



**ADDIS ABABA UNIVERSITY  
ADDIS ABABA INSTITUTE OF TECHNOLOGY  
SCHOOL OF CIVIL AND ENVIRONMENTAL  
ENGINEERING**

**MECHANICAL AND CHEMICAL STABILIZATION OF LATERITE  
GRAVEL WITH CRUSHED STONE AGGREGATE AND LIME  
FOR USE AS FLEXIBLE PAVEMENT MATERIAL**

**A Thesis submitted to the School of Graduate Studies of Addis Ababa University In  
partial fulfilment of the degree of Master of Science in Civil Engineering**

**(Road & Transport Engineering)**

**By**

Haftom Hagos

**Advisor**

Efrem G/Egziabher (Eng.)

July, 2017

Addis Ababa, Ethiopia



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## **Declaration**

I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor Eng. Efreem G/egziabher and has not been presented as a thesis for a degree in any other university. All sources of materials used for this thesis have also been duly acknowledged.

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Date        July, 2017

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## List of Abbreviations

AASHTO	American Association of Highway and Transportation Officials
ACV	Aggregate crushing value
AIV	Aggregate impact value
ASTM	American Society for Testing and Materials
BS	British Standard
CSA	Crushed Stone Aggregate
CBR	California Bearing Ratio
CIRIA	Construction Industry Research AND Information Association
ERA	Ethiopia Road Authority
GM	Grading Modulus
LL	Liquid Limit
LVSR	Low Volume Standard Roads
MDD	Maximum Dry Density
NP	Non Plastic
OMC	Optimum Moisture Content
OBC	Optimum Binder Content
PCA	Portland cement Association
PDM	Pavement Design Manual
PI	Plastic Index
PL	Plastic Limit
RH	Maximum relative humidity
SATCC	South African Transport and Communication Commission

TFV      Ten Per cent Fines Value

TRL      Transport and Road Research Laboratory

## ABSTRACT

Laterite gravels are marginal materials, which are locally available in southern and western part of Ethiopia, but not usually meet the specification to be used as pavement material for heavy traffic due to their gab grading and having higher plasticity.

The focus of this study is to use laterite gravels as flexible pavement materials through mechanical and chemical stabilization by Crushed Stone Aggregate and lime, respectively on the samples collected from western part of Ethiopia. Three sources were taken from two different routes and two of them have been subjected to Geo-chemical tests. In the first phase of investigation, mechanical stabilization was carried out at different proportion of Crushed Stone Aggregate blended with laterite gravel in order to determine the optimum Crushed Stone Aggregate with respect to strength. The second phase of investigation was focused on examining the possibility using of lime treated laterite gravel as lime stabilized pavement material with the addition of 4-16% of lime. In the last phase of investigation, mechanical as well as lime stabilization were carried out simultaneously in which 20-60% of Crushed Stone Aggregate and 4-16% of lime were blended with laterite gravel.

Based on the laboratory test results, the geo-chemical characteristics indicate that the blended laterite is pure laterite. It is shown that the laterite gravel fulfilled the CBR, LAA and TFV requirement when blended it with 40% and 60% Crushed Stone Aggregate for base course and 20% Crushed Stone Aggregate for subbase, regardless of having out values beyond the specification. The laterite stabilized with 12% lime fulfilled all of the requirements set for sub base materials. It was found out that the natural laterite gravel can be used for base course construction by stabilized it with a combination of 40% Crushed Stone Aggregate+8% lime and 60% Crushed Stone Aggregate+8% lime for CB2 and CB1, respectively.

Key words: Laterite gravel, laterite-CSA, laterite-lime, laterite-CSA-lime, stabilization, subbase , base course.

## 1. INTRODUCTION

### 1.1 General

Road construction pertaining to foster the expansion of transportation sector has very significant contribution to socio economic development of any country. Transportation uses for movement of people from place to place, utilize and mobilize resources, to improve agricultural production and market condition. It is also well recognized that transport plays a critical role in strengthening market linkage, advancing social relations and facilitating economic development. Due to this reason, the Ethiopian government has been investing a lot of billion birr per year for road construction.

It is in this context that road engineers are involved in trying to obtain materials that are readily available, less costly; and in cases that such materials are not readily available, to devise ways of improving deficiencies of the available materials. Improvement of such materials should be aimed at achieving high qualities in terms of strength, workability and durability at minimum construction costs as well as maintenance costs.

Marginal materials are abundant source of road building materials but do not always meet the quality requirements for flexible pavement materials and are frequently rejected in favor of expensive alternatives such as crushed stone aggregate. However, these alternatives are often not locally available and hence further cost is incurred as a result of transportation of large quantities in heavy vehicles. If, on the other hand, the properties of locally available marginal materials can be improved by stabilization techniques, then large financial and environmental benefits can be achieved. Laterite gravel is one of the marginal materials, which are locally available in many part of Ethiopian especially on the western part of Ethiopian, but not usually meet the specification to be used as a Flexible pavement material for heavy traffic.

Laterite soils are highly weathered and altered residual soils formed by the in situ weathering and decomposition of rocks under tropical condition; that occur mostly in tropical and sub-tropical regions with hot, humid climatic condition [6].

Laterite gravel has high percentage of clay fraction and consequently high plasticity. Some laterite gravels are also gab-graded with depleted sand size fraction, to contain variable percentage of fines, and to have coarse particles of variables strength which may break down in performance, limits their usefulness as pavement material on highly trafficked roads [8].

Hence, improvement of laterite gravel is unavoidable to be used as flexible pavement materials. The improvement methods usually employed including blending with a certain percentage of crushed aggregate, compaction and stabilization.

Therefore, to make safe plus economical pavement layer, stabilization of Laterite gravel using Mechanical and Chemical means is doubtless in order to be used the Laterite gravels as pavement material.

Various researches had been carried out to stabilize natural laterite with lime, cement, bitumen, cement-lime and cement-sand combination. A research by on Nigerian fine sand proved that it is possible to stabilize natural laterite gravel with a combination of cement-sand for flexible pavement construction [23]. Similarly a research on Nigerian laterite using lime, cement-lime and cement-geosta, confirmed the possibility of improving the strength of natural laterite to meet the flexible pavement requirement.

The purpose of this research is to stabilize the laterite gravel, with crushed stone aggregate and lime so it can be used as a flexible pavement material and to investigate the change in engineering properties of the laterite with certain percentage of stabilizing agent (crushed atone aggregate and lime).

## **1.2 Statement of Problem**

Laterite gravel encountered all over Ethiopia do not meet engineering properties to be used as flexible pavement materials in road construction because of their high plasticity and gradation as well as other criteria such as Los Angeles Abrasion (LAA), Soundness, Aggregate Crushing Value (ACV) etc. As a result of this problem, high quality aggregates with desirable engineering properties must be transported from place to place all over the country. The transportation of these large quantities of material has a negative impact on the environment and may lead to increase in project cost and time as it need for production and transportation of the selected materials. Besides the use of such high quality aggregate may lead to depletion of the natural resource.

Thus, we have to look for appropriate and economical soil stabilization techniques to minimize/avoid the environmental hazard, additional project cost and time required to transporting the selected materials.

## **1.3 Objective of the study**

### **1.3.1 General Objective**

The main objective of this study is to investigate the benefit obtained by the use of mechanical and chemical stabilization to render laterite gravel intended for flexible pavement construction in Ethiopia.

### **1.3.2 Specific Objectives**

Specifically, this research addresses the following objectives:

- To assess the possibility of using the locally available material as flexible pavement material.
- To improve the property of laterite gravel by increasing their USC, Resilient Modulus and durability.
- To evaluate the engineering properties of untreated and treated specimens with respect of Grading Limits, Plasticity Index, Modified Procter Compaction, California Bearing Ratio, Los Angeles Abrasion Value, Ten Per cent of Fines Value and Unconfined Compressive Strength.

- To determine optimum amount of stabilizing agent (lime and crushed stone aggregate) needed to attain the required properties of laterite gravel that can be used as flexible pavement material of paved roads.

#### **1.4 Research Questions**

- Is it possible to use the locally available natural laterite gravel for flexible pavement material?
- Do lime and crushed stone aggregate improve the strength, stiffness and durability of the laterite gravel?
- At what amount of the stabilizing agent do the laterite gravel can be used as flexible pavement material based on ERA specification?

#### **1.5 Scope of the Study**

The scope of the research is to investigate the effect of CSA and lime on laterite gravel for use as flexible pavement materials. Based on this background, laboratory tests are conducted on treated and untreated samples in order to make systematic comparison of the differences in the test results. In addition, some of the samples have been checked for their degree of laterization by determining the oxide contents.

#### **1.6 Organization of the Thesis**

This thesis has been organized and presented in six Chapters. In the first Chapter a general description on the use of stabilized locally available natural gravels has been made. Furthermore; the back ground and objective have also been discussed in this introductory part of the thesis. The second Chapter gives a brief literature review which discusses about formation, classification, Engineering properties and uses of laterite gravel as pavement material, and Function, characteristics and stabilization of granular materials. The third chapter presents a description of the study area as well as methodology of the study. Chapter four includes the laboratory test results, analysis and discussion. In this chapter, the different laboratory test results have been thoroughly conducted and analysed. Chapter five includes the mechanistic evaluation of layer response and performance of stabilized materials. Finally, conclusions and recommendations drawn from the research are discussed in chapter six. Following chapter six, reference materials used in this thesis work are appropriately sited and listed. The thesis ends with appendices which contain detail experimental results of laboratory investigation.

## 2. LITERATURE REVIEW

### 2.1 Marginal Materials

#### 2.1.1 Definition

Marginal materials are materials which do not in their present form possess quality levels as defined by current highway standards sufficient for their use as various pavement structural components including surfaces, bases, and/or sub bases [14].

According to The Permanent International Association of Road Congresses (PIARC) as cited by [9] has defined non-standard or marginal materials as:

"...any material not wholly in accordance with the specification in use in a country or region for normal road materials but which can be used successfully either in special conditions, made possible because of climatic characteristics or recent progress in road techniques or after having been subject to a particular treatment."

#### 2.1.2 Description and Classification

Marginal granular materials that could be considered for use in the upper pavement layers can effectively be grouped within a five tier system as shown in Table 2.1[9]. The Table identifies these five main groups as:

**Group 1: Hard Rocks:** usually comprising materials that require crushing and processing but retaining properties that result in the material does not fully meeting the requirements of a crushed stone base.

**Group 2: Weak rocks:** materials derived from weakly cemented, poorly consolidated or partially weathered parent deposits.

**Group 3: Natural Gravels:** transported and residual soils and gravels not meeting the minimum material standards for natural gravel road base.

**Group 4: Duricrusts:** indurated or partially indurated soils not meeting the minimum material standards for natural gravel road base.

**Group 5: Manufactured materials:** include a range of man-made materials that could effectively be re-processed as granular pavement materials.

Typical examples of materials that would commonly be associated with each of the groups are shown in Table 2.1.

Table 2.1: Marginal Materials Groups [9]

Groups		SUB GROUPS		Important Examples
I	Strong rocks			<ul style="list-style-type: none"> <li>✓ Foliated Metamorphic Rocks</li> <li>✓ Crystalline Basic Igneous &amp; Metamorphic Rocks</li> </ul>
II	Weak rocks	II a	Inherently Weak or Poorly Consolidated Rocks	<ul style="list-style-type: none"> <li>✓ Marls and Weak Limestones</li> <li>✓ Weak Conglomerates, Sandstones, and Volcanic Tuffs</li> <li>✓ Shale, Siltstone and Mudstone Deposits</li> </ul>
		II b	Partially Weathered and/or Highly Fractured Rocks	<ul style="list-style-type: none"> <li>✓ Fractured /Weathered (Rippable) Limestones</li> <li>✓ Weak Volcanic Agglomerates and Breccias</li> <li>✓ Other partially Weathered Rocks</li> </ul>
III	Natural Granular Deposits	II a	Transported Soils and Gravels	<ul style="list-style-type: none"> <li>✓ Alluvial Sand &amp; Gravel Deposits (including river bed, river terrace, shore and fan deposits)</li> <li>✓ Alluvial, Clayey Sand, Aeolian Sand and Colluvial Deposits</li> <li>✓ Volcanic Scoria (Cinder) Gravels and Volcanic Pumice/ Ash Deposits</li> </ul>
		II b	Residual Soils and Gravels	<ul style="list-style-type: none"> <li>✓ Quartz Gravels</li> <li>✓ Weathered Granite / Gneiss</li> <li>✓ Other Residual Gravelly Soils</li> <li>✓ Clayey Sand Deposits</li> </ul>
IV	Duricrust (Pedogenic Gravels)			<ul style="list-style-type: none"> <li>✓ Laterite Deposits and Silcrete Deposits</li> <li>✓ Calcrete Deposits (including Calc Tufa &amp; Caliche)</li> </ul>
V	Manufactured Materials			<ul style="list-style-type: none"> <li>✓ Bricks and Demolition Waste (Concrete and Brick)</li> <li>✓ Industrial By-products &amp; Waste Material</li> <li>✓ Products(i.e. blast furnace slag, mine tailings, demolished asphalt pavement)</li> </ul>

### **2.1.3 Usage of Marginal Materials for Highway Construction**

A primary requirement for increased use of marginal materials in highway construction is the willingness to take a calculated risk on paving materials which may not have “proven track records”. This is not to say that marginal materials should be used without knowledge of their properties and characteristics. However, evaluation of the marginal material’s properties and characteristics with existing tests and specifications may be too critical and thus unrealistic. [14].

Marginal materials derived from various sources have been used in road construction either routinely or on an experimental basis for many years by improving them with the addition of a stabilising agent or the use of these materials in their natural state.

In Ethiopia marginal materials have been successfully used in for pavement layers even though their use is restricted to low volume roads (T1 - T2). Some of these materials are lateritic soils, calcareous, and volcanic rocks [17].

## **2.2 Characteristics of Laterite**

### **2.2.1 General**

The soil name "laterite" was coined by Buchanan in India, from a Latin word "later" meaning brick. Laterite is a residual of rock decay that is red or reddish in colour. Soils under this classification are characterized by forming hard, Impenetrable and often irreversible pans when dried. However, there is confusion in the use of the term, because a variety of materials with many types of compositions and various origins have been called laterites, ranging from iron capping’s to the zonal soils of the humid tropics and the whole weathered profile beneath a laterite of Buchanan's meaning to the iron-rich breccias and slope wash accumulations. Because of this confusion, most workers now prefer to use the definitions based on hardening, such as "Ferric" for iron-rich cemented crusts, "Alcrete" or Bauxite for Aluminium-rich cemented crusts, "Calcrete" for Calcium Carbonate-rich crusts, and "Silcrete" for silica rich cemented crusts [40, 12].

[6] Describes laterites as highly weathered and altered residual soils formed by the in-situ weathering and decomposition of rocks under tropical condition. The three major agencies of weathering being physical, chemical and biological processes.

In the process the parent rock and rock minerals break down, releasing internal energy and forming soils having lower internal energy which are more stable. Physical processes increase surface area so that chemical attack increases. Biological weathering includes both physical and chemical actions.

Climate and topography influence the rate of weathering. Physical weathering is more predominant in dry climates while the extent and rate of chemical weathering is largely controlled by the availability of moisture and temperature. Topography on the other hand, controls the rate of weathering by partly determining the amount of available water and the rate at which it moves through the zone of weathering. It also controls the effective edge of the profile by controlling the rate of erosion of weathered material from the surface. Thus deeper profiles will generally be found in valleys and on gentle slopes rather than high ground or steep slopes [6].

Other definitions have been based on the silica ( $\text{SiO}_2$ ) to Sesquioxides ( $\text{Fe}_2\text{O}_3 + \text{Al}_2\text{O}_3$ ) ratio. In laterites the ratios are less than 1.33. Those between 1.33 and 2.0 are lateritic soils, and those greater than 2.0 are indicative of non- lateritic, tropical soils [6].

Laterites are rich in sesquioxides (secondary oxides of iron, aluminium or both) and low in bases and primary silicates but may contain appreciable amounts of quartz and kaolinite. Due to the presence of iron oxides lateritic soils are red in colour ranging from light through bright to brown shades.

### **2.2.2 Formation and Occurrence**

Laterites are formed as a result of intense chemical tropical weathering of parent rock having the following effects. The parent rock is changed chemically by being enriched with iron and Aluminium oxides. The clay mineral type becomes mainly kaolinite and the silica content is reduced. This process produces yellow, ochre, red or purple materials, while the typical red of the iron oxide, haematite, dominates many of the soils.

Laterite formation requires particular condition which concentrates the iron- and aluminium-rich weathering products sufficiently to allow concretionary development, resulting in a cemented horizon within the weathering profile. Thus three phases of action are necessary to produce a concretionary laterite [8].

- a. Tropical weathering to produce the mineral of laterite
- b. Formation of discrete horizon
- c. Concretionary development within the horizon.

**a) Tropical Weathering process and Weathering profile**

Disintegration and decomposition are the two major processes of weathering. The former is the physical breakdown of the rock without any chemical action, and decomposition, which is the chemical alteration of the primary minerals into secondary and residual products. These processes are highly influenced by the regional climatic zones. Laterite occurs mostly in the tropics and sub-tropical regions with hot and humid climatic conditions. In the tropical and sub-tropical regions, soil temperature is high throughout the year. For every 10degree rise in temperature the rate of chemical reaction is doubled, meaning a rate of chemical weathering four times faster in the tropics than in the temperate regions of the world and twice as fast as the sub-tropical regions [8].

Figure 2.1 below shows the relationship between climate and the formation of deep weathering profiles. It can be seen from the figure that there is significant chemical weathering and formation of clay minerals at the tropics where there is high rate of temperature and precipitation.

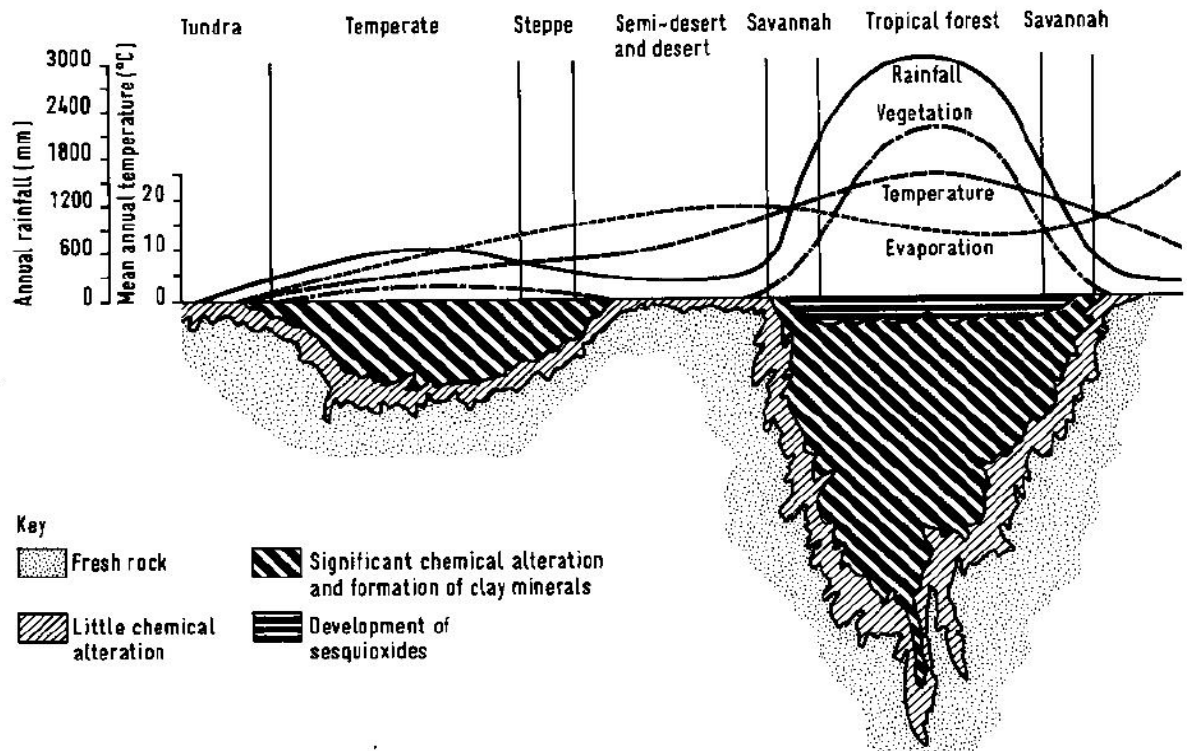


Figure 2.1: Schematic relationships between climate and weathering [8]

### **b) Formation of Discrete Horizon**

For the concretionary development of true laterite to take place, the process of formation of a weathering profile needs to undergo by the concentration of the weathering products within the residual soil. Open jointed rocks near the surface collect surface run off which eventually penetrates through joints and fractures. This penetration leads to the isolation of blocks of fresh rocks surrounded by the decomposed material, ultimately producing a residual soil.

Alteration starts on the surface of fractures, with the decomposition of the least parent material. Crystalline rock with low silica content such as gabbro, dolerite and basalt contains less stable, therefore easily weatherable minerals such as olivine, pyroxene and calcic plagioclase. Olivine alters first to serpentine and then to montmorillonite. Pyroxene to chlorite, calcite or montmorillonite; and feldspar to kaolinite.

The silica rich rock such as granite, gneiss and rhyolite contain more stable minerals such as quartz, potash and feldspar with micas and amphiboles. Feldspars weather to kaolinite, micas to montmorillonite then to kaolinite.

Iron and aluminium oxides are the main component of laterites. The iron oxide is usually present as haematite and the aluminium is present as its hydrated oxides, gibbsite and diaspora. It's due to the presence of iron oxide that lateritic soil get there red colour which ranges from light through bright to brown shades.

There are different factors that affect the development of tropical soils, thus it's weathering process. These are parent rock, climatic condition, topography and drainage. There are two aspects of parent rock that affect the development of laterites. One is the availability of iron and aluminium minerals, which are readily available in the basic rock and the other is the quartz content of the parent rock. Quartz remains as quartz grain in the weathering products because of its resistance and no mineralogical changes occur unless subjected to slow solution of leaching.

The other factors that affect the formation of laterites are topography and drainage. The amount of water moving downward through the weathering zone is basically controlled by the slope angle. On steep slopes run-off is greater than infiltration; erosion is active; and conditions are generally not suitable for the development of deep weathering. Conversely on flatter slopes run off is not so marked; only limited erosion takes place; and long

uninterrupted periods of weathering can occur, producing deep soil profiles. On level ground, however, where drainage is impeded and the ground is waterlogged, black montmorillonite soils dominate at the expense of red soils [8].

The formation of laterites requires hot and humid climatic condition where a mean annual temperature of around 25<sup>0</sup>C is required for their formation, and in seasonal situation there should be a coincidence of the warm and wet periods. If there is high rainfall during the cold season, laterites do not develop freely. The minimum annual rainfall required for laterite formation is generally at least 750 mm. The higher the rainfall above this value, the greater is the leaching effect thus reducing the silica/sesquioxides ratio as a result increases the degree of laterization [8].

### C) Process of Concretionary Development

The development of concretion requires sufficient concentration of hydrated oxides of iron and aluminium for cementation or precipitation growth to start. Soil scientists have called the uncemented horizons of material, plinthite. Because this horizon is present, concretionary development is possible; therefore the material can be called laterite [8].

The concentration of the materials in laterites depends on the relative mobility of silica and iron/aluminium oxides under certain physical and chemical conditions. Two processes are vital to cause a hardening or concretionary developments; chemical precipitation and loss of water of dehydration. The physical condition in these processes develop may be provided by a fluctuating ground water level, which causes reducing and oxidizing conditions alternately. Precipitation and dehydration can only take place by the lowering of the ground water level [8].

Based on the above discussion the three major processes responsible for the formation of laterites can therefore be summarized as follows:

**Decomposition:** physico-chemical breakdown of primary minerals and the release of constituent elements (SiO<sub>2</sub>, Al<sub>2</sub>O<sub>3</sub>, Fe<sub>2</sub>O<sub>3</sub>, CaO, MgO, K<sub>2</sub>O, Na<sub>2</sub>O, etc), which appear in simple ionic forms.

**Leaching:** removing of combined silica and bases and the relative accumulation or enrichment of oxides and hydroxides of sesquioxides which is called laterization.

The level to which the second stage is carried depends on the nature and extent of the chemical weathering of the primary minerals. Under conditions of low chemical and soil-forming activity, the physico-chemical weathering does not continue beyond the clay-forming stage, and tends to produce end products consisting of clay minerals predominantly represented by kaolinite and occasionally by hydrated or hydrous oxides of iron and aluminium.

**Desiccation (dehydration):** dehydration (either partial or complete) alters the composition and distribution of the sesquioxide rich materials in manner which is generally not reversible upon wetting. Dehydration also influences the formation processes of clay minerals. In case of total dehydration, strongly cemented soils with a unique granular soils structure may be formed [6]. Dehydration may be caused by climatic changes, upheaval of the land, or may also be induced by human activities, for example by clearing of forests.

### 2.2.3 Regional Distribution

Regional distribution of laterite is mainly governed by the world's climatic zones. Laterites cover vast areas in the tropical countries with intermittently moist climate. The six main regions of the world in which laterites occur are Africa, India, South East Asia, Australia, Central and South America. However, it should be noted that because of changes in climatic zone in the geological past, laterites can also be found in areas outside the tropics [8].

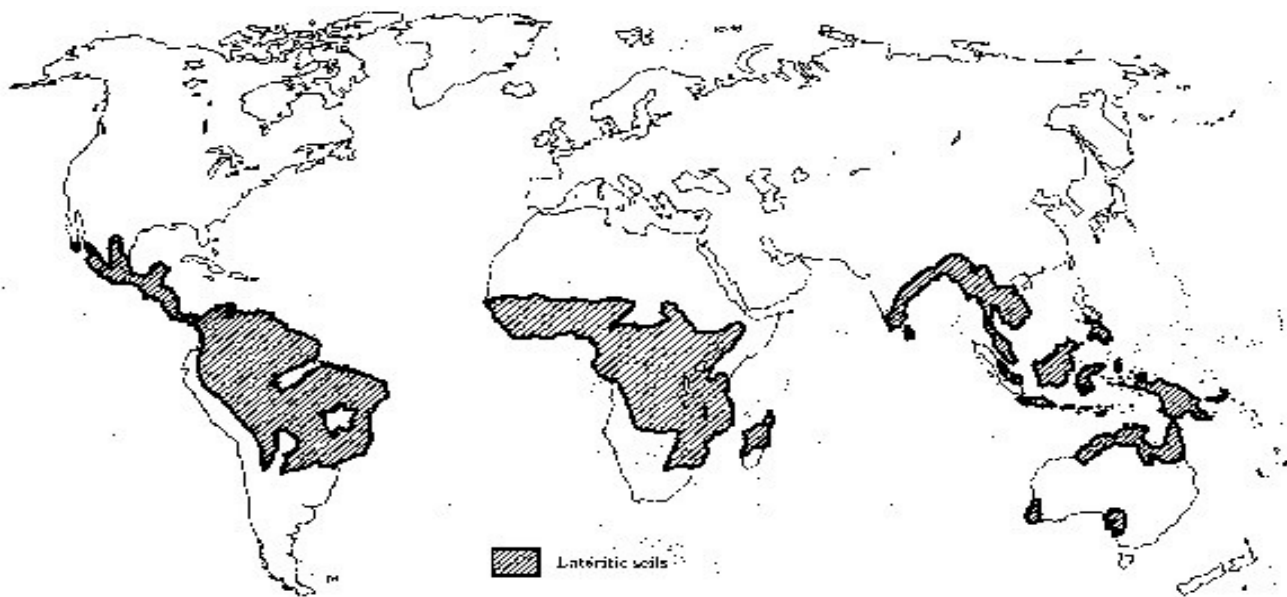


Figure 2.2: World-wide distributions of laterite soils [22, 42].

#### 2.2.4 Classification of Laterites

Laterites have specific features or characteristics that are not adequately covered by conventional methods of soil classification such as the Unified Soil Classification System. Among these features are the following [6]:

1. The unusual clay mineralogy of some tropical and subtropical soils results in characteristics that are not compatible with those normally associated with the group to which the soil belongs according to existing systems such as the Unified Soil Classification System.
2. The soil mass in-situ may display a sequence of materials ranging from a true soil to a soft rock depending on the degree of weathering, which cannot be adequately described using existing systems based on classification of transported soils in temperate climates.
3. Conventional soil classification systems focus primarily on the properties of the soil in its remoulded state; this is often misleading with residual soils, whose properties are likely to be most strongly influenced by in situ structural characteristics inherited from the original rock mass or developed as a consequence of weathering.

A classification system of residual soils based on mineralogical composition is suggested by Wesley L.D and Irfan T.Y. [6]. This system classifies residual soils into three groups.

1. Group A: Soils without a strong mineralogical influence.
2. Group B: Soils with a strong mineralogical influence deriving from clay minerals also commonly found in transported soils.
3. Group C: Soils with a strong mineralogical influence deriving from clay minerals only found in residual soils.

Group C is further grouped into three sub-groups according to clay minerals of soils. These are:  
Sub-group (a) Halloysitic soils  
Sub-group (b) Allophanic soils  
Sub-group (c) soils influenced by the presence of sesquioxides

Lateritic soils lay under group c, sub-group (c). The engineering properties of soils under this group are highly influenced by the presence of sesquioxides. Sesquioxides appear to act as cementing agent which bind the other mineral constituents into clusters or aggregations. The hard concretionary materials are formed as a result of sufficient concretion of sesquioxides [6].

Another classification system is also given by D' Hoore. This system broadly differentiates soils on a genetic basis, determined by soil forming factors. Three main units are used for the description and classification of red tropical soils [8].

**Ferruginous soils** show a marked separation of iron oxide, either leached out of the profile or precipitated within the profile as concretions. There may be a high proportion of weatherable primary minerals remaining. Kaolinite is the dominant clay mineral. These soils are generally found in areas with under 1850mm rainfall a year and pronounced dry seasons.

**Ferrallitic soils** are generally deep, with only slightly differentiated horizons. Kaolinite is the dominant clay mineral; and they contain free iron oxides and hydrated oxides of aluminium. They generally occur in more humid areas with more than 1500mm rainfall per year.

**Ferrisol soils** have profiles similar to ferrallitic soils, but with very few weatherable minerals remaining. The entire clay size fraction comprises kaolinite and amorphous oxides of iron and aluminium. Ferrisol tend to develop at deeper levels, because of surface erosion, and occur in regions of between 1250 and 2750mm rainfall per year. According to Morine W.J. and Todor P.C., Ethiopian laterites fall under this group [26].

On the other hand, Nascimento et al. (1959) have suggested an interesting lithological Classification of laterite as follows; [26]

Lateritic clays	< 0.002 mm
Lateritic silts	= 0.002-0.06mm
Lateritic sands	= 0.06-2 mm
Lateritic gravels	= 2-60 mm
and curisasse	> 60 mm

Moreover, Anthony has distinguished the following main types and sub-division of laterite [26]:

- I. **Massive laterite:** possesses a continuous hard fabric, subdivided-in to
  - a. Cellular laterite: With cavities approximately rounded
  - b. Vascular laterite: With cavities approximately tubular
- II. **Nodular laterite:** consists of individual particles approximately rounded (also called pisolithic laterite) sub-divided into:

- a. Cemented nodular laterite: individual concretions can be seen but are strongly joined together by the same iron stone material.
  - b. Partially cemented nodular laterite
  - c. Non-cemented nodular laterite: Concretions from over 60% by weight of the total soil.
  - d. Iron concretions: Are separated by soil-but forms less than 60% by weight of the total horizon.
- III. **Recemented laterite:** this contains fragments of massive laterite or ferruginized rock, broken and wholly or partly cemented.
- IV. **Ferruginized rock:** here, rock structure is still visible, but with substantial isomorphous replacement by iron.
- V. **Soft laterite:** Mottled iron-rich clay, which hardens irreversibly on exposure to air, repeated wetting and drying.

Classification of laterites is also possible depending on the degree of concretionary development. The development of concretions requires sufficient concentration of the hydrated oxides of iron and aluminium for cementation or precipitation growth to start. A recommended classification system for these is shown in Table 2.2 [8].



## 2.2.5 Engineering Characteristics of Laterite

### 2.2.5.1 General

The mode of formation of laterite makes the geotechnical characteristics of laterites different from soils of similar particle size distribution and plasticity characteristics developed from the same parent rocks under temperate climatic conditions. Some of the characteristics/behaviours that laterites exhibit differences from temperate zone materials are [19]:

- Most temperate zone soils have dispersed structure while laterites have concretionary structures. In concretionary structures ferric gels surround the siliceous particles and hydroxides of iron are adsorbed by the kaolinite mineral.
- The finer particles such as clay, silt and sand, which tend to become flocculated, aggregated during development may or may not be broken down during laboratory testing and construction.
- The clay mineral and the cementing and replacing minerals cemented into silt to gravel-sized particles of varying strength and porosity are different from mineral in temperate zone soils consisting of discrete particles from which much of our geotechnical experience and specifications have been derived.
- The presence of porous particles found in laterite will tend to increase all moisture content determinations, including Atterberg limits and grading, unlike the traditional soil mechanics it is usually assumed that all the water is outside the particles and grading is stable.
- Kaolinite, the dominant clay in most lateritic materials, has a non-expansive lattice which, compared to other clay mineral types such as smectite, makes the material less susceptible to volumetric expansion in the presence of moisture.

Based on the above behaviours, empirical correlations between soils properties developed from study of temperate soils may not be valid when applied to residual soils. So, to determine the engineering characteristics of a laterite material, it is generally required to conduct tests like particle size distribution, plasticity characteristics, specific gravity, compaction and bearing strength in addition to observe their behaviour in the field.

### 2.2.5.2 Particle Size Distribution

The successful use of laterites in pavement constructions depends largely upon their changed by the testing methods. A clear understanding of the conventions implicit in the test and calculation methods is therefore fundamental to the assessment of any analysis of particle size [8].

One of these conventions is the assumption of a constant specific gravity for the soil particles; when determined by measurement this value is usually an average over the full range of particle sizes. For some laterites, whose coarse fraction is iron-rich and whose fine fraction is kaolinite, this convention may be misleading. The coarse fraction usually has a specific gravity of between 3.0 and 3.5 (but sometimes very much higher), while the specific gravity of the finer material is about 2.7. The particle-size distribution curve is based on proportions by mass retained on successive sieves, and only represents a particular packing arrangement for a soil of constant specific gravity. Given this discrepancy in specific gravity, a conventionally calculated test applied to some laterites would overestimate the volume content of coarser particles and exaggerate any gap grading in the material, and would not represent the true packing and mechanical stability of the material as a whole [8].

It is important therefore, when judging grading analyses to inspect the material assess its composition and decide if separate specific gravity determinations of the fine and coarse fractions shall be made. If the specific gravities are significantly different, the grading should be calculated by modified mass proportions as well as by conventional mass proportions [9]. An example of this modification of grading analysis and the resulting grading curves are presented in Table 2.3 and Figure 2.3 respectively.

In addition to this, relatively weak coarse particles may also cause problems in grading analyses. Hence, for the analysis to represent the source material the sample preparation and test procedures should not fracture the coarse particles. It is also equally important that the fines adhering to the coarse particles shall be removed. It is equally important that the fines adhering to the coarse particles should be removed. It is therefore recommended by different authors that such soil particles should be soaked until the coating material is fully softened; that only the wet sieving procedure be used; and that a 'closed system' of washing be maintained so that no material is lost in the process. Any tendency for the coarse particles to fracture should be recorded on the test reports [8], [26].

Table 2.3: Modification of grading analysis by modified mass proportion for variation in specific gravity within sample [28]

Assuming constant specific gravity (S.G.)			But visual inspection and separate S.G. determinations reveal that	Therefore, mass retained on each sieve re-proportioned for variation in S.G.	Equivalent mass retained on each sieve for 100g sample	Therefore, Cumulative mass percentage passing each sieve
Sieve size (mm)	Cumulative mass percentage passing each sieve	Therefore, mass retained on each sieve in a 100g sample				
50	100	0	S.G. (fraction > 5mm) = 3.5	0.00	0.00	100.00
31.5	85	15	S.G. (fraction < 5mm) = 2.7	11.57	13.48	86.52
20	70	15	Therefore, mass retained on each sieve larger than 5 mm should be factored by	11.57	13.48	73.04
10	50	20	ratio = $\frac{2.7}{3.5}$ 0.77	15.43	17.98	55.06
5	38	12		9.26	10.79	44.27
2	30	8		8.00	9.32	34.95
0.5	25	5		5.00	5.83	29.13
0.075	20	5		5.00	5.83	23.30
		<u>20</u>		<u>20.00</u>	<u>23.30</u>	
		<u>100</u>		<u>85.83</u>	<u>100.00</u>	
			OR			
			Mass retained on each sieve smaller than 5 mm should be factored by	0.00	0.00	100.00
			ratio = $\frac{3.5}{2.7}$ 1.30	15.00	13.48	86.52
				15.00	13.48	73.04
				20.00	17.98	55.06
				12.00	10.79	44.27
				10.37	9.32	34.95
				6.48	5.83	29.13
				6.48	5.83	23.30
				<u>25.93</u>	<u>23.30</u>	
				<u>111.26</u>	<u>100.00</u>	

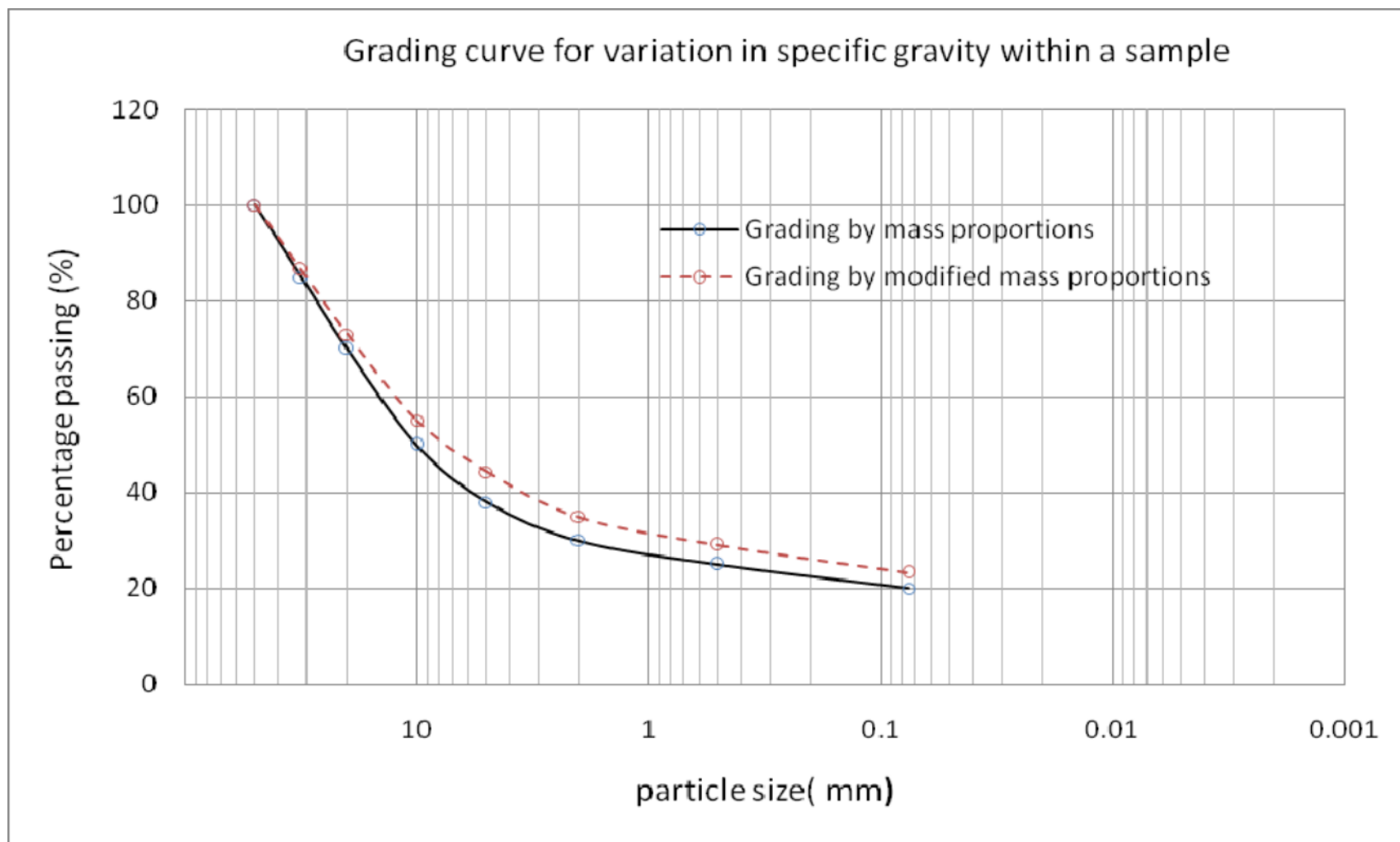


Figure 2.3: Grading curve for variation in specific gravity within sample [28]

### 2.2.5.3 Atterberg Limit

The plasticity of laterites varies widely, both from deposit to deposit and within a deposit. Laterites are very variable in their plasticity characteristics. They are ranging from non-plastic to highly plastic laterites; hence, each material source should be evaluated independently and published data should not be held typical of laterites in general [8].

For temperate region soils, Atterberg Limits are established indicators of engineering behaviour. However, for tropical soils they can be highly misleading without an awareness of the sensitivity of these soils to the testing procedures as a high proportion of hydrated oxides can change the properties of the fine fraction. Any sensitivity should be an initial series of tests.

Two sensitivities are experienced when carrying out Atterberg limit tests on laterites, these are:

**a) Effect of pre-testing drying on Atterberg limit**

The liquid limit and plasticity index of air dried soils are less than the liquid limit and plasticity index of soils at natural moisture content. This is caused by the increase in cementation and dehydration of allophone and halloysite in the air dried soils [37].

**b) Effect of duration and method of mixing on Atterberg limit**

In general, the greater the duration of mixing (i.e., the greater the energy applied to the soil prior to testing), the larger the value of the liquid limit, and to a lesser extent, the larger the plasticity index. This has been attributed to longer mixing resulting in more extensive break down of the cemented bonds between the clay clusters and within peds (disaggregation of the particles), and thus formation of greater proportions of fine particles. In order to address this problem, A.B. Fourie suggests five air dried test specimens should be mixed with water to give the range of water contents suitable for liquid and plastic limit determinations. The mixing time should be no more than 5 minute, and the mixed samples should be left to cure overnight before testing. On the following day the liquid limit should be determined with a minimum of further mixing. A sub-sample from each of the specimens used in the test should be used for the determination of moisture content, using the procedure. The remainder of each specimen should then be mixed continuously for a further 25 minutes before again determining the liquid limit. A significance difference (of >5% of the liquid limit obtained) between the liquid limit from test using 5 and 30 minutes mixing times indicates a disaggregation of the clay sized particles within the material [6]. If this

disaggregation is confirmed by further tests, the main test programme should include the following instructions.

1. limit the mixing times ( no more than 5 minutes)
2. Use fresh material for each moisture content point in compaction tests, for liquid and plastic limit determinations.

#### **2.2.5.4 Specific Gravity**

The available data indicate that specific gravities of laterite materials are vary not only with the textural soil groups but also within different fractions. In the first place lateritic soils have been found to have very high specific gravities of between 2.6 to 3.4 (de Graft-Johnson and Bhatia, 1969). For the same soil, gravel fractions were found to have higher specific gravities than fine fractions due to the concentration of iron oxide in the gravel fraction, while alumina is concentrated in the silt and clay fractions. It is common to see specific gravities reported for the gravel and fines separately. The average of the two values can be assumed to be more representative of the specific gravity for the whole soil [26].

#### **2.2.5.5 Compaction and Bearing Strength**

Compaction is the process of densifying soil and reducing air voids by application of mechanical energy. The densification leads to improvement in engineering properties of strength, compressibility and stiffness, and reduction of permeability.

The response of laterite material to compaction (i.e MDD & OMC) depends on the following factors

1. The significant extent on the particle size distribution
2. The coating of individual clay particles by a layer of hydrated oxides causes aggregation and a reduce ability to absorb water. The sesquioxide content may also cause an irreversible change in plasticity upon drying, and the measured moisture content may include the water of hydration of the oxides and therefore be misleading high.
3. Relatively weak concretionary particles present within the coarse fraction may fracture during laboratory or field compaction, with an attendant changes in the compaction characteristics because the particle size distribution has changed.

The density and compaction characteristics of laterite materials are highly affected by the following two factors.

**a) Effect of pre-test preparation and test procedures**

The OMC and MDD values are not constant values in laterites but vary with the degree of pre-test drying. This property is characteristics of all laterite soils which are not fully hydrated insitu. For the hydrated case, each increment of drying in the compaction test tend to produce some slight irreversible change of sesquioxides from mobile ferrous state to stable ferric state. In addition to this, dehydration of hydrated. The effects of the method of drying on the compaction characteristics have been investigated for soils varying in texture from clays (Newill, 1961; Tateishi, 1967) to gravels and gravelly soils (Brand and Hongsnoi, 1969). It was shown that oven-drying always gives the highest maximum dry densities and lowest optimum moisture contents, while soils at natural moisture content give the lowest maximum dry densities and highest optimum moisture contents [26].

**b) Effect of fresh and reused samples**

Re-compaction of soil yields values of maximum dry density and optimum moisture content that differ from values obtained by using new batch of soils for each point on compaction curves. In some cases, in the case of concretionary laterites, breakdown of coarse particles improves the nature of the grading curve giving rise to higher maximum dry density and lower optimum moisture content. In general it is observed that oven dried concretionary coarse laterite gravels tend to be more brittle and break down more frequently on the dry side of the optimum moisture content. The degree of breakdown may be a function of the degree of maturity (laterization) and the physical characteristics of the particles (e.g. angularity, flakiness, etc.). It is concluded that the nature of grading curve and strength of concretionary coarse fraction are crucial factors in determining the influence of fresh and reused samples on the compaction characteristics of gravelly laterite soils [37].

It is suggested to use Modified AASHTO compactive effort to lateritic gravels because it improves the gradation of these gravels by increasing the sand sized particles. Additionally, reuse of samples should be discouraged as much as possible.

The California Bearing Ratio (CBR) test remains the standard test for pavement design including laterites. This is due to advantages such as universally being known and used, cheap apparatus, rapid testing and results that can be directly related to the laboratory compaction tests.

### 2.2.6 Laterite Gravel as Pavement Material

Concretionary laterites are valuable road pavement materials, widely used in the tropics as sub-base, base material and for gravel roads. The term laterite, however, has tended to be indiscriminately applied in tropical highway engineering to any red soil, and as a result the usefulness of laterites for road construction has been underestimated [8].

Laboratory testing to check the suitability of concretionary laterites to be used as road pavement materials should take into account how these, materials are affected by the testing procedures [8]. Some lateritic soils are sensitive to pre-treatment and testing procedures. So laboratory testing should be simulated to site condition.

Main characteristics of lateritic gravels and gravelly soils as mentioned by Morine W.J. and Todor P.C. are the high content of fines. Consequently, such materials do not fit into the existing temperate zone classification systems for coarse grained soils [26]. In addition, laterites undergo property changes during construction. The most sensitive property is gradation as the nodules tend to crush under heavy compaction.

Laterite gravel is one of naturally occurring material that occurs extensively in the humid tropical and sub-tropical zones of the world, including much of central, southern and western Africa. Unfortunately, laterites have not been used to their fullest extent in the upper (base and sub base) layers of paved roads in the African region for a number of reasons including [13]:

- The variability in their engineering properties and their failure to meet traditional specifications. For example, these materials commonly exhibit gaps in the grading curve (e.g. in the sand fraction); high plasticity indices ( $PIs > 20$ ) and soaked CBR values lower than the minimum of 80 per cent normally specified.
- Lack of awareness of the more appropriate specifications that were first developed by the Portuguese in the mid-1970s in countries such as Angola and Mozambique and subsequently adapted for use in other countries, notably Brazil and Australia.

In view of the above, the use of neat (untreated) laterites for the construction of LVSRs in some African countries has been limited as the road authorities continue to use much tighter, restrictive standards that greatly suppress the use of this type of material. As a result, other more expensive options are adopted such as hauling over long distances other natural gravels

which meet the traditional specifications; stabilizing the laterites with cement and lime or using crushed stone for the base [13].

Fortunately, there are some relatively recent examples of the use of laterites in a number of African countries, such as Angola, Botswana, Kenya, Malawi, Mozambique, Zambia and Zimbabwe, where this type of material has been successfully used in the upper layers of both low and high-volume roads, despite their non-compliance with traditional specifications [13].

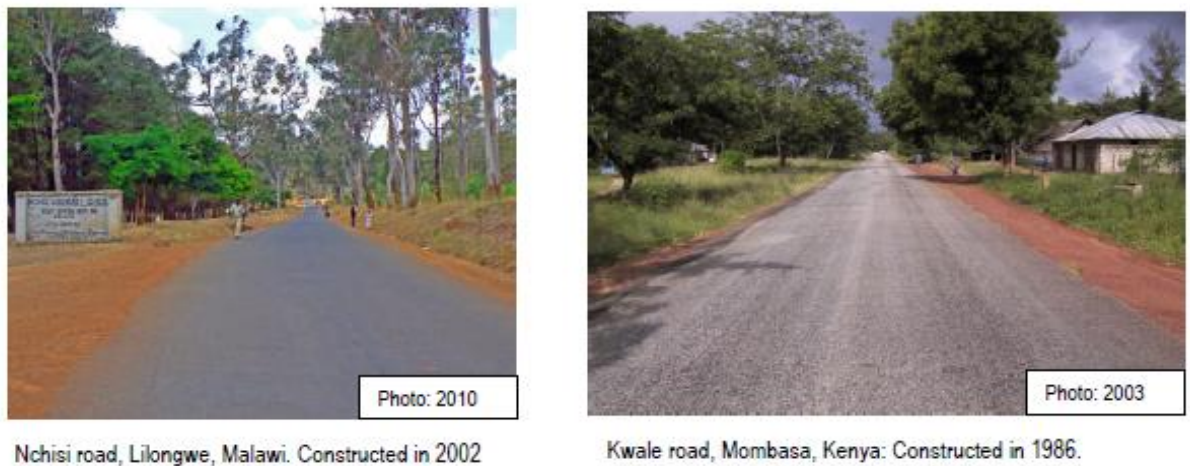


Figure 2.4: Typical examples of some low volume standard roads [13]

The ERA is currently conducting a research project by constructed demonstration section on Assosa-Kumuk road which located in Benushangul-Gumuz regionl state in order to examine the performance of laterite gravel as road base under double surfacing dressing. The demonstration section has 200mm laterite gravel and 150mm laterite gravel sub base thickness with 10m total width, 1.5m shoulder on both sides of the road. Now a day, the section is under monitoring in order to evaluate the performance of the laterite gravel as road base [16].



Figure 2.5 Profile picture of the Assosa-Kumruk demonstration section [16]

## 2.3 Function and General Characteristics of Granular Material

### 2.3.1 Subgrade Material

For designing the thickness of a road pavement, the strength of the subgrade should be taken as that of soil at a moisture content equal to the wettest moisture condition likely to occur in the subgrade after the road is opened to traffic. In classification of sub-grade soils with AASHTO method, Group index is an important indication of the load bearing capacity of the soil. It is a function of the liquid limit, Plasticity index, and the amount of material passing the 0.075mm sieve. Under average conditions of good drainage and thorough compaction, the group index is assumed as an inverse ratio of the supporting value of a soil, i.e., a group index of 0 indicates good bearing sub-grade material and group index of 20 or more is poor sub-grade material [17].

According to [17], sub-grade soils are classified in to 6 groups as S1, S2...S6 based on the CBR values. As follows

- S1- CBR value of 2%
- S2 – CBR values between 3 and 4%
- S3 – CBR values between 5 and 7%
- S4 – CBR values between 8 and 14%
- S5 – CBR values between 15 and 29% and
- S6 – CBR >30%

### **2.3.2 Subbase Material**

In completed pavement, the subbase is an important load spreading layer. It has functions like enabling keeping the traffic stress to at acceptable levels in the sub-grade, it acts as working platform for the construction of the upper pavement layer and it acts as separation layer between sub-grade and road base to avoid contamination of road bases. It can also act as a drainage layer or filter. In dry climatic conditions, in areas of good drainage and where the road surface remains well sealed, unsaturated moisture conditions prevail and the subbase specifications may be relaxed. The selection of the subbase material therefore depends on the design function of the layer and the anticipated moisture regime, both service and construction [17], [10].

Compressive stresses generated in a pavement by traffic decrease rapidly with depth. However, the material immediately above the subgrade is required to accept stresses which are only marginally greater than those in the subgrade hence provision of an unnecessary strong subbase is likely to be un economical. The subbase property is many times governed by its ability to support construction traffic without excessive deformation or raveling. A high quality subbase is therefore required where loading or climatic conditions during construction are severe [17], [37].

A minimum CBR of 30% is required at the highest anticipated moisture content when compacted to the specified field density, usually a minimum of 95% of the maximum dry density achieved in ASTM Test Method D 1557 (Heavy) compaction test, 4.5kg rammer. In tropical condition of prolonged, heavy rain, poor drainage and inadequate surface sealing, the bearing capacity should be determined on samples prepared at 95% of the maximum MDD obtained under heavy compaction (ASTM Test Method D 1557) and soaked for four days. [17].

### **2.3.3 Base course Material**

The principal function of base course in a flexible pavement are to produce sufficient cover over the sub base, to limit the stress and strain induced by the wheel load and uses as main load spreading layer.

The suitability of base course materials for use depends primarily on the design traffic level and the local environment. Road base materials are expected to have a particle size distribution and particle shape strength that will provide a high mechanical stability.

The grading should contain sufficient low plasticity fines (amount of material passing the 0.425 mm) to produce a dense material, with less than 5% air voids when compacted. In effects a maximum plasticity index (PI) of 6 is usually specified. A high PI indicates that the material will shrink or swell excessively during changes in water content. Materials meeting these requirements will normally meet the minimum CBR strength criteria of 80% after 4 days soaking.

Base materials are often divided into two general categories; unbound aggregate material and stabilized base materials. The limiting criteria such as strength, plasticity and grading set out in most conventional specifications for base material based on universal standards applied to all traffic levels. Where the materials fail to meet these criteria they are termed marginal or substandard [9]. They reported that marginal base gravels in terms of their grading can fall into three groups depending on whether the material is too coarse, too fine or gab graded. The too coarse materials generally tend to reduce stability, increase risk of shear and settlement, low in situ density and difficult to compact. The too many fines materials are known to have low compacted strength and increase risk of deformation and high potential for capillary rise and moisture susceptibility. The gab graded materials are difficult to compact, increased risk of deformation under traffic, increased moisture susceptibility and pumping of fines. The aggregate bases with higher fines content are susceptible to loss of strength and load supporting capability upon wetting [27].

Base aggregate may be considered as marginal in terms of shape if it is not flanky or elongated but also if its particle is oven-rounded with no angular faces. Rounded with smooth surface texture have poor inter-particle friction and loss of stability, compaction difficulty, low density and high air voids content & low stability [9].

Laterite gravel is one of the marginal materials and has high percentage of clay fraction and & consequently high plasticity.

Laterite gravels are gab-graded with depleted sand size fraction, to contain variable percentage of fines, and to have coarse particles of variables strength which may break down in performance, limits their usefulness as pavement material on highly trafficked roads [9]. Therefore, in order to use laterites for higher traffic intensities their performance characteristics has to be improved by appropriate stabilization methods.

## 2.4 Stabilization of Granular Material

Road construction encounters with the lack of good quality road building materials. The solution to overcome the shortage of road building materials, practicing road engineers and scientists have introduced a technique called stabilization, which means mixing poor or marginal quality materials with some amount of good quality materials and/or stabilizers to improve the engineering property and finally to use the stabilized materials for road construction.

The term stabilization is used to improve the strength, density, plasticity, volume stability, durability, permeability and workability or a combination of the mention parameters.

Due to their mineralogical composition, soils may be rather complex material. Stabilization is therefore not a straight forward application of a given stabilizing agent, a number of aspects should be taken into account in the selection of the proper stabilization techniques the factor that should be consider include physical and chemical composition of the soil to be stabilized, purpose of the stabilized layer to be used, availability and economic feasibility of stabilizing agent, ease of application, environmental consideration, curing time, and safety such factors should be taken into account in order to select the proper type of stabilization.

Stabilization can be summarized into four broad groups.

- Mechanical Stabilization: used to modify the physical property by combining two or more materials
- Chemical and Physical Stabilization: improve the engineering property of the material by adding stabilizers.
- Densification/Compaction: improve the strength of soil/aggregate by different compaction methods. Sometimes this also considered as mechanical stabilization.
- Thermal Stabilization: a method used for de-watering soil/aggregate

Since this research focuses on mechanical and lime stabilization, only these two stabilization method will be discussed in detail.

### 2.4.1 Mechanical Stabilization

Mechanical stabilisation of a material is usually achieved by adding a different material in order to improve the grading or decrease the plasticity of the original material.

The physical properties of the original material will be changed, but no chemical reaction is involved. For example, a material rich in fines could be added to a material deficient in fines in order to produce a material nearer to an ideal particle size distribution curve. This will allow the level of density achieved by compaction to be increased and hence improve the stability of the material under traffic. The proportion of material added is usually from 10 to 50 per cent [10].

Providing suitable materials are found in the vicinity, mechanical stabilisation is usually the most cost-effective process for improving poorly-graded materials. This process is usually used to increase the strength of a poorly-graded granular material up to that of a well-graded granular material. The stiffness and strength will generally be lower than that achieved by chemical stabilisation and would often be insufficient for heavily trafficked pavements. It may also be necessary to add a stabilising agent to improve the final properties of the mixed material.

An aggregate, with little or no fines content as shown in figure 2.6a, gains stability from grain-to-grain contact. An aggregate that contains no fines usually has a relatively low density but is pervious and not frost susceptible. This material is however difficult to handle during construction because of its non-cohesive nature.

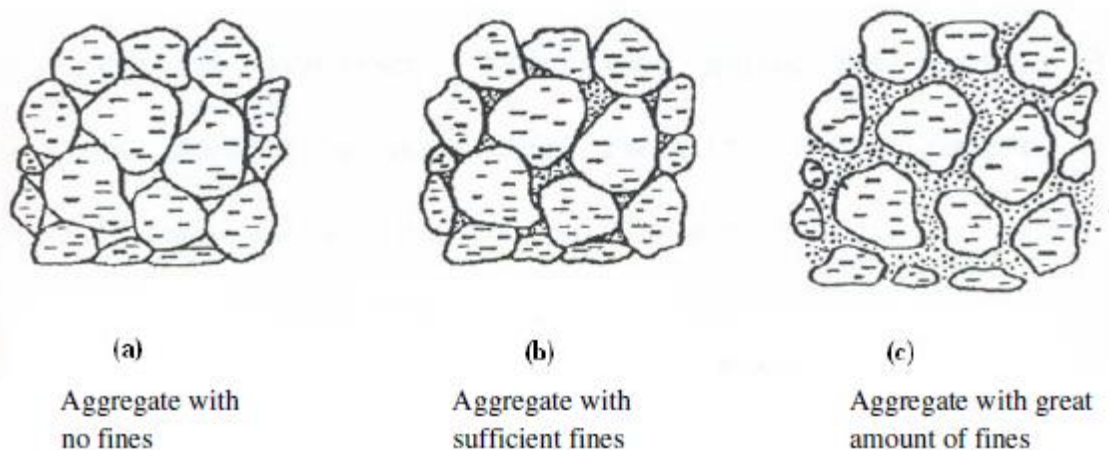


Figure 2.6: physical state of soil aggregate mixture [15]

An aggregate that contains sufficient fines to fill all voids between the aggregate grains will still gain its strength from grain-to-grain contact but has increased shear resistance as shown

in figure 2.6b. Its density is high and its permeability is low. This material is moderately difficult to compact but is ideal from the standpoint of stability.

As shown in figure 2.6c, material that contains a great amount of fines has no grain-to-grain contact and the aggregate merely 'float' in the soil. Its density is low; it is practically impervious and it is frost susceptible. In addition, the stability of this type of material is greatly affected by adverse water conditions. Paradoxically, the material at times is quite easy to handle during construction and compacts quite readily [16].

Mechanical stabilization of laterites usually requires the addition of sized material to achieve a suitable particle size distribution for maximum packing. Nodular laterites as a group tend to fall into category C in figure 2.6, and mechanical stabilization usually involves supplementing the sand-size material. This helps to reduce the plasticity of the mix and to increase the density achieved under a certain compactive energy [8].

Typical material used for mechanical stabilization includes river deposited sand, natural gravel, silt sand, sand clay, crushed run and waste quarry products, crushed stone, volcanic cinders and scoria, poorly graded laterite and beach sand, etc. In this research, crushed stone aggregate is used as mechanical stabilizer in order to improve the gradation as well as strength of the laterite gravel.

The mechanical stability of the mixed soil depends upon the following factors

- a) **Mechanical strength of the aggregate:** The mixed soil is stable if the aggregates used have high strength. However, if the mixture is properly designed and compacted, even the aggregates of relatively low strength can provide good mechanical stability.
- b) **Mineral composition:** The mechanical stability of the mixed soil depends upon the composition of the minerals in it. The minerals should be weather-resistant. Sodium sulphates and sodium carbonates cause large volume changes due to their hydration and dehydration. These are detrimental to the stability.
- c) **Gradation:** The gradation of the mixed soil should be such that the voids of the coarser particles are filled with finer particles so that a high density is obtained. It is found by experience that to obtain sufficient cohesion in the mixture, it is necessary to have a greater proportion of the material passing 75 $\mu$  sieve. Generally, an ideal mixture would include about 25% binder. The basic requirement is that the mixture shall

contain a fair proportion of different size particles. It should contain sufficient quantity of fines to provide cohesion, especially in the wearing surface.

- d) Plasticity characteristics:** Soils with high liquid limit and plasticity index are suitable as binders for soils used for surfacing. Such soils possess greater cohesion and better moisture retention capacity. They provide a better seal against the downward movement of surface water.

For soils used in base courses, the requirements of plasticity characteristics are quite different from those for surfacing. The soil in base course should have low plasticity to avoid excessive accumulation of water and the resulting loss of strength.

