ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

The Stablization of Lateritic Soil with Lime as a Pavement Material for the case of Wolayita – Sodo

By FevenAsmamaw

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A thesis submitted to the school of graduate studies of Addis Ababa University inpartial fulfillment of the requirements for the Degree of Masters of Science in CivilEngineering

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The Stablization of Lateritic Soil with Lime as a Pavement Material for the Case of Wolayita-Sodo Ethiopia

By Feven Asmamaw

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DECLARATION

I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor Dr. Mesele Haile 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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Feven Asmamaw

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LIST OF ABBREVIATIONS

AASHTO American Association of Highway and Transportation Officials
ASTM American Society for Testing and Materials
BS British Standard
CAH Calcium aluminate hydrate
CEC Cation Exchange Capacity
CBR California Bearing Ratio
CSH Calcium Silicate Hydrate
ERA Ethiopian Roads Authority
FSI Free swell index
FSR Free swell ratio
IS Indian Standard
LL Liquid Limit
PL Plastic Limit
MDD Maximum Dry Density
OMC Optimum Moisture Content
PI Plastic Index
PL Plastic Limit
SP Swelling pressure
UCS Unconfined compressive strength
USA United States of America
OD Oven dried at 105^{o}c
AD Air dried
AR As receive

Units

gm Gram
kg Kilogram
km Kilometer
kN Kilo Newton
mm millimeter
g/cm3 Gram per centimeter cube
µm micrometer
kN/m2 Kilo Newton per meter square
kPa Kilo Pascal
Pcf pounds per cubic foot
mm millimeter
meqMilliequivalent
^{o}C Degree Centigrade
cc Centimeter cube

ABSTRACT

Lateritic soil covers Western and dominantly Southern part of Ethiopia specially in Nekemete-Wollega, Dilla, Hossana, Chida and Wollayita Sodo area.

When lateritic soil consist of high plastic clay, the plasticity of the soil may cause cracks and damage on pavement, road ways, building foundations or any other civil engineering construction projects. The improvement in the strength and durability of lateritic soil in the recent time become imperative, this has geared up researchers toward using stabilizing materials.

In this study, the soil sample was collected from southern Ethiopia region called Wollayita-Sodo around Wadu area which was one of the sample area for previous study called "Index and shear strength study of Lateritic soil in Wollayita Sodo area" by (Tibebu,2008)

Index properties and Engineering properties of the soils were studied and soil is classified as A-7-5 according to AASHTO classification, the PI is about 22% and soaked CBR value of 21.9%.These values does not meet the desired values for competent road base and sub base material according to ERA pavement manual .Therefore, In this study an attempt was made to improve the property of the soil by stabilization with increasing percentage of lime until optimum percentage is found.

As shown from the study, the addition of lime decrease maximum dry density and increase the optimum moisture content with the same compaction effort and decrease the plasticity by decreasing the plastic index to 7%.The unconfined compressive strength of Tp1 and Tp2 has improved from 341.05kpa at 0% to 352.3kpa at 4% and 391.823kpa at 0% to 369.08kpa at 4% lime content respectively as well as the soaked CBR value has improved from 21.9% at 0% to 54.7% at 4% lime Content.

Therefore, samples do not qualify for base course but can be utilized as a sub base material with 4% lime stabilization.

1. INTRODUCTION

1.1. General

In the road construction industry, there is much need for soil material in the construction of pavements. When a section of the road is to be filled with soil, the material is either obtained from other cut sections along the road or from a borehole site where the suitable material is available, if the material from the road does not meet the required standards. A major cost that would be incurred by “borrowing” soil material is that involved in hauling the material from the borrow site to the construction site. This cost, in terms of finance, resources, and time, could however be avoided by simply improving the characteristics of the in situ material that was earlier rejected.

Lateritic soils have been one of the major highway and building materials in all the tropical and subtropical countries of the world for a long time. Base and sub base materials for most of the highways and walls of large percentage of residential houses in rural areas have been built with lateritic soils using different types of stabilizer. Many types of stabilizers have been used in different parts of the world in soil stabilization for various civil engineering works.

Lateritic soils are highly weathered and altered residual soils formed by the in-situ weathering and decomposition of rocks under tropical conditions. The soils have higher proportion of sesquioxide of iron and aluminum relative to the other mineral components.

Lateritic soils are generally used in road construction usually as a sub grade material in Southern Ethiopia. But they are highly plastic and have low CBR value. Therefore, if we stabilize this soil we may be able to improve its property and save economy and material.

This thesis deals with the stabilization of a lateritic soil identified in Wollayita Sodo region in Southern Ethiopia.

The index and shear strength properties of lateritic soils of Southern Ethiopia have been studied in similar previous works. For the areas of Wolayita Sodo, all lateritic soil-forming factors are fulfilled, and hence one can consider the soils as lateritic (Tibebu,2008).

To this end, a low-cost method of improving the mechanical behavior and strength of lateritic soils is desirable. It has been found that lateritic soils respond to cement stabilization and, in some cases, lime stabilization. But for clayey soils lime is very effective and better stabilizing agent(Lyon A, 1971).

In addition lime is a very effective stabilizing agent that can improve the behavior of plastic and expansive soils by significantly reducing PI and increasing CBR values provides it the appropriate strength and characteristics for road surface and subgrade material.
The main purpose of this research work is to investigate the influence of lime on atterberg limits, compaction characteristics, unconfined Compression test and CBR values of lateritic soil.

1.2. Background of the Problem

A detailed study and characterization of soil around Wollayita Sodo area were conducted by (Tibebu,2008).In the study all the necessary index and engineering properties of the soil with different pretreatment conditions have been investigated and as a result the soil was found to be lateritic.

However, the soil plasticity with PI 19-30% and the strength requirements such as CBR values do not satisfy the requirements for base material in pavement construction. For this reason, the soil has never been utilized for such purpose.

Also one of the major problem is, due to its high plasticity it becomes very sticky and slippery when it is wet and very dusty when dry. In addition it clogs the drainage structure during wet conditions damaging the constructed road.

1.3. Objectives of the Study

The aim of the study is to examine if the soil in the proposed area could be found suitable for pavement construction after lime treatment. The study describes the effect of lime mixes to improve the geotechnical properties of lateritic soil and recommends the optimum percentage of lime for the soil to satisfy as a road base or sub-base material in road pavement construction .The following index and shear strength properties of the natural and stabilized soil in the proposed area were conducted; Atterberglimits, compaction characteristics, CBR value and unconfined compression test with varying percentage of lime.

1.4. Methodology

Specimens were taken from one of the previously studied (Tibebu, 2008) areas of Wollayita Sodo which is located near Wadu Area. Disturbed samples were collected from two test pits on this area. The laboratory investigations were carried out with procedures given in ASTM. Effect of temperature variation on moisture content determination and different pretreatment and pretest condition were considered. Index property and shear strength tests like Atterberg limits, specific gravity, free swell, grain size analysis, compaction characteristics and unconfined compression test of remolded sample and CBR tests were carried out on the soil before stabilization.

Atterberg limits were also determined on the stabilized soil with increasing percentage of lime until the PI decreased to the desired values according to the ERA pavement manual. Attention was also given to changes on the compressive strength and unconfined compression strength by adding lime to the soil in dry state in percentage (by weight) and for 7 days of curing period. Using the optimum lime content, CBR values were determined. Finally, the optimum proportion of lime use in highway construction is identified.

2. LITERATURE REVEIW

2.1. Introduction

Laterites are highly weathered and altered residual soils formed by the in-situ weathering and decomposition of rocks under tropical condition. The three major agents of weathering are physical, chemical and biological processes (Blight 1997). In the process the parent rock and rock minerals break down forming soils having a 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 (Blight, 1997).

In many Countries of Africa and Asia, lateritic soils are the traditional material for road and airfield construction. The survey of the available data on lateritic soils gives the impression that the red color seems to have been accepted by most authors as the most important property by which these soils could be identified. Other obviously significant basic physical properties such as texture, structure, consistency, etc., often were ignored. It is also noted that lack of uniformity in pretreatment and testing procedures (resulting from association with different standards in different parts of Africa) makes it difficult to compare even textural data on the same soils (Lyon 1971).

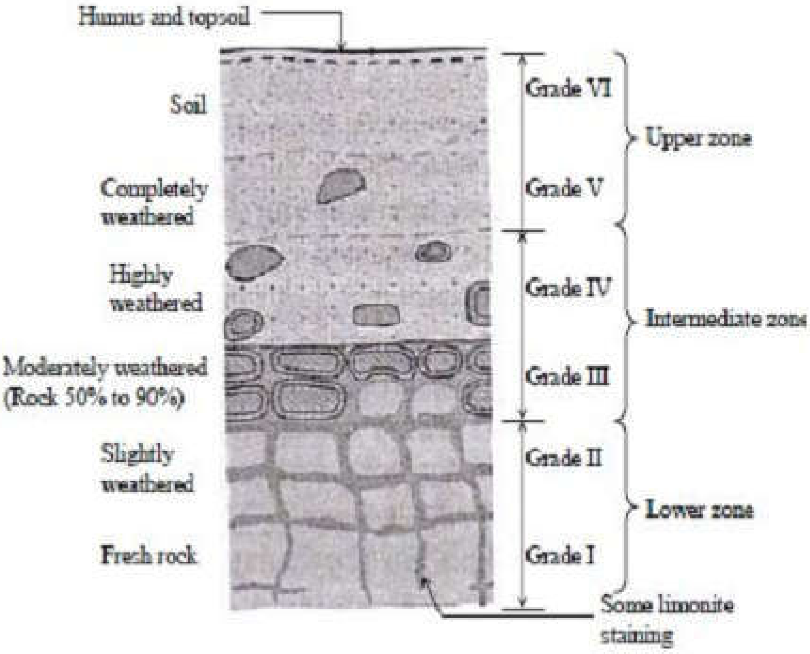
It is noted that three major factors influence the engineering properties and field performance of lateritic soils. They are:

1. Soil forming factors (e.g. parent rock, climatic vegetation conditions, topography and drainage conditions).

2. Degree of weathering (degree of laterization) and texture of soils, genetic soil type, the predominant clay mineral type and depth of sample.

3. Pretest treatments and laboratory test procedures as well as interpretation of test results.

In the review emphasis is given to these factors insofar as they affect the engineering properties, material selection and classification, design and construction control procedures, field performance and stabilization of lateritic soils (Lyon 1971).



Source: Rahardjo et al. (2004)

Figure 2.1: Typical Weathering Profiles of Residual Soil

Table 2.1 Weathering Products of Residual Soils

Weathering Products in Relation To Environments

Environment	pH (mean)	Eh	Behavior of Metallic Ions	Mineralogy
Nonleaching, hot rainfall 0-12 in	Alkaline	Oxidizing	Some loss of alkalies, iron present in ferric state	Partly decomposed parent minerals. Illite, chlorite montmorillonite, and mixed layered clay minerals, Hematite, carbonates and salts. Organic matter absent or sparse.
FERRUGINOUS– Moderate leaching hot climate rainfall 24-72 in.	6.3	Oxidizing	Loss of alkalies and alkaline earths. Some loss of silica. Concentration of alumina, ferric iron and titania	Hematite, goethite, and kaolinite. Organic matter absent or sparse.
FERRISOLS Intense leaching hot climate rainfall over 50 in.	5.9	Oxidizing	Loss of alkalies, alkaline earths and silica. Concentration of alumina, ferric oxide and titania.	Predominantly kaolinite with goethite; hematite and gibbsite less common- Organic matter sparse to abundant.
FERRALLITIC Intense leaching hot climate rainfall over 60 in.	4.7	Oxidizing to Reducing	Loss of alkalies, alkaline earths and silica. Concentration of alumina. Iron may be lost under reducing Eh and concentrated under oxidizing Eh.	Gibbsite, goethite with kaolinite. Organic matter may be abundant.

2.2. Formation of Lateritic Soils

The lateritic soil formation involves three major processes which are identified as follows (Makasa, 1998):

- Decomposition
- Leaching
- Desiccation

Decomposition: Physico-chemical breakdown of primary minerals and the release of constituent elements (SiO2, Al2O3, Fe2O3, CaO, MgO, K2O, Na2O, etc), which appear in simple ionic forms. Tropical decomposition tends to favor formation of the clay mineral kaolinite. This is the most common clay mineral in tropical residual soils. Under suitably moist conditions, halloysites will be formed. Under prolonged decomposition, silica can be removed to the extent that free alumina and iron oxides are present (Blight, 1997).

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 the 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 hydrate or hydrous oxides of iron and aluminum.

Desiccation /Dehydration/: Dehydration (either partial or complete) alters the composition and distribution of the sesquioxide rich materials in a manner which is generally not reversible upon wetting. Dehydration also influences the formation processes of clay minerals. In the case of total dehydration, strongly cemented soils with a unique granular soils structure may be formed (Blight, 1997). 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.

2.3.The Geological Distribution of Lateritic Soils

Lateritic soils covers extensive areas in tropical countries with intermittently moist climate which covers Africa, India, South East Asia, Australia, Central and South America.

However due to shifts of climatic zone in geological past coverage of laterites can be found outside tropics (CIRIA,1995).

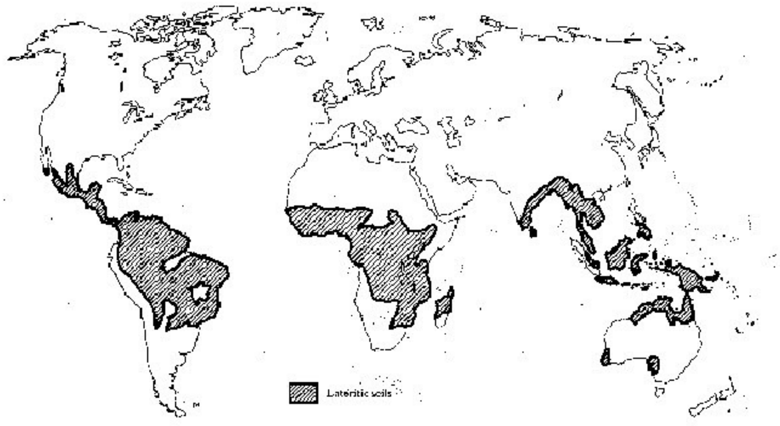


Fig 2.2 Worldwide distribution of Laterites

2.4. Pedological and Lithological Classification of Residual Soils

The first step in the grouping of residual soils is to divide them into groups on the basis of mineralogical composition alone, without referring to their undisturbed state. The following three groups are often suggested: (Blight,1997).

1. **Group A:** Soils without a strong mineralogical influence
2. **Group B:** Soils with strong mineralogical influence derived from clay minerals also commonly found in transported soils.
3. **Group C:** Soils with a mineralogical influence deriving from clay minerals only found in residual soils. These soils are strongly influenced by the presence of clay minerals not commonly found in transported soils. The two most important minerals involved are silicate clay minerals halloysite and allophone.

Group C is further grouped in to three sub-groups according to clay minerals of soils. These are:-

a) Halloysitic soils Sub-group
b) Allophanic soils Sub-group
c) Soils influenced by the presence of sesquioxides.

Halloysite: - Is a Crystalline mineral of tubular form and belongs to the same group as Kaolinite. The physical influence of halloysite appears to be that the engineering properties of the soil are good,despite a high clay fraction fairly high values of natural water content interms of Atterberg limits (i.e has high liquidity index)

Alophane:- Is very distinctive minerals with unusual properties it is described as amorphous however, recent researches suggest it may have a poorly developed crystalline structure. The influence of alophane is both dramatic and puzzling in that it results in soils having natural water content ranging from about 80% to 250% but which still perform satisfactorily as engineering material.

Sub-group C:-The soils in this group range from low plasticity silty clays through to the concretionary deposits made up predominantly of gravel sized material.

Laterites are usually highly weathered and altered residual soils, low in Silica that contains a sufficient concentration of Sesquioxides of Iron soils, and Aluminum to have been cemented to some degree.

The extent of which a residual soil has been laterized may be measured by the ratio of silica, SiO_2, remaining in the soil (except for discrete pebbles of free quartz that may remain) to the amount of Fe_2O_3 and Al_2O_3 that has accumulated.

The Silica to Sequioxide ratio give as:-

$$\frac{SiO_2}{R_2\,O_3} = \frac{SiO_2}{Fe_3O_3 + Al_2O_3}$$

It is basis for classification of residual soils ratios; less than 1.33 have sometimes been considered indicative of true laterites, those between 1.33 and 2 of lateritic soils and those greater than 2 of non lateritic tropically weathered soils.

Laterites are often excellent material for the building of roads and embankments. The high strength and low compressibility is suitable for placing of shallow foundations (Blight, 1997).

Peological classification system is given by D'Hoore. The soils are broadly differentiated on a genetic basis, determined by soil forming factors. it is a means of identification of lateritic soils (Lyon,1997).Three main units are used for the description and classification of red tropical soils (CIRIA,1995).

Ferruginous Soils:- show a marked separation of free 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 aluminum. They generally occur in more humid areas with more than 1500mm rainfall per year.

Ferrisol soils have profiles similar to ferralitic soils, but with very few weather able minerals remaining. The entire clay size fraction comprises kaolinite and amorphous oxides of iron and aluminum. Ferrisol tend to develop at deeper levels, because of surface erosion, and occur in regions of between 1250 and 2750 mm rainfall per year. According to Morine W.J. and Todor P.C., Ethiopian laterites fall under this group (Lyon, 1971).

Nascimento et al. (1959) have suggested an interesting lithological classification of lateritic soils as follows.

Lateritic clays <0.002 mm
Lateritic silts =0.002 - 0.06 mm
Lateritic sands ~ 0 . 0 6- 2 mm
Lateritic gravel =2 - 60 mm
Laterite stones and cuirasse X 0mm

2.5. Hydrated and Dehydrated Laterite

In continuously wet regions the end-products of the process of laterization are likely to be characterized by high natural water contents, high liquid limits, and irreversible changes upon drying. In as much as limonite and geothite are hydrated oxides of iron, and gibbsite is a hydrated oxide of aluminum, drying alters the characteristics of the material markedly. The plasticity decreases and the grain-size increases such that much of the clay-sized material agglomerates to the size of silt. When dealing with such materials, the engineer may need to determine the index properties and the engineering properties

On the basis of tests performed at the natural water content and again at various degrees of air drying in order to evaluate fully the range of properties associated with the physical conditions that may prevail on the job. Serious engineering and construction problems have arisen through failure to recognize materials likely to experience irreversible changes as a consequence of desiccation.

On the other hand, in regions subject to distinct wet and dry seasons, the products of laterization are likely to be characterized by low natural water contents, low plasticity, and the presence of concretions and cemented horizons. The alternate upward and downward movement of water causes concentration of the iron or of the alumina. Laboratory tests run from the natural water content or from the air-dried state lead to essentially the same results.

