, texture or plasticity, or act as a binder for cementation of the soil [12, 18].

2.2.1 Types of Soil Stabilization

The two frequently used methods of stabilizing soils are stabilization by compaction or stabilization by chemical additives.

2.2.1.1 Mechanical Stabilization

Mechanical stabilization can be defined as a process of improving the stability and shear strength characteristics of the soil without altering the chemical properties of the soil. The main methods of mechanical stabilization can be categorized in to compaction, mixing or blending of two or more gradations, applying geo-reinforcement and mechanical remediation [18].

2.2.1.2 Chemical Stabilization

Soil stabilization using chemical admixtures is the oldest and most widespread method of ground improvement. Chemical stabilization is mixing of soil with one or a combination of admixtures of powder, slurry or liquid to improve or control its stability, strength, swelling, permeability and durability [18].

Chemical soil stabilization has been utilized for many centuries. The Romans were one of the first to utilize a chemical stabilization process. Weak soils were mixed with pozzolana (volcanic ash containing alumina and silica) and lime to improve its bearing capacity. The modern day treatment of soils started in the late 1950s, in the US where weak clays were treated with hydrated lime. The development and improvement of construction equipment since these early days has seen significant utilization of the process globally [12,19].

Chemical stabilization of a soil eliminates the need to remove an inherently weak sub grade soil and replace it with a quarried, processed granular material. This process is not only cost effective, but it also lessens the demand on non-renewable resources and reduces the environmental footprint of a road construction project [12].

2.2.2 Soil stabilization- the chemical reaction

Hydraulic binders are most effective when inherently weak materials would normally be removed and replaced with materials that have superior engineering characteristics. The soil type and mineralogy of the soil will dictate the binder that is utilized. Where significant quantities of clay and silt are present, the favored stabilizing additives are either lime (either hydrated or quicklime) or a combination of lime and Portland cement [7]. Where a much coarser material is present, additives such as Portland cement, fly ash and CKD are preferable. To identify binder type and concentration, laboratory mix designs are performed. This ensures the optimum addition of a binder in order to meet the desired end performance criteria [5].

When a hydraulic binder is mixed with a soil in the presence of adequate water, the following chemical reactions occur:

• Cation exchange: replacement of exchangeable cations held by the host soil by higher valiancy calcium ions, which are held by the lime.

• Flocculation/agglomeration of the host soil particles and an increase in the effective grain size.

• Pozzolanic reaction: a long-term reaction producing cementations materials, typically calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH) gels.

Of these reactions, the first two are immediate and result in a **modification** of the host soil. [14] Modification is more rapid when lime is added to a soil, as more calcium ions are present compared to those present in Portland cement, fly ash and CKD. Providing an adequate concentration of binder has been introduced to the host soil and an alkaline environment has been maintained after modification, a pozzolanic reaction will occur. This reaction process is very time dependant and can continue over a long period of time. The reaction phase is generally referred to as **stabilization** of the soil [9].

2.2.3 Soil modification

During the modification process numerous alterations to the host soil occur. These alterations include dramatic reductions to the plasticity (and shrinkage characteristics) of a fine-grained soil, alteration of compaction characteristics, and increases to the stability of the host soil. These reactions occur immediately upon addition of a hydraulic binder and are typically complete within a 48-72 hour timeframe [9].

Changes to the plasticity of the soil are a result of the cation exchange resulting in particle flocculation and aggregation. This increases the effective particle size of the fine-grained soil resulting in a more silt-like material [5, 9]. This typically increases the plastic limit and decreases the liquid limit. In some instances the soil may even become non-plastic. For some soils the liquid limit may actually increase with lime concentration. Research tends to suggest that this is clay mineral dependent; all reported increases in liquid limit in soils where élite was the predominant clay mineral. Even with an increase in liquid limit, the accompanying increase in plastic limit is always greater – thus resulting in a net reduction in the plasticity index of the soil [5, 9].

The addition of hydraulic binders alters the compaction characteristics of the host soil. The maximum dry density (MDD) decreases and the optimum moisture content (OMC) increases. Typically, the higher the concentration of binder, the greater the alterations to the compaction characteristics are. The OMC increases due to the hydration effect and the affinity for more moisture during this reaction process [5].

Decreases in density are directly attributed to the flocculation/aggregation and the formation of weak cementations products. Flocculation/aggregation of the soil offers greater resistance to densification at a given level of compactive effort. The net result is a reduction in the MDD [5, 9].

In addition to the above, minor improvements to the stability and strength of the soil can also be observed. These immediate strength gains can generally improve adverse soil conditions, when soft, wet and highly plastic soils are encountered. Once treated, construction processes can be expedited and a satisfactory sub grade support can be achieved for construction traffic within several hours after binder application [9].

2.3 Lime Stabilization

Lime is one of the oldest and still popular additives used to improve fine-grained soils. Lime, either alone or in combination with other materials, can be used to treat a range of soil types. Lime treatment of soil facilitates the construction activity in three ways [6]. First, a decrease in the liquid limit and an increase in the plastic limit results in a significant reduction in plasticity index. Reduction in plasticity index facilitates higher workability of the treated soil. Second, as a result of chemical reaction between soil and lime a reduction in water content occurs. This facilitates compaction of very wet soils. Further, lime addition increases the optimum water content but decreases the maximum dry density and finally immediate increase in strength and results in a stable platform that facilitates the mobility of equipment [6, 12].

The interactions of lime with the soil particles can be described by a series of complex physical and chemical processes that affect the mechanical behavior of soils. There are two effects during lime treatment. At first, there is a so-called short-term or immediate effect, which occurs in the following hours of the contact between the lime and the soil and leads to flocculation / agglomeration of the soil particles. This results in a change in the texture of the soil. In a second step, there is an effect, said to be long-term, in which pozzolanic reactions occur. These reactions, which take place in the presence of water, between the lime and compounds composed of silicon and aluminum, lead to the formation of pozzolanic compounds that develop through time [13].

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, calcite 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 here in for hydrated lime should be reduced by 24 percent. Specifications for quicklime and hydrated lime are found in ASTM C 977 [4, 6].

2.3.1 Limestone

Calcitic limestone of dimension-stone quality is predominantly found within the Jurassic Antalo limestone (central part of the country) and the Hamanlei Series (east-central part). The best exposures and the most interesting deposits of the Antalo Limestone are found in the central part of the Abay Valley, and side valleys such as the Jema, Wonchit and Muger valleys. The Jema and Wonchit limestone deposits occur in the bottoms of the valleys of the same names. The lower part of the limestone unit is by far the most interesting, since this is the part where the bed thickness reaches more than one meter (Wondafresh et al. 1993). The limestone is essentially a calcareaous, fossilifereous sandstone with poorly developed structure; color varies from brown to off-white. Joint spacing varies considerably in the area, where the more massive parts of the deposits form small hills and plateaux. At the present time, these limestone deposits are not being exploited, due to difficult access (the access road is of poor quality) and locally closely spaced joints [21].

2.3.2 Types of lime

The most common form of commercial lime used in lime stabilization is hydrated high calcium lime, Ca(OH)2, but monohydrated dolomites lime, Ca $(OH)_2$, MgO, calcitic quick lime, CaO, and dolomites quicklime, CaO MgO are also used [5, 10].

For hydrated high calcium lime the majority of the free lime, which is defined as the calcium oxide and calcium hydroxide that is not combined with other constituents, should be present as calcium hydroxide. British Standard 890 requires a minimum free lime and magnesia content (CaO + MgO), of 65 per cent [5, 10].

In many parts of the world, lime has been produced on a small scale for many hundreds of years to make mortars and lime washes for buildings. Different types of kilns have been used and most appear to be relatively effective. Trials have been carried out by TRRL in Ghana (Ref. 11) to determine the output possible from small kilns and to assess the suitability of lime produced without commercial process control for soil stabilization. Small batch kilns have subsequently been used to produce lime for stabilized layers on major road projects [7].

The type of lime employed on a road project should be determined considering the lime supply, experience of the contractor, availability of equipment, location of a project –rural or urban- and availability of an appropriate nearby water source. For example, quicklime is excellent for drying wet soils. In addition, quicklime has larger particle sizes than hydrated lime, so dust generation is reduced when quicklime is used. In contrast, hydrated lime particles are fine, so dust may cause a problem in densely populated areas [3, 7].

A. Quick lime

Quicklime has a much higher bulk density than hydrated lime and it can be produced in various aggregate sizes. It is less dusty than hydrated lime but the dust is much more dangerous and strict safety precautions are necessary when it is used. For quicklime, British Standard 890 requires a minimum free lime and magnesia content, (CaO + MgO), of 85 per cent. ASTM C977 requires 90 per cent for both quicklime and hydrated lime [5, 10].

Quicklime is an excellent stabilizer if the material is very wet. When it comes into contact with the wet soil the quicklime absorbs a large amount of water as it hydrates. This process is exothermic and the heat produced acts as a further drying agent for the soil. The removal of water and the increase in plastic limit cause a substantial and rapid increase in the strength and traffic ability of the wet material [10].

Lime which has not been slaked, lump lime, burnt lime, calcium oxide, Cao, called quick because its affinity for water.

Commonly recognized sizes from ASTM C51-71 are;

- Large lump -200mm (8”) and smaller
- Pebble- 65mm (2.5”) and smaller
- Ground screed or granular – 6.5mm (0.25”) and smaller
- Pulverized mostly all passing No 20 (0.85mm) sieve

The research, development and availability of appropriate equipment and local manufacturers are ongoing.

This equipment includes the following:

Portable small jaw crusher, hammer mill, ball mill, grinder, or roller mixer for crushing and grinding quicklime and pozzolans to fine powder [7].

B. Hydrated lime

The quicklime is sprinkled with minimum water to form a dry hydrate powder. Chemically, both forms of hydrated lime (lime putty and dry hydrate) are calcium hydroxide [8]:

(Calcium oxide CaO) + Water (H_2O) forms calcium hydroxide Ca $(OH)_2$.

C. Natural hydraulic lime

Natural hydraulic lime is made by burning limestone which already contains active clay, with the calcium rich remains of sea creatures laid down at the same time, which eventually form a less pure limestone than that of non-hydraulic lime because of the added impurities of the ancient sediments [7]. It is the active clays in these sediments that are essential for creating a set under water required for flood resilience. The active clays in the limestone combine with lime when they are burnt together, to produce a natural hydraulic lime. Rock strata of natural hydraulic limestone will contain varying amounts of active clay which determines the degree of hydraulicity. There are classifications for natural hydraulic limes which cover a range of hydraulic set from weak to very strong. These are known as feebly, moderately and eminently hydraulic limes. (The stronger, eminently hydraulic limes have been used in mortar for water mills, embankments and light house) [5, 10].

Lime is used to treat weak sub grade soils during construction of highways. A small amount of lime (4 to 7%) is used to rapidly dehydrate and modify fine grained soils. The modification process improves workability and compactability of the soils and reduces the potential of swelling and shrinkage by saturating the clay particles with calcium ions. Although the lime modification process is primarily aimed at construction expediency, additional effects such as long-term improvement of stiffness and/or strength by pozzolanic and carbonation cementation reactions are expected [10,12]. Although lime treatment has been employed in Indiana over several decades, the long-term performance of lime treated soils has not been well quantified and no field tests have been done on roads in service. There is concern that repeated loading, weathering, change in water content, and potential for lime migration may cause with time a decrease in strength and/ or stiffness of lime-treated sub grade soils. For this reason, engineers do not usually account for the enhanced stiffness that the treatment may provide for pavement design. This results in a conservative design of the asphalt or concrete pavement layers [6].

Lime reacts with medium, moderately fine, and fine soils to produce decreased elasticity, increased workability, decreased swell, and increased strength. Lime may be effective for soils with clay content as low as 7%. Lime also works well when stabilizing (modifying) granular materials and lean clays. Cation exchange and flocculation agglomeration changes the texture of clay soils (called lime modification). This flocculation process causes a short-term increase in strength. In addition, pozzolanic reactions occur when lime, water, soil and silica react to form various cementing compounds. This process causes a long term strength gain that may be as high as 100 psin (690 kPa) at 28 days, 625 psi (4.3 MPa) at 56 days, and 1580 psi (10.9 MPa) at 75 days (cured at 120 F (49 C) with 5% lime). Soil properties including optimum pH (about 12.4, where the solubility of silica and alumina increase) influence the lime reactivity of a soil [1, 12, 24].

2.3.3. Benefits of Lime

There can be many reasons for the choice of lime as the preferred binder and stabilizer, not only because it is an excellent material for stabilizing clay soils. This has been well demonstrated in southern Pakistan where a great many local clay soils have reacted with small amounts of non-hydraulic lime to create a hydraulic set, sufficient to remain stable under water for many months.

In the context of communities in rural areas of the world, abundant limestone resources indicate that there is the opportunity for lime to be produced and used locally in many other ways. Lime has other attributes and uses in addition to those for construction. One of the most important of these is its contribution to improving human health and hygiene [7].

One of the ecological benefits of lime is its contribution to a sustainable environment. Efficient small scale local lime production results in lime binders having significantly less embodied energy than cement (the manufacture of which has a very high environmental impact), shorter transport distances and the re-absorption of carbon dioxide (CO^2) in its setting process. As such, pure lime production (non-hydraulic lime, as commonly found in southern Pakistan) can be almost carbon neutral [7]. Developing fuel wood plantations in conjunction with small scale lime production would further enhance the ecological benefits as part of a holistic and sustainable approach to the use of lime in rural [7].

2.4. Quicklime Preparation

Testing the reactivity of quicklime prior to the purchase and delivery of the quicklime is essential to ensure that it is of the best quality and is sufficiently reactive. Mixes that incorporate lime of a poor quality are likely to fail [7].

It is very important that the quicklime is well burnt, fresh from the kiln, and contains no under-burnt or over-burnt material. Confirm this through testing, and then either on a small scale with hand tools and a sieve, or on a larger scale with a machine such as a; jaw crusher, or ball mill, or a roller mixer, crushes the quicklime separately [7]. These machines are widely manufactured and many types are produced that can be hand or animal powered. The ideal crushing machine to select is one that is able to crush both quicklime to powder and pozzolans to the fine particle sizes [3].

2.4.1 Establishing Quicklime Proportions

These proportions will vary dependent upon soil type but are a guide for evaluating initial trial mixes. All percentages given are to the total amount of soil in the mix. The principal consistency of soil and lime are to a large extent to their particle size [3]. The precise definition varies from one country to another for practical purposes in this manual they are as set out below.

