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ADDIS ABABA UNIVERSITY

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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

STABILIZATION OF EXPANSIVE SUBGRADE SOILS WITH CEMENT AND WOOD ASH

SPONSOR ERA

A Thesis Submitted to School of Graduate Studies in Partial Fulfillment Of the Requirements for the Degree of Master of Science in Civil Engineering (Road and Transport Engineering)

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Addis Ababa University

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STABILIZATION OF EXPANSIVE SUBGRADE SOILS WITH CEMENT AND WOOD ASH:

BY

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Declaration

I, Getnet Meresa, hereby declare that this thesis entitled "Stabilization of Expansive Subgrade Soils With Cement And Wood ash: submitted by me to Addis Ababa University, Addis Ababa Institute of Technology (AAIT) for the award of the Degree of Master of Engineering in Road and Transport Engineering is original work and it hasn't been presented for the award of any other Degree, Diploma, Fellowship or other similar titles of any other university or institution. Besides, all views and opinions expressed therein remain the sole responsibility of the author and do not necessarily represent those of the Institute or the university, and all sources of the materials in the research thesis have been duly acknowledged.

Acknowledgment

First, and for most, I would like to express my deepest gratitude to Almighty God, who made it possible to begin and finish this work effectively. And also, I would like to thank my advisor Ephrem Taddesse (Associate professor) and my co-advisor Mastewal Shumet (M.Sc.), for their invaluable advice, kind encouragement, and consistent support throughout the study.

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Abstract

Expansive clay soils in their natural state are not adequate to be used as subgrade material in pavement road, hence they should be stabilized in order to improve strength and durability, improve workability and use of cost minimized locally available materials. The research was proceeded to evaluate the potential of cement-wood ash mix to stabilize expansive clay soil. The evaluation involved the determination of the geotechnical properties of Expansive soil in its natural state as well as when mixed with varying proportions of cement-wood ash additive. The parameters tested included the particle size distribution, specific gravity, Atterberg limits, compaction characteristics, California bearing ratio CBR, CBR swelling test, free swell and the unconfined compressive strength. Results showed that the geotechnical parameters of Expansive soil are improved substantially by the addition of cement-wood ash; plasticity was reduced by 27% and CBR and unconfined compressive strength increased as cement-wood ash addition increases and the values of CBR have increased from 1% to 24.3% while UCS values have increased from 161kpa to 450kpa. The results also showed that as the swelling potential values from CBR swelling test and free swell were found to be encouraging as CBR swell value of the expansive soil decreased from 13.1% to 2.43%. Hence, from the above mentioned results, it was concluded that the use of cement wood ash improves the engineering properties of expansive clay soil to a great extent.

Keywords: Stabilization, strength, Durability, plasticity, Expansive soil, cement, wood ash, cement-wood ash

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List of Abbreviations/Acronym

AASHTO……. American Association State Highways and Transportation Official

ACI………….. American Concrete Institute

ASTM……….. American society for testing materials

C ………………Cohesion

^{0}C.................... Degree centigrade

CBR…………...California Bearing Ratio

ERA………….. Ethiopian Road Authority

g ………………. Gram

GC……………. Capping layers

GI……………… Group Index

LL………………Liquid Limit

mm…………….. Millimeter

MDD………….. Maximum Dry Density

NP……………..Non-Plastic material

OMC………….. Optimum Moisture Content

OPC…………… Ordinary Portland cement

PI……………… Plasticity Index

PL……………… Plastic Limit

UCS……………. Unconfined Compressive Strength

USCS…………... Unified Soil Classification System

XRD…………….X-Ray Diffraction

Chapter One

1. Introduction

1.1 Background of the study

Expansive soils are those which show volumetric changes in response to changes in their moisture content. Such soils swell when the moisture content is increased and shrink when the moisture content is decreased. Expansive soils are a global problem that poses several challenges for civil engineers. They are considered a potential natural hazard, which can cause extensive damage to structures if not adequately treated. Expansive soils causes more damage to structures, particularly lighter buildings and pavements, than any other natural hazard, including earthquakes and floods [23].

Expansive clay soils have a world-wide distribution; their occurrence is not climatic specific though they are particularly widespread in arid to semi-arid climate and are problematic to engineering structures because of their tendency to heave during wet season and shrink during dry season. Although the extent and range of distribution of this problematic soil have not been studied thoroughly, expansive soil is known to be widely spread in Ethiopia [20].

In Ethiopia, there are several roads whose premature failures attributed to the volumetric changes of expansive clay soil; Modjo-Ejerie-Areti Road and Addis-Jimma Road could be examples of such failures [6], [8], [35]. Therefore, these soils are not fit for the construction of infrastructure on them as they have a high risk of settlement. Hence, it has become a challenging task for the construction industry to design and construct a structure on such soft soil.

There are different methods practiced to improve the engineering properties of such soils such as stabilization, grouting, removal and replacement, preloading, stone columns, and dynamic compaction and reinforcement using geo synthetics. Replacement methods could be the possible solution for organic soil, where the organic soil to a sufficient depth is replaced with granular soil such as sand and crushed stones or preloading to improve engineering properties but, with the increased global demand for energy and increasing local demand for aggregates, it has become expensive from a material point of view to remove inferior soils and replace them with foreign soils. Chemical stabilization is an alternative low-cost solution, where stabilizing agents such as cement, lime, fly ash, and other binders stabilize the organic soil rapidly through chemical reactions.

Stabilization is a technique introduced many years ago with the main purpose to render the soils capable of meeting the requirements of the specific engineering projects. Stabilized materials may be used as improved subgrades or capping layers or sub base for road or airfield pavements. It is the alteration of one or more soil properties, by mechanical or chemical means, to create an improved soil material possessing the desired engineering properties. Soils may be stabilized to increase strength and durability or to prevent erosion and dust generation [29], [33].