As the soil available at site may seldom meet both the gradation and the plasticity characteristics, it usually becomes necessary to mix soils from different sources to obtain the desired mixture. This is normally done by trial mixes. As far as possible, the maximum use of locally available soils should be made for economy.

- e) Compaction:** The mechanical stability of the stabilized soil mass depends upon the degree of compaction attained in the field. Normally, the compaction is done at or near the optimum water content.

### **Limitations of Mechanical Stabilizations**

- Original soils contains fines, use of coarser fractions for blending may be expensive.
- When compaction is not met in mechanical stabilization it is common to add fines to the blended soil.
- Adding “additional” fines to a “blended” soil can result in a number of problems:
  - Increase potential for frost heave
  - Due to the potential for a gap graded soil, it can become internally unstable and result in accelerated erosion
  - Internally instability will result in a lost of stiffness and strength
- It is not uncommon that the main problem with compaction is that improper equipment or techniques we used.
- Creation of dusts clouds results in nuisance to traffic.
- Reduction of soil cohesion or binding forces ultimately leads to material disintegration

### **2.4.2 Lime Stabilization**

Lime is produced from chalk or limestone by heating and combining with water. The term ‘lime’ is broad and covers the following three main types:

- a) Quicklime i.e. calcium oxide (CaO),
- b) Slaked or hydrated lime, i.e. calcium hydroxide (Ca(OH)<sub>2</sub>) and
- c) Carbonate of lime, i.e. calcium carbonate (CaCO<sub>3</sub>).

Only quicklime and hydrated lime are used as stabilisers in road construction. Hydrated lime is used extensively for the stabilisation of soil because it is much less caustic than quicklime, especially soil with high clay content where its main advantage is in raising the plastic limit of the clayey soil. Very rapid stabilisation of water-logged sites has been achieved with the use of quicklime. In this research, Grade-1 Hydrated lime is used as chemical stabilizer in order to improve the strength by decreasing the plasticity of the laterite gravel.

Lime is an effective stabilizing agent for clay material to improve both workability and strength. Lime is not effective with cohesion less or low cohesion materials without the addition of secondary additives (pozzolanic-fine materials which react with lime to form cementitious compounds). The strength of lime stabilized materials is dependent on the amount of lime, the curing time, curing temperature and compaction. Lime has more tendencies to produce granular material and consequently their major applications are in the modification of clays, plastic sands, and plastic gravel.

#### **2.4.2.1 Soil-lime Reaction**

The reactions between soil and lime are complex and not yet fully understood. However, the following form is part of the reactions in soil-lime reaction [35]:

- Rapid reactions (modification):
  - Cation exchange;
  - Flocculation;
- Long term reactions (cementation)
  - Pozzolanic reactions;
  - Carbonation.

Cation exchange can take place between the added lime and the clay particles in the mineral aggregate. The calcium (Ca<sup>2+</sup>) ion (and to a lesser extent the Magnesium (Mg<sup>2+</sup>) ion) from the lime can replace hydrogen (H<sup>+</sup>), sodium (Na<sup>+</sup>) and potassium (K<sup>+</sup>) ions from the clay particles. The replacing ions (calcium and magnesium) can hold substantially less water molecules than the ions being replaced.

This results in a reduction of the thickness of the bound water layer around the clay particle, which promotes the development of flocculent structures.

The rapid reactions result in reduction of the liquid limit and plasticity index, because of the reduction of bound water (the soil changes from hydrophilic to hydrophobic as a result of ion exchange) and improve the workability of the soil. Reduction in plasticity results in an increase in shear strength.

The long term reactions take place when there is sufficient lime added in order to have ‘free lime’ available in the soil for the pozzolanic reactions after the rapid reaction (ion exchange) has completed. Siliceous and aluminous materials in the soil react with lime to produce a gel of calcium silicates and aluminates, which cements the soil particles together in a way similar to hydrated cement. Minerals in the soil that react with lime to produce a cementing compound are known as pozzolans. The lime-cementing action takes considerable more time than required for hydration of Portland cement. This long term effect on strength, causing continuing strength improvement with time, often called pozzolanic reactions. The cementing action also depends on climatic conditions and a thorough compaction of the mixtures. High curing temperatures have a positive effect on the pozzolanic reactions. Temperatures lower than 13 and 16°C retard reaction; from this point of view it is obvious that lime stabilization is especially popular in tropical countries.

Carbonation occurs when hydrated lime reacts with the CO<sub>2</sub> from the air. Carbonates (CaCO<sub>3</sub>) add some strength but the carbonation reaction “eats” the lime and will therefore deter pozzolanic reaction. Carbonation is therefore an undesired chemical reaction.

### **2.4.2.2 Curing of Lime Treated Materials**

Curing is a very important process that prevents loss of moisture from the stabilized materials. Lime treated materials have to be cured at least for the first seven days after they are placed. Curing has multidimensional advantages to the stabilized materials among which the followings are worth mentioning. Curing helps to:

- ensure that sufficient moisture is retained in the layer so that the stabilizer continue to hydrate;
- limit drying shrinkage while the hydration reaction is proceeding and the material is getting strong;
- Reduce the risk of carbonation from the top of the stabilized layer.

There are different methods of curing of stabilized materials in the field. Among these, spraying of the water on the surface of the stabilized materials, and covering of the stabilized layer by water-tight medium are the most common methods.

If curing is to be carried out by spraying of water, it is important to cover the surface of the stabilized layer by 30-40mm thick sand so as to reduce leaching of the stabilizers. Watertight covers, on the other hand, can be applied using bituminous materials, water-proof paper, or plastic sheets. In this method, very light spray of water is first applied on the surface of the stabilized layer and viscous cutback bitumen like MC-3000 or slow setting emulsions, or the plastic sheets are applied afterwards. There should be no traffic movement on the covered surface for seven days [39].

For laboratory specimens, curing is carried out by keeping the samples in a sealed, air-tight bag at a constant temperature.

### 2.4.2.3 Engineering Properties of Lime Treated Material

Dallas little has characterized the material properties of lime-stabilized soils as related to their impact on overall pavement performance can be divided into four categories. These are Strength, durability, fracture and Resilient Modules/Stiffness [25]. Since in this research the properties of lime treated material is evaluated based on strength and durability, only these two properties will be see in detail:

**Strength:** The most obvious improvement in a lime-reactive soil or aggregate is strength gain over time. The various strength parameters impacted by the pozzolanic reactions that occur include unconfined compressive strength, tensile strength, flexural strength, and CBR. The stabilized soil should exhibit the following requirements to check whether the strength gain of the material is within the required range or not which is tabulated in table 2.4 below. There are two types of stabilized base layers in Tanzania Pavement and Materials Design Manual and Road Note 31. The two types have different strengths and serve the same purpose but under different traffic loading condition. The one with lower strength will be applied when the traffic load is light and the other with higher strength will be applied in heavy traffic zones.

Table 2.4: Strength specifications for Lime-Treated Pavement Layer in Flexible Pavement

Standards	Strength Requirement [MPa]	
	Base Course	Subbase
Road Note 31	3.0 – 6.0 for CB1	0.75 – 1.5
	1.5 – 3.0 for CB2	
US Army and Air Force	5.2	1.7
SATCC	1.5 – 3.0	0.5 – 1.5
Tanzania PMDM	1.0 for C1	-
	2.0 for C2	

The strength requirements are based on the 28-days unconfined compressive strength test in cylindrical specimens. The exception to this is the Road Note 31 strength requirement, which was set based on the 150mm cube strength. The SATCC strength requirement is based on cylindrical specimen with diameter-to-height ratio of 1:1.

**Durability:** Durability is the ability of material to retain stability and integrity over years of exposure to destructive forces of weathering. The ability of lime-stabilized materials to resist the detrimental effects of moisture and freeze-thaw cycling over time has been evaluated in several ways, in both the laboratory (e.g., soaking in conjunction with strength/stiffness tests, cyclic freeze-thaw tests) and the field. The results of these evaluations have often shown only slight detrimental effects of environment on the levels of strength/stiffness produced by the addition of lime.

The durability test is conducted by alternately wetting and drying of the lime treated specimen. The specimen is submerged in water for 5hrs followed by drying in the oven at 71°C for 42hrs, then the surface is cleared with a wire scratch brush and the procedure is repeated for 12 cycles. The weight loss is calculated as:

$$L_s = \frac{W_1 - W_2}{W_1} \times 100 \tag{1}$$

Where;  $L_s$  = Weight loss of specimen (%)

$W_1$  = Weight of specimen before durability test

$W_2$  = Weight of specimen after 12 cycles of wetting and drying

The durability requirement for the stabilized soil is tabulated in table 2.5 below [11], [32].

Table 2.5 Durability requirement for stabilized soils

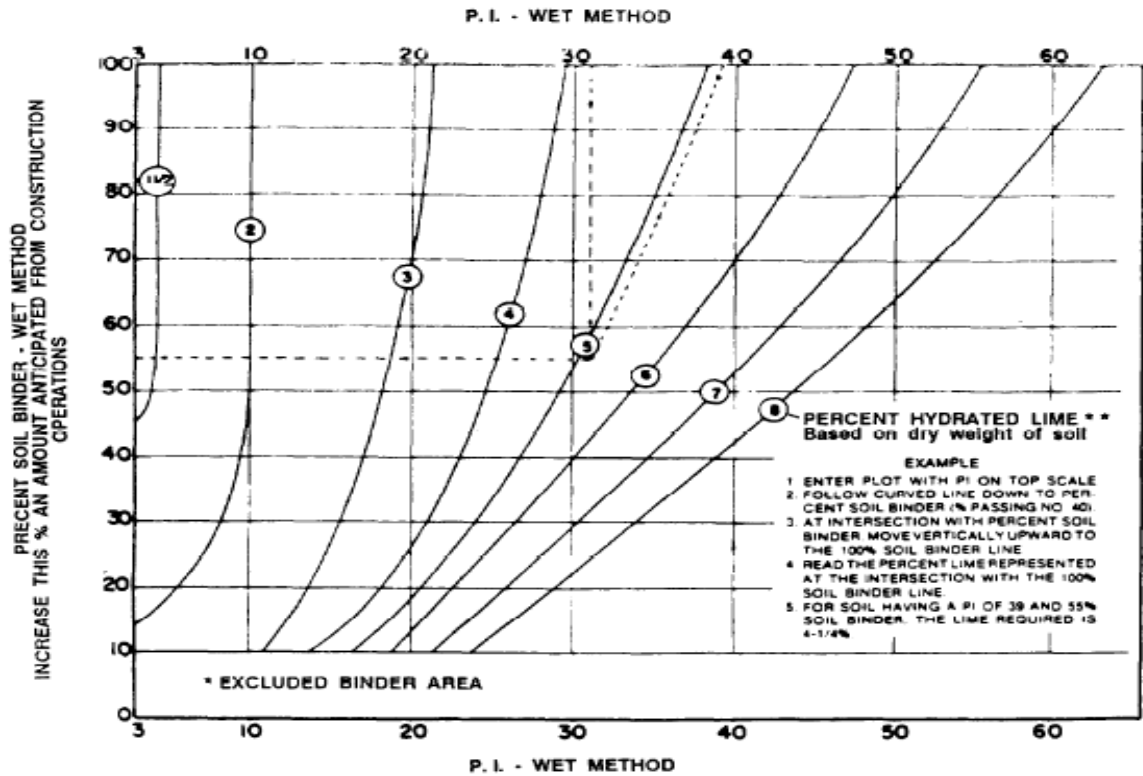
Standards	Durability Requirement
US Army and Air Forces	Max. 11 for Granular soil with PI < 10
	Max. 8 for Granular soil with PI > 10
	Max. 8 for silt
	Max. 6 for clay
Portland Cement Association	Max. 14 for A-1, A-2-4, A-2-5, and A-3 soils
	Max. 10 for A-2-6, A-2-7, and A-5 soils
	Max. 7 for A-6 and A-7 soils

**2.4.2.4 Determination of Design Lime Content**

The design lime content for lime-stabilized soil can be determined using the following steps which were discussed in the manual, TM 5-822-14/AFMAN 32-8010, prepared by Joint Departments of the Army and Air Force, USA, 1994. These steps are:

- I. The initial lime content is determined either from PH test or from fig. 2.7.
- II. Using the initial design lime content prepared soil-lime mix using ASTM D 3551 and moisture-density tests are conducted using ASTM D 1557 to determine the maximum dry density and optimum moisture content;
- III. Triplicate samples of soil-lime mixture are prepared for unconfined compression and durability tests at the initial lime content selected in step 2 and at the lime contents 2% above and 2% below that determined in step 2. The samples are prepared at the density and water content that are expected in field construction. The samples should be prepared in accordance with ASTM D 3551 except that when more than 35% of the material is retained on the No. 4 sieve, a 4-inch diameter mould is used to prepare the specimens. The specimens are cured for 28 days at 73<sup>0</sup>F.
- IV. Three specimens are tested using the unconfined compression test in accordance with ASTM D 1633 and the other three specimens are tested for durability using either wet-dry method (ASTM D 559) or freeze-thaw (ASTM D 560) as appropriate.
- V. The results of the unconfined compressive strength and durability tests are compared with the requirements given table 2.4 and 2.5, respectively. The lowest lime content which meets the required unconfined compressive strength requirement and satisfies the required durability is the design lime content. If the results of the specimens tested

do not meet the strength and durability requirements, then a higher lime content should be selected and the steps i through iv shall be repeated.



\* Exclude use of chart for materials with less than 10% - No. 40 and cohesionless materials (P. I. less than 3)

Figure 2.7: Chart for the initial determination of lime content

## 2.5 Laterite Stabilization

Studies on laterite stabilization have been made by Kedir A. and Worku B from Addis Ababa University and Ola, S.A., and Rahman, M.A. from abroad. With respect to these studies made on properties of stabilized laterite it is worth discussing the following details and findings.

Kedir A. (2010) studied the usefulness of Con-Aid as a stabilizer for lateritic soil from Gidole town (South Ethiopia). The experimental study involved Atterberg limit, moisture-density relation and soaked CBR for both treated and untreated soil. The findings drawn from the study are:

- The Con-Aid liquid chemical yielded in significant improvement on plasticity.
- The compaction tests showed a small improvement in maximum dry density and a small decrease in optimum moisture content.

- The laterite soils treated with Con-Aid have showed small increment of CBR value. This shows that CBR values are not promising after treatment.

Worku B. (2015) evaluates sugarcane bagasse ash for stabilization of lateritic soil from Mekenajo. The experimental study was made by Atterberg limit, free swell, linear shrinkage, compaction and CBR tests. The conclusions and finding of the study are:

- The plasticity, shrinkage limit and free swell showed an increase with increment of the bagasse ash.
- Addition of bagasse ash decreases the maximum dry density and increase the optimum moisture content.
- The application of bagasse ash results in increasing the un soaked CBR; however, it is ineffective in improving the soaked CBR value.

RAHMAN, M.A., (1987) investigated the effect of cement-lime mixes on the geotechnical properties of lateritic soils from University of Ife campus, Ile-Ife (Nigeria). The study aimed to use the lateritic soil in highway construction. The geotechnical properties of the stabilized lateritic soils were measured by using Atterberg limit, compaction characteristics, unconfined compressive strength and CBR characteristics. The main finding of the research can be summarized as:

- On the basis of compressive strength, California bearing ration and percentage of swelling, the lateritic soil investigated can be stabilized with lime-cement mixes for highway construction materials.
- From the point of view of economy, compressive strength and CBR value, a combination of 3% lime and 3% cement can be used for base materials in highway construction.
- The optimum moisture content increase with increase in both lime and cement.
- The maximum dry density decreases with increase in lime content but in the case of cement, the dry density increases almost linearly.

Ola, S.A., (1977) did a study on the potentials of lime stabilization of lateritic soils. Lateritic soils from A-1-a soil to A-7-6 soil from the Zaria area (Nigeria) were obtained from the study. Atterberg limit, compaction characteristics, CBR and durability test were used. The conclusions and finding drawn from the study are:

- All Nigerian lateritic soils from A-1-A soil to A-7-6 soil used in the investigation improved their engineering characteristics substantially by the addition of lime.
- The addition of lime reduced the plasticity index of the soils whereas the plastic limits increased; the liquid limits increased slightly.
- The application of lime reduced maximum dry density and increases the optimum moisture content.
- From CBR and durability test results, only the A-1 and A-2 soils have a potential as competent base material. However, the other soils may be utilized as sub base material.

Monasseh J. and Isaac O., (2011) have made a study on Mechanical and Cement stabilization of laterite for use as Flexible pavement material. 15-60% of sand by dry weight of laterite was used as modifier in the stabilization of the laterite with 3-12% by dry weight of cement in order to improve the physical and strength properties of a reddish-brown lateritic soil which obtained from a borrow pit at Ikpayongo (Nigeria). The experimental study was made by Atterberg, compaction, CBR and UCS. The conclusion and finding of the study are:

- The plasticity index decreased from 17% for untreated laterite to 2.5% when treated with a combination of 60% sand and 6% cement.
- An average 7 day UCS value of 2,115 KN/m<sup>2</sup> and CBR value of 230% were achieved at a combination of 45% sand and 6% cement. These values are higher than the conventional 1,720 KN/m<sup>2</sup> and 180% criteria for UCS and CBR, respectively, for adequate cement stabilization based on Nigerian specification. Generally, 45% sand and 6% cement is an optimum combination in order to satisfy the requirements of PI, UCS, and CBR.

## 2.6 Performance of Stabilized Materials

Pavements are constructed as per the standards and specifications after design. However, pavements usually do not serve for the design period efficiently, safely, comfortably and economically due to early deterioration. Pavement deterioration is broadly a function of the original design, material types, construction quality, traffic volume, axle load characteristics, road geometry, environmental conditions, age of pavement, and the maintenance policy pursued.

There are various modes in which the pavement fails. Cracking of the surface layer and permanent deformation of the pavement system which manifests as rutting on the pavement surface are the major one. Larger and more concentrated loads produce larger stresses and strains, with thicker layer carrying higher flexural stresses than thinner layers. In pavement analysis, loads on the surface of the pavement produce two strains which are believed to be critical for design purposes. These are the horizontal tensile strain;  $\epsilon_t$  at the bottom of the asphalt layer and the vertical compressive strain;  $\epsilon_c$  at the top of the subgrade layer. If the horizontal tensile strain; is excessive, cracking of the surface layer will occur, and the pavement distresses due to fatigue. If the vertical compressive strain;  $\epsilon_t$  is excessive, permanent deformation occurs at the surface of the pavement structure from overloading the subgrade, and the pavement distresses due to rutting. Damage analysis is performed for both fatigue cracking and permanent deformation as follows:

### a) Fatigue Criteria

The relationship between fatigue failure of asphalt concrete and tensile strain at the bottom of asphalt concrete layer is represented by the number of repetitions as suggested by Asphalt Institute in the following form:

$$Nf = f_1(\epsilon t)((\epsilon t)^{f_2}(E1)^{-f_3} \quad (2)$$

Where:  $Nf$  : number of load repetitions to prevent fatigue cracking.

$\epsilon t$ : tensile strain at the bottom of asphalt layer.

$E1$ : elastic modulus of asphalt layer.

$f_1$ ,  $f_2$ , and  $f_3$  are constants determined from laboratory fatigue tests.

b) Rutting Criteria

The relationship between rutting failure and compressive strain at the top of subgrade is represented by the number of load applications as suggested by Asphalt Institute in the following form

$$Nr = f_4 (\epsilon_c)^{-f_5} \tag{3}$$

Where; Nr: number of load repetitions to limit rutting.

$\epsilon_c$ : vertical compressive strain at the top of subgrade.

$f_4$  and  $f_5$  are constants determined from laboratory fatigue tests.

Different institutions have provided different distress models. The coefficients for rutting and cracking used by some of the institutions are given in Table 2.6.

Table 2.6: Coefficients for Rutting and Cracking Distress Models

No.	Distress Model	$f_1$	$f_2$	$f_3$	$f_4$	$f_5$	Source
1	AI model	0.0796	3.291	0.854	$1.365 \times 10^{-9}$	4.477	AI
2	Shell model	0.0685	5.671	2.363	$1.05 \times 10^{-7}$	4.0	Shell

## 2.7 KENPAVE Computer Program

KENPAVE computer programme contain two parts, KENLAYER and KENSLAB. The KENLAYER computer program applies only to flexible pavements with no joints or rigid layers. For pavements with rigid layers, such as PCC and composite pavements, the KENSLABS program should be used. The backbone of KENLAYER is the solution for an elastic multilayer system under a circular loaded area. The solutions are superimposed for multiple wheels, applied iteratively for non - linear layers, and collocated at various times for viscoelastic layers. As a result, KENLAYER can be applied to layered systems under single, dual, dual-tandem, or dual-tridem wheels with each layer behaving differently, linear elastic, nonlinear elastic, or viscoelastic. Damage analysis can be made by dividing each year into a maximum of 12 periods, each with a different set of material properties. Each period can have a maximum of 12 load groups, either single or multiple. The damage caused by fatigue cracking and permanent deformation in each period over all load groups is summed up to evaluate the design life [21].

## **3. RESEARCH METHDOLOGY**

### **3.1 Description of the Study Area**

#### **3.1.1 General**

Gedo-Nekemt and Gedo-Fincha-Lemlem Berha road project are routes where the laterite gravel samples for this research work were taken, are located in the National Regional State of Oromia.

The Gedo-Fincha-Lemlem Berha upgrading project is carried out by dividing into two contract: contract 1 and contract 2, which includes Gedo-Mana Begna and Mana Begna-Fincha-Lemlem Berha road project respectively. Gedo-Mana Begna road project, which has an approximate total project length of 80.5Km is located in West Shewa and Horogudru zones of the Oromia National Regional State; Western part of Ethiopia. The proposed road alignment starts at the junction of Addis-Nekemt-Assosa trunk road at Gedo town, which is about 175Km West of Addis Ababa then proceeds in a northerly directions towards a junction near Komboshaa town locally known as Mana Begnaa. Some of the controlling points and/or towns or rural settlement which are traversed by the project route includes Gedo, Lenchaa Matoria, Goben, Wayu, Shumbo, Gebete and Mana Begnaa or Kombosha Matoria. Functionally the road is a main access road that runs through the Western part of Ethiopian in area with great potential for cash crops and access to Fincha Sugar Factory and Fincha Hydropower Station. The client of the project is Ethiopian Road Authority (ERA). Currently the road project is under construction by Yemen Contractor HAWK consulted by Galander Engineering Consultant Co. (Sudan) in JV with Beza Engineering Consultant PLC (Ethiopia).

The Gedo-Nekemt road, which has a total length of 134 Km, is located in West Shewa and East Wollega Zones of the Oromia National Regional State. The road is part of the trunk road that connects the city of Addis Ababa via Gedo Nekemt town, which is a zonal capital. Between Gedo and Nekemt the road passes through about 8 towns including Ejaji, Tibe, Sheboka, Bako, Ano, and Sire, and about 6 villages including Kiltu, Kejo, Cheri, Chingi, Konchi and Gute.

#### **3.1.2 Climatic Condition**

The climatic condition of the country is classified into five temperature zones on the basis of temperature-altitude relationships [30].

Table 3.1: Climatic categories with respect of altitude and mean annual temperature

<b>Climatic Category</b>	<b>Altitude</b>	<b>Mean Annual Temperature</b>
“Wirch” which is cold moist temperature	3200-3500m a.s.l	Below 10 <sup>0</sup> C
“Dega” which is cool to cold humid	2500 - 3200m a.s.l	10 - 16 <sup>0</sup> C
“Weina-Dega” which is warm to cool semi-humid	1500 - 2500m a.s.l	16 - 20 <sup>0</sup> C
“Kola” which is warm to hot semi-arid	500 - 1500m a.s.l	20 - 28 <sup>0</sup> C
“Berha” which is hot arid	Below 500m a.s.l	28 - 34 <sup>0</sup> C
Note: A.S.L = Above Sea Level		

From the above classification, Gedo-Nekemt and Gedo-Mana Bagna road project routes are found within the “Weina Dega & Dega” and “Weina Dega” climatic zones respectively.

Furthermore, the mean annual rain fall and temperature of the Gedo-Nekemt and Gedo-Mana Bagna road project routes are in the range of 1200-2000mm & 15-20<sup>0</sup>C and 1443mm & 15-20<sup>0</sup>C respectively.

### 3.1.3 Topography and Soil Condition

The dominant soil types along the Gedo-Nekemt road and Gedo-Mana Bagna road project routes are reddish brown to light brown silty clays and clays with/without gravels. Black clay soils are also observed on some stretches of both routes.

Regarding topography most of the stretches of the Gedo-Nekemt road project traverses on a flat to rolling terrain whereas the Gedo-Mana Bagna road goes through a rolling, hilly and mountainous.

## 3.2 Materials and Methods

Relevant literature concerning laterite gravels are reviewed and designed to achieve the goals as follows.

- I. Sampling of the material (Laterite gravel)
- II. Experimental Plan
- III. Analysis Method

### 3.2.1 Material

The area under the study is located in the western part of the country. For this research, samples were collected from 2 different routes starting from Gedo town. These routes are Gedo-Mana Begna-Fincha and Gedo-Nekemt. From the route Gedo-Mana Bagna-Fincha, one sample was collected and from Gedo-Nekemt route, 2 samples were collected. Therefore a total of 3 samples were collected from different sources for the purpose of the research. Before the gravel samples were sampled, site visit was made with consultant’s material engineer (of Gedo-Mana Begna road project) and ERCC material inspector. The collected samples were visually observed as concretionary laterite gravel.

While collecting the samples, GPS coordinates were recorded to show the exact position of sources and photographic picture were also taken. Details are given in table 3.2.

Table 3.2: Location of samples for thesis work

Route	Location	Coordinate	Sample ID
Gedo-Mana Begna- Fincha	59+860 Km, 70m RHS	0324010	A
		1031165 2276	
Gedo- Nekemt	116+00Km, 50m LHS	0240517 0998505 1862	B
	115+500Km, 300m LHS	0241991 0997595 1855	C

The crushed stone aggregate and lime, which are used for mechanical and chemical stabilization, were collected from IFH Engineering PLC and Derba Lime & Chemicals PLC, respectively.



(a)

(b)

Figure 3.1 Profile picture of Sample A



(c)

(d)

Figure 3.2: profile picture of Sample B



(e)

(f)

Figure 3.3: profile picture of Sample C

Basic tests are conducted for the three sources (samples) without the addition of stabilizer in order to evaluate the basic engineering properties of each sample. The summary of the test results are shown in table 4.1.

From the basic laboratory tests results, sample A and B have higher strength and have similar classification while sample C has lower strength than Sample A and B. For this research, it is better to blend sample A & B in order to achieve higher strength for base course, and to reject sample C due to its low strength. Because of this, the research emphasis on stabilization of the blended laterite material (Sample A & B), which designated as laterite.

### 3.2.2 Experimental Plan

Various laboratory tests are available to identify and study the properties of soils. There are different tests to study the engineering properties of natural gravel and treated gravel. Before testing, materials were prepared in accordance with AASHTO T 87-86. These methods involves air drying of samples by spreading the material out in trays in the laboratory and leaving it open in the air for at least 10 days. The laboratory test of this research focused on five categories.

Category one, the engineering properties of the blended laterite material (sample A+B) was investigated using the following laboratory tests.

Table 3.3: Types of test for laterite gravel alone

No.	Method	Reference
1	Grain Size Analysis	AASHTO T-27
2	Atterberg Limits	AASHTO T-89 & T-90
3	Specific Gravity	AASHTO T-84
4	Water Absorption	AASHTO T-84
5	Modified Proctor's Compaction	AASHTO T 180-01 Method B
6	California Bearing Ratio [CBR]	AASHTO T-193
7	Los Angeles Abrasion	AASHTO T-96
8	Ten Per cent of Fines Value [TFV]	BS 812, Part 111
9	Geo-chemical Test	Atomic Absorption Spectrometer and Colorometric Analysis

Category two, quality of the crushed stone was tested using the following quality tests.

Table 3.4: Types of test for CSA alone

No.	Method	Reference
1	Grain Size Analysis	AASHTO T-27
2	Atterberg Limits	AASHTO T-89 & T-90
3	Modified Proctor's Compaction	AASHTO T 180-01 Method B
4	California Bearing Ratio [CBR]	AASHTO T-193
5	Aggregate Crushing Value [ACV]	BS 812, Part 105-1990
6	Ten Per cent of Fines value	BS 812, Part 111-1990
7	Flankiness Index	BS 812, Part 111-1990

Category three, the engineering properties of blended laterite material (sample A+B) was investigated with addition 20%, 40% and 60% CSA by volume of blended laterite. The following laboratory tests were conducted for this category.

Table 3.5: Types of test for laterite-CSA mix

No.	Method	Reference
1	Grain Size Analysis	AASHTO T-27
2	Atterberg Limits	AASHTO T-89 & T-90
3	Modified Proctor's Compaction	AASHTO T 180-01 Method B
4	California Bearing Ratio [CBR]	AASHTO T-193
5	Los Angeles Abrasion [LAA]	AASHTO T-96
6	Ten Per cent of Fines Value [TFV]	BS 812, Part 111

Category four, the engineering properties of blended laterite material (sample A +B) was investigated with addition of 4%, 8%, 12% and 16% lime by dry weight of blended laterite alone. The following laboratory tests were conducted for this category.

Table 3.6: Types of test for laterite-lime mix

No.	Method	Reference
1	Atterberg Limits	AASHTO T-89 & T-90
2	Modified Proctor's Compaction	AASHTO T 180-01 Method B
3	California Bearing Ratio [CBR]	AASHTO T-193
4	Unconfined Compression Strength [UCS]	ASTM D-5102
5	Durability [ Wet/Dry test]	ASTM D-559

Category five, the engineering properties of blended material (sample A & B) was investigated with a combination of CSA and lime addition (e.g. 20% CSA + 4%, 20% CSA + 8%, 20% CSA + 12%, 20% CSA + 16%, 40% CSA + 4%, 40% CSA + 8%, 40% CSA + 12%, 40% CSA + 16% etc.). The following laboratory tests were conducted for this category.

Table 3.7: Types of test for laterite-CSA-lime mix

No.	Method	Reference
1	Atterberg Limits	AASHTO T-89 & T-90
2	Modified Proctor's Compaction	AASHTO T 180-01 Method B
3	California Bearing Ratio [CBR]	AASHTO T-193
4	Unconfined Compression Strength [UCS]	ASTM D-5102
5	Durability [ Wet/Dry test]	ASTM D-559

### 3.2.3 Analysis Method

The data obtained from the laboratory tests of mechanical stabilization were analysed and interpreted using the ERA criteria. On the other hand, for lime treated laterite and laterite-CSA mix, the criteria set by Road note 31 and ERA were used. These criteria were tabulated in table 2.4 and 2.5.

## 4. LABORATORY RESULTS, ANALYSIS AND DISCUSSION

### 4.1 Test results for Laterite sources

Basic tests like Gradation, Atterberg limit, Proctor and CBR are conducted for the three sources (samples). The summary of the test results are shown in table 4.1 below and The detailed results of all of the laboratory tests, conducted on each source of laterite, were annexed as Appendix A.

Table 4.1: Basic laboratory test results for the three samples

Test Type	Test Results		
	Sample A (Source A)	Sample B (Source B)	Sample C (Source C)
Gradation: % passing			
50mm	100	96.6	100
37.5mm	100	87.5	96.3
19mm	93.6	53.8	61.8
9.5mm	52.5	28.7	33.7
4.75mm	15.5	20.2	25.5
2.36mm	9.4	17.6	22.8
0.425mm	7.2	14.6	18.9
0.075mm	5.3	11.3	13.8
Liquid Limit (%)	49.6	55.4	58.4
Plastic Limit (%)	30.8	33.8	33.3
Plastic Index (%)	18.8	21.6	25.1
OMC (%)	18.56	21.5	20.9
MDD (%)	2.08	1.97	1.90
CBR (%)	43.1	34.2	21.7
Soil Classification	A-2-7	A-2-7	A-2-7

### 4.2 Test results for Blended Laterite

#### 4.2.1 Atterberg Limits

Plasticity is an important factor in the performance of a gravel wearing course for the following reasons. Material with too low plasticity tends to loosen quickly as a result of diminished bonding and the rate of gravel loses is generally very high. Loose material is pushed off into the drains or washed away by run-off or blown away by wind when dry. High gravel lose reduces re gravelling cycle periods causing high maintenance cost and general whole life costs. High plasticity on the other hand causes the wearing course to be slippery when wet and the material may soften to an extent where the gravel layer may actually deform and fail instantly under traffic [15].

This test was conducted on materials passing No.40/ 0.425mm by using the standards of AASHTO T89&T90 Methods for liquid and plastic limit respectively. The method of preparation of samples is dry preparation. While conducting the liquid limit the sample was soaked in water for 24 hours for moisture equilibration, time of mixing used was 5minutes. Fresh materials were used for each moisture content point in accordance with [6].

Laboratory test result showed that the PI of the laterite gravel is 19.9% which do not meet the requirements of ERA technical specification for base course and subbase material in which the plasticity index shall not exceed 6% and 12%, respectively [18].

#### **4.2.2 Gradation**

This method is used primarily to determine the grading of materials proposed for use as aggregates or being used as aggregates. The results are used to determine compliance of particle size distribution with applicable specification requirements and to provide necessary data for control of the production of various aggregate products and mixtures containing aggregates [15].

A grain size analysis is used to determine the relative proportions of various particle sizes in laterite gravel. To do this analysis, a wet preparation method is performed which is given in AASHTO T-146; Blight, 1997 and weighed sample of dried aggregate is shaken over a nest of sieves having selected sizes of square openings. The laterite sample is shaken with a mechanical sieve shaker, and the weight of material retained on each sieve is determined and expressed as a percentage of the original sample.

The particle size distribution requirement depends on the nominal maximum particle size as shown in Table 4.2.

Table 4.2: Recommended Particle Size Distribution According to ERA for Naturals and Weathered Rocks for use as Base Course Material (GB2, GB3) [18]

Test sieve (mm)	Percentage by mass of total aggregate passing test sieve	
	Nominal maximum particle size	
	37.5 mm	19.5 mm
50	100	-
37.5	80-100	100
20	60-80	80-100
10	45-65	55-80
5	30-50	40-60
2.36	20-40	30-50
0.425	10-25	12-27
0.075	5-15	5-15

When we refer to table 4.2 two options of grain size distribution are presented depending on the nominal maximum particle size. In this study, grain size distribution test for the nominal maximum particle size of 37.5mm was carried out before and after compaction on the laterite gravel and the results are as shown in Fig 4.1.

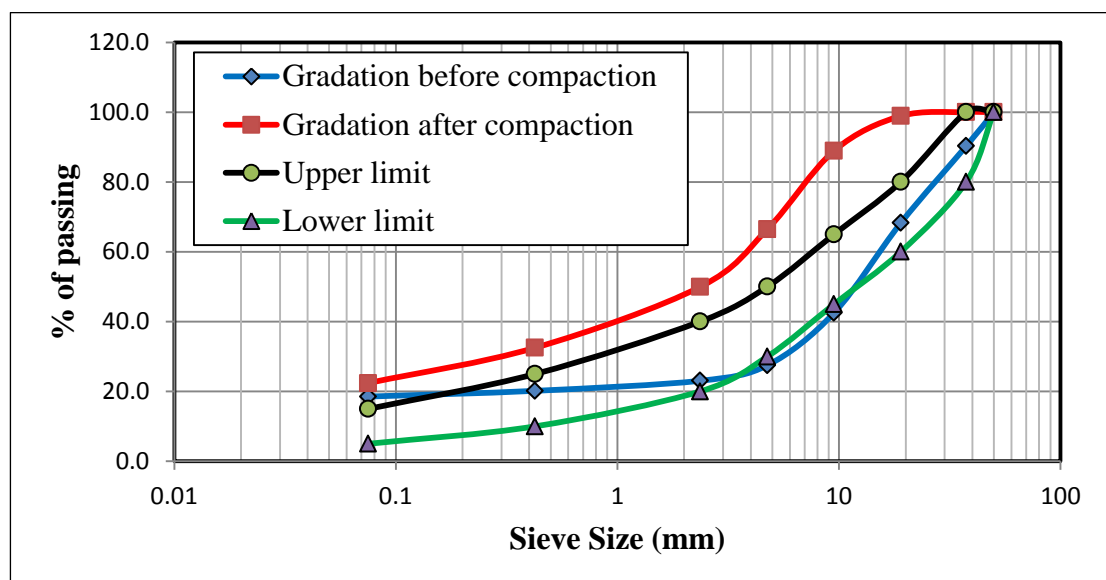


Figure 4.1: Gradation of natural laterite gravel before and after compaction with respect to base course specification

As it can be seen from fig.4.1 the gradation of laterite gravel on natural state does not satisfy the ERA gradation requirement for making base course materials i.e. its gradation curve is not parallel with upper and lower limits of the specification by having more fines and less sand size particles more than the recommended values. The gradation after compaction clearly shows that the gradation of laterite gravel changes significantly upon compaction due to its weakness to resist crushing.

According to ERA grading limit, the subbase material shall have a smooth continuous grading within the limits for grading A, B, C or D and it shall comply with one of the grading given below in Table 4.3.

Table 4.3: Ethiopia Road Authority grading limits for subbase material [18]

Sieve Size (mm)	Mass Percent Passing			
	A	B	C	D
63	100	-	-	-
50	90-100	100	100	-
37.5	-	-	80-100	-
25	51-80	55-85	-	100
20	-	-	60-100	-
9.5	-	40-70	-	51-85
5	-	-	30-100	-
4.75	35-70	30-60	-	35-65
2	-	20-51	-	25-51
1.18	-	-	17-75	-
0.425	-	10-30	-	15-30
0.3	-	-	9-50	-
0.075	5-15	5-15	5-25	5-15

For laterite gravel for use as subbase, the specification limits of Grade C is the most suitable one than the others why because Grade C has almost a complete specification of percent mass passing and relatively wider gap for each sieve sizes. In this study, grain size distribution test using Grade C was carried out before and after compaction on the laterite gravel and the results are as shown in Fig 4.2.

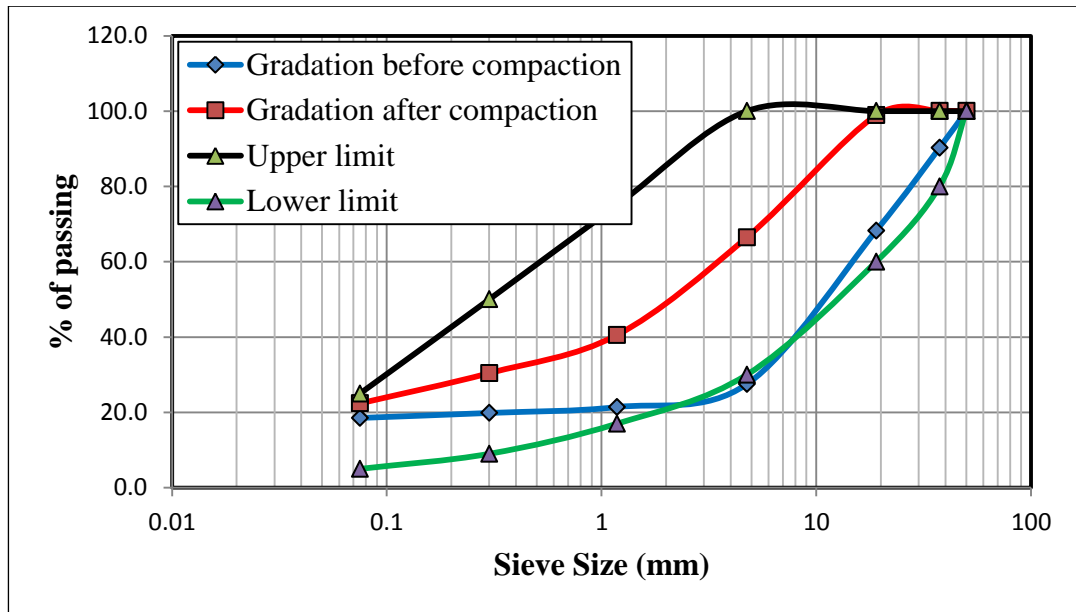


Figure 4.2: : Gradation of natural laterite gravel before and after compaction with respect of subbase specification

As it can be seen from fig.4.2 the gradation of laterite gravel on natural state satisfy the ERA gradation requirement for making subbase materials i.e. its gradation curve is parallel with upper and lower limits of the specification in after compaction state.

### 4.2.3 Specific Gravity and Water Absorption

Bulk specific gravity is the characteristic generally used for calculation of the volume occupied by the aggregate in various mixtures containing aggregate including Portland cement concrete and other mixtures that are proportioned or analysed on an absolute volume basis. The bulk specific gravity determined on the saturated surface-dry basis is used if the aggregates is wet, that is, if its absorption has been satisfied. Conversely, the bulk specific gravity determined on the oven-dry basis is used for computation when the aggregate is dry or assumed to be dry.

Specific gravity and absorption of the sample was determined using AASHTO T-84, procedures. Accordingly, laboratory test results revealed that the absorption & specific gravity of the laterite gravels that retain sieve 4.75 mm are 17.7 % and 2.6 respectively. Therefore, the laterite gavel has high water absorption capacity because of its high porosity.

#### 4.2.4 Modified Proctor Compaction

In this research, a heavily trafficked asphalt road was considered hence the modified proctor test is used. Compaction of laterite gravel is conducted by following the standard procedures stated in AASHTO T 180-01 Method B, in which the soil passing 19mm sieve is compacted in 5 layers in a 4 inch mold with compactive effort of 25 blows with a 4.5Kg hammer. While conducting the proctor test new batches of soil were used for each moisture content increments at each compactive effort in order to avoid the effect of degradation of concretionary particles.

Accordingly, the test was carried out which produced that the maximum dry density (MDD) of the laterite gravel is 1.75 g/cc at the optimum moisture content (OMC) of 17.6% as indicated in the curve shown below.

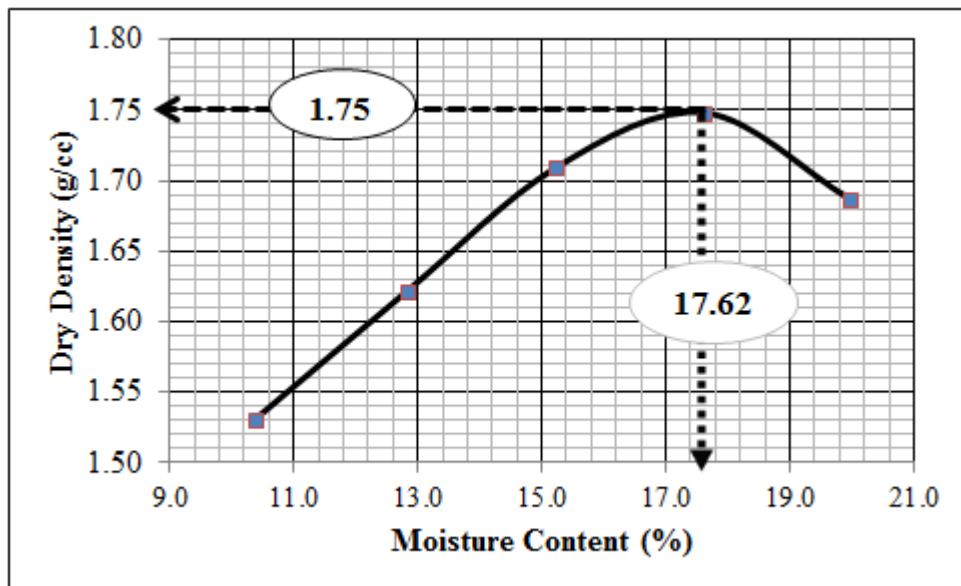


Figure 4.3: Moisture density relationships for laterite gravel only

#### 4.2.5 California Bearing Ratio (CBR)

The CBR test was carried out for the sample following the standard procedures stated in AASHTO T-193. Three point CBR test is used in order to get the CBR of the materials at different level of compaction. A material passing 19mm sieve was compacted in 5 layers with heavy hammer (4.5kg) for 10, 30 and 65 blows. A surcharge is placed on the surface to represent the mass of pavement material above base course. The sample was allowed to soak for 96 hours (4 days) before penetration to simulate the worst condition on the field. Expansion of the sample is measured during soaking to check for potential swelling. After the CBR test sample is taken for determination of moisture content.

According to ERA manual, the minimum soaked California Bearing Ratio (CBR) for the base course and subbase material shall be 80% and 30%, respectively, when determined in accordance with the requirements of AASHTO T-193. The Californian Bearing Ratio (CBR) shall be determined at a density of 98% and 95% of the maximum dry density for the base course and subbase material, respectively when determined in accordance with the requirements of AASHTO T-180 method B [18].

Table 4.4: determination of dry density before and after soaking

No. of Blows	Before Soaking		After Soaking	
	Dry Density (g/cc)	Moisture (%)	Dry Density (g/cc)	Moisture (%)
10	1.25	17.6	1.22	34.6
30	1.6	17.6	1.54	30.8
65	1.74	17.5	1.72	24.5

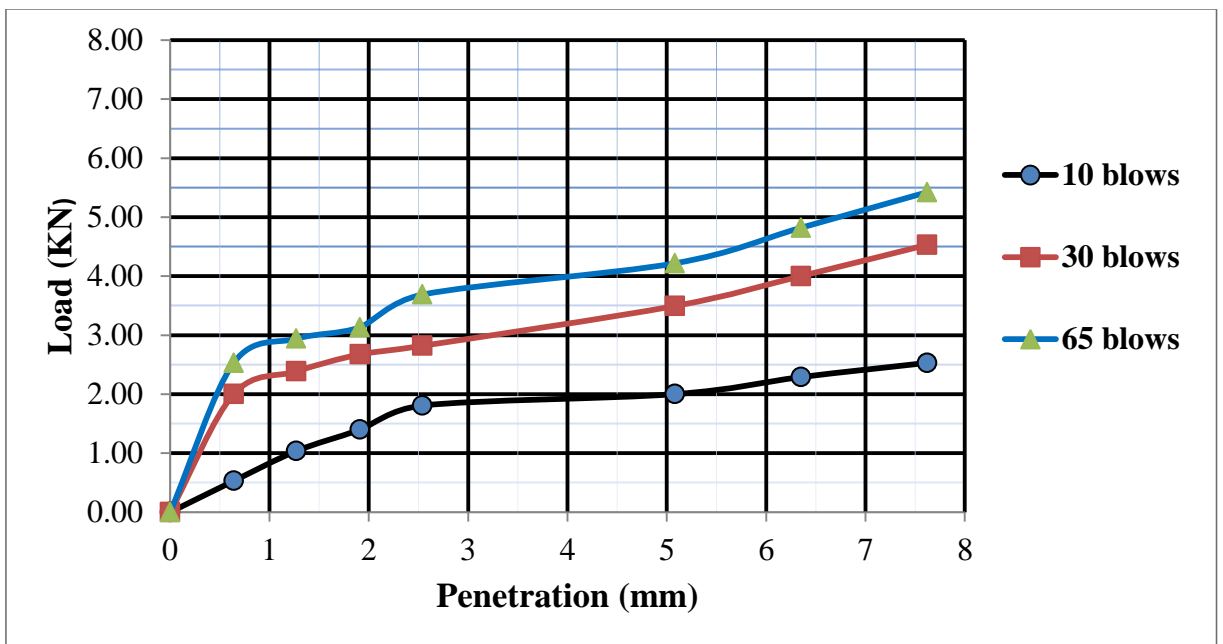


Figure 4.4: Load-Penetration curves of laterite gravel only

Table 4.5: Determination of CBR and Swell for laterite only

No. of Blows	Load (KN)		CBR (%)		Swell (%)
	2.54 mm	5.08mm	2.54mm	5.08mm	
10	1.81	2.00	13.9	10.0	1.67
30	2.99	3.50	23.0	17.5	1.39
65	4.06	4.35	31.2	21.7	1.21

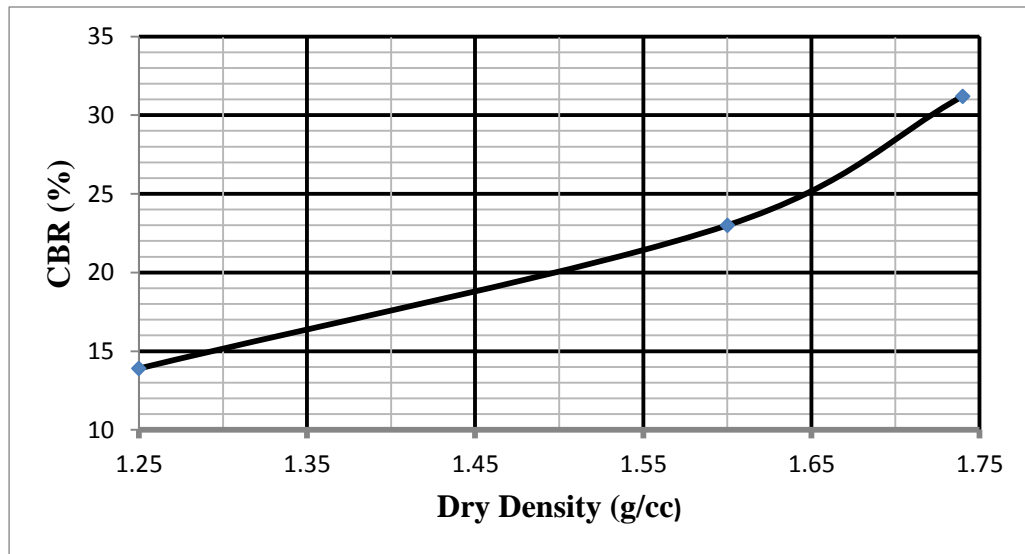


Figure 4.5: Dry Density Vs CBR.

Table 4.6: Determination of CBR at 98% and 95% of MDD of laterite gravel

No. of Blows	Dry Density (g/cc)	CBR (%)
10	1.16	13.9
30	1.42	23
65	1.53	31.2
CBR at 98% MDD		<b>28.64</b>
CBR at 95% MDD		<b>18.8</b>

#### **4.2.6 Abrasion Resistance**

Abrasion test is the test used to know how the aggregate is sufficiently hard to resist the abrasive effect of traffic over its service life.

The Ethiopia Road Authority standard technical specifications manual recommends that the Los Angeles Abrasion value shall not exceed 45% and 51% for base course and subbase materials, respectively when determined in accordance with the requirements of AASHTO T-96[18]. However the Los Angeles Abrasion value (LAAV) of the laterite gravel was found to be 70.3%. Thus, it has failed to satisfy the requirement of base and sub base materials of ERA technical specification.

#### **4.2.7 Ten Percent Fines Value (TFV)**

To ensure that the materials are sufficiently durable, the minimum soaked Ten percent Fines Value (TFV) according to BS 812, Part 111 shall be 50 KN [18]. However the Ten Per cent Fines Value (TFV) of the laterite gravel was found to be 17.3 KN. Thus, it has failed to satisfy the requirement of the technical specification of ERA.

#### **4.2.8 Unconfined Compression Strength (UCS)**

The unconfined compression strength test is used to measure the shearing resistance of cohesive soils which may be undisturbed or remolded specimens. This test is used to quickly obtain the compressive strength of soils that possess sufficient cohesion in the unconfined state. It is worth mentioning that the needs of UCS test to be conducted in order to determine the strength of laterite. Unfortunately, the compacted sample was collapsed, i.e. it is not stand by itself in the absence of lateral support. Hence, it was not possible to determine the UCS value of natural laterite gravel.

#### **4.2.9 Geochemical Tests**

Geochemical (oxide) tests are carried out to know quantitatively main oxides of the soil material. Almost all soils contain some amount of colloidal oxides and hydroxides. The oxides and hydroxides of Aluminium, iron and silicon are of greatest interest since they are the ones most frequently encountered. Iron and Aluminium oxides coat mineral particles, or cement particles of soils together. It may also occur as distinct crystalline units, such as Hematite, Gibbsite and Magnetite.

Geochemical tests were conducted at Geological Survey of Ethiopia Geochemical Laboratory. The test results are shown in Table 4.7.

The degree of laterization of the soil samples can be evaluated based on Silica-Sesquioxides (S-S) ratio as detailed in section 2.2.1. The Sesquioxide, designated as  $R_2O_3$ , is the combination of Aluminium oxide ( $Al_2O_3$ ) and Iron oxide ( $Fe_2O_3$ ). The chemical formula  $SiO_2$  designates the silica. Accordingly unlaterized soils have S-S ratio greater than 2. For lateritic soils S-S ratio lie between 1.33 and 2 and for true laterites the ratio is less than 1.33. Lateritic soil has not under gone a considerable degree of laterization as compared to true laterite.

The test results in Table 4.7 show that the laterite gravel used for this research have silica – sesquioxide ratio below 1.33. This indicates that the soil gavel are all true laterites. True laterites are simply referred as laterites. The soil of such kind are highly laterizd i.e., sesquioxides content are high.

Table 4.7: Oxide composition in percent.

Designation	$SiO_2$	$Al_2O_3$	$Fe_2O_3$	MgO	CaO	$Na_2O$	$K_2O$	MnO	$H_2O$	LOI	$Ti_2O$	$P_2O_5$	$\frac{SiO_2}{R_2O_3}$
Sample A	41.42	14.97	27.28	0.64	0.4	0.32	0.34	<0.01	2.36	9.21	1.48	0.43	0.98
Sample B	19.72	20.09	34.92	0.68	0.44	1.74	1.74	0.15	4.42	13.2	2.09	1.56	0.36

### 4.3 Quality test results for Crushed Stone Aggregate

Crushed stone aggregate is produced by the mining a suitable rock deposit and breaking to the desired sizes using crushers. Crushed stone aggregate makes use of the interaction between the particles to produce stiffness and strength of the material. Through crushing and sieving one can produce any possible grading, so this aspect is strictly controlled. Cost is an important factor in the process of requiring certain gradations for special applications.

The results of the laboratory tests that were carried out on the CSA are attached as Appendix C and summarized in table 4.8. The test results, as shown in table 4.8, indicated that the CSA fulfils all of the standard technical specification of ERA.

Table 4.8: Crushed Stone Aggregate Quality Specification and Test Results.

Test type	Specification as per ERA 2002 standard technical specification	Test results	Remark
Gradation: % passing			
50mm	100	100	OK
37.5mm	95-100	100	OK
20mm	60-80	79	OK
10mm	40-60	50	OK
5mm	25-40	29	OK
2.36mm	15-30	20	OK
0.425mm	7-19	11	OK
0.075mm	5-12	9	OK
Plastic index (%)	$\leq 6$	4.4	OK
CBR (%)	$\geq 100$	143	OK
ACV	$< 25$	18	OK
TFVT (KN)	$\geq 110$	200	OK
Flankiness index (%)	$\leq 30$	20	OK
Wet/dry Ratio (%)	$\geq 75$	87	OK

#### 4.4 Laterite Mechanically Blended with Crushed Stone Aggregate

For the gravel soil not comply with specifications, mechanical stabilization was carried out to improve their failed properties. Experimental trails were carried out by adding percentage of CSA to the marginal laterite gravel. The CSA is used to increase the strength and to improve the gradation of laterite gravel. The mixture specimens were prepared at 20%, 40%, and 60% of CSA by volume of dry laterite. This mixture were subjected to laboratory tests including Gradation, Atterberg limit, Modified proctor, California bearing ratio, Los Angeles abrasion and Ten percent of fines value.

##### 4.4.1 Gradation

Sieve analysis test for both before and after compaction were conducted on laterite gravel blended with 20%, 40%, and 60% of CSA by volume basis. The gradation of laterite gravel-CSA was compared with the requirements set by ERA for base course and subbase. The following figures show gradation before and after compaction of laterite gravel blended with different proportion of crushed stone aggregate. The details of the test are presented in Tables D-1.1 to D-1.3 of Appendix D-1.

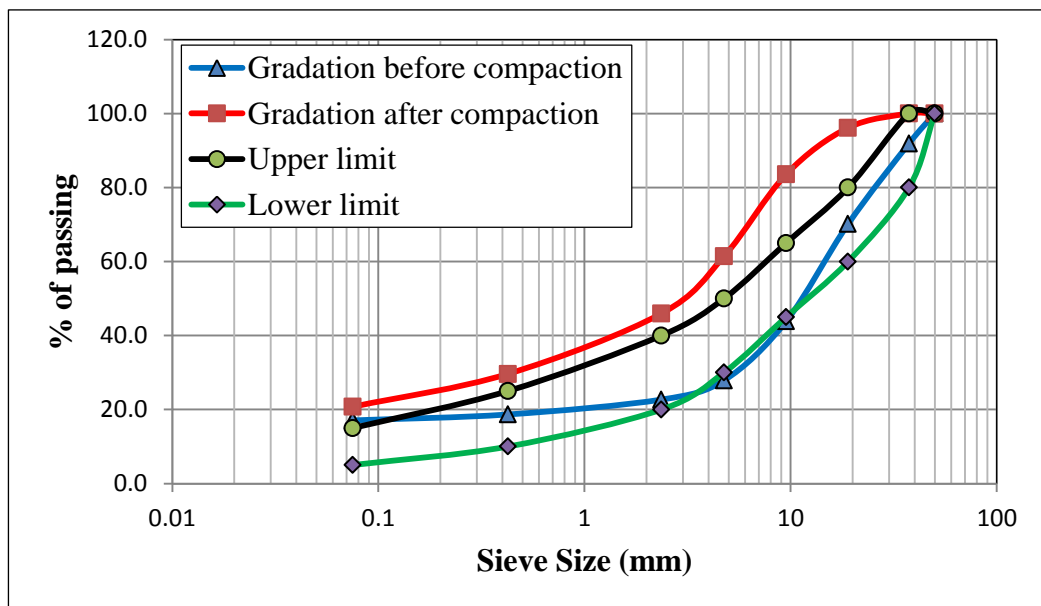


Figure 4.6: Gradation before and after compaction of laterite with 20 % CSA with respect to base course specification.

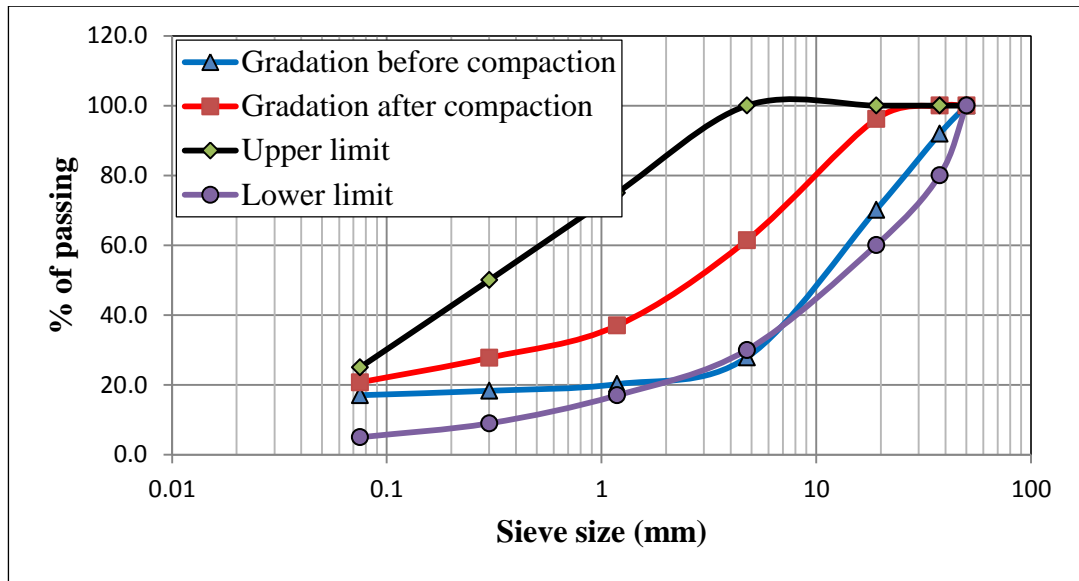


Figure 4.7: : Gradation before and after compaction of laterite with 20 % CSA with respect to subbase specification

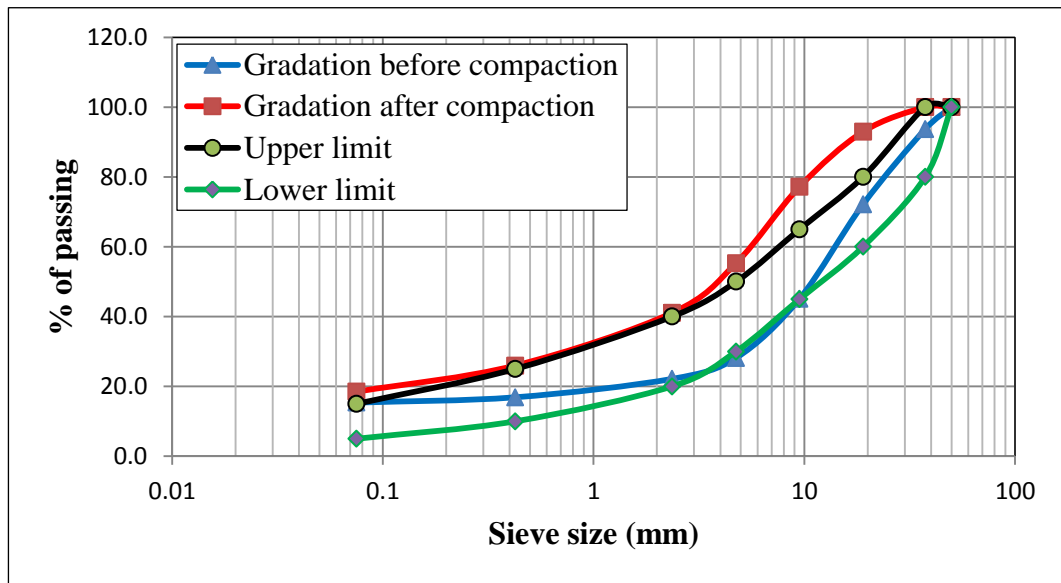


Figure 4.8: Gradation before and after compaction of laterite with 40 % CSA with respect to base course specification

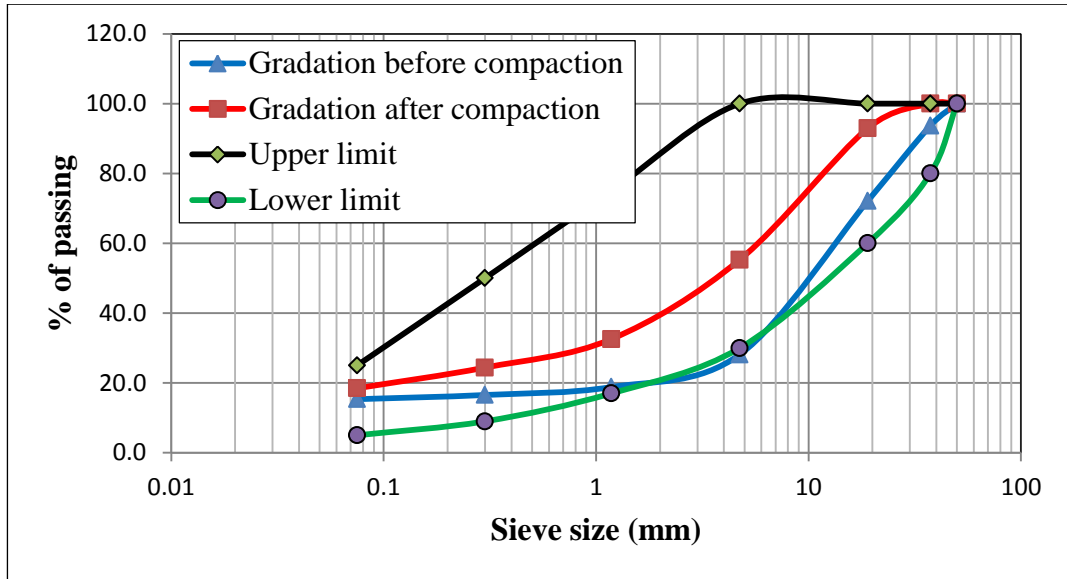


Figure 4.9: : Gradation before and after compaction of laterite with 40 % CSA with respect to subbase specification.