Particle size

Gravel	75mm to 5mm
Sand	5mm to 0.06mm
Silt	0.06mm to 0.002mm
Coarse Powdered quicklime	below 3.35mm (ASTM; 6)
Lime Dry Hydrate	below 0.6mm
Lime Putty	0.180mm
Fine powdered quicklime	0.85mm (ASTM; 20)

Table 2.2 Sieve sizes in the selection or grading of materials [10]

Sieve Size	(ASTM) Sieve No.	Material
5 mm	No. 4	Soil, gravel and course sand
3.35 mm	No. 6	Powdered quicklime
2.36 mm	No. 8	Lime putty for foundations
2.0 mm	No.10	Medium sand
850 micron	No. 20	Lime putty for finishing coat and quicklime powder for blocks
600 micro	No. 30	Lime dry hydrate fineness, fine sand
450 micro	No. 40	Soil testing
180 micron	No.80	Coarse pozzolans& lime putty for fine stucco and decorative work

2.4.2 Testing Quality of Lime

The powdered quicklime to be used should be fully reactive and pass through a 0.850mm mesh (No.20 sieve) for blocks or a 0.180mm (No.80) sieve for render. (As seen table above: Sieve

Sizes in the Selection or Grading of Materials). If the quicklime is good quality, it will quickly break down into powder. The quicker it breaks down, the better the quality. Sieve the powder before use [3, 4, 7].

A. Finest Test. Sieve testing will give an initial indication of the quality of a dry hydrate. If production, packaging and storage have been in accordance with the recommended National Standards, the lime should pass simple particle size tests.

B. Lime Reactivity Test.

1. Pour 1 liter of room temperature water into a 2 liter metal jug.
2. Add 500g of crushed quicklime to the water in the metal jug.
3. If the lime is good quality and lively, ready to use, it will boil the water within 5 minutes.

C. Lime Putty Density Test. An upper limit of 1.45g/ml is a standard set by several international standards for lime putty of standard consistency. The putty density can be calculated with a standard size (½ liter or 1 liter) or graduated container of sufficiently regular shape to maintain precise and constant volume each time the container is filled. Fill the container with exactly one liter of the putty and ensure all air is expelled by tapping it down until no further putty can be added. Carefully strike off surplus from the top. Continue to tap down, strike off and add putty until there is no increase in mass. The density is calculated by dividing the maximum mass of the putty in grams, by its volume in milliliters, or for field test purposes, kilograms per liter [7].

Table 2.3 ASTM Bulk Density Levels for Lime Dry Hydrate Classification

Dry hydrate of lime	Bulk density (g/ml)
White (pure) lime, Non hydraulic	0.5
Feebly(slightly) hydraulic	0.65
Moderately hydraulic	0.65-0.8
Eminently hydraulic	0.9-1.0

2.5. Design Factors for Cementing Agents

Mix design is done to improve various engineering properties such as Liquid Limit (LL), Plastic Limit (PL), Plasticity Index (PI), swell characteristics, cured strength, and uncured strength. The process involves analysis of the soil at various lime percentages. CBR methods are used to evaluate the mixes. 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. 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 stabilization with lime [17, 19]. Cementing agent content is usually specified as a percentage of dry soil weight. Samples are prepared dry and then blended with water.

Design criteria depend on the engineering objectives. Some common criteria are:

• No further decrease in PI with increased cementing agent percentage

• Acceptable PI reduction for a particular modification objective

• Acceptable reduction of swell potential, and

• Sufficient CBR increase for the proposed use.

2.5.1 Construction Steps for Lime Treatment

- ✓ Prepare the soil. (Must be careful of the fluff action that is possible when using lime).
- ✓ Apply the lime. Dry hydrated lime and dry quicklime may be applied by bulk application methods or by the single bag method**.** Quicklime must be applied with greater emphasis on safety. Lime may also be applied by the slurry method.
- ✓ Compact the soil. Most projects require 95% of AASHTO T-99 for sub bases and usually 98% for base courses. The compactive effort may be applied with a sheep foot roller followed by a multiple wheeled pneumatic roller. (A flat wheel may be used for finishing). Note that single lift compaction may be done with a vibratory roller or pneumatic roller followed by a light pneumatic or steel roller to finish.
- ✓ Cure the mix. Temperatures should be above 40 – 50 F (5 – 10 C). Moisture content should be kept close to optimum to aid compaction and curing. Curing may be done with moist cure or asphalt-membrane cure techniques [12].

2.6. Soil Demand of the Lime

This method describes the procedure to determine the degree to which a soil will react with calcium hydroxide through cationic exchange and pozzolanic responses from reactive clay minerals [1, 12, 15]. The method provides for the determination of the lime demand (percent lime), as measured using an extended pH test, and is used as a starting estimate of an optimum in design lime content. The lime demand test as performed by this procedure has been shown to provide lime contents that correspond well with optimum lime contents for long term effective stabilization [4, 5]. There is a lower limit of lime dosage below which mixing uniformity cannot be achieved with normal construction operations. The minimum percentage of lime is usually determined from the Eades and Grim procedure. The procedure is based on pH measurements. The amount of lime necessary to achieve a pH of 12.4 is considered to be the minimum. The dosage of lime applied to treat a soil may be also determined based on reduction of Plastic Index and/or improvement of strength properties such unconfined compressive strength. INDOT's design guide recommends that the lime dosage necessary for chemical treatment of a subgrade is determined from the Eades and Grim procedures [5, 24].

2.6.1 PH determination

The apparatus and procedures followed for PH determination are [19,23]:

1. PH meter
2. Balance of suitable capacity
3. Sieve, 2.36 mm.
4. Beakers, 100 ml.
5. Watch glasses, of appropriate size to cover the 100 ml beakers.
6. Measuring cylinder, of 100 ml capacity.
7. Magnetic stirrer.

9. Wash bottle.

Procedure for PH measurement

1 Prepare the required number of test portions with individual masses as calculated and place the test portions in suitably marked beakers.

2. Add the corresponding mass of the lime as calculated in Step 1 to each beaker and mix the constituents using a glass stirring rod. Add 5 g of the lime to another beaker, and then cover each beaker with a watch glass.

3. Add 75 ml of distilled water to the beaker containing the lime only.

4. Mix the suspension using the magnetic stirrer and then cover the beaker with a watch glass and allow it to stand for 2 hours.

5. Restart the stirrer and lower the pH electrode into the suspension until the bulb is just covered.

6. Read the pH meter at 1 minute intervals and continue readings until 3 successive values are within a range of 0.05 pH units. Record these values to the nearest 0.01 units together with the average pH value.

7. Remove the electrode from the beaker, wash it with distilled water and check the reading on the meter as detailed in Subsection 5.3 using the higher pH buffer solution.

8. Test each of the soil-lime mixtures in order, commencing with the lowest lime content, by adding 75 mL of distilled water to the beaker.

9. Continue testing until the average pH values of the 3 highest lime contents do not vary by more than 0.05 pH units.

Glass stirring rod

Calculations

1 Plot the average pH against its lime content and join each point. Next, draw a line parallel to the X axis corresponding to the pH for lime.

2 Record the lowest lime content (LC) where the pH just reaches a stable peak value, that is, a plateau where the pH values do not vary by more than 0.05 pH units over three successive soil-lime mixtures[13,16, 19].

CHAPTER THREE

MATERIALS AND RESEARCH METHODOLOGY

3.1 Study Area

Sheka zone is one of zonal administration in SNNP of Ethiopia and located at about 655 Km in Southwest of Addis Ababa and 884Kms from Hawasa which is capital city of SNNP. According to current government Sheka Zone is divided in three woreda, such as Masha, Yeki and Andracha woreda. The Geographical condition of the zone is approximately 7°36'N Latitude and 37°42'E Longitude [26]. This zone is one of densely forested area in Ethiopia; in which lands are fertile, suitable for agriculture and highly dominated with organic soil. This zone has a temperature of 20-31°C with an average annual rainfall 800-2400mm which occurred from April-October. It lies in the climatic zone locally known as Dega and Woynadega which is considered comfortable for human settlement.

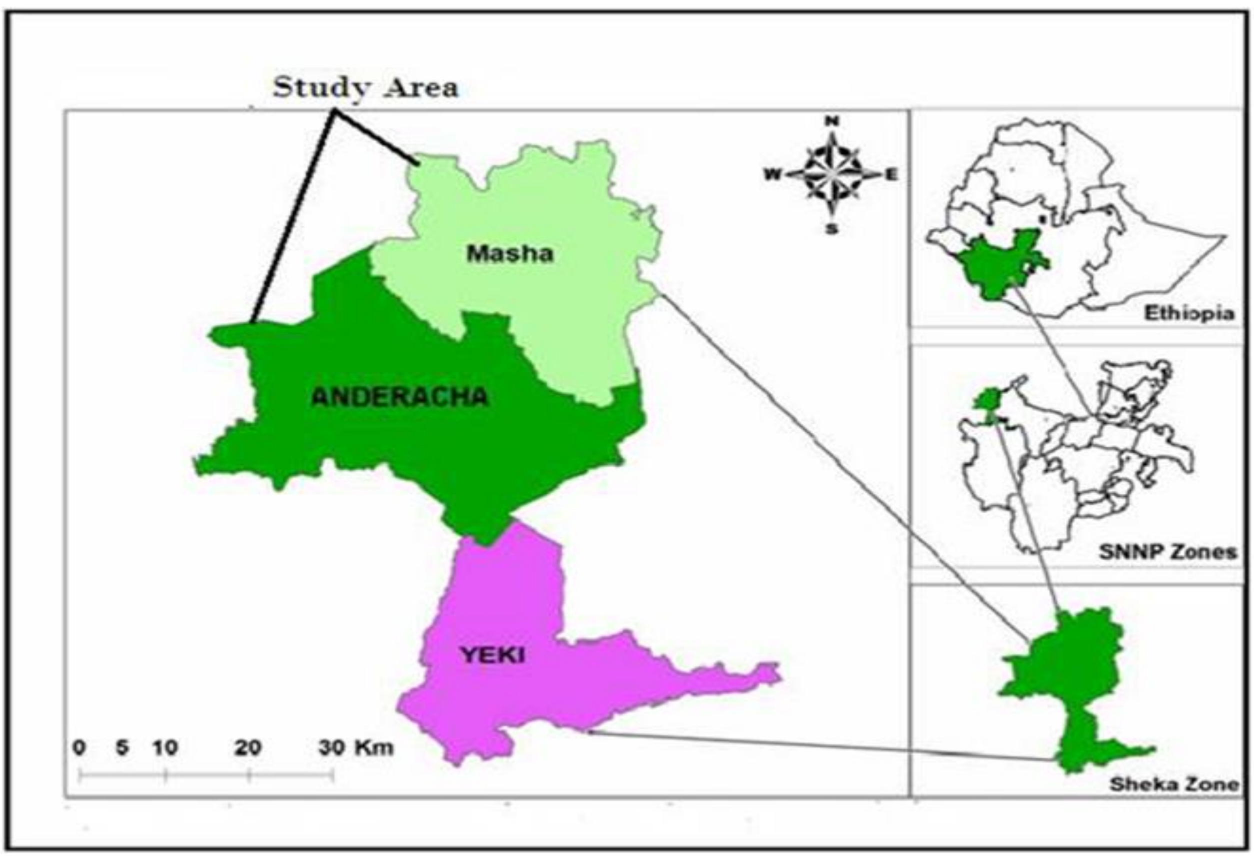

Figure 3.1 Study Area Map (Source: Lulu K. Msc. Thesis JU 2018)

3.2 Materials Used

3.2.1 Subgrade Soil

The soil sample used for this study is collected from local area at Masha in Sheka Zone at a depth of 1.5m using the method of disturbed sampling. The properties of the soil used in the investigation are given in Table 4.1.

Figure 4.2 Soil sample taking (photo taken by Anteneh)

3.2.2 Lime

The lime used in this study was prepared from the naturally occurred limestone which was collected from local area at Degele Kebele in Sheka Zone. The limestone was burned and crushed with ideal hand tools and sieved through 0.42micro m aperture in the form of powdered quicklime before use as per ASTM C 50-00 (2000). The oxide composition and reactivity properties of the lime used were shown in Table 4.5 respectively.

Figure 3.2: Preparation of Lime (photo taken by Anteneh)

3.3. Methodology

3.3.1 Study Design

This research was designed to answer the research questions and meet its objectives based on experimental findings. The first step in the research work was sample collection. The next step was laboratory tests on the untreated and treated expansive soil. The laboratory test data was analyzed and interpreted so that the properties of expansive soil and its performances on additives requirement was addressed. Finally, the research findings and recommendations were expressed based on the laboratory test results.

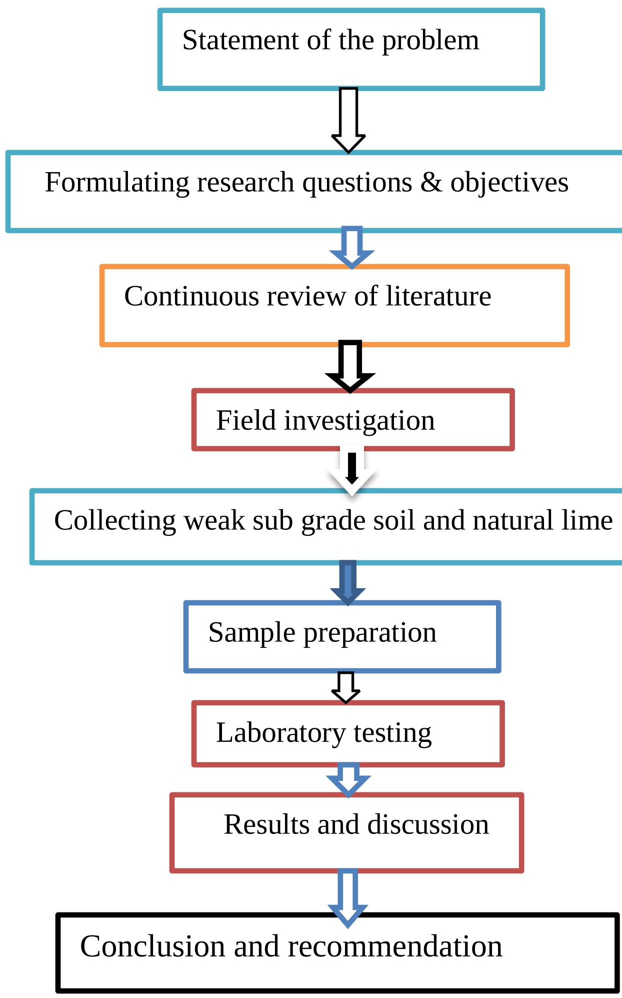

Figure 3.3; Study design flow chart

3.3.2 Sampling Techniques and Sample Size

3.3.2.1 Sampling Techniques

The sampling technique used for this research was a purposive sampling which is non-probability method. This sampling technique was proposed based on the intension to perform laboratory test on the selected sub grade soil to improve its strength using natural lime.