Purpose of the stabilization of flexible pavement resting on weak and problematic soil is to acquire desirable properties of subgrade which are high compressive and shear strength, permanency of strength under all weather and loading conditions, ease and permanency of compaction, ease of, drainage and low susceptibility to volume changes and frost action. Since subgrade soils vary considerably, the interrelationship of texture, density, moisture content and strength of subgrade materials is complex [29], [33].

Cement and lime are mostly known to be used to improve the geotechnical properties of clay soils [29]. In developing countries, however, cement and lime are expensive, hence it becomes essential to modify the properties of locally available soil to the extent that it can be used in the construction of roads and to make best utilization of locally available material residue powders left after the combustion of wood, such as burning wood in a home fireplace or an industrial power plant.

Various research works have been undertaken for improving the expansive nature of the black cotton soil using materials like wood ash as an admixture. Wood ash is a locally available material residue powder left after the combustion of wood, such as burning wood in a home fireplace or an industrial power plant. It is generally discarded as waste and dumped outside of house or landfill, which increase the volume of landfill. The experiments on wood ash showed that wood ash has an alkaline nature which clearly defines its potential as a raw material resource for the production of acidic and other different type soil improvers. Therefore as an alternative solution, wood ash can be used as a potential soil stabilizer through the chemical reaction [18].

It is found from the literature that the concentration of heavy metals present in the wood ash is very low [18]. The research by [36] showed that wood ash certainly meets some of these conditions, such as plasticity reduction, swell reduction, improved stability, and substantial strength gain but fails to meet the benefit of continued strength gain with time. These results from research imply that although wood ash provides most of the beneficial effects of lime in construction, it is unlikely to be a substitute for lime in soil stabilization. In this research however, cement and wood ash had been used on expansive soil-cement–wood ash mixtures and assess the potential of wood ash and cement mixture to stabilize expansive clay soils and also to assess if cement-wood ash mixture can replace some content cement which will have positive impact on the cost of the process.

1.2 Statement of the Problem

Expansive clay soils have caused persistent difficulties in road construction and are a relatively common problem in Ethiopia. The conventional stabilizing agents commonly used in expansive soils and replacement of the inferior sub-grade soils by foreign soils are fairly expensive. As a result, such roads are not adequately constructed and, therefore, frequently require close attention. Consequently, it becomes essential to modify the properties of locally available soil with cheaper stabilizers to the extent that it can be used in the construction of roads and to make utilization locally available material residue powders left after the combustion of wood, such as burning wood in a home fireplace or an industrial power plant.

1.3 Research Questions

- Is it possible to improve the strength of expansive soils using cement and wood ash?
- Is it possible to improve the different engineering properties of expansive soil using cement-wood ash mix?
- Is it possible to use the stabilized expansive soil as subgrade for pavement road construction?
- What composition of clay-cement- wood ash mix design will fulfill the requirement to use as pavement subgrade material?

- Is it possible to decrease the percentage of cement and replace it with locally available wood ash?

1.4 Objectives of the Study

1.4.1 General Objective

The main objective of the study is to stabilize expansive soils with locally available wood ash and cement.

1.4.2 Specific Objectives

The specific objectives of the study will include the following:-

- To improve the strength of expansive soils using cement and wood ash.
- To improve workability characteristics mainly by reducing the plasticity of the soil.
- To decrease the percentage of cement and replace it with locally available wood ash.

1.5 Scope and Limitations

The scope of the research was to evaluate laboratory performance of expansive clay soil with the addition of cement and wood ash mixture of various percentages.

In the present study, there were limitations on precisely identifying clay mineralogy of the soil and wood ash under study due to capacity limitations of the X-Ray Diffraction (XRD) soil test machine.

In this research, wood ash obtained from the combustion of Eucalyptus (Bahrizaf) wood at Bole 02 Keble for house use purpose is used. Wood ash other than this is not investigated.

Therefore, further studies and additional tests are required before implementing these results or finding for field applications, hence shall be considered indicative result only.

1.6 Organization of the thesis

This thesis consists of five chapters

- Chapter one contains background of the study, problem statement, research questions, objective of the study and scope & limitations.
- Chapter Two reviews related literature on expansive soils which include properties of expansive soil, classification of expansive soils, Stabilization of Expansive soils, treatment of expansive soil with Portland cement and mechanisms involved in soil stabilization with OPC, related works on expansive soil treated with wood ash.
- Chapter Three covers the Research Methods followed, Materials used to conduct the research. Study area and description of the conducted Laboratory tests.
- Chapter four presents test results obtained from stabilization of expansive soil and discussion on the obtained results.
- Chapter Five discusses the conclusions and recommendations drawn from the research study.
- The detailed test results are presented in the appendix.

CHAPTER TWO

2. Review of Related Literatures

2.1 The Characteristic of Expansive Soils

Soils in engineering are unconsolidated materials composed of solid particles produced by weathering of rocks due to mechanical disintegration or chemical decompositions. The top soil of the earth's crust contains a large quantity of organic matter and is not suitable as a construction material [3]. In pavement construction, the fine-grained soils are the main causes of road failures. It has been shown that if fine-grained materials have a high LL and a high PI, they will show excessive swelling and shrinkage as well as a low bearing capacity when wet. These serious problems are due to cohesive soils are mainly built up from clay minerals. These clay minerals are well-known for both their mineralogy and their particle size. Most particles are less than 2 microns (0.002 mm) in diameter. According to [32] there are three common groups of clay minerals in pavement engineering. These are kaolinite, illite and montmorillonite.

I. Kaolinite

It is stable clay with limited swelling and shrinking which consists of a silica sheet and a gibbsite Asheet. These sheets are tightly bonded together by common oxygen ions as shown in figure 2.1. The chemical bonds inside of a sheet of kaolinite are covalent bonds and are very strong. The kaolinite sheets are locked together by weaker hydrogen bonds between the oxygen of the silica sheet and the hydroxyls of the alumina sheet. Hence the particles have well-developed cleavage parallel to the sheets. The hydrogen bonds are strong enough, however, to provide considerable reinforcement between the sheets and as a result, the kaolinite flakes grow fairly large in nature (often 100 or more sheets in thickness). Water cannot enter between sheets to expand or shrink the particles as result the mineral is relatively stable.