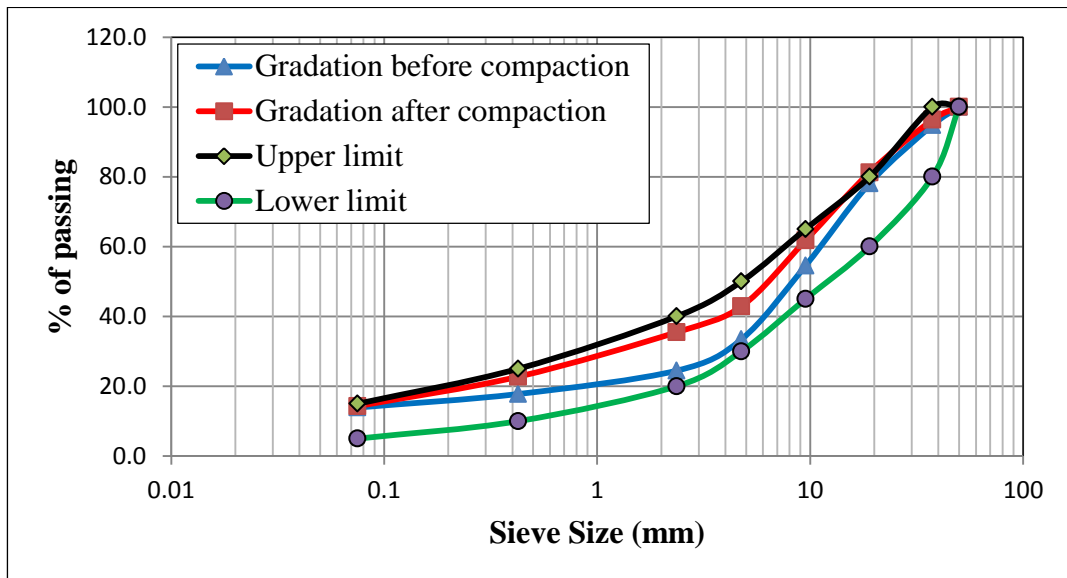


Figure 4.10: : Gradation before and after compaction of laterite with 60 % CSA with respect to base course specification.

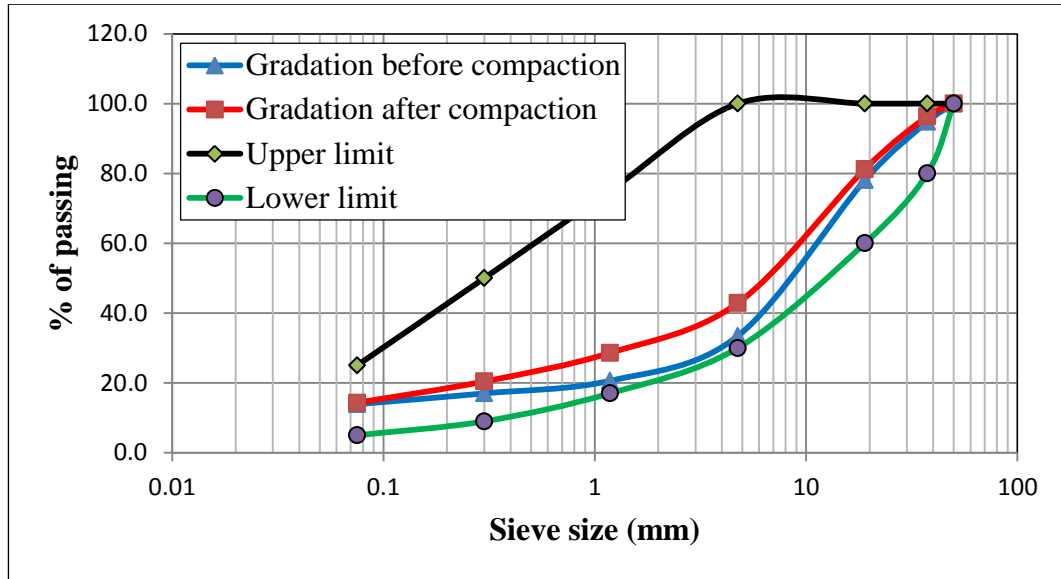


Figure 4.11: Gradation before and after compaction of laterite with 60 % CSA with respect to subbase specification.

The addition of CSA to laterite gravel produced corresponding improve in the gradation curves of laterite gravel (Figures 4.6, 4.8, 4.10).

For the laterite gravel with 20% CSA, the grading chart (Figure 4.6) has shown that deficiency in sand size particles and much in fine particles. This indicated that, addition of 20% CSA did not make a significant improvement in the content of sand size particles of laterite gravels. Therefore, it is out of the ERA specification for base course. Addition of 40% CSA to the laterite gravel showed that the gradation curve for base course (Figure 4.8) is marginally within the envelope of ERA specification before compaction while it lies outside the specification for the 4.75, 9.5 and 19mm sieves after compaction. The gradation curve of laterite (Figure 4.10), before and after compaction, falls within the envelope of ERA specification when 60% CSA was added.

From the gradation curve Figure 4.6-4.11 we can observe that, as the percentage of CSA increase the resistance of laterite gravel to crushing up on compaction is also increase due to the strong nature of CSA in order to resist crushing.

From gradation point of view, additions of 60% CSA to laterite gravel, before and after compaction, are within the upper and lower limit gradation requirement for base course material, but with respect to subbase material all the blend proportions lays within the specification of subbase, at both before and after compaction state.

#### 4.4.2 Atterberg Limit

Atterberg limit tests were carried out on laterite gravel with different percentage of CSA. The percentages of CSA were 20%, 40% and 60% (by volume of dry laterite gravel). The results of Atterberg limit test on the soil with various percentages of crushed stone aggregate are shown in table 4.9 and fig 4.12. The detailed laboratory test result is presented in Appendix D-2.

Table 4.9: Effect of crushed stone on Atterberg limits of laterite gravel.

CSA Content (%)	Liquid limit (%)	Plastic limit (%)	Plastic index (%)
0	52.1	32.2	19.9
20	43.3	26.5	16.8
40	35.1	22.1	13.0
60	33	21.9	11.1

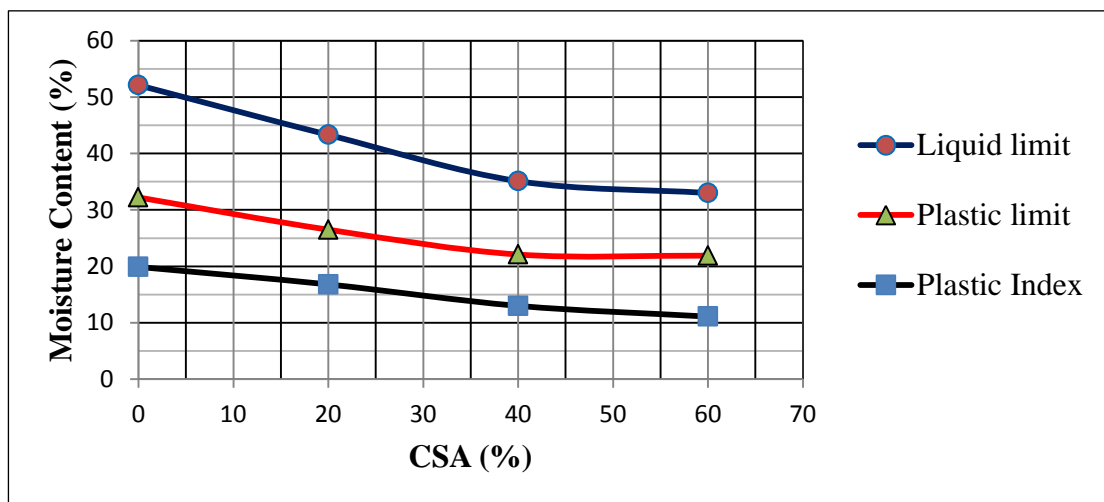


Figure 4.12: Plasticity index versus CSA content

As clearly shown in table 4.9 and fig 4.12, the plasticity index has decreased with the addition of CSA. The trend in the reduction of the PIs is the overall effect of reduction in LL and PL. The PIs decreased significantly from 20% for the natural laterite to 16.8, 13 and 11.1% with the addition of 20, 40 and 60% CSA, respectively.

However despite the continuous decrease in PI with increasing CSA addition, none of them meet the PI requirement for use as base course materials. However the laterites with 60% of CSA qualify PI requirement for use as subbase materials.

#### 4.4.3 Moisture Density Relation

To this end, compaction was carried out by blending laterite gravels with CSA at various proportions of 20%, 40% and 60% of CSA by volume of the dry laterite gravel. The method of compaction that was followed in this regard was AASHTO-T180 method B (Modified Proctor Test).

While conducting the proctor test new batches of soil were used for each moisture content increments at each compactive effort in order to avoid the effect of degradation of concretionary particles. The moisture density relations for various blending proportions of CSA are presented in Appendix D-3.

The summary of the test results of compaction of laterite blended with crushed stone aggregate at various proportions are shown in Table 4.10 and presented in Figure 4.13.

Table 4.10: Determination of MDD and OMC of laterite gravel with different proportion of CSA.

<b>CSA Content (%)</b>	<b>MDD (g/cc)</b>	<b>OMC (%)</b>
0	1.75	17.62
20	1.88	16.37
40	1.99	13.5
60	2.10	11.5

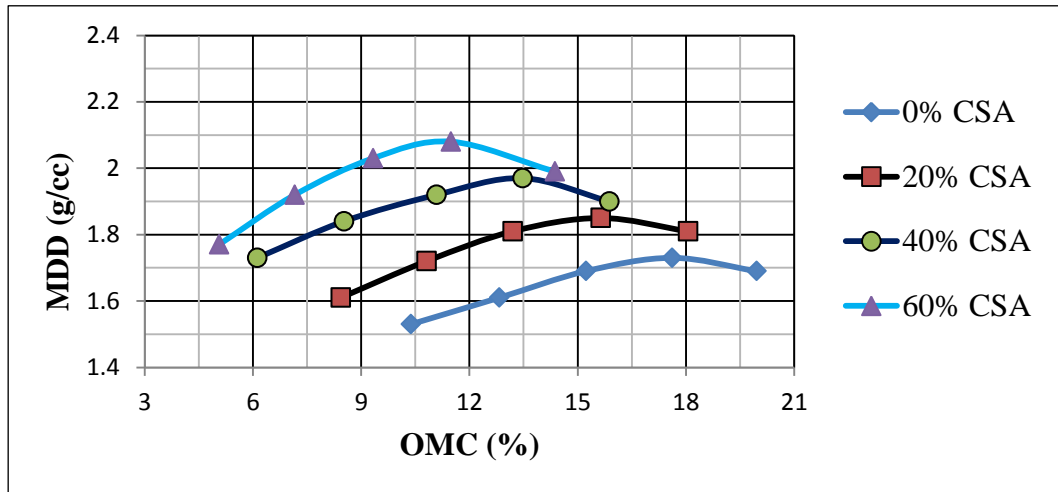


Figure 4.13: Variation of MDD and OMC with different content of CSA

It can be seen that from the above table and figure, the MDD shows a general increase and OMC shows a decrease in value with higher CSA content. The MDD increases from 1.75 to 2.10g/cc and OMC decrease from 17.6 to 11.5% for samples laterite alone and laterite with the addition of 60% CSA, respectively. The increase in MDD with the addition of CSA could be a result of both well graded property of CSA and filled the voids of natural laterite with the missing sand size particles, which resulted in a denser compact mass.

#### 4.4.4 California Bearing Ratio (CBR)

The CBR values are determined based on AASHTO T193-99. Tests were conducted with the addition of 20%, 40% and 60% CSA by volume of the dry laterite gravel. For each percentage of crushed stone aggregate, two CBR test were conducted in which one of them is for soaked condition and the remaining one is for the un soaked condition. For the unsoaked condition, penetration test took place upon the completion of the test while for the soaked condition, penetration test took place after 4 days of soaking time. Specimens are molded at respective optimum moisture content as determined in moisture density relationships. Results are tabulated Table 4.11 and illustrated in Figure 4.14 below. The details of the laboratory results are attached in Appendix D-4.

Table 4.11: Summary of laboratory results for soaked and un soaked CBR of laterite gravel with different proportion of CSA.

CSA Content (%)	CBR test results			CBR results			
	No. of Blows	Soaked	Un soaked	@ 98% of MDD		@ 95% of MDD	
		CBR	CBR	Soaked	Un soaked	Soaked	Un Soaked
0	10	14.4	21.3	28.6	41.2	25.4	39.9
	30	22.4	34.1				
	65	30.7	44.4				
20	10	27.1	37.4	63.4	72.8	55.9	65.0
	30	47.2	58.8				
	65	65	79.0				
40	10	33.2	42.6	84.6	95.5	75.7	86.2
	30	69.5	80.0				
	65	88.9	103.2				
60	10	75.8	48.4	109.5	119.8	98.9	109.5
	30	92.7	106.0				
	65	113.4	128.5				

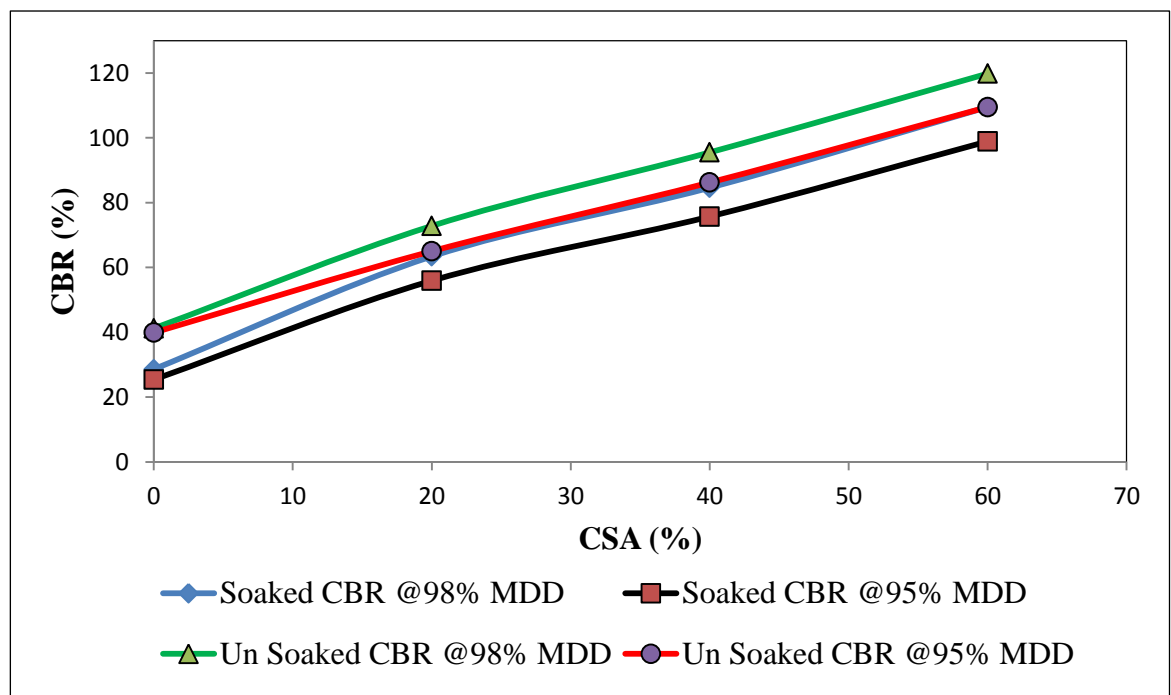


Figure 4.14: Variation of CBR with different content of CSA

It can be seen that from the above table and figure, CBR values increase almost linearly with increase in CSA content. For all content of CSA, the increase of The CBR value was as a result of the increase in maximum dry density and the decrease in optimum moisture content. CBR values are relatively higher for un soaked condition than soaked condition. The CBR value of natural laterite was 28.6% @98% MDD and 18.8 @95% MDD, which are not satisfy the ERA strength requirement to be used as a pavement material. But by the addition of crushed stone aggregate the CBR of the laterite has improved significantly. CBR values of 59.4%, 84.6% and 109.5% @98% of MDD and 52.4%, 76% and 98.4% @95% of MDD were obtained when the laterite gravel was treated by the addition of 20%, 40% and 60% CSA, respectively.

The ERA manual specification recommended that a CBR value of 80% and 30% should be attained in order to be used the material as base course and subbase, respectively. A CBR of 80% was attained by the addition of 40% and 60% CSA while 30% of CBR was attained by the addition of 20% of CSA on laterite. Accordingly the laterite with the addition of 40% and 60% CSA and with the addition of 20% CSA is the best mixture for base course and subbase material, respectively.

#### 4.4.5 Abrasion Resistance

Abrasion resistance tests were carried out on laterite gravel with different percentage of CSA in order to check the hardness to resist the abrasive effect of traffic and to judge their suitability according to ERA specification. The percentages of CSA were 20, 40 and 60% (by volume of dry laterite gravel). The summary of the test results is tabulated in Table 4.12 and presented in Figure 4.15. The detailed is presented in Appendix D-5.

Table 4.12: Results of LAA test of laterite gravel with different proportion of CSA.

CSA Content (%)	LAA result
0	70.3
20	48.7
40	37.5
60	28.8

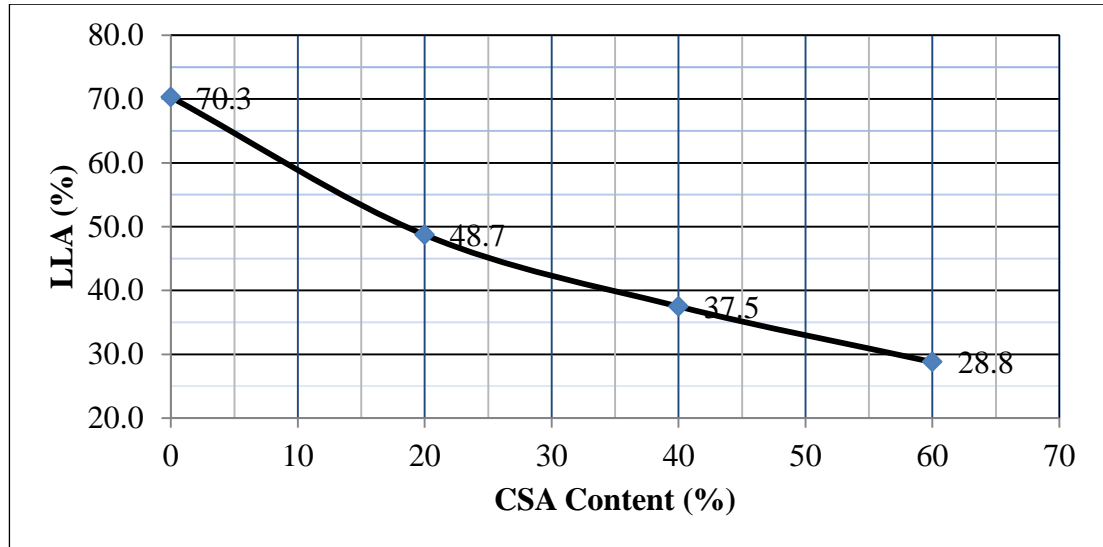


Figure 4.15: Variation of LAA value with variation CSA content

From the above figure it can be observed that, the addition of CSA to the laterite gravel was reduced the percentage of losses of material as a result of impact and abrasive action from 70.3% in the natural state to 48.7%, 37.5% and 28.8% with the addition of 20%, 40% and 60% CSA, respectively.

ERA manual set a minimum requirement of 45% and 51% LAA for a base course and sub base material, respectively. In this regard the laterite satisfy the ERA requirement to be used as base course with addition of 40% and 60% CSA and as sub base material with the addition of 20% CSA.

#### 4.4.6 Ten percent of Fines Value (TFV)

TFV tests were carried out with the addition of 20, 40 and 60 % CSA by volume of the dry laterite gravel. The summary of the test results is shown in table 4.13 and presented in figure 4.16. The detailed is also presented in Appendix D-6.

Table 4.13: Results of TFV test of laterite gravel with different proportion of CSA.

CSA Content (%)	TFV result (Soaked) [KN]
0	17.3
20	36.5
40	51.3
60	66.2

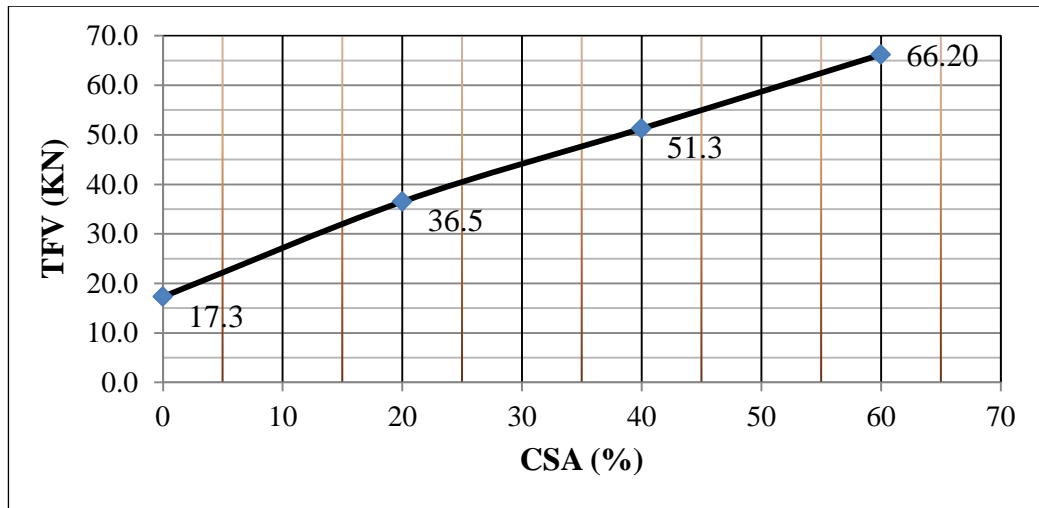


Figure 4.16: TFV of laterite at various content of CSA

It is shown in fig. 4.16 that TFV increased by 19.2KN, which is 110.9% of TFV at 0% CSA, as the CSA content increase from 0 to 20%. When increasing the amount of CSA to 40%, the TFV improved to 51.3KN, which is the 40.5% increase in TFV. Final increment of the CSA to 60% resulted in 29.1% increase in TFV. According to fig. 4.16, a laterite with 40% and 60% CSA fulfils the ERA TFV requirement for base course having a TFV above 50KN.

#### 4.5 Laterite-Lime and Laterite-Crushed stone Aggregate-Lime Mix

The Laterite-Crushed Stone Aggregate-Lime mixes were prepared by combining lime with laterite alone and lime with laterite and crushed stone aggregate. These mixtures were prepared mainly to reduce the plasticity of laterite and laterite-CSA mixes. The mixtures were prepared by four different lime contents (4%, 8%, 12%, and 16%) by dry weight of soil. The test that had been conducted on the trial mixes and the corresponding results were discussed in the following sections.

##### 4.5.1 Atterberg Limits

The test procedure adapted for the determination of Liquid limit, Plastic Limit and plasticity index are in accordance with AASHTO T89-96 and T90-00 respectively. Hand mixing in a porcelain pan is the method of mixing. The tests were carried out on laterite alone and laterite- CSA mix with different percentage of lime. The percentages of lime were 4, 8, 12 and 16% (by weight of dry soil). The result of Atterberg limit tests on each samples with various percentage of lime are shown in table 4.14.

The trend of changes of liquid limit, plastic limit and plasticity index are also presented in fig. 4.17, 4.18, and 4.19. The detailed laboratory test result is presented in Appendix E-1.

Table 4.14: Atterberg Limits of Laterite-Lime-CSA Mixtures

CSA content (%)	Parameter	Lime content (%)				
		0	4	8	12	16
0	LL	52.1	51.4	51.3	50.3	48.9
	PL	32.2	34	37.6	39.5	39.7
	PI	19.9	17.4	13.7	10.8	9.2
20	LL	43.3	43.6	42.5	40.4	39.8
	PL	26.5	28.9	30.1	30.5	31.0
	PI	16.8	14.7	12.4	9.9	8.8
40	LL	35.1	35.5	34.2	33.6	31.9
	PL	22.1	24.3	27.4	29.3	29.9
	PI	13.0	10.9	6.8	4.3	2.0
60	LL	33.0	30.7	30.0	-	-
	PL	21.9	23.1	25	-	-
	PI	11.1	7.7	5.1	NP	NP

Note: LL= Liquid Limit (%); PL= Plastic limit (%); PI = Plastic Index (%), NP = Non Plastic

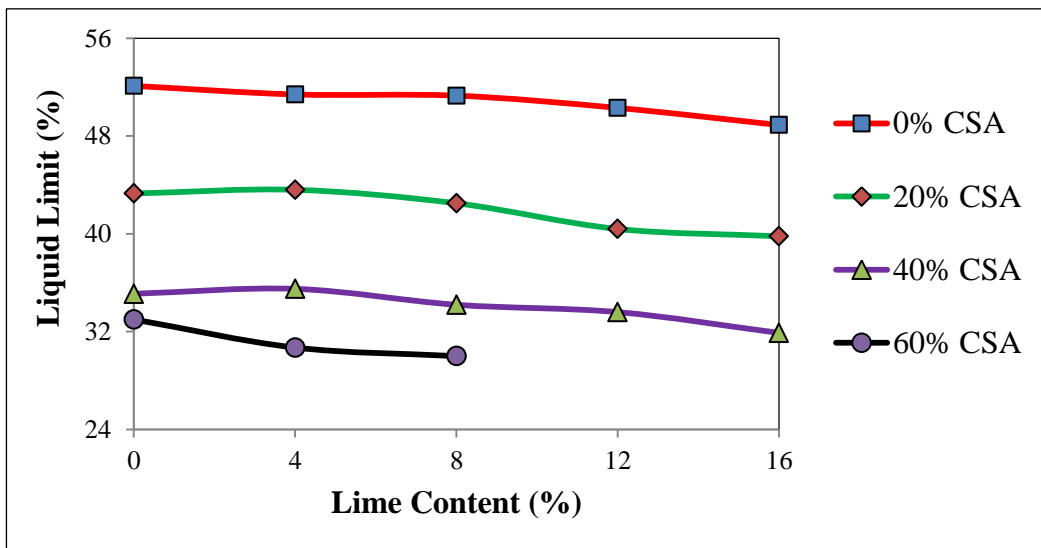


Figure 4.17: Variation of LL with different lime content for laterite-CSA-lime mixtures

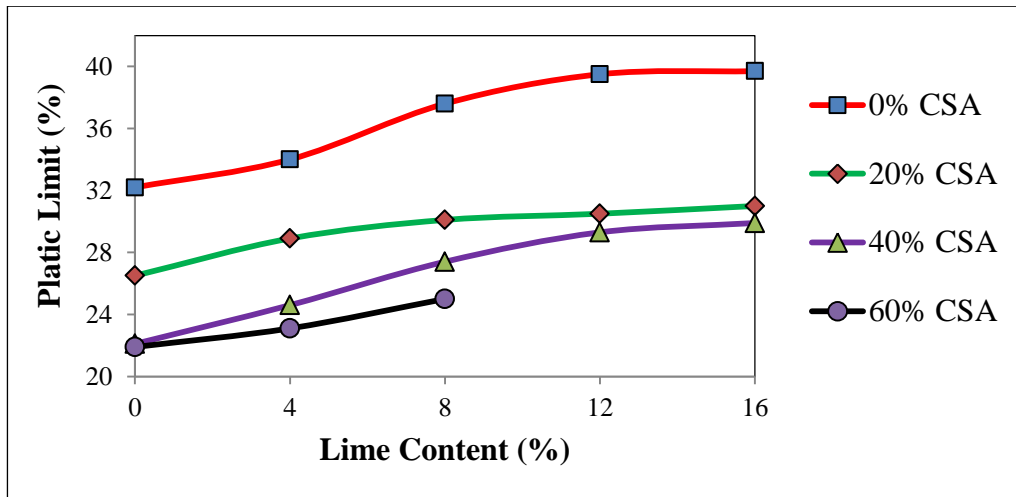


Figure 4.18: Variation of PL with different lime content for laterite-CSA-lime mixtures

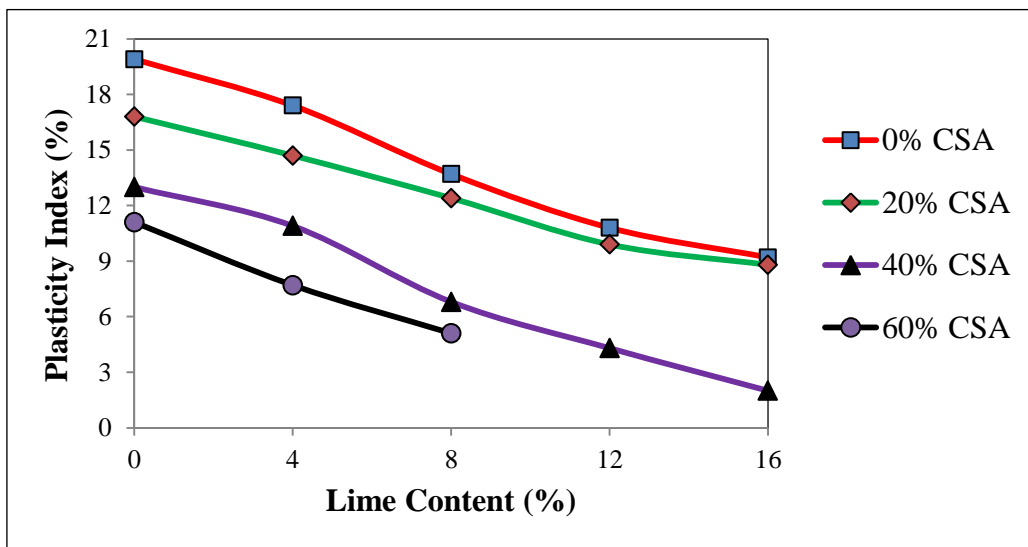


Figure 4.19: Variation of PI with different lime content for laterite-CSA-lime mixtures

In fig.4.17-4.19, it is shown that, the addition of lime produced a decrease in liquid limit and increase in plastic limit, thus causing a decrease in plasticity index. The reduction in the liquid limit with increasing lime content could be the result of the exchange reaction which flocculates the soil particles and reduces the clay- size fraction and hence the soil surface area. The increase plastic limit generally results in a reduction of the plastic properties of soil. This reaction is due to the alteration of water film surrounding the clay minerals in the soil [4]. [32] noted that increase and decrease in Atterberg limits depend on the type of soil and that increase in liquid limit occur for illitic clay soils whereas decrease in liquid limits occur for montmorillonitic clays.

ERA manual recommended 6% and 12% as maximum PI for base course and subbase material, respectively. These requirements were met at laterite-CSA-lime mixtures of 0% CSA+12% lime, 20% CSA +12% lime, 40% CSA+4% lime for subbase material and 40% CSA+12% lime, and 60% CSA+8% lime for base course material.

#### 4.5.2 Moisture Density Relation

The moisture density relations are determined based on AASHTO T99-97. Tests were conducted with the addition of 4, 8, 12 and 16% of lime on laterite alone and laterite-CSA mix. Summarized results are tabulated in Table 4.15 below. The details of the test results are attached in Appendix E-2.

Table 4.15: OMC and MDD of Laterite-Lime-CSA Mixtures.

CSA content (%)	Parameter	Lime content (%)				
		0	4	8	12	16
0	OMC	17.6	17.7	18	18.2	18.8
	MDD	1.75	1.77	1.73	1.71	1.70
20	OMC	16.4	17.2	18.2	19.2	20.3
	MDD	1.88	1.87	1.85	1.82	1.79
40	OMC	13.5	14.1	15.4	16.4	17.7
	MDD	1.99	1.98	1.95	1.93	1.90
60	OMC	11.5	11.6	12.7	13.6	13.7
	MDD	2.10	2.09	2.07	2.04	1.90

Note: OMC= Optimum Moisture Content (%) ; MDD= Maximum Dry Density (g/cc)

The nature of changes of optimum moisture content of the laterite with various lime and CSA percentage is presented in Fig. 4.20. It can be seen from the figure 4.20 that the optimum moisture content increases with increase in lime content for both Laterite-Lime and Laterite-CSA-Lime mix. This increase in optimum moisture content is a result of the pozzolanic reaction between the clay present in the soils and lime.

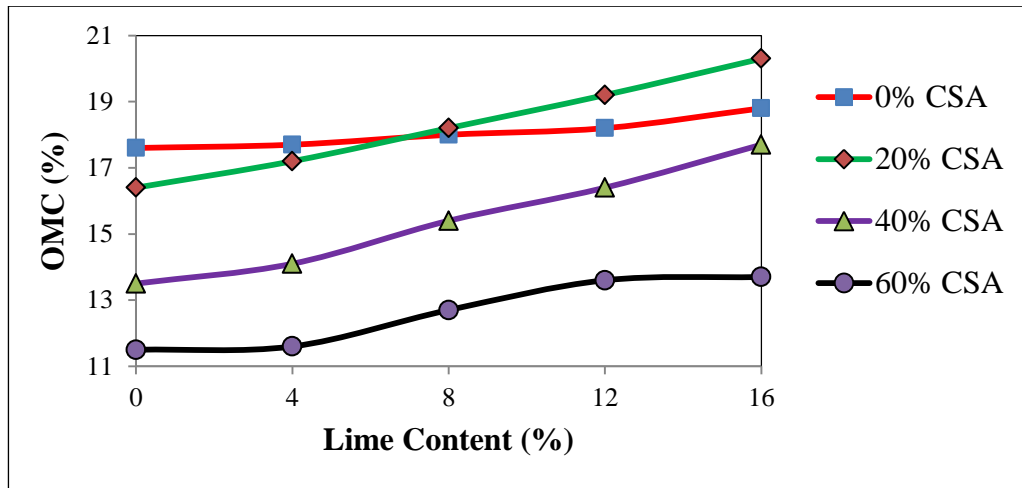


Figure 4.20: Variation of OMC with different lime content for laterite –CSA -lime mixtures

The variation of maximum dry density with lime is also shown in Fig. 4.21. The results in all cases show that addition of lime causes a decrease in the dry density. This reduction in maximum dry density may occur because of two reasons [36], [4]. Primarily, the lime causes aggregation of the particles to occupy larger spaces and hence alter the effective grading of the samples. Secondly, the specific gravity of lime generally is lower than the specific gravity of most lateritic soils.

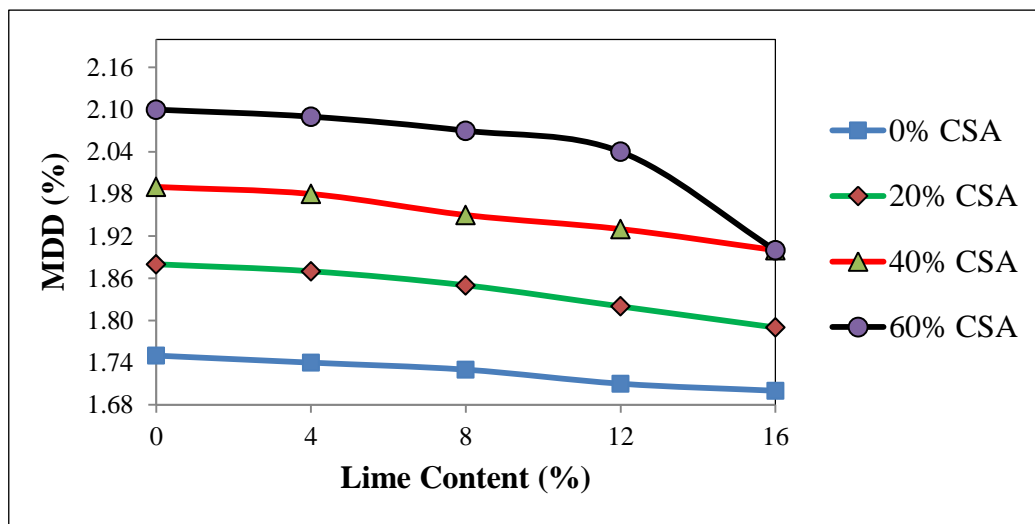


Figure 4.21: Variation of MDD with different lime content for laterite –CSA -lime mixtures

Generally, as shown in fig. 4.20 and fig. 4.21, the dry density and moisture content up on lime treatment has opposite effect with the increase of optimum moisture content and a decrease of MDD. This scenario is in agreement with the finding of [4], [32] and [36].

### **4.5.3 California Bearing Ratio (CBR)**

The California bearing ratio (CBR) test was conducted with the addition of 4, 8, 12 and 16 % of lime on laterite alone and laterite-CSA mix.

The discussion made in section 4.2.5 in relation to CBR test applies in the determination of the CBR of the blended laterite except that a one point CBR was determined in this scenario.

Before the determination of a one point CBR, the samples were kept in CBR molds by wrapping with a polyethylene sheet for the durations of 0 and 7 days.

In this case, two samples were prepared in CBR molds for each of the indicated durations in which one of them is for unsoaked condition while the other is for soaked condition. For the unsoaked condition, the durations after which the penetration test took place were 0 and 7 days; whereas for the soaked condition a further 4 days of soaking were required in addition to the indicated durations. The summary of the test results of both soaked and un soaked CBR test is shown in table 4.16. The changes of CBR values for different percentage of lime are presented in fig.4.22 and fig.4.23. The detailed laboratory test results are presented in Appendix E-3.

Table 4.16: CBR of Laterite-Lime-CSA Mixtures

CSA content %	Lime Content %	CBR Results (%)			
		0 days of curing		7 days of curing	
		soaked	Unsoaked	Soaked	Unsoaked
0	4	26.9	38.1	40.3	49.4
	8	27.5	42.4	42.9	52.4
	12	29.0	43.8	46.4	56.5
	16	31.6	46.4	51.3	57.2
20	4	51.1	63.2	62.8	74.3
	8	61.3	74.3	74.5	85.6
	12	69.7	82.7	83.2	93.8
	16	77.7	92.5	89.4	98.5
40	4	79.9	95.3	92.1	105.0
	8	90.1	104.0	101.4	113.3
	12	95.7	107.7	106.6	117.8
	16	105.9	116.1	115.0	124.1
60	4	109.6	122.6	122.4	135.6
	8	115.2	130.0	128.4	140.8
	12	122.6	139.0	134.3	146.0
	16	130.0	148.6	141.4	152.5

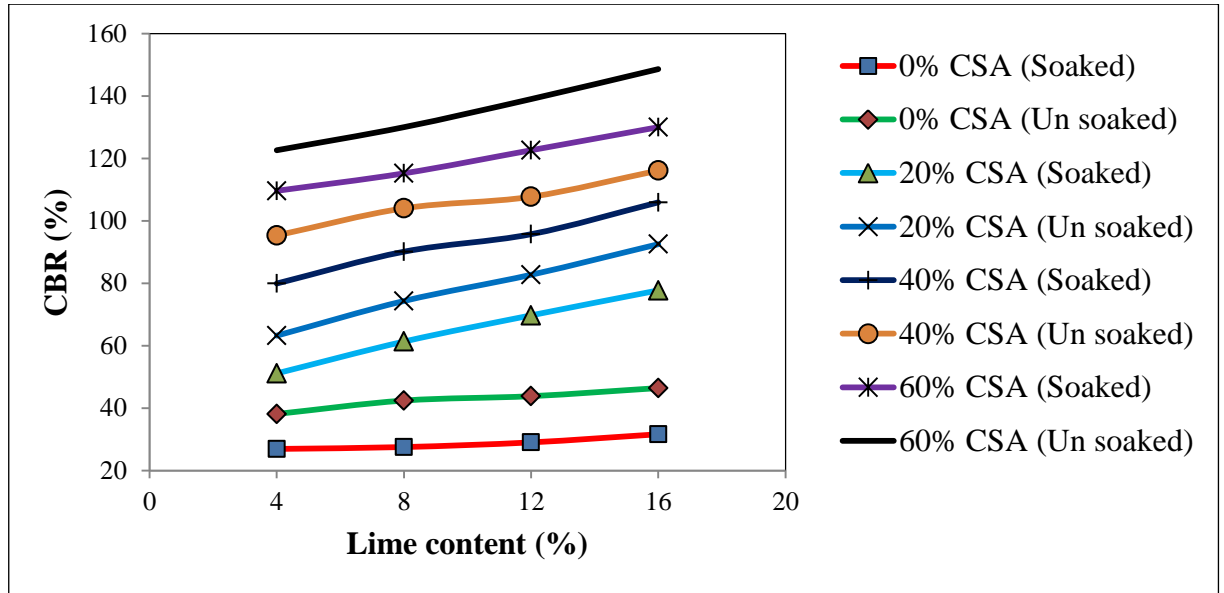


Figure 4.22: Variation of 0-day CBR with lime content for laterite-CSA-lime mixtures

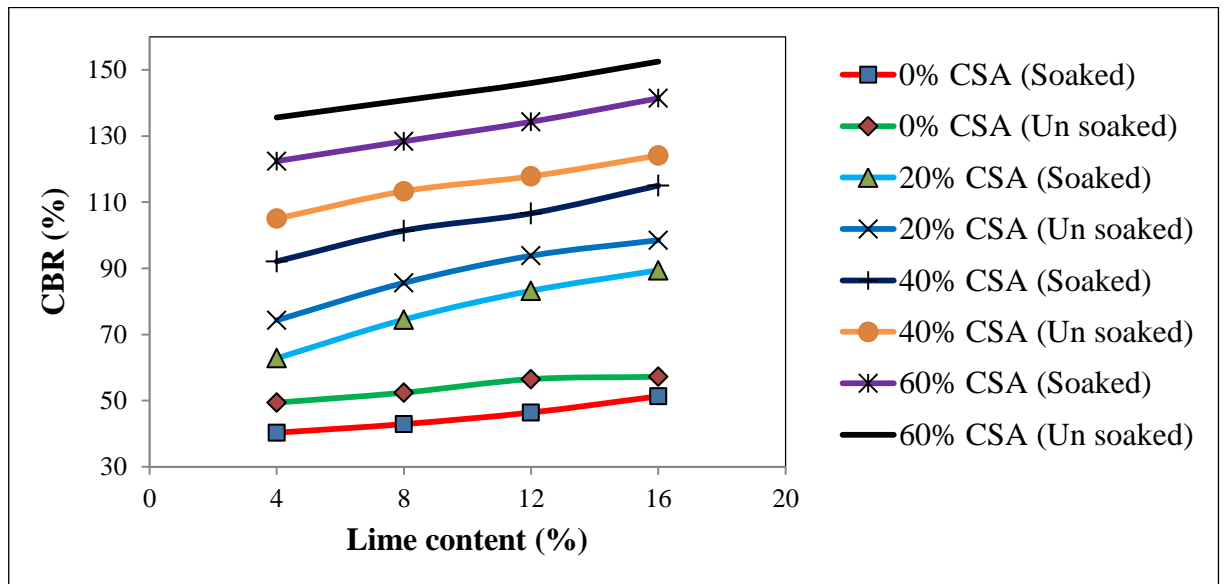


Figure 4.23: Variation of 7-day CBR with lime content for laterite-CSA-lime mixtures

It can be seen from the figure as well as the table that the CBR values increases almost linearly with increase in both lime content and durations of curing time, but with higher lime content the increase in CBR values were smaller. The increase in CBR is a result of formation of various cementing agents due to pozzolanic reaction between silica, which is available in both lime and laterite.

CBR values are relatively higher for unsoaked condition compared soaked condition. The difference in CBR between soaked and unsoaked specimen is as a result of water absorption which further weakened the specimen.

It can be seen from the above figures that soaked CBR of 29% and 46.4% were obtained when laterite was treated with 12% lime alone at 0 and 7 days of curing for soaked condition. The CBR increased significantly from 28.6% for the untreated laterite to 128.4% at a laterite –CSA-lime mixture of 60% CSA and 8% lime at 0 days of curing soaked condition. CBR of 100% for CB1 base course was attained with 40% CSA + 8% lime and 60% CSA + 4% lime at 7 days of curing soaked condition.

#### **4.5.4 Unconfined Compression Strength (UCS)**

The purpose of the unconfined compression test is to measure the strength of the compacted soil-lime mixture. In this study, the test was carried out according to ASTM D 5102, Method B in which 4-inch (101.6mm) diameter and 4.584-inch (116.4mm) mold is used for preparing the compacted specimen.

The compacted specimens were prepared based on ASTM D 698. The molded specimens were cured by covering them with plastic sheets for 7, 14 and 28 days. At the end of the curing period, the specimens subjected to compression tests.

The compressive strength of the specimens was then calculated by dividing the maximum compressive load with the cross sectional area of the specimens.

Since 4-inch mold was used, the length-to-diameter ratio (L/D) of the specimens is less than two. Consequently, the compressive strength of these specimens needs to be reduced using certain correction factors. The reduction factors in ASTM C-32 were utilized to correct the compressive strength of the specimens in this study. According to ASTM C-32, the correction factors in table 4.17 are recommended for various L/D ratios, and interpolated factors are suggested for values not given.

Table 4.17: Correction Factor [ASTEM C-32]

Length-to-Diameter Ratio [L/D]-X	Strength Correction Factors-Y
1.75	0.98
1.50	0.96
1.25	0.93
1.00	0.87
Interpolating Equation	$Y=0.2133X^3-1.04X^2+1.7667X-0.07$

The results of unconfined compression tests on the laterite with various mix proportions of lime and CSA are shown in Table 4.18.

Table 4.18: UCS of Laterite-Lime-CSA Mixtures

CSA content %	Lime Content %	USC Results (MPa)					
		USC of 4-inch Specimen(MPa)			Corrected USC (MPa)		
		7 days	14 days	28 days	7 days	14 days	28 days
0	4	0.92	1.30	1.42	0.84	1.18	1.29
	8	1.23	1.41	1.50	1.12	1.28	1.37
	12	1.31	1.46	1.56	1.19	1.33	1.42
	16	1.40	1.51	1.60	1.27	1.37	1.46
20	4	1.63	1.71	1.94	1.48	1.56	1.77
	8	1.70	1.79	2.11	1.55	1.63	1.92
	12	1.76	1.85	2.18	1.60	1.68	1.98
	16	1.88	1.96	2.20	1.71	1.78	2.00
40	4	2.28	2.40	2.53	2.07	2.18	2.30
	8	2.38	2.50	2.60	2.17	2.28	2.37
	12	2.50	2.67	2.77	2.28	2.43	2.52
	16	2.62	2.74	2.92	2.38	2.49	2.66
60	4	3.30	3.44	3.81	3.00	3.13	3.47
	8	3.59	3.70	4.00	3.27	3.37	3.64
	12	3.90	4.10	4.32	3.55	3.73	3.93
	16	4.15	4.23	4.51	3.78	3.85	4.10

Figures 4.24 to 4.26 also show the variation of UCS with lime for the laterite- CSA-lime mixtures for 7, 14, and 28 days, respectively. The results show clearly that the compressive strength increases with increase in both lime content and duration of curing time. As clearly observed in figures 4.24 to 4.26, there were only small increases in strength among specimens cured at 7, 14 and 28 days. The low increase in strength may be due to the small amount of pozzolanic material in the natural laterite. In order to form a cementation of particles, the aluminous and silicious material in the soil should be react with lime. Unfortunately, laterite gravels are usually low in content silicious material, which might be the main reason to gain low increase in strength with longer curing periods.

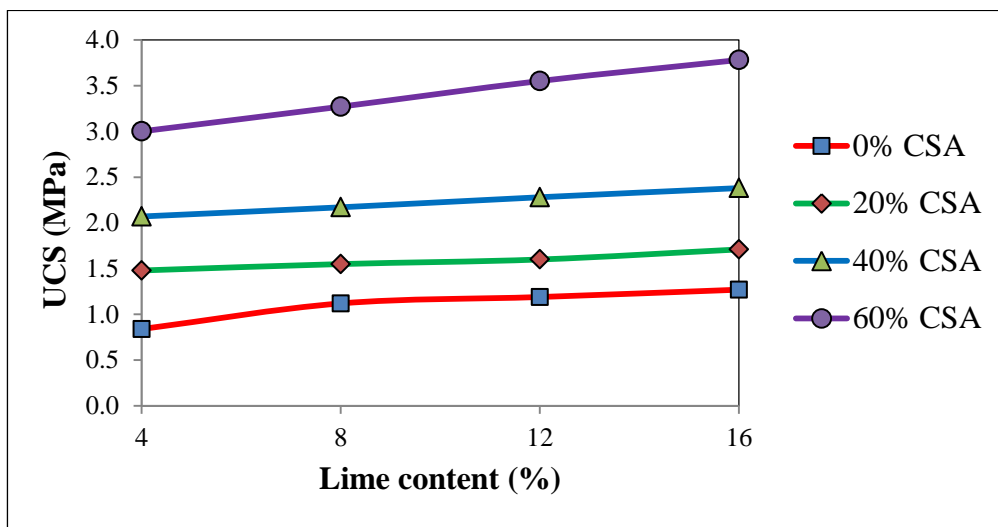


Figure 4.24: Variation of 7-day UCS with lime content for laterite-CSA-lime mixtures

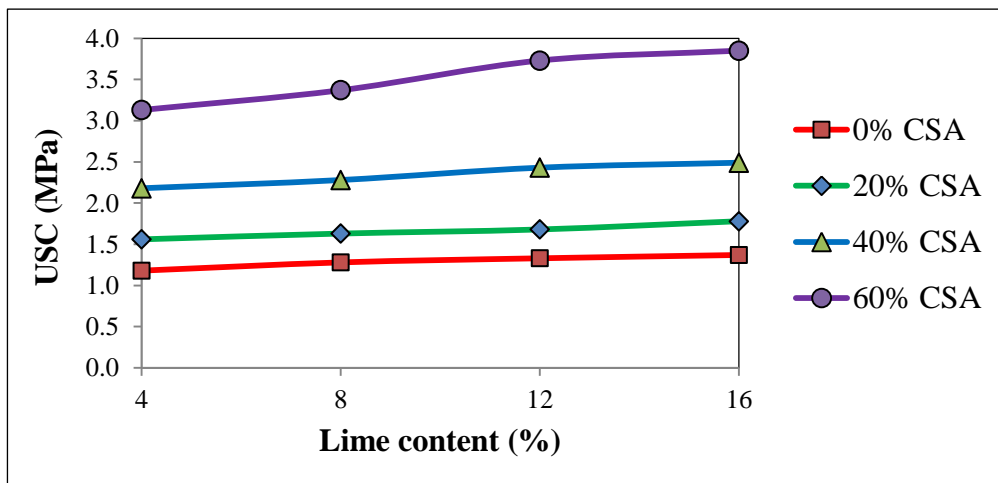


Figure 4.25: Variation of 14-day UCS with lime content for laterite-CSA-lime mixtures

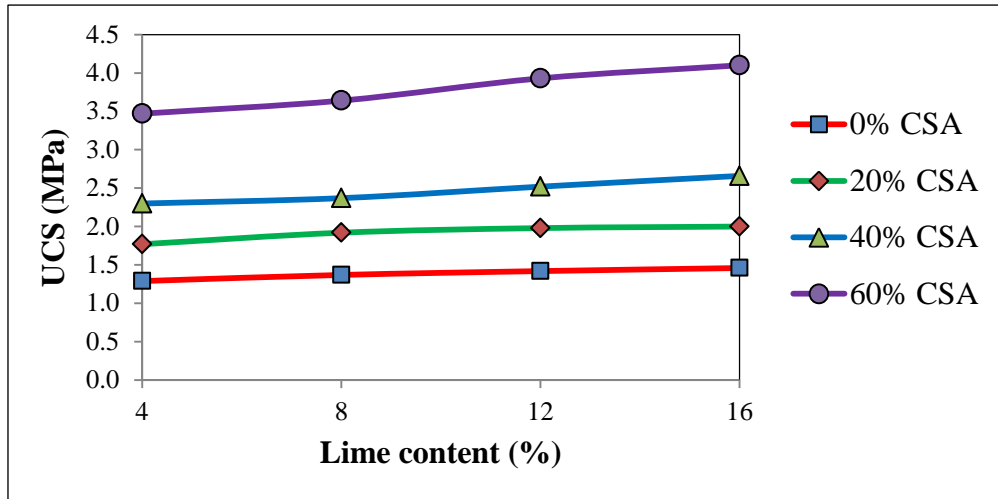


Figure 4.26: Variation of 28-day UCS with lime content for laterite-CSA-lime mixtures

#### 4.5.5 Wetting and Drying Test (Durability)

This is one of the durability tests which are used to determine the resistance of the compacted soil-lime specimen to various weathering conditions. The weathering conditions in this test method are simulated by frequent wetting and drying of the specimens. The test was conducted according to ASTM D 559. These test methods are applied to soil treated with cement, but it can be generalized for other soil-binders, such as soil treated with lime.

The test is carried out on laterite alone and laterite-CSA mix with the addition of 8 and 12% of lime as these specimens have strengths close to the minimum required for base course.

The specimens are submerged in potable water for 5-hours at room temperature after seven days curing period. Then they are weighed and measured, and put in an oven at 71<sup>o</sup>C for 42 hours. At the end of drying, the specimens are removed from the oven and weighed and measured after they are allowed to cool. The soil-lime loss specimens are then scratched with two firm strokes of a wire-brush within their full height and width. The specimens are then weighed and measured and the process repeated 12-times. After the 12-cycle process, the specimens are oven-dried at 110<sup>o</sup>C to constant weight. The data collected in due course are then used to compute the soil-lime loss of the specimens.

The results of the wet-dry test of the trial mixtures are summarised in table 4.19 and detail test results are appended as Appendix E-4.

Table 4.19: Wet-Dry Test Results for Laterite-Lime-CSA.

CSA Content (%)	Lime Content (%)	Design		Actual		Soil-Lime loss (%)
		MDD	OMC	MDD	OMC	
0	8	1.73	18.0	1.72	18.1	10.8
	12	1.71	18.2	1.69	18.0	9.7
20	8	1.85	18.2	1.83	18.4	8.1
	12	1.82	19.2	1.81	19.3	6.7
40	8	1.95	15.4	1.93	15.3	7.3
	12	1.93	16.4	1.92	16.5	6.5
60	8	2.07	12.7	2.06	12.9	7.0
	12	2.04	13.6	2.03	13.7	6.4

Note: OMC= Optimum Moisture Content (%) ; MDD= Maximum Dry Density (g/cc)

#### 4.5.6 Optimum Binder Content

Determination of the optimum binder content (OBC), so that the stabilized soil fulfils both the strength and durability requirements, is the primary objective of the soil-lime stabilization process. Among the various approaches stabilization guidelines and the requirements that had been specified in BS part: 2 and Road note 31 were employed for determining the OBC.

The minimum acceptable strength of a stabilized material depends on its position in the pavement structure and the level of traffic. It must be sufficiently strong to resist traffic stresses but upper limits of strength are usually set to minimize the risk of reflection cracking.

The unconfined compression test results indicated that the unconfined compressive strength (UCS) of the soil-lime specimens increases linearly with the cement content as shown figs. 4.24 to 4.26. Based on US army specification as discussed in section 2.4.2.3, a stabilized material should have a compressive strength value in terms of UCS test 5.2 MPa. In this regard none of the CSA-lime of cylindrical specimens achieved this minimum value of compressive strength.

To compare the strengths of the trial mixes with the requirements of ERA Pavement Design, the UCS values of the trial specimens have to be converted to equivalent value of the 150mm cube specimen since the values in the PDM has been established on the basis of 150 x150mm cubical specimens. Accordingly, the correction factors set by Road Note 31 were used to derive appropriate correction factor based on the height to diameter ratio of the specimens.

Table 4.20: Correlation factors for Cube and Cylindrical Specimens [39].

Sample Type	Height/Diameter	Correction Factor
200×100mm	2.0	1.25
115.5×105mm	1.1	1.04
127mm×152mm	0.8	0.96
116.4mm×101.6mm	1.15	1.05 <sup>#</sup>
Note: # interpolated factor for the actual sample type		

Accordingly, the results of UCS tests obtained at 28 days curing as specified in US army were changed by multiplying with 1.05 to values supposed to be equivalent with 150mm cubic specimens.

Table 4.21: Adjusted Unconfined Compressive Strength.

CSA Content %	Lime Content %	Corrected UCS of 116.4*101.6mm [MPa]	USC of 150mm cube [MPa]
0	4	1.29	1.36
	8	1.37	1.43
	12	1.42	1.49
	16	1.46	1.53
20	4	1.77	1.85
	8	1.92	2.02
	12	1.98	2.08
	16	2.00	2.10
40	4	2.30	2.42
	8	2.37	2.48
	12	2.52	2.65
	16	2.66	2.79
60	4	3.47	3.64
	8	3.64	3.82
	12	3.93	4.13
	16	4.10	4.31
ERA PDM and Road note 31 Requirements		CB1	3.0 – 6.0
		CB2	1.5 – 3.0
		CS	0.75 – 1.5

The results in table 4.21 indicate that all of the trial laterite-CSA-lime mixtures satisfy the strength requirement of ERA Pavement Design Manual and Road note 31 for stabilized pavement material.

Base on the results of the 12-cycle wet-dry test [Table 4.19], conducted on specimens with 8% and 12% lime contents for all laterite-CSA-lime mixtures, the soil-lime losses of all laterite-CSA-lime mixtures except 0% CSA + 8% lime, are below the maximum limit of 10% which is specified in Portland cement association for A-2-6 and A-2-7 soil types.

Depending on the strength and durability test results, the minimum binder content for each laterite- CSA mixtures are summarized in table 4.22.

Table 4.22: Optimum Binder Content

Sample type	Optimum Binder Content (%)
0% CSA	12% for sub base
20% CSA	8% for CB2 base course
40% CSA	8% for CB2 base course
60% CSA	8% for CB1 base course

Generally, based on CBR of 7 days curing soaked condition, 28 days of UCS, durability and PI test results and criteria's the following combination are suggested for use as flexible pavement material.

Table 4.23: Suggested Combinations for Use as Flexible Pavement material

Combination	CBR (%)	UCS (MPa)	Durability (%)	PI (%)	Suggestion
0% CSA+0% lime	28.6	-	-	19.9	Subgrade
0% CSA+12% lime	46.4	1.49	9.7	10.8	Subbase
20% CSA+12% lime	83.2	2.08	8.1	9.9	Subbase
40% CSA+12% lime	101.4	2.48	7.3	4.3	CB2 base course
60% CSA+8% lime	128.4	3.82	7.0	5.1	CB1 base course

#### 4.6 Statistical Analysis

Following the laboratory procedures and data collection described in the previous topics, to evaluate the significance of CSA and lime, a statistical analysis was performed using an Analysis of Variance (ANOVA). Due to time constraint, sample size and budget limitation, the analysis was limited to CBR, LAA and TFV test for laterite-CSA mix and Atterberg limit test for laterite-CSA-lime mix.

A one-way ANOVA in terms of partitioning the total variation of all sample scores used in to between group and within group variation. The four (0%, 20%, 40% and 60% CSA) and five (0%, 4%, 8%, 12% and 16% lime) groups of independent variables are considered for CSA and lime treatment, respectively. An individual result of the group on the dependent variable can be expressed in terms of four and five additive components.

**Testing the null hypothesis**

On this research the analysis of ANOVA consists of two random samples from each of four and five independent groups. The null hypothesis (H<sub>0</sub>) and hypothesis (H<sub>1</sub>) were defined as;

H<sub>0</sub>: Quantity of stabilizer does not affect the dependent variable significantly.

$$\mu_1 = \mu_2 = \mu_3 \dots = \mu_k$$

H<sub>1</sub>: Quantity of stabilizer affects the dependent variable significantly.

$$\mu_1 \neq \mu_2 \neq \mu_k$$

The null hypothesis is not rejected if there is no difference in the means of each independent group. However, when the means differ significantly the null hypothesis will be rejected. Using this analysis testing of the hypothesis is done at 0.05 level of significance.

The statistical analysis performed for all sample type tested is summarized in table 4.24. The detailed laboratory test results are presented in Appendix F.

Table 4.24: Summary of Statistical analysis for different sample type

Sample type tested	Dependent Variable	F-Value	P-Value	F <sub>cri</sub> -Value	Remark
Laterite-CSA mix	CBR	226.97	6.38E-05	6.59	Reject null hypothesis
Laterite-CSA mix	LAA	180626	1.02E-10	6.59	Reject null hypothesis
Laterite-CSA mix	TFV	2480.1	5.41E-07	6.59	Reject null hypothesis
Laterite-CSA-Lime	PI	14.31	6.03E-03	5.192	Reject null hypothesis
Laterite-CSA-Lime	PI	17.85	3.65E-03	5.192	Reject null hypothesis
Laterite-CSA-Lime	PI	191.52	1.20E-05	5.192	Reject null hypothesis
Laterite-CSA-Lime	PI	241.25	6.70E-06	5.192	Reject null hypothesis

The decision to accept or reject the null hypothesis (H<sub>0</sub>) is made by comparing the test statistics (computed F ratio) with critical F value from the table. If the computed F ratio exceeds the critical F value, the hypothesis is rejected; if not, the hypothesis is not rejected.