3.3.2.2 Sample Size

The soil samples used for this study were collected from road corridors of Masha and Gecha towns in Sheka Zone at a depth of 1.5m using the method of disturbed sampling. From those, one most weak soil was selected by observations due to time and transporting constraint and the intension to improve it and to adapt the use of natural lime in local road construction.

3.3.2.3 Sample preparation

The soil samples were first air dried, properly pulverized and additives were mixed in such a way that the additive is first added to the prepared sample and dry mixed with the soil. The weak subgrade soil was mixed with Lime by percentage of the weight of soil taken for each test starting from 5% to 9% within 1% difference for Lime. But initial lime content was determined by PH measurement of soil-lime mix. As the respective of each test procedures preparing uniform samples for Atterberg Limits, Compaction and Californian bearing ratio test was conducted. Soil sample was first dry mixed with the respective lime was added there after followed by a thorough mixing.

3.4 Study variables

There are two types of variables those had taken into consideration and those are the dependent and independent variables;

- **Dependent Variables**
  - Subgrade strength with Lime
- **Independent variables**
  - Atterberg test
  - Compaction test

- CBR test
- Sieve Analysis
- Specific gravity
- Optimum Lime Content (OLC)

3.5 Data Collection Methods

The primary research data was collected through experiments, site visit whereas the secondary data also collected through the existing relevant documents and literature review and analyze the issues related to the concerned objectives of the study.

3.6 Data Processing and Analysis

The research was conducted first by identification the effects of lime on weak sub grade soil through laboratory tests. The results of laboratory tests are going to be analyzed using excel tables and drawing different kind of graphs.

3.7 Laboratory Tests

Tests for soil classification which included grain size distribution, and Atterberg limits. These are indicative tests that are usually used for identifying whether the soil is expansive or not. The conducted tests however included wet sieve analysis, Atterberg limits, specific gravity, moisture density relation, CBR and CBR swell.

3.7.1 Expansive soil

3.7.1.1 Grain Size Analysis

This test was performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis was performed to determine the distribution of the coarser, larger-sized particles according to AASHTO T 088-93. Wet sieve analysis was used for this study.

Figure 3.4 Equipment prepared for Sieve analysis (photo by Eyuel)

3.7.1.2 Atterberg Limit Test

The test procedure adapted for the determination of Liquid limit, Plastic Limit and plasticity index for both untreated and treated soil sample was in accordance with AASHTO T89-94 and T90-94 respectively. A sample weighting about 50gm was prepared for liquid limit and plastic limit test for each samples. Soil samples were first air dried and pulverized and then sieved with number 40 sieve. Soil passing number 40 sieves was mixed with different proportion of lime-bagasse ashes at optimum water content and sealed with plastic for 24 hours in order to give sufficient time for chemical reaction before test. Hand mixing in a porcelain pan was the method of mixing. The liquid limit of the soil had been determined by using casagrande apparatus. The plastic limit of the soil was determined by using soil passing through a 425 µm sieve and rolling 3-mm diameter threads of soil until they began to crack.

Figure 3.5 Photos of liquid and plastic limit test

3.7.1.3 Soil Classification

The most widely used soil classification systems are AASHTO and USCS systems. The AASHTO Classification system is useful for classifying soils for high way. On this research each Soil was classified using the AASHTO and USCS Soil Classification System using particle size distribution and Atterberg limits.

3.7.1.4 Compaction Test

This laboratory test was conducted to determine optimum water content at maximum dry density of soil. Compaction is when mechanical loads applied to soil result in expulsion of air, increase in bulk density and resistance to penetration. The laboratory modified proctor test was performed as per AASHTO T 99-94. The test was performed on disturbed samples of soil passing sieve sizes 4.75mm or 19mm mixed with water to form samples at various moisture contents ranging from the dry state to wet state. These samples were compacted in five layers at 25 blows per layer in accordance with the specified nominal compaction energy of modified proctor test. Dry density was determined based on the moisture content. The corresponding water content at which the maximum dry density occurs is termed as the optimum moisture content.

Figure 3.7 Sample prepared for compaction test (Photo by Eyuel)

3.7.1.5 California Bearing Ratio Test (AASHTO T-193)

The CBR is expressed by force exerted by the plunger and the depth of its penetration into the specimen; it is aimed at determining the relationship between force and penetration. A three point CBR test at 10, 30 and 65 blows were conducted according to AASHTO T193 and the

CBR values at 95% MDD was determined. The CBR test indirectly measures the shearing resistance of a soil under controlled moisture and density conditions. The CBR is obtained as the ratio of load required to affect a certain depth of penetration of a standard penetration piston into a compacted specimen of the soil at some water content and density to the standard load required to obtain the same depth of penetration on a standard sample of crushed stone. The equation to be computing the CBR value is as follows.

$$CBR(\%) = \frac{\text{Applied load on sample}}{\text{standard load on the crushed stone}} \times 100$$

The required quantity of soil, lime and water for one specimen were calculated using dry density and moisture content determined from Proctor Test and the total quantity of each needed to prepare the required number of test specimens at each prescribed stabilizers percentage of maximum dry unit weight and water content was known.

Fig 3.8 Photos of CBR test and CBR Swelling (Photo by Eyuel)

3.7.2.1 Chemical Composition of Lime (locally prepared)

Chemical composition of the existing lime was tested as per ASTM C 25-99. An X-ray Diffractometer was used for this study. X-ray diffraction is a method to investigate the organization of solids at the atomic scale. The chemical composition test of the lime was conducted in south west soil laboratory (Teppi).

For this test 5 g of lime sample was taken and put it into the container and the container have a provided space in the equipment, which is digital and connected with computer, then finally the chemical composition of the sample was recorded from computer. From the XRD test results, the presence of calcium oxide (CaO), calcium hydroxide (Ca(OH)2), or calcium carbonate (Ca(CO)3) in the sample can be identified. Note that with this test the minerals are identified, but the test cannot provide a quantitative estimate of the mineral in the sample. The PH value of this lime and soil-lime mixture was tested in JIT chemical engineering lab. The test result of the chemical analysis is given in table 4.4. From the previous research, the chemical constituents of the lime used for soil stabilization is presented in the table below.

Table 3.1 Chemical constituents of hydrated lime from previous research [1].

Constituents	Weight by %
SiO2	4.11
Al2 O3	3.11
Fe2 O3	2.7
Ca CO3	3.8
CaO	63.7
CaSO4	19.26
MgO	1.62
Loss on ignition	1.7

3.7.2.2 Optimum Lime Content Determination

The optimum content of lime required for the stabilization of the soil was determined from the following procedure:

1. Perform mechanical and physical tests on the natural soil.
2. Determine pH of both the soil and lime.
3. Determine the optimum lime content using the Eades and Grim pH test [1, 23, 24].

This test was conducted to determine a sufficient amount of lime to be added to the soil to obtain a pH of 12.4 or equal to the pH of the lime itself [1.24]. The pH was measured in accordance with ASTM D 4972-01. A graph was plotted between pH and lime percent. The optimum lime content is the one associated with the maximum pH of the soil-lime mixture [6, 24]. The result of this test is presented in table 4.5.

Calculation of test masses

For each lime increment calculate the mass of soil and mass of lime to be used, based on a combined dry mass of 30 g as follows:

1. Mass of Soil Test Portion has been obtained from the following equation

$$mw = \left[\frac{30}{1+\frac{P}{100}}\right]\left[1+\frac{w1}{100}\right]$$

Where mw = mass of soil (g)

W1 = Optimum moisture content (%)

P = lime content (%)

2. Mass of lime has been determined from the following equation

$$m1 = 30 - \left[\frac{30}{1+\frac{p}{100}}\right]$$

Where m1= mass of lime (g)

P = lime content (%)

Table 3.2 Calculated masses of soil and lime for soil-lime PH test

P (%)	m1(g)	w1 (%)	mw(g)
1	0.31	31.9	39.18
2	0.59	31.9	38.79
3	0.88	31.9	38.17
4	1.15	31.9	38.04
5	1.43	31.9	37.68
6	1.71	31.9	37.33
7	1.96	31.9	36.96
8	2.22	31.9	36.63
9	2.48	31.9	36.3

CHAPTER FOUR

RESULTS AND DISCUSSIONS

This chapter presents the results of laboratory tests and a discussion pertinent to the results. The engineering property of the soil is evaluated both in unstabilized and stabilized state. Chemical constituent, PH test for soil and lime and for soil-lime mixture with different lime content was conducted. The test on both natural (untreated) and treated soil includes; Atterberg limits, moisture density- relationship (compaction), California bearing ratio (CBR) and CBR Swell test.

4.1 Property of material used in the study

4.1.2 Natural Subgrade soil

The results of the laboratory tests conducted for identification and determination of the engineering properties of the untreated soil before applying the lime were presented in table 4.1.

Table 4.1: Summary of Geotechnical properties of the natural soil

Parameters	Result in %
Natural Moisture content (%)	40.64
Percent of Passing No-200 sieve %	93.23
Liquid Limit	77.04
Plastic Limit	35.08
Plastic Index	41.24
AASHTO Soil Classification	A-7-5
USCS	CH
Specific Gravity	2.71
Maximum Dry Density (g/cm^3)	1.39
Optimum Moisture Content %	31.94
Soaked CBR Value%	2.66
CBR Swell %	2.52
PH value	4.85

Generally Liquid limit less than 35% is low plasticity, between 35% and 50% intermediate plasticity, between 50% and 70% high plasticity and between 70% and 90% very high plasticity (Whitlow, 1995). ERA manual volume I (2000) foundation and subgrade construction describes Soil having a liquid limit exceeding 60% or a plasticity index exceeding 30 when determined in accordance with the requirements of AASHTO T-89 and T-90 which are sufficiently wet and soft. This subgrade shrink and swell easily and does not resist internal and external load. Therefore, this soil requires initial modification and/or stabilization to improve its workability and engineering property [25].

4.1.1 Grain Size Analysis (AASHTO T 27-93)

The distribution of different grain sizes affects the engineering properties of the given soil. Grain size analysis provides the grain size distributions, and it was required in classifying the soil. Distribution of particle sizes greater than 0.075 mm is determined by sieving, To determine the distribution of coarser particles, 1000g of the natural subgrade soil is taken and washed on sieve size of 75μm and oven dried.

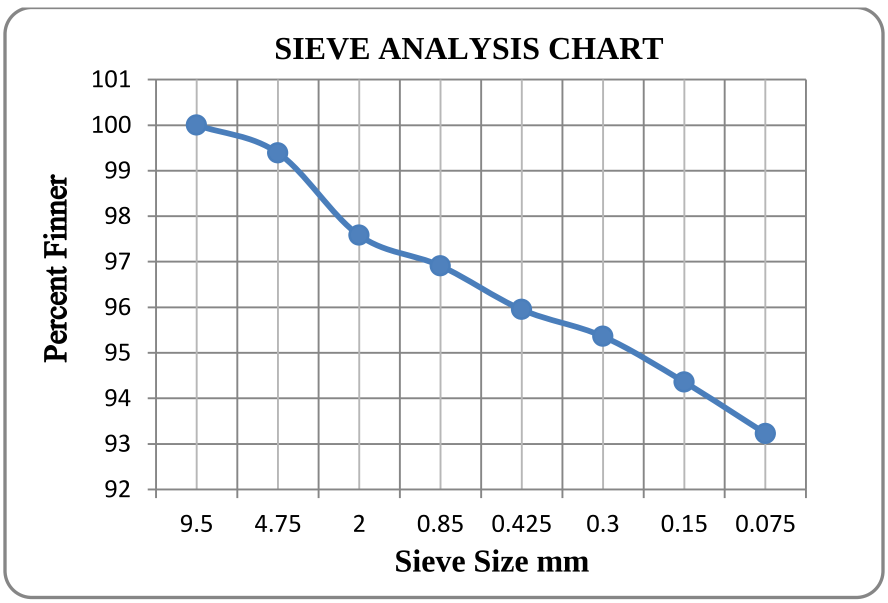

Figure 4.1 Grain Size Analysis Graph for Natural soil

According to AASHTO soil classification soils 35% minimum percent pass sieve no.200 are classified as silty-clay materials. The minimum percent pass sieve no.200 for the soil under study is 93.23% and the soil is categorized as poor clay subgrade soil.

4.1.2 Atterberg limit test on natural subgrade

Atterberg limits (liquid limit, plastic limit) were determined according to AASHTO T 89 and 90 standard test method. The detailed tabular results of the Atterberg limits were shown in appendix A. Based on the Atterberg test result, summary of soil samples is tabulated below.

Table 4.2: Summary of Atterberg limit for the natural subgrade soil

Soil Sample	Average LL %	Average PL%	PI %
Test	77.04	35.8	41.24

According to Atterberg limit test result as shown above Table 4.2 The soil sample changed from liquid state to plastic state and got an average liquid limit of 77.04 The given soil sample translate from plastic state to semisolid state and got an average plastic limit of 35.8. At this state the soil rolled into threads. The difference between the liquid limit and plastic limit is called Plastic Index. The soil sample also has Plastic Index of 41.24.