II. Illite

Each individual sheet of illite consists of an octahedral layer inserted between two tetrahedral layers as shown in figure 2.1. Illite sheets have very strong covalent bonds. The octahedral layer may contain Al, Mg, Fe or other cations. In the tetrahedral layer 1 in 7 Si^{4+} ions are replaced by an Al^{3+}ion giving the sheet a large deficit of positive charge. Some deficiency also results from insufficient charges in the octahedral layer. Hence, the illite sheet must not only adsorb cations to balance broken bonds at the edges of the sheet but must also adsorb cations to balance charge deficiencies inside each sheet. To balance the positive charge deficit in the tetrahedrons potassium ions, K^+ ions enter between two tetrahedral layers.

III. Montmorillonite

The expansive behavior of clay soils is due to the presence of montmorillonite mineral which has an expanding frame structure. Water can readily be absorbed in the frame causing it to expand and the specific surface (surface area per unit mass) of montmorillonite is also more than other clay minerals. Montmorillonite sheet consists of an octahedral layer inserted between two tetrahedral layers. The charge scarcity in montmorillonite comes from the octahedral layer. This charge deficit must be balanced in the same way as it is done with illite, i.e. by means of cations between the montmorillonite sheets. Since the distance between the charge deficit and the cation is larger with the montmorillonite than with the illite and the bond between the montmorillonite sheets is much less than that of the illite sheets.

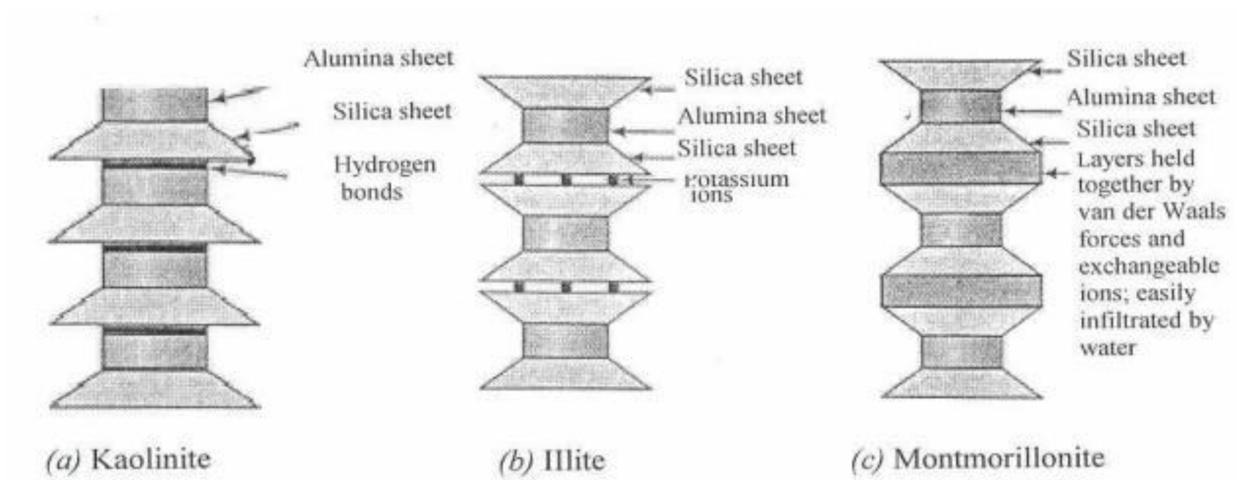


Figure 2:1: Mineralogy of expansive soils

2.2 Identification and Classification of Expansive Soils

2.2.1 Field Identification of Expansive Soils

Some of the important field identification methods that indicate the potential for expansiveness of a soil during reconnaissance and preliminary stages are summarized in Table 2.1.

Table 2:1: Filed identification methods for expansive soils

Field Assessment	
Field description identification	Detailed soil profiling required.
Consistency with respect to moisture content	High shear strength when dry. Soft and sticky when wet.
Structure	Shrinkage fissures and cracks, Shear surfaces have a glazed or shiny appearance
Color	May be of value on a regional or local level
Suction	Expansive soils have a high suction towards water when partly dry
Local Knowledge	Local authority engineers and builders may be a valuable source of information.

Source (Prof Alemayehu Teferra, 2007)

2.2.2 Laboratory Identification and Testing Techniques of Expansive Soils

Generally there are three different methods of identifying expansive soil in the laboratory. These are mineralogical identification, indirect methods and direct methods.

I. Mineralogical Identification

This method claims that the swelling potential of any clay can be evaluated by identification of the constituent mineral of this clay. The various techniques under these methods are: X-ray diffraction, differential thermal analysis, methylene blue, electron microscope, cation exchange capacity, etc. But these methods are not suitable for routine laboratory tests because of the following reasons; they are time consuming, require expensive test equipment and, the results are interpreted by specially trained technicians [30].

Table 2:2: Mineralogical identification of expansive soils

Test	Properties investigation	Parameters determined
X-ray diffraction	Characteristics crystal dimension	Proportion of various minerals present in colloidal clay
Differential Thermal analysis	Characteristics reaction to heat treatment	Area and amplitude of reaction peaks on thermos-grams
Electron microscopy	Size and shape of clay particle	Visual record of particles
Cation exchange capacity	Charge deficiency and surface activity of clay particles	CEC (Meg/100gm)

Source (Prof Alemayehu Teferra, 2007)

II. Indirect Methods

Indirect methods in which one or more of the related intrinsic properties are measured and complemented with experience to provide indicators of potential volume change of expansive clay soil. These methods include, such as the index property, potential volume change (PVC) method, and activity method, etc. [30] strongly states that erroneous conclusion can be drawn if the indirect methods are used independently.