The ANOVA result of CBR, LAA, TFV and Atterberg limit shows that, the F ratio exceeds the critical value at 0.05 level of significance, and therefore the null hypothesis is rejected on all tests. This indicates that, the quantity of CSA and lime alters CBR, LAA, TFV and PI values of laterite gravel significantly. Hence this research accepted that CSA and lime treatment have a significant effects on the strength and plasticity of laterite gravel, respectively.

## 5. MECHANISTIC EVALUATION OF LAYER RESPONSE

The performance of the stabilized base was analysed considering utilization of the crushed granular base as base case scenario. The deflection, fatigue and rutting of the pavement structure with crushed granular base was compared against stabilized base.

For the sake of performance evaluation, pavement structures, which accommodate T6 [6-10m ESA] class traffic load over S5 [15% – 30% dCBR] class subgrade, was selected from structural catalogue of ERA PDM 2013. The T6 traffic class was chosen considering that the development of vast farmlands within the study area induces massive traffic volume, and S5 subgrade class was used based on the CBR result of natural laterite gravel.

Material property and thickness of different layers used in KENPAVE are given in table 5.1

Table 5.1: Material property used in KENPAVE Analysis

	HMA	Base layer		Subbase	Subgrade
		CGB	SB		
Thickness (cm)	10	15	15	15	-
Resilient Modulus (KPa) RS=3000*CBR <sup>0.65</sup> for CBR>10%	4e <sup>6</sup>	521121.2	485889.6	150,000	80,000
Poisson's ratio	0.35	0.4	0.2	0.4	0.45
Note; CGB= Crushed Granular Base, SB= Stabilized Base					

The detail outputs of KENPAVE for both Crushed granular and stabilized base are appended in Appendix G.

### 5.1 Deflection

Cementitiously stabilized pavement layers have been known to provide greater structural support than traditional flexible pavements, while sharing similar noise, friction, and smoothness properties. High structural support of a pavement structure has been traditionally associated with low deflections at the surface (i.e., deflection measurements are known to be reduced when the bearing capacity of the road is high). In addition, a reduction of deflection under an applied load reduces the traffic-induced stresses and strains within the layers of the structure. Therefore, a structure that provides lower deflection measurements would tend to reduce the layers' state of stress and strain, causing the pavement structure to be less affected (damaged) by the loading conditions. The deflection analysis performed is shown in table 5.2 and presented in figure 5.1.

Table 5.2: Surface Pavement Deflections of Various Structures with Depth

Depth	Deflection	
	Crushed granular base	Stabilized base
0.001	0.65791	0.65170
9.99	0.65689	0.65089
25	0.59933	0.59644
40.001	0.51429	0.51266

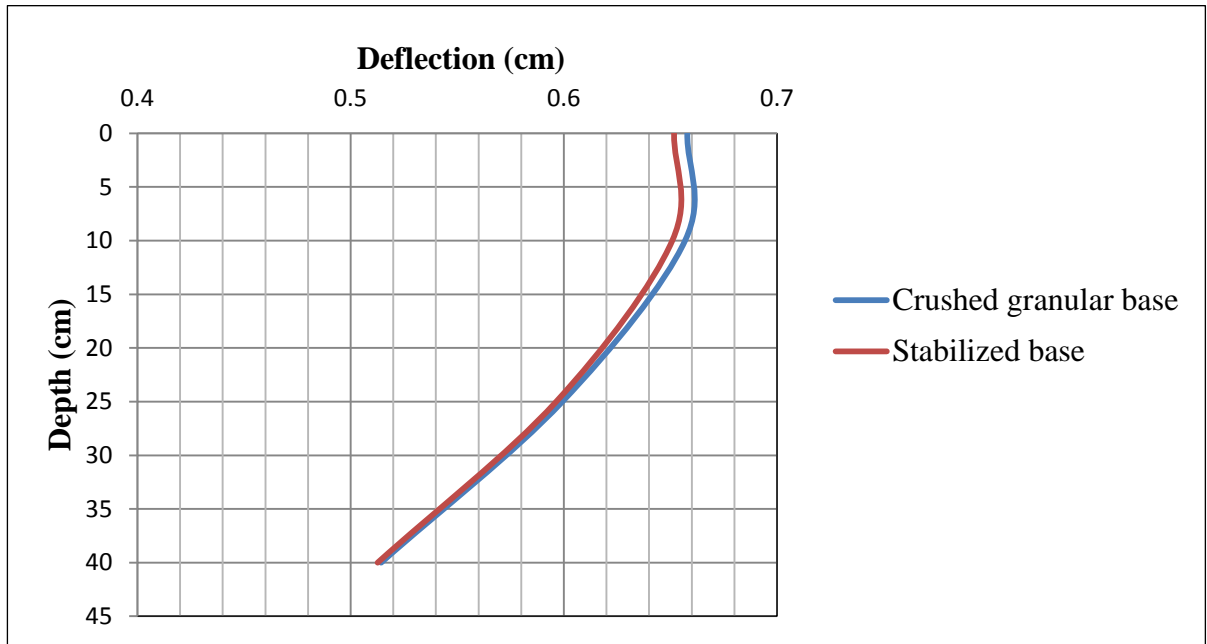


Figure 5.1: Surface Pavement Deflections of different base materials

The figure 5.1 shows that the deflections at the pavement surface are increase as the Poisson’s ratio of the base increase. This is somewhat expected since an increase in Poisson’s ratio is an indication of greater volume change.

A larger volume change (i.e., horizontal movement in the base and sub grade layers) would indicate that the asphalt layer has to vertically deflect more. In this case, the Poisson’s ratio of the base increased from stabilized base [0.2] to Crushed granular base [0.4]. The relatively maximum deflection predicted when the crushed granular base was used was 0.66mm.

## 5.2 Horizontal stress and strain

A pavement structure, when subjected to a load, presents stress and strain responses that are a function of the load magnitude, load location, pressure, and material properties, among other factors. Horizontal stresses have been investigated to understand their effect on failure of HMA. In addition, horizontal strains have also been investigated to predict HMA and stabilized material fatigue. The output of horizontal stress analysis is presented in figure 5.2.

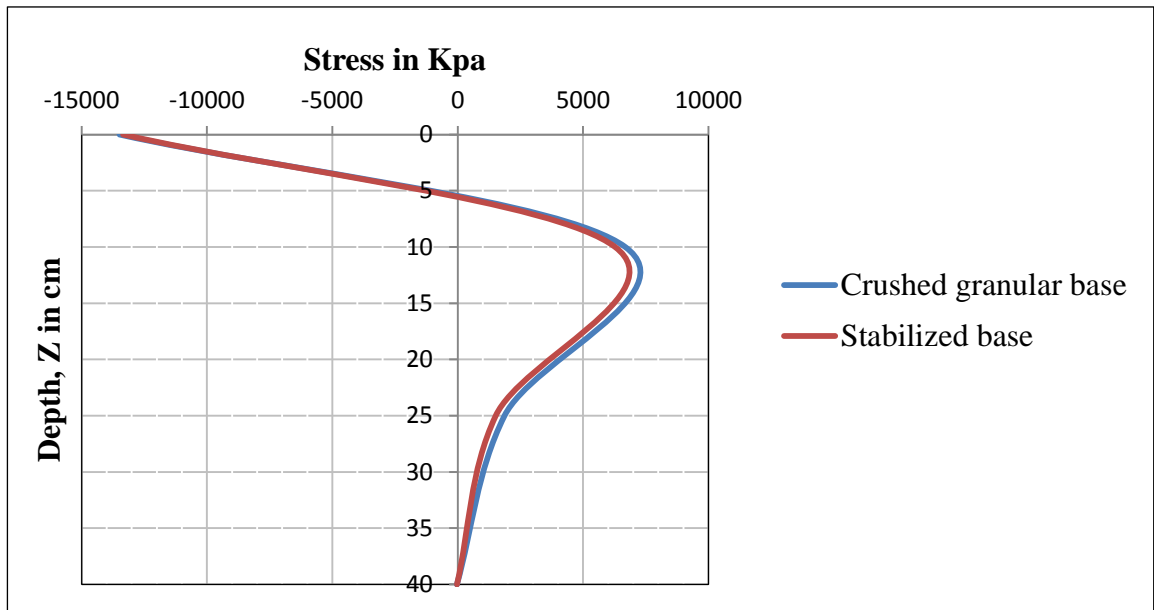


Figure 5.2: Horizontal Stresses Analysis with Varying Base layer

It can be seen from the figure 5.2 that that relatively higher compressive and tensile stresses can be observed in the HMA layer of the Crushed granular base scenario. In the case of stabilized base, the magnitude of both compressive and tensile stresses is reduced. For both scenario (i.e, Crushed granular base and stabilized base), the highest compressive stress is located at the top of the HMA layer, whereas the highest tensile stress is located at the bottom of the HMA layer.

The horizontal strain output obtained from KENPAVE is presented in figure 5.3.

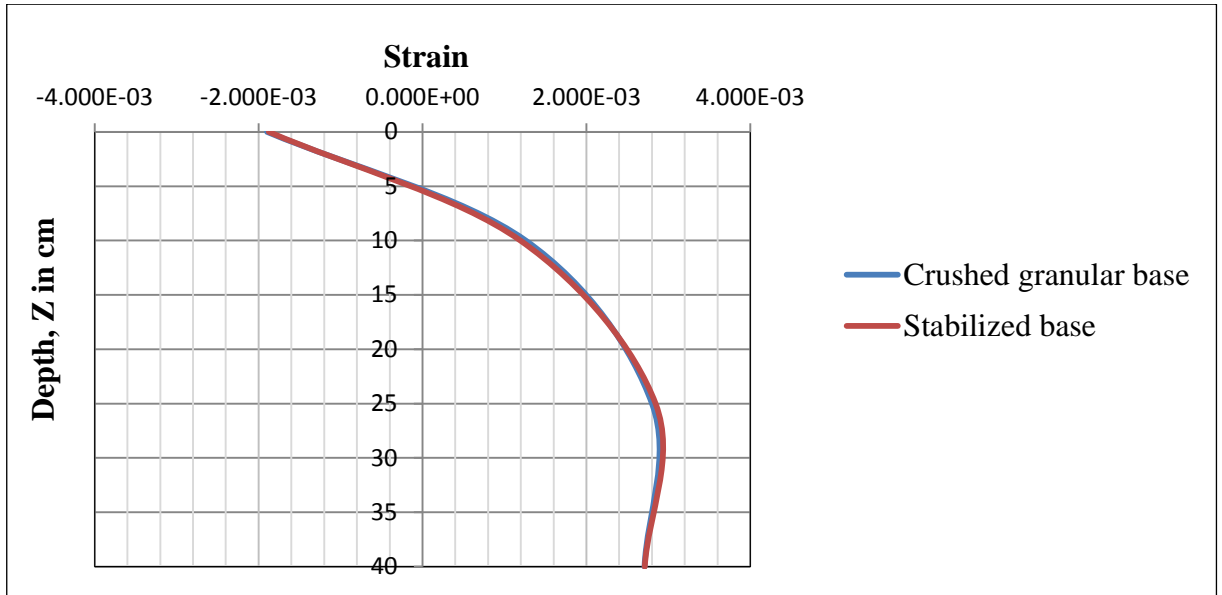


Figure 5.3: Horizontal strains Analysis with Varying Base layer

It is shown in the figure 5.3 that the tensile strain at the bottom of HMA, which is the most commonly used point of interest when investigating flexural fatigue damage, is larger in Crushed granular base than when a stabilized base was used. This suggests that the chance of having fatigue failure in the HMA when using a stabilized base is lower than Crushed granular base.

### 5.3 Vertical strains

Vertical strains have been used in the past to determine how much deformation is likely to occur on top of subgrade and thus help determine rutting due to subgrade permanent deformation [21]. The output of vertical strain analysis of KENPAVE is presented in figure 5.4.

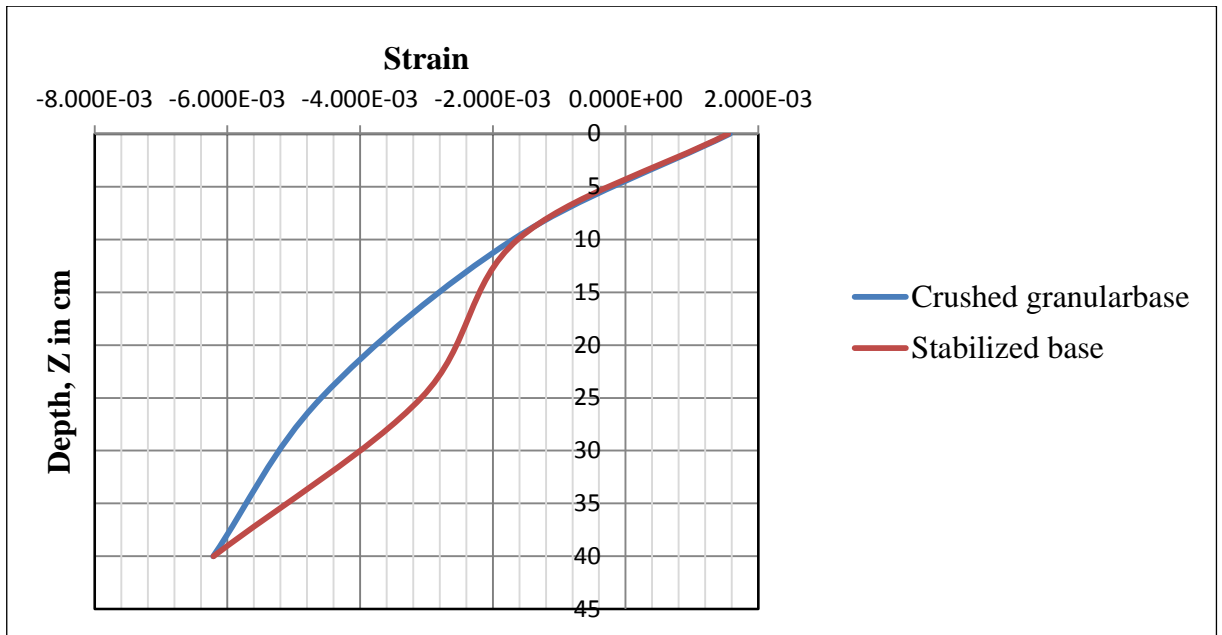


Figure 5.4: Vertical strains Analysis with Varying Base layer

From the vertical strain comparison shown in fig. 5.4, it can be noted that stabilized base has lower vertical strain than Crushed granular base. The significant reduction of vertical strains at top of the subgrade at a depth just below 40 cm suggests that rutting due to permanent deformation of the subgrade is greatly minimized or even unlikely to occur.

## **6. CONCLUSIONS AND RECOMMENDATIONS**

### **6.1 Conclusions**

In this study, the engineering characteristics of laterite gravels in order to evaluate their uses as flexible pavement materials based on four options were studied. The first one was to evaluate the appropriateness of natural laterite gravel as flexible pavement material. The second option investigated was using laterite gravel as mechanically stabilized layer with the addition of 20%, 40% and 60% CSA. The third and fourth option were to examine the appropriateness of natural laterite gravel with the addition of lime on laterite alone and mechanically stabilized laterite for flexible pavement material according to ERA specification. So based on the result of this experimental study on various types of stabilization the following conclusions are made.

#### **6.1.1 Natural Laterite Gravel**

- The geochemical test results indicate that sample A and B have sesquioxide ratio of less than 1.33. This shows that the blended laterite (Sample A + B) is pure laterite.
- Laterite gravel can be considered as marginal (low quality) material for a variety of reasons such as inadequate gradation, inadequate plasticity and inadequate strength.
- Best utilization of laterites for sub base can be achieved by using envelope 'C' of ERA's technical specification manual among the other envelopes. However, with respect to base course laterite has not been fulfilling the gradation of ERA specification due to the lacking of sufficient sand size particles.
- The laterite gravel is a weak material with CBR value <30% and LAA value >51% in addition to having high plasticity. Due to this reason, laterite gravels are not suitable for sub base as well as for base course materials for high traffic volume unless improved their strength in some ways. However, with respect of sub grade specification laterite gravel can be used as sub grade material satisfactorily.

#### **6.1.2 Mechanical Stabilization**

- CSA stabilization of laterite gravel was conducted with the addition of 20%, 40% and 60% CSA by volume of laterite gravel.
- The sieve analysis was carried out on mechanically stabilized laterite before and after compaction. Accordingly, it has been showed that all the blend proportions satisfy gradation requirements for sub base material as determined in after compaction state.

Whereas laterite with 60% of CSA fulfil the requirement for base course material in addition to sub base.

- With regard to particle strength and bearing capacity, laterite with 60% and 40% CSA fulfil CBR, LAA and TFV requirement of ERA manual to be used as base course and sub base material, respectively. However none of them meet the PI requirement.

### **6.1.3 Lime Stabilization**

- The plasticity of laterite was reduced with the addition of lime as a result of increased in plastic limit and a slight decreased in liquid limit.
- The addition of lime has resulted in an increase in optimum moisture content and reduction of maximum dry density for the same compaction effort.
- Optimum lime content in improving the strength of laterite gravel for sub base quality is found out as 12%. At this lime content, the laterite satisfies both the strength and durability requirements that were set by different agencies for sub base material.

### **6.1.4 Mechanical-Lime Composite Stabilization**

- Lowest PI values 4.3 and 5.1% were recorded at a combination of 40% CSA+12% lime and 60% CSA+8% lime, respectively. These values are lower as compared to the allowable maximum limit of 6% which is specified in ERA manual. These mixtures are also satisfying both the strength and durability requirements for flexible pavement materials.
- CSA and lime can be concluded as effective stabilizing agents for improvement of laterite gravel for use as flexible pavement materials.

### **6.1.5 Deflection, Stress and Strain**

The use of a stabilized base layer under the HMA surface course provided the following benefits:

- Deflections at the HMA surface are reduced as the stiffness of the base layer increases.
- Fatigue (bottom-up) cracking in the HMA, due to high tensile strain at the bottom of the layer, is minimized.
- Permanent deformations (rutting) due to vertical compressive strains and stresses in the unbound subbase and, most importantly, subgrade layer are minimized.

## 6.2 Recommendations

Based on the finding of the research the following are the main recommendations.

1. A special gradation requirement should be provided for marginal materials, such as laterite gravel whose gradation changes significantly up on compaction.
2. Preparation of guidelines for laterite material is essential to utilize laterite material which is found thorough out the country.
3. A full scale experiments should be carried out in Ethiopia in order to examine the field performance of the laterite gravel blended with crushed stone aggregate and lime as base course materials for different area of laterites in Ethiopia.
4. The result obtained may be used as basis for further research in the area of laterite material found in different part of the country.
5. It is recommended to use mechanically stabilized laterite with 60% CSA as base course and check its performance.
6. The following topics are recommended for future studies:
  - The effect of higher PI value beyond the spec. on the performance of Base course material.
  - Check the economic advantage of laterite stabilized with different CSA and lime content.
  - The effect of fly ash on lime stabilized laterite gravel.

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## APPENDICES

## APPENDIX A

### BASIC TEST RESULTS FOR EACH LATERITE SAMPLES

#### APPENDIX A-1 SIEVE ANALYSIS TEST

Table A-1.1 Result of sieve analysis test for sample A (before compaction)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		4100		GM	PP	
Mass dry sample after wash = (M2)		3881.1				
Total passing 0.075mm sieve = (M1-M2)		218.9		2.8	100.7	
Sieve Size (mm)	Weight retained (g)	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	0.0	0.0	0.0	100.0	<b>80</b>	<b>100</b>
19	260.8	6.4	6.4	93.6	<b>60</b>	<b>80</b>
9.5	1686.1	41.1	47.5	52.5	<b>45</b>	<b>65</b>
4.75	1518.1	37.0	84.5	15.5	<b>30</b>	<b>50</b>
2.36	248.8	6.1	90.6	9.4	<b>20</b>	<b>40</b>
0.425	90.3	2.2	92.8	7.2	<b>10</b>	<b>25</b>
0.075	77.1	1.9	94.7	5.3	<b>5</b>	<b>15</b>
Pan	218.9	5.3	100.0			

Table A-1.2 Result of sieve analysis test for sample B (before compaction)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		3570		GM	PP	
Mass dry sample after wash = (M2)		3167.1				
Total passing 0.075mm sieve = (M1-M2)		402.9		2.6	282.5	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	119.9	3.4	3.4	96.6	<b>100</b>	<b>100</b>
37.5	327.8	9.2	12.5	87.5	<b>80</b>	<b>100</b>
19	1202.1	33.7	46.2	53.8	<b>60</b>	<b>80</b>
9.5	896.9	25.1	71.3	28.7	<b>45</b>	<b>65</b>
4.75	300.6	8.4	79.8	20.2	<b>30</b>	<b>50</b>
2.36	93.2	2.6	82.4	17.6	<b>20</b>	<b>40</b>
0.425	106.9	3.0	85.4	14.6	<b>10</b>	<b>25</b>
0.075	119.7	3.4	88.7	11.3	<b>5</b>	<b>15</b>
Pan	402.9	11.3	100.0			

Table A-1.2 Result of sieve analysis test for sample C (before compaction)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		3190		GM	PP	
Mass dry sample after wash = (M2)		2750.6				
Total passing 0.075mm sieve = (M1-M2)		439.4		2.45	303.6	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	118.8	3.7	3.7	96.3	<b>80</b>	<b>100</b>
19	1100.3	34.5	38.2	61.8	<b>60</b>	<b>80</b>
9.5	897.3	28.1	66.3	33.7	<b>45</b>	<b>65</b>
4.75	260.1	8.2	74.5	25.5	<b>30</b>	<b>50</b>
2.36	87.0	2.7	77.2	22.8	<b>20</b>	<b>40</b>
0.425	123.2	3.9	81.1	18.9	<b>10</b>	<b>25</b>
0.075	163.9	5.1	86.2	13.8	<b>5</b>	<b>15</b>
Pan	439.4	13.8	100.0			

**APPENDIX B**

**LABORATORY TEST RESULTS FOR BLENDED LATERITE**

**APPENDIX B-1 SIEVE ANALYSIS TEST**

Table B-1.1 Result of sieve analysis test for laterite

I. before compaction (with respect to base course specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		3133		GM	PP	
Mass dry sample after wash = (M2)		2549.5				
Total passing 0.075mm sieve = (M1-M2)		583.5		3.2	368.7	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	305.1	9.7	9.7	90.3	<b>80</b>	<b>100</b>
19	689.8	22.0	31.8	68.2	<b>60</b>	<b>80</b>
9.5	806.3	25.7	57.5	42.5	<b>45</b>	<b>65</b>
4.75	470.4	15.0	72.5	27.5	<b>30</b>	<b>50</b>
2.36	139.2	4.4	76.9	23.1	<b>20</b>	<b>40</b>
0.425	90.0	2.9	79.8	20.2	<b>10</b>	<b>25</b>
0.075	51.7	1.7	81.5	18.5	<b>5</b>	<b>15</b>
Pan	583.5	18.6	100.1			

II. before compaction (with respect to sub base specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		3133		GM	PP	
Mass dry sample after wash = (M2)		2549.5				
Total passing 0.075mm sieve = (M1-M2)		583.5		3.2	368.7	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	305.1	9.7	9.7	90.3	<b>80</b>	<b>100</b>
19	689.9	22.0	31.8	68.2	<b>60</b>	<b>100</b>
4.75	1276.7	40.8	72.5	27.5	<b>30</b>	<b>100</b>
1.18	190.5	6.1	78.6	21.4	<b>17</b>	<b>75</b>
0.3	49.0	1.6	80.2	19.8	<b>9</b>	<b>50</b>
0.075	41.4	1.3	81.5	18.5	<b>5</b>	<b>25</b>
Pan	583.5	18.6	100			

III. after compaction (with respect to base course specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		4000		GM	PP	
Mass dry sample after wash = (M2)		3108.8				
Total passing 0.075mm sieve = (M1-M2)		891.2		3.6	445.6	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	0.0	0.0	0.0	100.0	<b>80</b>	<b>100</b>
19	44.5	1.1	1.1	98.9	<b>60</b>	<b>80</b>
9.5	398.2	10.0	11.1	88.9	<b>45</b>	<b>65</b>
4.75	898.9	22.5	33.5	66.5	<b>30</b>	<b>50</b>
2.36	661.2	16.5	50.1	49.9	<b>20</b>	<b>40</b>
0.425	696.6	17.4	67.5	32.5	<b>10</b>	<b>25</b>
0.075	405.0	10.1	77.6	22.4	<b>5</b>	<b>15</b>
Pan	895.6	22.4	100.0			

IV. after compaction (with respect to sub base specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		4000		GM	PP	
Mass dry sample after wash = (M2)		3104.4				
Total passing 0.075mm sieve = (M1-M2)		895.6		3.6	445.6	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	0.0	0.0	0.0	100.0	<b>80</b>	<b>100</b>
19	44.5	1.1	1.1	98.9	<b>60</b>	<b>100</b>
4.75	1297.1	32.4	33.5	66.5	<b>30</b>	<b>100</b>
1.18	1037.1	25.9	59.5	40.5	<b>17</b>	<b>75</b>
0.3	404.5	10.1	69.6	30.4	<b>9</b>	<b>50</b>
0.075	321.2	8.0	77.6	22.4	<b>5</b>	<b>25</b>
Pan	895.6	22.4	100			

**APPENDIX B-2 ATTERBERG LIMIT TEST**

Table B-2.1 Result of Atterberg limit test for laterite

No. of blows	Liquid limit			Plastic limit	
	32	26	20		
Wt. of cont. + wet soil (g)= W1	34.92	33.85	32.38	24.58	18.52
Wt. of cont. + dry soil (g) = W2	25.00	23.93	22.65	23.08	17.06
Wt. of container (g) = W3	5.14	4.82	4.62	18.42	12.53
Mass of moisture (g) (W1 - W2) = X	9.92	9.92	9.73	1.50	1.46
Wt. of dry soil (g) (W2 - W3) = Y	19.86	19.11	18.03	4.66	4.53
Moisture content (%) = (X/Y)*100	49.95	51.91	53.97	32.19	32.23
	52.1			32.21	
	<b>Plasticity index</b>			<b>19.9</b>	

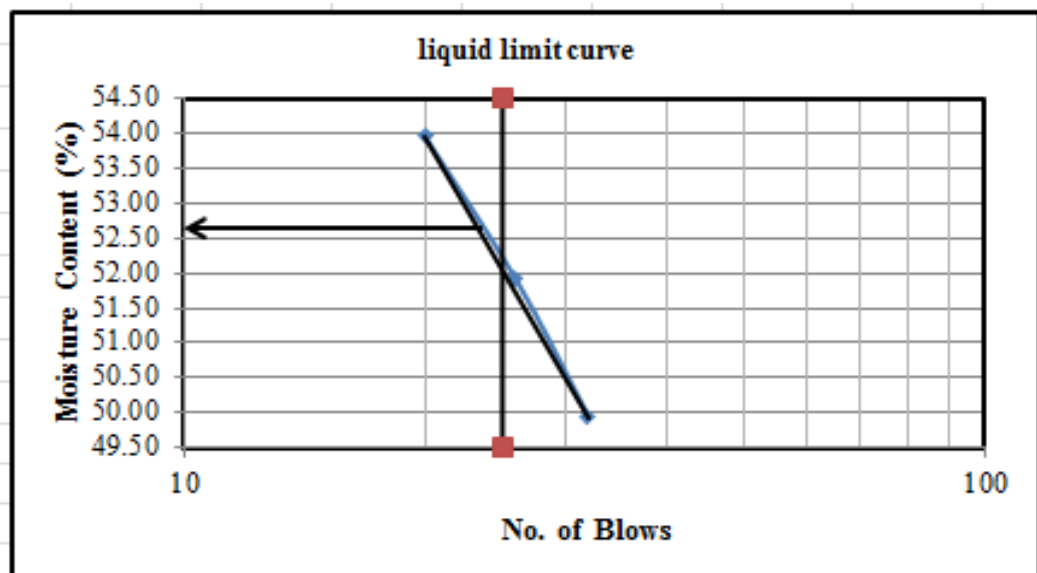


Figure B-2.1 liquid limit curve for laterite

**APPENDIX B-3 SPECIFIC GRAVITY AND WATER ABSORPTION TEST**

Table B-3.1 Result of Specific gravity and water absorption test for laterite

<b>Specific gravity and absorption of aggregate</b>		
<b>AASHTO T-84</b>		
	1	2
Trial No	1	2
Pycnometer no	P <sub>1</sub>	P <sub>2</sub>
Mass of saturated surface dry test sample in air, (A) (g)	500	500
Mass of pycnometer+soil+ water (M <sub>2</sub> ) (g)	1585	1596
Mass of pycnometer+ water (M <sub>1</sub> ) (g)	1280.5	1281.6
Mass of oven dry test sample in air (B) (g)	427.3	425.2
Absorption (%) = (A-B)/B	17.0	17.6
Average Absorption (%)	<b>17.3</b>	
Specific gravity of laterite at 23°C = A/(M <sub>1</sub> +A-M <sub>2</sub> )	2.6	2.7
Average specific gravity of laterite at 23°C	<b>2.6</b>	

**APPENDIX B-4 MODIFIED PROCTOR TEST**

Table B-4.1 Result of modified proctor test for laterite

Trial No.	1	2	3	4	5	
Weight of Mould + Wet soil (g)	4500	4630	4760	4840	4810	
Weight of Mould (g)	2930	2930	2930	2930	2930	
Weight of Wet soil (g)	1570	1700	1830	1910	1880	
Volume of Mould (cc)	929	929	929	929	929	
Wet density (g/cc)	1.69	1.83	1.97	2.06	2.02	
<b>Moisture Content Determination</b>						
Weight of Wet soil + cont. (g)	351.8	282.2	212.2	259.9	237.3	
Weight of Dry soil + cont. (g)	322.0	254.5	188.8	226.6	203.6	
Weight of Container (g)	34.9	38.6	35.2	37.6	34.8	
Weight of water (moisture) (g)	29.8	27.7	23.4	33.3	33.7	
Weight of Dry soil (g)	287.1	215.9	153.6	189.0	168.8	
Moisture content (%)	10.38	12.83	15.23	17.62	19.96	
Dry Density (g/cc)	1.53	1.62	1.71	1.75	1.69	
	<b>MDD, g/cc</b>		<b>1.75</b>	<b>OMC, %</b>		<b>17.62</b>

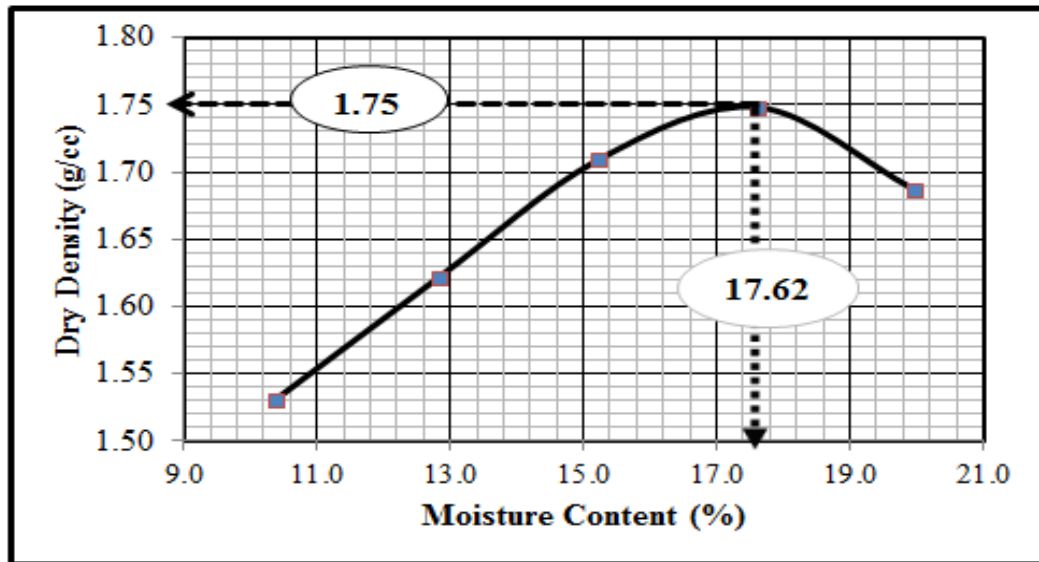


Figure B-4.1 compaction curve for laterite

**APPENDIX B-5 CALIFORNIA BEARING RATIO [CBR] TEST**

Table B-5.1 Result of CBR test for laterite

Unit Weight Determination													
No. of blows per layer		10 Blows		30 Blows		65 Blows							
		Before Soaking	After Soaking	Before Soaking	After Soaking	Before Soaking	After Soaking	Before Soaking	After Soaking				
Condition of Sample													
Wt. of wet sample + mold, g	W1	10615	10980	11235	11505	10420	10625						
Wt. of mould, g	W2	7500	7500	7240	7240	6080	6080						
Wt. of wet sample, g	W3= W1 -W2	3115	3480	3995	4265	4340	4545						
Volume of mold, cc	V	2123	2123	2123	2123	2123	2123						
Wet unit weight, g/cc	Dw = W3/V	1.47	1.64	1.88	2.01	2.04	2.14						
Dry unit weight, g/cc	Dw/(1+W8/100)	1.25	1.22	1.60	1.54	1.74	1.72						
Moisture Content Determination													
Wt. of wet sample + cont., g	W3	218.8	251.3	237.6	305.8	277.1	237.7						
Wt. of dry sample + cont., g	W4	191.2	195.5	207.3	242.2	241.2	197.6						
Wt. of water, g	W5= W3-W4	27.6	55.8	30.3	63.6	35.9	40.1						
Wt. of Container, g	W6	34.3	34.3	35.3	35.7	36.6	35.6						
Wt. of dry sampe, g	W7=W4-W6	156.9	161.2	172.0	206.5	204.6	162.0						
% Moisture Content	W8= (W5/W7) *100	17.6	34.6	17.6	30.8	17.5	24.8						
Swell Data (Surcharge Weight 9Kg)													
No. of Blows		10				30				65			
Initial Height of Sample: 116mm	Gauge reading		Swell		Gauge reading		Swell		Gauge reading		Swell		
	Initial	Final	mm	%	Initial	Final	mm	%	Initial	Final	mm	%	
	619	813	1.94	1.67	744	905	1.61	1.39	810	950	1.40	1.21	
CBR DATA													
Penetra-tion (mm)	Std. load (KN)	Gauge reading	Load KN	Corrected CBR		Gauge reading	Load KN	Corrected CBR		Gauge reading	Load KN	Corrected CBR	
0		0	0			0	0			0	0		
0.64		22	0.53			83	2			105	2.54		
1.27		43	1.04			99	2.39			122	2.95		
1.91		58	1.4			111	2.68			145	3.5		
2.54	13	75	1.81	1.81	<b>13.9</b>	124	2.99	2.99	<b>23.0</b>	168	4.06	4.06	<b>31.2</b>
5.08	20	83	2.00	2.00	<b>10.0</b>	145	3.5	3.50	<b>17.5</b>	180	4.35	4.35	<b>21.8</b>
6.35		95	2.29			166	4.01			200	4.83		
7.62		105	2.54			188	4.54			225	5.43		
Soaked CBR, %	13.9			23.0			31.2						
Dry Density, g/cc	1.25			1.60			1.74						
Swell, %	1.67			1.39			1.21						
Density Requirement	98%			Target Density			1.71			CBR			28.64
Density Requirement	95%			Target Density			1.66			CBR			18.8

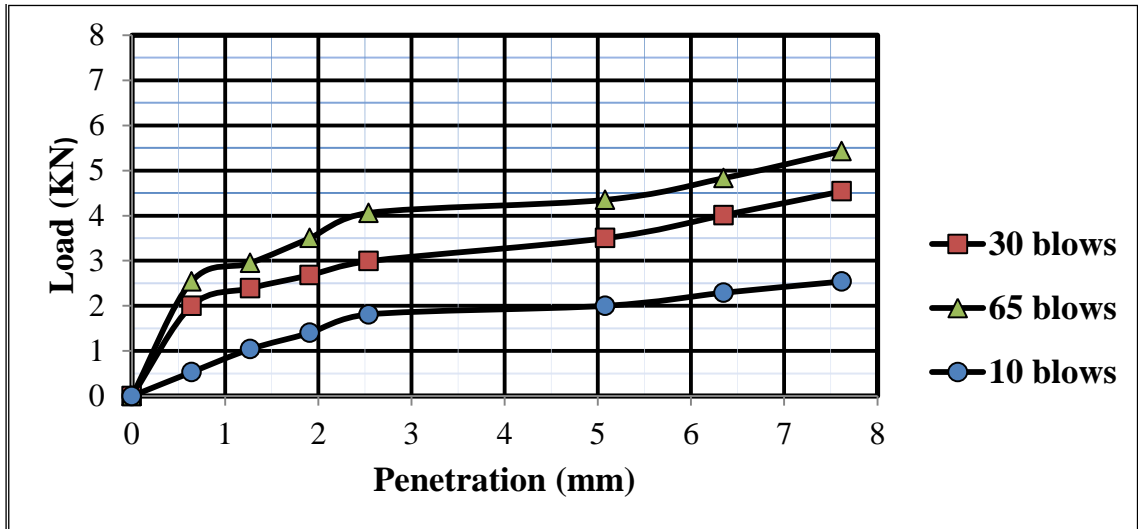


Figure B-5.1 Load penetration curve for laterite

**APPENDIX B-6 LOS ANGELES ABRASION TEST**

Table B-6.1 Result of Los angeles abrasion test for laterite

Los Angeles Abrasion Test									
AASHTO T-96									
Sieve Sizes	1 1/2 - 1"	1 - 3/4"	3/4 - 1/2"	1/2 - 3/8"	3/8 - 1/4"	1/4" - No. 4	No.4 - No.3		
Grade	A				C		D		
Number of Balls	12 Balls				8 Balls		6 Balls		
Wt. of Indicated Size	1250 ± 25	1250 ± 25	1250 ± 10	1250 ± 10	2500 ± 10	2500 ± 10	5000 ± 10		
Wt. of Tested Sample									
Grade	B			C					
Number of Balls								12 Balls	
Wt. of Indicated Size								2500 ± 10	2500 ± 10
Wt. of Tested Sample									
Test Results									
Trial					1	2			
Number of Revolution					500	500			
Total Wt. of Sample Tested (g)					5006.8	4995.4			
Wt. of tested Sample Retained On No. 12 Sieve (g)					1490.8	1481.3			
Percent loss (%)					70.22	70.35			
Average of Percent loss (%)					<b>70.3</b>				

**APPENDIX B-7 TEN PERCENT FINES VALUE TEST**

Table B-7.1 Result of Ten percent fines value test for laterite

<b>TEN PER CENT FINES VALUE (TFV)</b>		
<b>BS 812 Part 111: 1990</b>		
Condition of Aggregate Tested	Soaked	
Trials	1	2
Mass of aggregate passing 14mm and retained on 10mm (Before Compression) = (M1)	2635.20	2640.50
Mass of aggregate passing 2.36mm (After Compression) = (M2)	289.80	286.40
% of material passing 2.36mm = (M2/M1)*100	11.00	10.85
Duration of testing, min	10.00	10.02
Maximum load , f (KN)	17.90	18.90
Force to required to produce 10% fines in KN = $14*f/(m+4)$	16.71	17.82
Average Force Ttfv in KN	<b>17.3</b>	
<b>Remarks :</b> Per cent passing 2.36mm should be between 7.5 % and 12 %		
$F = (14+f)/(M+4)$		
Penetration of Plunger : 15 mm for rounded or partialy rounded aggregate		
20mm for normally crushed aggregate		
24mm for vesicular (honeycombed) aggregates		

APPENDIX B-8 GEOCHEMICAL TEST

Table B-8.1 Result of Geochemical test for laterite

Geological Survey of Ethiopia:Geochemical Laboratory Directorate  
 Geochemical Laboratory Complete Silicate Analysis Report Format Form G0004  
 FILE ID ;4964/16 pvt Originator; Haftom Hagos  
 Sample type; Rock Date Submitted; 05/18/2016

Preparation :-200 MESH

Number of Sample: 2


Element to be determined Major Oxides & Minor Oxides

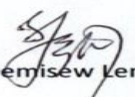
Analytical Method: LIBO2 FUSION , HFattack,GRAVIMETERIC,COLORIMETRIC and AAS


Analytical Results in PERCENT

FIELD NO	Lab No	SiO <sub>2</sub>	Al <sub>2</sub> O <sub>3</sub>	Fe <sub>2</sub> O <sub>3</sub>	CaO	MgO	Na <sub>2</sub> O	K <sub>2</sub> O	MnO	P <sub>2</sub> O <sub>5</sub>	TiO <sub>2</sub>	H <sub>2</sub> O	LOI
sample -A	4964/16	41.42	14.97	27.28	0.40	0.64	0.32	0.34	<0.01	0.43	1.48	2.36	9.21
sample-B	4965	19.72	20.09	34.92	0.44	0.68	1.74	1.74	0.15	1.56	2.09	4.42	13.18

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DATE REPORTED  
 06-02-2016



**APPENDIX C**
**QUALITY TEST RESULTS FOR CRUSHED STONE AGGREGATE [CSA]**
**APPENDIX C-1 SIEVE ANALYSIS TEST**

Table C-1.1 Result of sieve analysis test for CSA

PARTICLE ANALYSIS DISTRIBUTION						
TEST METHOD: AASHTO T-27						
Mass dry sample before wash = (M1)		2550				
Mass dry sample after wash = (M2)		2320.5				
Total passing 0.075mm sieve = (M1-M2)		229.5				
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	0.0	0.0	0.0	100.0	<b>95</b>	<b>100</b>
19	535.5	21.0	21.0	79.0	<b>60</b>	<b>80</b>
9.5	739.5	29.0	50.0	50.0	<b>40</b>	<b>60</b>
4.75	535.5	21.0	71.0	29.0	<b>25</b>	<b>40</b>
2.36	229.5	9.0	80.0	20.0	<b>15</b>	<b>30</b>
0.425	229.5	9.0	89.0	11.0	<b>7</b>	<b>19</b>
0.075	51.0	2.0	91.0	9.0	<b>5</b>	<b>12</b>
Pan	229.5	9.0	100.0			

**APPENDIX C-2 ATTERBERG LIMIT TEST**

Table C-2.1 Result of Atterberg limit test for CSA

No. of blows	Liquid limit			Plastic limit	
	33	26	20		
Wt. of cont. + wet soil (g)= W1	30.50	32.30	35.10	24.90	18.40
Wt. of cont. + dry soil (g) = W2	26.80	28.90	30.20	23.80	17.40
Wt. of container (g) = W3	11.90	15.90	12.40	18.80	12.80
Mass of moisture (g) (W1 - W2) = X	3.70	3.40	4.90	1.10	1.00
Wt. of dry soil (g) (W2 -W3) = Y	14.90	13.00	17.80	5.00	4.60
Moisture content (%) = (X/Y)*100	24.83	26.15	27.53	22.00	21.74
	26.3			21.9	
	<b>Plasticity index</b>			<b>4.4</b>	

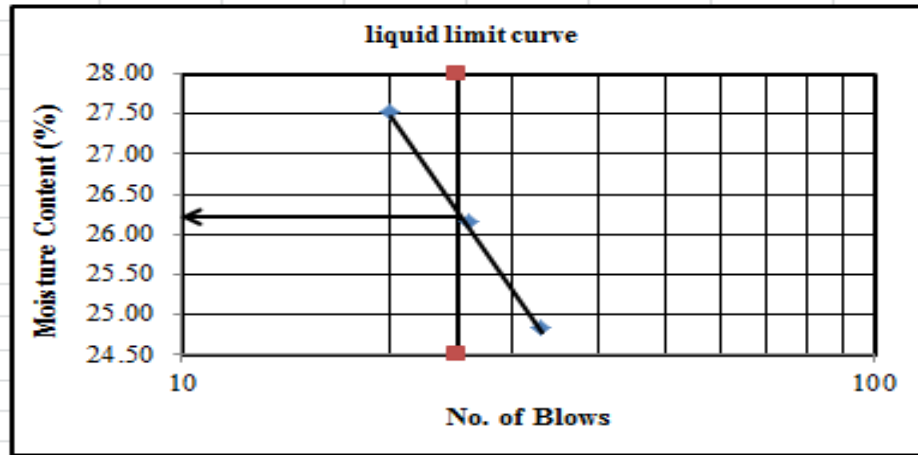


Figure C-2.1 liquid limit curve for CSA

**APPENDIX C-3 MODIFIED PROCTOR TEST**

Table C-3.1 Result of modified proctor test for CSA

Trial No.	1	2	3	4	5	
Weight of Mould + Wet soil (g)	4900	4970	5110	5200	5200	
Weight of Mould (g)	2930	2930	2930	2930	2930	
Weight of Wet soil (g)	1970	2040	2180	2270	2270	
Volume of Mould (cc)	929	929	929	929	929	
Wet density (g/cc)	2.12	2.20	2.35	2.44	2.44	
<b>Moisture Content Determination</b>						
Weight of Wet soil + cont. (g)	287.4	263.3	210.8	211.2	194.2	
Weight of Dry soil + cont. (g)	284.5	258.8	203.8	200.2	180.8	
Weight of Container (g)	35.7	34.1	36.2	34.3	32.3	
Weight of water (moisture) (g)	2.9	4.5	7.0	11.0	13.4	
Weight of Dry soil (g)	248.8	224.7	167.6	165.9	148.5	
Moisture content (%)	1.17	2.00	4.18	6.63	9.02	
Dry Density (g/cc)	2.10	2.15	2.25	2.29	2.24	
	<b>MDD, g/cc</b>		<b>2.29</b>	<b>OMC, %</b>		<b>6.63</b>

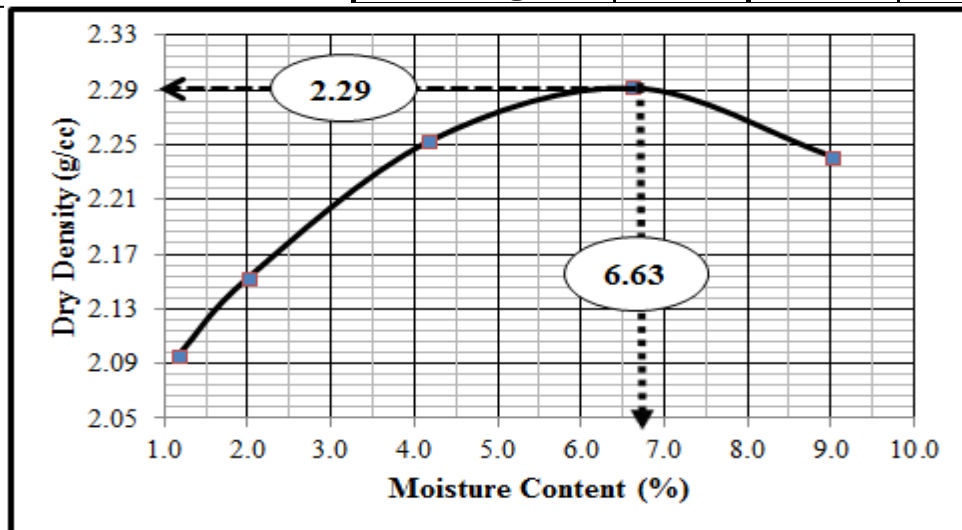


Figure C-3.1 compaction curve for CSA

**APPENDIX C-4 CALIFORNIA BEARING RATIO [CBR] TEST**

Table C-4.1 Result of CBR test for CSA

Unit Weight Determination													
No. of blows per layer		10 Blows				30 Blows				65 Blows			
Condition of Sample		Before Soaking	After Soaking	Before Soaking	After Soaking	Before Soaking	After Soaking	Before Soaking	After Soaking	Before Soaking	After Soaking	Before Soaking	After Soaking
Wt. of wet sample + mold, g	W1	10620	10750	10850	10990	11200	11310						
Wt. of mould, g	W2	6300	6300	5950	5950	5980	5980						
Wt. of wet sample, g	W3= W1 -W2	4320	4450	4900	5040	5220	5330						
Volume of mold, cc	V	2123	2123	2123	2123	2123	2123						
Wet unit weight, g/cc	Dw = W3/V	2.03	2.10	2.31	2.37	2.46	2.51						
Dry unit weight, g/cc	Dw/(1+W8/100)	1.92	1.86	2.17	2.15	2.31	2.32						
Moisture Content Determination													
Wt. of wet sample + cont., g	W3	205.5	267.6	277.5	288.5	294.2	213.3						
Wt. of dry sample + cont., g	W4	195.6	240.8	263.3	264.6	278.5	199.9						
Wt. of water, g	W5= W3-W4	9.9	26.8	14.2	23.9	15.7	13.4						
Wt. of Container, g	W6	37.3	33.3	35.5	37.2	31.8	37.2						
Wt. of dry sampe, g	W7=W4-W6	158.3	207.5	227.8	227.4	246.7	162.7						
% Moisture Content	W8= (W5/W7) *100	6.3	12.9	6.2	10.5	6.4	8.2						
Swell Data (Surcharge Weight 9Kg)													
No. of Blows	10				30				65				
Initial Height of Sample: 116mm	Gauge reading		Swell		Gauge reading		Swell		Gauge reading		Swell		
	Initial	Final	mm	%	Initial	Final	mm	%	Initial	Final	mm	%	
	755	820	0.65	0.56	1313	1350	0.37	0.31897	1210	1235	0.25	0.22	
CBR DATA													
Penetration(mm)	Std. load (KN)	Gauge reading	Load KN	Corrected CBR KN	%	Gauge reading	Load KN	Corrected CBR KN	%	Gauge reading	Load KN	Corrected CBR KN	%
0		0	0			0	0			0	0		
0.64		120	2.60			200	4.34			420	9.11		
1.27		200	4.34			420	9.11			620	13.45		
1.91		300	6.51			635	13.78			780	16.93		
2.54	13	400	8.68	8.68	<b>66.8</b>	780	16.93	16.93	<b>130.2</b>	925	20.07	20.0725	<b>154.4</b>
5.08	20	620	13.45	13.45	<b>67.3</b>	1010	21.92	21.92	<b>109.6</b>	1190	25.82	25.823	<b>129.1</b>
6.35		700	15.19			1170	25.39			1315	28.54		
7.62		800	17.36			1280	27.78			1455	31.57		
Soaked CBR, %	67.3				130.2				154.4				
Dry Density, g/cc	1.92				2.17				2.31				
Swell, %	0.56				0.32				0.22				
Density Requirement	98%				Target Density				2.24 CBR 142.66				

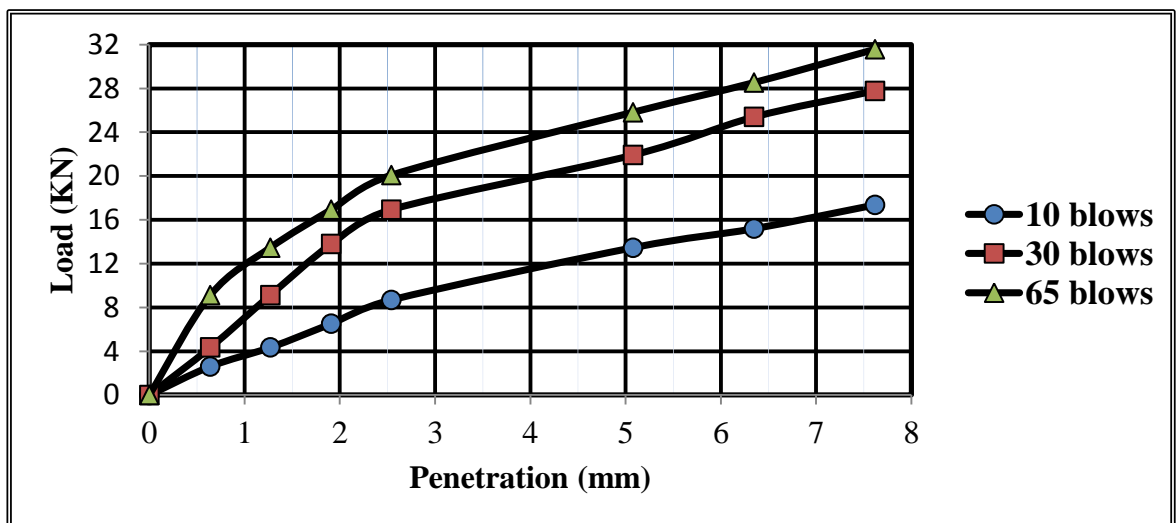


Figure C-4.1 Load penetration curve for CSA

**APPENDIX C-5 AGGREGATE CRUSHING VALUE TEST**

Table C-5.1 Result of Aggregate crushing value test for CSA

<b>Aggregate Crushing Value Test</b>		
Test No.	1	2
Mass of aggregate before test, passing 14.0mm and retainn 10.0mm sieves M1 (g)	3020	3015
Mass of aggregate before test, passing 2.36mm sieves M2 (g)	554	549
ACV (%) = (M2/M1)*100	18	18
Average ACV (%) = (Test 1 + Test 2)	18	

**APPENDIX C-6 TEN PERCENT FINES VALUE TEST**

Table C-6.1 Result of Ten per cent fines value test for CSA

<b>TEN PER CENT FINES VALUE (TFV)</b>				
<b>BS 812 Part 111: 1990</b>				
Condition of Aggregate Tested	Un soaked		Soaked	
Trials	1	2	1	2
Mass of aggregate passing 14mm and retained on 10mm (Before Compression) = (M1)	2980.0	2970.0	3015.0	3018.0
Mass of aggregate passing 2.36mm (After Compression) = (M2)	322.0	238.4	331.0	262.0
% of material passing 2.36mm = (M2/M1)*100	10.8	8.0	11.0	8.7
Duration of testing, min	10.0	10.1	9.9	10.0
Maximum load , f (KN)	228.0	222.0	205.0	189.0
Force to required to produce 10% fines in KN = 14*f/(m+4)	215.6	258.4	191.6	208.7
Average Force Ttfv in KN	237.0		200.1	
Ratio of Soaked and Un soaked (%)	84.4			
<b>Remarks :</b> Per cent passing 2.36mm should be between 7.5 % and 12 %				
$F = (14+f)/(M+4)$				
Penetration of Plunger : 15 mm for rounded or partialy rounded aggregate				
20mm for normally crushed aggregate				
24mm for vesicular (honeycombed) aggregates				

**APPENDIX C-7 FLAKINESS INDEX TEST**

Table C-7.1 Result of Flakiness index test for CSA

Fraction prepared for the test		Weight of fraction consisting of at least 200 pieces W1(g)	Thickness gauge (mm)	Weight of aggregates passing thickness gauge. W2 (g)
passing through BS sieve	Retained on BS sieve			
60	50	1500	33.9	254
50	37.5	1200	26.3	189
37.5	28	800	19.7	169
28	20	758	14.4	123
20	14	451	10.2	85
14	10	364	7.2	81
10	6.3	275	4.9	67
Total		5348	968	
Flakiness index (%) = $(w2/w1)*100$			<b>18.10</b>	

**APPENDIX D**

**LABORATORY TEST RESULTS FOR MECHANICAL STABILIZATION**

**APPENDIX D-1 SIEVE ANALYSIS TEST**

Table D-1.1 Result of sieve analysis test for laterite with 20% CSA

I. before compaction (with respect to base course specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		3020		GM	PP	
Mass dry sample after wash = (M2)		2504.4				
Total passing 0.075mm sieve = (M1-M2)		515.6		3.2	287.3	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	244.1	8.1	8.1	91.9	<b>80</b>	<b>100</b>
19	656.9	21.8	29.8	70.2	<b>60</b>	<b>80</b>
9.5	795.4	26.3	56.2	43.8	<b>45</b>	<b>65</b>
4.75	481.3	15.9	72.1	27.9	<b>30</b>	<b>50</b>
2.36	157.0	5.2	77.3	22.7	<b>20</b>	<b>40</b>
0.425	121.5	4.0	81.3	18.7	<b>10</b>	<b>25</b>
0.075	48.2	1.6	82.9	17.1	<b>5</b>	<b>15</b>
Pan	515.6	17.1	100			

II. before compaction (with respect to sub base specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		3020		GM	PP	
Mass dry sample after wash = (M2)		2504.4				
Total passing 0.075mm sieve = (M1-M2)		515.6		3.2	287.3	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	244.1	8.1	8.1	91.9	<b>80</b>	<b>100</b>
19	656.9	21.8	29.8	70.2	<b>60</b>	<b>100</b>
4.75	1276.7	42.3	72.1	27.9	<b>30</b>	<b>100</b>
1.18	231.0	7.7	79.8	20.2	<b>17</b>	<b>75</b>
0.3	57.5	1.9	81.7	18.3	<b>9</b>	<b>50</b>
0.075	38.2	1.3	82.9	17.1	<b>5</b>	<b>25</b>
Pan	515.6	17.1	100			

III. after compaction (with respect to base course specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		3720		GM	PP	
Mass dry sample after wash = (M2)		2948.3				
Total passing 0.075mm sieve = (M1-M2)		771.7		3.5	347.8	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	0.0	0.0	0.0	100.0	<b>80</b>	<b>100</b>
19	143.1	3.8	3.8	96.2	<b>60</b>	<b>80</b>
9.5	468.9	12.6	16.5	83.5	<b>45</b>	<b>65</b>
4.75	824.1	22.2	38.6	61.4	<b>30</b>	<b>50</b>
2.36	574.6	15.4	54.1	45.9	<b>20</b>	<b>40</b>
0.425	606.8	16.3	70.4	29.6	<b>10</b>	<b>25</b>
0.075	330.8	8.9	79.3	20.7	<b>5</b>	<b>15</b>
Pan	771.7	20.7	100			

IV. after compaction (with respect to sub base specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		3720		GM	PP	
Mass dry sample after wash = (M2)		2948.3				
Total passing 0.075mm sieve = (M1-M2)		771.7		3.5	347.8	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	0.0	0.0	0.0	100.0	<b>80</b>	<b>100</b>
19	143.1	3.8	3.8	96.2	<b>60</b>	<b>100</b>
4.75	1293.0	34.8	38.6	61.4	<b>30</b>	<b>100</b>
1.18	908.3	24.4	63.0	37.0	<b>17</b>	<b>75</b>
0.3	341.9	9.2	72.2	27.8	<b>9</b>	<b>50</b>
0.075	262.1	7.0	79.3	20.7	<b>5</b>	<b>25</b>
Pan	771.7	20.7	100			

Table D-1.2 Result of sieve analysis test for laterite with 40% CSA

I. before compaction (with respect to base course specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		2905		GM	PP	
Mass dry sample after wash = (M2)		2459.2				
Total passing 0.075mm sieve = (M1-M2)		445.8		3.2	198.9	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	183.1	6.3	6.3	93.7	<b>80</b>	<b>100</b>
19	627.0	21.6	27.9	72.1	<b>60</b>	<b>80</b>
9.5	784.4	27.0	54.9	45.1	<b>45</b>	<b>65</b>
4.75	492.2	16.9	71.8	28.2	<b>30</b>	<b>50</b>
2.36	174.8	6.0	77.8	22.2	<b>20</b>	<b>40</b>
0.425	153.1	5.3	83.1	16.9	<b>10</b>	<b>25</b>
0.075	44.6	1.5	84.7	15.3	<b>5</b>	<b>15</b>
Pan	445.8	15.3	100			

II. before compaction (with respect to sub base specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		2905		GM	PP	
Mass dry sample after wash = (M2)		2459.2				
Total passing 0.075mm sieve = (M1-M2)		445.8		3.2	198.9	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	183.1	6.3	6.3	93.7	<b>80</b>	<b>100</b>
19	627.0	21.6	27.9	72.1	<b>60</b>	<b>100</b>
4.75	1276.7	43.9	71.8	28.2	<b>30</b>	<b>100</b>
1.18	271.5	9.3	81.2	18.8	<b>17</b>	<b>75</b>
0.3	66.0	2.3	83.5	16.5	<b>9</b>	<b>50</b>
0.075	35.1	1.2	84.7	15.3	<b>5</b>	<b>25</b>
Pan	445.8	15.3	100			

III. after compaction (with respect to base course specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		3425		GM	PP	
Mass dry sample after wash = (M2)		2792.2				
Total passing 0.075mm sieve = (M1-M2)		632.8		3.5	240.5	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	0.0	0.0	0.0	100.0	<b>80</b>	<b>100</b>
19	241.7	7.1	7.1	92.9	<b>60</b>	<b>80</b>
9.5	539.6	15.8	22.8	77.2	<b>45</b>	<b>65</b>
4.75	749.3	21.9	44.7	55.3	<b>30</b>	<b>50</b>
2.36	488.0	14.2	58.9	41.1	<b>20</b>	<b>40</b>
0.425	517.0	15.1	74.0	26.0	<b>10</b>	<b>25</b>
0.075	256.6	7.5	81.5	18.5	<b>5</b>	<b>15</b>
Pan	632.8	18.5	100			

IV. after compaction (with respect to sub base specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		3425		GM	PP	
Mass dry sample after wash = (M2)		2792.2				
Total passing 0.075mm sieve = (M1-M2)		632.8		3.5	240.5	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	0.0	0.0	0.0	100.0	<b>80</b>	<b>100</b>
19	241.7	7.1	7.1	92.9	<b>60</b>	<b>100</b>
4.75	1288.9	37.6	44.7	55.3	<b>30</b>	<b>100</b>
1.18	779.4	22.8	67.4	32.6	<b>17</b>	<b>75</b>
0.3	279.3	8.2	75.6	24.4	<b>9</b>	<b>50</b>
0.075	203.0	5.9	81.5	18.5	<b>5</b>	<b>25</b>
Pan	632.8	18.5	100			

Table D-1.3 Result of sieve analysis test for laterite with 60% CSA

I. before compaction (with respect to base course specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		3265		GM	PP	
Mass dry sample after wash = (M2)		2810.7				
Total passing 0.075mm sieve = (M1-M2)		454.3		3.3	154.3	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	169.8	5.2	5.2	94.8	<b>80</b>	<b>100</b>
19	545.0	16.7	21.9	78.1	<b>60</b>	<b>80</b>
9.5	770.1	23.6	45.5	54.5	<b>45</b>	<b>65</b>
4.75	689.2	21.1	66.6	33.4	<b>30</b>	<b>50</b>
2.36	292.0	8.9	75.5	24.5	<b>20</b>	<b>40</b>
0.425	218.3	6.7	82.2	17.8	<b>10</b>	<b>25</b>
0.075	126.3	3.9	86.1	13.9	<b>5</b>	<b>15</b>
Pan	454.3	13.9	100			