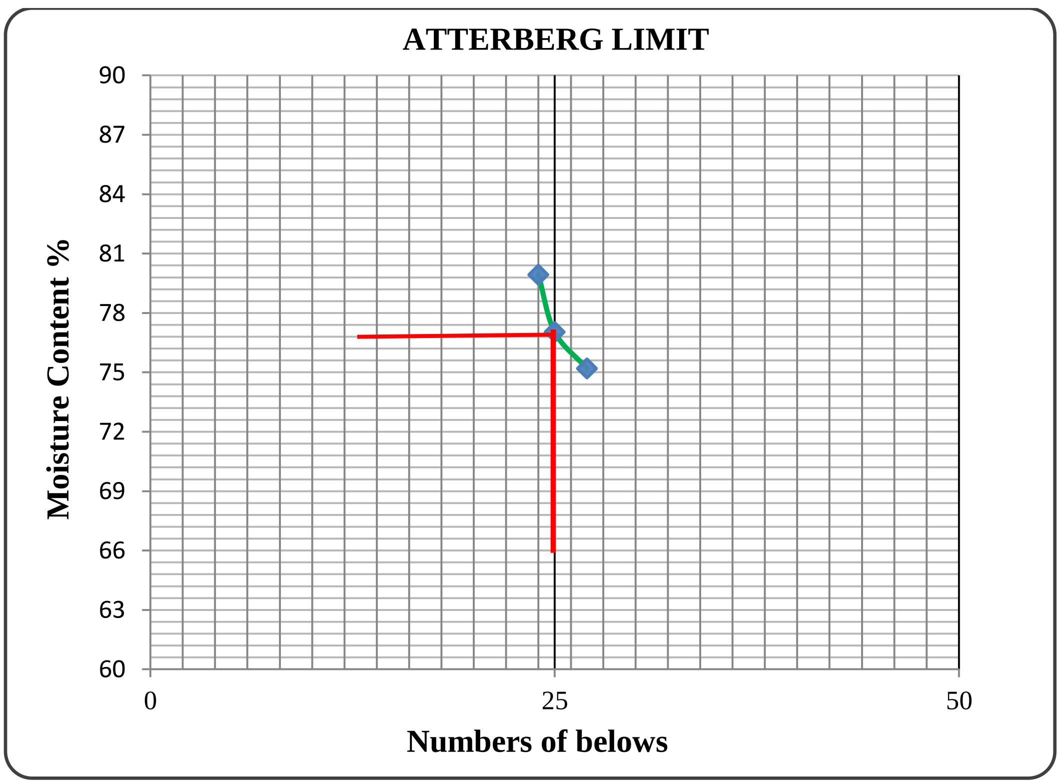

Figure 4.2 Liquid Limit Graph of natural soil

As result of Plastic Index indicates the native subgrade soil sample is poor for sub grade material unless it treated.

4.1.3 Soil Classification

4.1.3.1 AASHTO Classification system

The AASHTO system uses similar techniques as that of USC but the dividing line has an equation of the form PI= LL-30. It generally classifies a soil broadly into granular material and silt-clay material. The granular material is further divided into three groups which are called A-1, A-2 and A-3. The silt-clay material is in turn divided into four groups namely, A-4, A-5, A-6 and A-7. As it can be observed from AASHTO Classification system plasticity chart was as Follows in Fig.4.3

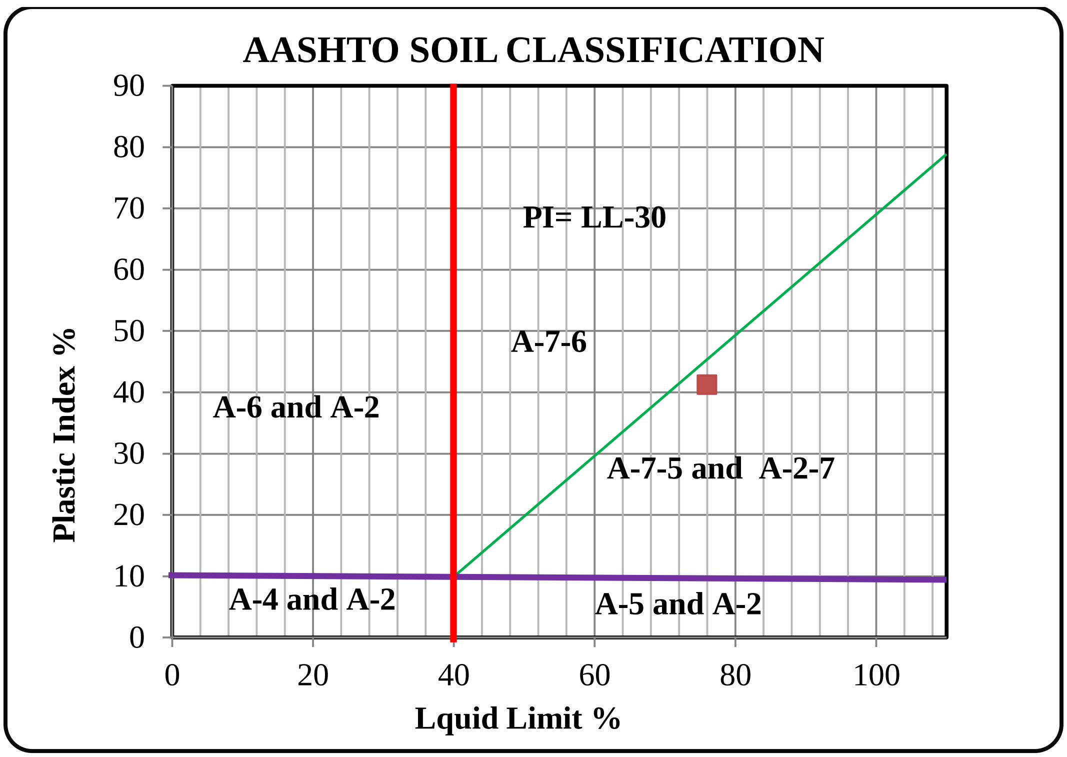

Figure 4.3 Soil classifications according to AASHTO system

According to AASHTO soil classification system Atterberg limit result soil sample has classified under group A-7-5 with rating Fair-to- Poor to be used as subgrade material. Thus, the natural subgrade material is unsuitable to be used as subgrade material without employing some improvement. 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 stabilization with lime [17].

4.1.3.2 Unified Soil Classification System

This system describes a system for classifying minerals and oregano-mineral soils for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit and plasticity index and shall be used when precise classification is required. The classification of the soil is presented in Fig 4.4.

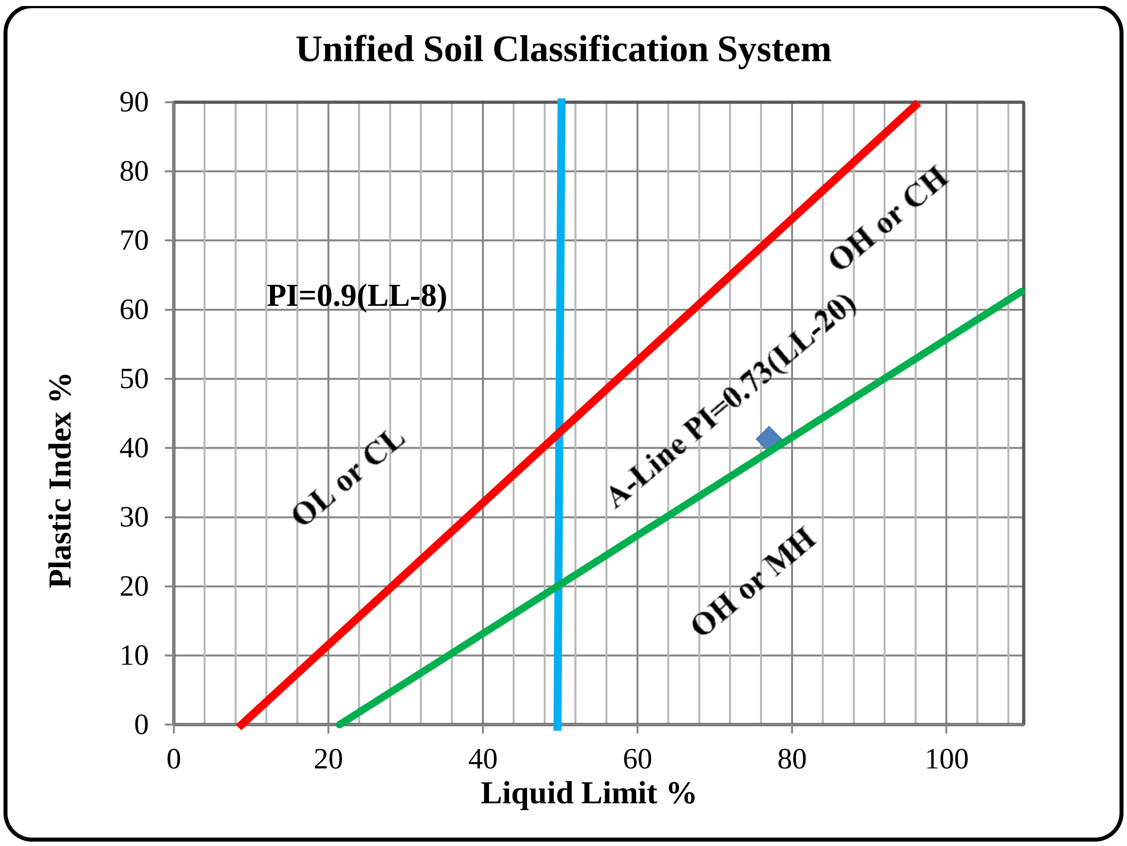

Figure 4.4 Soil Classification according to Unified soil classification System.

According to USCS, if the Liquid limit are greater or equal to 50% the soil can be clay, silt, or organic depends on whether the soil coordinates plot above or below the A line. Since soil sample has Liquid limit more than 50% and above A-Line, so the soil is classified under high CH.

4.1.3 Specific Gravity of natural subgrade soil

This test was conducted on fine grained particles of materials used for the study and summary of the test results was tabulated as followed in Table 4.3.

Table 4.3: Specific Gravity of Natural subgrade soil Sample

Test	Specific Gravity (Gs)
Soil Sample	2.72

As Table 4.3 showed that soil sample has an average specific gravity of 2.72. The specific gravity of solid particles of most soils varies from 2.5 to 2.9. Therefore, as Table 4.3 This result indicated that the sample was dividing under clay soil.

4.2.3.6 Compaction test results of natural subgrade soil

Standard Proctor compaction tests were conducted on the soil to determine the relationship between the moisture content and dry density for specific compaction effort according to AASHTO T99-94. The soil sample has optimum moisture content 31.94% and the maximum dry density is 1.39gm/cm^3 as shown below in Fig 4.4

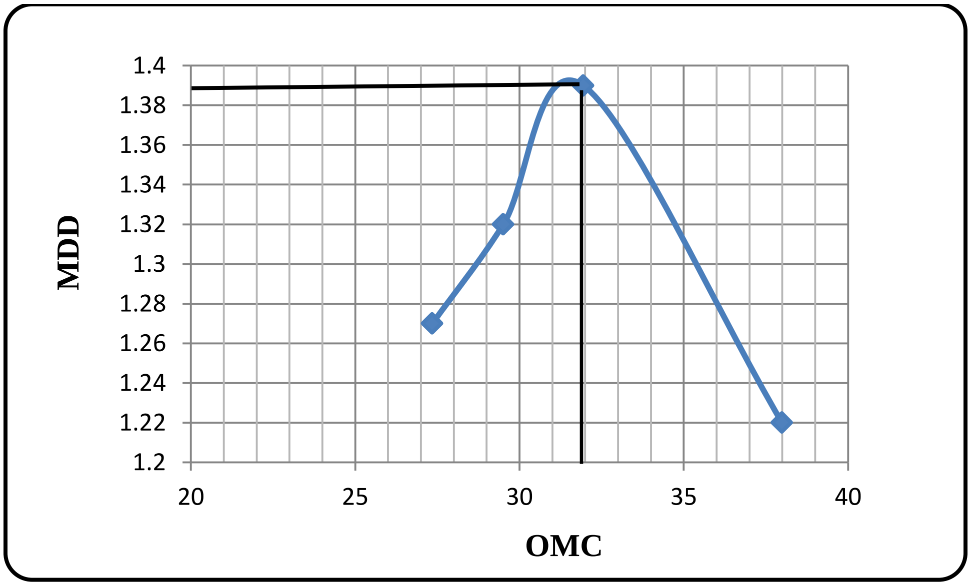

Figure 4.5 Moisture-density relations of natural soil

4.1.3 Soaked California Bearing Ratio (CBR) and CBR swell Tests

Strength of the soil has also been determined. A soaked CBR test was conducted according to AASHTO T193, and the result attached to appendix A.

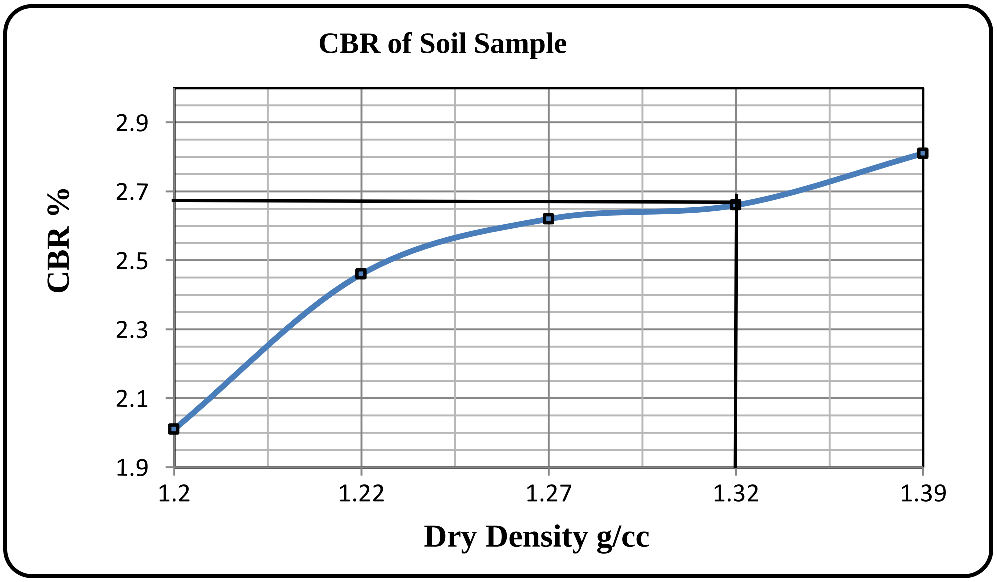

Figure 4.6 CBR Chart for natural soil

As shown in figure 4.6, soil sample had 2.66% CBR value at maximum dry density with 2.52% CBR swell. The test result showed that the soil sample has low CBR value, which does not satisfy the minimum requirements as sub-grade material. According to ERA standard specification a CBR value of less than 3% needs special treatment to be used as subgrade [25].