Table 2:3: Indirect Methods of Expansive soil identification

Test	Properties Investigation	Parameters determined
Activity method	Plasticity index (PI) Percent by weight finer than 0.002	Activity (Ac) =plasticity Index/(% by weight finer than 0.002mm)
Potential volume change method	One dimensional swell and pressure of compacted, remolded sample under semi-strained controlled conditions	Swelling pressure lb/ft2 PVC(potential volume change)

Source (Prof Alemayehu Teferra, 2007)

III. Direct Method

The direct methods which involve actual measurement of volume change in an odometer type testing apparatus are generally grouped into swell or swell pressure tests. These testing methods are necessary to obtain measurable properties for predicting or estimating the magnitude of volume change the material will experience in order to ascertain approximate treatment and/or design alternatives [41].

2.3 Classification of Expansive Soils

The different classification systems are categorized into two:

I. General classification systems which have evolved over many years and are based on largely on correlation with actual performance.

II. Those devised specifically for classification of expansive soils. These systems are based on indirect and direct prediction of swell potential, as well as combinations, to arrive at a rating.

2.3.1 General Classification

Soils are classified in the general schemes; Unified Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials Method (AASHTO) according to index properties. Soils rated CL or CH by USCS, and A6 or A7 by AASHTO, may be considered potentially expansive soils [34].

2.3.2 Classification Specific to Expansive soils

The parameters determined from expansive soil identification tests have been combined in a number of different classification schemes to give a qualitative assessment of the degree of probability expansion. Unfortunately, there has not yet evolved a standard classification procedure, and a different scheme is used in practically every different location [28]. Some of the classification methods discussed in the literatures is given in the following section:

I. Skempton's Method

Skempton classified clays into three classes according to their activities as indicated in Table 2.4.

$$Activity = \left(\frac{PI}{\text{Percent of Clay} < 0.002mm}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(2.1)}$$

Table 2:4: Types of clay based on Activity

Degree of Activity	Activity
Inactive clay	Ac<0.75
Normal clay	0.75<Ac<1.25
Active clay	Ac>1.25

Based on the above classification, montmorillonitic clay (Expansive Clay) is defined as active, illitic clay as normal and Kaolinitic clay as inactive [17].

II. U.S.B.R. Classification Method

This method was developed by Holtz and Gibbs to establish the degree of expansion based on simultaneous consideration of shrinkage limit, plasticity index, percent smaller than 0.001mm, free swell (FS) and percent swell under a pressure of 1 psi. Thus, the abovementioned four parameters are used to indicate the criteria for identification of expansive soils by U.S. bureau of reclamation (1960) and are reproduced in Table-2.5.

Table 2:5: U.S.B.R classification Method

Classification of potential swell	Colloid content % <0.001mm	Plasticity Index (PI),%	Shrinkage Limit (SL), %	Potential Swell (%)
Low	<15	<18	>15	<10
Medium	13-23	15-28	10-15	10-20
High	20-31	25-41	7-12	20-30
Very high	>28	>35	<11	>30

III. Activity Method (Seeds's Classification Method)

The activity of a soil is taken as the dimensionless ratio of plasticity index to clay contents (C), both taken in percent. Seed et al, classify clayey soil according to its swelling potential defined as the percent vertical swell under a pressure of 1 psi of laterally confined sample compacted to the maximum dry density and optimum moisture content. According to the Seed et al, 1962 model Activity is calculated using Equation (2.2),