The difference in behavior of the hydrated and dehydrated forms of lateritic soils deserves attention because of the danger of being misled by reports in the literature in which the significance of the distinction may not be realized.

2.6. Chemical, Mineralogical Aridphysico-Chemical Characteristics

A distinctive feature of laterite and lateritic soils is the higher proportion of sesquioxides of iron and/or aluminum relative to the other chemical components (Lyon 1971). The amount of alumina or iron oxides is an important factor in differentiating aluminous and ferruginous varieties. The base (alkalis and alkaline earths) are almost absent in lateritic horizons, except in some ferruginous crusts developed in alluvium and some concretionary horizons in ferruginous tropical soils. Other common lateritic constituents are manganese, titanium, chromium and vanadium oxides.

Chemical analysis do not usually reveal the origin, nature or even the composition of laterites or lateritic soils. For example, physically similar laterites may have different chemical composition and chemically similar laterites may display different physical properties (Lyon,1971).

The mineralogical composition is considered to be more important in explaining the physical properties of indurated laterite and lateritic soil. Maignien (1966) has divided the mineralogical constituents into major elements, which are essential to laterization, and minor elements which do not effect the laterization process. The major constituents are oxides and hydroxides of aluminum and iron, with clay minerals and, to a lesser extent, manganese, titanium and silica. The minor constituents are residual remnants or clastic materials. Iron occurs in many forms.

The most common minerals are goethite or limonite and hematite. Maghemite, magnetite and ilmenite have also been reported. Alumina is common as gibbsite. Boehmite and amorphous forms are also fairly common, but diaspore and corundum are relatively rare. Titanium minerals include rutile, anatase and ilmenite.

The clay minerals most common in lateritic soils is kaolinite. Halloysite is also reported. Illite and montmorillonite are rare. The most common primary minerals encountered arequartz,mica (muscovite) feldspar (microcline) and hornblende. The particular mineral available depends upon the parent rock type. The chief clay mineral generally observed is kaolinite with some mica, sericite and illite.

The secondary minerals resulting mainly from the laterization processes are gibbsite, goethite, limonite and hematite. Neither manganese nor titanium minerals were observed in significant amounts. The chemical and mineralogical characteristics of all lateritic soils exhibit some similarities (Clare and O'Reilly, 1960; Terzaghi, 1958; Matyas, 1969; Ruddock, 1967; Morin and Parry, 1969; Baldovin, 1969; and Bawa,1957).

There is very little information on the physico-chemical characteristics of lateritic soils. A few data published on this subject have been confined to the PH, organic content and calcium carbonate content (Clare and O’Reilly,1960). The conclusion reached show that the lateritic medium is either neutral or acidic and the organic content is generally low,below 2%.The calcium carbonate content is either very small or lacking in most lateritic soil profiles.

The significance of the physicochemical properties to the success or otherwise of soil stabilization can hardly be over-emphasized. Lack of data of physico-chemical properties is also a major factor hindering the understanding of the soil-water relationships in lateritic soils.

Experience with soils in stable temperate zones has revealed that particle-size distribution exerts great influence on the engineering properties of soils. It is also one of the most important properties by which soils can be easily identified and classified on the basis of simple field and laboratory tests. Consequently great importance has also been accorded to particle-size distribution in dealing with lateritic soils.

Recent studies have revealed however, that lateritic soils are strikingly different from temperate zone soils in terms of genesis and structure. Their concretionary structure as compared to the dispersed temperate zone soils have necessitated modifications to mechanical or grading tests (Remillion,1967,1955).Consistent reports of variations in the particle-size distribution with methods of pretreatment and testing have been widely reported on lateritic soils.

Schofield, 1957 found that wet sieving increased the silt and clay fraction from 7 to 20% as compared to the dry sieving. It has been found that sodium hexametaphosphate generally gives better dispersion of the fine fraction.

In the study of the particle-size distribution of lateritic soils, three sources of confusion were noted. The first confusion arises from the belief by some authors, e.g. (Bawa ,1957), that lateritic soils represent a group of materials that can be defined within a specific range of particle-size distribution. The second source of confusion seems to arise out of attempts by some authors to confine the word laterite to concretionary lateritic gravels. The third source of confusion arises out of the attachment of unnecessary importance to the soil color.

The most important differentiating factor with regard to indurated laterite seems to be hardness. Attempts have been made to group these materials and some success has been attained. The summary of the results so far obtained will be discussed under appropriate sections together with those of other lateritic soils.

One of the main characteristics of lateritic gravels and gravelly soils is the high content of fines. Consequently, such materials do not fit into the existing temperate-zone classification systems for coarse-grained soils. The grading curves often have flat platforms between 4 mm and 0.5 mm revealing the absence of this fraction. Studies on lateritic gravels by de Graft (Johnsonet al.1969) among others have shown that the grading, though important for identification purposes, cannot alone form the basis for grouping lateritic gravels in terms of mechanical properties. The strength of the aggregates was found to be an important factor. On the basis of studies of lateritic aggregates in Nigeria, Ackroyd (1960) distinguished hard, medium and weak lateritic gravels. It was also established that the strength of the aggregates is mainly a function of the degree of maturity of the lateritic concretionary particles and the predominant sesquioxide in the aggregates (Novais-Ferreira and Correia, 1965). Soils with hard aggregates generally have stable grading curves while weak gravels give different grading curves with different methods of pretreatment.

The problem of particle-size distribution of the fine-grained lateritic soils is more complex than those of the gravels and gravelly soils. Information on the grading of fine-grained soils is rather scanty, because they have been least studied. The silt and clay contents reported vary from 12% (Winterkorn and Chandrasekharan, 195 1) to over 82% (Clare and O'Reilly,1960). One reason for the wide variation in these soils is the tendency to form aggregations or clusters of clay-minerals and clay-size mineral fragments. This tendency has been noted by many authors (Tateishi, 1967; Quinones, 1963; Newill, 1961;Clare and O'Reilly, 1960; Terzaghi, 1958; Hirashinla, 1948; and Fruhauf, 1946).

Halloysite is particularly susceptible to aggregation. Robertson (in Terzaghi, 1958) found that the halloysite in the Sasumua clay does not deflocculate like kaolinite or mortmorillonite even in efficient dispersing agents. Aggregate sizes up to 10 microns were measured. He concluded that the clay is strongly aggregated in its natural state and the aggregates will not lose their strength unless deflocculated which is unlikely to occur (in this case, in a dam fill). Halloysite soils often fall below the A-line on Casagrande's plasticity chart, indicating they are silt-like in character (Clare and O'Reilly, 1960,Terzaghi, 1958).

Tateishi (1967) has proposed the "aggregation index" as a measure of the change in physical properties of a tropical soil due to dehydration. He defined this index as the sand equivalent value of the oven-dried sample divided by the sand equivalent of the soil in its natural state. The sand equivalent test measures the relative percentage of fines in a soil (AASHO Designation T-176-65). Tateishi classified soils with an aggregation index of 1 as those which display no difference in properties with dehydration; an index of 2 would indicate a moderately sensitive soil.

2.7. Highways and Airfields

A number of authors have described the use of lateritic soils in highways. Remillon (1967, 1955) and Christophe (1949) have given details of highway construction in western and central Africa. De Medina (1964) has given details of lateritic and other papers on specific road projects Clare and O'Reilly (1960) have provided data on a number of tropical red clays used for highways.

Generally speaking, good quality lateritic soils perform satisfactorily in highways. Specifications need not be as stringent as with temperate zone soils. Plastic limits are usually higher and swelling lower than in other soils with comparable clay content. However, inferior lateritic soils have sometimes been employed which have not been adequate.

The stipulated grading specifications for the base and surfacing materials have been summarized by Ackroyd (1959) for soils of Nigeria. These grading and plasticity limits are based on the ASTM and British Road Research Laboratory specifications. Since the CBR method of pavement design is generally used, the strength requirements are based on CBR tests. A minimum CBR of 80% after 24 hours soaking for mechanically stable road-base material, using the Ghana method of compaction, is adopted in Ghana and Nigeria. Specifications regarding the use of concretionary aggregates are based on the studies by Ackroyd (1960) as a result of which gravels are rated according to strengths of the aggregates.

In Angola, ASTM Specification D1241-551 is generally accepted with the following reservations (De Castro, 1969):

1. In base and sub-base course materials the Los Angeles abrasion should not exceed 65%.
2. The requirements for maximum values of liquid limit, LL=25% and PI=6% have been relaxed to 40 and 15 respectively provided the swelling does not exceed does not exceed 10%
3. The requirements for maximum values of LL = 35% and PI = 4 to 9% for surface courses have been relaxed respectively to 40% and 6 - 15%, provided the swelling does not exceed 10%.

Airfields constructed on lateritic soils have been described in a few papers. Nixon and Skipp (1957) and Dreyfus (1952) provide general papers on various lateritic soils. Da Silva (1967) gives the details on four airfields in Angola with lateritic soil base courses that are performing satisfactorily. As with highways, lateritic soils of good quality are satisfactory for airfield construction. The same specifications can be applied.

2.8. Classification and Index Tests

The conventional test for the determination of moisture content is based on the loss of water when a soil is dried to a constant mass at a temperature between 105 and 110 ºC. In many residual soils however, some moisture exits as water of crystallization, within the structure of minerals presented in the soils particle. Some of this structural moisture may be removed by drying at the above temperature assuring the behavior of the soil. The following procedure is therefore recommended:

Two test specimens should be prepared for moisture content determinations. One specimen should be oven dried at 105 ºC until successive weighing show that no further loss of mass. The moisture content should then be calculated in normal way. The second sample should be air dried (if feasible); or oven dried at a temperature of no more than 50ºC and a maximum relative humidity (RH) Of 30% until successive weighing show that no further loss of mass.

The two moisture content results should then be compared; a significant difference (4-6% of moisture content obtained by oven drying at 105 ºC) indicates that structural water is present. This water forms part of soil solids, and should therefore be excluded from the calculation of moisture content. If a difference is detected using the two different drying process, all subsequent tests for moisture content determination (including those associated with Atterberg Limit tests, etc) should be carried out by drying at lower temperature (i.e. either air-drying, or oven–drying at 50ºC and 30% RH) if possible, the lower drying temperature of50ºC should be used (Blight, 1997).

2.8.1. Grain Size Distribution

Variations in particle-size distribution with methods of pretreatment and testing have been widely reported on laterite soils. The particle size distribution of residual soils is affected by

i. **Effect of Drying:** The most widely reported effect of drying is reduce the percentage that is reported as the clay fraction (finer than 2µm).It is accordingly recommended that drying of the soil prior to testing be avoided. Oven dried lateratic soils were found to give the least amount clay fraction, as compared to air dried or as received (natural moisture content) samples.

ii. **Chemical Pre-treatment:** If it is considered necessary to eliminate Carbonates or sesquioxides, then pretreatment with hydrochloric acid is recommended.

iii. **Sedimentation:** is essential to achieved complete dispersion of fine particles prior to carrying out a sedimentation test. The sample should be immersed in a solution of dispersant such as dilute alkaline Sodium Hexametaphosphate and therefore washed through the standard nest of sieves (Blight, 1997).

2.8.2. The Effect of Pretest Drying

The effect of air drying specimen prior to carrying out the atterberg limit tests rather than testing, starting at natural moisture content has been observed to result in a decrease in the liquid limit and plastic index (Blight 1997).

According to Townsend (1985) the effect of drying prior to testing may be attributed to

a) Increase cementation due to oxidation of the iron and aluminum Sesquioxides or
b) Dehydration of allophane and halloysite or
c) Both (a) and (b) above

2.8.3. Effects of Pretreatment in Atterberg Limits

In general, the greater the duration of mixing (i.e. the greater the energy applied to the soil prior to testing) (Blight 1997), the larger the resulting liquid limit, and the lesser extent, the larger the plasticity index this has been attributed to longer mixing resulting in more extensive breaking down of cemented bonds between clay cluster and thus the formation of greater proportion of fine particles. The number of blows to close the groove in the liquid limit cup increased from 20 after minimal mixing to 235 after 25 minutes of mixing. The soil in this case was residual mud rock. In order to address this problem the following procedure is recommended. Five test specimens should be mixed with water to give a range of moisture content suitable for liquid and plastic limit determination. The minimal amount of air drying should be used and preferable none at all. This should not be too difficult the in situ moisture content of the majority of soil is at or below the relevant plastic limit. The mixing time should be standardized at 5 minutes and the mixed specimen should be left for moisture equilibration overnight before testing (Blight 1997).

The four methods of pretreatment resulted in slight changes in the liquid limit between the AD and OD and between the AR and S but these changes were within the normal reproducible range of the liquid limit (Lyon 1971). Liu and Thornburn (1964) reported a range of limits of approximately 2% using an experienced operator under uniform laboratory conditions. Normally, 5 to 6 percent would be the average range of reproducible results.

The AD liquid limit was about 5 percent higher than the OD liquid limit. The AR liquid limit was about 3percent lower than the S liquid limit. The differences are not large compared to the reproducibility of the liquid limit test.

The OD results gave an extreme limit of the change in liquid limit, not actually encountered under field conditions. The comparison of liquid limits is shown in Figs 2.3a and 2.3b
Because of these relatively small differences, subsequent testing of moist soils was performed only on the AD and AR condition.

A soil was considered to be in the moist state if its field moisture content was about 13 percent. Some soils sampled dry were soaked to distinguish reversible trends. However, there were no significant changes in the physical properties of most soils when soaked.

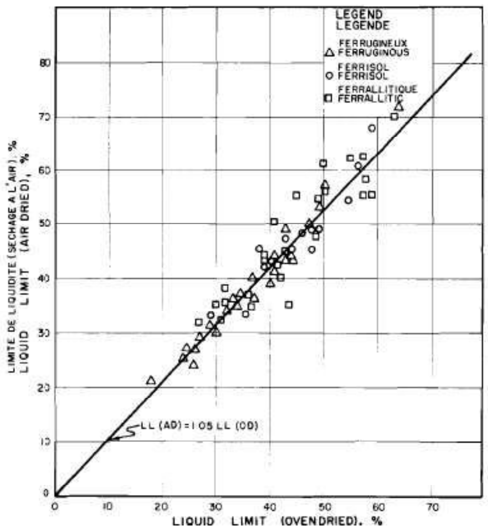


Fig 2.3a COMPARISON BETWEEN AD & OD LIQUID LIMIT

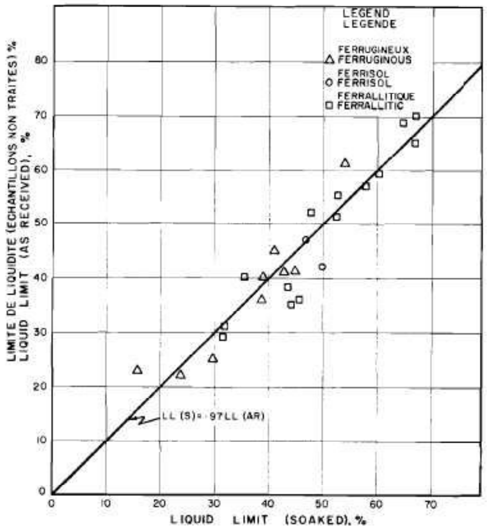


Fig2.3bCOMPARISON BETWEEN AS & S LIQUID LIMITS

2.8.4. Effect of Various Temperatures and Drying Periods

A special study was initiated to compare the effect of various temperatures and drying periods on the atterberg limits. In addition to the AD and AR pretreatment the samples were oven dried at 50^{o}c and 105^{o}c for 6 and 24 hours at each temperature. The changes in Atterberg limits when the samples were dried in the oven at the different temperatures and times, the decrease in the liquid limit and plasticity index was small. The aggregations formed during drying were easily dispersed during the soaking period. The aggregations were apparently due to only partial dehydration of the iron minerals, and thus, were dispersed during soaking.

2.8.5. Mechanical Analysis

Particle size analyses were performed on the AD and OD samples in accordance with ASTM D422-63. Wet sieve analyses were performed on AR and S samples on the portion retained on the No. 10 sieve. The dry weights used in the computations for the gradation were calculated by using the wet weight and moisture contents of the samples before sieving. The moisture content of the portion passing the No. 10 sieve was used to determine the dry weight for the hydrometer analysis (Lyon, 1971).

Remillon (1967) states that it is nearly impossible to completely deflocculate lateritic soils. Terzaghi (1958) found that when the Sasumua clay was dispersed with sodiumoxolate the percentage of particles smaller than 2 micron ranged between 20 and 30 percent and when dispersed with sodium hexametaphosphate ranged between 40 and 50 percent.

The mineralogical investigation showed that the clay contained between 70 and 100 percent clay mineral particles with a size less than 1 micron. Here again, as in the liquid limit test, complete deflocculation or disaggregation is not meaningful with tropically weathered soils. The dispersant should be standardized as well as the dispersing time.

It was decided to use sodium hexametaphosphate as a dispersing agent as recommended in ASTM. All samples were dispersed for 15 minutes, the longest time recommended in the ASTM procedures.

The test results disclosed very little change in the gradations between the OD and AD and between the AR and S samples. The differences between the AD and AR samples were not often appreciable.

2.9. Compaction

Compaction is the process of densifying and soil and reducing air voids by applying mechanical energy. Densification leads to improvement in engineering properties of strength compressibility and stiffness and to a reduction in permeability.

There are some basic scientific principles involved in the compaction process. However the means of achieving the desired degree of compaction involves the combination of technology and judgment and the recognition that soil materials are inherently variable. Efficient compaction is an art that is dependent of engineering skill and judgment. Compaction is undertaken in the field using the best available technology. The compacted product requires compromises between energy and cost expended and the value of result obtained. Engineering design must recognize the reality of what can be achieved in the field.

Most of the results available on the density and compaction characteristics of African lateritic soils have limited applicability because no systematic attempt has been made to consider all the factors in any single study (Lyon, 1971). The result is that there are no clear conclusions and some findings are conflicting. Perusal of data on the density and compaction characteristics of these soils shows that the factors which influence these properties can be separated into two groups. The first group of factors derives from the genesis of the soil while the second group derives from pretreatment procedures before testing.

The most important genetic factor is texture. The densities of indurated laterites are generally above 140 pcf. with natural moisture contents between 0% and 6%. For the gravels the maximum dry densities range between 115-145 pcf and the optimum moisture contents between 6%- 16%; for gravel sand-clays and for sand-clays the corresponding densities are 100-120 pcf and the optimum moisture contents are 16% to 24%. The maximum dry densities are generally below 100 pcf with optimum moisture contents above 24% for the clays. The variations as shown are also due to different compactive efforts.

Another genetic factor identified with the compaction characteristics is the position of the sample in the soil profile (depth of sample). The significance of this factor is elaborated upon below but it is associated with depth of desiccation in lateritic soils.