4.2 Properties of the lime used for this study

The rate of the pozzolanic reaction is dependent on the basic characteristics of the Pozzolana such as the density, surface area and the chemical composition. Therefore, the chemical constituents and its quality test results of the lime are presented in the table 4.5 below.

Table 4.4 Chemical constituents and property of the lime used

Constituent	**Weight by %**
SiO_2	1.3
Al_2O_3	1.12
Fe_2O_3	1.8
$CaCO_3$	5.32
CaO	78.6
$CaSO_4$	4.23
MgO	6.53
Loss on ignition	1.1
Quick lime property test	
Reactivity test	It boils the water in 4.17min
Density test	0.72g/ml
Observation test	It is quite light in color

From ERA pavement design manual in selecting types of lime, for quicklime, British Standard 890 requires a minimum free lime and magnesia content, (CaO + MgO), of 85 per cent and ASTM C977 requires 90 per cent for both quicklime and hydrated lime. Therefore, this lime satisfied the BS 890 for quicklime and according to ASTM bulk Density Levels for Lime Dry Hydrate Classification, this lime is moderately hydraulic and reactive [7, 25].

4.3 Initial Content of Lime (ICL) Determination

This test was conducted to determine the lime content which is suitable to treat the soil. From ERA pavement design manual volume I under "CEMENT AND LIME STABLIZED MATERIALS", if the amount of lime exceeds the ICL, the stabilized material will generally be non-plastic or only slightly plastic. This indicates that there is a lower limit of lime dosage below which mixing uniformity cannot be achieved with normal construction operations. According to Chulmin J. and Antonio B. (2008), the minimum percentage of lime is usually determined from the Eades and Grim procedure. The procedure is based on pH measurements. Indian Department of Transportation (2002) design guide recommends that the lime dosage necessary for chemical treatment of a subgrade is determined from the Eades and Grim procedures [1, 24]. The test specifies that a sufficient amount of lime is to be added to the soil to obtain a pH of 12.4 or equal to the pH of the lime itself. The PH value of the lime used is 12.36 and the result of PH of the soil-lime mixture was presented in the table 4.5 below.

Table 4.5 PH values of soil-lime mixture with different lime content

Lime content (%)	PH value
0	4.85
1	7.27
2	9.43
3	10.41
4	11.12
5	11.64
6	11.96
7	12.36
8	12.37
9	12.38

INDOT's design guide on the lime treatment of a subgrade specifies that the percentage of lime of 3 to 10 % is used for soil modification and stabilization. For soil modification, hydrated lime or quicklime and lime by-products are used within the range of 4 ± 0.5% and 5 ± 1% by weight of natural soil. The optimum lime content is the one associated with the maximum pH of the soil-lime mixture. But Queensland Gov. Dept. of Transport and Main Road, the stable pH value of the soil-lime mixture should be the same as the pH of the hydrated lime mixture. Also Central Material Testing Laboratory of the United Republic of Tanzania specifies, record the lowest hydrated lime content (HLC) where the PH just reaches a stable peak value, that is, a plateau where the PH values do not vary by more than 0.05 PH units over three successive soil-lime mixtures. Therefore, graph below was obtained from the measured PH of soil- lime mixture and it was plotted between PH and lime percentage

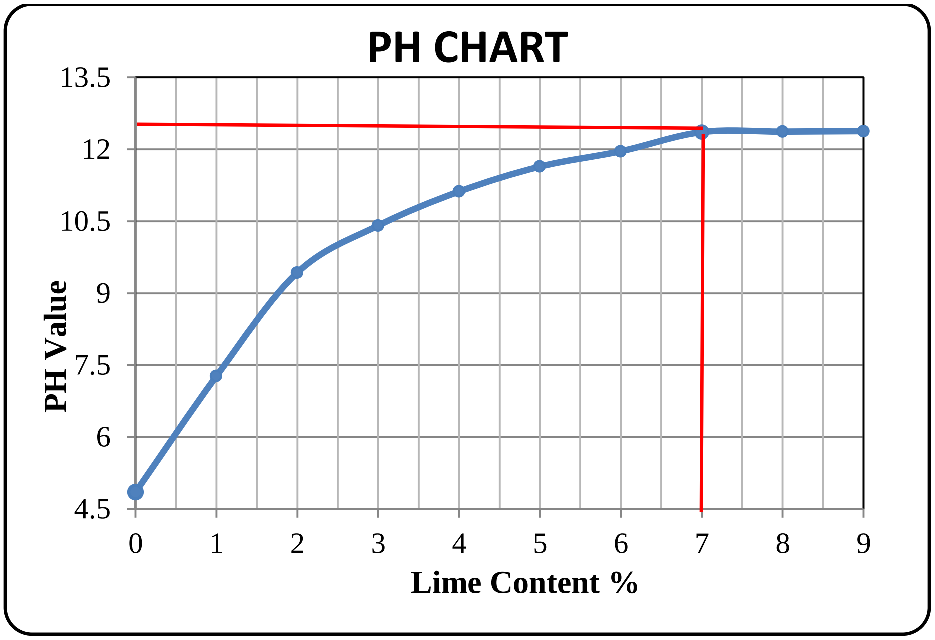

Figure 4.7 PH Chart of lime-soil mixture

The soil-lime pH test is performed as a test to indicate the soil-lime proportion needed to maintain the elevated pH necessary for sustaining the reactions required to stabilize a soil [1]. From the above graph, the lime content that of 7% satisfies the specifications of Eades and Grim procedures. This test specifies that a sufficient amount of lime is to be added to the soil to obtain a pH of 12.4 or equal to the pH of the lime itself. Also it fulfills the Central Material Testing Laboratory of the United Republic of Tanzania because of the variation of PH values over three successive soil-lime mixtures is within 5% PH units. As a result, 7% of lime was selected as optimum lime content for the study but in order to determine its effects above the optimum content and below the optimum content, the lime was used from 5% to 9% for different tests.

4.4. Properties of Lime Stabilized Soil

4.4.1 Atterberg Limits

One of the important and principal aims of the present study was to evaluate the changes of liquid limits, plastic limits and plasticity index with addition of lime to the soil sample. To achieve this objective, liquid limit and plastic limit tests were conducted on lime-soil mixtures according to consistency test of AASHTO T89 and T90, respectively.

Table 4.6: Atterberg limit test result of soil-lime mixture on different lime content

Lime content (%)	LL (%)	PL (%)	PI (%)	% Decrease of PI	ERA requirement of PI %	Remark
0	77.04	35.8	41.24	0	<30%	Poor
5	74.57	37.42	37.45	9.19		Poor
6	72.46	38.72	33.74	18.18		Poor
7	70.36	44.18	26.28	36.27		Satisfied
8	68.87	44.3	24.57	40.42		Satisfied
9	67.78	44.32	23.46	43.11		Satisfied

According to the results observed from the laboratory test, one can judge that the behavior of soil sample was changed from high plasticity soil to low plasticity soil. The Plastic Index (PI) is the parameter most commonly used to measure consistency changes of soils due to physicochemical effects produced by changes in water content. Little (1995) Reduction in plasticity translates into the improvement of workability and compactability of the soils. One of the ability of lime treatment is to reduce plasticity and improve workability (Eades and Dimond). Therefore, when the percentage of lime increased, plasticity index of the treated soil sample is significantly decreased but from the PH test, lime content of 7%, 8% and 9% were selected as initial lime content and the graph below shows significant improvement in PI the results are attached to appendix B.

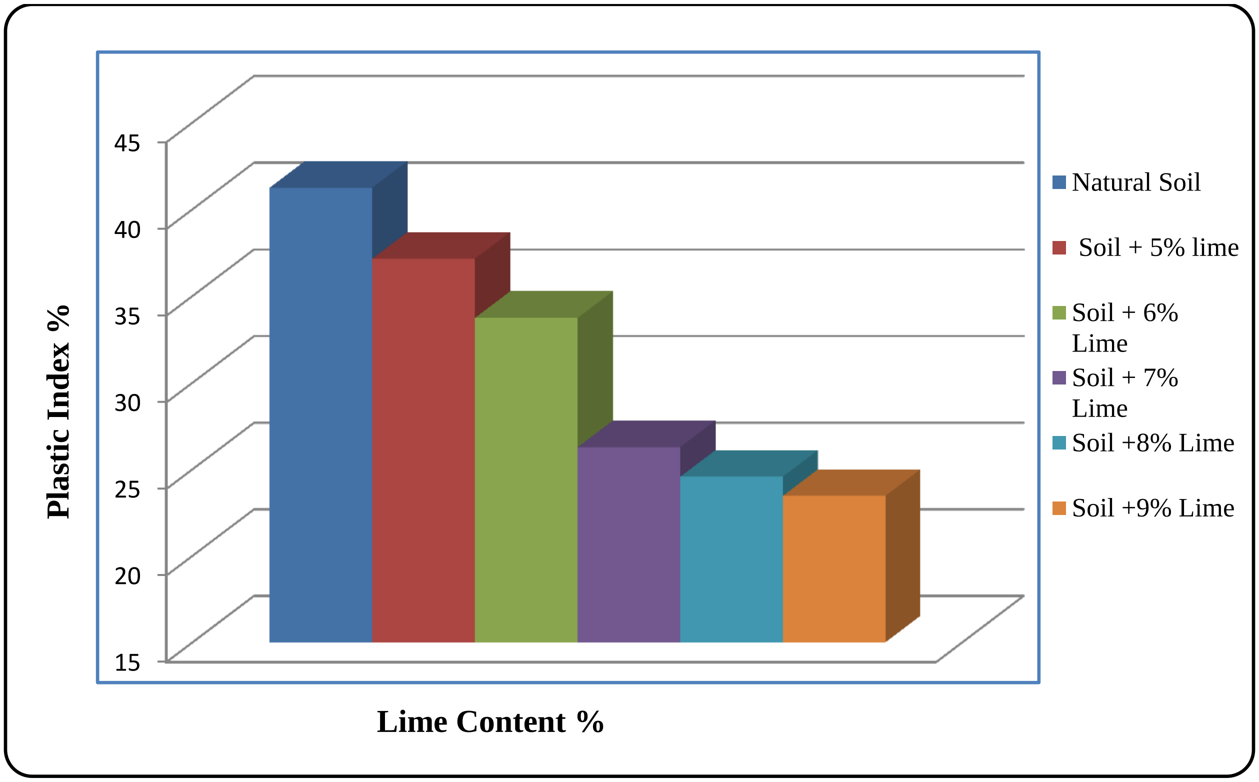

Figure 4.8: Plasticity index chart for Stabilized soil Sample

It is observed from the figure, as the dosage of lime increases, the PI of the lime-treated soil decreases. The Liquid limit decreases with slight changes on the soil from its natural value 77.04% to 67.78%. Changes to the plasticity of the soil are a result of the cation exchange resulting in particle flocculation and aggregation. This increases the effective particle size of the fine-grained soil resulting in a more silt-like material. This typically increases the plastic limit and decreases the liquid limit, Thompson (1967) and TRB (1987). In some instances the soil may even become non-plastic. For some soils the liquid limit may actually increase with lime concentration. Research tends to suggest that this is clay mineral dependant; Rowlands et al (1987), Cobbe (1988) and Thompson (1967) all reported increases in liquid limit in soils where illite was the predominant clay mineral. Even with an increase in liquid limit, the accompanying increase in plastic limit is always greater – thus resulting in a net reduction in the plasticity index of the soil.

Holtz (1969) investigated the effect of hydrated lime on the reduction of PI in active clays. Reduction in plasticity translates into the improvement of workability and compactability of the soils. The dosage of lime applied to treat a soil may be also determined based on reduction of Plastic Index and/or improvement of strength properties [12]. However, the lime content below the initial lime content from PH test do not shown significant improvement in PI, as a result they didn't fulfill the requirements of ERA.

4.4.2 Compaction Characteristics of Treated Soil

The moisture density relations are determined based on AASHTO T99-94. Tests were conducted with different percentage of lime. Moisture content versus dry density graph is plotted and the optimum Moisture Content (OMC) and Maximum Dry Density (MDD) are determined from the graph. Summarized results are tabulated in Table 4.8 below. The details of the test results are attached in Appendix B.

Table 4.7 Moisture Density Relation test results of lime Treated Soil

Mix No	% Soil	% Lime	MDD(g/cm^3)	OMC %
1	100	0	1.39	31.94
2	95	5	1.36	32.33
3	94	6	1.33	34.46
4	93	7	1.29	37
5	92	8	1.26	38.42
6	91	9	1.24	40.11

As observed from table 4.7 above, the MDD of untreated sample was observed to be 1.390g/cm^3. Even though the compaction curve is normal and the curve shifted to the right down ward in the case of treating the soil with lime, which also means additions of lime slightly decrease the MDD and increase the OMC of soil sample. Little (1995) Reduction in plasticity translates into the improvement of workability and compactability of the soils.