II. before compaction (with respect to sub base specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		3265		GM	PP	
Mass dry sample after wash = (M2)		2810.7				
Total passing 0.075mm sieve = (M1-M2)		454.3		3.3	154.3	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	169.8	5.2	5.2	94.8	<b>80</b>	<b>100</b>
19	545.0	16.7	21.9	78.1	<b>60</b>	<b>100</b>
4.75	1459.3	44.7	66.6	33.4	<b>30</b>	<b>100</b>
1.18	417.9	12.8	79.4	20.6	<b>17</b>	<b>75</b>
0.3	118.2	3.6	83.0	17.0	<b>9</b>	<b>50</b>
0.075	100.5	3.1	86.1	13.9	<b>5</b>	<b>25</b>
Pan	454.3	13.9	100			

III. after compaction (with respect to base course specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		3280		GM	PP	
Mass dry sample after wash = (M2)		2810.7				
Total passing 0.075mm sieve = (M1-M2)		469.3		3.4	158.7	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	119.8	3.7	3.7	96.3	<b>80</b>	<b>100</b>
19	495.0	15.1	18.7	81.3	<b>60</b>	<b>80</b>
9.5	640.1	19.5	38.3	61.7	<b>45</b>	<b>65</b>
4.75	619.2	18.9	57.1	42.9	<b>30</b>	<b>50</b>
2.36	242.0	7.4	64.5	35.5	<b>20</b>	<b>40</b>
0.425	418.3	12.8	77.3	22.7	<b>10</b>	<b>25</b>
0.075	276.3	8.4	85.7	14.3	<b>5</b>	<b>15</b>
Pan	469.3	14.3	100			

IV. after compaction (with respect to sub base specification)

<b>PARTICLE ANALYSIS DISTRIBUTION</b>						
<b>TEST METHOD: AASHTO T-27</b>						
Mass dry sample before wash = (M1)		3280		GM	PP	
Mass dry sample after wash = (M2)		2810.7				
Total passing 0.075mm sieve = (M1-M2)		469.3		3.4	158.7	
Sieve Size	Weight retained	% retained	Commulative retained	% passing	Specification	
					lower	upper
50	0.0	0.0	0.0	100.0	<b>100</b>	<b>100</b>
37.5	119.8	3.7	3.7	96.3	<b>80</b>	<b>100</b>
19	495.0	15.1	18.7	81.3	<b>60</b>	<b>100</b>
4.75	1259.3	38.4	57.1	42.9	<b>30</b>	<b>100</b>
1.18	467.9	14.3	71.4	28.6	<b>17</b>	<b>75</b>
0.3	268.2	8.2	79.6	20.4	<b>9</b>	<b>50</b>
0.075	200.5	6.1	85.7	14.3	<b>5</b>	<b>25</b>
Pan	469.3	14.3	100			

**APPENDIX D-2 ATTERBERG LIMIT TEST**

Table D-2.1 Result of Atterberg limit test for laterite with 20% CSA

No. of blows	Liquid limit			Plastic limit	
	34	27	20		
Wt. of cont. + wet soil (g)= W1	33.74	36.92	38.27	24.75	12.69
Wt. of cont. + dry soil (g) = W2	25.19	29.42	27.87	23.39	11.04
Wt. of container (g) = W3	4.65	11.98	4.53	18.26	4.82
Mass of moisture (g) (W1 - W2) = X	8.55	7.50	10.40	1.36	1.65
Wt. of dry soil (g) (W2 - W3) = Y	20.54	17.44	23.34	5.13	6.22
Moisture content (%) = (X/Y)*100	41.63	43.00	44.56	26.51	26.53
	43.3			26.5	
	<b>Plasticity index</b>			<b>16.8</b>	

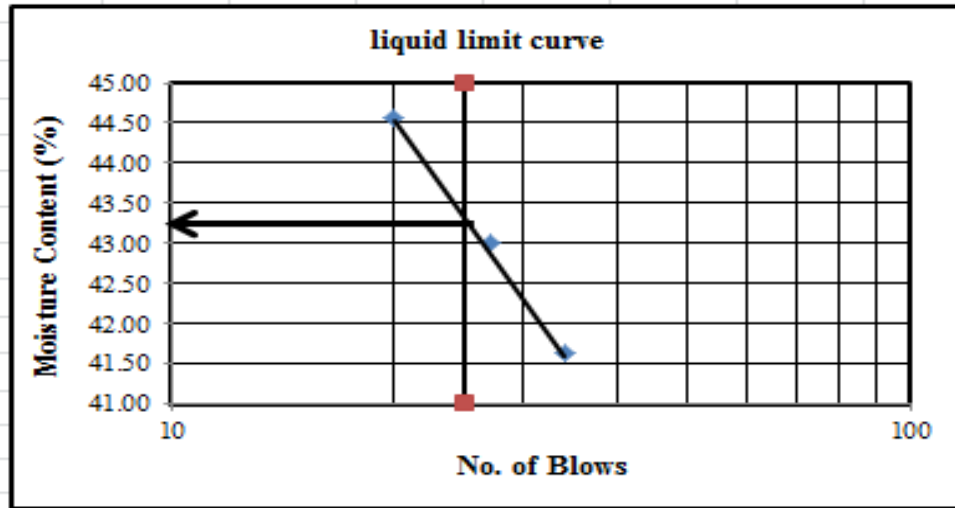


Figure D-2.1 liquid limit curve for laterite with 20% CSA

Table D-2.2 Result of Atterberg limit test for laterite with 40% CSA

No. of blows	Liquid limit			Plastic limit	
	34	27	20		
Wt. of cont. + wet soil (g)= W1	39.25	41.26	46.68	18.94	26.58
Wt. of cont. + dry soil (g) = W2	30.56	33.83	37.52	17.79	25.68
Wt. of container (g) = W3	4.61	12.47	12.24	12.56	21.62
Mass of moisture (g) (W1 - W2) = X	8.69	7.43	9.16	1.15	0.90
Wt. of dry soil (g) (W2 - W3) = Y	25.95	21.36	25.28	5.23	4.06
Moisture content (%) = (X/Y)*100	33.49	34.78	36.23	21.99	22.17
	35.1			22.1	
	<b>Plasticity index</b>			<b>13.0</b>	

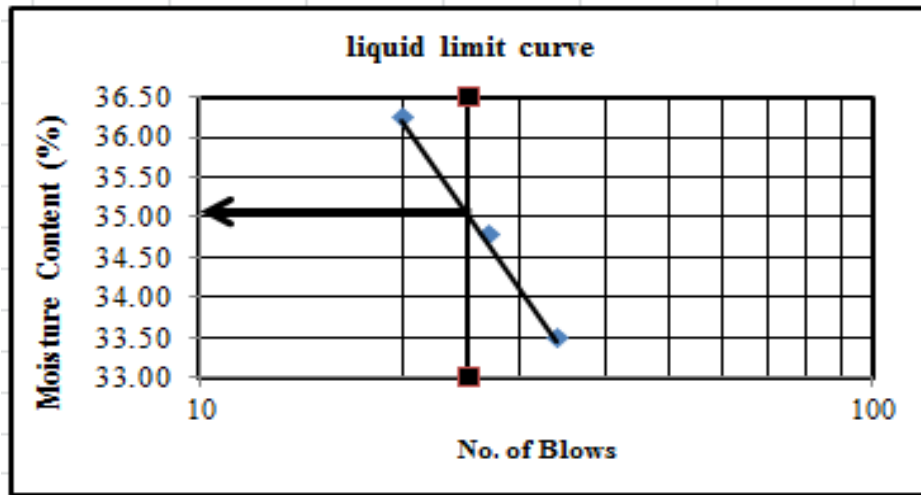


Figure D-2.2 liquid limit curve for laterite with 40% CSA

Table D-2.3 Result of Atterberg limit test for laterite with 60% CSA

No. of blows	Liquid limit			Plastic limit	
	32	27	23		
Wt. of cont. + wet soil (g)= W1	38.27	35.00	37.94	26.37	18.69
Wt. of cont. + dry soil (g) = W2	30.38	27.64	29.63	25.32	17.48
Wt. of container (g) = W3	4.89	4.85	5.00	20.52	11.96
Mass of moisture (g) (W1 - W2) = X	7.89	7.36	8.31	1.05	1.21
Wt. of dry soil (g) (W2 - W3) = Y	25.49	22.79	24.63	4.80	5.52
Moisture content (%) = (X/Y)*100	30.95	32.29	33.74	21.88	21.92
	33			21.9	
	<b>Plasticity index</b>			<b>11.1</b>	

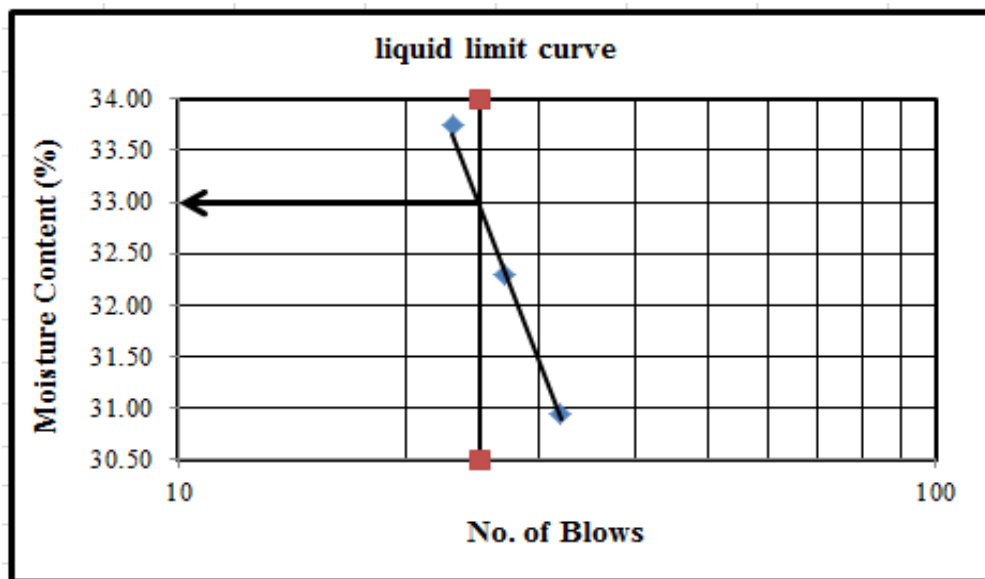


Figure D-2.3 liquid limit curve for laterite with 60% CSA

**APPENDIX D-3 MODIFIED PROCTOR TEST**

Table D-3.1 Result of modified proctor test for laterite with 20% CSA

Trial No.	1	2	3	4	5	
Weight of Mould + Wet soil (g)	4570	4720	4850	4965	4950	
Weight of Mould (g)	2930	2930	2930	2930	2930	
Weight of Wet soil (g)	1640	1790	1920	2035	2020	
Volume of Mould (cc)	929	929	929	929	929	
Wet density (g/cc)	1.77	1.93	2.07	2.19	2.17	
<b>Moisture Content Determination</b>						
Weight of Wet soil + cont. (g)	219.9	237.3	241.5	247.9	258.2	
Weight of Dry soil + cont. (g)	205.4	217.5	217.6	218.6	224.4	
Weight of Container (g)	33.3	34.3	36.6	39.6	37.2	
Weight of water (moisture) (g)	14.5	19.8	23.9	29.3	33.8	
Weight of Dry soil (g)	172.1	183.2	181.0	179.0	187.2	
Moisture content (%)	8.43	10.81	13.20	16.37	18.06	
Dry Density (g/cc)	1.63	1.74	1.83	1.88	1.84	
	<b>MDD, g/cc</b>		<b>1.88</b>	<b>OMC, %</b>		<b>16.37</b>

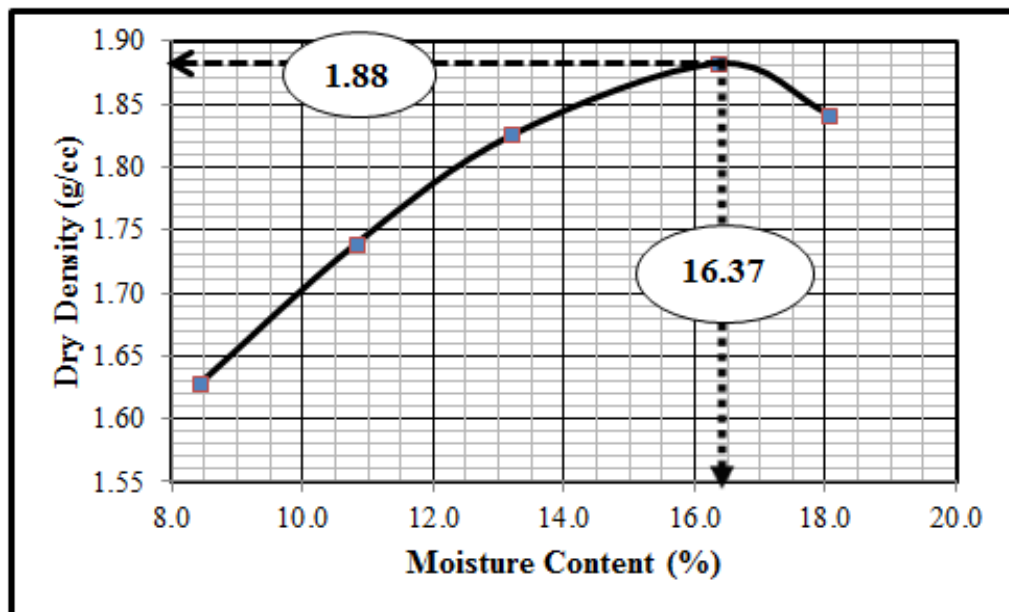


Figure D-3.1 compaction curve for laterite with 20% CSA

Table D-3.2 Result of modified proctor test for laterite with 40% CSA

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4640	4790	4935	5025	4980
Weight of Mould (g)	2930	2930	2930	2930	2930
Weight of Wet soil (g)	1710	1860	2005	2095	2050
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.84	2.00	2.16	2.26	2.21
<b>Moisture Content Determination</b>					
Weight of Wet soil + cont. (g)	213.3	264.6	251.5	258.8	243.3
Weight of Dry soil + cont. (g)	202.8	246.6	229.8	232.5	215.0
Weight of Container (g)	31.3	35.3	34.2	37.3	36.8
Weight of water (moisture) (g)	10.5	18.0	21.7	26.3	28.3
Weight of Dry soil (g)	171.5	211.3	195.6	195.2	178.2
Moisture content (%)	6.12	8.52	11.09	13.47	15.88
Dry Density (g/cc)	1.73	1.84	1.94	1.99	1.90
	<b>MDD, g/cc</b>		<b>1.99</b>	<b>OMC, %</b>	<b>13.5</b>

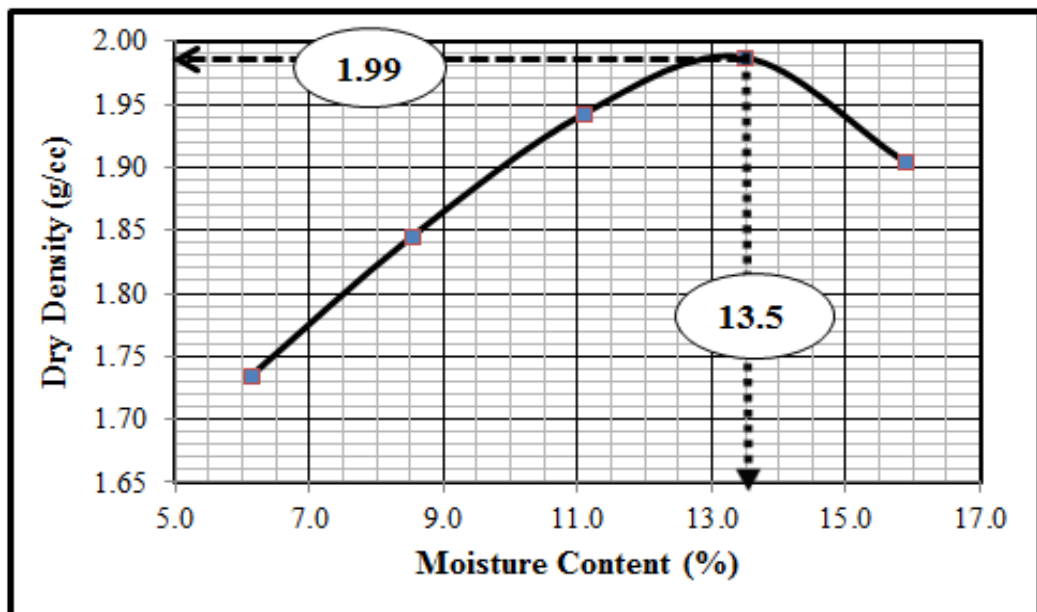


Figure D-3.2 compaction curve for laterite with 40% CSA

Table D-3.3 Result of modified proctor test for laterite with 60% CSA

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4660	4840	5000	5100	5040
Weight of Mould (g)	2930	2930	2930	2930	2930
Weight of Wet soil (g)	1730	1910	2070	2170	2110
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.86	2.06	2.23	2.34	2.27
<b>Moisture Content Determination</b>					
Weight of Wet soil + cont. (g)	245.3	197.5	315.7	261.2	279.6
Weight of Dry soil + cont. (g)	235.2	186.2	292.2	237.8	248.8
Weight of Container (g)	35.7	28.4	40.2	34.1	34.5
Weight of water (moisture) (g)	10.1	11.3	23.5	23.4	30.8
Weight of Dry soil (g)	199.5	157.8	252.0	203.7	214.3
Moisture content (%)	5.06	7.16	9.33	11.49	14.37
Dry Density (g/cc)	1.77	1.92	2.04	2.10	1.99
	<b>MDD, g/cc</b>		<b>2.10</b>	<b>OMC, %</b>	<b>11.5</b>

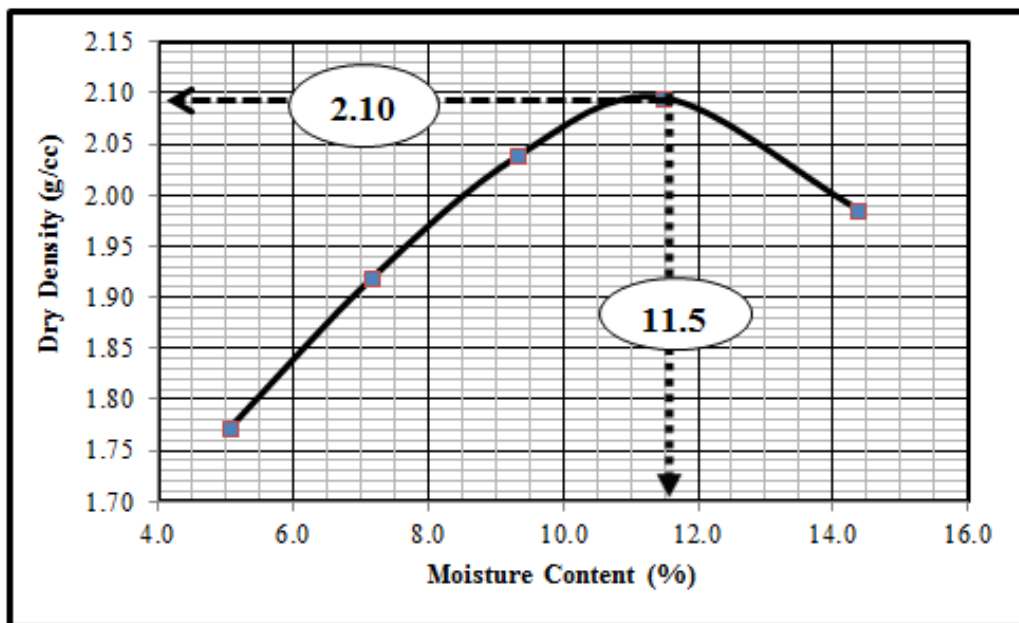


Figure D-3.3 compaction curve for laterite with 60% CSA

**APPENDIX D-4 CALIFORNIA BEARING RATIO [CBR] TEST**

Table D-4.1 Result of CBR test for laterite with 20% CSA [Soaked condition]

Unit Weight Determination													
No. of blows per layer		10 Blows		30 Blows		65 Blows							
Condition of Sample		Before Soaking	After Soaking	Before Soaking	After Soaking	Before Soaking	After Soaking						
Wt. of wet sample + mold, g	W1	10124	10330	10080	10230	10450	10600						
Wt. of mould, g	W2	6600	6600	5840	5840	5900	5900						
Wt. of wet sample, g	W3= W1 -W2	3524	3730	4240	4390	4550	4700						
Volume of mold, cc	V	2123	2123	2123	2123	2123	2123						
Wet unit weight, g/cc	Dw = W3/V	1.66	1.76	2.00	2.07	2.14	2.21						
Dry unit weight, g/cc	Dw/(1+W8/100)	1.43	1.42	1.72	1.70	1.85	1.88						
Moisture Content Determination													
Wt. of wet sample + cont., g	W3	216.6	251.5	258.8	264.6	249.9	234.3						
Wt. of dry sample + cont., g	W4	191.2	210.2	228.5	224.2	220.6	204.2						
Wt. of water, g	W5= W3-W4	25.4	41.3	30.3	40.4	29.3	30.1						
Wt. of Container, g	W6	31.3	35.3	36.6	34.8	32.3	37.3						
Wt. of dry sampe, g	W7=W4-W6	159.9	174.9	191.9	189.4	188.3	166.9						
% Moisture Content	W8= (W5/W7) *100	15.9	23.6	15.8	21.3	15.6	18.0						
Swell Data (Surcharge Weight 9Kg)													
No. of Blows	10				30				65				
Initial Height of Sample: 116mm	Gauge reading		Swell		Gauge reading		Swell		Gauge reading		Swell		
	Initial	Final	mm	%	Initial	Final	mm	%	Initial	Final	mm	%	
	524	690	1.66	1.43	413	558	1.45	1.25	519	648	1.29	1.11	
CBR DATA													
Penetration(mm)	Std. load (KN)	Gauge reading	Load KN	Corrected CBR		Gauge reading	Load KN	Corrected CBR		Gauge reading	Load KN	Corrected CBR	
0		0	0			0	0			0	0		
0.64		55	1.19			101	2.19			141	3.06		
1.27		98	2.13			148	3.21			245	5.32		
1.91		124	2.69			224	4.86			324	7.03		
2.54	13	160	3.47	3.47	<b>26.7</b>	280	6.08	6.08	<b>46.7</b>	390	8.46	8.463	<b>65.1</b>
5.08	20	215	4.67	4.67	<b>23.3</b>	360	7.81	7.81	<b>39.1</b>	477	10.35	10.3509	<b>51.8</b>
6.35		266	5.77			424	9.20			541	11.74		
7.62		305	6.62			500	10.85			610	13.24		
Soaked CBR, %	26.7			46.7			65.1						
Dry Density, g/cc	1.43			1.72			1.85						
Swell, %	1.43			1.25			1.11						
Density Requirement	98%			Target Density			1.84			CBR			59.44
Density Requirement	95%			Target Density			1.76			CBR			52.36

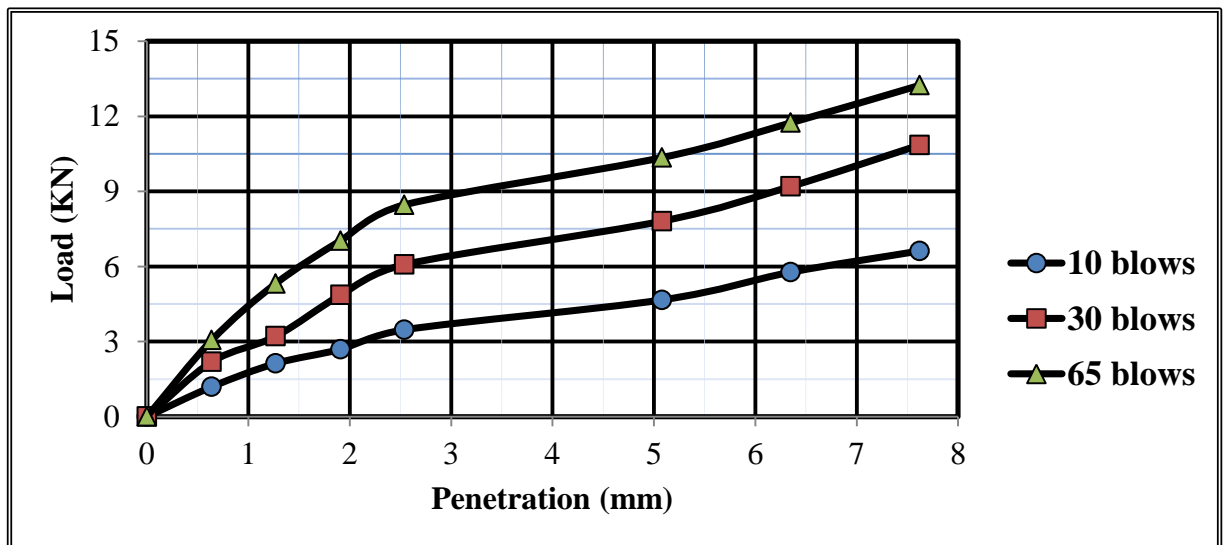


Figure D-4.1 Load penetration curve for laterite with 20% CSA [Soaked condition]

Table D-4.2 Result of CBR test for laterite with 20% CSA [Un soaked condition]

		Unit Weight Determination													
		No. of blows per layer		10 Blows		30 Blows		65 Blows							
		Condition of Sample		Before Soaking	After Soaking	Before Soaking	After Soaking	Before Soaking	After Soaking						
Wt. of wet sample + mold, g	W1	10134		10100		10490									
Wt. of mould, g	W2	6610		5860		5940									
Wt. of wet sample, g	W3= W1 -W2	3524		4240		4550									
Volume of mold, cc	V	2123		2123		2123									
Wet unit weight, g/cc	Dw = W3/V	1.66		2.00		2.14									
Dry unit weight, g/cc	Dw/(1+W8/100)	1.43		1.72		1.85									
		Moisture Content Determination													
Wt. of wet sample + cont., g	W3	220.6		260.8		253.5									
Wt. of dry sample + cont., g	W4	194.8		229.9		223.9									
Wt. of water, g	W5= W3-W4	25.8		30.9		29.6									
Wt. of Container, g	W6	32.3		34.6		34.3									
Wt. of dry sampe, g	W7=W4-W6	162.5		195.3		189.6									
% Moisture Content	W8= (W5/W7) *100	15.9		15.8		15.6									
No. of Blows		10		30		65									
Initial Height of Sample: 116mm	Gauge reading		Swell		Gauge reading		Swell								
	Initial	Final	mm	%	Initial	Final	mm	%							
CBR DATA															
Penetr-ation (mm)	Std. load (KN)	Gauge reading	Load KN	Corrected CBR KN	%	Gauge reading	Load KN	Corrected CBR KN	%	Gauge reading	Load KN	Corrected CBR KN	%		
0		0	0			0	0			0	0				
0.64		115	2.50			173	3.75			224	4.86				
1.27		158	3.43			220	4.77			328	7.12				
1.91		185	4.01			296	6.42			407	8.83				
2.54	13	224	4.86	4.86	<b>37.4</b>	352	7.64	7.64	<b>58.8</b>	473	10.26	10.2641	<b>79.0</b>		
5.08	20	278	6.03	6.03	<b>30.2</b>	432	9.37	9.37	<b>46.9</b>	560	12.15	12.152	<b>60.8</b>		
6.35		326	7.07			496	10.76			624	13.54				
7.62		365	7.92			572	12.41			693	15.04				
Un soaked CBR, %					37.4				58.8				79.0		
Dry Density, g/cc					1.43				1.72				1.85		
Swell, %					0.00				0				0.00		
Density Requirement					98%				Target Density				1.81	CBR	72.78
Density Requirement					95%				Target Density				1.79	CBR	65.02

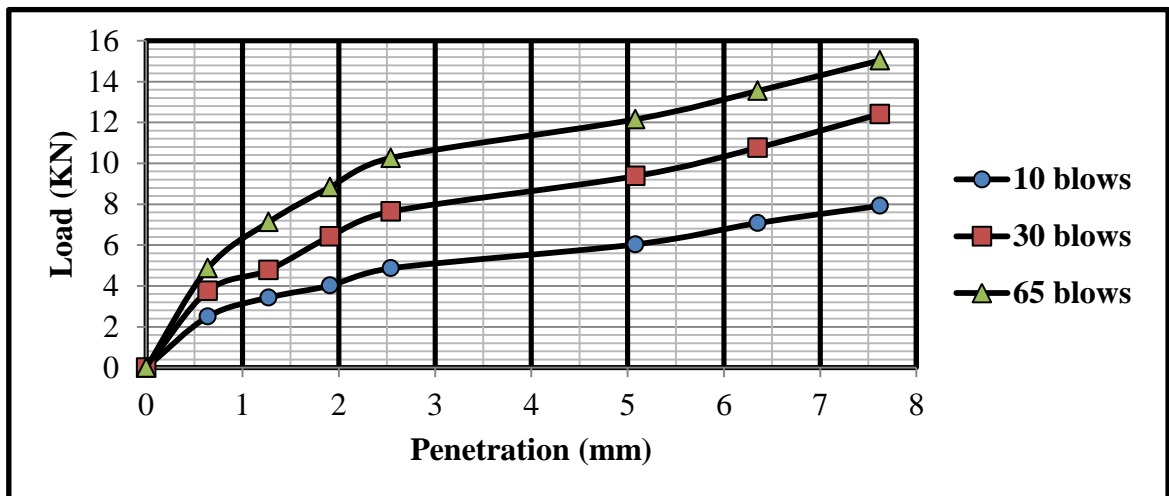


Figure D-4.2 Load penetration curve for laterite with 20% CSA [ Un soaked condition]

Table D-4.3 Result of CBR test for laterite with 40% CSA [Soaked condition]

Unit Weight Determination													
No. of blows per layer		10 Blows		30 Blows		65 Blows							
		Before Soaking	After Soaking	Before Soaking	After Soaking	Before Soaking	After Soaking	Condition of Sample					
Wt. of wet sample + mold, g		W1		10050	10200	10330	10450	10724	10960				
Wt. of mould, g		W2		6240	6240	5900	5900	5950	5950				
Wt. of wet sample, g		W3= W1 - W2		3810	3960	4430	4550	4774	5010				
Volume of mold, cc		V		2123	2123	2123	2123	2123	2123				
Wet unit weight, g/cc		Dw = W3/V		1.79	1.87	2.09	2.14	2.25	2.36				
Dry unit weight, g/cc		Dw/(1+W8/100)		1.58	1.51	1.83	1.79	1.98	2.01				
Moisture Content Determination													
Wt. of wet sample + cont., g		W3		204.6	213.3	265.6	245.5	261.6	238.3				
Wt. of dry sample + cont., g		W4		183.3	180.1	237.8	210.6	234.8	208.5				
Wt. of water, g		W5= W3- W4		21.3	33.2	27.8	34.9	26.8	29.8				
Wt. of Container, g		W6		30.4	36.6	35.2	31.6	37.3	38.2				
Wt. of dry sampe, g		W7= W4- W6		152.9	143.5	202.6	179.0	197.5	170.3				
% Moisture Content		W8= (W5/W7) *100		13.9	23.1	13.7	19.5	13.6	17.5				
Swell Data (Surcharge Weight 9Kg)													
No. of Blows		10				30				65			
Initial Height of Sample: 116mm		Gauge reading		Swell		Gauge reading		Swell		Gauge reading		Swell	
		Initial	Final	mm	%	Initial	Final	mm	%	Initial	Final	mm	%
		768	923	1.55	1.34	593	722	1.29	1.11	127	231	1.04	0.90
CBR DATA													
Penetration(mm)	Std. load (KN)	Gauge reading	Load KN	Corrected CBR		Gauge reading	Load KN	Corrected CBR		Gauge reading	Load KN	Corrected CBR	
0		0	0	KN	%	0	0	KN	%	0	0	KN	%
0.64		78	1.69			140	3.04			224	4.86		
1.27		130	2.82			235	5.10			360	7.81		
1.91		155	3.36			330	7.16			480	10.42		
2.54	13	200	4.34	4.34	<b>33.4</b>	421	9.14	9.14	<b>70.3</b>	550	11.94	11.935	<b>91.8</b>
5.08	20	258	5.60	5.60	<b>28.0</b>	625	13.56	13.56	<b>67.8</b>	740	16.06	16.058	<b>80.3</b>
6.35		300	6.51			735	15.95			844	18.31		
7.62		356	7.73			833	18.08			935	20.29		
Soaked CBR, %		33.4				70.3				91.8			
Dry Density, g/cc		1.58				1.83				1.98			
Swell, %		1.34				1.11				0.90			
Density Requirement		98%				Target Density		1.95		CBR		84.63	
Density Requirement		95%				Target Density		1.87		CBR		76.03	

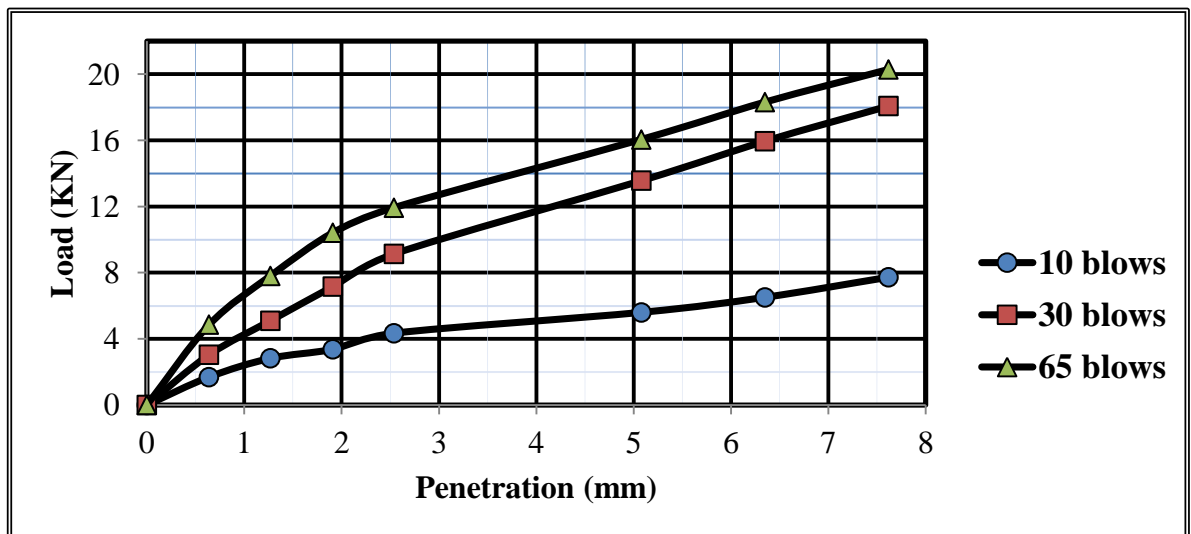


Figure D-4.3 Load penetration curve for laterite with 40% CSA [Soaked condition]

Table D-4.4 Result of CBR test for laterite with 40% CSA [Un soaked condition]

		Unit Weight Determination											
		No. of blows per layer		10 Blows		30 Blows		65 Blows					
		Condition of Sample		Before Soaking	After Soaking	Before Soaking	After Soaking	Before Soaking	After Soaking				
Wt. of wet sample + mold, g	W1	10220		10280		10580							
Wt. of mould, g	W2	6410		5860		5810							
Wt. of wet sample, g	W3= W1 -W2	3810		4420		4770							
Volume of mold, cc	V	2123		2123		2123							
Wet unit weight, g/cc	Dw = W3/V	1.79		2.08		2.25							
Dry unit weight, g/cc	Dw/(1+W8/100)	1.58		1.83		1.98							
		Moisture Content Determination											
Wt. of wet sample + cont., g	W3	215.3		254.8		248.7							
Wt. of dry sample + cont., g	W4	192.9		228.5		222.7							
Wt. of water, g	W5= W3-W4	22.4		26.3		26							
Wt. of Container, g	W6	31.3		36.6		31.4							
Wt. of dry sampe, g	W7=W4-W6	161.6		191.9		191.3							
% Moisture Content	W8= (W5/W7) *100	13.9		13.7		13.6							
No. of Blows		10		30				65					
Initial Height of Sample: 116mm	Gauge reading		Swell		Gauge reading		Swell		Gauge reading		Swell		
	Initial	Final	mm	%	Initial	Final	mm	%	Initial	Final	mm	%	
CBR DATA													
Penetr-ation (mm)	Std. load (KN)	Gauge reading	Load KN	Corrected CBR KN	%	Gauge reading	Load KN	Corrected CBR KN	%	Gauge reading	Load KN	Corrected CBR KN	%
0		0	0			0	0			0	0		
0.64		130	2.82			198	4.30			289	6.27		
1.27		182	3.95			293	6.36			423	9.18		
1.91		207	4.49			388	8.42			545	11.83		
2.54	13	255	5.53	5.53	<b>42.6</b>	479	10.39	10.39	<b>80.0</b>	618	13.41	13.4106	<b>103.2</b>
5.08	20	313	6.79	6.79	<b>34.0</b>	683	14.82	14.82	<b>74.1</b>	802	17.40	17.4034	<b>87.0</b>
6.35		352	7.64			793	17.21			909	19.73		
7.62		408	8.85			891	19.33			1000	21.70		
Un soaked CBR, %		42.6				80.0				103.2			
Dry Density, g/cc		1.58				1.83				1.98			
Swell, %		0.00				0				0.00			
Density Requirement		98%		Target Density		1.95		CBR		95.47			
Density Requirement		198%		Target Density		1.89		CBR		86.19			

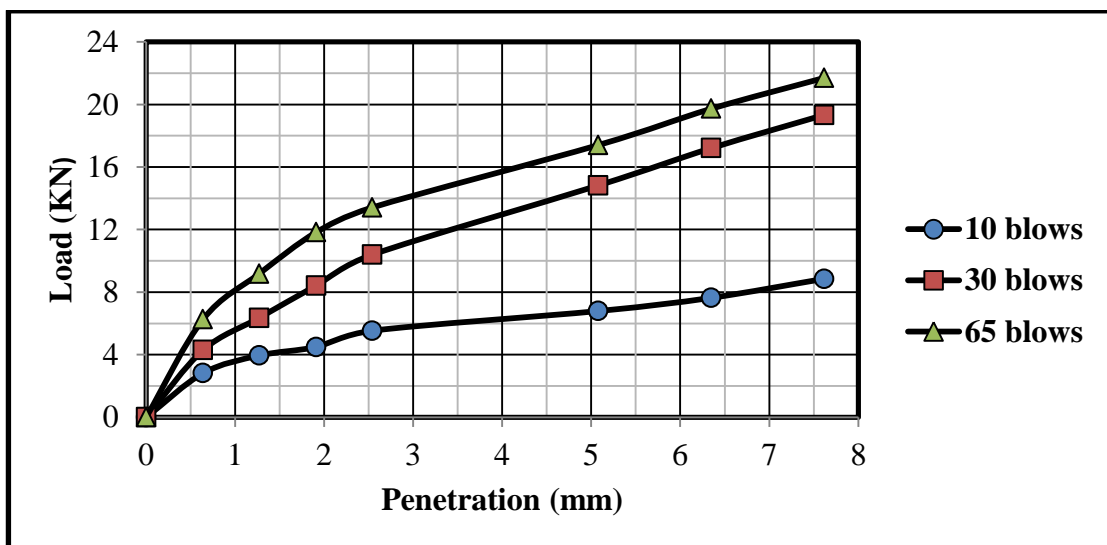


Figure D-4.4 Load penetration curve for laterite with 40% CSA [Un soaked condition]

Table D-4.5 Result of CBR test for laterite with 60% CSA [Soaked condition]

Unit Weight Determination													
No. of blows per layer		10 Blows		30 Blows		65 Blows							
Condition of Sample		Before Soaking	After Soaking	Before Soaking	After Soaking	Before Soaking	After Soaking						
Wt. of wet sample + mold, g	W1	9900	10260	10310	10630	10850	11070						
Wt. of mould, g	W2	5950	5950	5700	5700	5950	5950						
Wt. of wet sample, g	W3= W1 -W2	3950	4310	4610	4930	4900	5120						
Volume of mold, cc	V	2123	2123	2123	2123	2123	2123						
Wet unit weight, g/cc	Dw = W3/V	1.86	2.03	2.17	2.32	2.31	2.41						
Dry unit weight, g/cc	Dw/(1+W8/100)	1.67	1.70	1.95	2.00	2.08	2.11						
Moisture Content Determination													
Wt. of wet sample + cont., g	W3	211.2	264.6	213.3	245.5	267.6	241.2						
Wt. of dry sample + cont., g	W4	193.3	226.6	195.5	216.6	244.9	216.6						
Wt. of water, g	W5= W3-W4	17.9	38	17.8	28.9	22.7	24.6						
Wt. of Container, g	W6	33.3	34.3	35.3	37.3	36.2	42						
Wt. of dry sampe, g	W7=W4-W6	160.0	192.3	160.2	179.3	208.7	174.6						
% Moisture Content	W8= (W5/W7) *100	11.2	19.8	11.1	16.1	10.9	14.1						
Swell Data (Surcharge Weight 9Kg)													
No. of Blows		10				30				65			
Initial Height of Sample: 116mm	Gauge reading		Swell		Gauge reading		Swell		Gauge reading		Swell		
	Initial	Final	mm	%	Initial	Final	mm	%	Initial	Final	mm	%	
	524	648	1.24	1.07	668	765	0.97	0.84	425	506	0.81	0.70	
CBR DATA													
Penetration(mm)	Std. load (KN)	Gauge reading	Load KN	Corrected CBR KN	%	Gauge reading	Load KN	Corrected CBR KN	%	Gauge reading	Load KN	Corrected CBR KN	%
0		0	0			0	0			0	0		
0.64		85	1.84			165	3.58			280	6.08		
1.27		124	2.69			280	6.08			430	9.33		
1.91		175	3.80			440	9.55			575	12.48		
2.54	13	224	4.86	4.86	<b>37.4</b>	570	12.37	12.37	<b>95.1</b>	700	15.19	15.19	<b>116.8</b>
5.08	20	270	5.86	5.86	<b>29.3</b>	610	13.24	13.24	<b>66.2</b>	810	17.58	17.577	<b>87.9</b>
6.35		310	6.73			735	15.95			920	19.96		
7.62		375	8.14			854	18.53			1010	21.92		
Soaked CBR, %				37.4				95.1				116.8	
Dry Density, g/cc				1.67				1.95				2.08	
Swell, %				1.07				0.84				0.70	
Density Requirement				98%		Target Density		2.05		CBR		109.5	
Density Requirement				95%		Target Density		1.99		CBR		98.4	

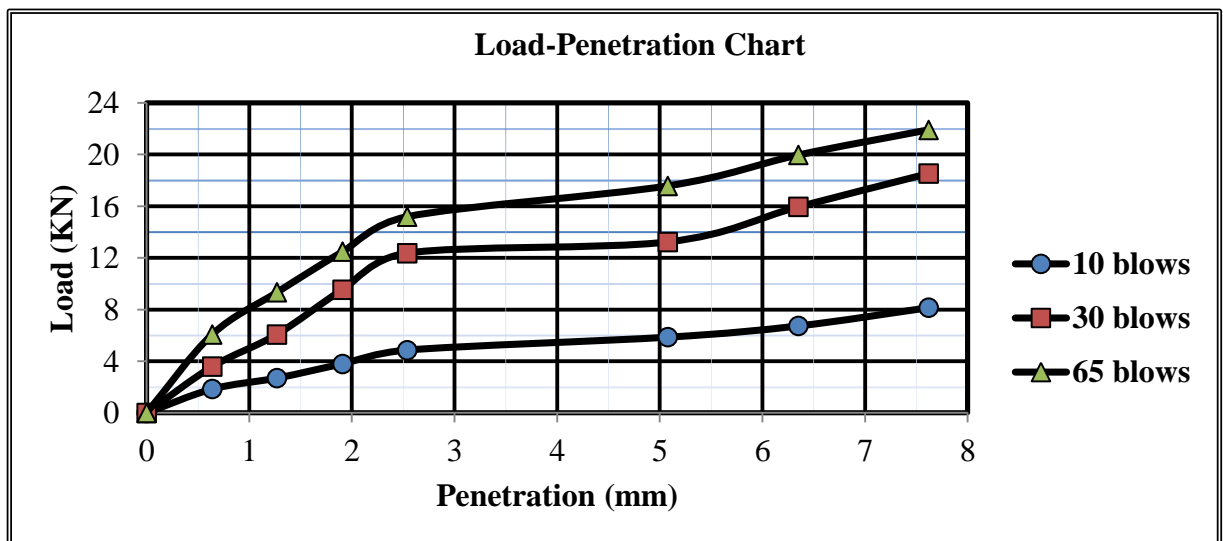


Figure D-4.5 Load penetration curve for laterite with 60% CSA [Soaked condition]

Table D-4.6 Result of CBR test for laterite with 60% CSA [Un soaked condition]

		Unit Weight Determination											
		No. of blows per layer		10 Blows		30 Blows		65 Blows					
		Condition of Sample		Before Soaking	After Soaking	Before Soaking	After Soaking	Before Soaking	After Soaking				
Wt. of wet sample + mold, g	W1	10295		10400		10790							
Wt. of mould, g	W2	6350		5810		5900							
Wt. of wet sample, g	W3= W1 -W2	3945		4590		4890							
Volume of mold, cc	V	2123		2123		2123							
Wet unit weight, g/cc	Dw = W3/V	1.86		2.16		2.30							
Dry unit weight, g/cc	Dw/(1+W8/100)	1.67		1.95		2.08							
		Moisture Content Determination											
Wt. of wet sample + cont., g	W3	215.2		252.5		245.9							
Wt. of dry sample + cont., g	W4	196.7		230.9		224.9							
Wt. of water, g	W5= W3-W4	18.5		21.6		21							
Wt. of Container, g	W6	31.3		36.6		32.2							
Wt. of dry sampe, g	W7=W4-W6	165.4		194.3		192.7							
% Moisture Content	W8= (W5/W7) *100	11.2		11.1		10.9							
No. of Blows		10		30		65							
Initial Height of Sample: 116mm	Gauge reading		Swell		Gauge reading		Swell		Gauge reading		Swell		
	Initial	Final	mm	%	Initial	Final	mm	%	Initial	Final	mm	%	
CBR DATA													
Penetr-ation (mm)	Std. load (KN)	Gauge reading	Load KN	Corrected CBR KN	Corrected CBR %	Gauge reading	Load KN	Corrected CBR KN	Corrected CBR %	Gauge reading	Load KN	Corrected CBR KN	Corrected CBR %
0		0	0			0	0			0	0		
0.64		151	3.28			230	4.99			350	7.60		
1.27		190	4.12			345	7.49			500	10.85		
1.91		241	5.23			505	10.96			645	14.00		
2.54	13	290	6.29	6.29	<b>48.4</b>	635	13.78	13.78	<b>106.0</b>	770	16.71	16.709	<b>128.5</b>
5.08	20	336	7.29	7.29	<b>36.5</b>	675	14.65	14.65	<b>73.2</b>	880	19.10	19.096	<b>95.5</b>
6.35		376	8.16			800	17.36			990	21.48		
7.62		441	9.57			919	19.94			1080	23.44		
Un Soaked CBR, %		48.4				106.0				128.5			
Dry Density, g/cc		1.67				1.95				2.08			
Swell, %		0.00				0				0.00			
Density Requirement			98%		Target Density			2.05		CBR		119.8	
Density Requirement			95%		Target Density			1.99		CBR		109.5	

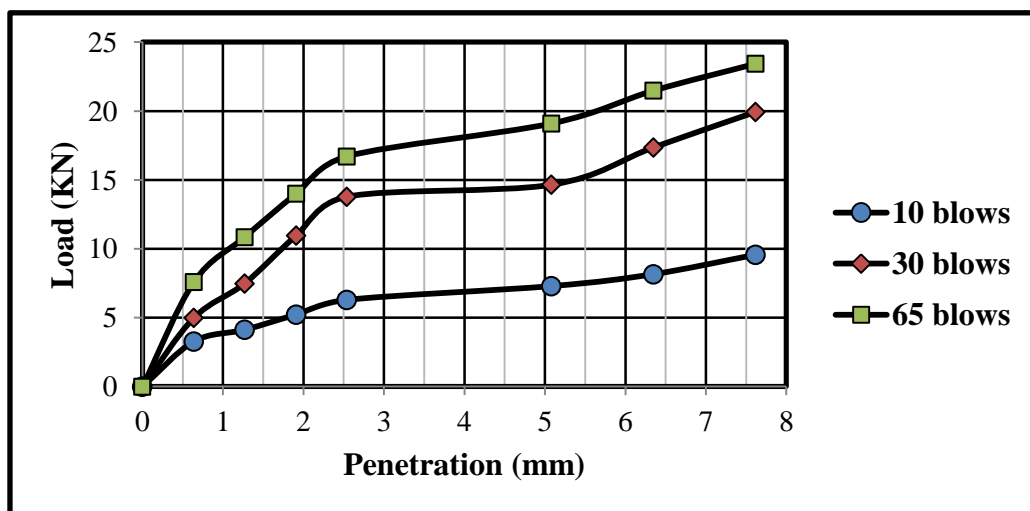


Figure D-4.6 Load penetration curve for laterite with 60% CSA [Un soaked condition]

**APPENDIX D-5 LOS ANGELES ABRASION TEST**

Table D-5.1 Result of Los Angeles abrasion test for laterite with 20% CSA

Los Angeles Abrasion Test							
AASHTO							
Sieve Sizes	1 1/2 - 1"	1 - 3/4"	3/4 - 1/2"	1/2 - 3/8"	3/8 - 1/4"	1/4" - No. 4	No.4 - No.3
Grade	A				C		D
Number of Balls	12 Balls				8 Balls		6 Balls
Wt. of Indicated Size	1250 ± 25	1250 ± 25	1250 ± 10	1250 ± 10	2500 ± 10	2500 ± 10	5000 ± 10
Wt. of Tested Sample							
Grade	/		B		/		
Number of Balls			12 Balls				
Wt. of Indicated Size			2500 ± 10	2500 ± 10			
Wt. of Tested Sample							
Test Results							
Trials						1	2
Number of Revolution						500	500
Total Wt. of Sample Tested (g)						4993.2	4986.5
Wt. of tested Sample Retained On No. 12 Sieve (g)						2562.1	2554.7
Percent loss (%)						48.69	48.77
Average of Percent loss (%)						48.73	

Table D-5.2 Result of Los Angeles abrasion test for laterite with 40% CSA

Los Angeles Abrasion Test							
AASHTO							
Sieve Sizes	1 1/2 - 1"	1 - 3/4"	3/4 - 1/2"	1/2 - 3/8"	3/8 - 1/4"	1/4" - No. 4	No.4 - No.3
Grade	A				C		D
Number of Balls	12 Balls				8 Balls		6 Balls
Wt. of Indicated Size	1250 ± 25	1250 ± 25	1250 ± 10	1250 ± 10	2500 ± 10	2500 ± 10	5000 ± 10
Wt. of Tested Sample							
Grade	/		B		/		
Number of Balls			12 Balls				
Wt. of Indicated Size			2500 ± 10	2500 ± 10			
Wt. of Tested Sample							
Test Results							
Trials						1	2
Number of Revolution						500	500
Total Wt. of Sample Tested (g)						5016.5	5010.3
Wt. of tested Sample Retained On No. 12 Sieve (g)						3134.5	3132.6
Percent loss (%)						37.52	37.48
Average of Percent loss (%)						37.50	

Table D-5.3 Result of Los Angeles abrasion test for laterite with 60% CSA

Los Angeles Abrasion Test									
AASHTO T-96									
Sieve Sizes	1 1/2 - 1"	1 - 3/4"	3/4 - 1/2"	1/2 - 3/8"	3/8 - 1/4"	1/4" - No. 4	No.4 - No.3		
Grade	A				C		D		
Number of Balls	12 Balls				8 Balls		6 Balls		
Wt. of Indicated Size	1250 ± 25	1250 ± 25	1250 ± 10	1250 ± 10	2500 ± 10	2500 ± 10	5000 ± 10		
Wt. of Tested Sample									
Grade	B				C				
Number of Balls								12 Balls	
Wt. of Indicated Size								2500 ± 10	2500 ± 10
Wt. of Tested Sample									
Test Results									
Trials					1	2			
Number of Revolution					500	500			
Total Wt. of Sample Tested (g)					4987	4992			
Wt. of tested Sample Retained On No. 12 Sieve (g)					3552.9	3553.2			
Percent loss (%)					28.76	28.82			
Average of Percent loss (%)					28.79				

**APPENDIX D-6 TEN PERCENT FINES VALUE TEST**

Table D-6.1 Result of Ten per cent fines value test for laterite with 20% CSA

TEN PER CENT FINES VALUE (TFV)		
BS 812 Part 111: 1990		
Condition of Aggregate Tested	Soaked	
Trials	1.00	2.00
Mass of aggregate passing 14mm and retained on 10mm (Before Compression) = (M1)	2679.20	2681.40
Mass of aggregate passing 2.36mm (After Compression) = (M2)	242.10	246.80
% of material passing 2.36mm = (M2/M1)*100	9.04	9.20
Duration of testing, min	9.80	9.90
Maximum load , f (KN)	33.67	34.83
Force to required to produce 10% fines in KN = 14*f/(m+4)	36.16	36.93
Average Force Ttfv in KN	<b>36.54</b>	
<b>Remarks :</b> Per cent passing 2.36mm should be between 7.5 % and 12 %		
$F = (14+f)/(M+4)$		
Penetration of Plunger :	15 mm for rounded or partialy rounded aggregate	
	20mm for normally crushed aggregate	
	24mm for vesicular (honeycombed) aggregates	

Table D-6.2 Result of Ten per cent fines value test for laterite with 40% CSA

<b>TEN PER CENT FINES VALUE (TFV)</b>		
<b>BS 812 Part 111: 1990</b>		
Condition of Aggregate Tested	Soaked	
Trials	1.00	2.00
Mass of aggregate passing 14mm and retained on 10mm (Before Compression) = (M1)	2671.30	2674.20
Mass of aggregate passing 2.36mm (After Compression) = (M2)	308.10	310.30
% of material passing 2.36mm = $(M2/M1)*100$	11.53	11.60
Duration of testing, min	9.80	9.90
Maximum load , f (KN)	56.80	57.20
Force to required to produce 10% fines in KN = $14*f/(m+4)$	51.19	51.32
Average Force Ttfv in KN	51.26	
<b>Remarks :</b> Per cent passing 2.36mm should be between 7.5 % and 12 %		
$F = (14+f)/(M+4)$		
Penetration of Plunger : 15 mm for rounded or partialy rounded aggregate		
20mm for normally crushed aggregate		
24mm for vesicular (honeycombed) aggregates		

Table D-6.3 Result of Ten per cent fines value test for laterite with 60% CSA

<b>TEN PER CENT FINES VALUE (TFV)</b>		
<b>BS 812 Part 111: 1990</b>		
Condition of Aggregate Tested	Soaked	
Trials	1	2
Mass of aggregate passing 14mm and retained on 10mm (Before Compression) = (M1)	2755.20	2751.00
Mass of aggregate passing 2.36mm (After Compression) = (M2)	299.30	297.60
% of material passing 2.36mm = $(M2/M1)*100$	10.86	10.82
Duration of testing, min	10.10	10.20
Maximum load , f (KN)	69.80	70.64
Force to required to produce 10% fines in KN = $14*f/(m+4)$	65.75	66.74
Average Force Ttfv in KN	66.2	
<b>Remarks :</b> Per cent passing 2.36mm should be between 7.5 % and 12 %		
$F = (14+f)/(M+4)$		
Penetration of Plunger : 15 mm for rounded or partialy rounded aggregate		
20mm for normally crushed aggregate		
24mm for vesicular (honeycombed) aggregates		

**APPENDIX E**

**LABORATORY TEST RESULTS FOR MECHANICAL - LIME COMPOSITE STABILIZATION**

APPENDIX E-1 ATTERBERG LIMIT TEST

Table E-1.1 Result of Atterberg limit test for 0% CSA + 4% lime

No. of blows	Liquid limit			Plastic limit	
	35	27	20		
Wt. of cont. + wet soil (g)= W1	30.32	32.42	38.64	12.57	18.96
Wt.of cont. + dry soil (g) = W2	22.16	25.52	31.60	10.80	17.33
Wt. of container (g) = W3	5.46	11.95	18.31	5.42	12.68
Mass of moisture (g) (W1 - W2) = X	8.16	6.90	7.04	1.77	1.63
Wt. of dry soil (g) (W2 - W3) = Y	16.70	13.57	13.29	5.38	4.65
Moisture content (%) = (X/Y)*100	48.86	50.85	52.97	32.90	35.05
	51.4			34.0	
	<b>Plasticity index</b>			<b>17.4</b>	

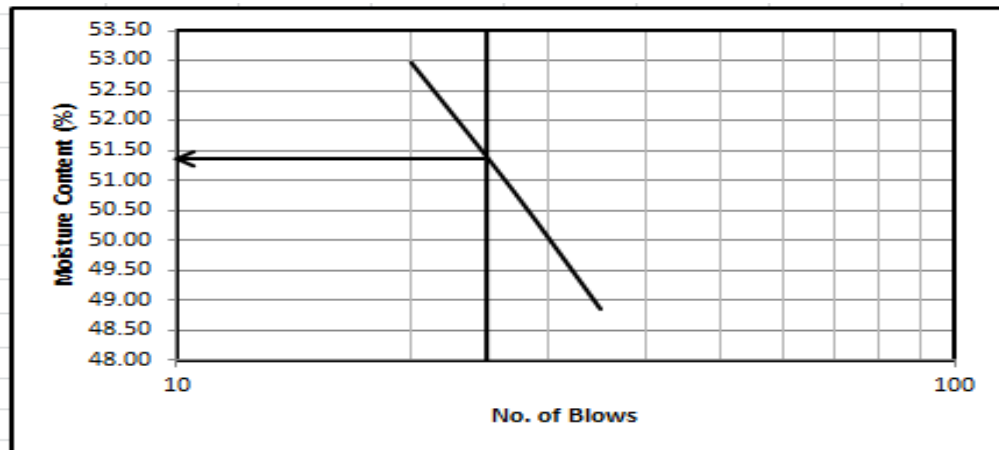


Figure E-1.1 liquid limit curve for 0% CSA + 4% lime

Table E-1.2 Result of Atterberg limit test for 0% CSA + 8% lime

No. of blows	Liquid limit			Plastic limit	
	35	27	20		
Wt. of cont. + wet soil (g)= W1	33.12	40.31	32.16	18.94	26.31
Wt.of cont. + dry soil (g) = W2	23.93	33.02	22.86	17.17	25.10
Wt. of container (g) = W3	5.16	18.67	5.25	12.63	21.75
Mass of moisture (g) (W1 - W2) = X	9.19	7.29	9.30	1.77	1.21
Wt. of dry soil (g) (W2 - W3) = Y	18.77	14.35	17.61	4.54	3.35
Moisture content (%) = (X/Y)*100	48.96	50.80	52.81	38.99	36.12
	51.3			37.6	
	<b>Plasticity index</b>			<b>13.7</b>	

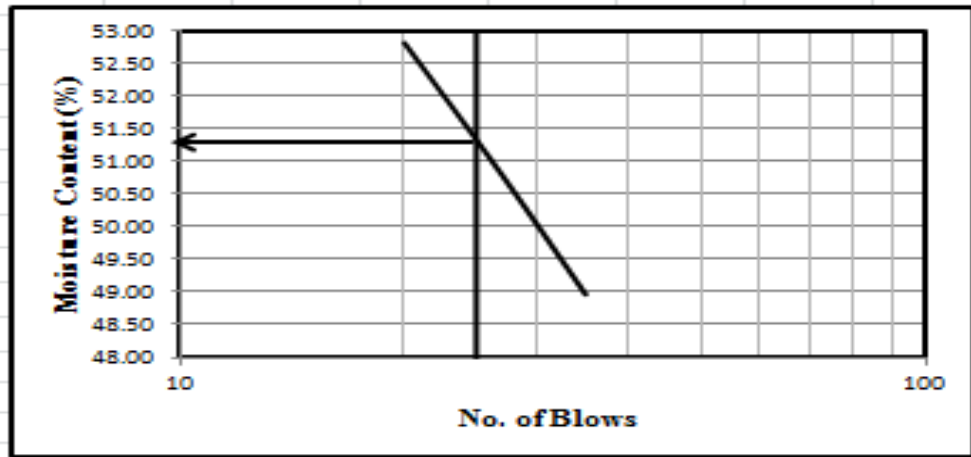


Figure E-1.2 liquid limit curve for 0% CSA + 8% lime

Table E-1.3 Result of Atterberg limit test for 0% CSA + 12% lime

No. of blows	Liquid limit			Plastic limit	
	34	27	20		
Wt. of cont. + wet soil (g)= W1	33.86	32.16	39.87	22.64	18.34
Wt. of cont. + dry soil (g) = W2	24.38	23.04	27.89	21.04	16.60
Wt. of container (g) = W3	4.52	4.68	4.74	16.54	12.59
Mass of moisture (g) (W1 - W2) = X	9.48	9.12	11.98	1.60	1.74
Wt. of dry soil (g) (W2 - W3) = Y	19.86	18.36	23.15	4.50	4.01
Moisture content (%) = (X/Y)*100	47.73	49.67	51.75	35.56	43.39
	50.3			39.5	
	<b>Plasticity index</b>			<b>10.8</b>	