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 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.

Compaction curves developed from the air-dried or oven-dried condition display higher maximum densities and lower optimum moistures than compaction curves developed by determining points as the soil dries from its natural moisture content. Such differences can be significant for soils with in-situ moisture contents well above the optimum.

The effect of hydrated halloysite on the compaction curves has been investigated by Newill (1961). Soil from Sasumua, Kenya which had been dried produced a higher dry density than one which had not been dried. However, he considers the difference illusory and due to the loss of interlayer water in changing hydrated halloysite to metahalloysite in oven-drying the moisture samples. When corrected for the loss of interlayer water, both curves are in close agreement. It is nevertheless desirable to perform compaction tests from the natural moisture content for those soils which display irreversible changes with drying. Compaction curves so developed include the effect of the loss of interlayer water and will be valid criteria for controlling compaction during construction.

The breakdown of weak concretionary lateritic gravels during compaction also affects the compaction characteristics. Whether higher or lower densities are obtained after compaction depends on whether the breakdown leads to improvement in grading or deterioration. 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.). The conclusions reported on the effect of compaction on the particle breakdown have revealed that the reuse of concretionary lateritic soils should be discouraged (Remillon 1967) all soft concretions that do not perform satisfactorily.

Remillon (1967) used a field penetration test to determine the strength-depth variations for the ferrallitic soil and ferruginous soil profiles in western and central Africa. The variations in strength with depth are due to variation in the degree of laterization and homogeneity in the profiles.

The strength and hardness of indurated laterite is a function of chemical composition, age and homogeneity. The iron-rich indurated laterites are harder than alumina-rich ones (Nascimento et a,1959). The greater the homogeneity and the less the content of foreign bodies, the harder the material older indurated laterites are generally harder than younger ones.

2.9.1. Effects of Pretest Treatment in Compaction

A special study was undertaken to understand how the method of preparation could affect the compaction characteristics of lateritic materials (Lyon,1971). Fresh and re-used samples were tested AR, AD and OD. The samples were compacted in accordance with the Ghana compactive effort which uses a 10 pound rammer falling 18 inches with 25 blows on each of 5 layers in a 6 inch mold. The AD and OD samples were prepared in accordance with ASTM Dl 557-70 Method D. The AR samples were tested from the wet to dry state starting from the natural moisture content, or adding moisture when the natural moisture content was less than the optimum moisture content.

The results of this study indicate that the compaction characteristics of lateritic soils change after various pretreatment. Generally OD samples have higher maximum densities and lower optimum moistures than the AD or S samples. A similar relationship was reported by Grady (1949) who observed that significant variations in the compaction characteristics of Hawaiian lateritic soils appeared to be related to the natural moisture content. The drier the initial state, the higher the maximum density and the lower the optimum moisture content.

There was significant increase in the maximum density and a decrease in the optimum moisture content when the same sample was re-used (Lyon, 1971). When the re-used samples were subjected to the same pretreatment as fresh samples, the highest maximum densities and lowest optimum moisture contents were obtained with the OD soil. During compaction the gravel size particles tended to break down to sand size particles and thus improved .the gradation of the sample and increased the maximum density. The amount of breakdown of coarse gravel particles varied, but generally was less for soils compacted at the natural moisture content than for air or oven dried soils. In all cases, Fresh sample should be used for each point of compaction to avoid desegregation of particles for lateritic soils.

2.10. Lime Stabilization

Lime stabilization is one of the oldest process of improving the engineering properties of soils and can be used for stabilizing both base and sub base materials (Garber and Hoel, 2000). The addition of lime to reactive fine-grained soils has beneficial effects on their engineering properties, including reduction in plasticity and swells potential, improved workability, increased strength and stiffness, and enhanced durability. In addition, lime has been used to improve the strength and stiffness properties of unbound base and sub base materials. Lime can be used to treat soils to varying degrees, depending upon the objective. The least amount of treatment is used to dry and temporarily modify soils. Such treatment produces a working platform for construction or temporary roads. A greater degree of treatment--supported by testing, design, and proper construction techniques--produces permanent structural stabilization of soils.

Generally, the oxides and hydroxides of calcium and magnesium are considered as ‘lime’, but the materials commonly used for lime stabilization are calcium hydroxide (Ca(OH)2) and dolomite ($Ca(OH)_2$ + MgO) (Garber and Hoel, 2000). Calcium hydroxide (hydrated lime) is a fine, dry powder formed by ‘slaking’ quicklime (calcium oxide, CaO) with water; quicklime is produced by heating natural limestone (calcium carbonate, Ca(CO)3) in a kiln until carbon dioxide is driven out (Thagesen, 1996). Quicklime is also an effective stabilizer used but not usually used for stabilization because it is caustic hence dangerous to handle, susceptible to moisture uptake in storage, and gives off much heat during hydration (McNally, 1998). Dolomite used as stabilizing agent contains not more than 36 % by weight of magnesium oxide (MgO).

The percentage of lime used for any project depends on the type of soil being stabilized. The determination of the quantity of lime is usually based on an analysis of the effect that different lime percentages have on the reduction of plasticity and the increase in strength of the soil. However, most fine-grained soil can be effectively stabilized with 3%-10% of lime, based on the dry weight of the soil. Lime is used extensively to change the engineering properties of fine-grained soils and the fine-grained fractions of more granular soils. It is most effective in treating plastic clays capable of holding large amounts of water. The particles of such clays have highly negative-charged surfaces that attract free cations (i.e., positive charged ions) and water dipoles. The addition of lime to a fine-grained soil in the presence of water initiates several reactions. The two primary reactions, cation exchange and flocculation agglomeration, take place rapidly and produce immediate improvements in soil plasticity, workability, uncured strength, and load-deformation properties.

The effects of lime treatment or stabilization on pertinent soil properties can be classified as immediate and long-term. Immediate modification effects are achieved without curing and are of interest primarily during the construction stage. They are attributed to the cation exchange and flocculation–agglomeration reactions that take place when lime is mixed with the soil. Long-term stabilization effects take place during and after curing, and are important from a strength and durability standpoint. While these effects are generated to an extent by cation exchange and flocculation–agglomeration, they are primarily the result of pozzolanic strength gain (Malaga et al, 2004)

2.11. Carbonation

If cement or lime-stabilized materials are exposed to air, the hydration products may react with carbon dioxide thereby reducing the strength of the material. This reaction is associated with a decrease in the pH of the material from more than 12 to about 8.5. The presence and depth of carbonation can be detected by testing the pH of the stabilised layer with phenolphthalein indicator and checking for the presence of carbonates with hydrochloric acid. A reasonable indication of whether the material being stabilized will be subject to serious carbonation can be obtained from the wet/dry test for durability. Good curing practices, as outlined above, are the best means of preventing carbonation in road bases. The risk of carbonation can be reduced by taking the following precautions:

i. Avoid wet/dry cycles during the curing phase.
ii. Seal as soon as possible to exclude carbon dioxide.
iii. Compact as early as possible to increase the density and to reduce the permeability.
iv. Reduce the possibility of reflection cracks.

There may be some conflict between the last two points and care should be taken not to over compact the layer. Checks should be made during construction and if the depth of carbonated material is more than 2 to 3 mm the carbonated layer should be removed by heavy brushing or grading before the surfacing is applied.

2.12. Atterberg Limits

Almost all research on lime stabilization has indicated the ability of the lime to alter the plasticity of the soil. Addition of lime reduces the plasticity index. The amount of reduction varies directly with the quantity of lime added and with the type of clay mineral and organic matter present in the soil. The liquid limit normally decreases with increasing amounts of lime (Herrin and Mitchell, 1961), although the converse has also been reported for some soils.

Samples were prepared in sealed plastic bags at stabilizer content for 7 day curing periods. The samples were prepared at about the plastic limit (PL) and water contents were checked during the curing periods. Water was added when necessary to allow proper action between the stabilizer and soil. A minimum of three plastic limit determinations were made for each sample. Liquid limits (LL) were determined in accordance with B.S. 1377
All samples were cured at approximately 100% relative humidity at temperatures varying between 21°c and 23°c.

In the tests performed for previous study, almost all the lateritic samples tested experienced a decrease in liquid limit with the addition of lime. The only two soils that showed an increase of the liquid limit were soils with sandy parent material. However, the increase of the liquid limit was not as great as the accompanying increase in plastic limit, and the plasticity index decreased accordingly. Increases in the liquid limit occurred in the majority of soils stabilized with cement.

An increase in the liquid limit of soil stabilized with lime may be due to calcium saturation of the clay particles. This explanation could apply to the lateritic soils studied. The phenomenon may be caused by flocculation and cementing of the individual particles, where upon the soils becomes silty and stiffens therefore it flows with great difficulty in the liquid limit cup.

Clare and Cruchley (1957) reported that an increase in the liquid limit of a soil stabilized with lime may be due to calcium saturation of the clay particles.

There was very little difference in liquid limit between the 7 and 28 day curing periods for lime or cement stabilized soils.

The plasticity indexes of lateritic soils are higher than normal soils with similar strength characteristics. In almost all cases where comparative data were obtained, a greater reduction in the plasticity index occurred with the addition of lime than with cement. This finding is in agreement with previous research with both types of stabilizers. Lime as compared with an equal weight of cement has more calcium available for cationic exchange. This reaction will alter the water holding capacity of the clay to form a less plastic soil.

The plasticity index of the natural soils ranged from 10 to 34. All the other materials tested showed a decrease of the plasticity index to less than 15, except for one soil which exhibited a reduction from 31 to only 20 with 8 percent lime.

The ferruginous soils and soils derived from sandy parent materials all showed a great reduction in the plasticity index with the addition of 2% lime. Higher percentages of lime resulted in smaller reductions. The other soils showed a steady reduction in the plasticity index with increasing lime content. All cement treated materials showed steady reduction

2.13. Wet Dry Durability

In most tropical countries the durability requirements of soil-cement mixtures are assumed to be satisfactory if the stabilized soil is in conformance with their minimum strength requirements. Stabilized with cement, for which the 7-day compressive strength was 150 pounds per square inch, which possessed adequate stability to withstand the wet-dry durability test.

The assessment of the wet-dry durability properties of lime-soil mixtures by means of laboratory tests is difficult. Results from standard wet-dry tests using brushed samples, when compared to criteria developed for soil-cement mixtures, indicate that lime-soil mixtures have little resistance to weathering. However, field research has shown that lime-soil mixtures when protected from weathering by a bituminous wearing surface are more durable than indicated by most laboratories.

2.14. Compaction

The changes of maximum density and OMC for those soils stabilized with lime or cement are similar to those reported for other soils, including those in temperate zones and other lateritic soils (Lyon Associates, 1971). The maximum density decreased with an increase of lime content. With the addition of 2% lime, the maximum density decreased from about 1 to 6 pounds per cubic foot. With additional lime the maximum densities decreased further, up to 8 pounds per cubic foot less than the natural maximum density. The OMC increased from 1 to 2 percent with 2% lime, but further addition of lime produced little increase in most soils.

The changes in maximum density and OMC for soil cement were smaller than for lime. The most significant change took place with the addition of 2% cement, but with increased cement contents the changes were small. Except for soils with the maximum densities and OMC did not vary by more than 3 pounds per cubic foot and 2 percent moisture.

2.15. California Bearing Ratio (CBR)

For the lime-treated soils there did not seem to be any distinction in the behavior of samples from the different lateritic soil groups. There was some distinction with respect to cement-treated soils, as will be discussed subsequently (Lyon A,1971).

The CBR values for cement treated soil were much higher than for lime treated soil. In most cases the highest CBR value with lime treatment was less than the 28 day CBR for 2% cement content. In no case was the highest soil-lime CBR value more than the 28 day CBR value for 4% cement by weight.

The CBR value for most of the lime treated soils increased linearly with lime at low lime contents, but there was a tendency toward a smaller increase or even a slight decrease in strength with higher lime contents. Peaks in CBR values occurred at about 4 to 6 percent lime by weight. There were small increases in strength between the 7 and 28 day curing periods.

The smallness of the gain in strength may be due to the lack of pozzolanic material in the natural soil. Usually aluminous and siliceous materials in soil react with the lime.

Lateritic soils are usually low in siliceous materials, which might explain the lack of increase in strength with longer curing periods. Herrin and Mitchell(1961) state that neither base exchange capacity, the pH of the soil nor the plasticity characteristics seem to be satisfactory indicators as to when a soil will give large strength gains with lime.

Further research may be necessary to determine whether the addition of a pozzolan, such as fly ash, might be helpful to aid the cementing action in lateritic soils. Irwin (1958) found that after 26 weeks of curing, lateritic gravels from Uganda stabilized with 4 percent of lime were approaching the 7 day strength of the same gravels stabilized with 4 percent of cement.

Hayter and Cairns (1966) reported a very marked gain in strength between 7 and 28 days for a lime treated lateritic soil. They did not give any explanation for this reaction. The long time required for the gain in strength is due to the slow cementing reaction. The time may be accelerated with the addition of pozzolans.

Most of the cement-treated soils in the present study developed high CBR values with low percentages of cement. The values generally increased linearly with increase of cement content, although some soils exhibited smaller increases at 4 to 6 percent. Sherwood and Sullivan (1960) observed smaller measures when testing Gambian lateritic gravel at optimum moisture content. They showed that the decreased rate was due to insufficient water being available for the larger quantities of cement to hydrate fully. When the gravel was tested at higher moisture contents, the unconfined compressive strength increased linearly with an increase on cement content.

On the basis of the CBR test results it appears that most lateritic soils can be successfully stabilized as base course material with 3 to 4 percent cement. Many of the lime-treated soils did not reach a CBR value of 180 even with the higher percentages of lime, but could be considered for sub-base. Lime treatment could also be considered for improvement of sub-grade soils.

2.16. Curing Time

Lime addition causes significant improvement in a short time in clayey soil properties by reducing plasticity and increasing strength of soil. Bell (1996) indicated that the optimum gain of strength in clay is achieved with the lime addition of 4-6%. The increase in strength is affected by the amount of mix water, the curing time length and the temperature at which curing takes place. When the mix water content is just in excess of the optimum moisture content, the gain in strength is the highest. According to the tests conducted by Bell, the most notable increase of strength occurs within the first 7 days (Bell,F.G,1996)

2.17. Related Researches

2.17.1. The Potential of Lime Stabilization of Lateritic soil Ola,S.A,1977

The potential of lime stabilization of Lateritic soil Ola.S.A, 1977.The paper reviews lime stabilization of lateritic soils and shows that all Nigerian lateritic 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 plasticity indices of the soil were reduced whereas the plastic limit increased the liquid limits increased slightly, the maximum dry density decreased and the optimum moisture content increased.

From the results of durability and CBR tests only the A-1 soil and A-2 soil have any potential as competent base material and consequently, only these require any further field tests.6% lime is recommended for these field tests. The other soils that do not qualify for bases may be utilized as base material.

2.17.2. Stabilization of Expansive Soil Using Bagasse Ash and Lime (Meron W and Samuel T, 2014)

The Expansive soil collected from Addis Ababa, Bole sub city, classified as an A-7-5 soil on the AASHTO classification was stabilized using 3% lime, 15% bagasse ash and 15% bagasse ash in combination with 3% lime by dry weight of the soil. The effect of the additives on the soil was investigated with respect to plasticity, compaction and California bearing ratio (CBR) tests.

The results obtained indicate an increase in optimum moisture content (OMC) and CBR value; and a decrease in maximum dry density (MDD) and plasticity of the soil for all additives. But there was also a tremendous improvement in the CBR value when the soil is stabilized with a combination of lime and bagasse ash. This shows a potential of using bagasse ash as admixture in lime stabilized expansive soil.

2.17.3. Effects of Cement Lime Mixes on Lateritic Soils for Use in Highway Construction (M.A Rahman, 1987)

The study describes the effect of cement lime mixes on the geotechnical properties of lateritic soils and recommends the optimum mix proportion of cement and lime for use in highway construction. The following geotechnical properties of the stabilized lateritic soils were measured. Atterberg limits, compaction characteristics, Unconfined compressive strength and Compressive characteristics the test results shows that cement lime mixes can be utilized for base material in highway construction with a combination of 3% lime and 3% cement.

3. WOLAYITA SODO AREA DESCRIPTION AND IN-SITU PROPERTIES

3.1. Wolayita Sodo Area Description

Wolayita Sodo is a town and separate Woreda in South-Central Ethiopia. It is the administrative center of the Wolayita zone of the Southern Nation, Nationalities and Peoples Region. Two trunk roads connect the town with Addis Ababa through Shasemene 390km, through Butajera Hossana, 307kms. The town is located at a junction of four roads connecting the nearby towns of Arbaminch, Hossana, Jimma and Sawla.

3.1.1. Geography

The soil around Wollayta is of heavy red color as shown in Fig 3.1. It becomes brown and dark during the rains and has the fragility and the softness of sand. The dry period makes the soil hard as brick, this is one of the reasons why people can plough and dig only after the rains. One can hardly find any stones except along the river banks, where the soil is light and easy to excavate. The soil cover is very thick with an average thickness of 30 meters in both the plains and the hills, as verified during the drilling of wells.

3.1.2. Climate

There are two prominent rainy seasons; the “Belg” season with infrequent rains generally fall between March and May and the heavy rains of “Kiremt” occur between June and September. The mean annual temperature and rainfall are obtained from National Metrological Service Agency.

The Mean annual rain fall ranges between 938.2 - 1619.2 mm

The Mean annual maximum temperature ranges between 24.8 - 25.8ºc

The Mean annual Minimum temperature ranges between 13.8 – 14.9ºc

3.1.3. Soil and Geology

When visually inspected almost similar soil color, predominantly red brown soil, starts from around Bodity town which is 370 km from Addis Ababa, dominantly covers Soddo – Houssana, Soddo-Chida and Gamo-Goffa road direction. These are alkaline and peralkaline stratoidsilicics, ignimbrites, unweldedtuffs, ash flows, rhyolites, domes and trachytes. The soils range in age between 2 and 9 million years and mostly located on the escarpments. The soil flow thicknesses vary widely, from 1 to 30 meters on the plateau and up to 250 meters in rift (Scott, 1998).

Fig 3.1 Site Photograph

3.1.4. Geochemical tests and X-Ray diffraction (XRD) test

Geochemical test were Conducted at the Geochemical Laboratory of the Geological Survey of Ethiopia by (Tibebu,2008) Atomic Absorption Spectrometer and Colorometer Analysis methods were used to get the percentage of oxide composition of the soils under investigation.