$$A = \frac{PI}{C-5} \qquad (2.2)$$

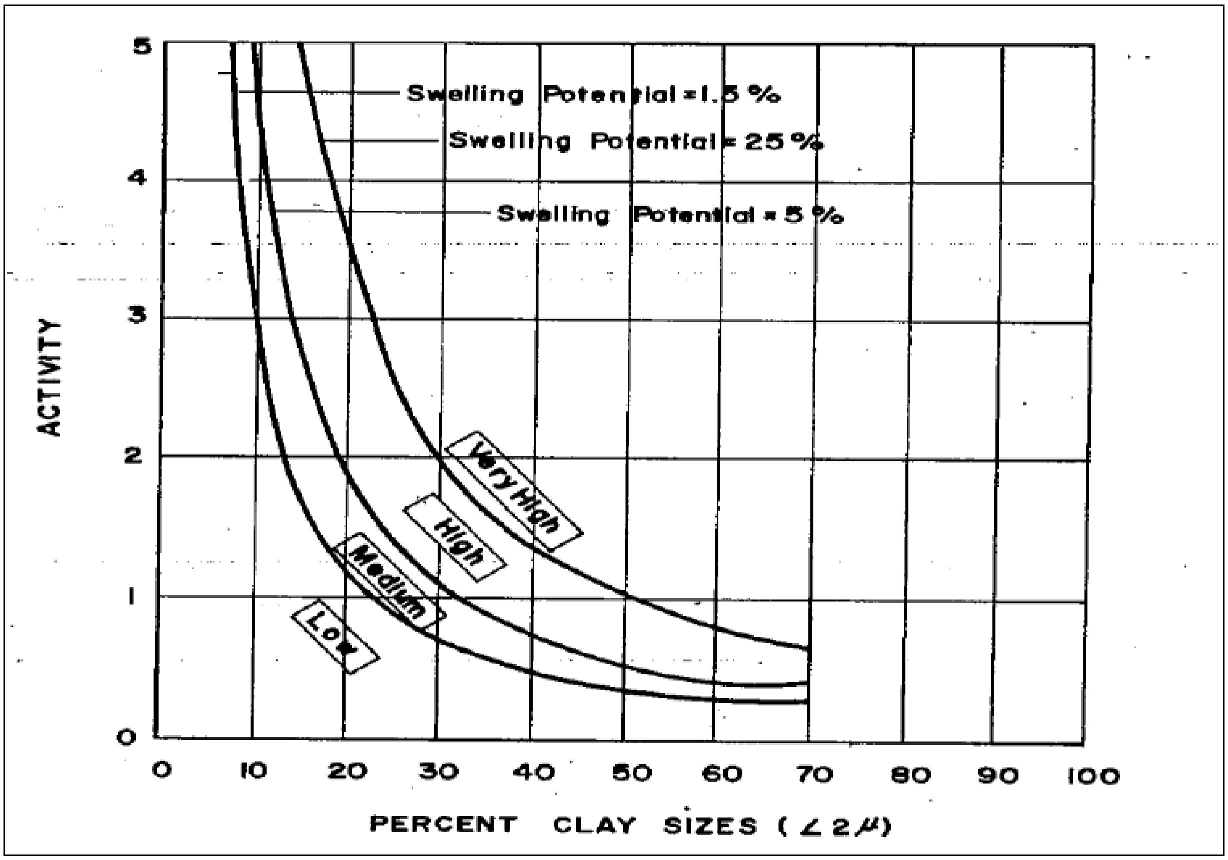


Figure 2:2: Classification chart for swelling potential

2.4 Distribution of Expansive Soils in Ethiopia

Distribution of expansive soil is generally a result of geological history, sedimentation and local climatic conditions. Arid climatic conditions and severe weathering environment prevailing in north eastern part of Africa promote the widespread occurrence of expansive soils.

In Ethiopia, expansive soils covered nearly 40% surface area of the country [26]. Expansive soils are observed in areas such as central Ethiopia, following the major trunk road like Addis Ababa - Ambo, Addis Ababa - Weliso, Addis Ababa – DebereBerehan, Addis Ababa- Gohatsion, and Addis Ababa - Mojo. Also covers the area like Mekelle, Bahirdar, Gambela, Arba Minch and the most Southern, South-west and south east part of the capital Addis Ababa area in which the most major recent construction are being carried out [10].

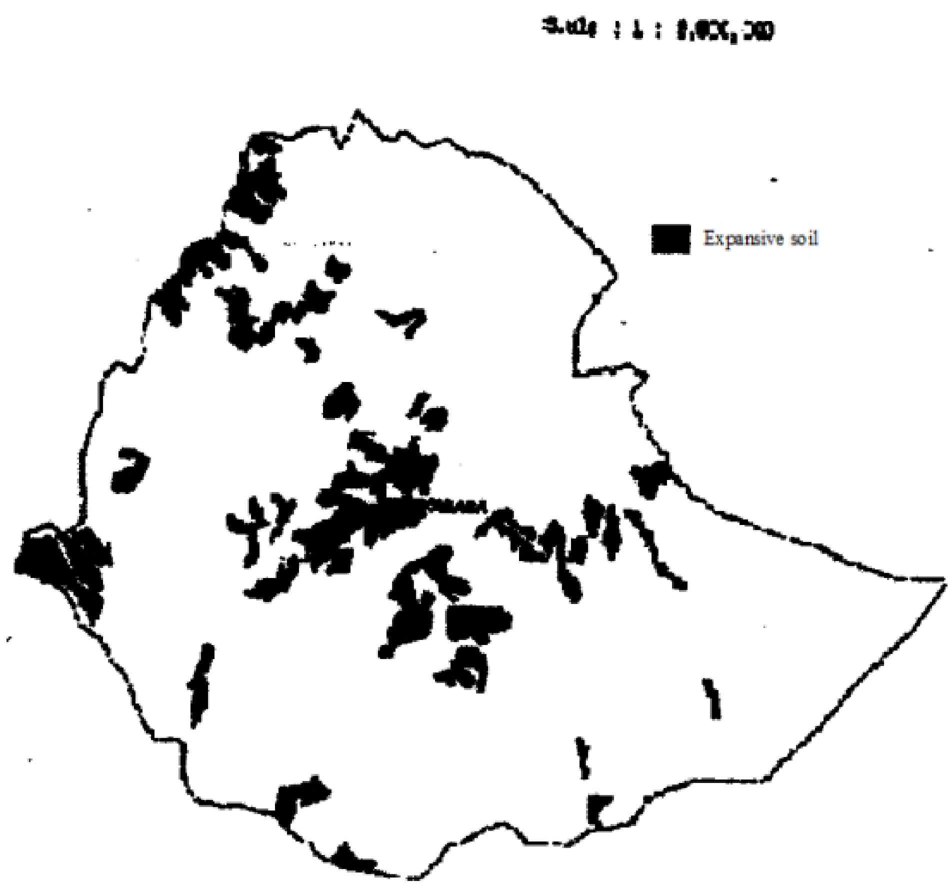


Figure 2:3: Distribution of expansive soils in Ethiopia

2.4.1 Availability of Wood Ash in Ethiopia

Rural areas in developing countries mainly depend on traditional biomass as fuel. For Ethiopia the main sources are woody biomass (78%), dung (8%), crop residue (7%) and petroleum (5%) Eshete et al.[45]. High demand for fuel wood and population growth in Ethiopia causes an acute scarcity of wood. Therefore, households are turning to dung and crop residue for energy Bewket [46]. Improved stoves and alternative cooking methods may decrease consumption of wood. However, wood consumption even increases in villages with good market access while remaining constant in remote villages Chen [47].

Wood ash is locally available and financially cheap. Most people of Ethiopia use wood as energy source as a result much amount of wood ash produced from wood combustion in the fire place.

In Ethiopia the major demand for forest products is for fuel wood. It is estimated that about 24 million cubic meters of wood is produced annually, of which 10 % is used for industrial and building purpose and the remainder for fuel wood and charcoal. Estimated consumption demand for fuel wood for energy varies from 49 to 64 million m3. According to Bios Bio energy system GmbH, infield gasses 21b, A-8010 Graz, Austria; one percent of ash is produced from unit combustion of wood, thus 0.49 to 0.64 million m3 wood ashes is produced annually in Ethiopia.