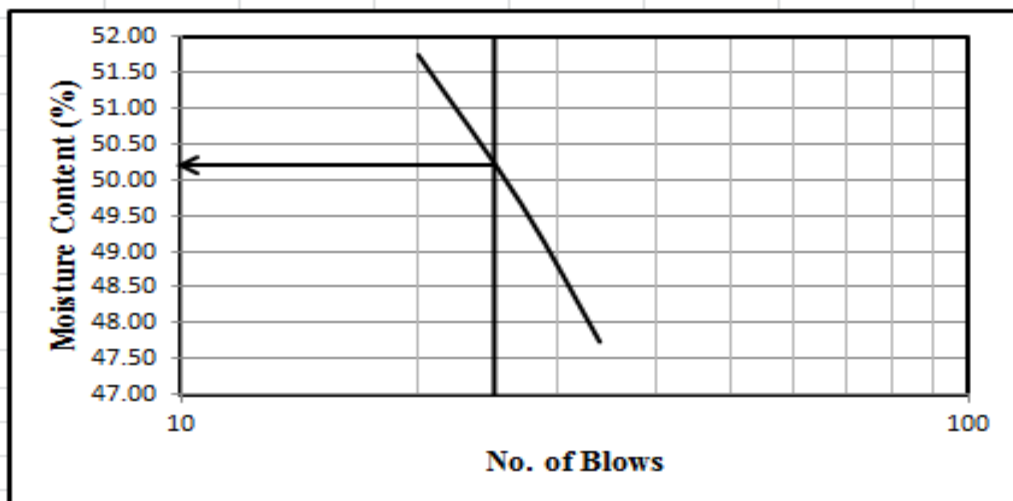


Figure E-1.3 liquid limit curve for 0% CSA + 12% lime

Table E-1.4 Result of Atterberg limit test for 0% CSA + 16% lime

No. of blows	Liquid limit			Plastic limit	
	35	27	20		
Wt. of cont. + wet soil (g) = W1	41.67	33.87	33.74	18.25	12.34
Wt. of cont. + dry soil (g) = W2	34.43	26.95	24.09	16.70	10.31
Wt. of container (g) = W3	18.85	12.62	4.90	12.94	4.98
Mass of moisture (g) (W1 - W2) = X	7.24	6.92	9.65	1.55	2.03
Wt. of dry soil (g) (W2 - W3) = Y	15.58	14.33	19.19	3.76	5.33
Moisture content (%) = (X/Y)*100	46.47	48.29	50.29	41.22	38.09
	48.9			39.7	
	<b>Plasticity index</b>			<b>9.2</b>	

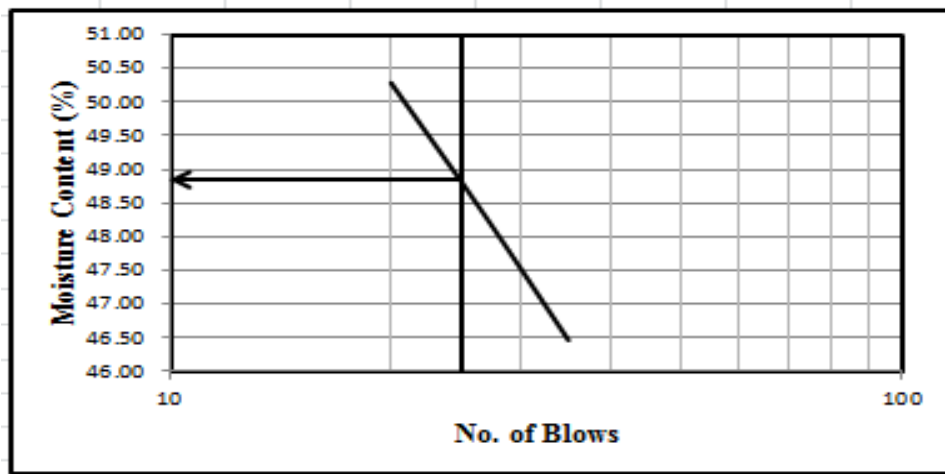


Figure E-1.4 liquid limit curve for 0% CSA + 16% lime

Table E-1.5 Result of Atterberg limit test for 20% CSA + 4% lime

No. of blows	Liquid limit			Plastic limit	
	34	26	19		
Wt. of cont. + wet soil (g) = W1	33.12	35.49	46.35	12.73	18.48
Wt. of cont. + dry soil (g) = W2	24.73	28.57	36.91	11.01	17.22
Wt. of container (g) = W3	4.68	12.62	16.00	5.16	12.78
Mass of moisture (g) (W1 - W2) = X	8.39	6.92	9.44	1.72	1.26
Wt. of dry soil (g) (W2 - W3) = Y	20.05	15.95	20.91	5.85	4.44
Moisture content (%) = (X/Y)*100	41.85	43.39	45.15	29.40	28.38
	43.6			28.9	
	<b>Plasticity index</b>			<b>14.7</b>	

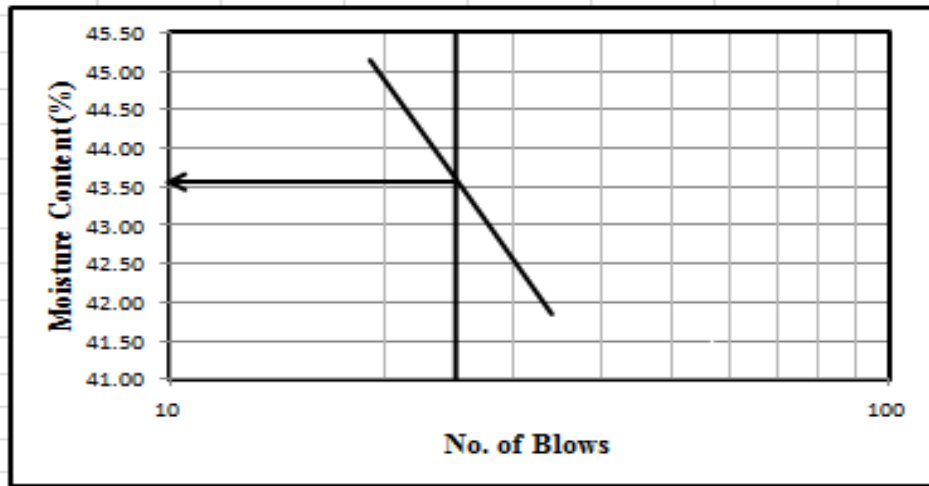


Figure E-1.5 liquid limit curve for 20% CSA + 4% lime

Table E-1.6 Result of Atterberg limit test for 20% CSA + 8% lime

No. of blows	Liquid limit			Plastic limit	
	33	26	19		
Wt. of cont. + wet soil (g) = W1	34.26	42.54	38.92	12.85	26.97
Wt. of cont. + dry soil (g) = W2	25.62	35.42	30.85	11.10	25.70
Wt. of container (g) = W3	4.53	18.61	12.54	5.24	21.51
Mass of moisture (g) (W1 - W2) = X	8.64	7.12	8.07	1.75	1.27
Wt. of dry soil (g) (W2 - W3) = Y	21.09	16.81	18.31	5.86	4.19
Moisture content (%) = (X/Y)*100	40.97	42.36	44.07	29.86	30.31
	42.5			30.1	
	<b>Plasticity index</b>			<b>12.4</b>	

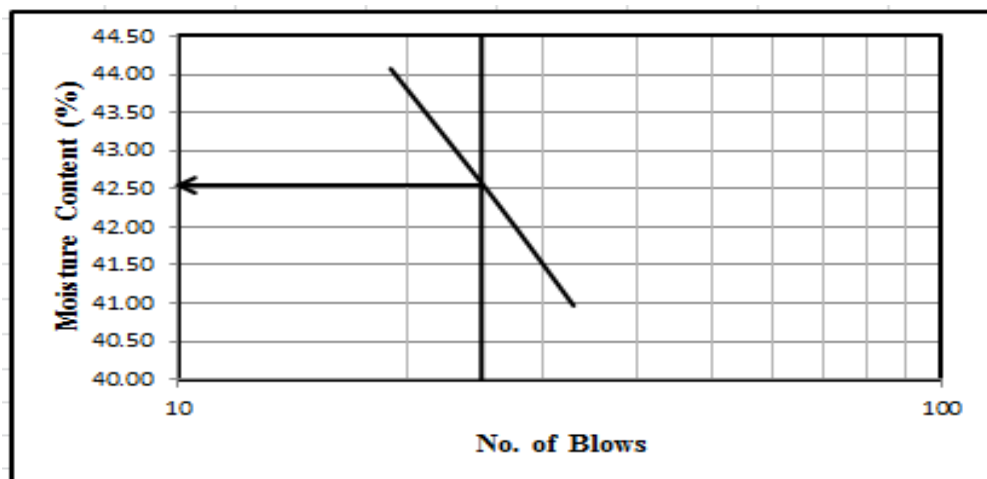


Figure E-1.6 liquid limit curve for 20% CSA + 8% lime

Table E-1.7 Result of Atterberg limit test for 20% CSA + 12% lime

No. of blows	Liquid limit			Plastic limit	
	33	26	20		
Wt. of cont. + wet soil (g)= W1	32.45	37.14	34.28	12.97	26.52
Wt. of cont. + dry soil (g) = W2	24.75	31.25	27.89	11.18	25.20
Wt. of container (g) = W3	4.83	16.58	12.60	5.34	20.85
Mass of moisture (g) (W1 - W2) = X	7.70	5.89	6.39	1.79	1.32
Wt. of dry soil (g) (W2 - W3) = Y	19.92	14.67	15.29	5.84	4.35
Moisture content (%) = (X/Y)*100	38.65	40.15	41.79	30.65	30.34
	40.4			30.5	
	<b>Plasticity index</b>			<b>9.9</b>	

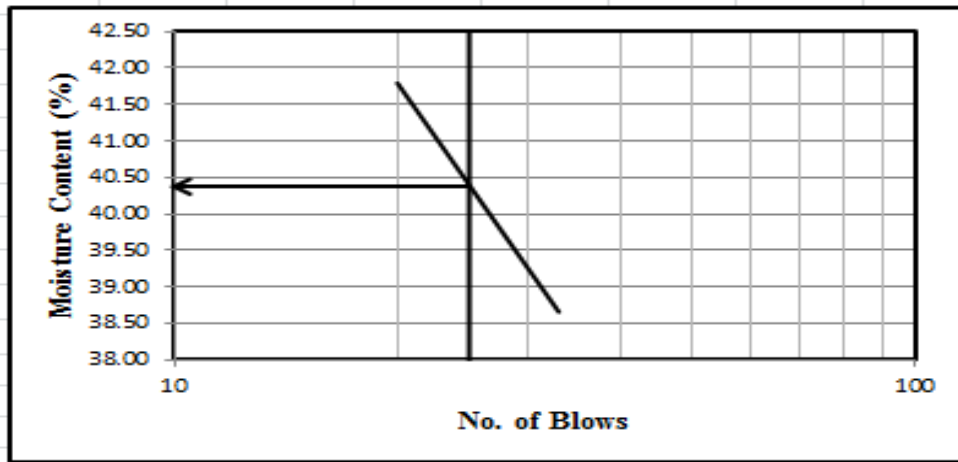


Figure E-1.7 liquid limit curve for 20% CSA + 12% lime

Table E-1.8 Result of Atterberg limit test for 20% CSA + 16% lime

No. of blows	Liquid limit			Plastic limit	
	33	26	20		
Wt. of cont. + wet soil (g)= W1	33.12	35.49	46.35	12.73	18.48
Wt. of cont. + dry soil (g) = W2	25.25	29.00	37.50	10.94	17.13
Wt. of container (g) = W3	4.68	12.62	16.00	5.16	12.78
Mass of moisture (g) (W1 - W2) = X	7.87	6.49	8.85	1.79	1.35
Wt. of dry soil (g) (W2 - W3) = Y	20.57	16.38	21.50	5.78	4.35
Moisture content (%) = (X/Y)*100	38.26	39.62	41.16	30.97	31.03
	39.8			31.0	
	<b>Plasticity index</b>			<b>8.8</b>	

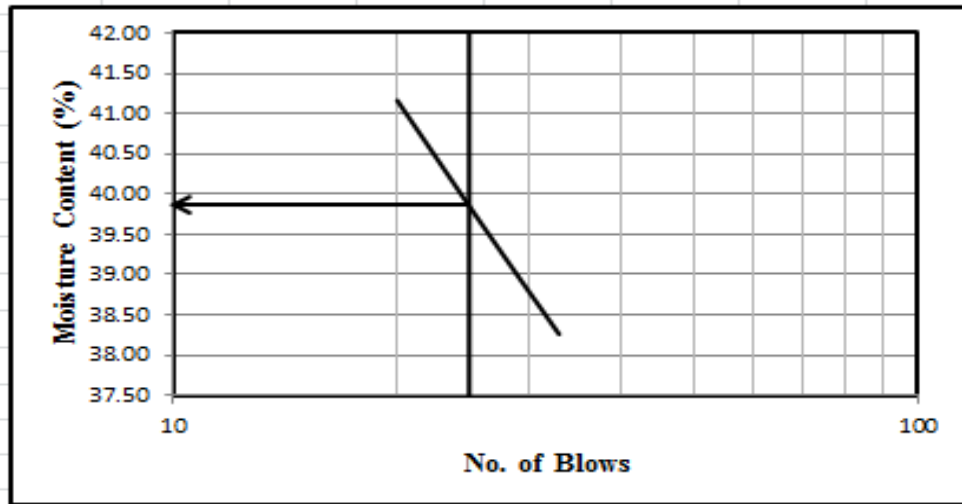


Figure E-1.8 liquid limit curve for 20% CSA + 16% lime

Table E-1.9 Result of Atterberg limit test for 40% CSA + 4% lime

No. of blows	Liquid limit			Plastic limit	
	34	28	23		
Wt. of cont. + wet soil (g)= W1	38.91	32.74	33.78	12.85	18.53
Wt. of cont. + dry soil (g) = W2	33.78	27.59	26.03	11.26	17.35
Wt. of container (g) = W3	18.32	12.69	4.56	4.80	12.55
Mass of moisture (g) (W1 - W2) = X	5.13	5.15	7.75	1.59	1.18
Wt. of dry soil (g) (W2 - W3) = Y	15.46	14.90	21.47	6.46	4.80
Moisture content (%) = (X/Y)*100	33.18	34.56	36.10	24.61	24.58
	35.5			24.6	
	<b>Plasticity index</b>			<b>10.9</b>	

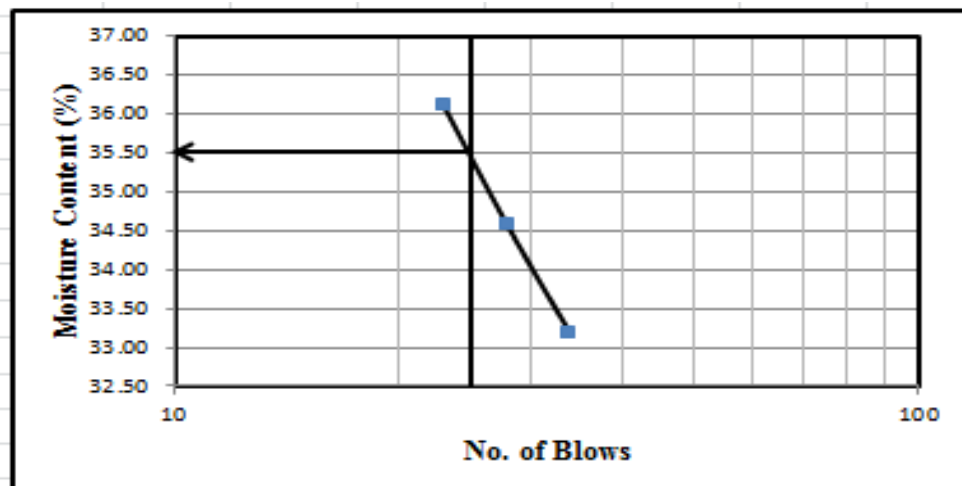


Figure E-1.9 liquid limit curve for 40% CSA + 4% lime

Table E-1.10 Result of Atterberg limit test for 40% CSA + 8% lime

No. of blows	Liquid limit			Plastic limit	
	33	26	20		
Wt. of cont. + wet soil (g)= W1	33.96	29.78	41.25	24.58	18.52
Wt. of cont. + dry soil (g) = W2	26.92	23.46	35.85	23.26	17.23
Wt. of container (g) = W3	5.32	4.83	20.61	18.42	12.53
Mass of moisture (g) (W1 - W2) = X	7.04	6.32	5.40	1.32	1.29
Wt. of dry soil (g) (W2 - W3) = Y	21.60	18.63	15.24	4.84	4.70
Moisture content (%) = (X/Y)*100	32.59	33.92	35.43	27.27	27.45
	34.2			27.4	
	<b>Plasticity index</b>			<b>6.8</b>	

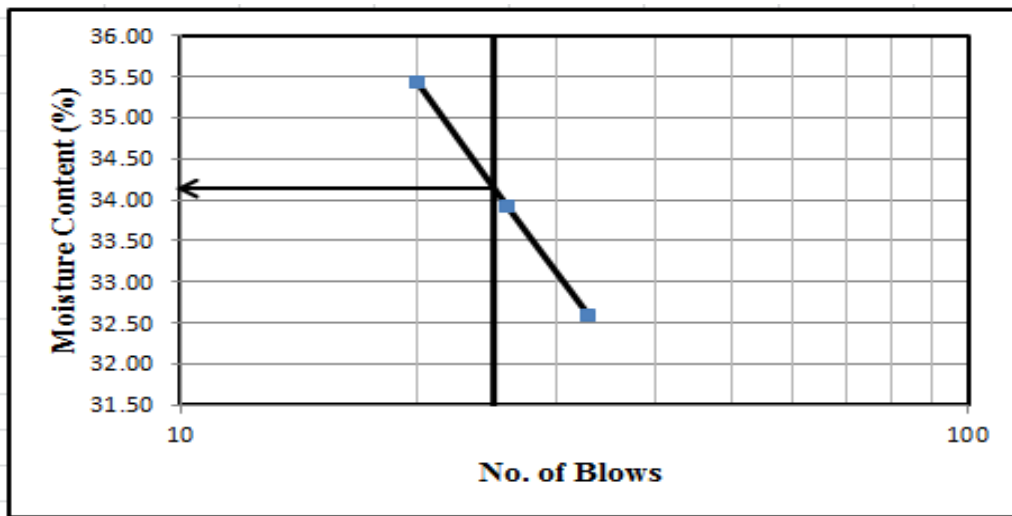


Figure E-1.10 liquid limit curve for 40% CSA + 8% lime

Table E-1.11 Result of Atterberg limit test for 40% CSA + 12% lime

No. of blows	Liquid limit			Plastic limit	
	34	27	21		
Wt. of cont. + wet soil (g)= W1	30.56	34.17	36.52	24.58	12.50
Wt. of cont. + dry soil (g) = W2	24.31	28.75	28.46	23.18	10.76
Wt. of container (g) = W3	4.69	12.43	5.17	18.37	4.86
Mass of moisture (g) (W1 - W2) = X	6.25	5.42	8.06	1.40	1.74
Wt. of dry soil (g) (W2 - W3) = Y	19.62	16.32	23.29	4.81	5.90
Moisture content (%) = (X/Y)*100	31.86	33.21	34.61	29.11	29.49
	33.6			29.3	
	<b>Plasticity index</b>			<b>4.3</b>	

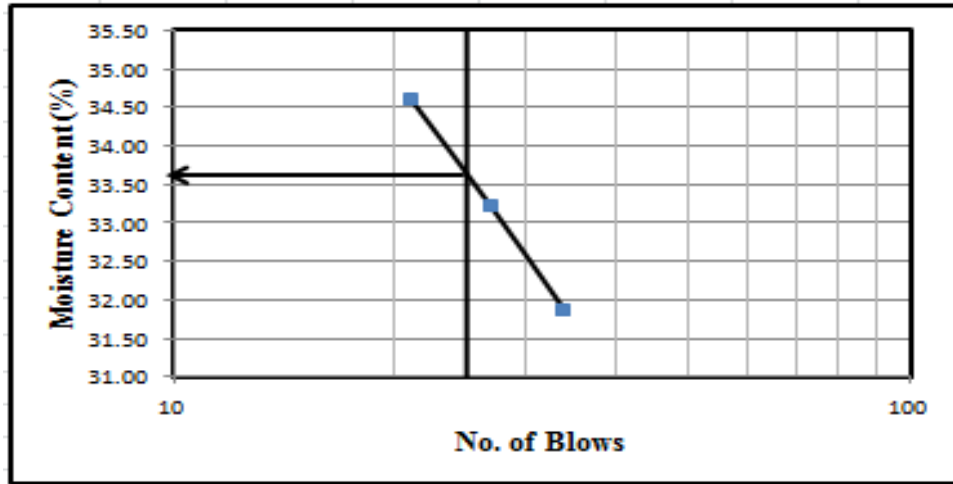


Figure E-1.11 liquid limit curve for 40% CSA + 12% lime

Table E-1.12 Result of Atterberg limit test for 40% CSA + 16% lime

No. of blows	Liquid limit			Plastic limit	
	33	26	20		
Wt. of cont. + wet soil (g) = W1	46.13	35.28	38.64	24.18	18.65
Wt. of cont. + dry soil (g) = W2	38.30	29.63	33.15	22.81	17.19
Wt. of container (g) = W3	12.46	11.78	16.58	18.20	12.34
Mass of moisture (g) (W1 - W2) = X	7.83	5.65	5.49	1.37	1.46
Wt. of dry soil (g) (W2 - W3) = Y	25.84	17.85	16.57	4.61	4.85
Moisture content (%) = (X/Y)*100	30.30	31.65	33.13	29.72	30.10
	31.9			29.9	
	<b>Plasticity index</b>			<b>2.0</b>	

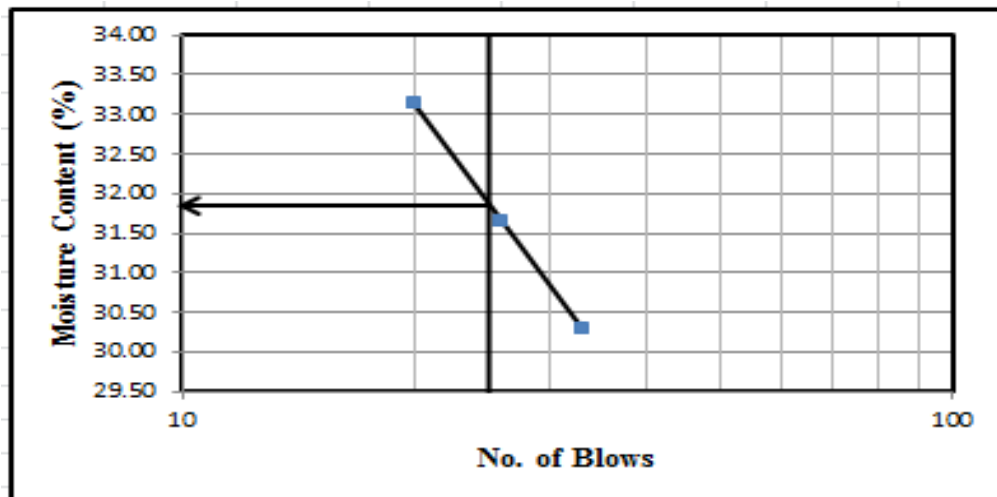


Figure E-1.12 liquid limit curve for 40% CSA + 16% lime

Table E-1.13 Result of Atterberg limit test for 60% CSA + 4% lime

No. of blows	Liquid limit			Plastic limit	
	33	26	20		
Wt. of cont. + wet soil (g)= W1	30.85	36.94	38.45	18.64	24.58
Wt.of cont. + dry soil (g) = W2	25.01	31.25	30.28	17.52	23.43
Wt. of container (g) = W3	5.03	12.61	4.73	12.59	18.47
Mass of moisture (g) (W1 - W2) = X	5.84	5.69	8.17	1.12	1.15
Wt. of dry soil (g) (W2 - W3) = Y	19.98	18.64	25.55	4.93	4.96
Moisture content (%) = (X/Y)*100	29.23	30.53	31.98	22.72	23.19
	30.7			23.0	
	<b>Plasticity index</b>			<b>7.7</b>	

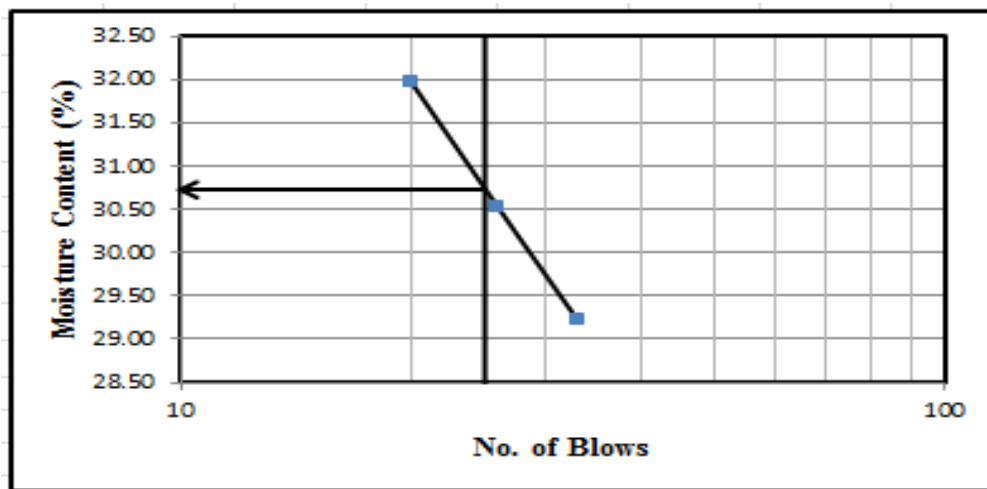


Figure E-1.13 liquid limit curve for 60% CSA + 4% lime

Table E-1.14 Result of Atterberg limit test for 60% CSA + 8% lime

No. of blows	Liquid limit			Plastic limit	
	32	26	21		
Wt. of cont. + wet soil (g)= W1	38.20	34.16	37.95	12.95	18.62
Wt.of cont. + dry soil (g) = W2	33.12	29.07	31.82	11.34	17.39
Wt. of container (g) = W3	15.24	11.96	12.16	4.83	12.47
Mass of moisture (g) (W1 - W2) = X	5.08	5.09	6.13	1.61	1.23
Wt. of dry soil (g) (W2 - W3) = Y	17.88	17.11	19.66	6.51	4.92
Moisture content (%) = (X/Y)*100	28.41	29.75	31.18	24.73	25.00
	30.0			24.9	
	<b>Plasticity index</b>			<b>5.1</b>	

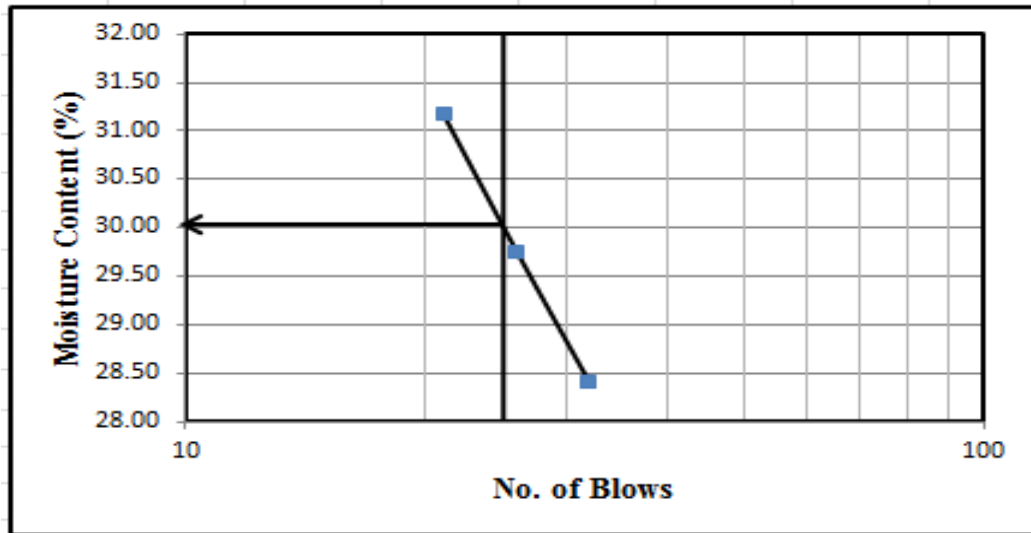


Figure E-1.14 liquid limit curve for 60% CSA + 8% lime

Table E-1.15 Result of Atterberg limit test for 60% CSA + 12% lime

No. of blows	Liquid limit		Plastic limit
Wt. of cont. + wet soil (g) = W1	<b>NP</b>		
Wt. of cont. + dry soil (g) = W2			
Wt. of container (g) = W3			
Mass of moisture (g) (W1 - W2) = X			
Wt. of dry soil (g) (W2 - W3) = Y			
Moisture content (%) = (X/Y)*100			

Table E-1.16 Result of Atterberg limit test for 60% CSA + 16% lime

No. of blows	Liquid limit		Plastic limit
Wt. of cont. + wet soil (g) = W1	<b>NP</b>		
Wt. of cont. + dry soil (g) = W2			
Wt. of container (g) = W3			
Mass of moisture (g) (W1 - W2) = X			
Wt. of dry soil (g) (W2 - W3) = Y			
Moisture content (%) = (X/Y)*100			

**APPENDIX E-2 Modified Proctor Test**

Table E-2.1 Result of modified proctor test for 0% CSA+4% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4480	4600	4750	4840	4820
Weight of Mould (g)	2940	2940	2940	2940	2940
Weight of Wet soil (g)	1540	1660	1810	1900	1880
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.66	1.79	1.95	2.05	2.02
<b>Moisture Content Determination</b>					
Weight of Wet soil + cont. (g)	341.2	249.2	302.9	291.5	210.2
Weight of Dry soil + cont. (g)	310.6	224.6	267.3	254.8	179.1
Weight of Container (g)	47.9	35.7	34.8	47.7	24.2
Weight of water (moisture) (g)	30.6	24.6	35.6	36.7	31.1
Weight of Dry soil (g)	262.7	188.9	232.5	207.1	154.9
Moisture content (%)	11.65	13.02	15.31	17.72	20.08
Dry Density (g/cc)	1.48	1.58	1.69	1.74	1.69
	<b>MDD, g/cc</b>		<b>1.74</b>	<b>OMC, %</b>	<b>17.72</b>

Figure E-2.1 Relationship between moisture and density for 0% CSA + 4% lime

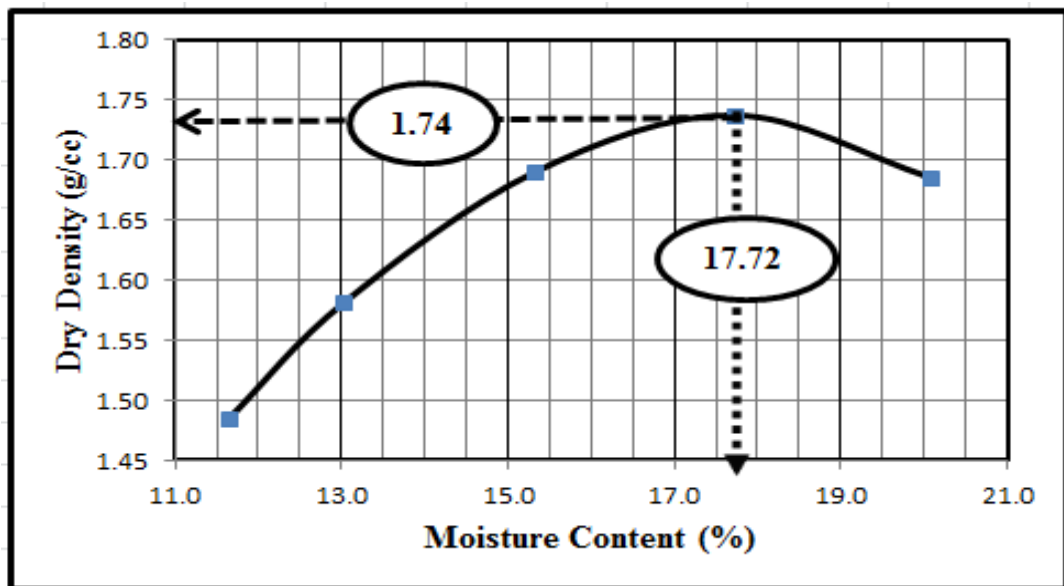


Table E-2.2 Result of modified proctor test for 0% CSA+8% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4480	4600	4745	4840	4840
Weight of Mould (g)	2940	2940	2940	2940	2940
Weight of Wet soil (g)	1540	1660	1805	1900	1900
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.66	1.79	1.94	2.05	2.05
Moisture Content Determination					
Weight of Wet soil + cont. (g)	323.7	339.3	299.5	216.3	204.2
Weight of Dry soil + cont. (g)	293.3	303.8	264.3	188.1	175.2
Weight of Container (g)	34.9	35.7	38.6	31.3	33.3
Weight of water (moisture) (g)	30.4	35.5	35.2	28.2	29.0
Weight of Dry soil (g)	258.4	268.1	225.7	156.8	141.9
Moisture content (%)	11.76	13.24	15.60	17.98	20.44
Dry Density (g/cc)	1.48	1.58	1.68	1.73	1.70
	<b>MDD, g/cc</b>		<b>1.73</b>	<b>OMC, %</b>	<b>18.0</b>

Figure E-2.2 Relationship between moisture and density for 0% CSA + 8% lime

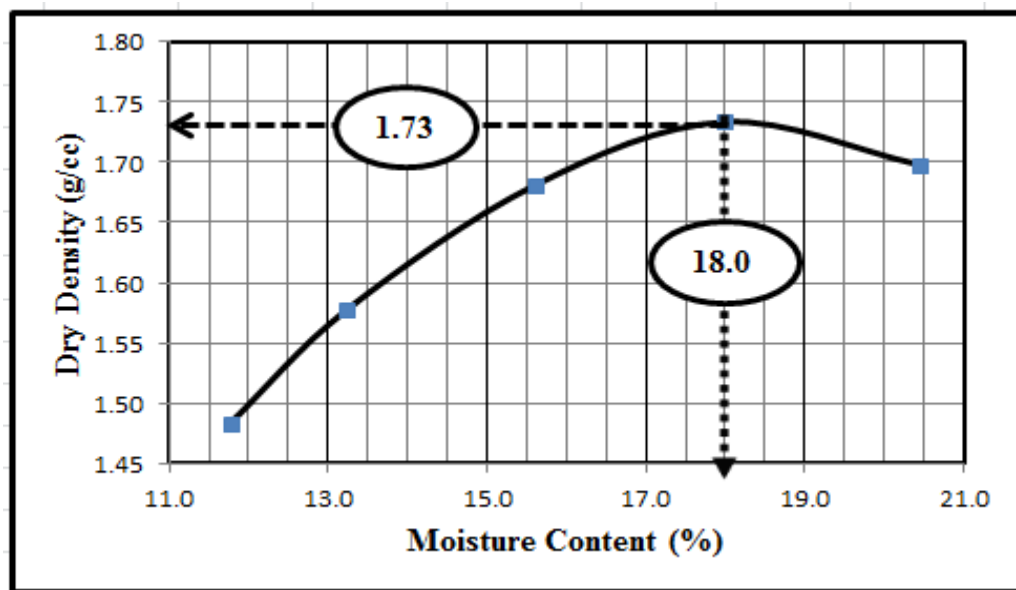


Table E-2.3 Result of modified proctor test for 0% CSA+12% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4400	4550	4680	4770	4750
Weight of Mould (g)	2895	2895	2895	2895	2895
Weight of Wet soil (g)	1505	1655	1785	1875	1855
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.62	1.78	1.92	2.02	2.00
<b>Moisture Content Determination</b>					
Weight of Wet soil + cont. (g)	265.5	307.7	298.8	210.3	264.6
Weight of Dry soil + cont. (g)	242.5	275.2	262.7	183.0	224.7
Weight of Container (g)	34.1	33.3	34.0	33.3	30.8
Weight of water (moisture) (g)	23.0	32.5	36.1	27.3	39.9
Weight of Dry soil (g)	208.4	241.9	228.7	149.7	193.9
Moisture content (%)	11.04	13.44	15.78	18.24	20.58
Dry Density (g/cc)	1.46	1.57	1.66	1.71	1.66
	<b>MDD, g/cc</b>		<b>1.71</b>	<b>OMC, %</b>	<b>18.2</b>

Figure E-2.3 Relationship between moisture and density for 0% CSA + 12% lime

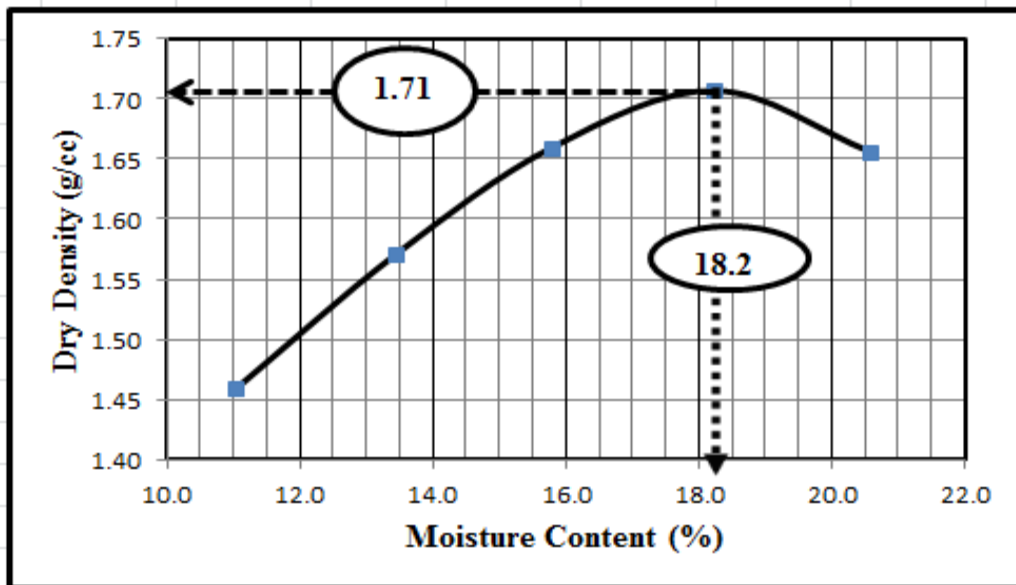


Table E-2.4 Result of modified proctor test for 0% CSA+16% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4410	4550	4700	4810	4770
Weight of Mould (g)	2940	2940	2940	2940	2940
Weight of Wet soil (g)	1470	1610	1760	1870	1830
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.58	1.73	1.89	2.01	1.97
Moisture Content Determination					
Weight of Wet soil + cont. (g)	299.9	380.8	367.2	204.2	216.6
Weight of Dry soil + cont. (g)	273.1	339.4	321.1	177.0	184.5
Weight of Container (g)	34.5	35.4	34.6	31.3	30.7
Weight of water (moisture) (g)	26.8	41.4	46.1	27.2	32.1
Weight of Dry soil (g)	238.6	304.0	286.5	145.7	153.8
Moisture content (%)	11.23	13.62	16.09	18.67	20.87
Dry Density (g/cc)	1.42	1.53	1.63	1.70	1.63
	<b>MDD, g/cc</b>		<b>1.70</b>	<b>OMC, %</b>	<b>18.7</b>

Figure E-2.4 Relationship between moisture and density for 0% CSA + 16% lime

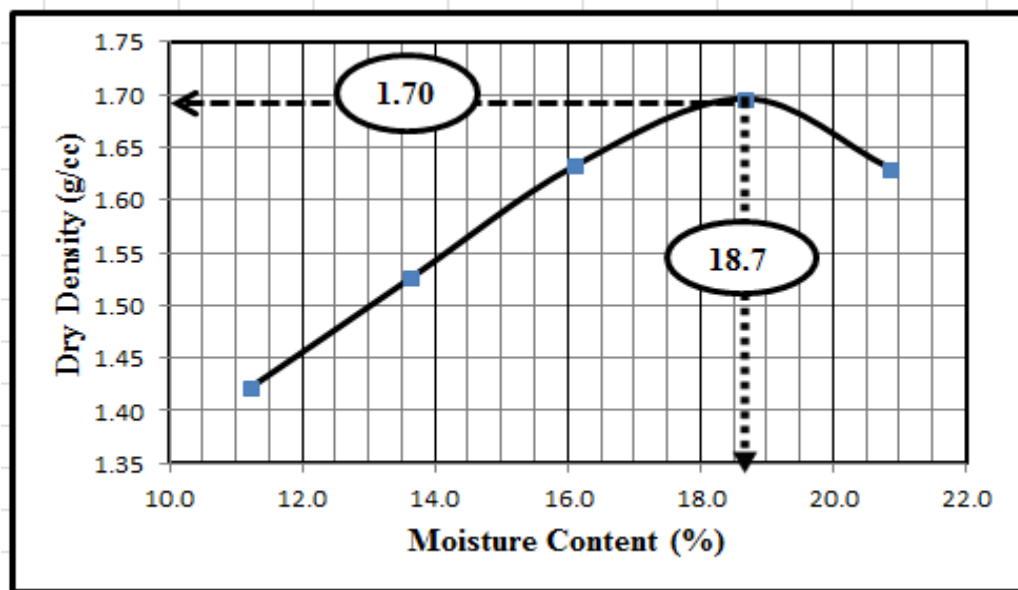


Table E-2.5 Result of modified proctor test for 20% CSA+4% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4550	4690	4820	4925	4905
Weight of Mould (g)	2895	2895	2895	2895	2895
Weight of Wet soil (g)	1655	1795	1925	2030	2010
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.78	1.93	2.07	2.19	2.16
Moisture Content Determination					
Weight of Wet soil + cont. (g)	322.7	328.8	353.3	267.8	249.3
Weight of Dry soil + cont. (g)	297.6	297.0	312.9	233.3	213.6
Weight of Container (g)	41.4	36.7	35.9	31.3	30.7
Weight of water (moisture) (g)	25.1	31.8	40.4	34.5	35.7
Weight of Dry soil (g)	256.2	260.3	277.0	202.0	182.9
Moisture content (%)	9.80	12.22	14.58	17.08	19.52
Dry Density (g/cc)	1.62	1.72	1.81	1.87	1.81
	<b>MDD, g/cc</b>		<b>1.87</b>	<b>OMC, %</b>	<b>17.1</b>

Figure E-2.5 Relationship between moisture and density for 20% CSA + 4% lime

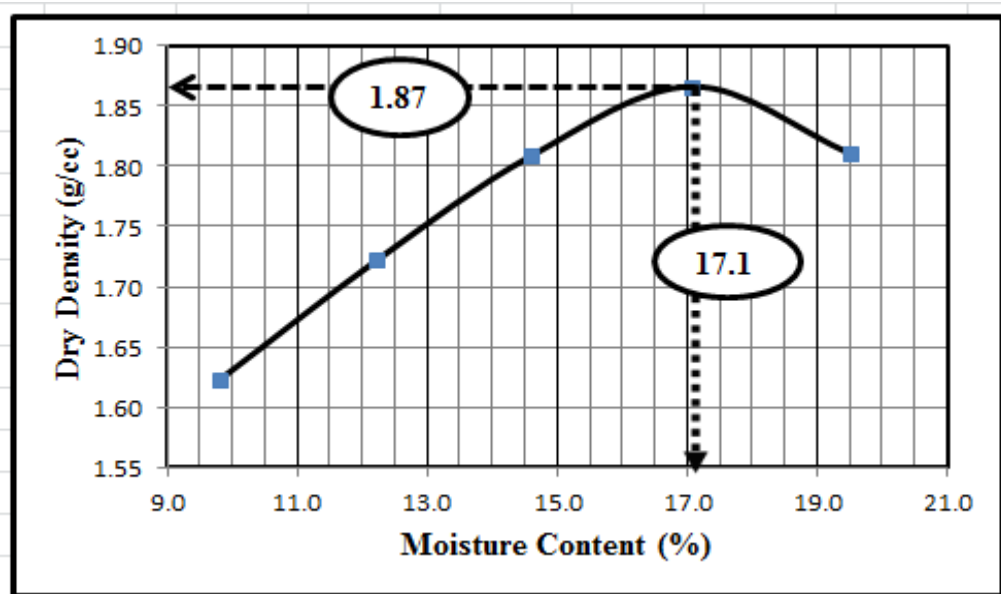


Table E-2.6 Result of modified proctor test for 20% CSA+8% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4550	4720	4880	4970	4950
Weight of Mould (g)	2940	2940	2940	2940	2940
Weight of Wet soil (g)	1610	1780	1940	2030	2010
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.73	1.92	2.09	2.19	2.16
Moisture Content Determination					
Weight of Wet soil + cont. (g)	302.4	286.6	215.6	216.3	207.8
Weight of Dry soil + cont. (g)	275.8	256.8	190.9	187.8	177.6
Weight of Container (g)	34.8	34.3	34.5	31.3	30.8
Weight of water (moisture) (g)	26.6	29.8	24.7	28.5	30.2
Weight of Dry soil (g)	241.0	222.5	156.4	156.5	146.8
Moisture content (%)	11.04	13.39	15.79	18.21	20.57
Dry Density (g/cc)	1.56	1.69	1.80	1.85	1.79
	<b>MDD, g/cc</b>		<b>1.85</b>	<b>OMC, %</b>	<b>18.2</b>

Figure E-2.6 Relationship between moisture and density for 20% CSA + 8% lime

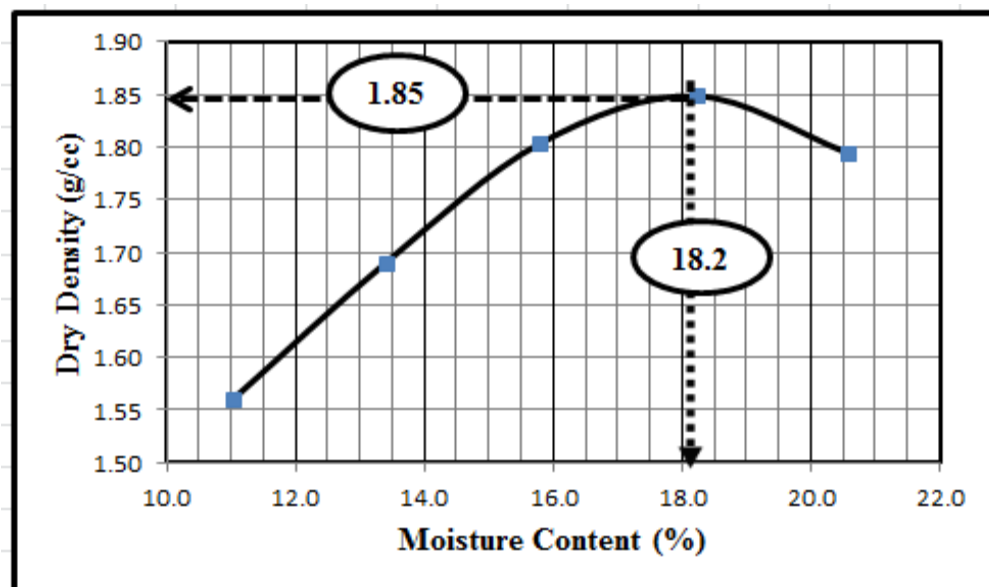


Table E-2.7 Result of modified proctor test for 20% CSA+12% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4580	4720	4860	4960	4940
Weight of Mould (g)	2940	2940	2940	2940	2940
Weight of Wet soil (g)	1640	1780	1920	2020	2000
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.77	1.92	2.07	2.17	2.15
<b>Moisture Content Determination</b>					
Weight of Wet soil + cont. (g)	270.7	244.7	207.6	213.3	264.9
Weight of Dry soil + cont. (g)	245.8	218.3	182.2	183.9	224.5
Weight of Container (g)	38.0	35.4	31.3	30.6	37.3
Weight of water (moisture) (g)	24.9	26.4	25.4	29.4	40.4
Weight of Dry soil (g)	207.8	182.9	150.9	153.3	187.2
Moisture content (%)	11.98	14.43	16.83	19.18	21.58
Dry Density (g/cc)	1.58	1.67	1.77	1.82	1.77
	<b>MDD, g/cc</b>		<b>1.82</b>	<b>OMC, %</b>	<b>19.2</b>

Figure E-2.7 Relationship between moisture and density for 20% CSA + 12% lime

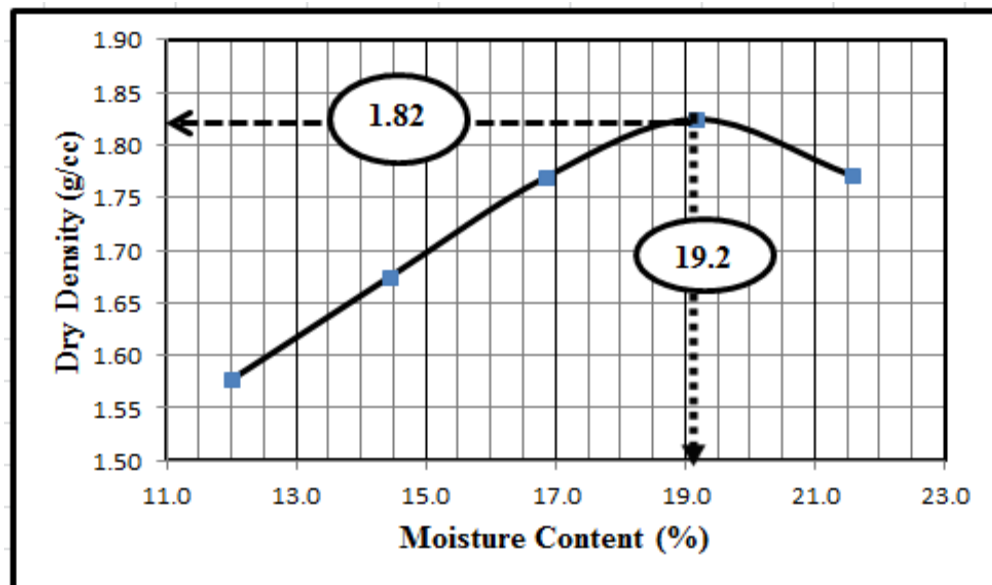


Table E-2.8 Result of modified proctor test for 20% CSA+16% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4600	4720	4850	4940	4920
Weight of Mould (g)	2940	2940	2940	2940	2940
Weight of Wet soil (g)	1660	1780	1910	2000	1980
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.79	1.92	2.06	2.15	2.13
<b>Moisture Content Determination</b>					
Weight of Wet soil + cont. (g)	313.3	324.7	296.1	213.3	207.2
Weight of Dry soil + cont. (g)	280.9	285.8	256.3	183.1	175.2
Weight of Container (g)	33.7	34.4	34.1	34.1	34.1
Weight of water (moisture) (g)	32.4	38.9	39.8	30.2	32.0
Weight of Dry soil (g)	247.2	251.4	222.2	149.0	141.1
Moisture content (%)	13.11	15.47	17.91	20.27	22.68
Dry Density (g/cc)	1.58	1.66	1.74	1.79	1.74
	<b>MDD, g/cc</b>		<b>1.79</b>	<b>OMC, %</b>	<b>20.3</b>

Figure E-2.8 Relationship between moisture and density for 20% CSA + 16% lime

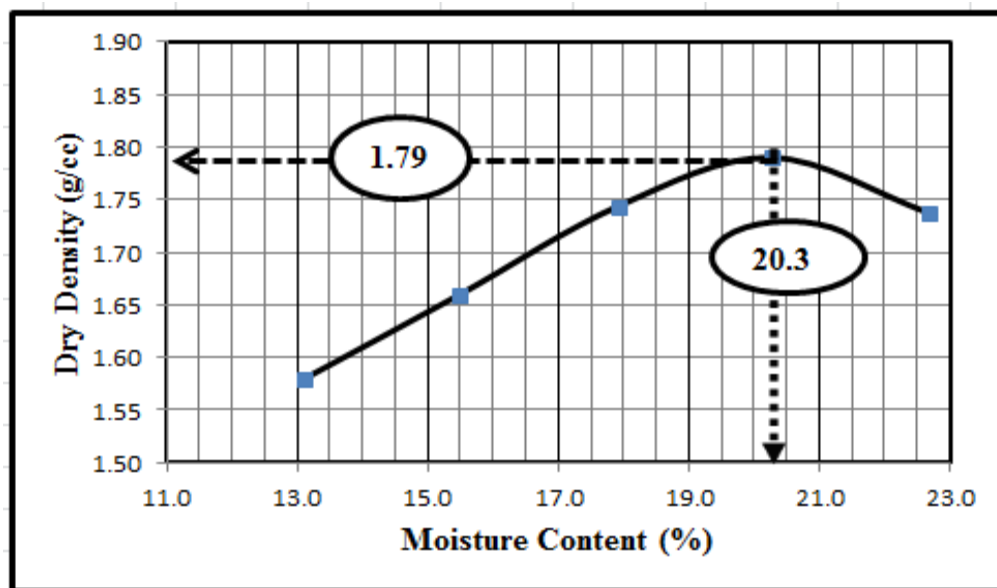


Table E-2.9 Result of modified proctor test for 40% CSA+4% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4700	4830	4960	5040	5020
Weight of Mould (g)	2940	2940	2940	2940	2940
Weight of Wet soil (g)	1760	1890	2020	2100	2080
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.89	2.03	2.17	2.26	2.24
Moisture Content Determination					
Weight of Wet soil + cont. (g)	337.2	277.1	213.3	211.6	264.6
Weight of Dry soil + cont. (g)	317.8	256.4	194.2	189.5	232.4
Weight of Container (g)	34.7	34.4	31.3	32.3	37.2
Weight of water (moisture) (g)	19.4	20.7	19.1	22.1	32.2
Weight of Dry soil (g)	283.1	222.0	162.9	157.2	195.2
Moisture content (%)	6.85	9.32	11.72	14.06	16.50
Dry Density (g/cc)	1.77	1.86	1.95	1.98	1.92
	<b>MDD, g/cc</b>		<b>1.98</b>	<b>OMC, %</b>	<b>14.1</b>

Figure E-2.9 Relationship between moisture and density for 40% CSA + 4% lime

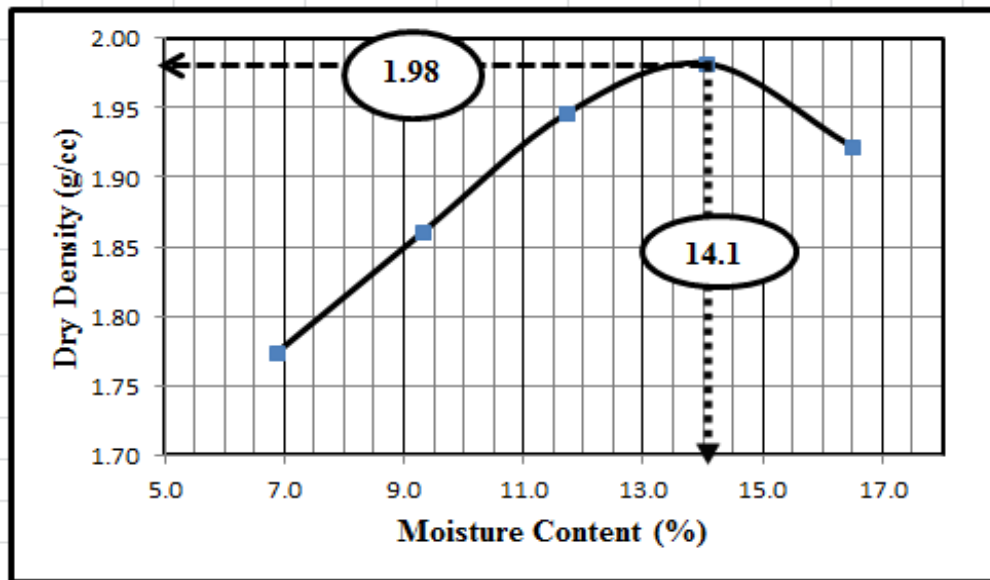


Table E-2.10 Result of modified proctor test for 40% CSA+8% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4680	4820	4940	5030	5030
Weight of Mould (g)	2940	2940	2940	2940	2940
Weight of Wet soil (g)	1740	1880	2000	2090	2090
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.87	2.02	2.15	2.25	2.25
Moisture Content Determination					
Weight of Wet soil + cont. (g)	313.2	298.3	270.8	211.2	204.2
Weight of Dry soil + cont. (g)	292.2	273.8	243.6	187.2	178.0
Weight of Container (g)	35.7	42.1	34.7	31.3	30.7
Weight of water (moisture) (g)	21.0	24.5	27.2	24.0	26.2
Weight of Dry soil (g)	256.5	231.7	208.9	155.9	147.3
Moisture content (%)	8.19	10.57	13.02	15.39	17.79
Dry Density (g/cc)	1.73	1.83	1.90	1.95	1.91
	<b>MDD, g/cc</b>		<b>1.95</b>	<b>OMC, %</b>	<b>15.4</b>

Figure E-2.10 Relationship between moisture and density for 40% CSA + 8% lime

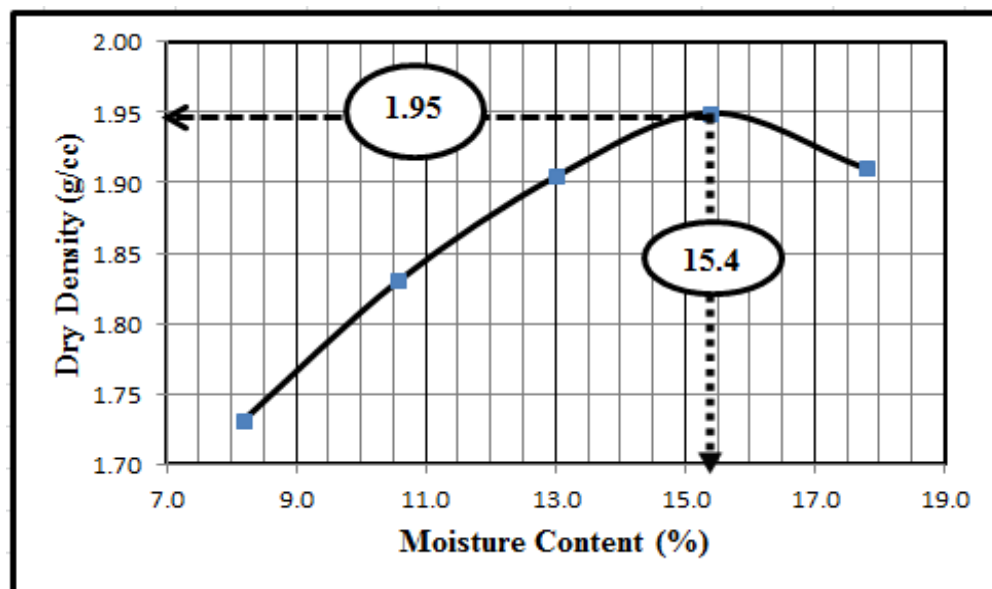


Table E-2.11 Result of modified proctor test for 40% CSA+12% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4520	4700	4870	4980	4960
Weight of Mould (g)	2895	2895	2895	2895	2895
Weight of Wet soil (g)	1625	1805	1975	2085	2065
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.75	1.94	2.13	2.24	2.22
<b>Moisture Content Determination</b>					
Weight of Wet soil + cont. (g)	305.6	331.9	247.9	207.6	218.8
Weight of Dry soil + cont. (g)	282.6	301.1	221.9	182.8	189.6
Weight of Container (g)	33.4	36.1	36.2	31.3	34.5
Weight of water (moisture) (g)	23.0	30.8	26.0	24.8	29.2
Weight of Dry soil (g)	249.2	265.0	185.7	151.5	155.1
Moisture content (%)	9.23	11.62	14.00	16.37	18.83
Dry Density (g/cc)	1.60	1.74	1.86	1.93	1.87
	<b>MDD, g/cc</b>		<b>1.93</b>	<b>OMC, %</b>	<b>16.4</b>

Figure E-2.11 Relationship between moisture and density for 40% CSA + 12% lime

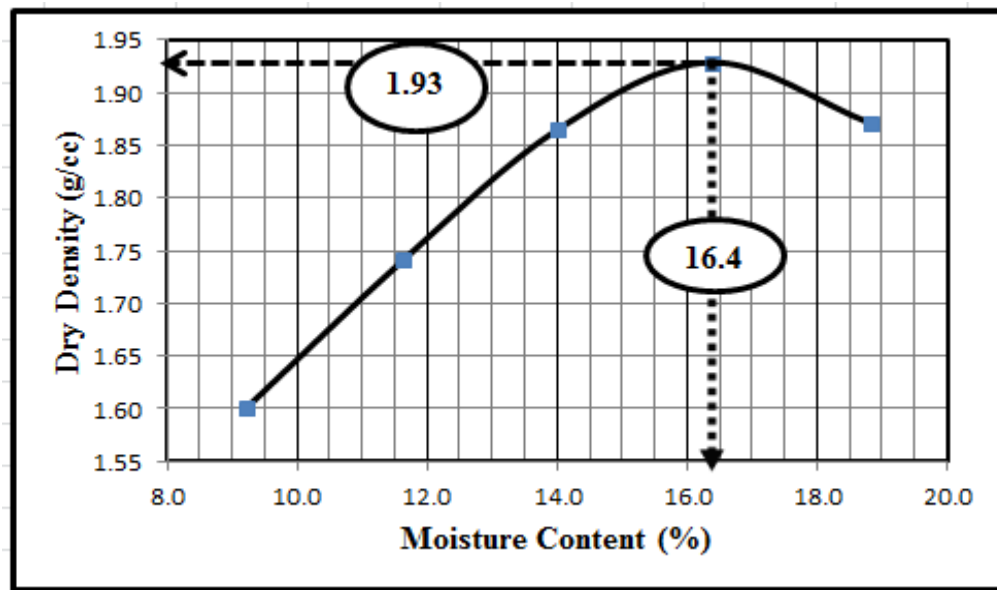


Table E-2.12 Result of modified proctor test for 40% CSA+16% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4640	4800	4940	5020	5015
Weight of Mould (g)	2940	2940	2940	2940	2940
Weight of Wet soil (g)	1700	1860	2000	2080	2075
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.83	2.00	2.15	2.24	2.23
<b>Moisture Content Determination</b>					
Weight of Wet soil + cont. (g)	292.6	252.1	350.1	213.3	216.6
Weight of Dry soil + cont. (g)	268.2	227.5	308.5	185.9	186.2
Weight of Container (g)	35.6	37.2	37.2	31.3	35.3
Weight of water (moisture) (g)	24.4	24.6	41.6	27.4	30.4
Weight of Dry soil (g)	232.6	190.3	271.3	154.6	150.9
Moisture content (%)	10.49	12.93	15.33	17.72	20.15
Dry Density (g/cc)	1.66	1.77	1.87	1.90	1.86
	<b>MDD, g/cc</b>		<b>1.90</b>	<b>OMC, %</b>	<b>17.7</b>