The degree of laterization of the soil samples can be evaluated based on the ratio of Silica/Sesquioxides

$$\frac{SiO_2}{R_2O_3} = \frac{SiO_2}{Fe_3O_3 + Al_2O_3}$$

The Sesquioxide, designated as R_2O_3, is the combination of Aluminum oxide (Al_2O_3) and Iron oxide (Fe_2O_3). The chemical formula, SiO_2 designates the silica. Ratios less than 1 have been considered as true laterites, those between 1.33 and 2.00 as lateritic soils and those greater than 2.00 as non-lateritic tropically weathered soils (Blight,1997).

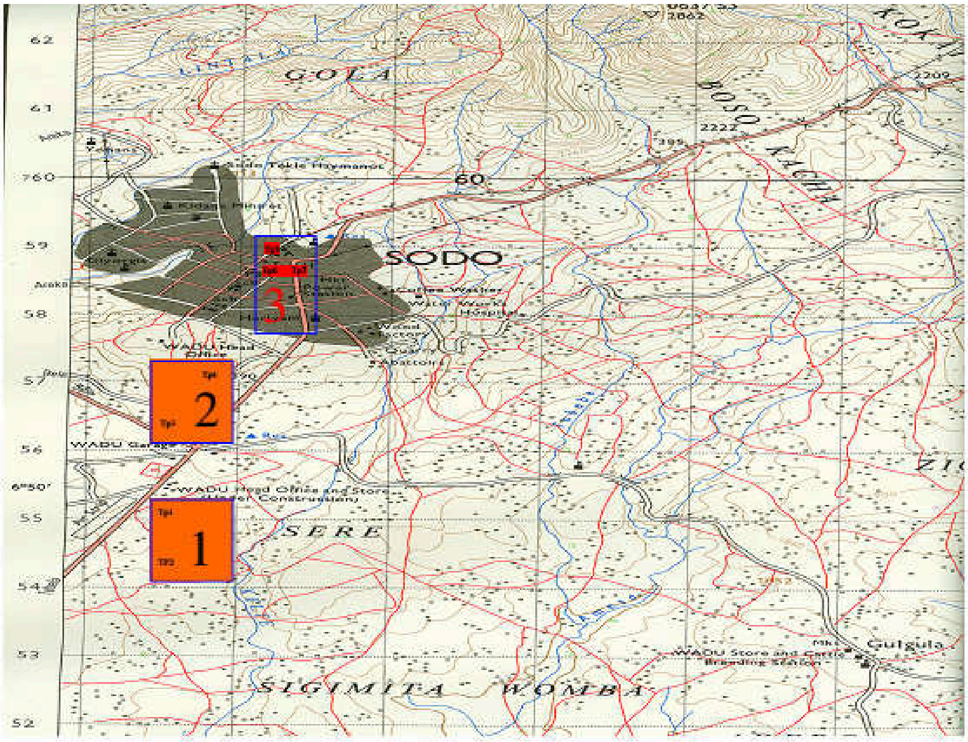


Fig 3.2 Test pit location map of Wollayita-Sodo (Hanna.T,2008)

The test results show that Silica – Sesquioxide ratio are all below 1.33 except for one test pit out of seven pits. This indicates that the soils are between true laterites and lateritic soil except for one test pit which has a ratio of 1.5 which is lateritic. True laterites are simply referred as laterites. The soil of such kind are highly laterized i.e., Sesquioxides content are high.

The degree of laterization increases with depth for TP4 and TP5 while for TP2 degree of laterization almost similar with depth Fig 3.2.

3.1.5. X-Ray Diffraction

X-Ray Diffraction is the most widely used technique for the identification and characterization of clay minerals. Clay minerals consist of tiny crystals which are themselves made up of ordered arrays of atoms, arranged in periodic or repetitive way. Using X-ray diffraction method the major constituent minerals were determined.

These are Kaolinite, Halloysite and Montormorillonite refer to Table 3.1. XRD test results were taken from previous study (Tibebu,2008). It was undertaken at the Geochemical and Petrography Laboratory of the Geological Survey of Ethiopia using X-Ray Diffraction analysis by powder diffraction method after grinding the bulk soil sample without separating the fraction powder diffraction method after grinding the bulk soil sample without separating the fraction.

Table 3.1 Mineralogical composition of Sample (HannaT, 2008)

Test Pit	Sampling depth (m)	Mineral Identification	Chemical Formula
Tp1	-2m	Quartz	SiO_2
		Magnetite	Fe_3O_4
		Kaolinite	$Al_2(Sio_5)(OH)_4$
		Hematite	Fe_2O_3
TP2	-2m	Low Quartz	SiO_2
		Magnetite	Fe_3O_4
		Kaolinite	$Al_2(Sio_5)(OH)_4$
		Hematite	Fe_2O_3
TP4	-2m	Phosposiderite	$FePO_4(H_2O)$
		Kaolinite	$Al_2(Si_2O_5)(OH)_4$
		Hematite	Fe_2O_3
		Low Quartz	SiO_2

3.2.Sample Collection for Geothechnical Tests

The soil specimen for this thesis work was collected from Wolayita Sodo which was previously studied by Tibebu, 2008. For the purpose of this study, three different areas were selected.

In this study, one of the previously studied areas was selected, where two test pits were opened. The selected area is located near the agricultural research office (WADU) behind Wollayita Sodo University. From these test pits, which are almost 100m apart, about 300kg of disturbed samples were collected from a depth of 80cm to avoid the top soil. Other samples were packed in tight plastic bags and covered with moist towel to maintain the in situ moisture. They are used for moisture content determination.

4. LABORATORY TEST RESULT AND INTERPRETATIONS

4.1. Moisture Content Determination

Two test specimens were prepared for moisture content determination. The first sample was oven dried at 105°c until successive weighing shows no further loss of mass. The moisture content then calculated according to ASTM. The second one was air dried for about 15 days until successive weighing shows no further loss of mass. The difference detected is greater than 4% as shown in Table 4.1 below which means that the soil under investigation still contains loosely bound water of hydration. However, as the differences are not that high it is assumed that oven drying will not affect the moisture content determination significantly. Therefore, oven drying was used for subsequent tests like Atterbergs limits, specific gravity, free swell etc. at a temperature of 105°c.

Table 4.1 Moisture content comparison of oven dried and Air dried sample

Sample Description	Depth	Condition	Moisture content	Difference
TP1	80cm	Oven dried at 105°c	28.5	5.93
		Air dried	22.57	
TP2	80cm	Oven dried at 105°c	27.8	5.65
		Air dried	22.15	

4.2. Atterberg Limits

Atterberg limits of lateritic soils are affected by test procedures such as excessive manipulations during testing as this leads to crumbling and disaggregating of the soil structure producing fines resulting in higher liquid limit points (Tibebu,2008). In order to reduce these effects the soil was broken down by soaking in distilled water and not by drying and grinding.

The soil was immersed in distilled water to form slurry, which is then washed through a 425 µm sieves until the water runs clear. The material passing the sieve is collected and used for Atterberg limit test. The mixing time for liquid limit test was kept to a minimum generally about 5 min for each limit points and the mixed samples were left for 24 hours before testing for moisture equilibrium. A fresh mixed sample was used for each limit points (Blight,1997). Atterberg limit test results of the non-stabilized soil is shown in Table 4.2.

Table 4.2Atterberg limit test results of un-stabilized soil

Test pit No	Depth	Liquid limit (%)	Plastic limit (%)	Plastic Index (%)
Tp1	80cm	56	34	22
Tp2	80cm	55	35	20

4.3. Free Swell

Soil mineralogy and structure, fabric and several physico-chemical aspects of the soil determines the amount of swelling and magnitude of swelling pressure. From clay minerals Montimorillonite affects the magnitude of swelling compared to illites and kaolinites (Haile Mariam, 1993).

The tests were performed according to ASTM, sample that passed No 40 sieve, oven dried at 100±5°c. A 10ml sample is placed in graduated cylinder filled with distilled water and letting the content stay for 24 hours and the final volume is recorded, Free swell was determine as :-

$$Free\ Swell = \frac{Final\ Volume - Initial\ Volume\ of\ the\ Soil}{Initial\ Volume}\ x\ 100\%$$

From the test table 4.3 below one can see that the free swell of the soil under investigation are below 50% which indicates that sample has low degree of expansion.

Table 4.3 Free swell values

Sample Designation	Free Swell (%)
Tp1	25
Tp2	30

4.4. Specific Gravity

Specific gravity of the soils samples under investigation was determined using ASTM test designation D854 – 92 procedures. The specific gravity is used to calculate parameters such as clay fraction, void ratio and porosity. In residual soils the specific gravity maybe unusually high or unusually low (Blight, 1997).

The specific gravity values of an oven dried sample is less than the specific gravity result of as received and air dried sample. Hence specific gravity significantly changes upon drying prior to testing and oven drying temperature (Tibebu,2008).Pre-test drying at higher temperature should be avoided as it tends to reduce the measured specific gravity (Blight 1997).

The soil in the test was air dried. The measured specific gravity is as shown in the Table 4.4. The results obtained are within the common range for lateritic soil from2.6 to 2.9.

Table 4.4Specific gravity test result

Sample Designation	Specific gravity
Tp1	2.70
Tp2	2.65

4.5. Grain Size Analysis

4.5.1. Wet Preparation

If the soil contains substantial quantities say more than 5% of fine particles a wet sieve is required with representative quantity of soil (Arrora,K.R,2000). For grain size distribution sample were air dried and wet sieved through No 200 sieve with a No 52 sieve placed on top for protection, washing was done until the water runs clean then the remaining sample was oven dried and dry sieved. For sample finer than 0.0075 sieve hydrometer test with dispersing agent is carried out according to ASTM. The combined grain size distribution curves are as follows in Fig 4.1and 4.2.

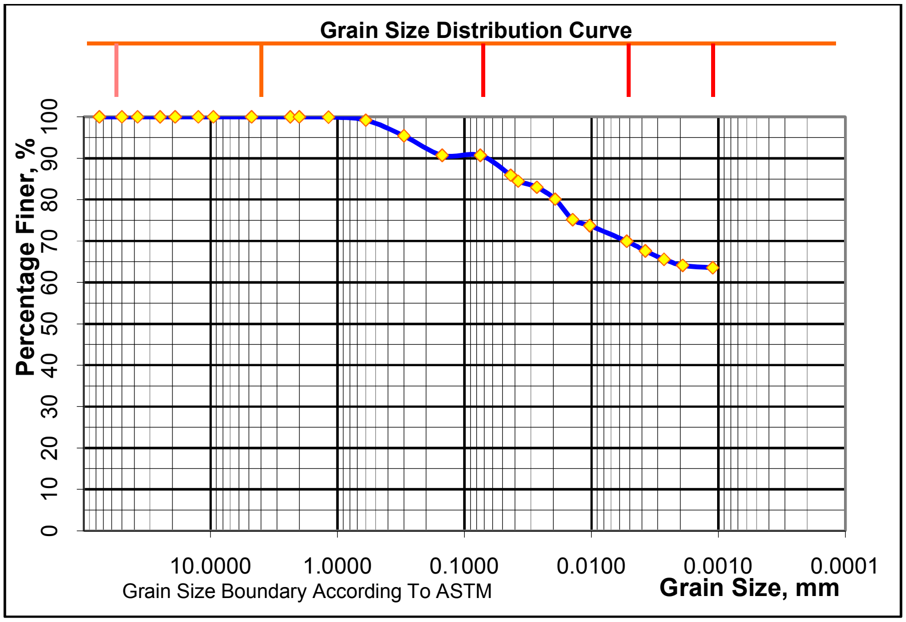


Fig 4.1 Grain size distribution curve for TP1

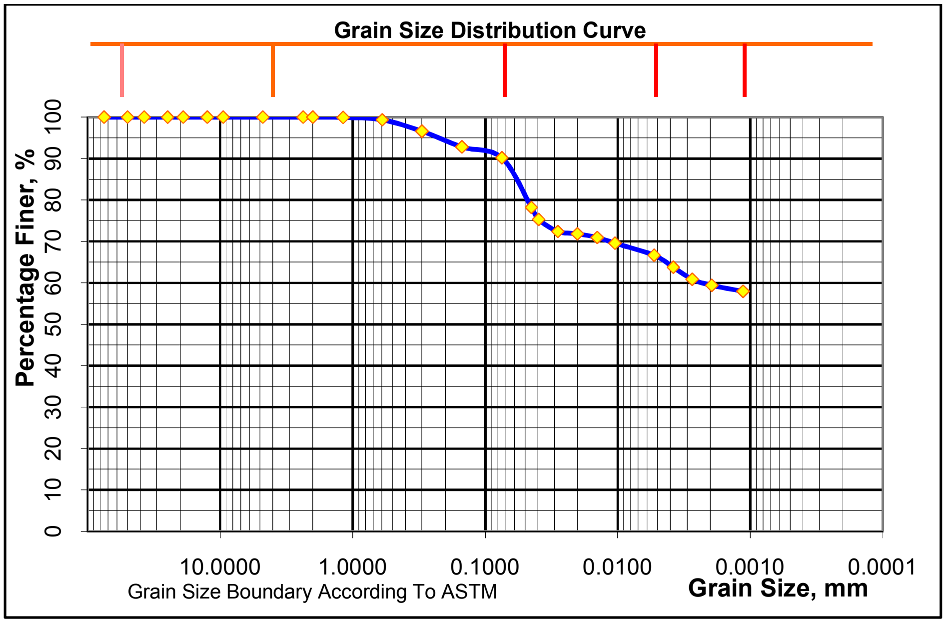


Fig 4.2 Grain size distribution curve for TP1

4.6. Classification of Soils

The AASHTO classification system is convenient as a basis for classifying tropically weathered soils. Some road construction authorities use conventional soil classification system using the grain size distribution and the Atterberg limits as shown in Table 4.5 .The soils under investigation have been classified according to AASHTO M-145 and USCS.

Table 4.5 AASHTO Classification

General Classification	**Granular Materials (35 per cent or less passing No. 200)**							**Silt-clay Materials (More than 35 percent passing No. 200)**			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7 A-7-5 A-7-6
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				
Sieve analysis per cent passing											
No. 10	50 max										
No. 40	30 max	50 max	51 min								
No. 200	15 max	10 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40 sieve											
Liquid limit	6 (max)		N.P	40 (max)	41 (min)	40 (max)	41 (min)	40 (max)	41 (min)	40 (max)	41 (min)
Plasticity - Index				10 (max)	10(max)	11 (min)	11 (min)	10 (max)	10 (max)	11 (min)	11 (min)
Group index	0		0	0		4 (max)		8 (max)	12 (max)	16 (max)	20 (max)
Usual types of significant constituent materials	Stone fragments gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as sub- grade	Excellent to good							Fair to poor			

According to USCS plasticity chart places both samples below the A line and on the chart PI versus LL both samples fall in MH (inorganic silt).The soil under investigation fall under group A-7-5(GI=20) according to AASHTO.

4.7. Compaction

The compaction characteristics of residual soils may be very dependent on the method of applying the compactive energy. Compaction often results in progressive break down of quartzitic gravel and lateritic gravel particles both residual from weathering of granite. Therefore it is imperative to use fresh soil sample to establish each point on compaction curve, otherwise the compaction characteristics of the soil will change progressively as the compaction test proceeds, and the test result may be meaningless (Blight, 1997).Refer Table 4.6 below for compaction test results of MDD and OMC of the non-stabilized soil using fresh soil sample for each point in compaction curve.

Table 4.6 Compaction result of non-stabilized soil

Sample Designation	Maximum dry density(MDD)Kn/m^3	Optimum Moisture Content(OMC) %
Tp1	12.81	31
Tp2	13.6	27

4.8. Lime Stabilization

The percentage of lime used for any project depends on the type of soil being stabilized. The determination of the quantity of lime is usually based on an analysis of the effect that different lime percentages have on the reduction of plasticity and the increase in strength of the soil (Lyon A,1971). However, most fine-grained soils can be effectively stabilized with 3%-10% of lime, based on the dry weight of the soil. Lime is used extensively to change the engineering properties of fine-grained soils and the fine-grained fractions of more granular soils. It is most effective in treating plastic clays capable of holding large amounts of water. The particles of such clays have highly negative-charged surfaces that attract free cations (i.e. positive charged ions) and water dipoles. The addition of lime to a fine-grained soil in the presence of water initiates several reactions. The two primary reactions, cation exchange and flocculation agglomeration, take place rapidly and produce immediate improvements in soil plasticity, workability, uncured strength, and load-deformation properties.

The hydrated lime used in this study is commercially available atDerba Cement Ethiopia and contains more than 90% of $Ca(OH)_2$. The composition of the lime used for this study is as shown in Table 4.7.

Table 4.7 Hydrated lime Composition

Lime components	Composition
SiO2	2% max
Al2O3	1% max
FeO3	1% max
Ca(OH)2	90% min
MgO	1% max
Dead burnt	1% max
Specific gravity	2.2

4.8.1. Effects of Lime Content on Atterberg Limits

Atterberg limits were determined for uncured and 7 days cured lime stabilized soil samples in accordance with AASHTO T89-90 and T90-96 testing procedures. The samples were prepared in sealed plastic bags for 7 days curing periods.

The samples were prepared at about the plastic limit (PL) and water contents were checked during the curing periods. Water was added when necessary to allow proper action between the stabilizer and soil. A minimum of three plastic limit determinations were made for each sample. Liquid limits (LL) were determined in accordance with B.S. 1377

The effects of lime content on the Atterberg limits are shown in Table 4.8 and graphically in Fig 4.3a and 4.3b for the sample TP1 and TP2.

Table 4.8 Plastic index versus Lime content

Test pit Designation	Lime Content	Liquid Limit (LL%)	Plastic Limit (PL%)	Plastic Index (PI%)
TP1	0	56	34	22
	2	53	34	19
	4	49	41	8
	6	59	39	20
	8	59.4	39	20.4
TP2	0	54	34	20
	2	45	33	13
	4	46	39	7
	6	57	39	18
	8	58	39	19

The results show that, with an increase of lime content the liquid limit decreased and the plastic limit increased proportionally up to 4% lime content. As a result, the PI reduced to 8% and 7% for TP1 and TP2 respectively.

Whereas above 4% lime content the liquid limit increased and the plastic limit showed constant value, as a result the plastic index increased proportionally.

An increase in the liquid limit of lime stabilized soils may be due to calcium saturation of the clay particles (Lyon. A, 1971) this explanation could apply to the lateritic soils studied. The phenomenon may be caused by flocculation and cementation of the individual particles, where upon the soils become silty and stiffens. Therefore, it flows with great difficulty in the liquid limit cup.