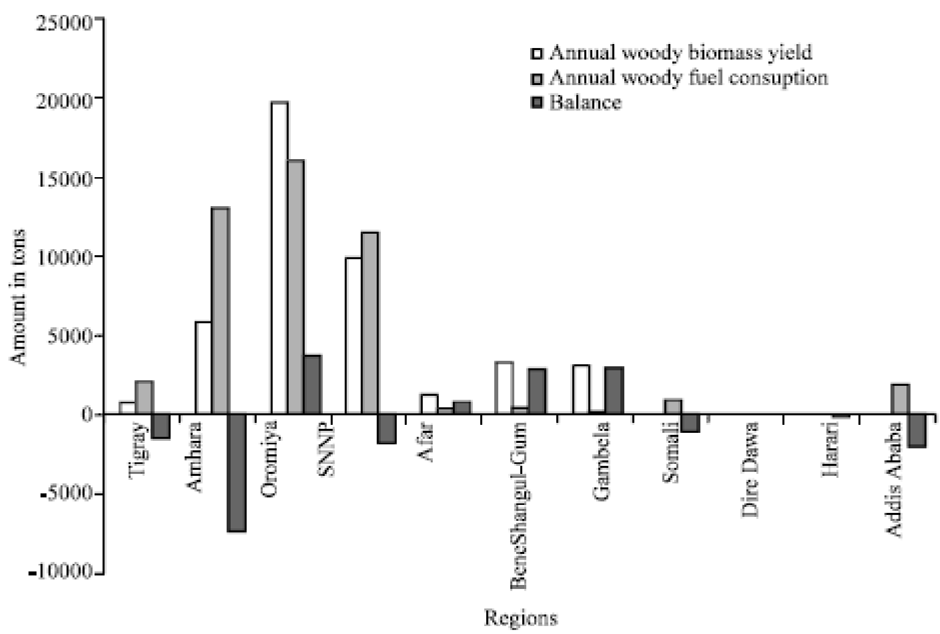


Figure 2:4 National woody biomass annual yields, wood fuel consumption balance

2.5 Pavement and Expansive Soils

Pavements are vulnerable to expansive soil damage because of their relative light weight nature extended over a relatively large area. Damage to pavements on expansive soil comes in four major forms:

- Severe unevenness along significant lengths- cracks may or may not be visible (particularly important for airport runways),
- Longitudinal cracking,
- Lateral cracking developed from significant localized deformations, and
- Localized pavement failure associated with disintegration of the surface [25].

Pavement designs are considered based on either flexible or rigid pavement systems [34]. However, whenever we want to assume the effect of the expansive soil, a number of issues should be considered:

- Choose an alternative route and avoid path on expansive soil,
- Remove and replace expansive soil with a non expansive alternative,
- Design for a lower strength and allow regular maintenance,
- Physically modify expansive soils through disturbance and re-compaction,
- Stabilization through chemical additives, such a lime treatment, and
- Control water content changes although very difficult over the life of a pavement [25].

2.6 Treatment of expansive soils

Treatment procedures are available in order to reduce or completely eliminate the swelling potential of expansive clays. The problematic soil is removed and replaced by a good quality material or treated using mechanical and/or chemical stabilization.

Different procedures can be used to improve the geotechnical characteristics of problematic soils by treating in situ.

These procedures are:

- Prewetting
- Moisture control
- Chemical stabilization
- Compaction control
- Soil Replacement
- The use of reinforcing elements (such as geotextiles and stone columns)

Preliminary soil investigation must be undertaken before choosing the appropriate type of soil treatment. The successful application of soil treatment procedures requires considerable experience and judgment regarding the soils in-situ, method of procedure application and limitations of procedures. The procedures require a specific methodology for every situation.

2.6.1 Prewetting

The procedure is being based on the theory that increasing the moisture content in the expansive foundation will cause swelling to occur prior to construction and if the high soil moisture content is maintained, the soil volume will remain essentially constant, achieving a no-heave state and therefore structural damage will not occur. There are great limitations regarding this procedure, being that expansive soils generally exhibit low hydraulic conductivity, therefore the time required for adequate wetting can be up to several years. Another major limitation to this procedure is that after a long period of time applying water to the soil, it can conduct serious loss of soil strength and can cause a reduction in bearing capacity and soil stability. [15] Regarding a prewetting project in which the soil moisture content did not increase appreciably after the first month of prewetting. For 5months thereafter, soil swelling continued. It was suggested that the first infiltration of water was probably taken by seams and fissures present in the clay and, therefore, full soil expansion did not occur. As time passed, the water moved from the fissures into the blocky soil mass, and swelling took place throughout the mass of the soil and not merely a seepage path.

This method may be best suited for soils with low or moderate expansion potential. Therefore a good assessment of the expansion potential should be undertaken.

2.6.2 Soil replacement

The procedure consists in replacing the expansive soil with non-swelling soils, this being the first requirement, followed by details concerning type of material, depth of replacement and extent of replacement. For man-made fill the extent of the layer should be limited. The depth of replacement is determined based on the swelling characteristics of the soil determined by laboratory testing, uplift pressure and the actual heave in the field. The cost of soil replacing is comparatively smaller to other techniques, like chemical treatments.

2.6.3 Compaction and moisture control

Refer basically to the determination of the optimum density of the soil and apply the compaction with 4-5% higher than the optimum in the field. It is a very difficult operation. Moisture content is the most important factor in the calculations but the process of re-compacting swelling clays at moisture contents slightly above their natural moisture content and at a low density should be the preferred approach. The main advantage of the technique is that it increases the strength of the soil, reduces swelling potential without adverse effects. With modern construction techniques, it is possible to scarify, pulverize, and re compact the natural soil effectively without substantially increasing the construction cost.