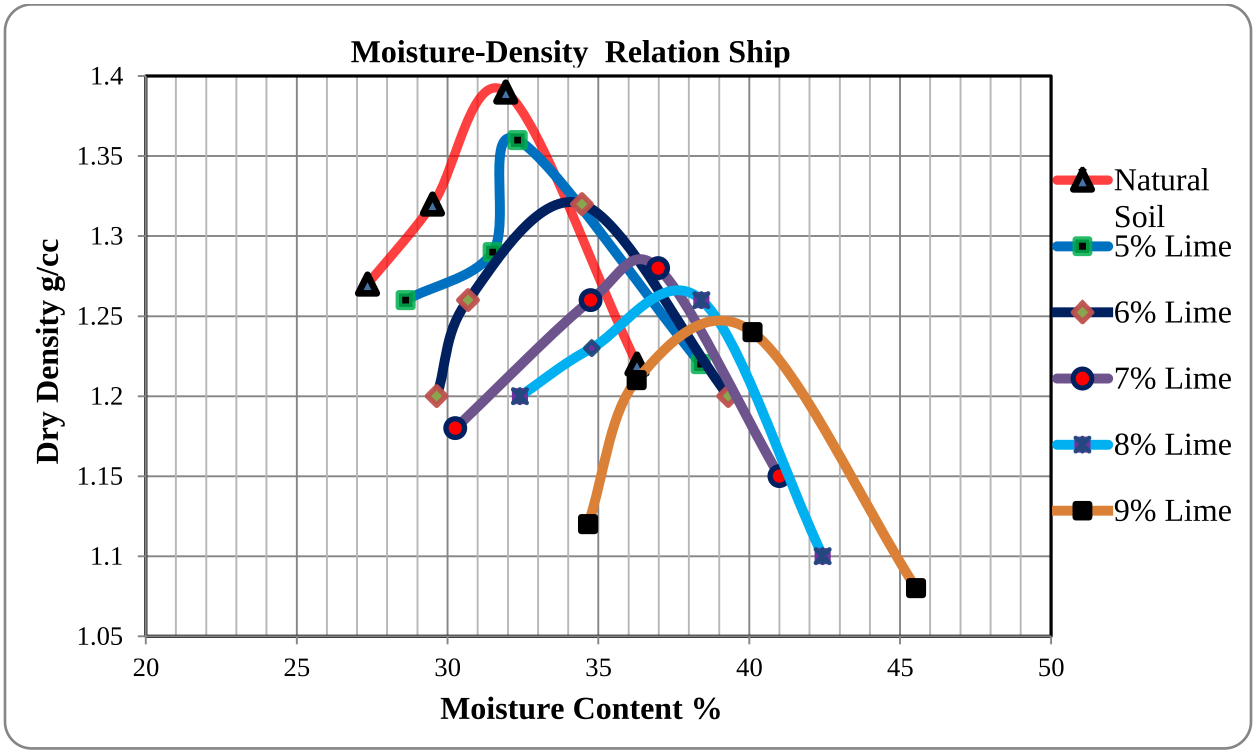

Figure 4.9 Dry-density and Moisture relation of lime treated soil

The addition of hydraulic binders alters the compaction characteristics of the host soil. The maximum dry density (MDD) decreases and the optimum moisture content (OMC) increases. Typically, the higher the concentration of binder, the greater the alterations to the compaction characteristics are. The OMC increases due to the hydration effect and the affinity for more moisture during this reaction process (Thompson (1967)).

Decreases in density are directly attributed to the flocculation/aggregation and the formation of weak cementitious products. Flocculation/aggregation of the soil offers greater resistance to densification at a given level of compactive effort. The net result is a reduction in the MDD Cobbe (1988) and Thompson (1967) [12].

The advantage of the increase in OMC and corresponding decrease in MDD of the soil is that it allowed compaction to be easily achieved with wet soil. Any adverse effect on strength due to reduction in density is unlikely to occur due to the expected substantial gain in strength of treated soils due to the pozzolanic properties of lime [6].

4.4.3 Effects of Lime on the CBR Value

CBR is a parameter which is used to measure the strength of subgrade soil. The CBR value was determined after soil samples have been soaked in water for 96 hours. That means if soil is stabilized using sufficient amount of stabilizer and hardening occurs, the soaking acts as an efficient means of curing providing hydration and preventing carbonation resulting in higher strength than can be achieved in the field. The soaked CBR test results for different percentage of the lime is summarized in the table 4.9. The details of the laboratory results are attached in Appendix B.

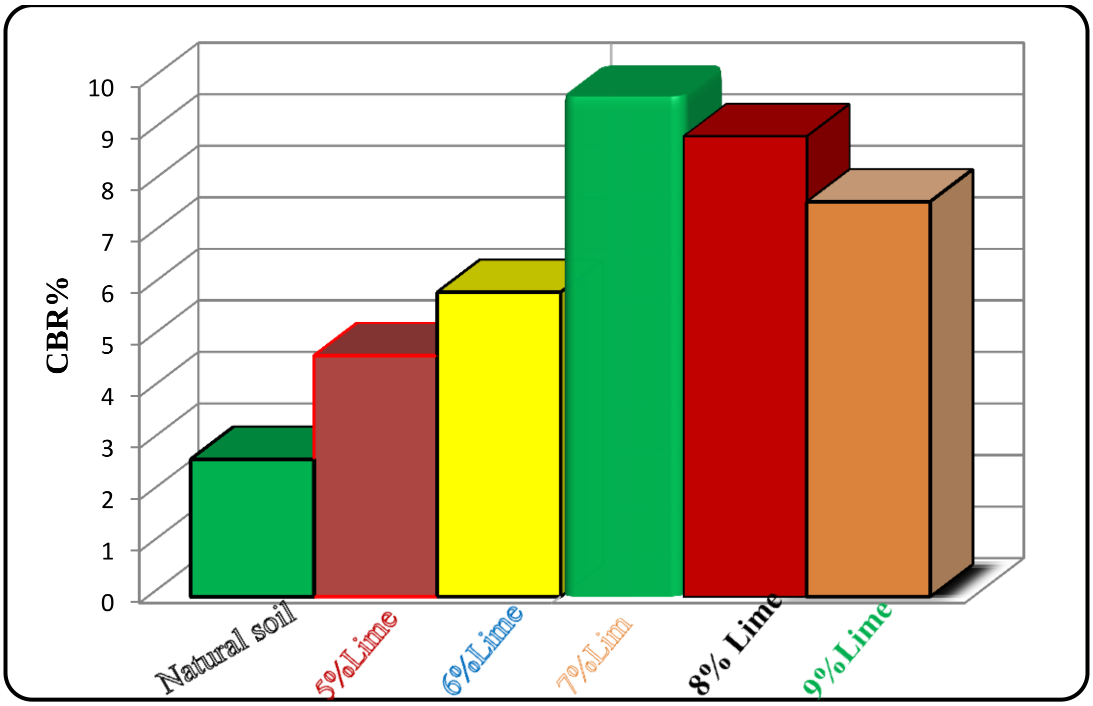

Figure 4.10: Graph of CBR test results of the soil with different lime content

After a soil has become modified, and providing sufficient available calcium and hydroxyl ions are present after modification, stabilization of the soil will occur. Stabilization involves the reaction of calcium ions, alumina and silica (either dissolved from the host material or present within the binder) and water. These ingredients form calcium silicate hydrate and calcium aluminate hydrate gels. These gels are similar to those produced in the production of concrete and will enhance the strength, bearing capacity and durability characteristics of the treated soil (Van Ganse (1973/74)).

The CBR value increases with the increase of lime percentage. The solubilities of silica and alumina are greatly increased in the stabilized clay soil with a resultant increase in the strength of the soils [Eades and Dimond et al]. As seen from table 4.9, CBR result showed significant improvement in strength as compared to untreated soil sample. The CBR value is found to increase appreciably with the increase in lime content. The maximum CBR value of 9.7% was found to occur for 7% of lime under soaked state. From soil-lime PH test, 7% is selected as optimum lime content at which the PH value is greater than 12 and similarly CBR value was maximum. This maximum improvement can confirm the relation between the PH value and the CBR values. One of the effects of cation exchange at the surface of clay particles is an increase of the pH of the pore water. The increase in pH facilitates the dissolution of alumina and silica from the clay minerals. In other words, silica and alumina can more easily be released from the clay mineral [12]. The silica and alumina react with the calcium from the added lime and creates pozzolanic compounds such as calcium-aluminate-hydrate (CAH) and calcium-silicate-hydrates (CSH). The pozzolanic compounds have cementing effects because they bind the soil structure together and increase the strength and/or stiffness of the soil [5].

The CBR value was slightly decreasing above the optimum lime content. The decrease in CBR at lime content of 8% and 9% may be due to extra lime that could not be mobilized for the reaction which consequently occupies spaces within the sample. This reduced the bond in the soil lime mixture[6].

Table 4.9 Summary of the CBR test results

<table>
<tr><th colspan="11">CBR Value (%)</th></tr>
<tr><th rowspan="2">% Lim</th><th colspan="2">10 Blows</th><th colspan="2">30Blows</th><th colspan="2">65Blows</th><th rowspan="2">CBR@ 95% MDD</th><th rowspan="2">Swell (%)</th><th rowspan="2">ERA Reqt.</th><th rowspan="2">Subgrade Class</th></tr>
<tr><th>2.54mm</th><th>5.08mm</th><th>2.54mm</th><th>5.08mm</th><th>2.54mm</th><th>5.08mm</th></tr>
<tr><td>0</td><td>2.24</td><td>2.01</td><td>2.61</td><td>2.22</td><td>2.81</td><td>2.47</td><td>2.66</td><td>2.51</td><td rowspan="6">> 3%</td><td>S1</td></tr>
<tr><td>5</td><td>3.28</td><td>2.62</td><td>4.23</td><td>3.86</td><td>6.03</td><td>5.72</td><td>4.67</td><td>2.3</td><td>S2</td></tr>
<tr><td>6</td><td>5.73</td><td>4.86</td><td>5.87</td><td>5.56</td><td>6.24</td><td>5.92</td><td>5.9</td><td>1.75</td><td>S2</td></tr>
<tr><td>7</td><td>8.06</td><td>7.37</td><td>8.64</td><td>8.31</td><td>10.27</td><td>9.62</td><td>9.7</td><td>1.47</td><td>S4</td></tr>
<tr><td>8</td><td>7.54</td><td>7.24</td><td>8.16</td><td>7.56</td><td>9.46</td><td>8.91</td><td>8.93</td><td>1.41</td><td>S4</td></tr>
<tr><td>9</td><td>6.26</td><td>6.03</td><td>7.85</td><td>7.26</td><td>8.12</td><td>7.74</td><td>7.66</td><td>1.37</td><td>S4</td></tr>
</table>

4.3.4 CBR Swell of treated soil sample

The swells of lime mixed with expansive soil is measured and determined from Soil with various percentage of lime was conducted on CBR tests. From these Swell measurements are taken at the time of soaking and after four days of soaking. Results are tabulated below in Table 4.10.

Table 4.10: Swell from CBR test

Mix No	% Lime	%Swell	Remark
1	0	2.51	Poor
2	5	2.3	Poor
3	6	1.75	Satisfied
4	7	1.47	Satisfied
5	8	1.41	Satisfied
6	9	1.37	Satisfied

As the results indicted above, mix proportion from 6% to 9% met the requirement specified by ERA pavement design manual as criterion for suitable material.

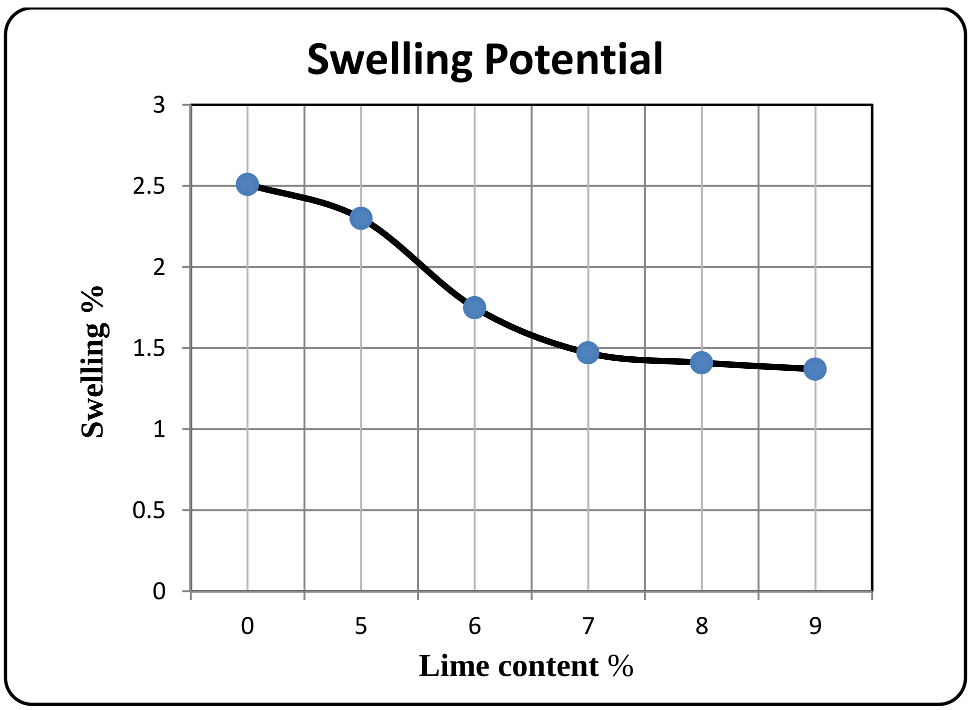

Figure 4.11 Swelling chart of lime treated soil

Expansive soils treated with different lime content showed reduction in CBR swell when compared to 2.52 of untreated soil. The practical effects of the treatment of a soil with lime was reduction of swelling and shrinking potentials of the soil by saturating the clay fraction with calcium ions and compressibility (Chulmin J. and Antonio B. (2008)), However, it was observed that swelling potential decreases with increasing in lime content. These reduced swell characteristics are generally attributed to increase in consolidation and settlement of soil with the reaction produced from lime.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The following conclusions are drawn from the present investigation on the basis of the laboratory test results of lime stabilized with the soft high clay soil:

- The soil used in this study, classified by the IS, USCS and AASHTO methods is of clay of high plasticity(CH).The value of soaked CBR at the optimum moisture content is about 2.66%. Hence the soil is required for stabilization before the construction of flexible pavement.
- The lime used for this study was prepared from naturally occurring limestone by using ideal hand tools. The chemical composition of this lime test result indicates the combined percent composition of its main oxides (CaO + MgO) was 85.13% which is above the minimum of (85%) specified by BS890 and good quality material.
- The addition of admixture with the soft sub-grade decreases the Maximum Dry Density and increases the Optimum Moisture content. The treatment of soil with the addition of admixture such as lime has a general trend of decrease in liquid limit, increase in plastic limit and decrease in plasticity index.
- In CBR test, there was an appreciably increase from the control value of 2.66% to 9.7% with different lime content. But from maximum CBR value, 7% lime was the optimum lime content for this study. However, all mix proportions satisfied the minimum requirements as per ERA specification used as a road subgrade material.
- Based on the above investigation, naturally occurring stabilizers appreciably improves the engineering properties of the soil. Also it plays significant role in stabilization and it optimizes suitability of natural stabilizer..
- Generally, the most parameters of ERA (2013) specification requirement were achieved and the Engineering properties of expansive soil were improved by lime in different mix-proportion.