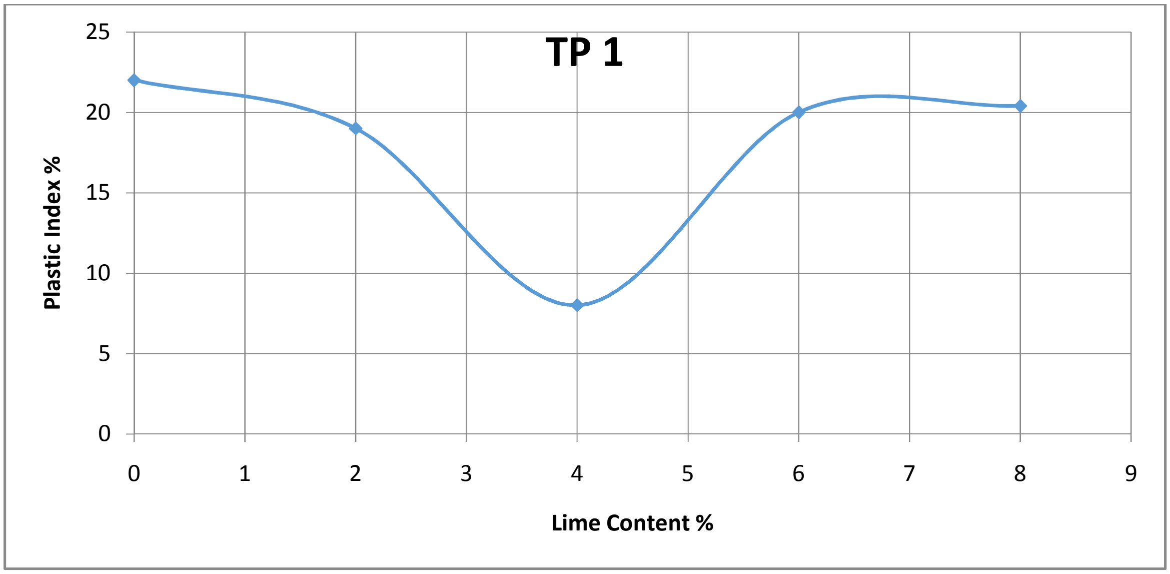


Fig 4.3a Variation of Plastic Index with Lime Content

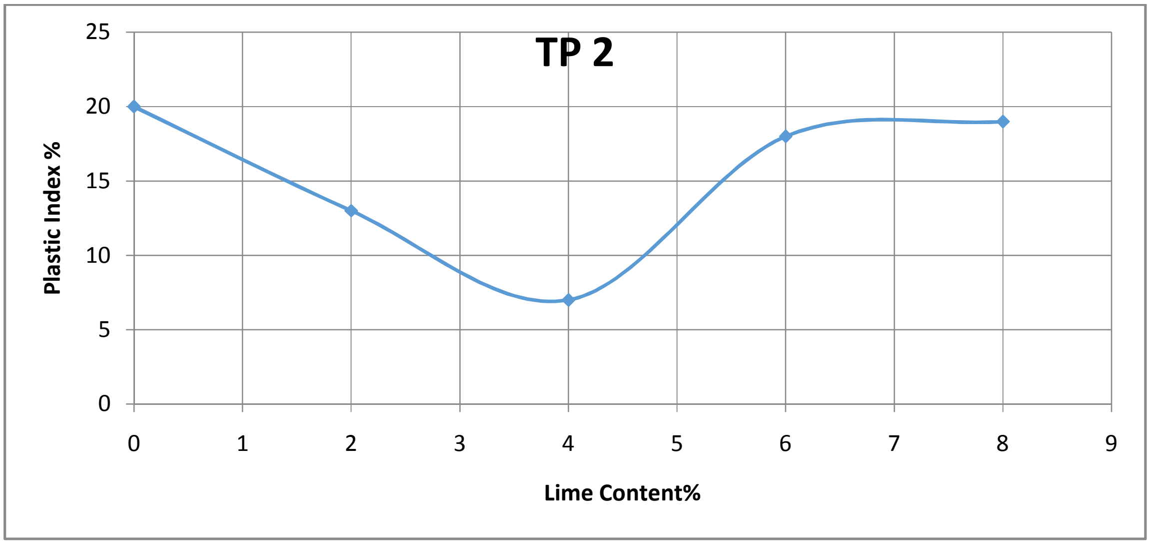


Fig 4.3b Variation of Plastic Index with Lime Content for TP2

4.8.2. Compaction with Lime

The mixing procedure prior to compaction was as recommended by Portland Cement Association for soil cement. The soil was air dried, pulverized and divided into batches before each batch was thoroughly mixed with lime to uniform color in dry state as shown in Fig 4.4. Water is then added as rapidly as practicable before compaction. The tests are conducted for uncured and 7days cured stabilized soil samples with varying percentages by weight of lime in accordance with AASHTO T99-94 testing procedures.

Fig 4.4 Picture of sample blending with lime for compaction in dry state

Tables 4.9 and 4.10 show results of the maximum dry density and the optimum moisture content with varying percentage of lime for the uncured and 7 days cured samples. In general, Increase in lime content decreases maximum dry density and increase the optimum moisture content with the same compactive effort.

Table 4.9 Uncured Compaction results for increasing percentage of lime

Test pit Designation	Lime Content	MDD(Kg/m^3)	OMC(%)
TP1	0	1281	31
	2	1314	30
	4	1237	35
	6	1265	37
TP2	0	1357	27
	2	1253	30
	4	1258	33
	6	1280	35

Table 4.10 Compaction results for increasing percentage of lime for 7 days of curing

Test pit Designation	Lime Content	MDD(Kg/m^3)	OMC(%)
TP1	0	1281	31
	2	1310	32
	4	1240	37
	6	1280	38
TP2	0	1360	27
	2	1290	31
	4	1230	37
	6	1263	39

This can be linked to the additional water needed to enable the pozzolanic soil-lime reaction necessary for the stabilization process. The OMC increased constantly along with the percentage of lime, while MDD decreased as the percentages of lime increased to an optimum value after which it increases suddenly and falls again. This maximum value represents the optimum percentage of lime required for stabilization.

The compaction curves are shown in Fig 4.5 and Fig 4.6for uncured and 7 days cured samples respectively. The details of the MDD and OMC results are shown in the Appendix.

For sample TP1, the OMC increased from 31% at 0% lime to 37% at 6% lime content with a corresponding decrease in MDD from 1281 to 1260kg/m3 and sudden increase at 2% lime to 1320kg/m3 and decrease to 1240at 4% lime content and increased to 1260kg/m3 at 6% which in this case the optimum value for TP1 obtained at 4% lime percentage.

The OMC of sample Tp2 increased from 27% to 35% with Corresponding decrease of MDD from 1360 to 1280kg/m^{3}at 4% of lime MDD reduced to 1264kg/m^3 and increased to 1280kg/m3 at 6% Therefore, 4% lime is the optimum lime content for both cases.

The MDD of 7 days cured sample is slightly less than the uncured sample. Whereas the OMC of cured sample is almost constant.

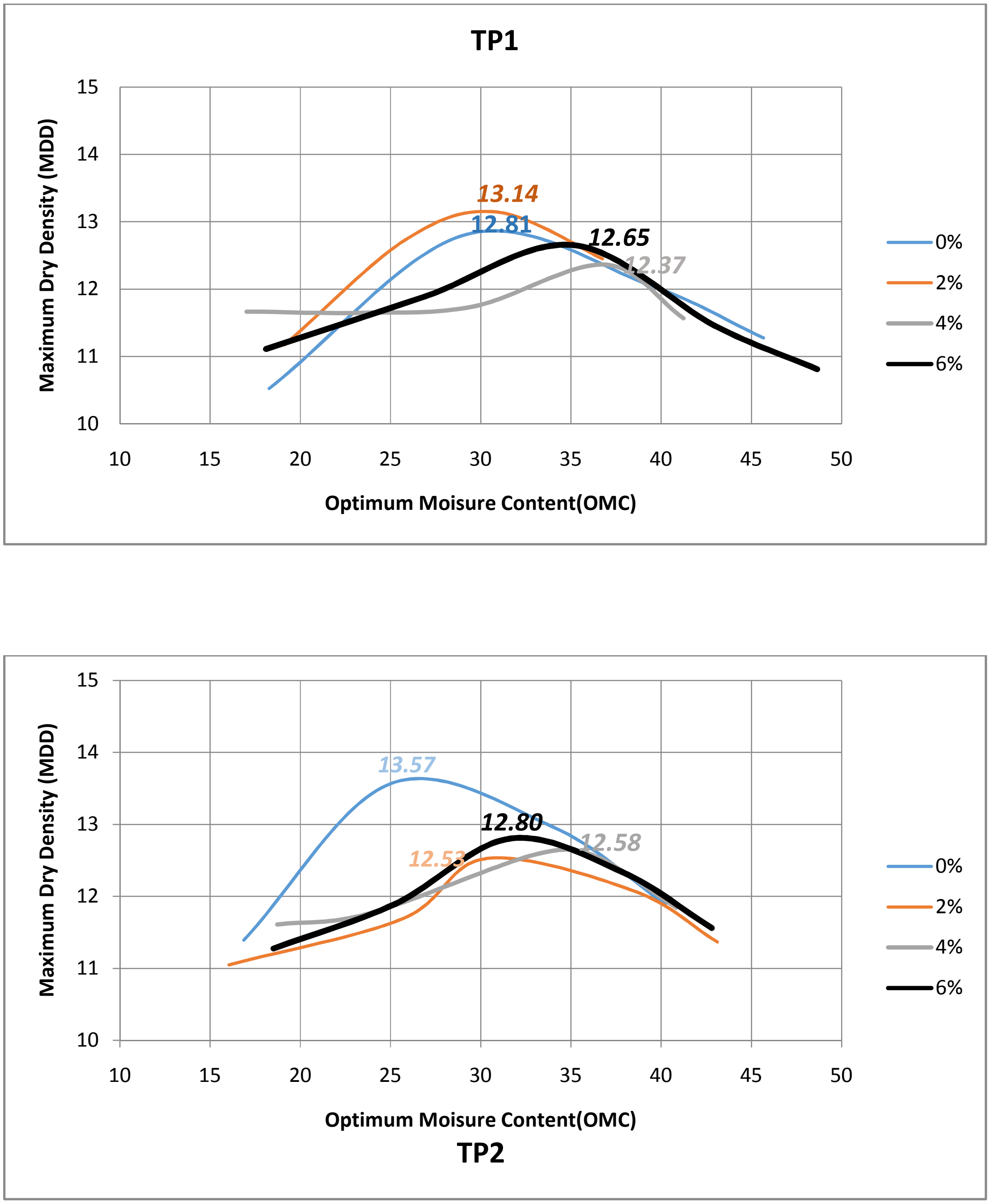


Fig 4.5 Compaction versus Lime Content for 0 days of curing

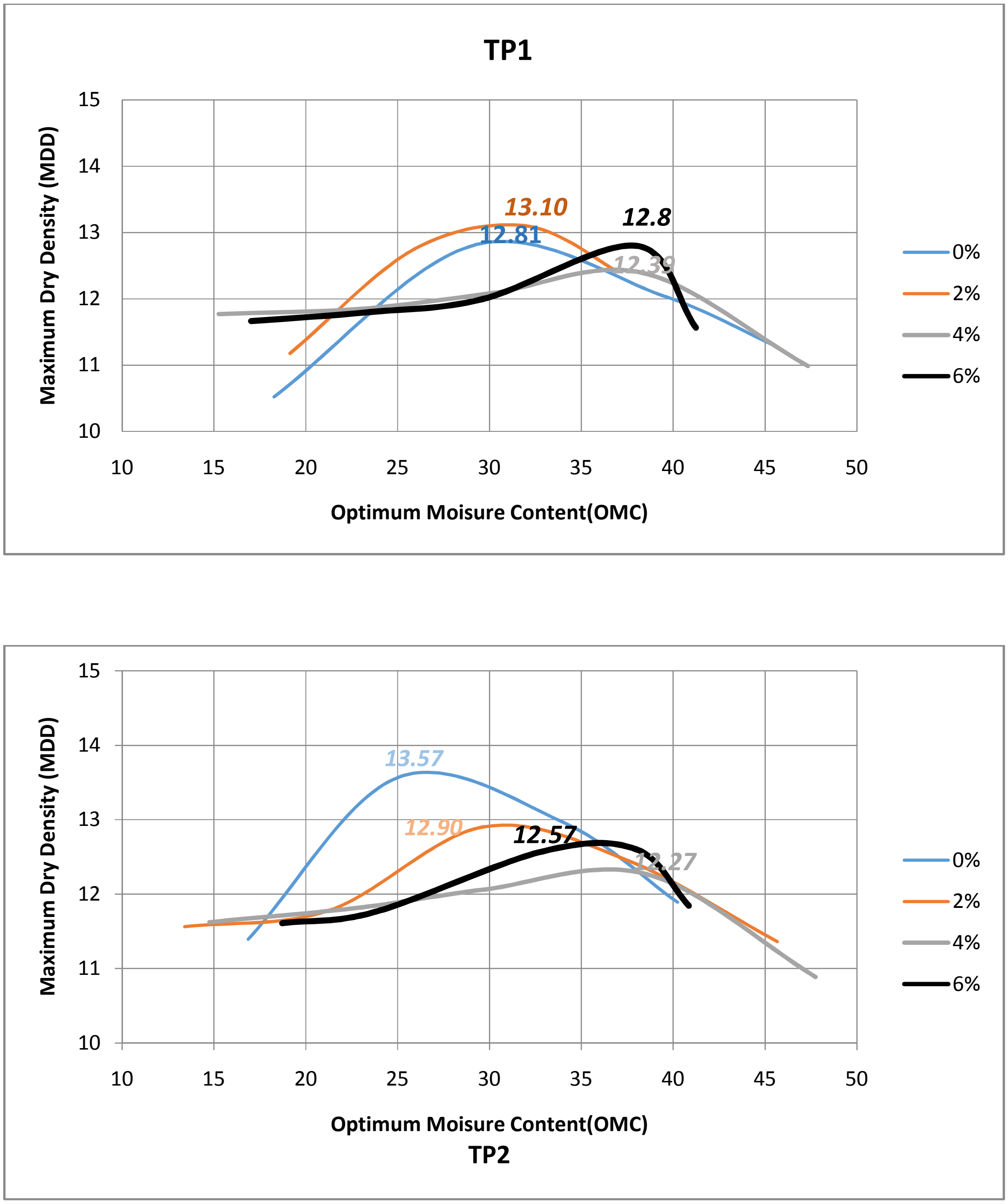


Fig 4.6 Compaction versus Lime Content for 7 days of curing

4.9. Unconfined Compression Test (UCT)

The Unconfined Compression Test is a special case of the un-drained test. No confining pressure is applied to the soil specimen throughout the test. The test can be performed by applying a load in a simple loading frame. At the start of the test, the unsaturated soil in specimen has negative pore – water pressure, and pore air pressure is assume to be atmospheric. The soil matric suction (ua-uw), is therefore numerically equal to pore –water pressure.

The soil specimen is sheared by applying an axial load and failure is reached. The Deviator stress, (s_1-s_3), is equal to the major principal stress and, s_3, is equal to zero. The compressive load is applied quickly in order to maintain conditions. This should apply to both in pore – air and pore – water phases. The pore - air and pore – water pressures are not measured during compression. The excess pore pressure developed during Unconfined Compression Test can be theoretically related to the major principal stress through use of the D or B pore pressure parameter (Fredlund,1993).

4.9.1. Test procedure for Unconfined Compression Test (UCT)

The Unconfined Compression Test procedure similar to the UU test procedure, except that no confining pressure is applied to the specimen (i.e.s_3 is equal to zero).The test is commonly performed in a simple loading frame by applying an axial load to the soil specimen (Fredlund, 1993), Fig 4.7 shows samples after unconfined compression tests.

Fig 4.7 Samples in Unconfined Compression test

Table 4.11 shear strength parameters of UC test with Lime content

Lime content (%)	Unconfined compressive strength (qu) kPa	
	Tp1	Tp2
0%	341.05	297.151
2%	343.144	311.242
4%	371	391.823
6%	312.044	369.823

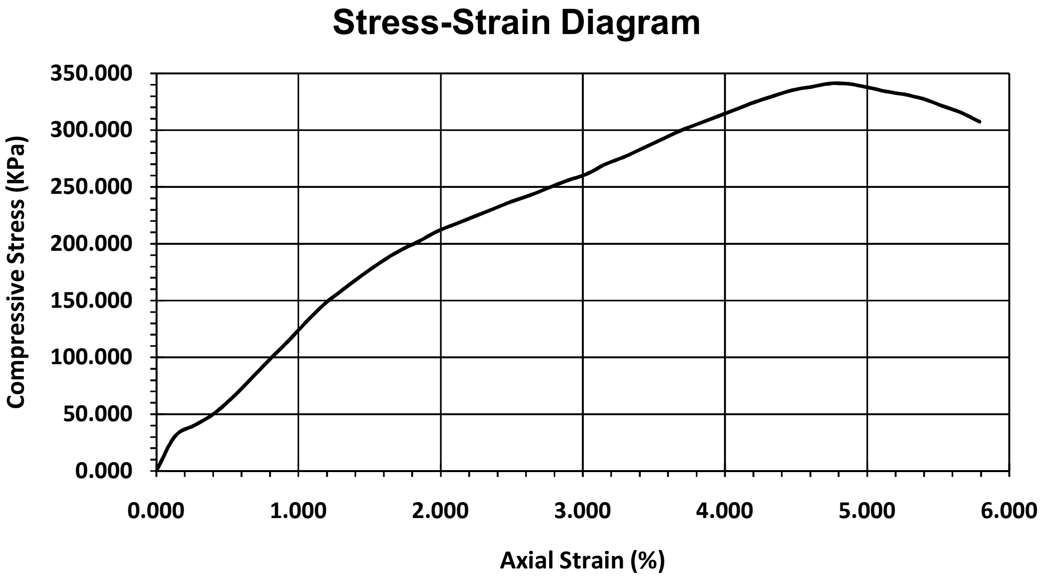


Fig 4.8 Axial stress Vs strain at 0%Lime content at TP1 (UC test)