2.6.4 Subgrade stabilization

The process requires extent knowledge of the characteristics of the geotextiles, thus they must have specific mechanical and hydraulic properties in order to perform properly. Stone columns are extensively used to improve the bearing capacity of low grade soils. A stone column is one of the soil stabilization methods that are used to increase strength, decrease the compressibility of soft and loose fine graded soils and accelerate a consolidation effect. They are mainly used for stabilization soft soil such as soft clays, silts and silty-sands. It is believed that this method was used first in France in 1830s.The columns consist of compacted gravel or crushed stone arranged by a vibrator.

2.6.5 Chemical stabilization

Practice includes the use of lime and/or other chemicals, both organic and inorganic to stabilize expansive soils. These products can be cement, fly ash, and combinations between them in different percentages depending on the stabilized soil desired characteristics. Other chemicals are sodium or calcium based compounds (calcium hydroxide, sodium chloride etc) [11]. In general soil stabilization is a practice used in the field of highway construction and road pavements, but has been used successfully in the stabilization of the subgrade soil for individual buildings. [17] Classify chemical stabilizers in to three groups:

- Traditional stabilizers such as hydrated lime, Portland cement and fly ash.
- Non-traditional stabilizers**:** comprised of sulfonated oils, ammonium chloride, enzymes, polymers, potassium compounds.
- And by-product stabilizers**:** which include cement kiln dust, lime kiln dust etc.

A. Portland cement Stabilization

When stabilization of soil is done by mixing of pulverized soil and measured amount of cement and water it is known as soil cement stabilization. Cement has been found to be effective in stabilizing a wide variety of soils and waste materials such as pulverized bituminous pavements and crushed concrete. Cement-stabilized materials generally fall into two classes: soil-cement and cement modified soil.

 Soil-Cement(S-C) is a mixture of pulverized soil material and/or aggregates, measured amounts of Portland cement, and water that is compacted to a high density to serve as the primary structural base layer in a flexible pavement or as a sub-base for rigid pavements [17].

 Cement-Modified Soil (CMS) is a soil or aggregate material that has been treated with a relatively small proportion of Portland cement with the objective of altering undesirable properties of soils or other materials so they are suitable for use in construction. CMS is typically used to improve subgrade soils or to amend local aggregates for use as base in instead of more costly transported aggregates [17].

Table 2:6Cement requirements by volume for an effective stabilization of various soils, Das

Soil Type		Percent Cement by Volume
AASHTO Classification System	Unified Soil Classification System	
A-2 and A-3	GP, SP and SW	6-10
A-4 and A-5	CL, ML and MH	8-12
A-6 and A-7	CL, CH	10-14

The improvement of soils/aggregates containing clay through the addition of Portland cement involves four distinct processes discussed in the order of their occurrence: Cation exchange, flocculation and agglomeration/Particle restructuring, cementitious hydration, and pozzolanic reaction.

I. Cation Exchange

The first reaction in calcium reach stabilizers is hydration reaction which occurs between the reaction of cement and water. This reaction is an exothermic reaction which produces calcium ions and hydrogen ions. The calcium ions produced from the hydration process and other elements / compounds which are the constituents of the stabilizer are responsible for cation exchange.

CaO (lime) + $H_2O \rightarrow Ca^{++} + 2(OH)^-$ ………….. (2.3)

Cation exchange includes an immediate reaction of the clay with the stabilizer within few minutes of mixing, resulting in a soil with improved texture. The tetrahedral (T) and octahedral (O) combination of clay minerals in 1:1 (1T and 1O) or 2:1 (2T and 1O) have charge deficiency that results in the attraction of the cations or water molecule. Generally, sodium or potassium (Na^+ or K^+) are prevalent in clay minerals along with water. However, these cations can be replaced by the higher valance cations like Al^{+3}, Ca^{+2}, and Mg^{+2} etc. so called cation exchange. During this process calcium rich chemical stabilizer provides enough cations to replace the monovalent cations resulting in a reduced thickness of diffused double layer.

II. Flocculation and Agglomeration

Flocculation and agglomeration, which is made possible through cation exchange [22], is the process of clay particles altering their arrangement from a flat, parallel structure to a more random edge-to-face orientation (Figure 2.5). The restructuring of modified soil/aggregate particles changes the texture of the material from that of a plastic, fine-grained material to one more resembling a friable, granular soil/aggregate. The reduced size of the double layer due to cation exchange, as well as the increased internal friction of clay particles due to flocculation and agglomeration, result in a reduction in plasticity, an increase in shear strength, and an improvement in texture. As with cation exchange, the particle restructuring process happens rapidly.