5.2 Recommendation

For further study the following pointes are recommended:-

- ✓ This study was done for specific area and on specific stabilizers, it is recommended as more investigation shall be performed on different parts of the country by mixing with other stabilizers such as human hair fiber.
- ✓ The present study was conducted by taking limited parameter such as Atterberg limit, moisture density relation, CBR and CBR swell potential on stabilization by using locally prepared lime. It is recommended to test additional parameter like unconfined compressive strength and mineralogical tests to obtain more realistic test results.
- ✓ This is conducted through locally crushed lime using hand tools. It is recommended to adapt local crushing technologies to minimize construction time, demand and cost of industrial products

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APPENDEXIS A

Table 4.12 Natural moisture content of the soil

Natural moisture content of the soil			
Can number	1	2	3
Mass of moisture can (M.C)	17.44	17.76	18.05
Mass of moisture can + Mass of moist soil (Mcms)	104.28	97.44	94.52
Mass of Moisture can + Mass of oven dried soil (Mcds)	79.1	75.28	71.62
Mass of water (Mw)	25.18	23.16	21.9
Mass of dry soil (Ms)	61.66	56.52	54.57
Water Content(w) %	40.84	40.97	40.13
Average water content(w) %	**40.64**		

Table 4.13 Grain Size Analysis

Sieve size (mm)	mass of retain on each sieve(g)	Parentage of retained soil	cumulative % of retain soil	percentage of passing particle
9.5	0.00	0.00	0.00	100.00
4.75	6.06	0.61	0.61	99.39
2	18.05	1.81	2.42	97.58
0.85	6.70	0.67	3.09	96.91
0.425	9.61	0.96	4.05	95.95
0.3	5.87	0.59	4.64	95.36
0.15	9.94	1.00	5.64	94.36
0.075	11.30	1.13	6.77	93.23
Pan	930.10	93.23	100.00	0.00
Sum	997.6			

Table 4.14 Proctor Compaction test of Natural Soil

Test No.	1	2	3	4
Mass of sample (g)	2000	2000	2000	2000
Water Added(cc)	400	480	560	640
Mass of Mold+ Wet soil(g)(A)	3301.4	3484.4	3502.1	3394.2
Mass of Mold(g)(B)	1810.1	1816.9	1814.3	1808.4
Mass of Wet Soil(g)A-B=C	1529.28	1699.2	1727.52	1585.8

Volume of Mold cm^3(D)	944	944	944	944
Bulk Density g/cm^3 C/D=(E)	1.62	1.71	1.83	1.68
Container Code.	J41	3	G3T2	12
Mass of Wet soil+ Can(g)(F)	169.91	175.74	156.2	151.13
Mass of dry soil+ can (g)(G)	140.43	135.08	118.12	119.85
Mass of container(g)(H)	32.64	40.66	34.8	41.16
Mass of moisture(g)F-G=(I)	29.48	31.47	29.08	30.28
Mass of Dry soil(g)G-H=(J)	107.79	106.61	92.32	79.69
Moisture content % (I/J)*100=K	**27.35**	**29.51**	**31.94**	**37.99**
Dry Density g/cm^3 E/(1+K)*100	**1.27**	**1.32**	**1.39**	**1.22**

Table 4.15 Natural Soil Atterberg Limit Test Result

liquid limit				**Plastic limit**	
No of below	34	24	18		
Trial	1	2	3	1	2
Can	B8	3L	AA	Md1	A16
Wt. of can + wet soil	29.54	36.38	36.8	22.09	21.36
Wt. of can + dry soil	24.9	29.93	29.25	21.09	20.25
Wt. of container	18.73	19.61	19.31	18.08	17.36
Wt. of water	4.64	7.35	7.55	1	1.11
Wt. of dry soil, g	6.17	9.32	9.94	3.01	2.89
Moisture content,%	75.2	79.93	76	33.2	38.4
Ave Moisture content %	77.04			35.8	
Plastic Index	**41.24**				

Table 4.16 CBR test of natural soil

PENETRATION AND LOAD DETERMINATION OF NATURAL SOIL						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.54	0.37	2.81	0.34	2.62	0.29	2.24
5.08	0.48	2.46	0.44	2.23	0.39	2.01
CBR RESULT SUMMARY OF NATURAL SOIL						
MMDD	1.39					
Dry Density at 95% of MDD	1.32					
No of Blows	65	30	10			
CBR Values (%)	2.81	2.61	2.24			
DDBS g/cc	1.27	1.22	1.2			
CBR at 95% MDD	2.66					

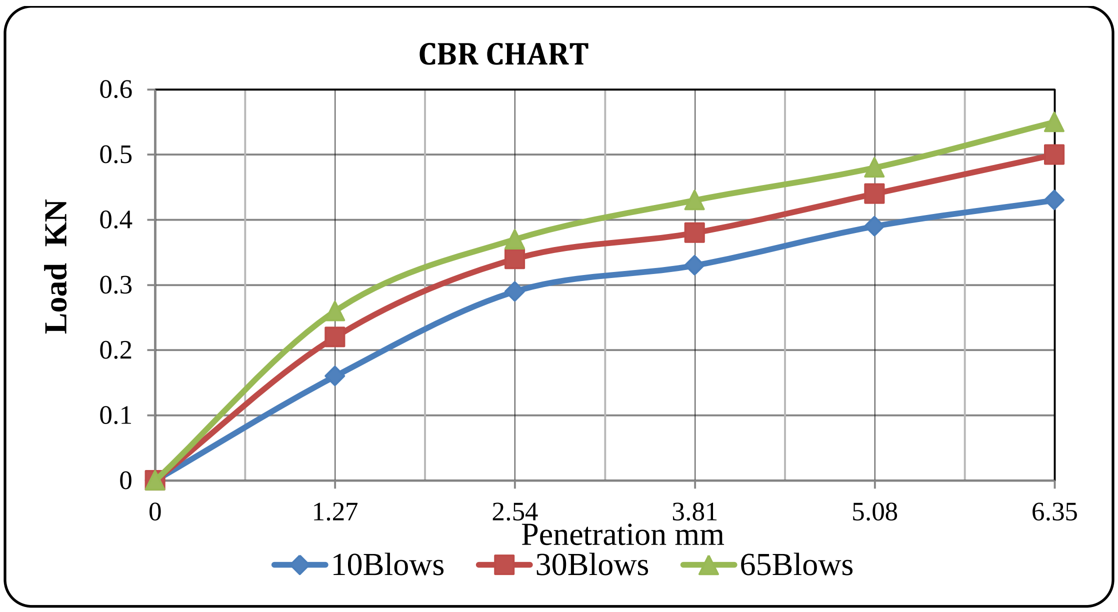


Figure 4.12 Load-penetration graph of natural soil

Appendix B

Table 4.17 Compaction characteristics of treated soil

Natural soil+5% Lime				
Test No.	1	2	3	4
Mass of soil (g)	1860	1860	1860	1860
Mass of lime	140	140	140	140
Water Added(cc)	450	530	610	690
M of Mold+ Wet soil(g)	3242.6	3377.2	3480.8	3345.2
Mass of Mold(g)	1816.6	1814.1	1816.6	1814.1
Mass of Wet Soil(g)	1426	1563.1	1664.2	1531.1
Volume of Mold cm^3	944	944	944	944
Bulk Density g/cm^3	1.63	1.7	1.81	1.7
Container Code.	2	G5-4	AD	J41
Mass of Wet soil+ Can(g)	152.3	183.14	118.8	193.98
M dry soil+ container(g)	126.2	152.12	95.2	144.76
Mass of container(g)	35	53.65	30.39	32.65
Mass of moisture(g)	26.1	31.02	21.6	47.22
Mass of Dry soil(g)	91.2	98.47	66.81	122.96
Moisture content %	28.62	31.5	32.33	38.4
Dry Density (g/c3	1.26	1.29	1.36	1.22

Soil +6%lime				
Trial No.	1	2	3	4
Wet Mold + wet soil(g)	4420	4505	4600	4575
Wet Mold(g)	2990	2990	2990	2990
Wet Soil(g)	1430	1415	1610	1585
Volume of Mold cm^3	944	944	944	944
Wet Density, (g/cm3)	1.56	1.5	1.78	1.68
Moisture Content Determination				
Trial No.	1	2	3	4
Can wt.(g)	7.6	5.7	5.49	5.32
Wet soil+can(g)	73.2	81.8	71.8	78.4
Dry soil+can(g)	59.2	65	55.8	59.7
Mass of moisture(g),	15	17.8	16.9	20.6
Dry soil(g)	50.6	58.1	49.4	52.4
Moisture content (%)	29.64	30.68	34.46	39.31

Dry Density(g/cm3)	1.2	1.26	1.32	1.2

Soil+ 7% lime				
Test No.	1	2	3	4
M of Mold+ Wet soil(g)	3192.34	3335.64	3402.52	3315.06
Mass of Mold(g)	1814.1	1815.8	1816.6	1814.1
M of Wet Soil(g)	1463.2	1595.36	1661.44	1469.08
Volume of Mold cm^3	944	944	944	944
Bulk Density g/c^3	1.53	1.69	1.76	1.63
Container Code.	T n	T6	Tm8	12
M of Wet soil+ Can(g)	163.1	168.4	127.2	139.4
M of dry soil+ can(g)	136.2	132.25	108.3	106.5
M of can(g)	43	36	41.2	34
M of moisture(g)	27.9	34.15	18.9	30.9
M of Dry soil(g)	92.2	98.25	51.1	75.5
M content %	30.26	34.75	36.98	41
D Density g/c^3	1.18	1.26	1.28	1.15

Soil+ 8%lime				
Trial No.	1	2	3	4
Wet Mould + wet soil(g)	4490	4605	4560	4550
Wet Mold(g)	2990	2991	2990	2989
Wet Soil(g)	1500	1615	1570	1560
Wet Density, (g/cm3)	1.59	1.66	1.75	1.57
Moisture Content Determination				
Trial No.	1	2	3	4
Can wt.(g)	6.39	17.96	17.74	17.75
Wet soil+can(g)	60.84	99.6	87.09	101.7
Dry soil+can(g)	49.5	80.53	68.56	77.68
Mass of moisture(g),	13.34	21.07	19.53	25.01
Dry soil(g)	41.11	60.57	50.82	58.93

Moisture content (%)	**32.4**	**34.78**	**38.42**	**42.44**
DD(g/cm3)	**1.2**	**1.23**	**1.26**	**1.1**

Soil +9% Lime				
Trial No.	1	2	3	4
Wet Mould + wet soil(g)	4425	4580	4595	4560
Wet Mold(g)	2990	2990	2990	2990
Wet Soil(g)	1435	1590	1605	1570
Wet Density, (g/cm3)	1.52	1.65	1.73	1.57
Moisture Content Determination				
Trial No.	1	2	3	4
Can wt.(g)	17.82	18.01	18.64	17.14
Wet soil+can(g)	101.7	100.6	106.94	105.1
Dry soil+can(g)	83.1	80.71	82.66	78.25
Mass of moisture(g),	21.59	21.98	25.28	27.51
Dry soil(g)	62.28	60.6	63.02	60.41
Moisture content (%)	34.66	36.27	40.11	45.53
Dry Density(g/cm3)	1.12	1.21	1.24	1.08

Table 4.18 Atterberg limit of treated soils

Natural Soil + 5% Lime					
liquid limit				**Plastic limit**	
No of below	31	22	19		
Trial	1	2	3	1	2
Can	BS	Aa	Pxc	bgt	ml
Wt. of can + wet soil	33.96	45.99	36.76	26.47	23.94
Wt. of can + dry soil	24.3	33.93	28.29	24.35	22.21
Wt. of container	17.08	18.3	16.7	19.03	17.27
Wt. of water	7.11	12.06	8.47	2.12	1.73
Wt. of dry soil, g	9.67	15.63	11.59	5.32	4.94
Moisture content,%	73.5	77.12	73.08	39.81	35.03
Ave Moisture content %	74.57			37.42	
Plastic Index	**37.15**				

Soil+ 6% lime					
liquid limit				**Plastic limit**	
No of below	31	19	22		
Trial	1	2	3	1	2
Can	1SW	M2	TY	bgt	ml
Wt. of can + wet soil	36.38	35.42	35.38	31.74	34.43
Wt. of can + dry soil	29.88	28.22	28.04	28.54	30.29
Wt. of container	21	18.2	17.9	20.16	19.78
Wt. of water	6.5	7.2	7.34	3.2	4.13
Wt. of dry soil, g	8.88	10.02	10.14	8.38	10.51
Moisture content,%	73.17	71.8	72.41	38.15	39.29
Ave Moisture content %	72.46			38.72	
Plastic Index	**33.74**				

soil+7% lime					
liquid limit				**Plastic Limit**	
Number of blows	32	23	19		
Trial	1	2	3	1	2
Can Code	TH1	N6	A7	TM01	M1
Wt. of can + wet soil	42.5	46.24	38.6	28.95	33.5
Wt. of can + dry soil	38.61	41.09	32.4	27.69	31.77
Wt. of container	32.91	33.9	23.6	24.67	27.78
Wt. of water	3.89	5.15	6.2	1.33	1.73
Wt. of dry soil, g	5.7	7.1	8.8	2.95	3.99
Moisture content,%	68.2	72.5	70.4	45.08	43.28
Ave Moisture content%	70.36			44.18	
Plastic Index	**26.28**				

Soil+ 8% lime					
liquid limit				**Plastic Limit**	
Number of blows	34	22	18		
Trial	1	2	3	1	2
Can Code	T1m	Ny6	tm71	PS01	La1
Wt. of can + wet soil	39.38	43.26	37.82	26.9	23.4
Wt. of can + dry soil	36.61	38.9	32.4	25.69	21.77