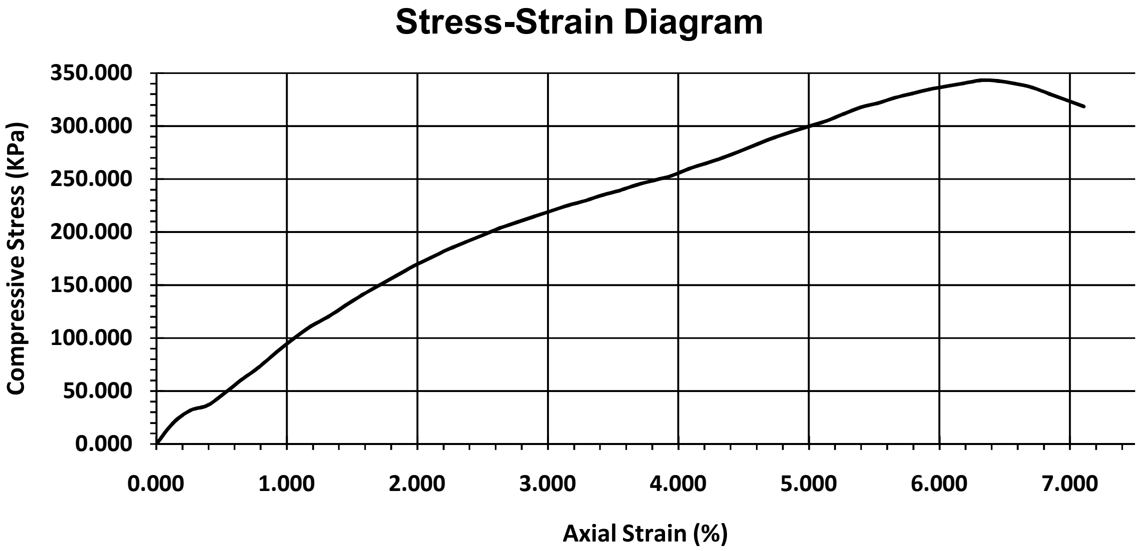


Fig 4.9 Axial stress Vs strain at 2% Lime content at TP1 (UC test)

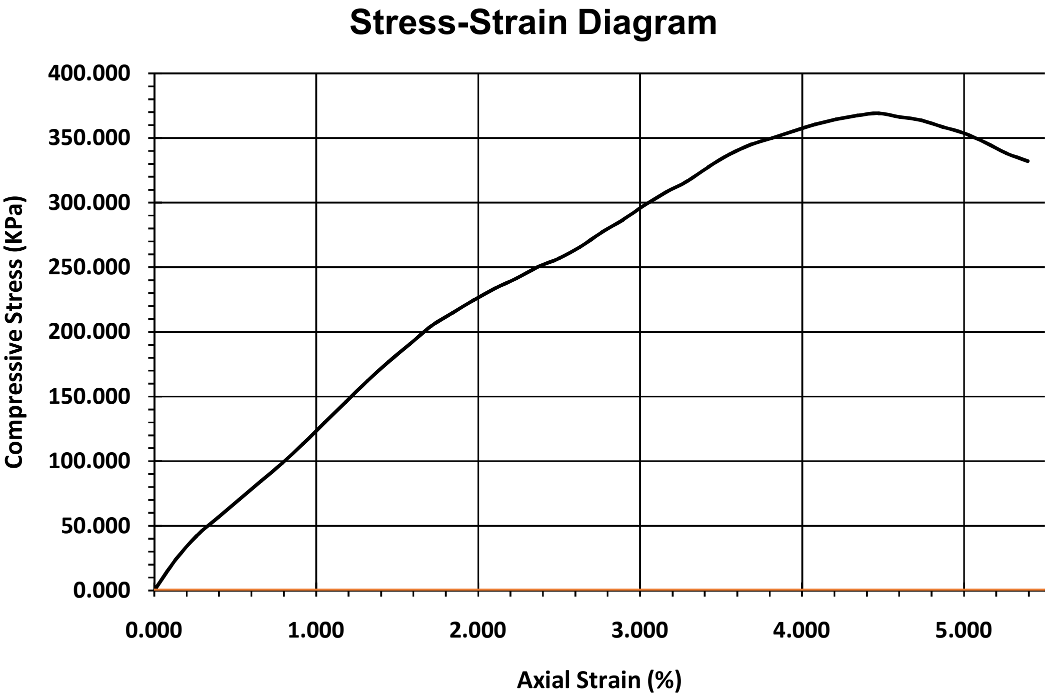


Fig 4.10 Axial stress Vs strain at 4% Lime content at TP1 (UC test)

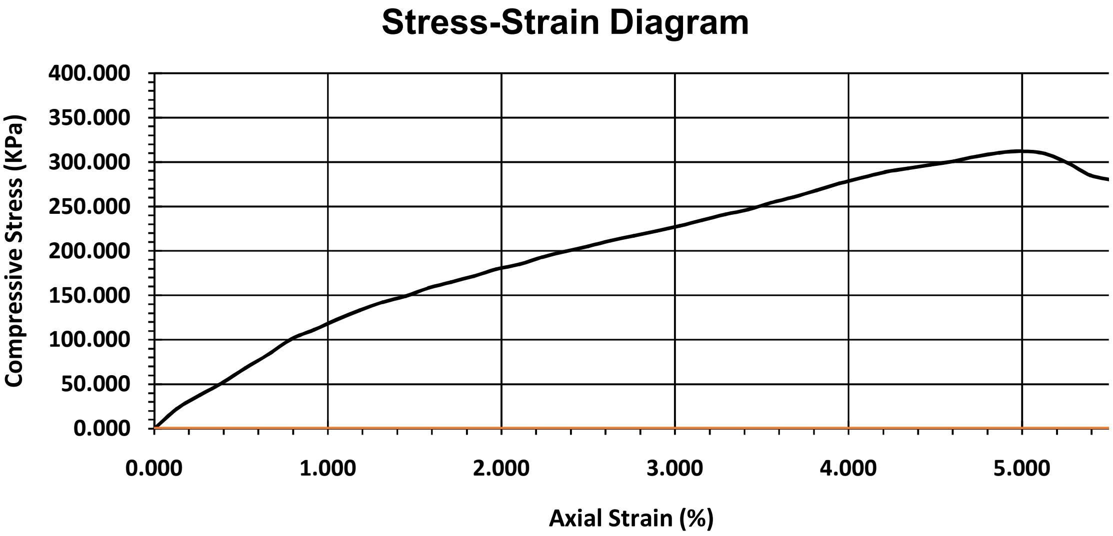


Fig 4.11 Axial stress Vs. strain at 6% Lime content at TP1 (UC test)

As shown in Table 4.11 the uncured compressive strength of Tp1 and Tp2 has improved from 341.05kPa at 0% to 352.3kPa at 4% and 391.823kPa at 0% to 369.08kPa at 4% lime content, respectively. The stress-strain curves for all tests shown from Fig 4.8 to fig 4.11.

4.10. California Bearing Ratio (CBR)

The CBR test indirectly measures the shearing resistance of a soil under controlled moisture and density conditions.CBR is obtained as the ratio of load required to effect a certain depth of penetration of a standard penetration piston into a compacted specimen of the soil at some water content and density to the standard load required to obtain the same depth of penetration on a standard sample of crushed stone. It is expressed by the force exerted by the plunger and the depth of its penetration into the specimen; it is aimed at determining the relationship between force and penetration.

CBR= (test load on the sample/ standard load on the crushed stone)*100 %

The CBR tests were conducted in accordance with AASHTO T193-93 for non-stabilized and stabilized soil sample for comparison. 4% lime content was blended in dry state for stabilized sample as it is the optimum percentage based on the compaction and USCS test results. A minimum of three points CBR was performed for each samples to determine the CBR accurately at the optimum moisture content from compaction test.

Accordingly 5.0kg of the natural non stabilized soil and with 4% lime content blended sample are mixed at their respective optimum moisture contents in 2124 cubic centimeters mould. Each sample is compacted in three layers with 56 blows from the 2.5kg rammer.

CBR tests were conducted on the compacted specimens at the optimum moisture content using standard compaction test. The compacted soil samples of the CBR mold are soaked for 96 hours in a water bath to get the soaked CBR value. The CBR value has improved from 21.9% at 0%to 54.7 at 4% lime Content as shown in Table 4.12and Table 4.13 below. The detail calculation is shown in the Appendix.

Table 4.12 Swell and Penetration data of three point CBR for 0% Lime Content

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	%
	960	1180	2.20	1.90	1525	1698	1.73	1.49	586	688	1.02	0.88

Penet-ration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge reading	Load	Corrected CBR		Gauge reading	Load	Corrected CBR	
			KN	KN	%		KN	KN	%		KN	KN	%
0		0	0			0	0			0	0		
0.64		20	0.48			34	0.82			48	1.16		
1.27		31	0.75			60	1.45			80	1.93		
1.91		40	0.97			74	1.79			108	2.61		
2.54	13	48	1.16	1.16	8.9	91	2.20	2.20	16.9	138	3.33	3.33	25.6
5.08	20	67	1.62	1.62	8.1	136	3.28	3.28	16.4	200	4.83	4.83	24.2
6.35		77	1.86			160	3.86			236	5.70		
7.62		83	2.00			178	4.30			258	6.23		
Soaked CBR, %		8.9				16.9				25.6			
Dry Density, g/cc		1.20				1.34				1.53			
Swell, %		1.90				1.49				0.88			
Density Requirement:			95%		Target Density:			1.44		CBR		21.9	

Table 4.13Swell and Penetration data of three point CBR for 4% Lime Content

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	%
	1173	1355	1.82	1.57	1731	1812	0.81	0.70	548	559	0.11	0.09

Penet-ration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge reading	Load	Corrected CBR		Gauge reading	Load	Corrected CBR	
			KN	KN	%		KN	KN	%		KN	KN	%
0		0	0			0	0			0	0		
0.64		62	1.50			121	2.92			220	5.31		
1.27		102	2.46			222	5.36			402	9.71		
1.91		123	2.97			252	6.09			460	11.11		
2.54	13.7	133	3.21	3.21	23.4	258	6.23	6.23	45.5	503	12.15	12.15	88.7
5.08	19.6	220	5.31	5.31	27.1	370	8.94	8.94	45.6	600	14.49	14.49	73.9
6.35		290	7.00			560	13.52			802	19.37		
7.62		302	7.29			670	16.18			1000	24.15		
Soaked CBR, %		27.1				45.6				88.7			
Dry Density, g/cc		1.26				1.41				1.52			
Swell, %		1.57				0.70				0.90			
Density Requirement:			95%		Target Density:			1.43		CBR		54.7	

5. DISCUSSION

The soil is classified as A-7-5 and MH group according to AASHTO classification and USCS, respectively. However, USCS does not consider the cementation effects of lateritic soils which increase the clayey grain classification into silt.

In order to utilize the natural soil of the proposed area as competent base or sub base material for heavy traffic, the obtained PI value of the natural soil sample is about 20% which does not satisfy according to the ERA pavement manual 2002(PI<15). In addition the CBR (21.9%) does not meet the desired values for sub base material.

However, by stabilizing with increasing percentage of lime for 0 and 7 days of curing time the optimum percentage by weight is found to be 4% of lime which reduced the PI to 7% for both samples which is a satisfactory result, because it decreases the plasticity nature of the soil which makes it dusty during dry season and sticky when it is wet.

The Unconfined Compression strength also improved modestly from 341kPa to 371kPa and from 297.15kPa to 391.82kPa for Tp1 and Tp2, respectively, with 4% lime stabilization.

The soaked CBR value has shown significant improvement from 21.9% at 0% to 54.7% at 4% lime stabilization qualifying the stabilized soil as a sub base material according to ERA pavement Manual 2002.

Comparison of the two strength test has shown that CBR has increased substantially by almost three fold after stabilization while the UCS improved modestly by 10-30%.The smallest of the gain of unconfined compressive strength may be due to the test was carried out in un cured state while soaked CBR test was carried out.

In General, there is no any clear correlation between the unconfined compressive strength and the CBR values, this may be because of the absence of lateral pressure in unconfined compressive testing whereas such pressure exists in CBR test.

However, the unconfined compressive test appears to have limited application as the basis for design of stabilized materials in road construction. There is very little agreement with design criteria based on the unconfined compressive strength of stabilized materials.(Lyon.A, 1979).

6. CONCLUSION AND RECOMMENDATION

The following Conclusions can be drawn from the result of the investigation carried out within the scope of the study.

1. The soil is classified as A-7-5 according to AASHTO classification and according to USCS samples fall under MH or inorganic clay group of soil.
2. The plasticity index decreased significantly with addition of lime content from 0% to 4% by weight of lime where there is significant increment from 4 to 8%.
3. The optimum moisture content increased while the maximum dry density values decreased with increment of lime content.
4. The uncured Unconfined Compression strength increased slightly with the addition of lime.
5. The soaked CBR value has improved from 21.9% at 0% to 54.7 at 4% lime Content. This shows that the load bearing capacity of the sample increased significantly with lime treatment.
6. Therefore, this material with 4% Lime stabilization can be utilized as a sub base with crushing of basaltic rocks material to fulfill the gradation according to ERA Pavement Manual 2002.
7. Further detailed analysis of the study should be done on different areas of Wollayita Sodo.
8. Cost benefit analysis should also be made to use lime or other stabilizing agents for construction of heavy traffic roads as compared with crushing of basaltic rocks or other alternatives.

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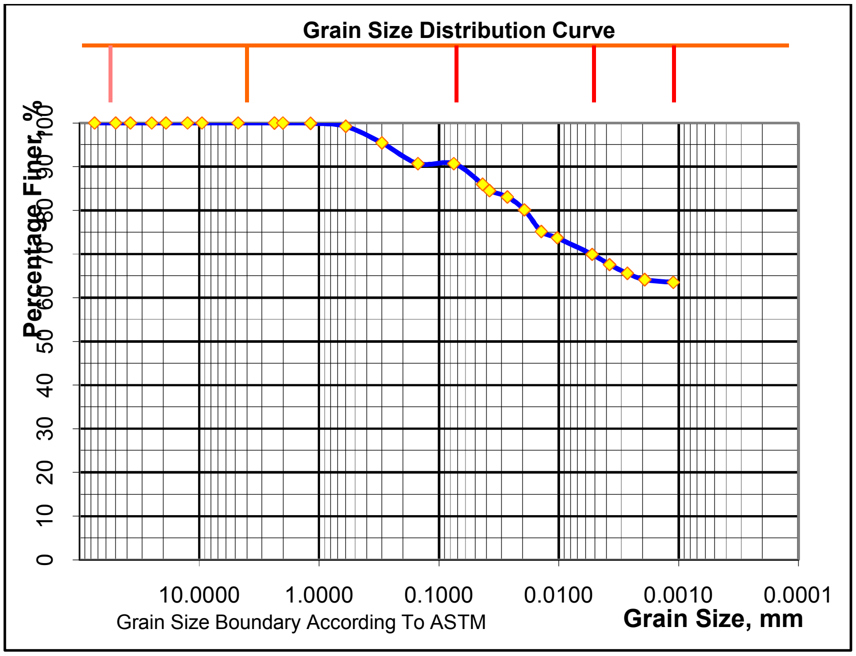
16. Stewart Scott and AEC 1998 Engineering Report for Alemgena-Hossana-Sodo road design project Addis Ababa, Ethiopia.

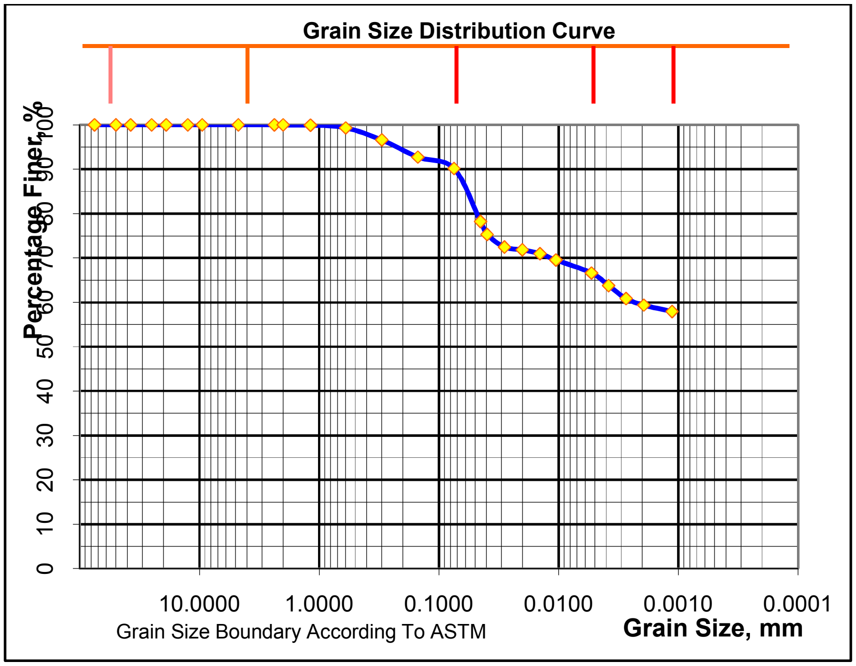
Appendix

Grain Size Analysis

a) TP1



b) TP2



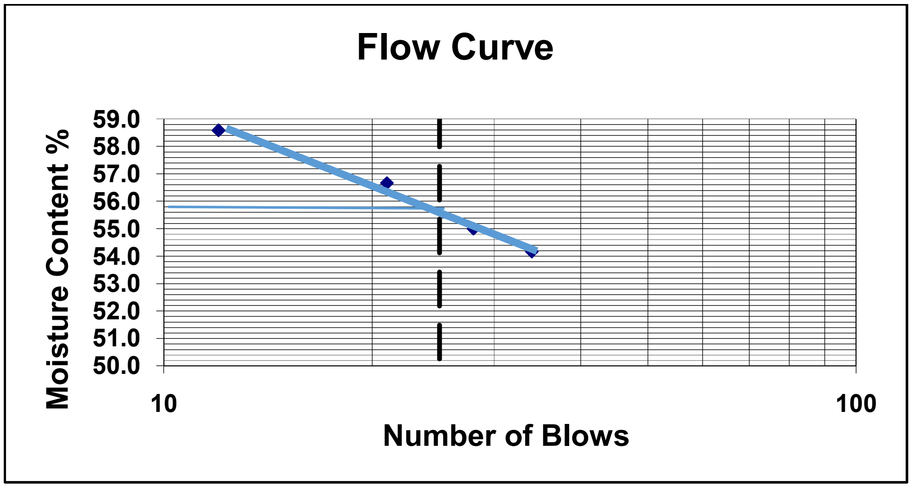
Appendix 1.1 Atterberg limits with varying percentage of Lime for 0days of Curing.