Figure E-2.12 Relationship between moisture and density for 40% CSA + 16% lime

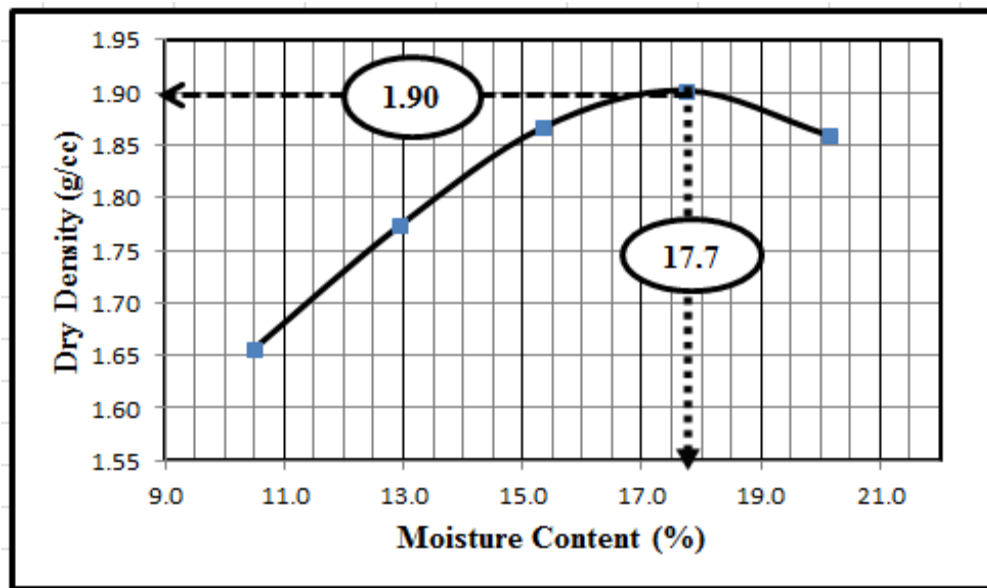


Table E-2.13 Result of modified proctor test for 60% CSA+4% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4700	4840	4970	5060	5040
Weight of Mould (g)	2895	2895	2895	2895	2895
Weight of Wet soil (g)	1805	1945	2075	2165	2145
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.94	2.09	2.23	2.33	2.31
Moisture Content Determination					
Weight of Wet soil + cont. (g)	336.2	295.5	394.4	204.2	213.3
Weight of Dry soil + cont. (g)	323.9	278.9	364.2	186.5	190.9
Weight of Container (g)	42.7	34.6	37.7	34.3	31.3
Weight of water (moisture) (g)	12.3	16.6	30.2	17.7	22.4
Weight of Dry soil (g)	281.2	244.3	326.5	152.2	159.6
Moisture content (%)	4.37	6.79	9.25	11.63	14.04
Dry Density (g/cc)	1.86	1.96	2.04	2.09	2.02
		<b>MDD, g/cc</b>	<b>2.09</b>	<b>OMC, %</b>	<b>11.6</b>

Figure E-2.13 Relationship between moisture and density for 60% CSA + 4% lime

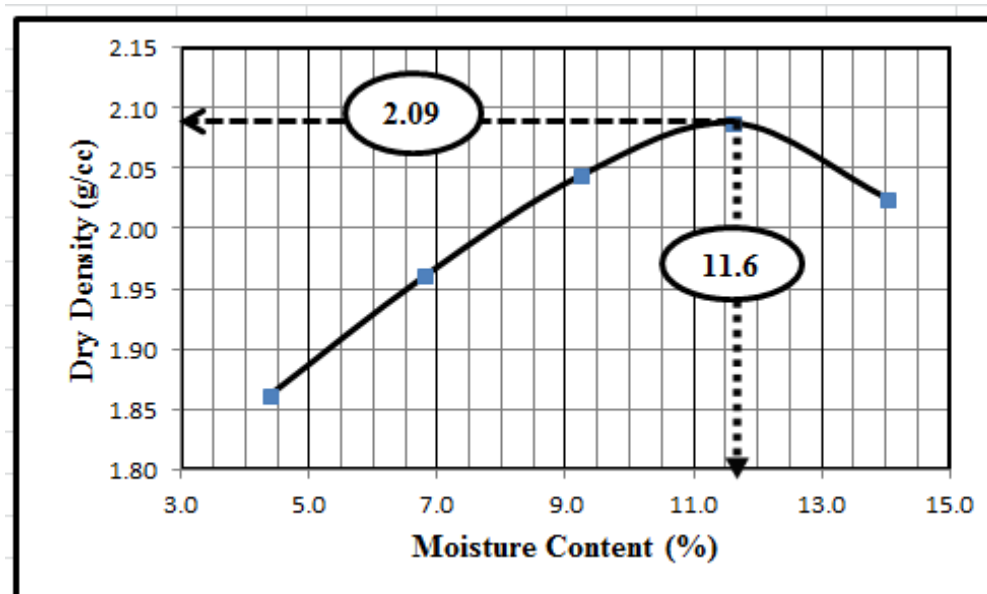


Table E-2.14 Result of modified proctor test for 60% CSA+8% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4670	4820	4970	5060	5040
Weight of Mould (g)	2895	2895	2895	2895	2895
Weight of Wet soil (g)	1775	1925	2075	2165	2145
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.91	2.07	2.23	2.33	2.31
<b>Moisture Content Determination</b>					
Weight of Wet soil + cont. (g)	298.5	296.4	328.2	264.6	218.2
Weight of Dry soil + cont. (g)	284.8	277.8	301.4	238.3	193.2
Weight of Container (g)	38.2	45.5	41.5	31.3	27.6
Weight of water (moisture) (g)	13.7	18.6	26.8	26.3	25.0
Weight of Dry soil (g)	246.6	232.3	259.9	207.0	165.6
Moisture content (%)	5.56	8.01	10.31	12.71	15.10
Dry Density (g/cc)	1.81	1.92	2.02	2.07	2.01
	<b>MDD, g/cc</b>		<b>2.07</b>	<b>OMC, %</b>	<b>12.7</b>

Figure E-2.14 Relationship between moisture and density for 60% CSA + 8% lime

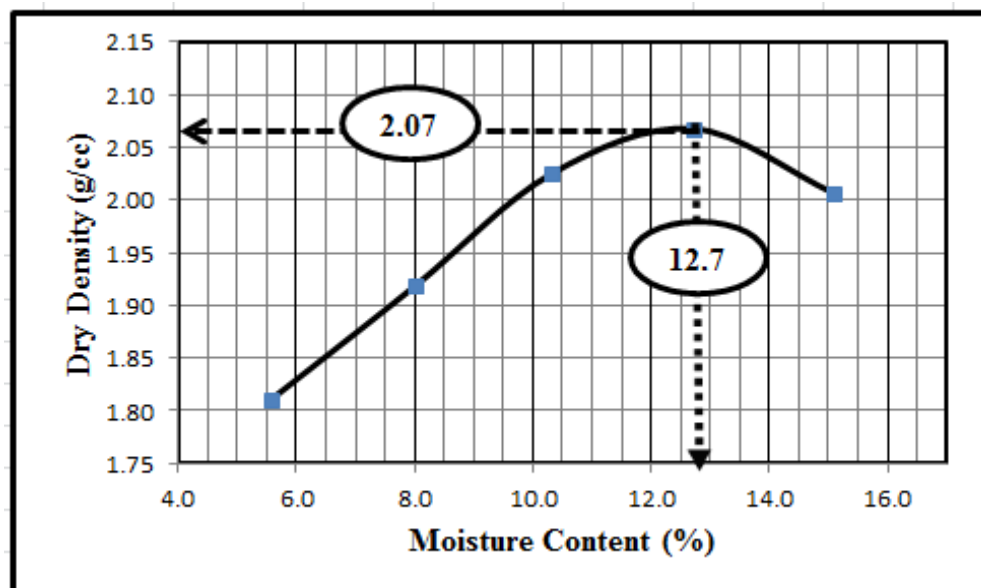


Table E-2.15 Result of modified proctor test for 60% CSA+12% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4700	4850	4990	5090	5060
Weight of Mould (g)	2940	2940	2940	2940	2940
Weight of Wet soil (g)	1760	1910	2050	2150	2120
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.89	2.06	2.21	2.31	2.28
Moisture Content Determination					
Weight of Wet soil + cont. (g)	249.4	290.3	256.2	216.6	207.2
Weight of Dry soil + cont. (g)	236.6	269.6	233.9	194.4	183.5
Weight of Container (g)	36.9	34.5	34.3	31.3	35.6
Weight of water (moisture) (g)	12.8	20.7	22.3	22.2	23.7
Weight of Dry soil (g)	199.7	235.1	199.6	163.1	147.9
Moisture content (%)	6.41	8.80	11.17	13.61	16.02
Dry Density (g/cc)	1.78	1.89	1.98	2.04	1.97
	<b>MDD, g/cc</b>		<b>2.04</b>	<b>OMC, %</b>	<b>13.6</b>

Figure E-2.15 Relationship between moisture and density for 60% CSA + 12% lime

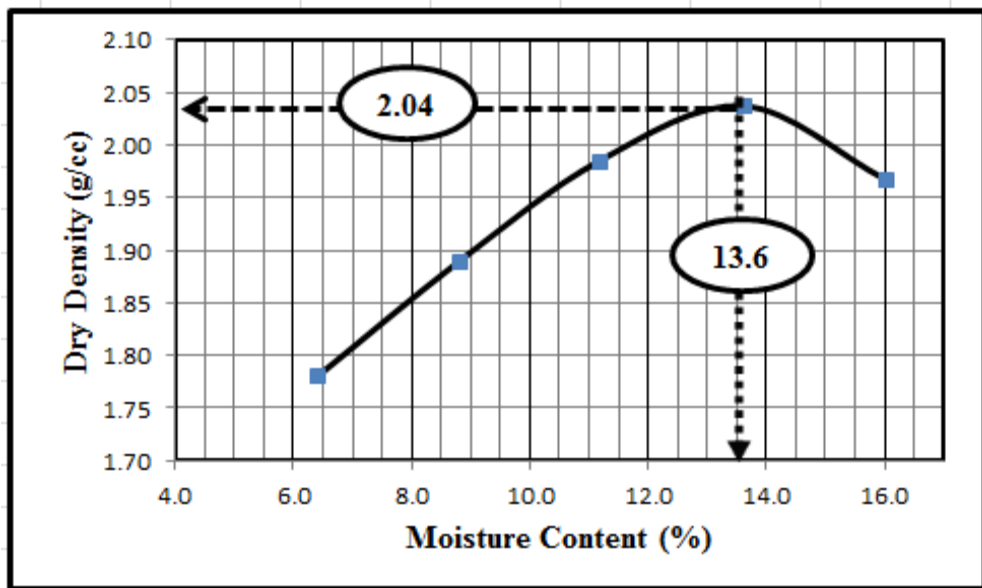
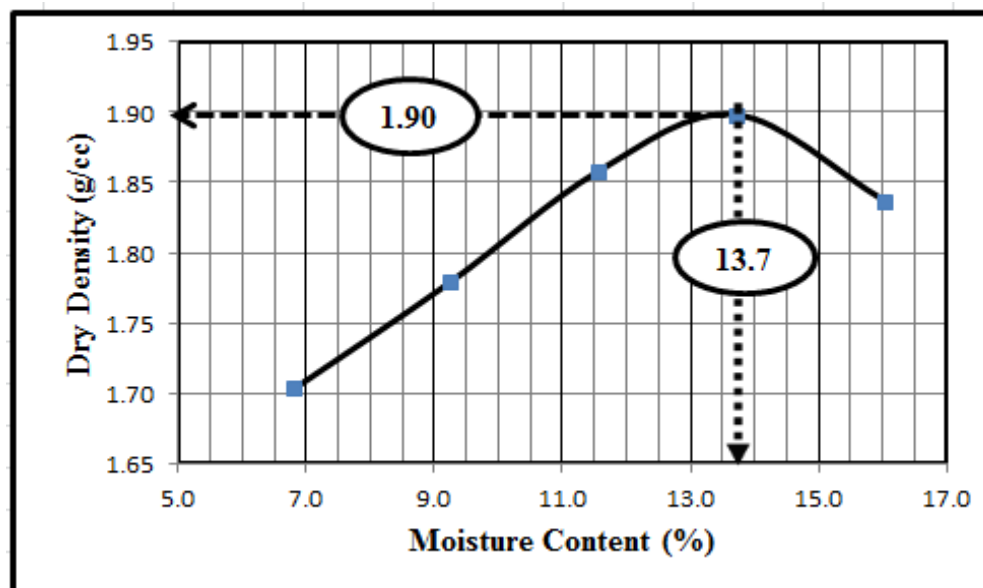


Table E-2.16 Result of modified proctor test for 60% CSA+16% lime

Trial No.	1	2	3	4	5
Weight of Mould + Wet soil (g)	4585	4700	4820	4900	4875
Weight of Mould (g)	2895	2895	2895	2895	2895
Weight of Wet soil (g)	1690	1805	1925	2005	1980
Volume of Mould (cc)	929	929	929	929	929
Wet density (g/cc)	1.82	1.94	2.07	2.16	2.13
Moisture Content Determination					
Weight of Wet soil + cont. (g)	240.6	281.1	339.0	396.9	454.8
Weight of Dry soil + cont. (g)	227.5	260.8	308.4	354.6	398.6
Weight of Container (g)	35.1	41.0	43.4	45.8	48.2
Weight of water (moisture) (g)	13.1	20.3	30.6	42.3	56.2
Weight of Dry soil (g)	192.4	219.8	265.0	308.8	350.4
Moisture content (%)	6.81	9.24	11.55	13.70	16.04
Dry Density (g/cc)	1.70	1.78	1.86	1.90	1.84
	<b>MDD, g/cc</b>		<b>1.90</b>	<b>OMC, %</b>	<b>13.7</b>

Figure E-2.16 Relationship between moisture and density for 60% CSA + 16% lime



**APPENDIX E-3 California Bearing Ratio Test**

Table E-3.1 Result of CBR test for 0% CSA+4% lime [0 days of curing + 4 days of soaking]

Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	10820	11020
Wt.of mould, g	$W_2$	6480	6480
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4340	4540
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.04	2.14
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.74	1.77
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	277.2	307.3
Wt. of dry sample + cont., g	$W_4$	240.6	260.2
Wt. of water, g	$W_5 = W_3 - W_4$	36.6	47.1
Wt. of container, g	$W_6$	33.3	31.3
Wt. of dry sample, g	$W_7 = W_4 - W_6$	207.3	228.9
% Moisture Content	$W_8 = W_5/W_7*100$	17.7	20.6

CBR Data, Bottom Face Penetration						SWELL DATA		
Penetration (mm)	Std load (KN)	Gauge reading	Load KN	Corrected CBR		Gauge Reading	Initial	Final
				KN	%			
0		0	0				276	420
0.64		66	1.59			Swell	mm	%
1.27		92	2.22				1.44	1.24
1.91		114	2.75					
2.54	13	145	3.50	3.50	<b>26.9</b>	Soaked CBR, %		<b>26.9</b>
5.08	20	175	4.23	4.23	<b>21.1</b>	Swell, %		<b>1.24</b>
6.35		202	4.88			Dry Density, g/cc		<b>1.74</b>
7.62		230	5.55					

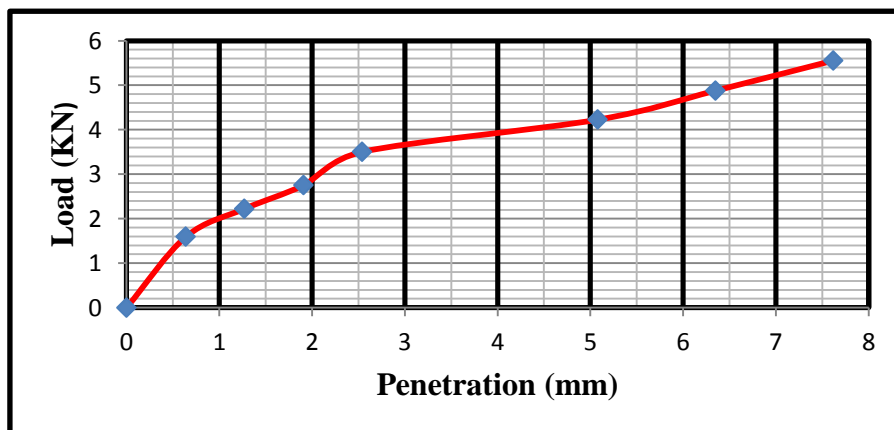


Figure E-3.1 Penetration-Load curve for 0% CSA+4% lime [0 days of curing + 4 days of soaking]

Table E-3.2 Result of CBR test for 0% CSA+4% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt. of wet sample + mould, g	$W_1$	10730	11144
Wt. of mould, g	$W_2$	6375	6375
Wt. of wet sample, g	$W_3 = W_1 - W_2$	4355	4769
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.05	2.25
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.74	1.88
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	322.8	317.2
Wt. of dry sample + cont., g	$W_4$	279.6	271.8
Wt. of water, g	$W_5 = W_3 - W_4$	43.2	45.4
Wt. of container, g	$W_6$	35.1	35.4
Wt. of dry sample, g	$W_7 = W_4 - W_6$	244.5	236.4
% Moisture Content	$W_8 = W_5/W_7*100$	17.7	19.2

CBR Data, Bottom Face Penetration						SWELL DATA		
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				315	398
0.64		138	3.33			Swell	mm	%
1.27		164	3.96				0.83	0.72
1.91		186	4.49					
2.54	13	217	5.24	5.24	<b>40.3</b>	Soaked CBR, %		<b>40.3</b>
5.08	20	247	5.97	5.97	<b>29.8</b>	Swell, %		<b>0.72</b>
6.35		274	6.62			Dry Density, g/cc		<b>1.74</b>
7.62		302	7.29					

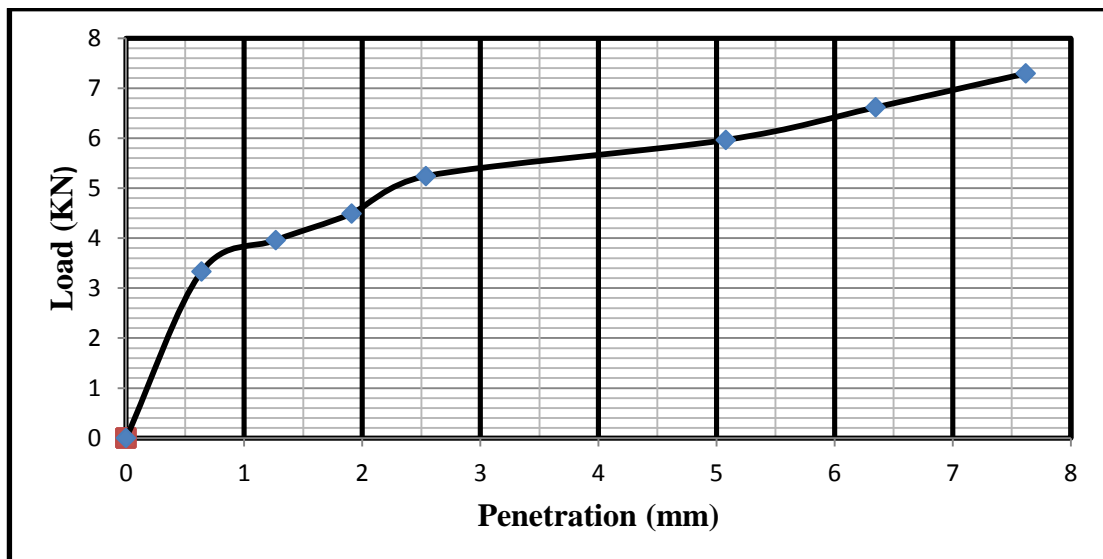


Figure E-3.2 Penetration-Load curve for 0% CSA+4% lime [7 days of curing + 4 days of soaking]

Table E-3.3 Result of CBR test for 0% CSA+8% lime [0 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	10230	10345
Wt.of mould, g	$W_2$	5900	5900
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4330	4445
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.04	2.09
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.73	1.71
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	313.3	214.6
Wt. of dry sample + cont., g	$W_4$	270.9	181.2
Wt. of water, g	$W_5 = W_3 - W_4$	42.4	33.4
Wt. of container, g	$W_6$	34.2	30.8
Wt. of dry sample, g	$W_7 = W_4 - W_6$	236.7	150.4
% Moisture Content	$W_8 = W_5/W_7*100$	17.9	22.2

CBR Data, Bottom Face Penetration						SWELL DATA		
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				298	451
0.64		71	1.71			Swell	mm	%
1.27		96	2.32				1.53	1.32
1.91		117	2.83					
2.54	13	148	3.57	3.57	<b>27.5</b>	Soaked CBR, %		<b>27.5</b>
5.08	20	182	4.40	4.40	<b>22.0</b>	Swell, %		<b>1.32</b>
6.35		212	5.12			Dry Density, g/cc		<b>1.73</b>
7.62		239	5.77					

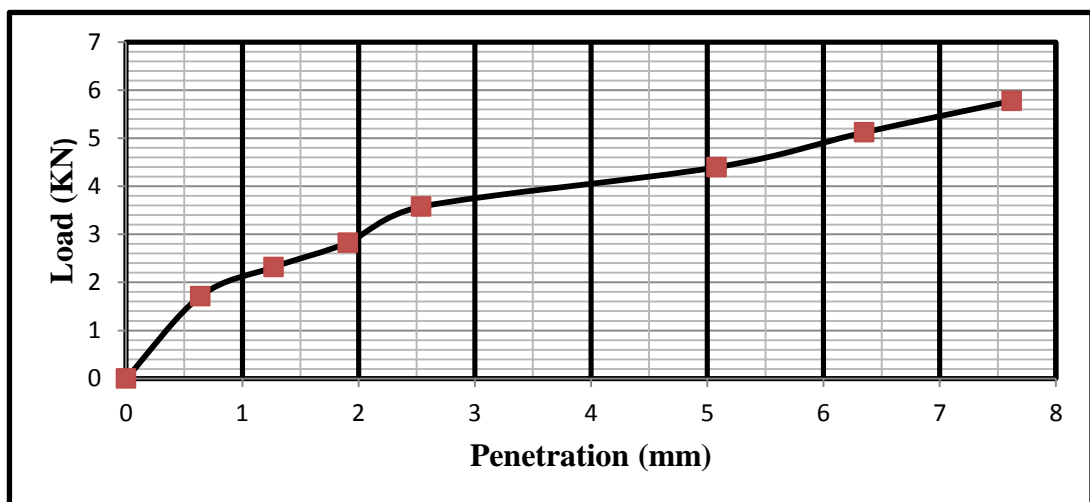


Figure E-3.3 Penetration-Load curve for 0% CSA+8% lime [0 days of curing + 4 days of soaking]

Table E-3.4 Result of CBR test for 0% CSA+8% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	10315	10430
Wt.of mould, g	$W_2$	5995	5995
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4320	4435
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.03	2.09
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.73	1.70
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	388.6	306.1
Wt. of dry sample + cont., g	$W_4$	335.1	254.9
Wt. of water, g	$W_5 = W_3 - W_4$	53.5	51.2
Wt. of container, g	$W_6$	36	33.3
Wt. of dry sample, g	$W_7 = W_4 - W_6$	299.1	221.6
% Moisture Content	$W_8 = W_5/W_7*100$	17.9	23.1

CBR Data, Bottom Face Penetration						SWELL DATA		
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				675	746
0.64		154	3.72			Swell	mm	%
1.27		179	4.32				0.71	0.61
1.91		200	4.83					
2.54	13	231	5.58	5.58	<b>42.9</b>	Soaked CBR, %		<b>42.9</b>
5.08	20	265	6.40	6.40	<b>32.0</b>	Swell, %		<b>0.61</b>
6.35		295	7.12			Dry Density, g/cc		<b>1.73</b>
7.62		322	7.78					

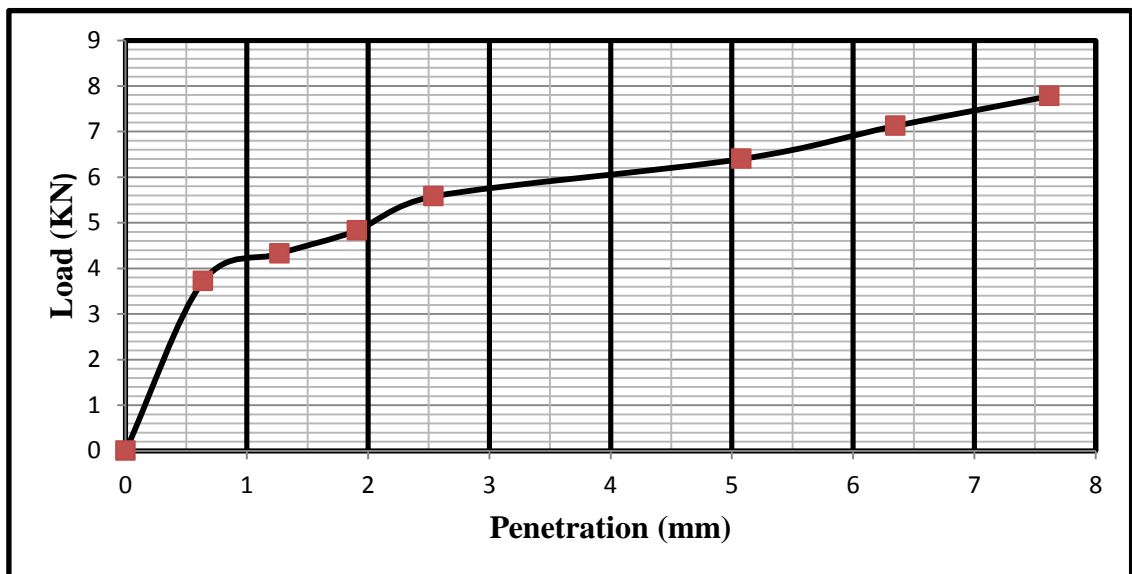


Figure E-3.4 Penetration-Load curve for 0% CSA+8% lime [7 days of curing + 4 days of soaking]

Table E-3.5 Result of CBR test for 0% CSA+12% lime [0 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	10575	10845
Wt.of mould, g	$W_2$	6260	6260
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4315	4585
Volume of mould, cc	V	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.03	2.16
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.72	1.76
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	207.8	216.6
Wt. of dry sample + cont., g	$W_4$	180.8	182.2
Wt. of water, g	$W_5 = W_3 - W_4$	27.0	34.4
Wt. of container, g	$W_6$	31.3	30.7
Wt. of dry sample, g	$W_7 = W_4 - W_6$	149.5	151.5
% Moisture Content	$W_8 = W_5/W_7*100$	18.1	22.7

CBR Data, Bottom Face Penetration					SWELL DATA			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				713	863
0.64		91	2.20			Swell	mm	%
1.27		107	2.58				1.5	1.29
1.91		128	3.09					
2.54	13	156	3.77	3.77	<b>29.0</b>	Soaked CBR, %	<b>29.0</b>	
5.08	20	202	4.88	4.88	<b>24.4</b>	Swell, %	<b>1.29</b>	
6.35		225	5.43			Dry Density, g/cc	<b>1.72</b>	
7.62		251	6.06					

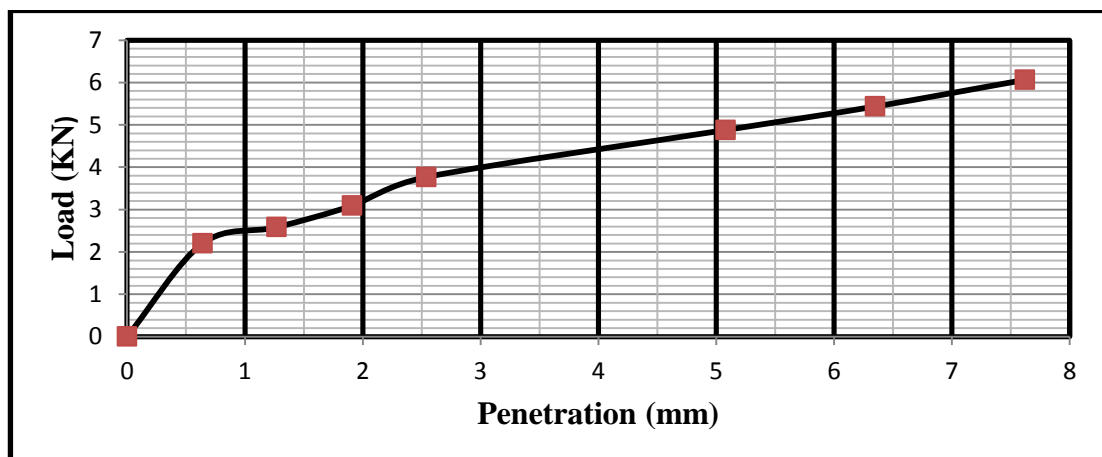


Figure E-3.5 Penetration-Load curve for 0% CSA+12% lime [0 days of curing + 4 days of soaking]

Table E-3.6 Result of CBR test for 0% CSA+12% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	10195	10380
Wt.of mould, g	$W_2$	5895	5895
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4300	4485
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.03	2.11
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.72	1.77
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	354.8	280.1
Wt. of dry sample + cont., g	$W_4$	306.1	240.2
Wt. of water, g	$W_5 = W_3 - W_4$	48.7	39.9
Wt. of container, g	$W_6$	36.8	33.4
Wt. of dry sample, g	$W_7 = W_4 - W_6$	269.3	206.8
% Moisture Content	$W_8 = W_5/W_7*100$	18.1	19.3

CBR Data, Bottom Face Penetration						SWELL DATA		
Penetration (mm)	Std load (KN)	Gauge reading	Load KN	Corrected CBR		Gauge Reading	Initial	Final
				KN	%		mm	%
0		0	0				513	578
0.64		185	4.47			Swell	mm	%
1.27		201	4.85				0.65	0.56
1.91		222	5.36					
2.54	13	250	6.04	6.04	<b>46.4</b>	Soaked CBR, %		<b>46.4</b>
5.08	20	296	7.15	7.15	<b>35.7</b>	Swell, %		<b>0.56</b>
6.35		319	7.70			Dry Density, g/cc		<b>1.72</b>
7.62		345	8.33					

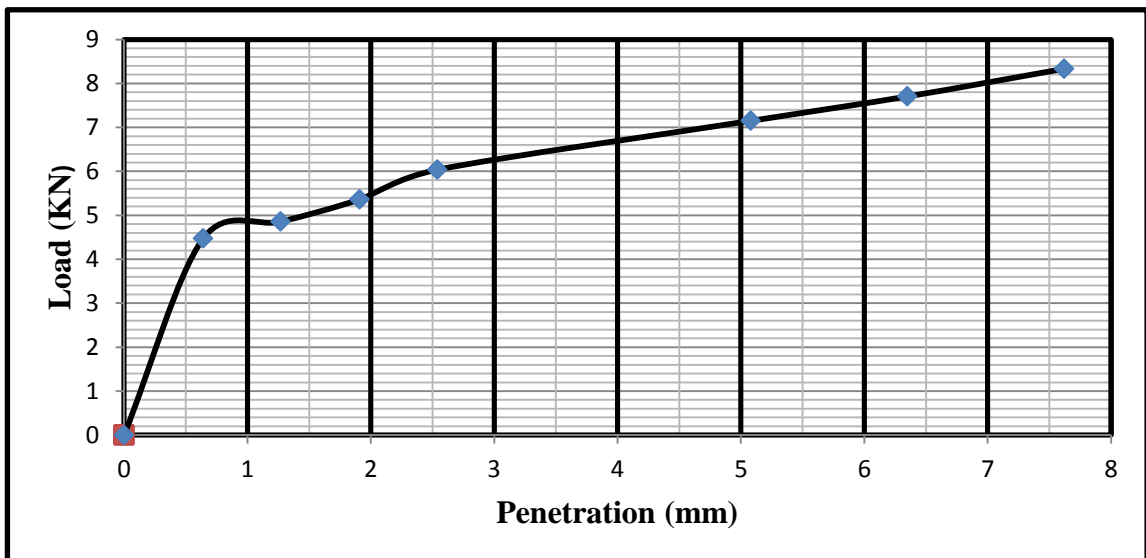


Figure E-3.6 Penetration-Load curve for 0% CSA+12% lime [7 days of curing + 4 days of soaking]

Table E-3.7 Result of CBR test for 0% CSA+16% lime [0 days of curing + 4 days of soaking]

<b>CBR TEST, 56 Blows, 5 Layers</b>			
<b>Unit Weight Determination</b>			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	10190	10340
Wt.of mould, g	$W_2$	5880	5880
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4310	4460
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.03	2.10
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.71	1.73
<b>Moisture Content Determination</b>			
Wt. of wet sample + cont., g	$W_3$	204.6	218.8
Wt. of dry sample + cont., g	$W_4$	177.2	186.8
Wt. of water, g	$W_5 = W_3 - W_4$	27.4	32
Wt. of container, g	$W_6$	33.3	37.3
Wt. of dry sample, g	$W_7 = W_4 - W_6$	143.9	149.5
% Moisture Content	$W_8 = W_5/W_7*100$	19.0	21.4

<b>CBR Data, Bottom Face Penetration</b>					<b>SWELL DATA</b>			
Penetration (mm)	Std load (KN)	Gauge reading	Load KN	Corrected CBR		Gauge Reading	Initial	Final
				KN	%			
0		0	0				519	668
0.64		101	2.44			Swell	mm	%
1.27		126	3.04				1.49	1.28
1.91		142	3.43					
2.54	13	170	4.11	4.11	<b>31.6</b>	Soaked CBR, %		<b>31.6</b>
5.08	20	213	5.14	5.14	<b>25.7</b>	Swell, %		<b>1.28</b>
6.35		251	6.06			Dry Density, g/cc		<b>1.71</b>
7.62		288	6.96					

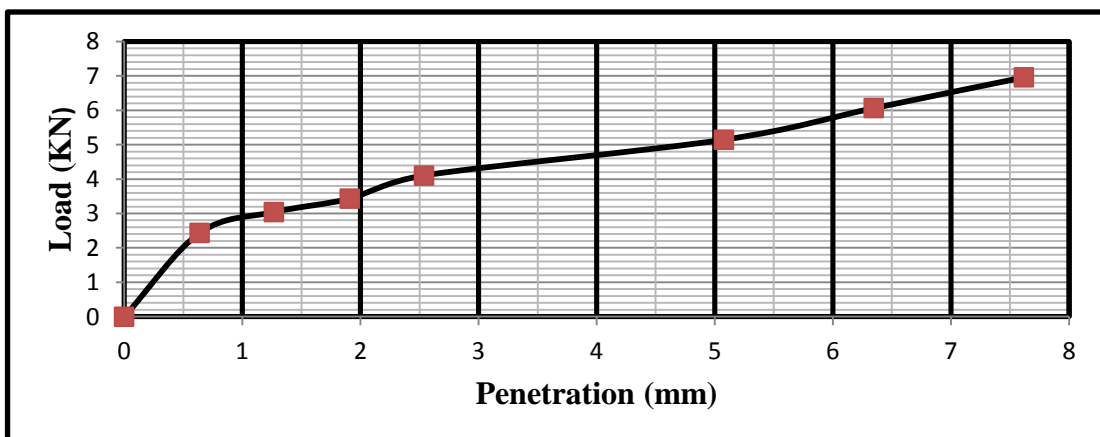


Figure E-3.7 Penetration-Load curve for 0% CSA+16% lime [0 days of curing + 4 days of soaking]

Table E-3.8 Result of CBR test for 0% CSA+ 16% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt. of wet sample + mould, g	$W_1$	10115	10515
Wt. of mould, g	$W_2$	5785	5785
Wt. of wet sample, g	$W_3 = W_1 - W_2$	4330	4730
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.04	2.23
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.71	1.74
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	336.2	292.3
Wt. of dry sample + cont., g	$W_4$	288.1	236
Wt. of water, g	$W_5 = W_3 - W_4$	48.1	56.3
Wt. of container, g	$W_6$	34.3	36.7
Wt. of dry sample, g	$W_7 = W_4 - W_6$	253.8	199.3
% Moisture Content	$W_8 = W_5/W_7*100$	19.0	28.2

CBR Data, Bottom Face Penetration						SWELL DATA		
Penetration (mm)	Std load (KN)	Gauge reading	Load KN	Corrected CBR		Gauge Reading	Initial	Final
				KN	%			
0		0	0				241	295
0.64		207	5.00			Swell	mm	%
1.27		232	5.60				0.54	0.47
1.91		248	5.99					
2.54	13	276	6.67	6.67	<b>51.3</b>	Soaked CBR, %		<b>51.3</b>
5.08	20	319	7.70	7.70	<b>38.5</b>	Swell, %		<b>0.47</b>
6.35		357	8.62			Dry Density, g/cc		<b>1.71</b>
7.62		394	9.52					

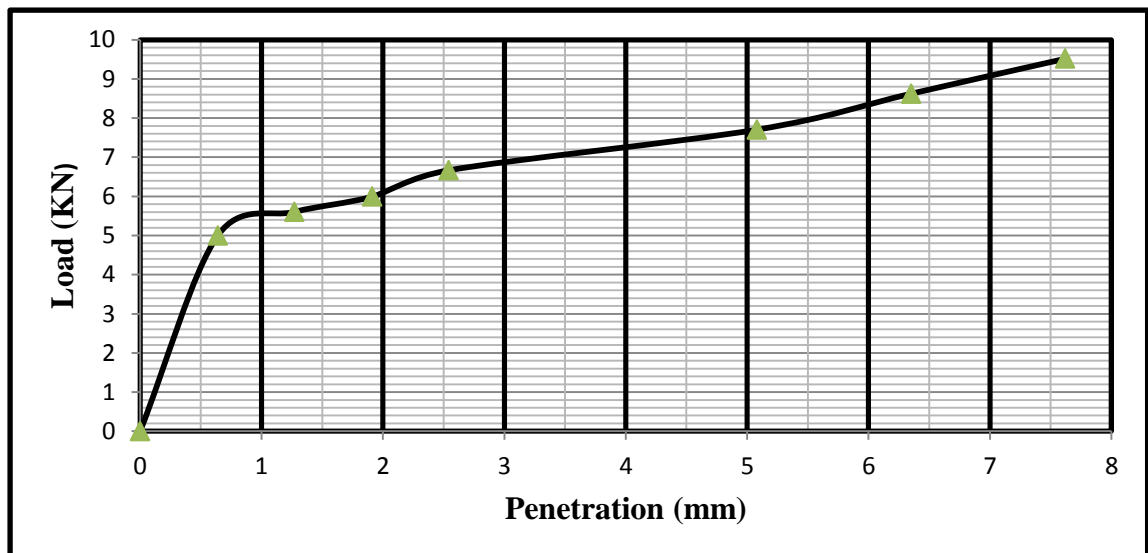


Figure E-3.8 Penetration-Load curve for 0% CSA+ 16% lime [7 days of curing + 4 days of soaking]

Table E-3.9 Result of CBR test for 20% CSA+ 4% lime [0 days of curing + 4 days of soaking]

<b>CBR TEST, 56 Blows, 5 Layers</b>			
<b>Unit Weight Determination</b>			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	11023	11170
Wt.of mould, g	$W_2$	6320	6320
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4703	4850
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.22	2.28
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.88	1.85
<b>Moisture Content Determination</b>			
Wt. of wet sample + cont., g	$W_3$	251.4	224.8
Wt. of dry sample + cont., g	$W_4$	218.5	188.5
Wt. of water, g	$W_5 = W_3 - W_4$	32.9	36.3
Wt. of container, g	$W_6$	31.3	33.7
Wt. of dry sample, g	$W_7 = W_4 - W_6$	187.2	154.8
% Moisture Content	$W_8 = W_5/W_7*100$	17.6	23.4

<b>CBR Data, Bottom Face Penetration</b>						<b>SWELL DATA</b>		
Penetration (mm)	Std load (KN)	Gauge reading	Load KN	Corrected CBR		Gauge Reading	Initial	Final
				KN	%			
0		0	0				704	850
0.64		122	2.95			Swell	mm	%
1.27		175	4.23				1.46	1.26
1.91		227	5.48					
2.54	13	275	6.64	6.64	<b>51.1</b>	Soaked CBR, %		<b>51.1</b>
5.08	20	362	8.74	8.74	<b>43.7</b>	Swell, %		<b>1.26</b>
6.35		400	9.66			Dry Density, g/cc		<b>1.88</b>
7.62		472	11.40					

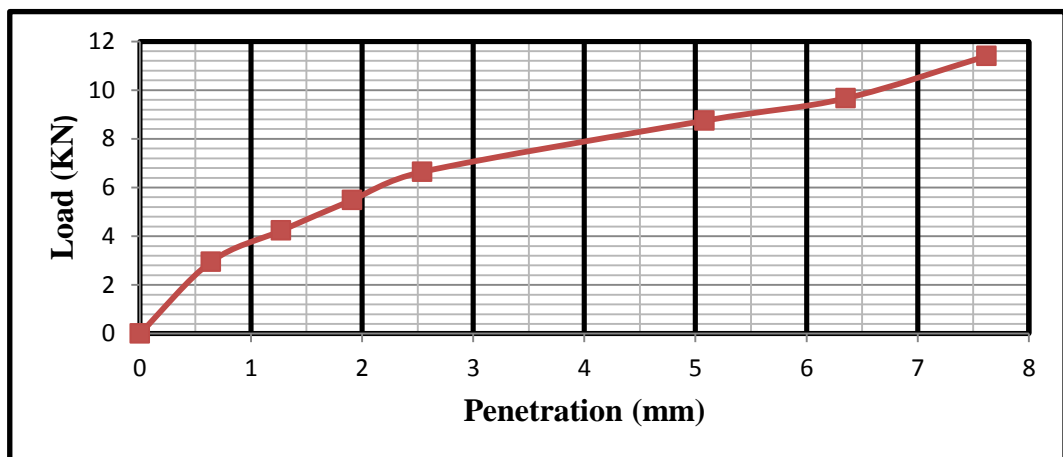


Figure E-3.9 Penetration-Load curve for 20% CSA+ 4% lime [0 days of curing + 4 days of soaking]

Table E-3.10 Result of CBR test for 20% CSA+ 4% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt. of wet sample + mould, g	$W_1$	10780	10970
Wt. of mould, g	$W_2$	6095	6095
Wt. of wet sample, g	$W_3 = W_1 - W_2$	4685	4875
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.21	2.30
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.88	2.02
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	358.2	267
Wt. of dry sample + cont., g	$W_4$	311.2	239
Wt. of water, g	$W_5 = W_3 - W_4$	47	28
Wt. of container, g	$W_6$	43.4	34.3
Wt. of dry sample, g	$W_7 = W_4 - W_6$	267.8	204.7
% Moisture Content	$W_8 = W_5 / W_7 * 100$	17.6	13.7

CBR Data, Bottom Face Penetration						SWELL DATA		
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				262	406
0.64		185	4.47			Swell	mm	%
1.27		238	5.75				1.44	1.24
1.91		290	7.00					
2.54	13	338	8.16	8.16	<b>62.8</b>	Soaked CBR, %		<b>62.8</b>
5.08	20	425	10.26	10.26	<b>51.3</b>	Swell, %		<b>1.24</b>
6.35		463	11.18			Dry Density, g/cc		<b>1.88</b>
7.62		535	12.92					

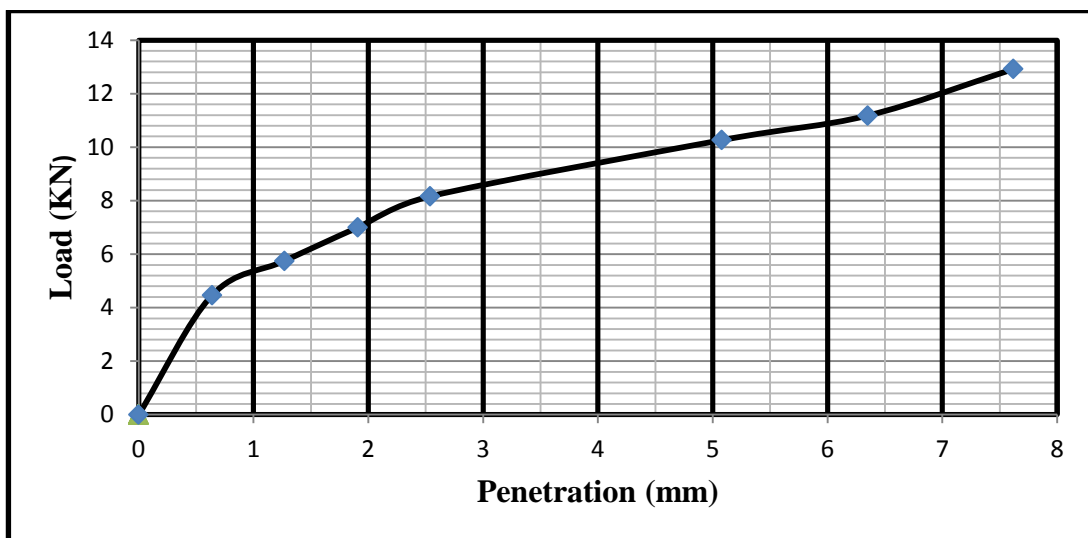


Figure E-3.10 Penetration-Load curve for 20% CSA+ 4% lime [7 days of curing + 4 days of soaking]

Table E-3.11 Result of CBR test for 20% CSA+ 8% lime [0 days of curing + 4 days of soaking]

<b>CBR TEST, 56 Blows, 5 Layers</b>			
<b>Unit Weight Determination</b>			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	10675	10785
Wt.of mould, g	$W_2$	6030	6030
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4645	4755
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.19	2.24
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.85	1.86
<b>Moisture Content Determination</b>			
Wt. of wet sample + cont., g	$W_3$	313.3	345.5
Wt. of dry sample + cont., g	$W_4$	270.5	292.2
Wt. of water, g	$W_5 = W_3 - W_4$	42.8	53.3
Wt. of container, g	$W_6$	34.3	31.6
Wt. of dry sample, g	$W_7 = W_4 - W_6$	236.2	260.6
% Moisture Content	$W_8 = W_5/W_7*100$	18.1	20.5

<b>CBR Data, Bottom Face Penetration</b>					<b>SWELL DATA</b>			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0			Swell	424	560
0.64		105	2.54				mm	%
1.27		224	5.41			1.36	1.17	
1.91		280	6.76					
2.54	13	330	7.97	7.97	<b>61.3</b>	Soaked CBR, %	<b>61.3</b>	
5.08	20	424	10.24	10.24	<b>51.2</b>	Swell, %	<b>1.17</b>	
6.35		475	11.47			Dry Density, g/cc	<b>1.85</b>	
7.62		535	12.92					

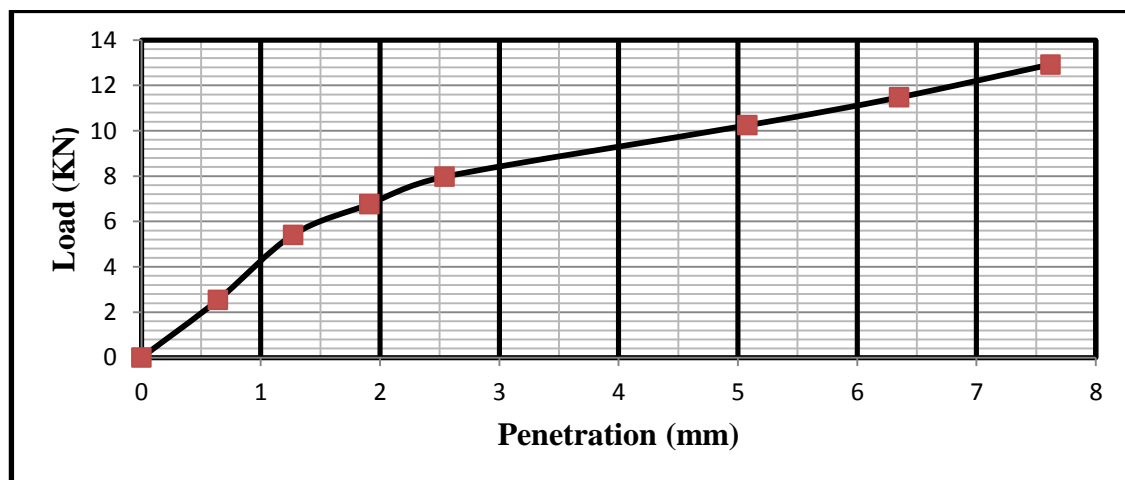


Figure E-3.11 Penetration-Load curve for 20% CSA+ 8% lime [0 days of curing + 4 days of soaking]

Table E-3.12 Result of CBR test for 20% CSA+ 8% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers					
Unit Weight Determination					
CONDITION OF SAMPLE		Before soaking	After soaking		
Wt. of wet sample + mould, g	$W_1$	10835	11215		
Wt. of mould, g	$W_2$	6190	6190		
Wt. of wet sample, g	$W_3 = W_1 - W_2$	4645	5025		
Volume of mould, cc	$V$	2123	2123		
Wet unit weight, g/cc	$D_w = W_3 / V$	2.19	2.37		
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.85	1.98		
Moisture Content Determination					
Wt. of wet sample + cont., g	$W_3$	383.5	309.7		
Wt. of dry sample + cont., g	$W_4$	331.2	264.8		
Wt. of water, g	$W_5 = W_3 - W_4$	52.3	44.9		
Wt. of container, g	$W_6$	41.6	35.7		
Wt. of dry sample, g	$W_7 = W_4 - W_6$	289.6	229.1		
% Moisture Content	$W_8 = W_5 / W_7 * 100$	18.1	19.6		

CBR Data, Bottom Face Penetration					SWELL DATA			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%		mm	%
0		0	0				326	380
0.64		176	4.25			Swell	mm	%
1.27		295	7.12				0.54	0.47
1.91		351	8.48					
2.54	13	401	9.68	9.68	<b>74.5</b>	Soaked CBR, %		<b>74.5</b>
5.08	20	495	11.95	11.95	<b>59.8</b>	Swell, %		<b>0.47</b>
6.35		546	13.19			Dry Density, g/cc		<b>1.85</b>
7.62		606	14.63					

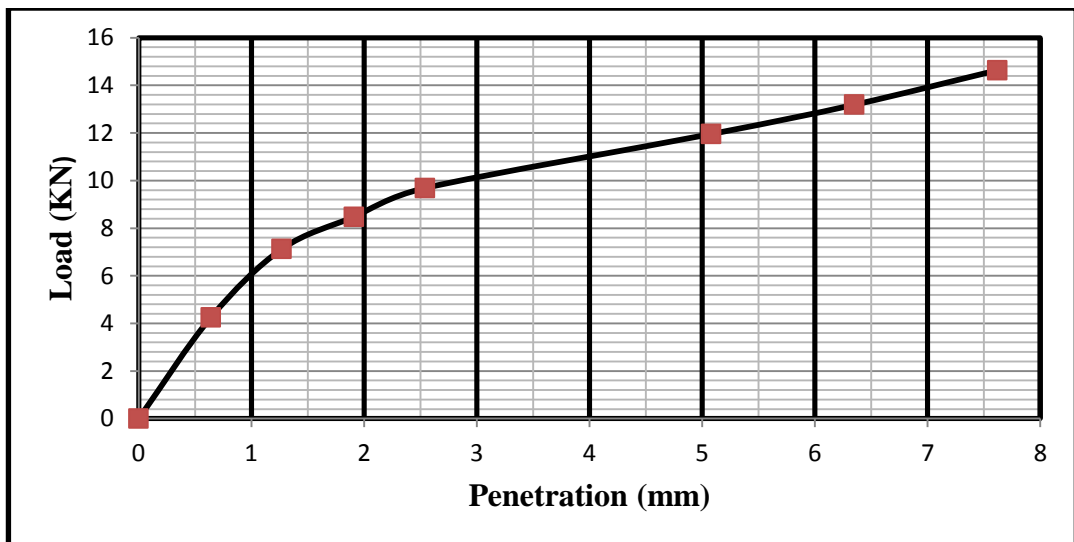


Figure E-3.12 Penetration-Load curve for 20% CSA+ 8% lime [7 days of curing + 4 days of soaking]

Table E-3.13 Result of CBR test for 20% CSA+ 12% lime [0 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt. of wet sample + mould, g	$W_1$	10330	10480
Wt. of mould, g	$W_2$	5700	5970
Wt. of wet sample, g	$W_3 = W_1 - W_2$	4630	4510
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.18	2.12
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.83	1.70
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	213.3	216.6
Wt. of dry sample + cont., g	$W_4$	184.5	181.2
Wt. of water, g	$W_5 = W_3 - W_4$	28.8	35.4
Wt. of container, g	$W_6$	34.2	37.3
Wt. of dry sample, g	$W_7 = W_4 - W_6$	150.3	143.9
% Moisture Content	$W_8 = W_5/W_7*100$	19.2	24.6

CBR Data, Bottom Face Penetration					SWELL DATA			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				313	458
0.64		168	4.06			Swell	mm	%
1.27		265	6.40				1.45	1.25
1.91		325	7.85					
2.54	13	375	9.06	9.06	<b>69.7</b>	Soaked CBR, %		<b>69.7</b>
5.08	20	451	10.89	10.89	<b>54.5</b>	Swell, %		<b>1.25</b>
6.35		508	12.27			Dry Density, g/cc		<b>1.83</b>
7.62		574	13.86					

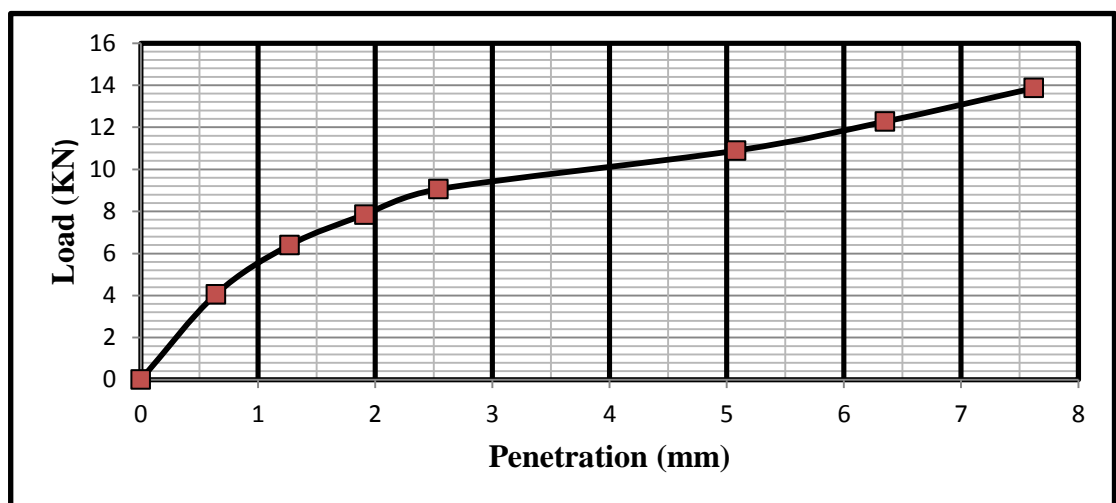


Figure E-3.13 Penetration-Load curve for 20% CSA+ 12% lime [0 days of curing + 4 days of soaking]

Table E-3.14 Result of CBR test for 20% CSA+ 12% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	10585	10750
Wt.of mould, g	$W_2$	5945	5755
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4640	4995
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.19	2.35
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.83	1.94
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	362.3	298
Wt. of dry sample + cont., g	$W_4$	311.2	252.2
Wt. of water, g	$W_5 = W_3 - W_4$	51.1	45.8
Wt. of container, g	$W_6$	45.3	36.9
Wt. of dry sample, g	$W_7 = W_4 - W_6$	265.9	215.3
% Moisture Content	$W_8 = W_5/W_7*100$	19.2	21.3

CBR Data, Bottom Face Penetration					SWELL DATA			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				276	320
0.64		241	5.82			Swell	mm	%
1.27		338	8.16				0.44	0.38
1.91		398	9.61					
2.54	13	448	10.82	10.82	83.2	Soaked CBR, %	83.2	
5.08	20	524	12.65	12.65	63.3	Swell, %	0.38	
6.35		581	14.03			Dry Density, g/cc	1.83	
7.62		647	15.63					

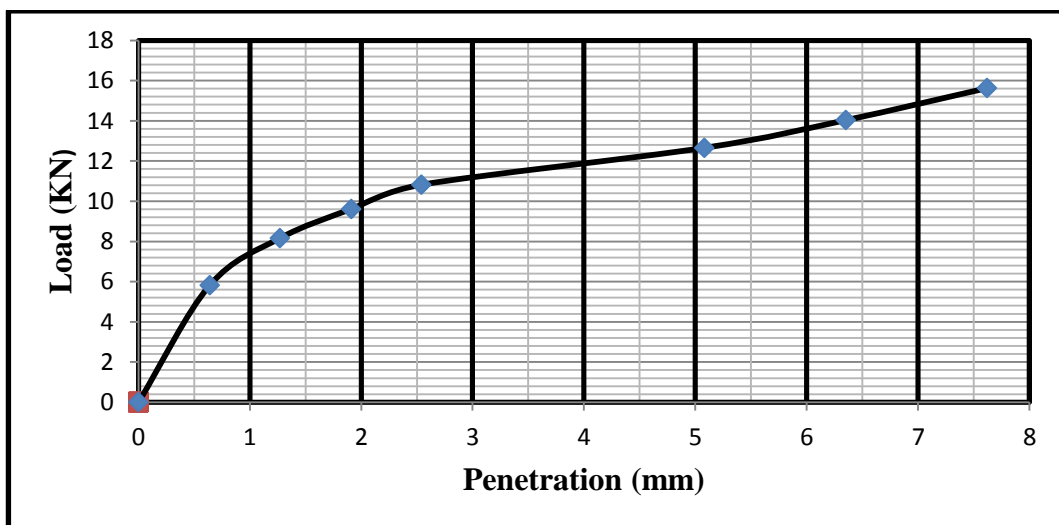


Figure E-3.14 Penetration-Load curve for 20% CSA+ 12% lime [7 days of curing + 4 days of soaking]

Table E-3.15 Result of CBR test for 20% CSA+ 16% lime [0 days of curing + 4 days of soaking]

<b>CBR TEST, 56 Blows, 5 Layers</b>			
<b>Unit Weight Determination</b>			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt. of wet sample + mould, g	$W_1$	10740	10850
Wt. of mould, g	$W_2$	6160	6160
Wt. of wet sample, g	$W_3 = W_1 - W_2$	4580	4690
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.16	2.21
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.79	1.76
<b>Moisture Content Determination</b>			
Wt. of wet sample + cont., g	$W_3$	246.8	237.3
Wt. of dry sample + cont., g	$W_4$	210.4	195.9
Wt. of water, g	$W_5 = W_3 - W_4$	36.4	41.4
Wt. of container, g	$W_6$	31.3	34.3
Wt. of dry sample, g	$W_7 = W_4 - W_6$	179.1	161.6
% Moisture Content	$W_8 = W_5 / W_7 * 100$	20.3	25.6

<b>CBR Data, Bottom Face Penetration</b>					<b>SWELL DATA</b>			
Penetration (mm)	Std load (KN)	Gauge reading	Load KN	Corrected CBR		Gauge Reading	Initial	Final
				KN	%		653	810
0		0	0				mm	%
0.64		265	6.40			Swell	1.57	1.35
1.27		313	7.56					
1.91		374	9.03					
2.54	13	418	10.09	10.09	<b>77.7</b>	Soaked CBR, %	<b>77.7</b>	
5.08	20	490	11.83	11.83	<b>59.2</b>	Swell, %	<b>1.35</b>	
6.35		546	13.19			Dry Density, g/cc	<b>1.79</b>	
7.62		602	14.54					

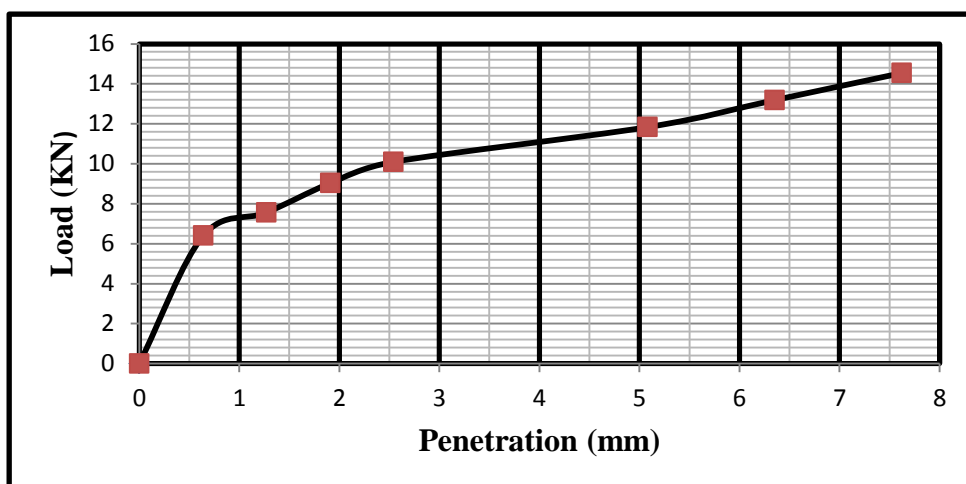


Figure E-3.15 Penetration-Load curve for 20% CSA+ 16% lime [0 days of curing + 4 days of soaking]

Table E-3.16 Result of CBR test for 20% CSA+ 16% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt. of wet sample + mould, g	$W_1$	10885	11220
Wt. of mould, g	$W_2$	6315	6315
Wt. of wet sample, g	$W_3 = W_1 - W_2$	4570	4905
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.15	2.31
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.79	1.91
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	339.6	279
Wt. of dry sample + cont., g	$W_4$	288.1	237
Wt. of water, g	$W_5 = W_3 - W_4$	51.5	42
Wt. of container, g	$W_6$	34.1	33.7
Wt. of dry sample, g	$W_7 = W_4 - W_6$	254	203.3
% Moisture Content	$W_8 = W_5/W_7*100$	20.3	20.7

CBR Data, Bottom Face Penetration					SWELL DATA			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				275	329
0.64		328	7.92			Swell	mm	%
1.27		376	9.08				0.54	0.47
1.91		437	10.55					
2.54	13	481	11.62	11.62	<b>89.4</b>	Soaked CBR, %		<b>89.4</b>
5.08	20	553	13.35	13.35	<b>66.8</b>	Swell, %		<b>0.47</b>
6.35		609	14.71			Dry Density, g/cc		<b>1.79</b>
7.62		665	16.06					