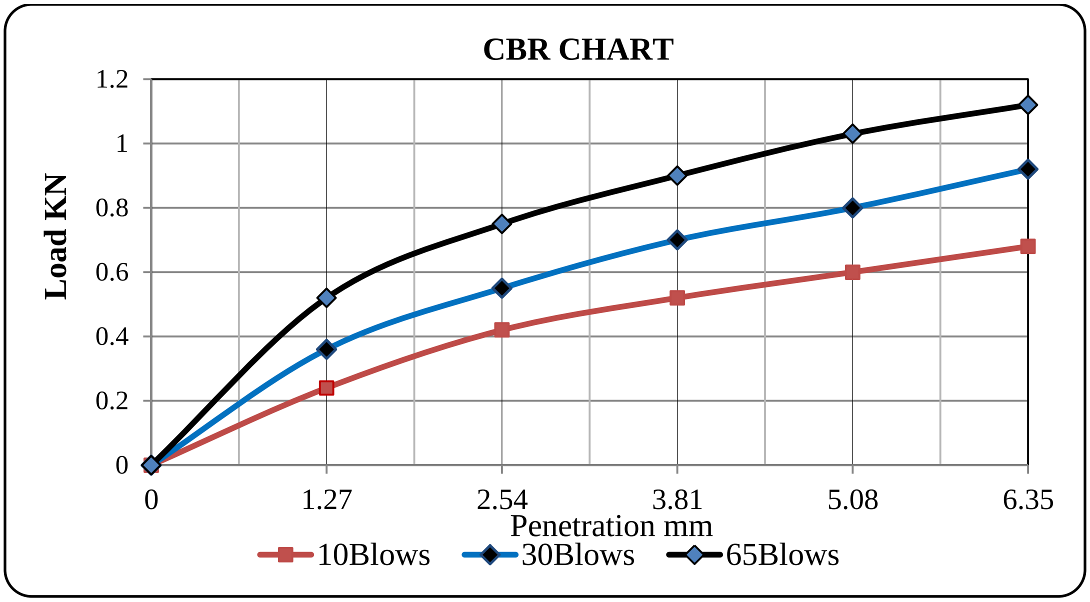
Wt. of container	32.46	32.5	24.39	22.8	18.09
Wt. of water	2.87	4.36	5.42	1.26	1.63
Wt. of dry soil, g	4.05	6.4	8.01	2.84	3.68
Moisture content,%	70.86	68.12	67.64	44.36	44.25
Ave Moisture content%			68.87	44.3	
Plastic Index	**24.57**				

Natural Soil +9% Lime					
liquid limit				**Plastic limit**	
No of below	31	22	19		
Trial	1	2	3	1	2
Can	Q1	ZL	Ny	Ds	Wz
Wt. of can + wet soil	31.13	32.8	33.54	22.07	24.7
Wt. of can + dry soil	26.73	27.65	28.74	20.25	22.4
Wt. of container	20.1	19.7	21.8	16	17.2
Wt. of water	4.4	5.3	4.8	2	2.1
Wt. of dry soil, g	6.63	7.8	6.94	4.08	5.3
Moisture content,%	66.31	67.94	69.11	49.01	39.62
Ave Moisture content %	67.78			44.32	
Plastic Index	**23.46**				

Table 4.19 CBR test results of treated soil

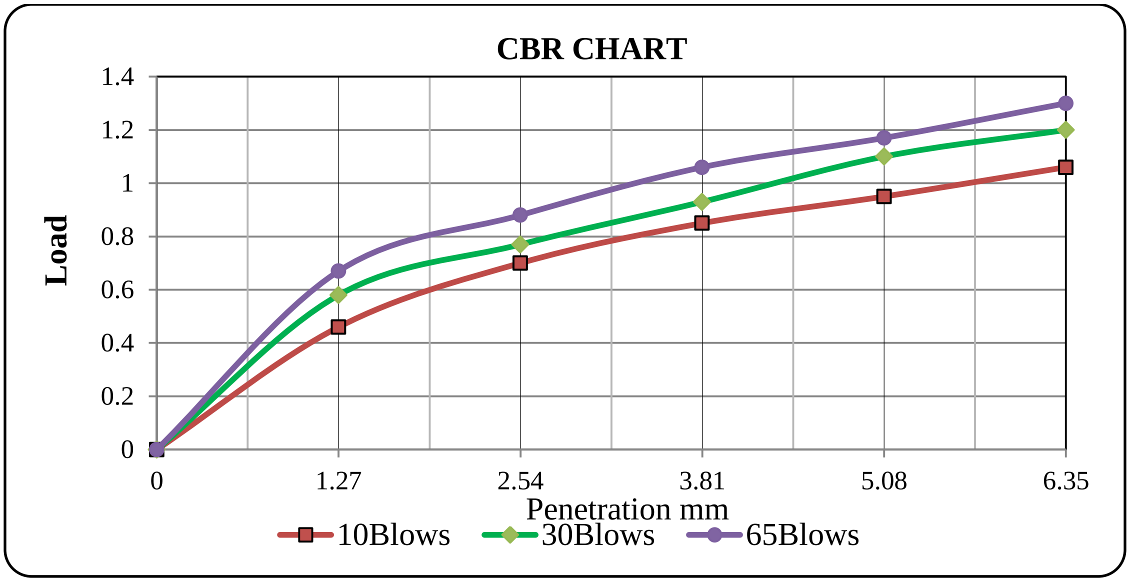
PENETRATION AND LOAD DETERMINATION OF 5%LIME + SOIL						
Penetration Data After 96-hours Soaking						
Penetration (mm)	**65-Blows**		**30-Blows**		**10-Blows**	
	Load (KN)	**CBR (%)**	**Load (KN)**	**CBR (%)**	**Load (KN)**	**CBR (%)**
2.54	0.79	6.03	0.55	4.23	0.43	3.28
5.08	1.13	5.72	0.76	3.86	0.51	2.62

CBR RESULT SUMMARY OF 5%LIME + SOIL			
MMDD	1.36		
Dry Density at 95% of MDD	1.29		
No of Blows	65	30	10
CBR Values (%)	6.03	4.23	3.28
DDBS g/cc	1.29	1.26	1.22
CBR at 95% MDD	4.67		



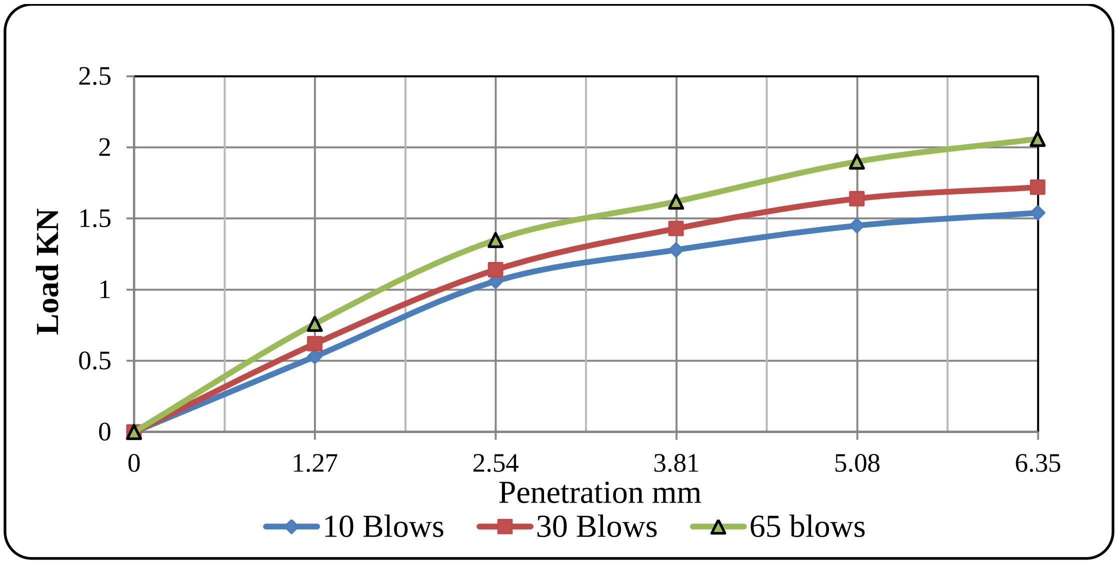
PENETRATION AND LOAD DETERMINATION OF 6%LIME + SOIL						
Penetration Data After 96-hours Soaking						
Penetration (mm)	**65-Blows**		**30-Blows**		**10-Blows**	
	Load (KN)	**CBR (%)**	**Load (KN)**	**CBR (%)**	**Load (KN)**	**CBR (%)**
2.54	0.82	6.24	0.77	5.87	0.7	5.31
5.08	1.17	5.92	1.1	5.56	0.96	4.86

CBR RESULT SUMMARY OF 6%LIME + SOIL			
MMDD	1.32		
Dry Density at 95% of MDD	1.25		
No of Blows	65	30	10
CBR Values (%)	6.24	5.87	5.31
DDBS g/cc	1.21	1.2	1.19
CBR at 95% MDD	5.9		



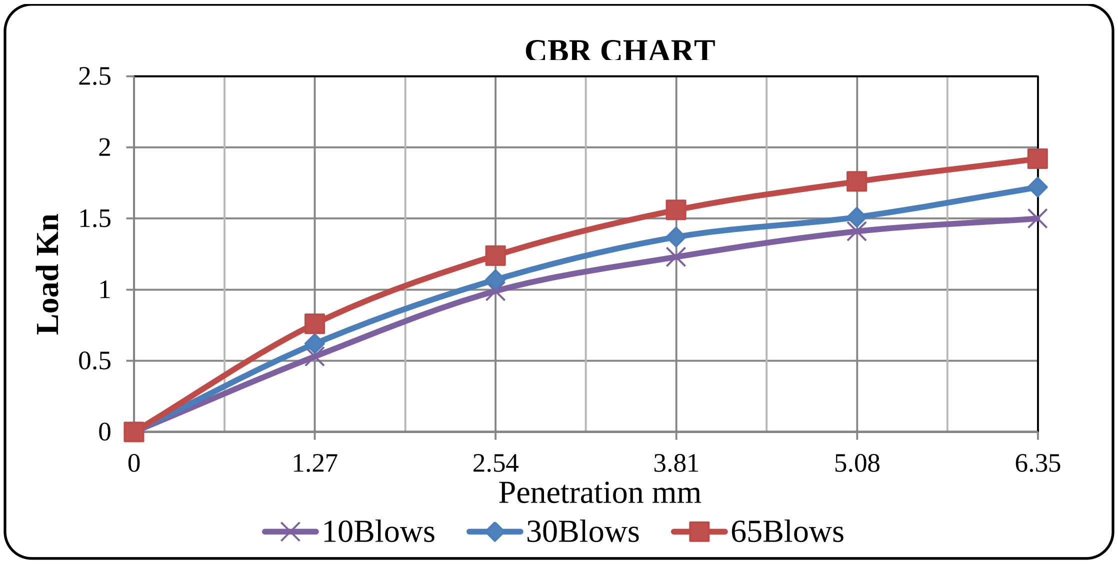
PENETRATION AND LOAD DETERMINATION OF 7%LIME + SOIL						
Penetration Data After 96-hours Soaking						
Penetration (mm)	**65-Blows**		**30-Blows**		**10-Blows**	
	Load (KN)	**CBR (%)**	**Load (KN)**	**CBR (%)**	**Load (KN)**	**CBR (%)**
2.54	1.35	10.27	1.14	8.64	1.06	8.06
5.08	1.9	9.62	1.64	8.31	1.45	7.37

CBR RESULT SUMMARY OF 7%LIME + SOIL			
MMDD	1.29		
Dry Density at 95% of MDD	1.21		
No of Blows	65	30	10
CBR Values (%)	10.27	8.64	8.06
DDBS g/cc	1.21	1.18	1.15
CBR at 95% MDD	9.7		



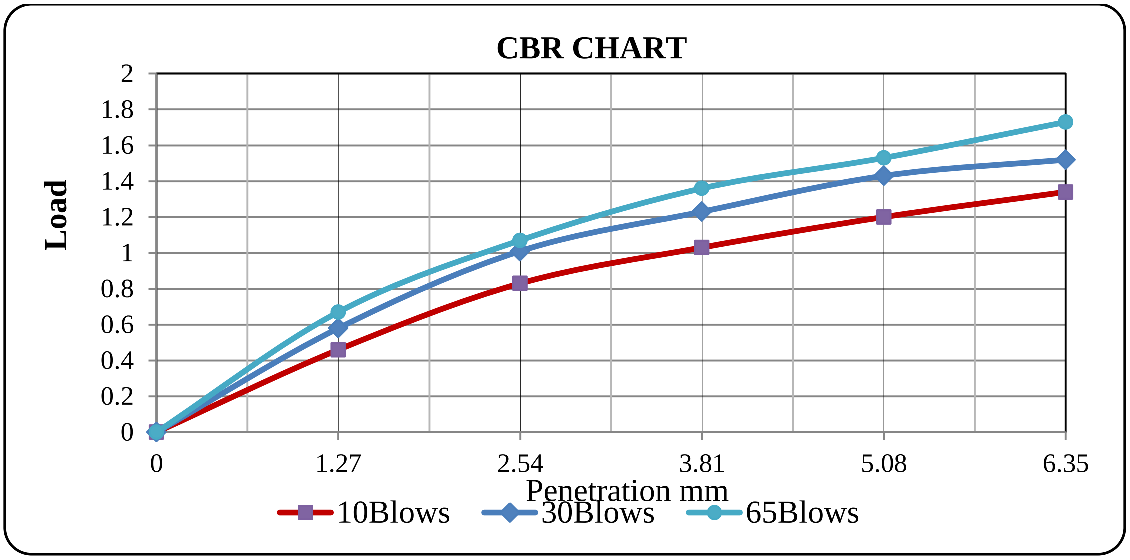
PENETRATION AND LOAD DETERMINATION OF 8%LIME + SOIL						
Penetration Data After 96-hours Soaking						
Penetration (mm)	**65-Blows**		**30-Blows**		**10-Blows**	
	Load (KN)	**CBR (%)**	**Load (KN)**	**CBR (%)**	**Load (KN)**	**CBR (%)**
2.54	1.24	9.46	1.07	8.16	0.99	7.54
5.08	1.76	8.91	1.49	7.56	1.43	7.24

CBR RESULT SUMMARY OF 8%LIME + SOIL			
MMDD	1.26		
Dry Density at 95% of MDD	1.19		
No of Blows	65	30	10
CBR Values (%)	9.46	8.16	7.54
DDBS g/cc	1.2	1.15	1.1
CBR at 95% MDD	8.93		



PENETRATION AND LOAD DETERMINATION OF 9%LIME + SOIL						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.54	1.07	8.12	1.03	7.85	0.83	6.36
5.08	1.53	7.74	1.43	7.26	1.2	6.07

CBR RESULT SUMMARY OF 9%LIME + SOIL			
MMDD	1.24		
Dry Density at 95% of MDD	1.17		
No of Blows	65	30	10
CBR Values (%)	8.12	7.85	6.36
DDBS g/cc	1.21	1.12	1.08
CBR at 95% MDD	7.66		



Figure; A Density test of lime and B Specific gravity test of soil

Fig; A crushing of lime, B &E measuring of soil and lime, C&D 5minute reactivity test on lime and finest test of lime respectively

Figure: PH measurements for soil and soil-lime mixture