a) **Lime Content 0% TP1**

LL=56%

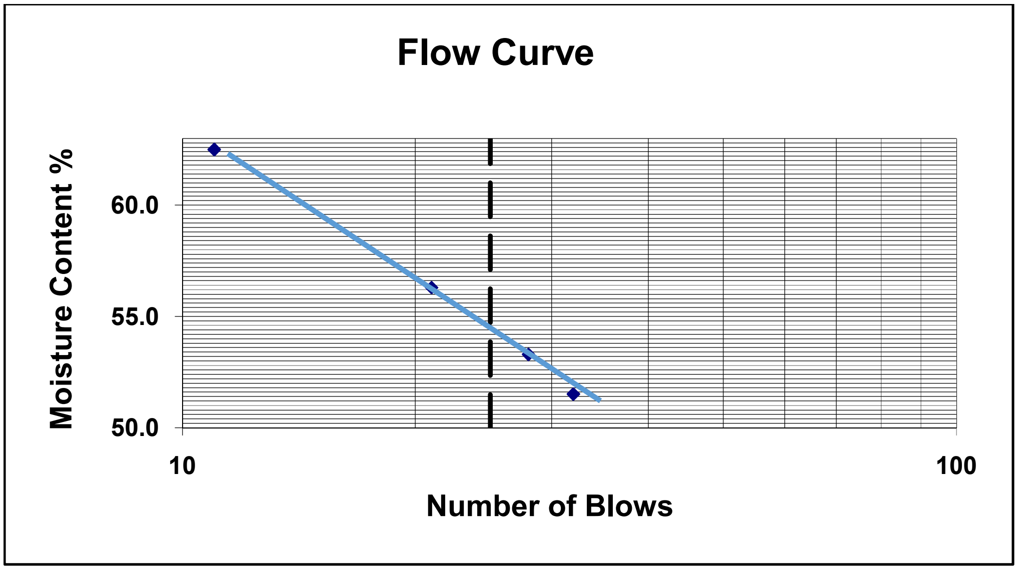
PL=34

PI=22



b) **Lime Content 0% TP2**

LL=54%
PL=34%
PI=20%



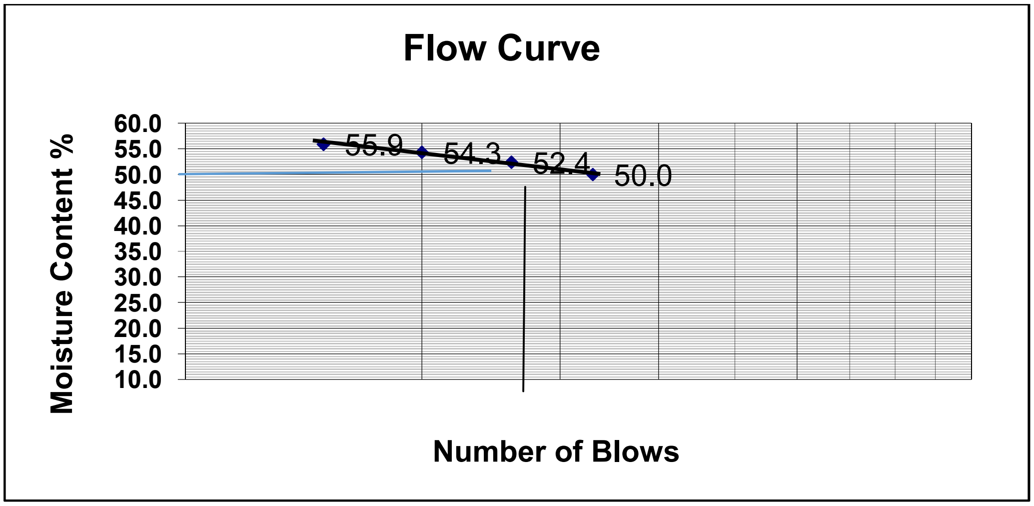
Appendix 1.2 Atterberg limits with varying percentage of Lime for 7 days of Curing

a) **2% lime Tp1**

LL=50%

PL=33%

PI=17%

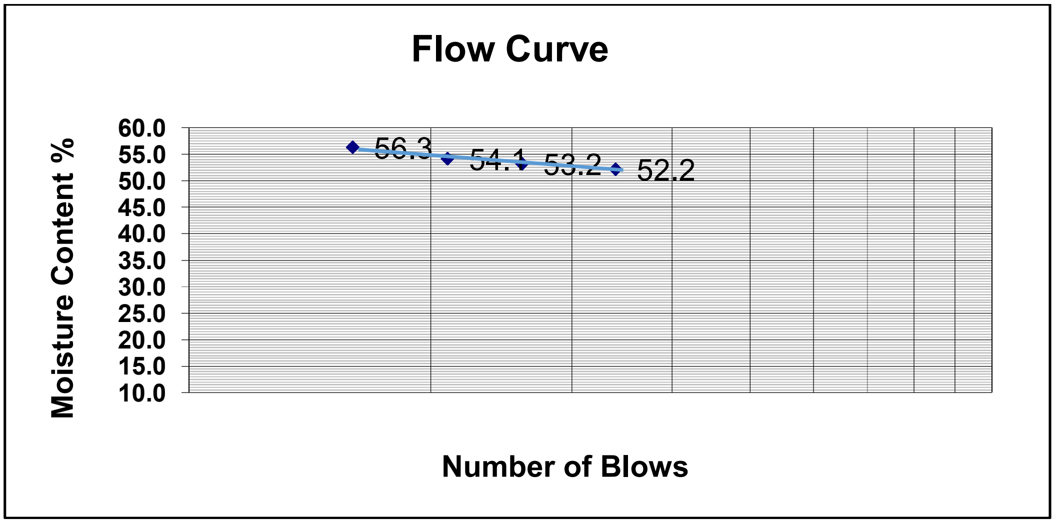


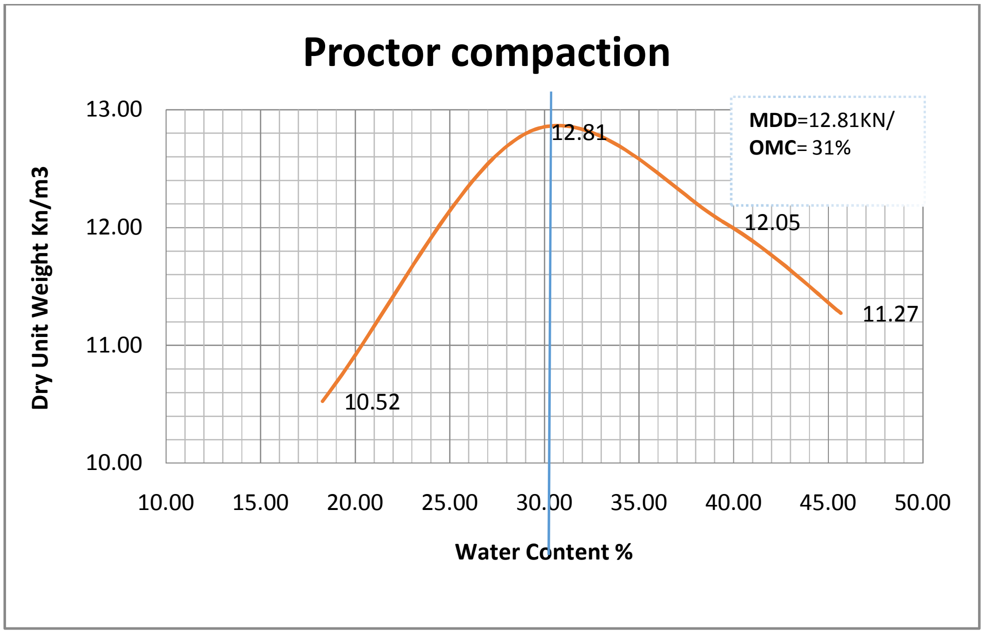
b) **4% lime TP1**

LL=52.5%

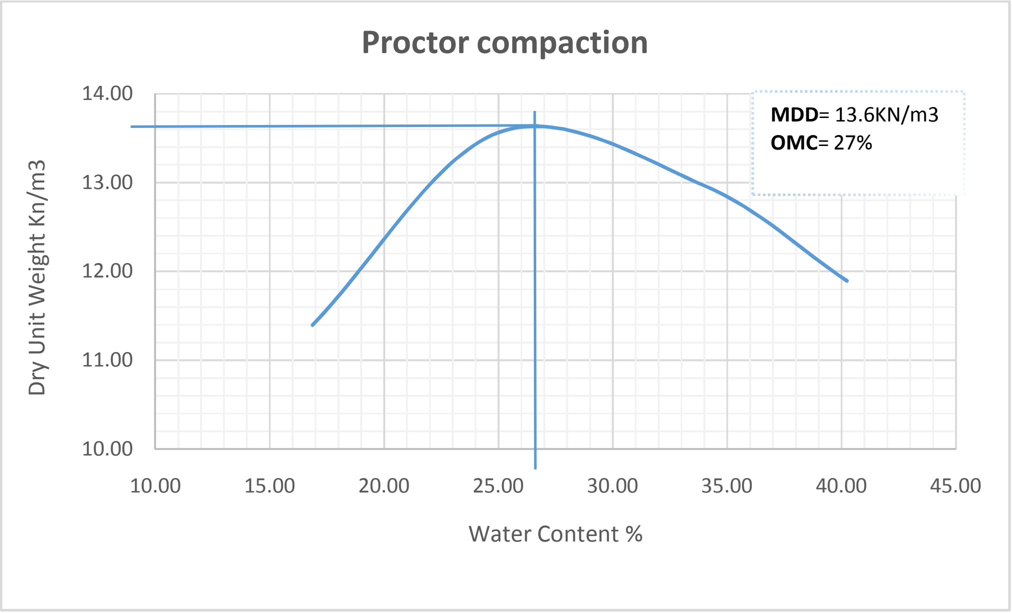
PL=32%

PI=18%



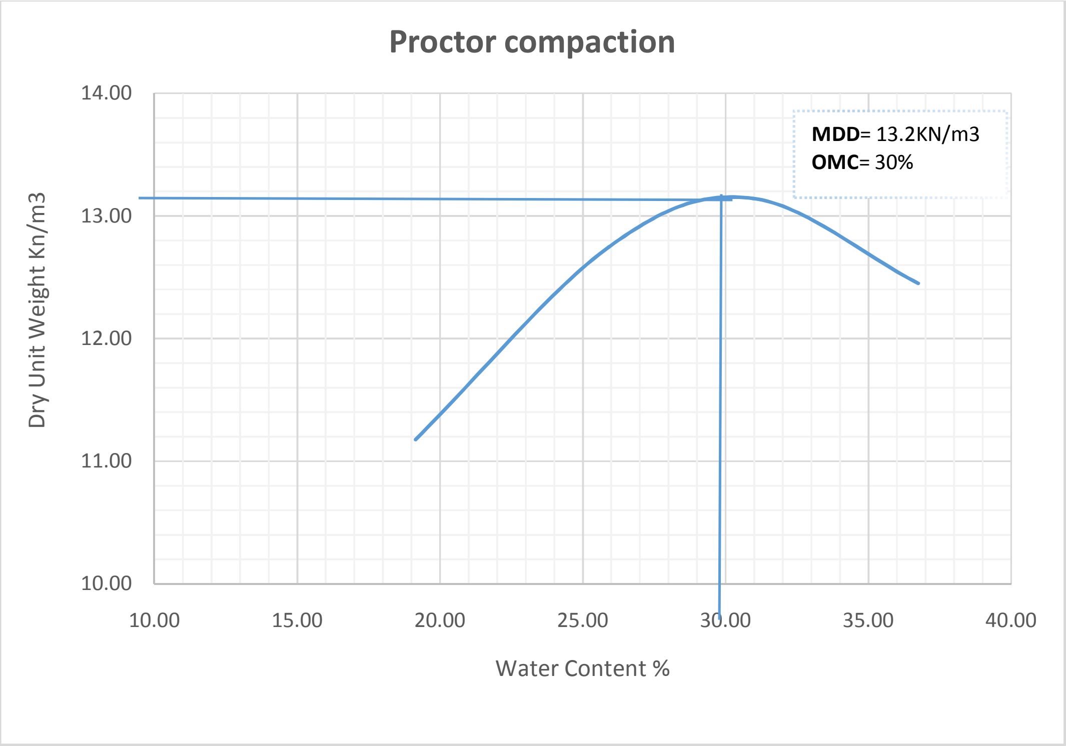

Proctor compaction
Dry Unit Weight Kn/m3
13.00
12.00
11.00
10.00
10.00
15.00
20.00
25.00
30.00
35.00
40.00
45.00
50.00
Water Content %
12.81
12.05
11.27
10.52
MDD=12.81KN/
OMC= 31%

c) 0% lime Tp2

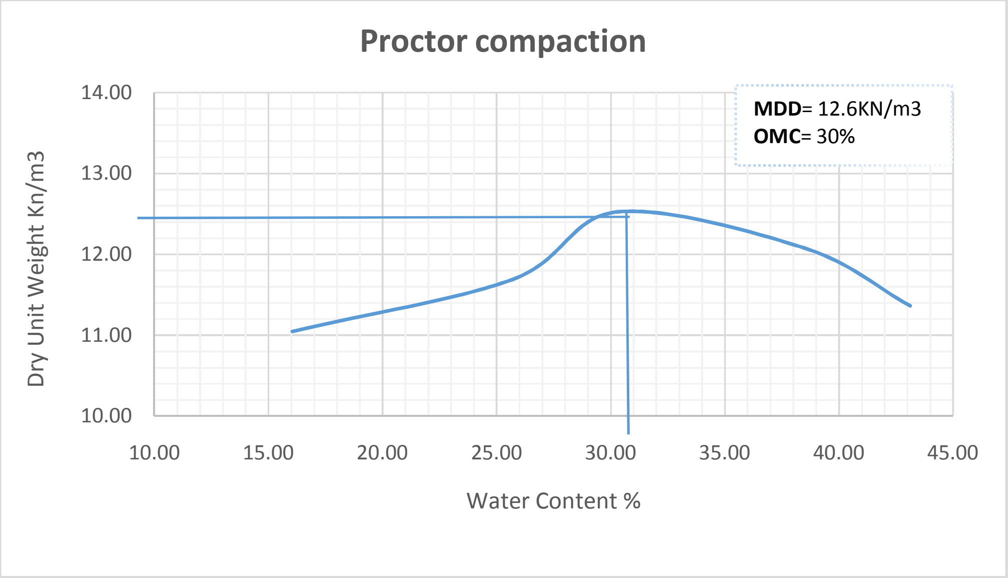

Proctor compaction
Dry Unit Weight Kn/m3
14.00
13.00
12.00
11.00
10.00
10.00
15.00
20.00
25.00
30.00
35.00
40.00
45.00
Water Content %
MDD= 13.6KN/m3
OMC= 27%