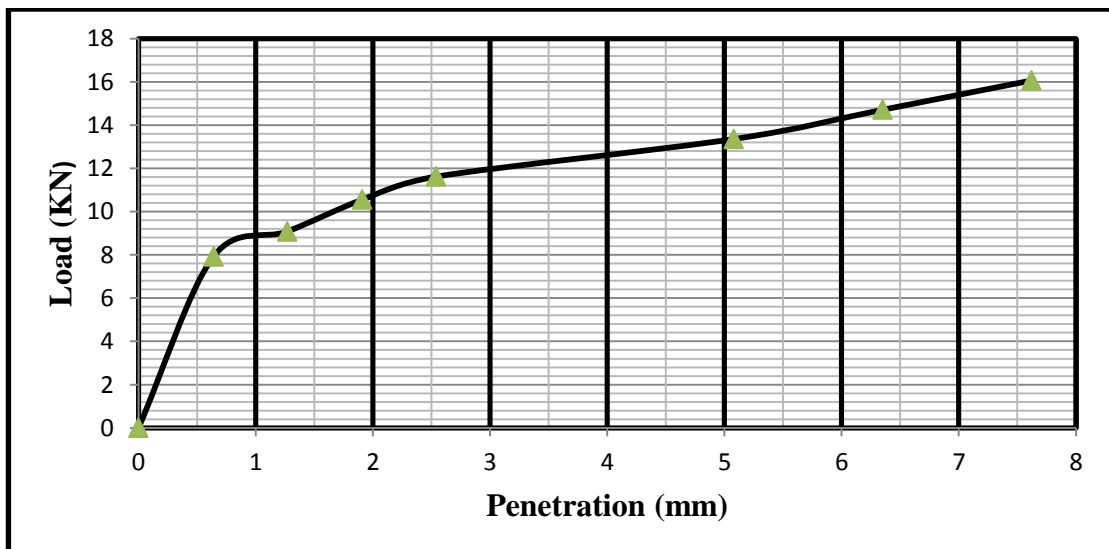


Figure E-3.16 Penetration-Load curve for 20% CSA+ 16% lime [7 days of curing + 4 days of soaking]

Table E-3.17 Result of CBR test for 40% CSA+ 4% lime [0 days of curing + 4 days of soaking]

<b>CBR TEST, 56 Blows, 5 Layers</b>			
<b>Unit Weight Determination</b>			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	10975	11090
Wt.of mould, g	$W_2$	6370	6370
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4605	4720
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.17	2.22
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.90	1.88
<b>Moisture Content Determination</b>			
Wt. of wet sample + cont., g	$W_3$	316.6	307.3
Wt. of dry sample + cont., g	$W_4$	281.2	264.6
Wt. of water, g	$W_5 = W_3 - W_4$	35.4	42.7
Wt. of container, g	$W_6$	31.3	34.3
Wt. of dry sample, g	$W_7 = W_4 - W_6$	249.9	230.3
% Moisture Content	$W_8 = W_5/W_7*100$	14.2	18.5

<b>CBR Data, Bottom Face Penetration</b>					<b>SWELL DATA</b>			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%		313	471
0		0	0				mm	%
0.64		164	3.96			Swell	1.58	1.36
1.27		227	5.48					
1.91		370	8.94					
2.54	13	430	10.38	10.38	<b>79.9</b>	Soaked CBR, %		<b>79.9</b>
5.08	20	490	11.83	11.83	<b>59.2</b>	Swell, %		<b>1.36</b>
6.35		545	13.16			Dry Density, g/cc		<b>1.90</b>
7.62		606	14.63					

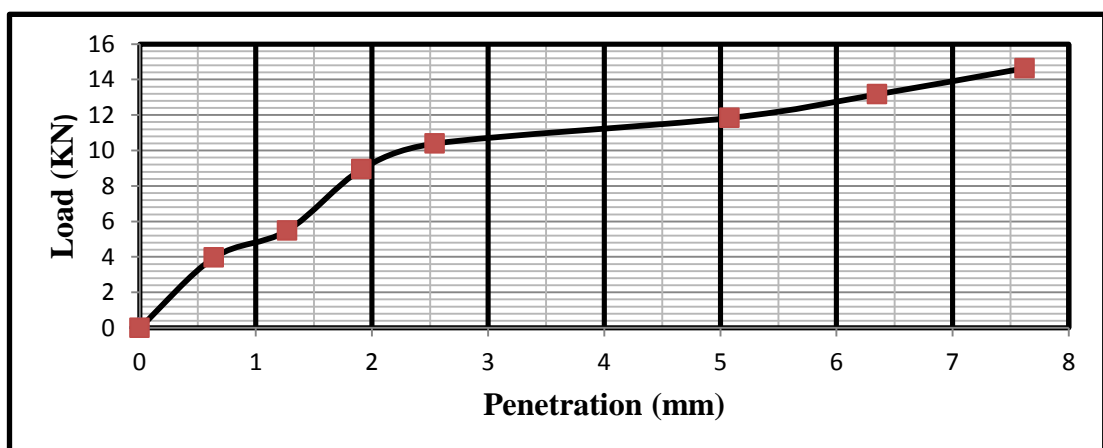


Figure E-3.17 Penetration-Load curve for 40% CSA+ 4% lime [0 days of curing + 4 days of soaking]

Table E-3.18 Result of CBR test for 40% CSA+ 4% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt. of wet sample + mould, g	$W_1$	10555	10980
Wt. of mould, g	$W_2$	5945	5945
Wt. of wet sample, g	$W_3 = W_1 - W_2$	4610	5035
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.17	2.37
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.90	2.04
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	345.9	283.4
Wt. of dry sample + cont., g	$W_4$	307.4	249
Wt. of water, g	$W_5 = W_3 - W_4$	38.5	34.4
Wt. of container, g	$W_6$	36.6	34.8
Wt. of dry sample, g	$W_7 = W_4 - W_6$	270.8	214.2
% Moisture Content	$W_8 = W_5/W_7*100$	14.2	16.1

CBR Data, Bottom Face Penetration					SWELL DATA			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				351	401
0.64		230	5.55			Swell	mm	%
1.27		293	7.08				0.5	0.43
1.91		436	10.53					
2.54	13	496	11.98	11.98	<b>92.1</b>	Soaked CBR, %	<b>92.1</b>	
5.08	20	556	13.43	13.43	<b>67.1</b>	Swell, %	<b>0.43</b>	
6.35		611	14.76			Dry Density, g/cc	<b>1.90</b>	
7.62		672	16.23					

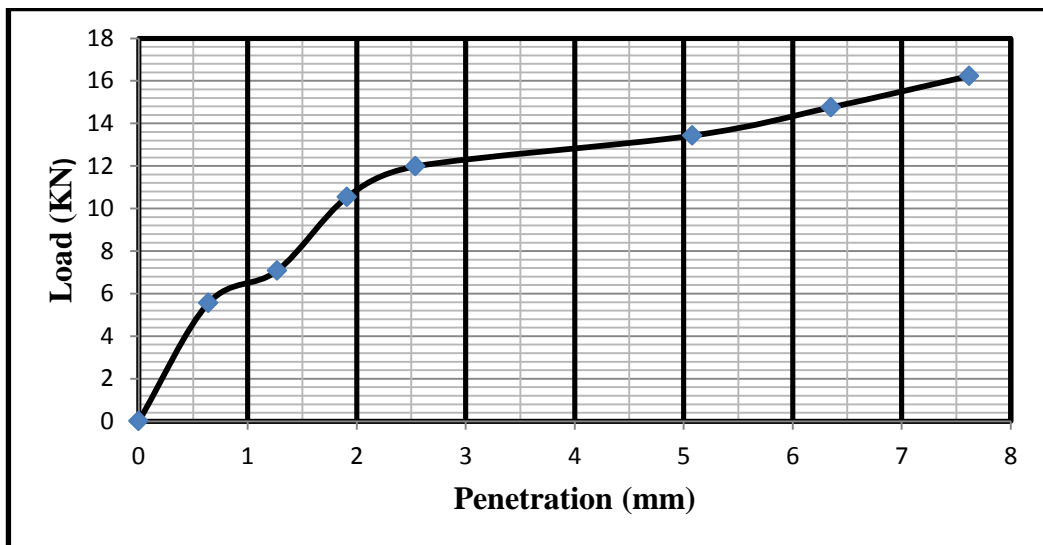


Figure E-3.18 Penetration-Load curve for 40% CSA+ 4% lime [7 days of curing + 4 days of soaking]

Table E-3.19 Result of CBR test for 40% CSA+ 8% lime [0 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	10790	11020
Wt.of mould, g	$W_2$	5990	5990
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4800	5030
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.26	2.37
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.96	2.00
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	216.6	281.2
Wt. of dry sample + cont., g	$W_4$	192.6	243.5
Wt. of water, g	$W_5 = W_3 - W_4$	24	37.7
Wt. of container, g	$W_6$	37.2	36.5
Wt. of dry sample, g	$W_7 = W_4 - W_6$	155.4	207
% Moisture Content	$W_8 = W_5/W_7*100$	15.4	18.2

CBR Data, Bottom Face Penetration						SWELL DATA		
Penetration (mm)	Std load (KN)	Gauge reading	Load KN	Corrected CBR		Gauge Reading	Initial	Final
				KN	%			
0		0	0				167	315
0.64		259	6.25			Swell	mm	%
1.27		327	7.90				1.48	1.28
1.91		402	9.71					
2.54	13	485	11.71	11.71	<b>90.1</b>	Soaked CBR, %		<b>90.1</b>
5.08	20	571	13.79	13.79	<b>68.9</b>	Swell, %		<b>1.28</b>
6.35		618	14.92			Dry Density, g/cc		<b>1.96</b>
7.62		697	16.83					

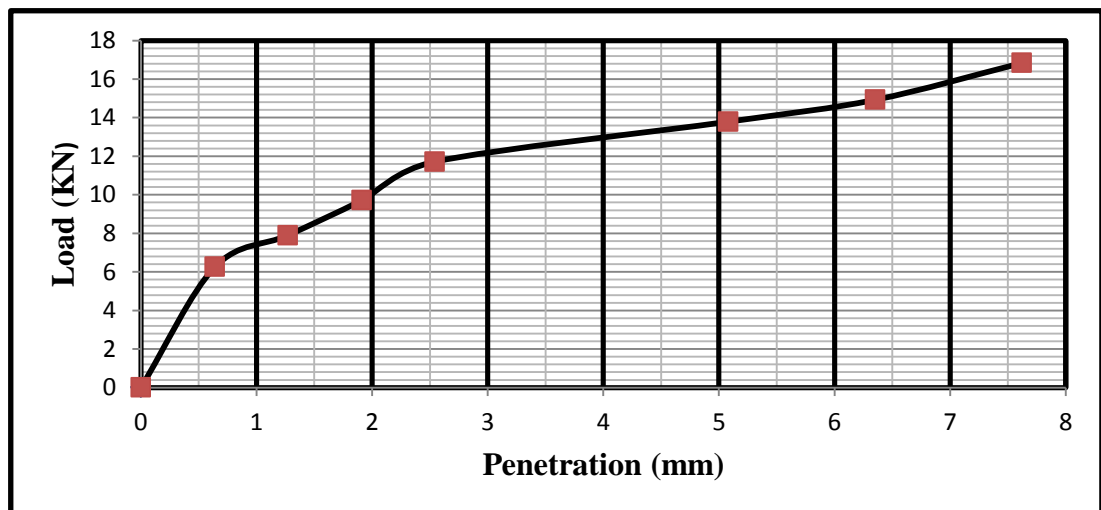


Figure E-3.19 Penetration-Load curve for 40% CSA+ 8% lime [0 days of curing + 4 days of soaking]

Table E-3.20 Result of CBR test for 40% CSA+ 8% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	11245	11450
Wt.of mould, g	$W_2$	6430	6430
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4815	5020
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.27	2.36
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.96	2.00
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	418.6	317.8
Wt. of dry sample + cont., g	$W_4$	368.8	275.7
Wt. of water, g	$W_5 = W_3 - W_4$	49.8	42.1
Wt. of container, g	$W_6$	46.1	43.8
Wt. of dry sample, g	$W_7 = W_4 - W_6$	322.7	231.9
% Moisture Content	$W_8 = W_5/W_7*100$	15.4	18.2

CBR Data, Bottom Face Penetration						SWELL DATA		
Penetration (mm)	Std load (KN)	Gauge reading	Load KN	Corrected CBR		Gauge Reading	Initial	Final
				KN	%			
0		0	0				275	247
0.64		320	7.73			Swell	mm	%
1.27		388	9.37				-0.28	-0.24
1.91		463	11.18					
2.54	13	546	13.19	13.19	<b>101.4</b>	Soaked CBR, %		<b>101.4</b>
5.08	20	632	15.26	15.26	<b>76.3</b>	Swell, %		<b>-0.24</b>
6.35		679	16.40			Dry Density, g/cc		<b>1.96</b>
7.62		758	18.31					

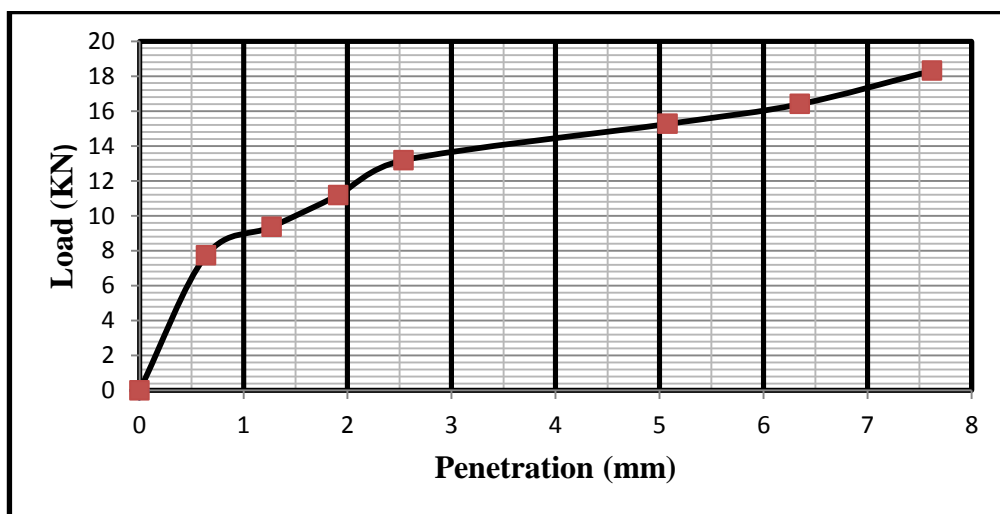


Figure E-3.20 Penetration-Load curve for 40% CSA+ 8% lime [7 days of curing + 4 days of soaking]

Table E-3.21 Result of CBR test for 40% CSA+ 12% lime [0 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt. of wet sample + mould, g	$W_1$	10480	10795
Wt. of mould, g	$W_2$	5700	5700
Wt. of wet sample, g	$W_3 = W_1 - W_2$	4780	5095
Volume of mould, cc	V	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.25	2.40
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.93	2.00
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	211.3	216.6
Wt. of dry sample + cont., g	$W_4$	185.8	186.6
Wt. of water, g	$W_5 = W_3 - W_4$	25.5	30
Wt. of container, g	$W_6$	31.3	35.3
Wt. of dry sample, g	$W_7 = W_4 - W_6$	154.5	151.3
% Moisture Content	$W_8 = W_5/W_7*100$	16.5	19.8

CBR Data, Bottom Face Penetration						SWELL DATA		
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				548	690
0.64		288	6.96			Swell	mm	%
1.27		391	9.44				1.42	1.22
1.91		451	10.89					
2.54	13	515	12.44	12.44	<b>95.7</b>	Soaked CBR, %	<b>95.7</b>	
5.08	20	600	14.49	14.49	<b>72.5</b>	Swell, %	<b>1.22</b>	
6.35		674	16.28			Dry Density, g/cc	<b>1.93</b>	
7.62		788	19.03					

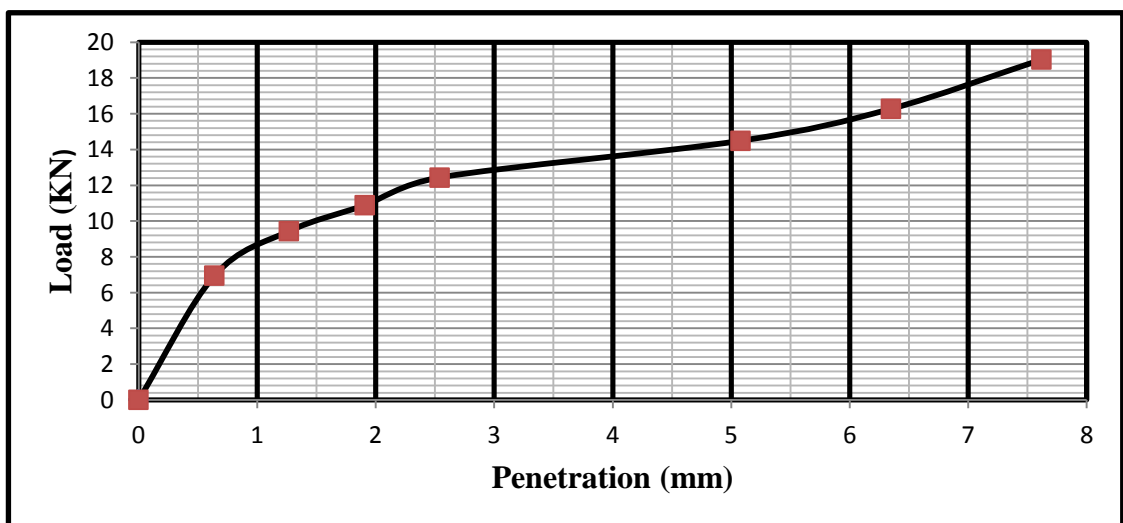


Figure E-3.21 Penetration-Load curve for 40% CSA+ 12% lime [0 days of curing + 4 days of soaking]

Table E-3.22 Result of CBR test for 40% CSA+ 12% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt. of wet sample + mould, g	$W_1$	10685	10790
Wt. of mould, g	$W_2$	5900	5900
Wt. of wet sample, g	$W_3 = W_1 - W_2$	4785	4890
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.25	2.30
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.93	1.92
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	241.2	283
Wt. of dry sample + cont., g	$W_4$	212.4	242
Wt. of water, g	$W_5 = W_3 - W_4$	28.8	41
Wt. of container, g	$W_6$	37.7	35.7
Wt. of dry sample, g	$W_7 = W_4 - W_6$	174.7	206.3
% Moisture Content	$W_8 = W_5 / W_7 * 100$	16.5	19.9

CBR Data, Bottom Face Penetration					SWELL DATA			
Penetration (mm)	Std load (KN)	Gauge reading	Load KN	Corrected CBR		Gauge Reading	Initial	Final
				KN	%			
0		0	0				527	576
0.64		347	8.38			Swell	mm	%
1.27		450	10.87				0.49	0.42
1.91		510	12.32					
2.54	13	574	13.86	13.86	<b>106.6</b>	Soaked CBR, %		<b>106.6</b>
5.08	20	659	15.91	15.91	<b>79.6</b>	Swell, %		<b>0.42</b>
6.35		733	17.70			Dry Density, g/cc		<b>1.93</b>
7.62		847	20.46					

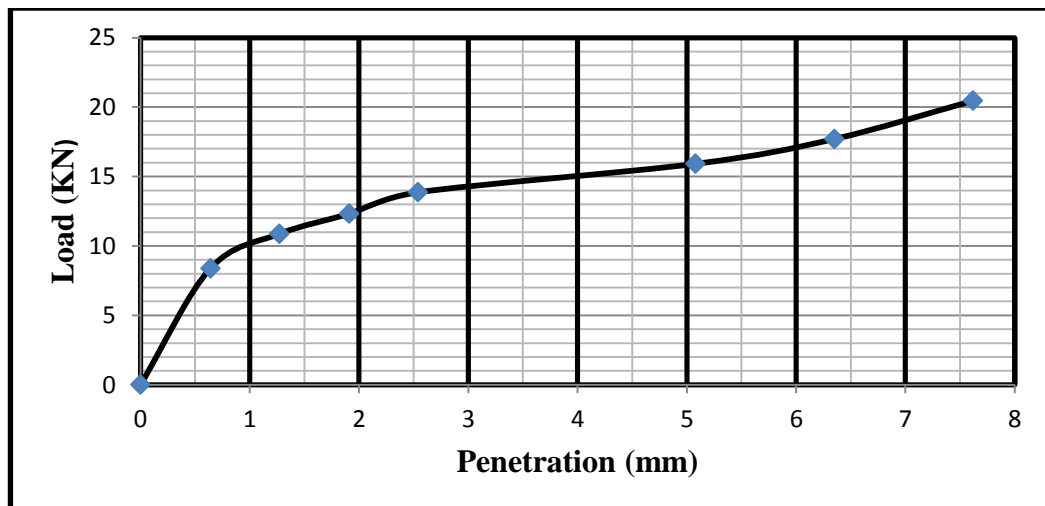


Figure E-3.22 Penetration-Load curve for 40% CSA+ 12% lime [7 days of curing + 4 days of soaking]

Table E-3.23 Result of CBR test for 40% CSA+ 16% lime [0 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	10365	10635
Wt.of mould, g	$W_2$	5630	5630
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4735	5005
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.23	2.36
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.90	1.94
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	313.3	267.6
Wt. of dry sample + cont., g	$W_4$	271.8	225.5
Wt. of water, g	$W_5 = W_3 - W_4$	41.5	42.1
Wt. of container, g	$W_6$	34.3	30.6
Wt. of dry sample, g	$W_7 = W_4 - W_6$	237.5	194.9
% Moisture Content	$W_8 = W_5/W_7*100$	17.5	21.6

CBR Data, Bottom Face Penetration						SWELL DATA		
Penetration (mm)	Std load (KN)	Gauge reading	Load KN	Corrected CBR		Gauge Reading	Initial	Final
				KN	%			
0		0	0				167	302
0.64		216	5.22			Swell	mm	%
1.27		334	8.07				1.35	1.16
1.91		420	10.14					
2.54	13	570	13.77	13.77	<b>105.9</b>	Soaked CBR, %		<b>105.9</b>
5.08	20	660	15.94	15.94	<b>79.7</b>	Swell, %		<b>1.16</b>
6.35		774	18.69			Dry Density, g/cc		<b>1.90</b>
7.62		865	20.89					

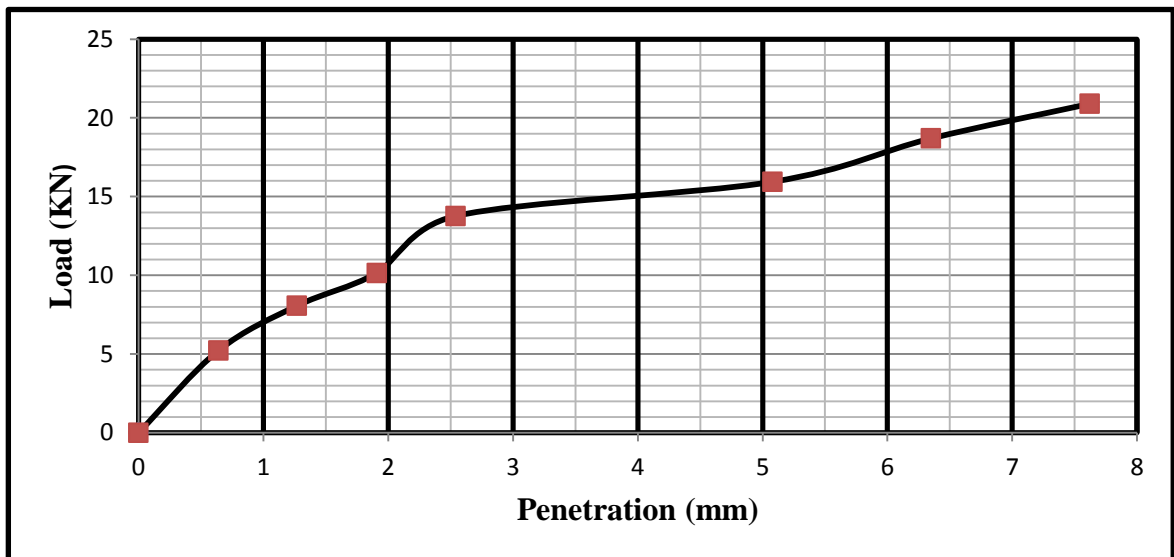


Figure E-3.23 Penetration-Load curve for 40% CSA+ 16% lime [0 days of curing + 4 days of soaking]

Table E-3.24 Result of CBR test for 40% CSA+ 16% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers					
Unit Weight Determination					
CONDITION OF SAMPLE			Before soaking	After soaking	
Wt.of wet sample + mould, g	$W_1$		11085	11210	
Wt.of mould, g	$W_2$		6355	6355	
Wt.of wet sample, g	$W_3 = W_1 - W_2$		4730	4855	
Volume of mould, cc	$V$		2123	2123	
Wet unit weight, g/cc	$D_w = W_3 / V$		2.23	2.29	
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$		1.90	2.08	
Moisture Content Determination					
Wt. of wet sample + cont., g	$W_3$		371.6	328	
Wt. of dry sample + cont., g	$W_4$		322.1	302.2	
Wt. of water, g	$W_5 = W_3 - W_4$		49.5	25.8	
Wt. of container, g	$W_6$		38.6	36.2	
Wt. of dry sample, g	$W_7 = W_4 - W_6$		283.5	266	
% Moisture Content	$W_8 = W_5/W_7*100$		17.5	9.7	

CBR Data, Bottom Face Penetration					SWELL DATA			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%		mm	%
0		0	0				452	493
0.64		265	6.40			Swell		
1.27		383	9.25				0.41	0.35
1.91		469	11.33					
2.54	13	619	14.95	14.95	<b>115.0</b>	Soaked CBR, %	<b>115.0</b>	
5.08	20	709	17.12	17.12	<b>85.6</b>	Swell, %	<b>0.35</b>	
6.35		823	19.88			Dry Density, g/cc	<b>1.90</b>	
7.62		914	22.07					

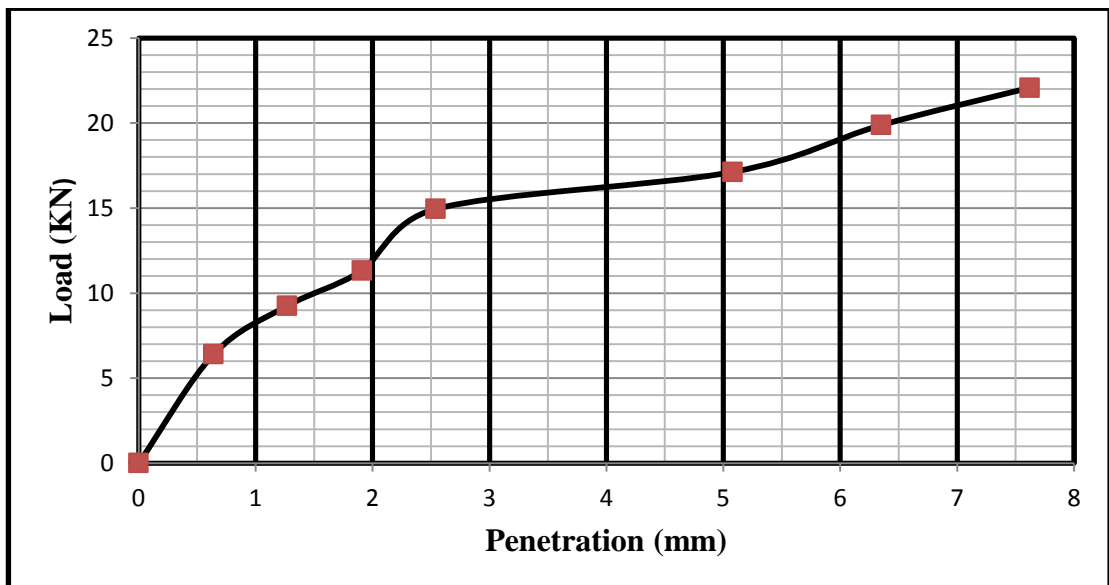


Figure E-3.24 Penetration-Load curve for 40% CSA+ 16% lime [7 days of curing + 4 days of soaking]

Table E-3.25 Result of CBR test for 60% CSA+ 4% lime [0 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	11175	11225
Wt.of mould, g	$W_2$	6220	6220
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4955	5005
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.33	2.36
Dry unit weight, g/cc	$D_w / (1 + W_g/100)$	2.09	2.09
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	213.3	262.6
Wt. of dry sample + cont., g	$W_4$	194.5	236.5
Wt. of water, g	$W_5 = W_3 - W_4$	18.8	26.1
Wt. of container, g	$W_6$	31.3	36.6
Wt. of dry sample, g	$W_7 = W_4 - W_6$	163.2	199.9
% Moisture Content	$W_8 = W_5/W_7*100$	11.5	13.1

CBR Data, Bottom Face Penetration					SWELL DATA			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				168	320
0.64		245	5.92			Swell	mm	%
1.27		361	8.72				1.52	1.31
1.91		455	10.99			Soaked CBR, %		109.6
2.54	13	590	14.25	14.25	109.6	Swell, %		1.31
5.08	20	695	16.78	16.78	83.9	Dry Density, g/cc		2.09
6.35		804	19.42					
7.62		909	21.95					

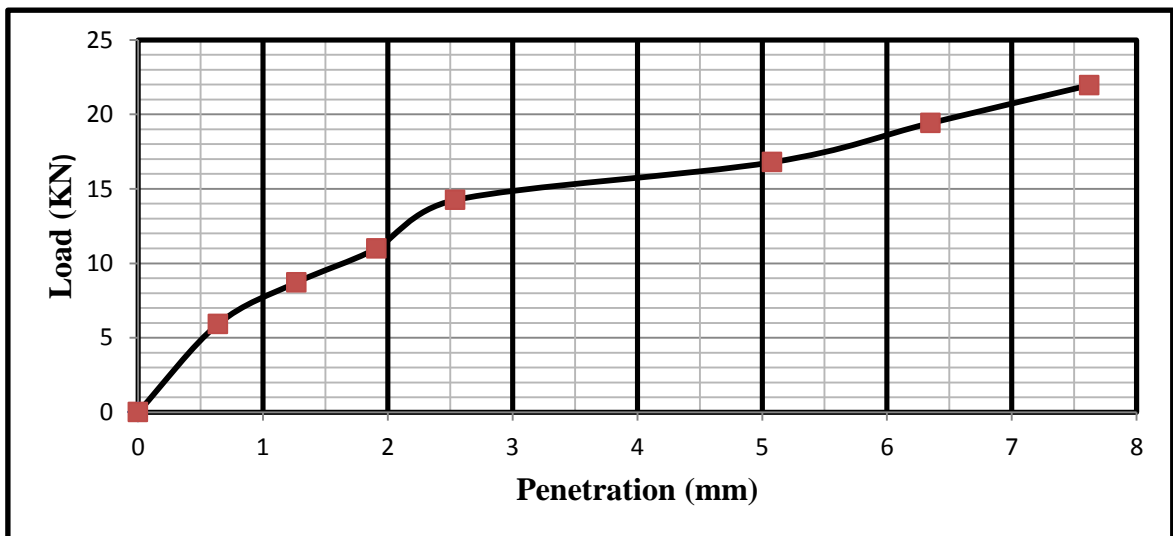


Figure E-3.25 Penetration-Load curve for 60% CSA+ 4% lime [0 days of curing + 4 days of soaking]

Table E-3.26 Result of CBR test for 60% CSA+ 4% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	11185	11560
Wt.of mould, g	$W_2$	6235	6235
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4950	5325
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.33	2.51
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	2.09	2.20
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	268.2	298.7
Wt. of dry sample + cont., g	$W_4$	244.3	266.9
Wt. of water, g	$W_5 = W_3 - W_4$	23.9	31.8
Wt. of container, g	$W_6$	35.9	37.7
Wt. of dry sample, g	$W_7 = W_4 - W_6$	208.4	229.2
% Moisture Content	$W_8 = W_5/W_7*100$	11.5	13.9

CBR Data, Bottom Face Penetration					SWELL DATA			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%		mm	%
0		0	0				649	684
0.64		314	7.58			Swell	mm	%
1.27		430	10.38				0.35	0.30
1.91		524	12.65					
2.54	13	659	15.91	15.91	<b>122.4</b>	Soaked CBR, %	<b>122.4</b>	
5.08	20	764	18.45	18.45	<b>92.3</b>	Swell, %	<b>0.30</b>	
6.35		873	21.08			Dry Density, g/cc	<b>2.09</b>	
7.62		978	23.62					

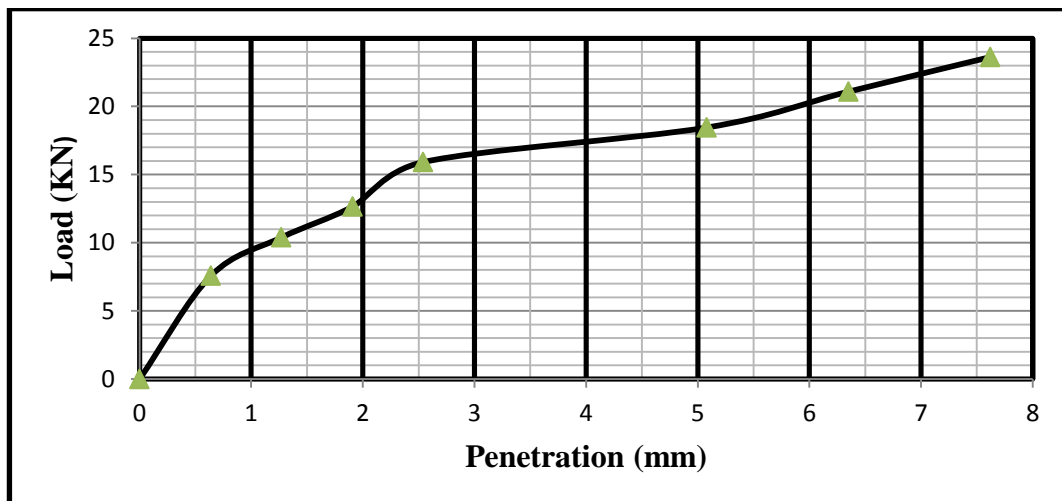


Figure E-3.26 Penetration-Load curve for 60% CSA+ 4% lime [7 days of curing + 4 days of soaking]

Table E-3.27 Result of CBR test for 60% CSA+ 8% lime [0 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	11040	11205
Wt.of mould, g	$W_2$	6100	6100
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4940	5105
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.33	2.40
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	2.07	2.05
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	285.6	264.6
Wt. of dry sample + cont., g	$W_4$	257.3	230.3
Wt. of water, g	$W_5 = W_3 - W_4$	28.3	34.3
Wt. of container, g	$W_6$	33.3	30.6
Wt. of dry sample, g	$W_7 = W_4 - W_6$	224	199.7
% Moisture Content	$W_8 = W_5/W_7*100$	12.6	17.2

CBR Data, Bottom Face Penetration					SWELL DATA			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				416	562
0.64		330	7.97			Swell	mm	%
1.27		424	10.24				1.46	1.26
1.91		508	12.27					
2.54	13	620	14.97	14.97	<b>115.2</b>	Soaked CBR, %		<b>115.2</b>
5.08	20	740	17.87	17.87	<b>89.4</b>	Swell, %		<b>1.26</b>
6.35		851	20.55			Dry Density, g/cc		<b>2.07</b>
7.62		995	24.03					

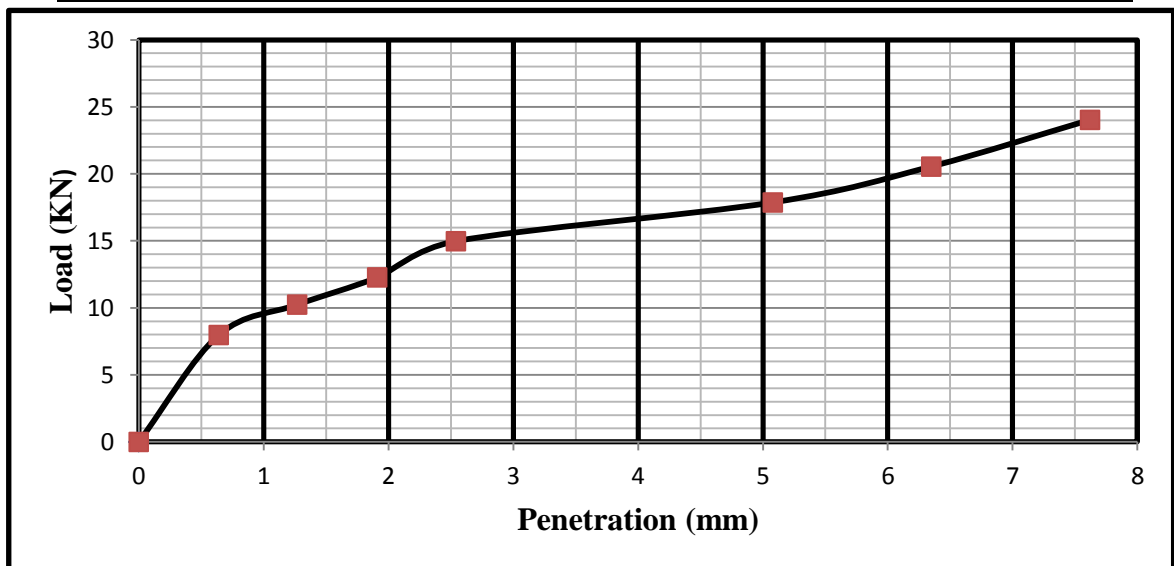


Figure E-3.27 Penetration-Load curve for 60% CSA+ 8% lime [0 days of curing + 8 days of soaking]

Table E-3.28 Result of CBR test for 60% CSA+ 8% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt. of wet sample + mould, g	$W_1$	11230	11210
Wt. of mould, g	$W_2$	6285	6285
Wt. of wet sample, g	$W_3 = W_1 - W_2$	4945	4925
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.33	2.32
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	2.07	1.98
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	307.9	326.8
Wt. of dry sample + cont., g	$W_4$	278.2	285.6
Wt. of water, g	$W_5 = W_3 - W_4$	29.7	41.2
Wt. of container, g	$W_6$	43	43.4
Wt. of dry sample, g	$W_7 = W_4 - W_6$	235.2	242.2
% Moisture Content	$W_8 = W_5/W_7*100$	12.6	17.0

CBR Data, Bottom Face Penetration					SWELL DATA			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				313	347
0.64		401	9.68			Swell	mm	%
1.27		495	11.95				0.34	0.29
1.91		579	13.98					
2.54	13	691	16.69	16.69	<b>128.4</b>	Soaked CBR, %	<b>128.4</b>	
5.08	20	811	19.59	19.59	<b>97.9</b>	Swell, %	<b>0.29</b>	
6.35		922	22.27			Dry Density, g/cc	<b>2.07</b>	
7.62		1066	25.74					

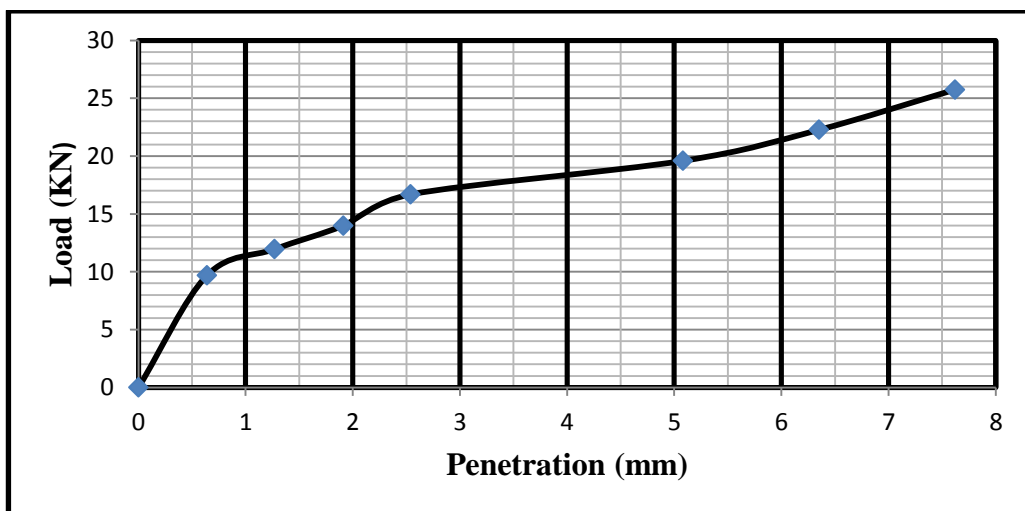


Figure E-3.28 Penetration-Load curve for 60% CSA+ 8% lime [7 days of curing + 4 days of soaking]

Table E-3.29 Result of CBR test for 60% CSA+ 12% lime [0 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt.of wet sample + mould, g	$W_1$	10385	10660
Wt.of mould, g	$W_2$	5430	5430
Wt.of wet sample, g	$W_3 = W_1 - W_2$	4955	5230
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.33	2.46
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	2.05	2.08
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	264.6	218.8
Wt. of dry sample + cont., g	$W_4$	236.3	190.5
Wt. of water, g	$W_5 = W_3 - W_4$	28.3	28.3
Wt. of container, g	$W_6$	31.6	35.5
Wt. of dry sample, g	$W_7 = W_4 - W_6$	204.7	155
% Moisture Content	$W_8 = W_5/W_7*100$	13.8	18.3

CBR Data, Bottom Face Penetration						SWELL DATA		
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				515	652
0.64		325	7.85			Swell	mm	%
1.27		440	10.63				1.37	1.18
1.91		540	13.04					
2.54	13	660	15.94	15.94	<b>122.6</b>	Soaked CBR, % <b>122.6</b>		
5.08	20	840	20.29	20.29	<b>101.4</b>	Swell, % <b>1.18</b>		
6.35		990	23.91			Dry Density, g/cc <b>2.05</b>		
7.62		1070	25.84					

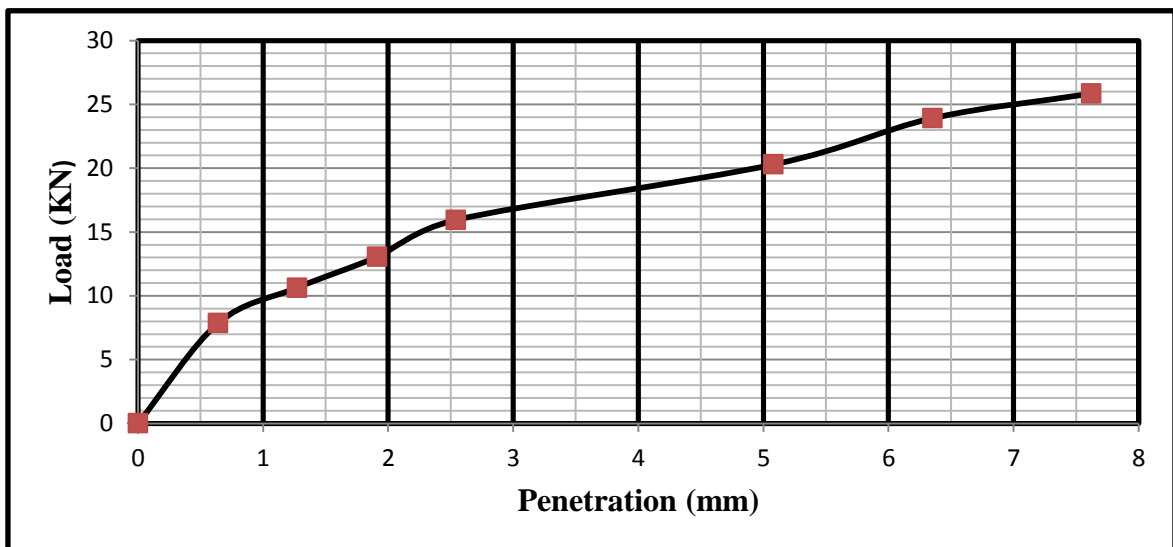


Figure E-3.29 Penetration-Load curve for 60% CSA+ 12% lime [0 days of curing + 4 days of soaking]

Table E-3.30 Result of CBR test for 60% CSA+ 12% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt. of wet sample + mould, g	$W_1$	11365	11390
Wt. of mould, g	$W_2$	6400	6400
Wt. of wet sample, g	$W_3 = W_1 - W_2$	4965	4990
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	2.34	2.35
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	2.05	2.15
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	267.5	322.6
Wt. of dry sample + cont., g	$W_4$	239.1	298.5
Wt. of water, g	$W_5 = W_3 - W_4$	28.4	24.1
Wt. of container, g	$W_6$	34	33.4
Wt. of dry sample, g	$W_7 = W_4 - W_6$	205.1	265.1
% Moisture Content	$W_8 = W_5/W_7*100$	13.8	9.1

CBR Data, Bottom Face Penetration						SWELL DATA		
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				479	501
0.64		388	9.37			Swell	mm	%
1.27		503	12.15				0.22	0.19
1.91		603	14.56					
2.54	13	723	17.46	17.46	<b>134.3</b>	Soaked CBR, %		<b>134.3</b>
5.08	20	903	21.81	21.81	<b>109.0</b>	Swell, %		<b>0.19</b>
6.35		1053	25.43			Dry Density, g/cc		<b>2.05</b>
7.62		1133	27.36					

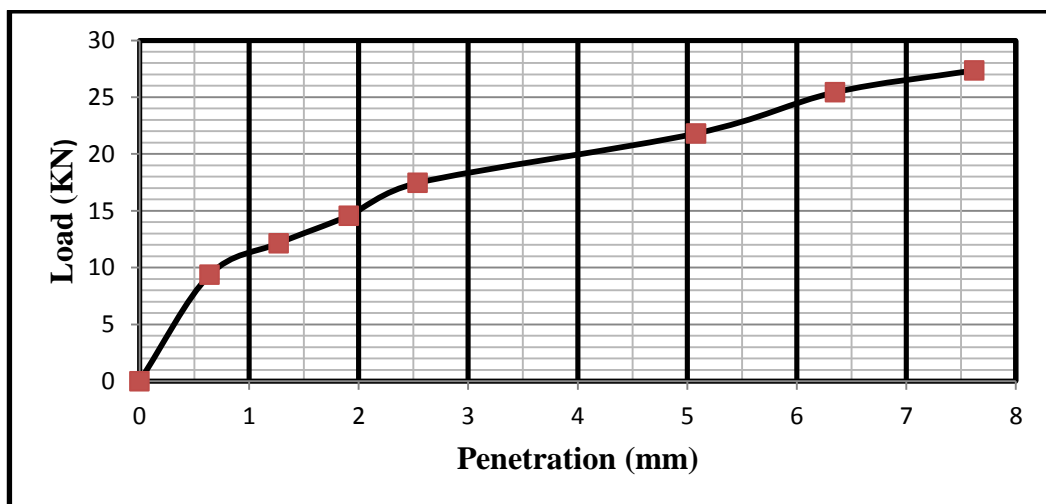


Figure E-3.30 Penetration-Load curve for 60% CSA+ 12% lime [7 days of curing + 4 days of soaking]

Table E-3.31 Result of CBR test for 60% CSA+ 16% lime [0 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers			
Unit Weight Determination			
CONDITION OF SAMPLE		Before soaking	After soaking
Wt. of wet sample + mould, g	$W_1$	10205	10490
Wt. of mould, g	$W_2$	6050	6050
Wt. of wet sample, g	$W_3 = W_1 - W_2$	4155	4440
Volume of mould, cc	$V$	2123	2123
Wet unit weight, g/cc	$D_w = W_3 / V$	1.96	2.09
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$	1.48	1.78
Moisture Content Determination			
Wt. of wet sample + cont., g	$W_3$	362.3	350.3
Wt. of dry sample + cont., g	$W_4$	282.6	303.4
Wt. of water, g	$W_5 = W_3 - W_4$	79.7	46.9
Wt. of container, g	$W_6$	33.7	35.9
Wt. of dry sample, g	$W_7 = W_4 - W_6$	248.9	267.5
% Moisture Content	$W_8 = W_5/W_7*100$	32.0	17.5

CBR Data, Bottom Face Penetration				SWELL DATA				
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge Reading	Initial	Final
			KN	KN	%			
0		0	0				741	868
0.64		365	8.81			Swell	mm	%
1.27		480	11.59				1.27	1.09
1.91		580	14.01					
2.54	13	700	16.91	16.91	<b>130.0</b>	Soaked CBR, %	<b>130.0</b>	
5.08	20	880	21.25	21.25	<b>106.3</b>	Swell, %	<b>1.09</b>	
6.35		1030	24.87			Dry Density, g/cc	<b>1.48</b>	
7.62		1110	26.81					

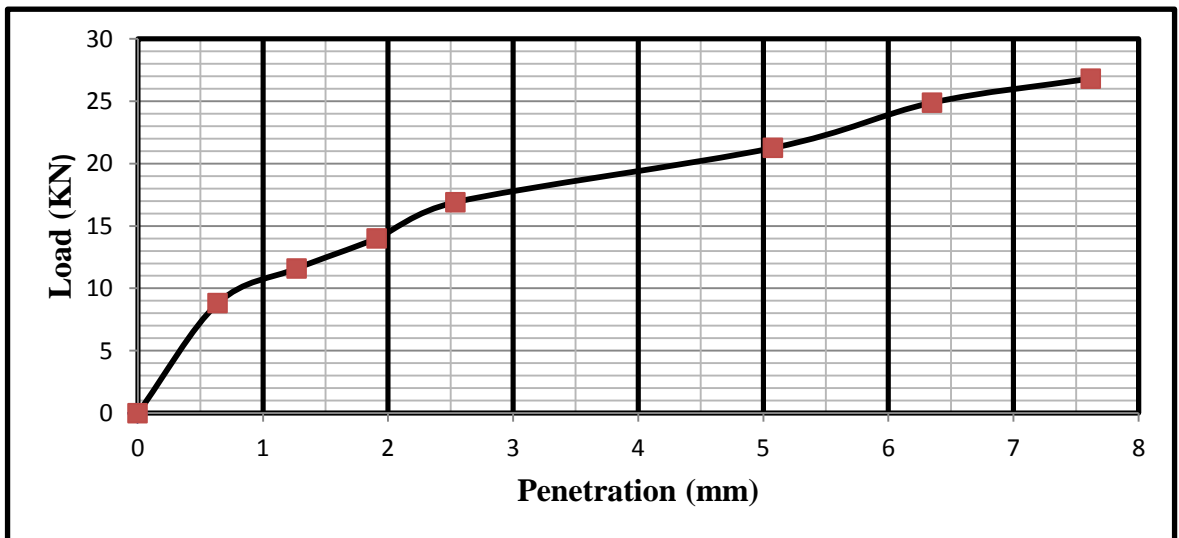


Figure E-3.31 Penetration-Load curve for 60% CSA+ 16% lime [0 days of curing + 4 days of soaking]

Table E-3.32 Result of CBR test for 60% CSA+ 16% lime [7 days of curing + 4 days of soaking]

CBR TEST, 56 Blows, 5 Layers					
Unit Weight Determination					
CONDITION OF SAMPLE			Before soaking	After soaking	
Wt. of wet sample + mould, g	$W_1$		10975	11110	
Wt. of mould, g	$W_2$		6025	6025	
Wt. of wet sample, g	$W_3 = W_1 - W_2$		4950	5085	
Volume of mould, cc	$V$		2123	2123	
Wet unit weight, g/cc	$D_w = W_3 / V$		2.33	2.40	
Dry unit weight, g/cc	$D_w / (1 + W_8/100)$		2.03	2.16	
Moisture Content Determination					
Wt. of wet sample + cont., g	$W_3$		331.9	344.3	
Wt. of dry sample + cont., g	$W_4$		292.8	313.4	
Wt. of water, g	$W_5 = W_3 - W_4$		39.1	30.9	
Wt. of container, g	$W_6$		33.7	35.9	
Wt. of dry sample, g	$W_7 = W_4 - W_6$		259.1	277.5	
% Moisture Content	$W_8 = W_5/W_7*100$		15.1	11.1	

CBR Data, Bottom Face Penetration					SWELL DATA			
Penetration (mm)	Std load (KN)	Gauge reading	Load KN	Corrected CBR		Gauge Reading	Initial	Final
				KN	%		mm	%
0		0	0				382	398
0.64		426	10.29			Swell	mm	%
1.27		541	13.07				0.16	0.14
1.91		641	15.48					
2.54	13	761	18.38	18.38	<b>141.4</b>	Soaked CBR, %		<b>141.4</b>
5.08	20	941	22.73	22.73	<b>113.6</b>	Swell, %		<b>0.14</b>
6.35		1091	26.35			Dry Density, g/cc		<b>2.03</b>
7.62		1171	28.28					

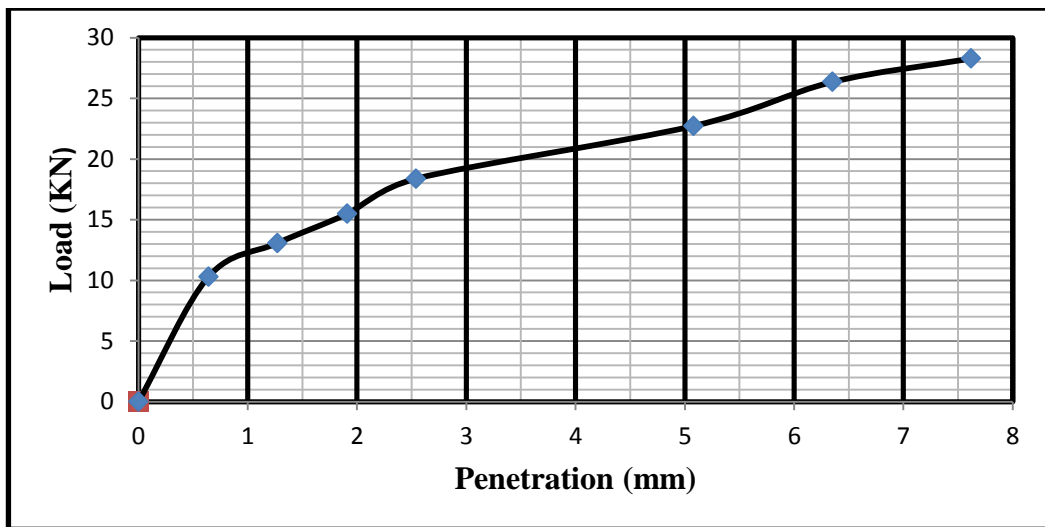


Figure E-3.32 Penetration-Load curve for 60% CSA+ 16% lime [7 days of curing + 4 days of soaking]

**APPENDIX E-4 Durability (Wet/Dry) Test**

Table E-4.1 Result of 12 cycle wet/dry test for Laterite-CSA-Lime mixtures

Lime Content %	No. of Cycle	CSA Content (%)			
		0%	20%	40%	60%
		Wt. after brushing	Wt. after brushing	Wt. after brushing	Wt. after brushing
8%	0	1617.2	1820	1715.3	1776.2
	1	1612.3	1779.3	1697.9	1771.6
	2	1608.1	1775.5	1695.2	1768.6
	3	1571.1	1772.3	1690.2	1765.3
	4	1559.4	1768.3	1680	1759.4
	5	1553	1761.7	1675.2	1752.2
	6	1546.2	1756.9	1671.2	1747.4
	7	1538.6	1751.6	1667.3	1743
	8	1526.2	1748.8	1658.4	1730.2
	9	1519.6	1745.1	1657.3	1727.3
	10	1512.3	1744.2	1655.1	1722.3
	11	1501.7	1740.4	1653.8	1719.5
	12	1495.1	1737.2	1651	1715.3
12%	0	1646.9	1719.4	1720.1	1705.2
	1	1638.9	1714.9	1716.2	1702.1
	2	1634.4	1712.3	1711.8	1697.1
	3	1604.7	1709.1	1708.3	1693.2
	4	1602	1701.5	1703.7	1690.4
	5	1595.3	1695.7	1697.3	1685.2
	6	1589.5	1694.1	1693.6	1680.2
	7	1585.2	1689.5	1685.9	1672.2
	8	1579.8	1681.4	1979.8	1668.7
	9	1575.6	1677.6	1974.4	1664
	10	1555.2	1673.5	1972.3	1661.2
	11	1548.6	1670.3	1970.1	1658.7
	12	1542.3	1665	1668.5	1656.1

Table E-4.1 Result of weight of loss for Laterite-CSA-Lime mixtures

CSA Content (%)	Lime Content							
	8%				12%			
	Initial oven dry wt	Final oven dry wt	corrected oven dry wt	loss	Initial oven dry wt	Final oven dry wt	corrected oven dry wt	loss
0%	1617.2	1493.1	1442.6	10.8	1646.9	1539.3	1487.2	9.7
20%	1820	1732.2	1673.4	8.1	1719.4	1659.8	1603.7	6.7
40%	1715.3	1645.2	1589.6	7.3	1720.1	1665.1	1608.8	6.5
60%	1776.2	1710.4	1652.6	7.0	1705.2	1652.1	1596.2	6.4

**APPENDIX F- STATISTICAL ANALYSIS OUTPUT**

Table F-1: Statistical analysis of CBR for laterite-CSA mix using ANOVA

		CSA Content					
		0%	20%	40%	60%		
		27.1	66.4	86.3	112.0		
		30.1	60.3	82.9	107.0		
n		2.00	2.00	2.00	2.00	N=8.00	
Anova: Single Factor							
SUMMARY							
Groups	Count	Sum	Average	Variance			
0%	2	57.2	28.6	4.50			
20%	2	126.7	63.35	18.61			
40%	2	169.2	84.6	5.78			
60%	2	219	109.5	12.50			
ANOVA							
Source of Variation	SS	df	MS	F	P-value	F crit	
Between Groups	7044.88	3	2348.29	226.971	6.3827E-05	6.59138	
Within Groups	41.385	4	10.3463				
Total	7086.27	7					

Table F-2: Statistical analysis of LAA for laterite-CSA mix using ANOVA

		CSA Content					
		0%	20%	40%	60%		
		70.22	48.69	37.52	28.76		
		70.35	48.77	37.48	28.82		
n		2.00	2.00	2.00	2.00	N= 8.00	
Anova: Single Factor							
SUMMARY							
Groups	Count	Sum	Average	Variance			
0% CSA	2	140.57	70.285	0.00845			
20% CSA	2	97.46	48.73	0.0032			
40% CSA	2	75	37.5	0.0008			
60% CSA	2	57.58	28.79	0.0018			
ANOVA							
Source of Variation	SS	df	MS	F	P-value	F crit	
Between Groups	1930.44	3	643.482	180626	1.02167E-10	6.59138	
Within Groups	0.01425	4	0.00356				
Total	1930.46	7					

Table F-3: Statistical analysis of TFV for laterite-CSA mix using ANOVA

		CSA Content					
		0%	20%	40%	60%		
		16.71	36.16	51.19	65.75		
		17.82	36.93	51.32	66.74		
n		2.00	2.00	2.00	2.00	N= 8.00	
Anova: Single Factor							
SUMMARY							
Groups	Count	Sum	Average	Variance			
0%	2	34.53	17.265	0.61605			
20%	2	73.09	36.545	0.29645			
40%	2	102.51	51.255	0.00845			
60%	2	132.49	66.245	0.49005			
ANOVA							
Source of Variation	SS	df	MS	F	P-value	F crit	
Between Groups	2624.63	3	874.876	2480.16	5.41222E-07	6.59138	
Within Groups	1.411	4	0.35275				
Total	2626.04	7					

Table F-4: Statistical analysis of Atterbeg limit for 0% CSA with different lime content using ANOVA

		Lime Content						
		0%	4%	8%	12%	16%		
		18.9	18.4	12	9.6	10.1		
		20.8	16.4	15.4	11.9	8.3		
n		2.00	2.00	2.00	2.00	2.00	N=10.00	
Anova: Single Factor								
SUMMARY								
Groups	Count	Sum	Average	Variance				
0%	2	39.7	19.85	1.805				
4%	2	34.8	17.4	2				
8%	2	27.4	13.7	5.78				
12%	2	21.5	10.75	2.645				
16%	2	18.4	9.2	1.62				
ANOVA								
Source of Variation	SS	df	MS	F	P-value	F crit		
Between Groups	158.626	4	39.6565	14.3164	0.00603	5.192168		
Within Groups	13.85	5	2.77					
Total	172.476	9						

Table F-5: Statistical analysis of Atterbeg limit for 20% CSA with different lime content using ANOVA

		Lime Content					
		0%	4%	8%	12%	16%	
		17.6	13.5	11.9	10.6	8.1	
		16	15.8	12.8	9.2	9.4	
n		2.00	2.00	2.00	2.00	2.00	N=10.00
Anova: Single Factor							
SUMMARY							
Groups	Count	Sum	Average	Variance			
0%	2	33.6	16.8	1.28			
4%	2	29.3	14.65	2.645			
8%	2	24.7	12.35	0.405			
12%	2	19.8	9.9	0.98			
16%	2	17.5	8.75	0.845			
ANOVA							
Source of Variation	SS	df	MS	F	P-value	F crit	
Between Groups	87.914	4	21.9785	17.8542	0.00365	5.192168	
Within Groups	6.155	5	1.231				
Total	94.069	9					

Table F-6: Statistical analysis of Atterbeg limit for 40% CSA with different lime content using ANOVA

		Lime Treatment					
		0%	4%	8%	12%	16%	
		13.6	10.6	6.6	4.5	2.1	
		12.4	11.2	7	4.1	1.9	
n		2.00	2.00	2.00	2.00	2.00	N=10.00
Anova: Single Factor							
SUMMARY							
Groups	Count	Sum	Average	Variance			
0%	2	26	13	0.72			
4%	2	21.8	10.9	0.18			
8%	2	13.6	6.8	0.08			
12%	2	8.6	4.3	0.08			
16%	2	4	2	0.02			
ANOVA							
Source of Variation	SS	df	MS	F	P-value	F crit	
Between Groups	165.48	4	41.37	191.528	1.2E-05	5.19217	
Within Groups	1.08	5	0.216				
Total	166.56	9					

Table F-6: Statistical analysis of Atterbeg limit for 60% CSA with different lime content using ANOVA

	Lime Content					N=10.00
	0%	4%	8%	12%	16%	
	11.4	8.1	5.5	0	0	
	10.7	7.3	4.6	0	0	
n	2	2	2	2	2	
Anova: Single Factor						
SUMMARY						
Groups	Count	Sum	Average	Variance		
0%	2	22.1	11.05	0.245		
4%	2	15.4	7.7	0.32		
8%	2	10.1	5.05	0.405		
12%	2	0	0	0		
16%	2	0	0	0		
ANOVA						
Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	187.214	4	46.8035	241.2552	6.7E-06	5.19217
Within Groups	0.97	5	0.194			
Total	188.184	9				