0 days of Curing

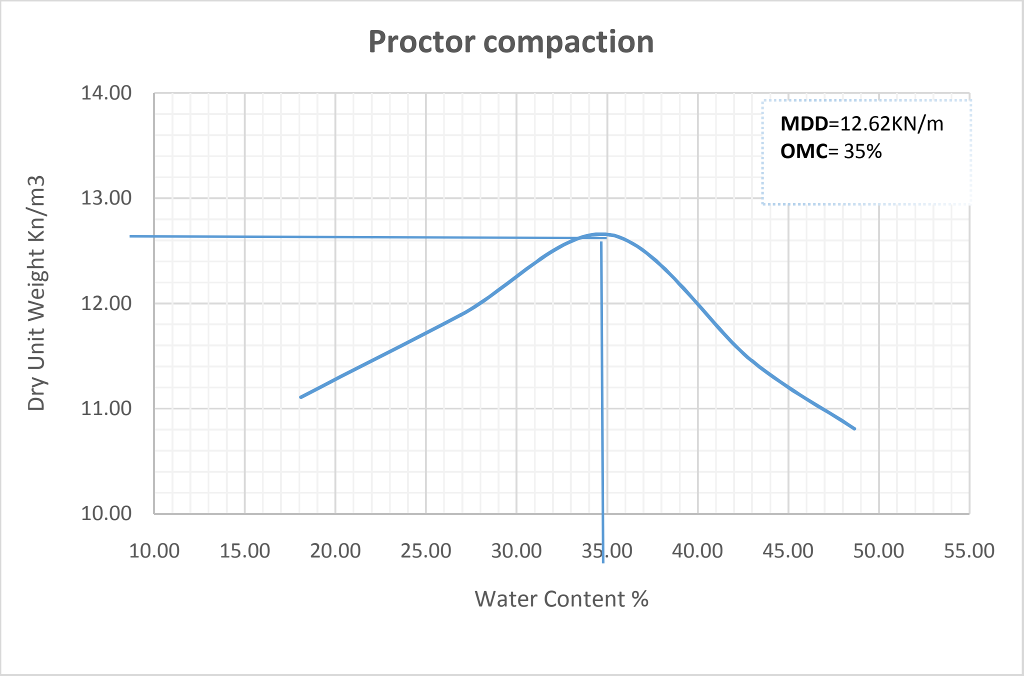
a) 2% lime for Tp1



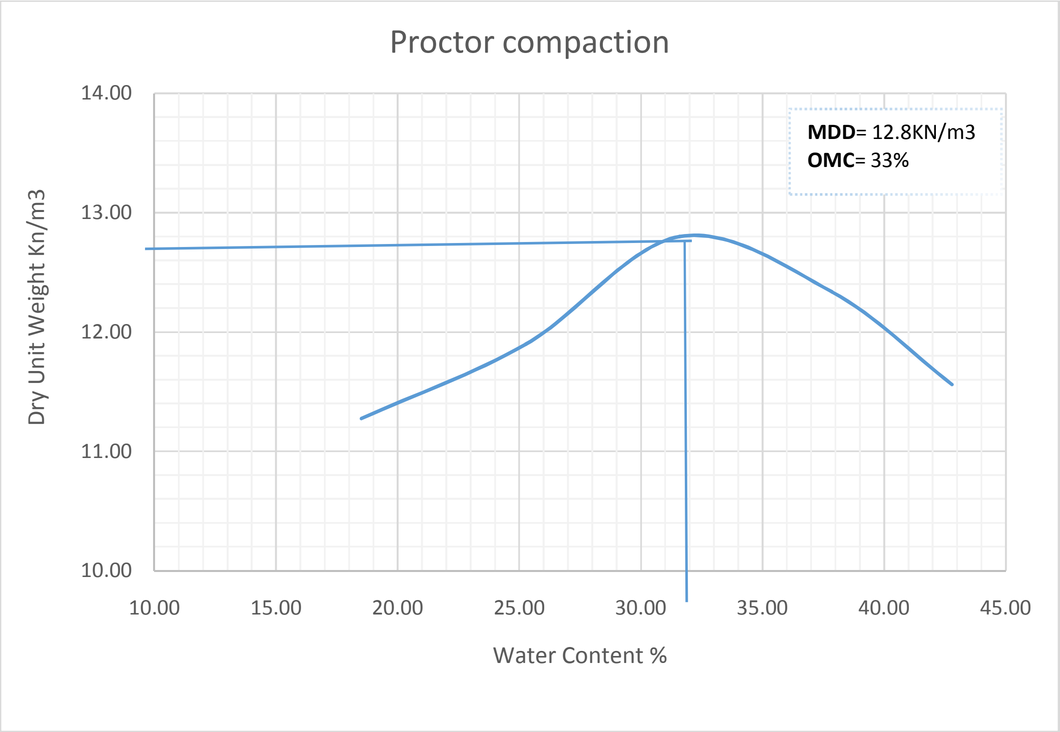
b) 2% lime Tp2



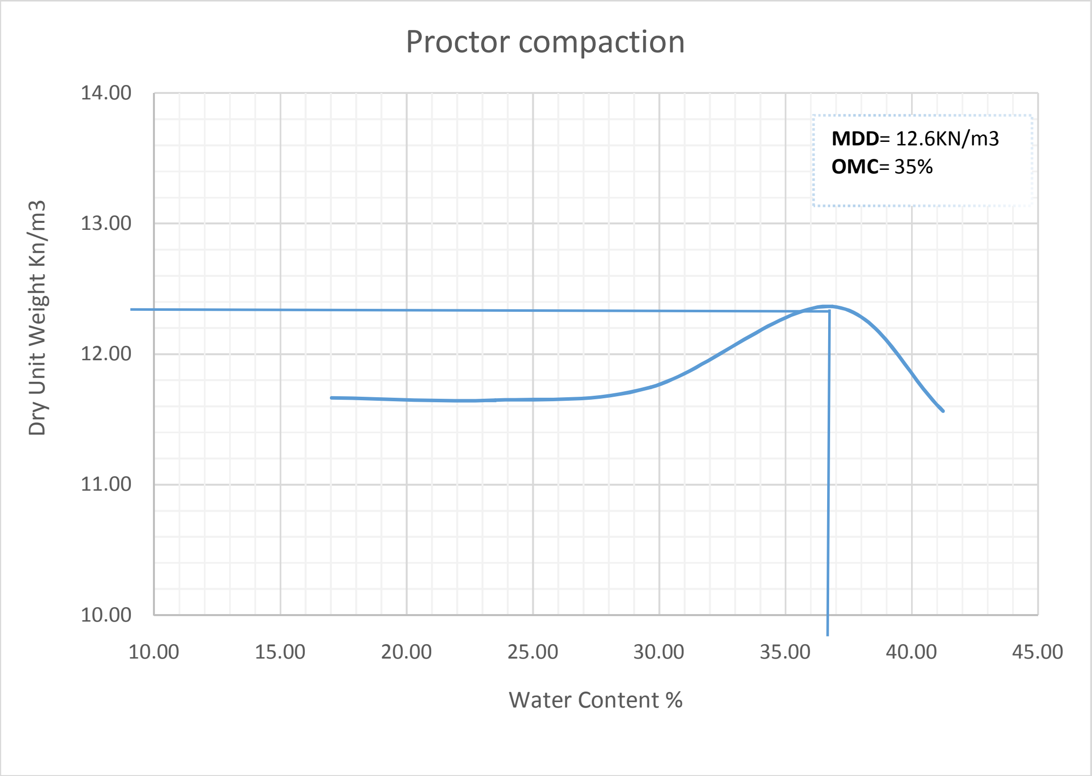
c) 4% lime Tp1



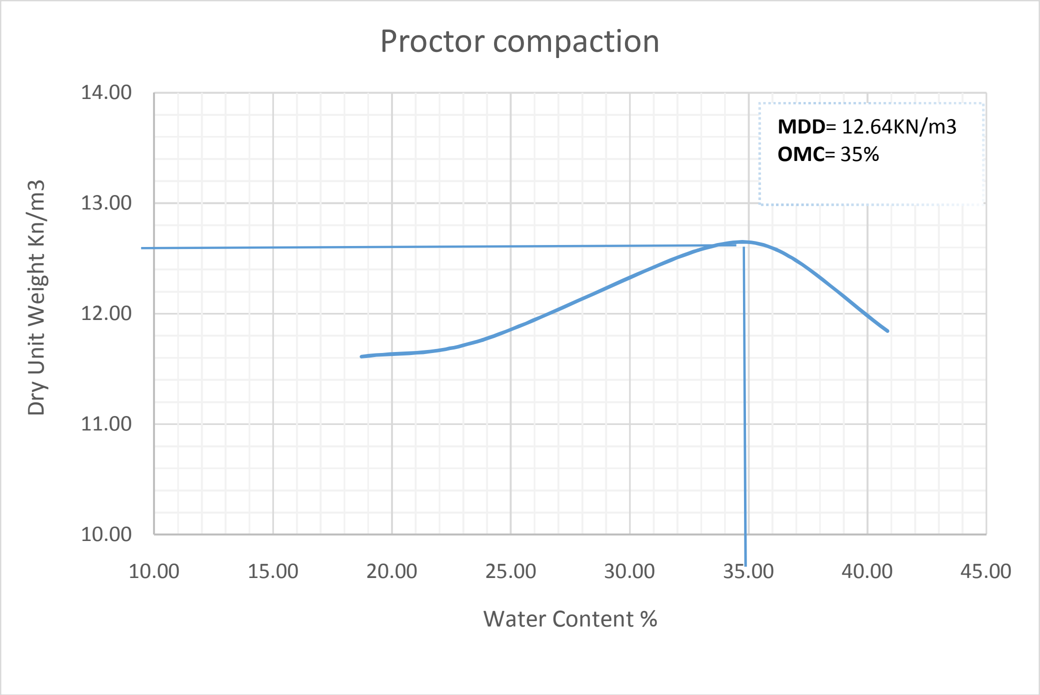
d) 4% lime Tp2



e) 6% lime Tp1

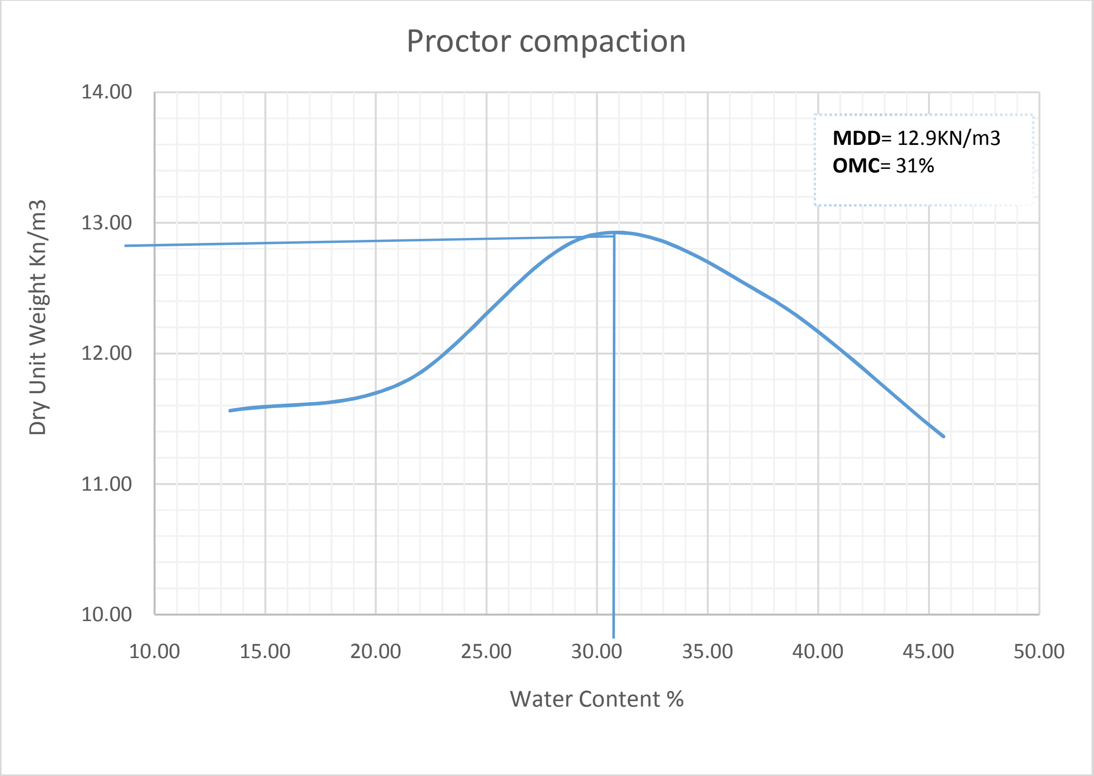


f) 6% lime Tp2

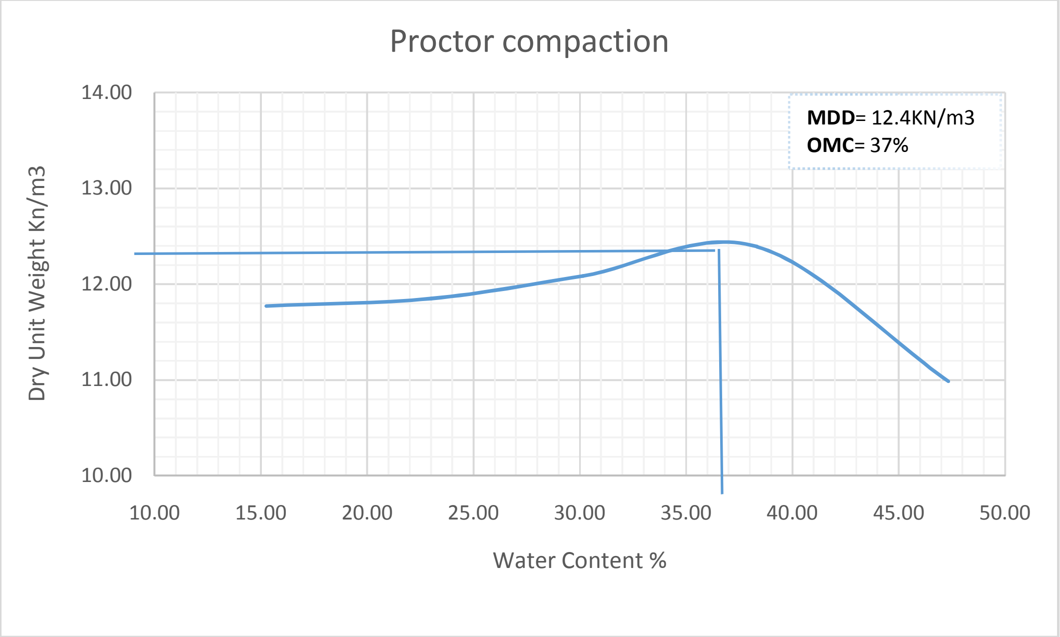


7 days of Curing

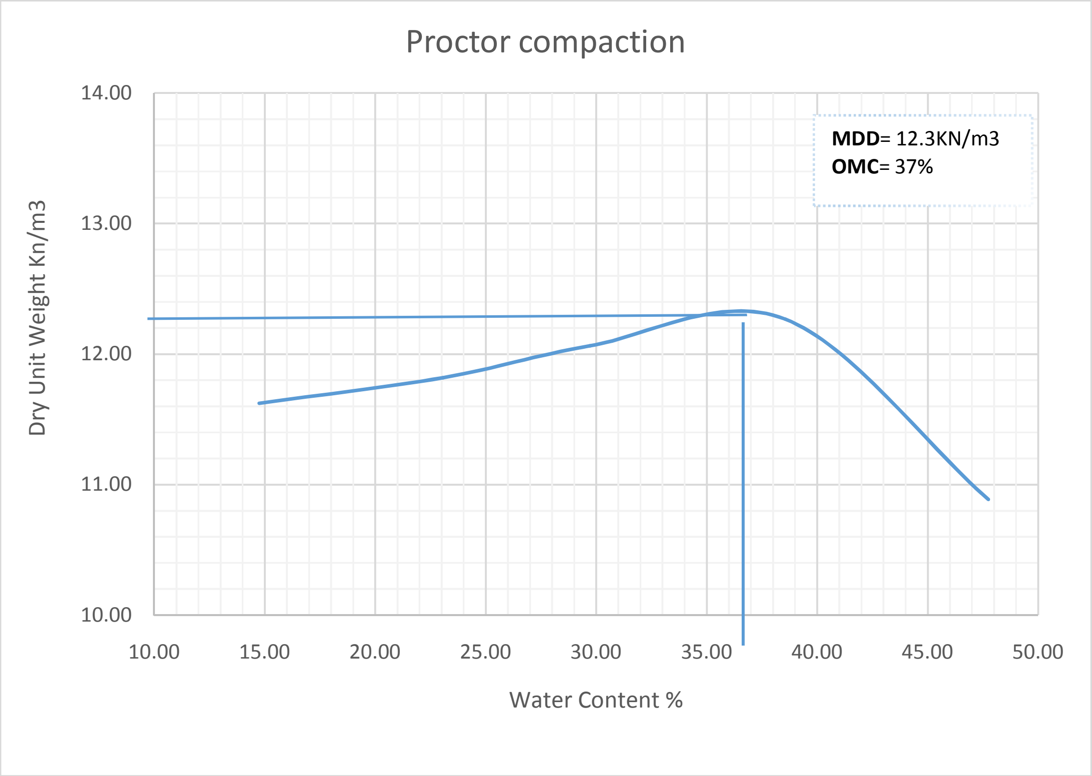
a) 2% lime for Tp2



b) 4% lime Tp1



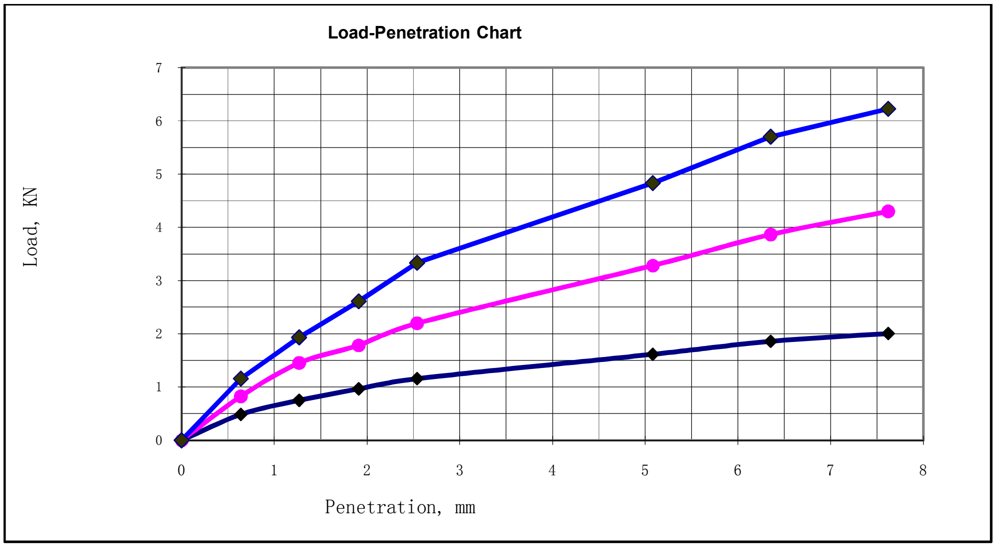
C) 4% lime TP2



CBR Test data for Natural soil

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	%
	960	1180	2.20	1.90	1525	1698	1.73	1.49	586	688	1.02	0.88

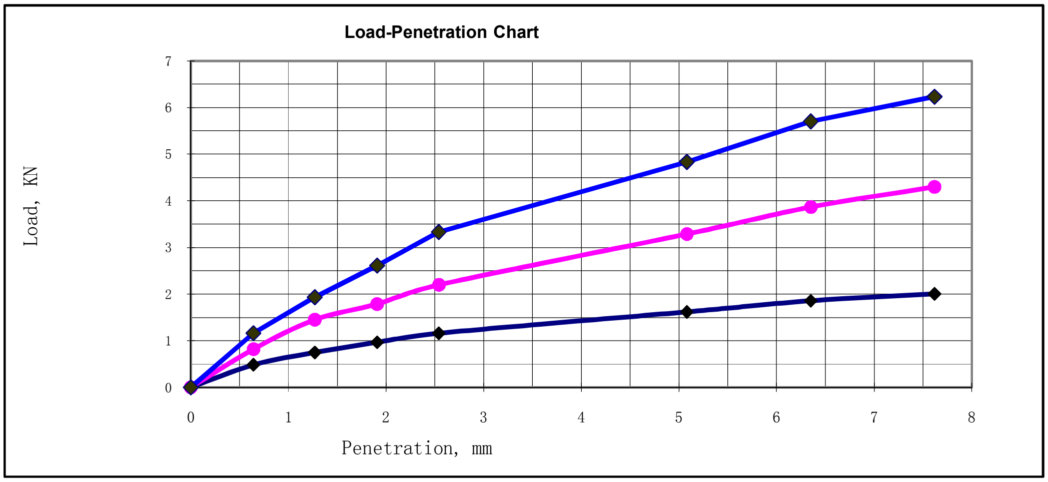
Penet-ration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge reading	Load	Corrected CBR		Gauge reading	Load	Corrected CBR	
			KN	KN	%		KN	KN	%		KN	KN	%
0		0	0			0	0			0	0		
0.64		20	0.48			34	0.82			48	1.16		
1.27		31	0.75			60	1.45			80	1.93		
1.91		40	0.97			74	1.79			108	2.61		
2.54	13	48	1.16	1.16	8.9	91	2.20	2.20	16.9	138	3.33	3.33	25.6
5.08	20	67	1.62	1.62	8.1	136	3.28	3.28	16.4	200	4.83	4.83	24.2
6.35		77	1.86			160	3.86			236	5.70		
7.62		83	2.00			178	4.30			258	6.23		
Soaked CBR, %		8.9				16.9				25.6			
Dry Density, g/cc		1.20				1.34				1.53			
Swell, %		1.90				1.49				0.88			
Density Requirement:			95%		Target Density:			1.44		CBR		21.9	



CBR Test data for 4% stabilized Sample

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	%
	1173	1355	1.82	1.57	1731	1812	0.81	0.70	548	559	0.11	0.09

Penet - ration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge reading	Load	Corrected CBR		Gauge reading	Load	Corrected CBR	
			KN	KN	%		KN	KN	%		KN	KN	%
0		0	0			0	0			0	0		
0.64		62	1.50			121	2.92			220	5.31		
1.27		102	2.46			222	5.36			402	9.71		
1.91		123	2.97			252	6.09			460	11.11		
2.54	13.7	133	3.21	3.21	23.4	258	6.23	6.23	45.5	503	12.15	12.15	88.7
5.08	19.6	220	5.31	5.31	27.1	370	8.94	8.94	45.6	600	14.49	14.49	73.9
6.35		290	7.00			560	13.52			802	19.37		
7.62		302	7.29			670	16.18			1000	24.15		
Soaked CBR, %		27.1				45.6				88.7			
Dry Density, g/cc		1.26				1.41				1.52			
Swell, %		1.57				0.70				0.90			
Density Requirement:			95%		Target Density:			1.43		CBR		54.7	


Load-Penetration Chart
Load, KN
Penetration, mm
0
1
2
3
4
5
6
7
0
1
2
3
4
5
6
7
8