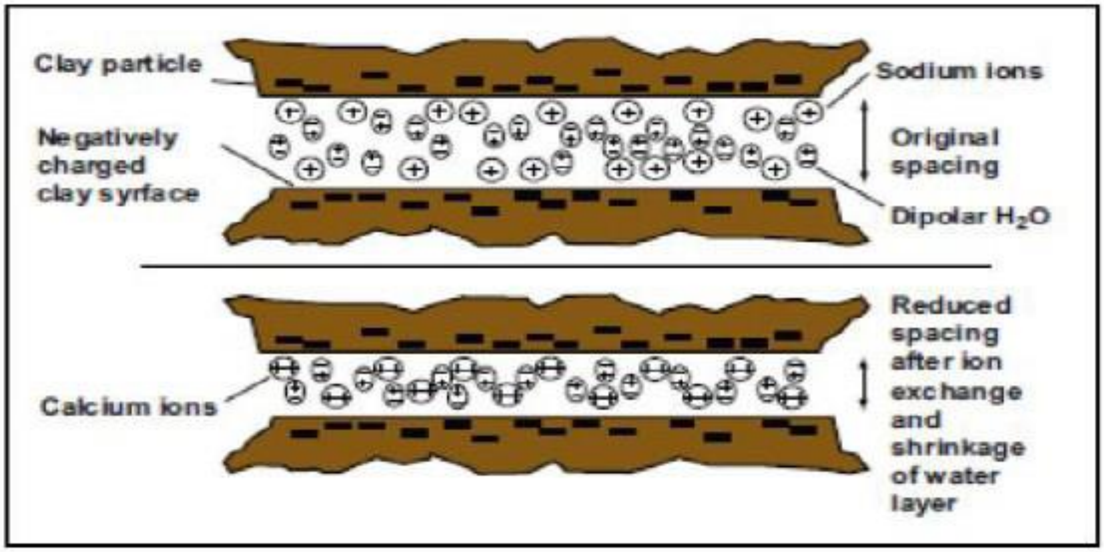


Figure 2:5: Cation Exchange

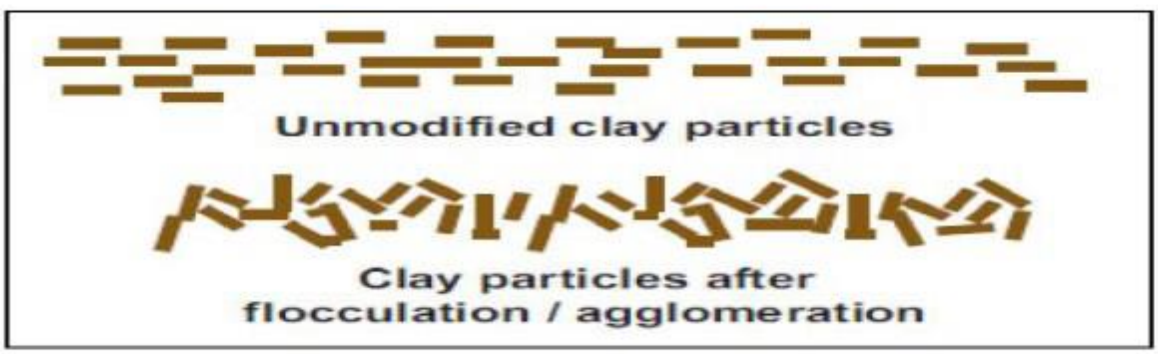


Figure 2:6: Particle Restructuring

III. Cementitious Hydration

Cementitious hydration (Figure 2.6) is a process that is unique to cement, and produces cement hydration products referred to in cement chemistry as calcium-silicate-hydrate (CSH) and calcium-aluminum-hydrate (CAH). CSH and CAH act as the “glue” that provides structure in a cement-modified soil/aggregate by stabilizing flocculated clay particles through the formation of clay-cement bonds. This bonding between the hydrating cement and the clay particles improves the gradation of the modified clay by forming larger aggregates from fine-grained particles. This process happens between one day and one month after mixing.

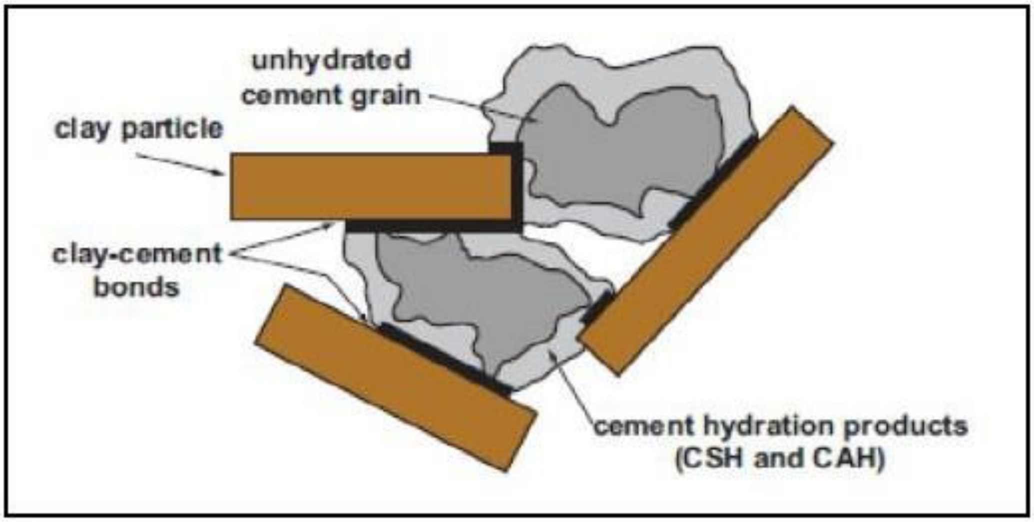


Figure 2:7: Cementinous hydration

IV. Pozzolanic Reaction

In addition to CSH and CAH, hydrated Portland cement also forms calcium hydroxide, or Ca $(OH)_2$, which enters into a pozzolanic reaction. The calcium which is released during hydration process is in suspension of stabilizer-soil-water and will be available for the stabilization of soil. The general reaction of the cement with water that yields calcium is presented in equations 2.2 and 2.3. This phenomenon is illustrated in Figure 2.7.

C3S + H ------------------------ C-S-H + Ca $(OH)_2$ ……………… (2.4)

C2S + H ------------------------ C-S-H + Ca $(OH)_2$ ……………… (2.5)

Where, H= H_2O, C = Ca, S= SiO_2, C3S = Tri-calcium silicate, C2S= Di-calcium silicate and C-S-H = $C_3S_2H_3$.

This secondary soil modification process takes the calcium ions supplied by the incorporation of portland cement and combines them with the silica and alumina dissolved from the clay structure (at highly alkaline solution pH >12.4) to form additional CSH and CAH (Figure 2.7)[22][23]. The pozzolanic reactions take place slowly, over months and years, and can further strengthen a modified soil/aggregate as well as reduce its plasticity and improve its gradation. The pH environment in the system initiates further reaction of the silica and alumina with the clay particles, hence proving extra strength to the stabilized soils [22]. The minimum PH of 12.4 is necessary in order to maintain the pozzolanic reaction [23].

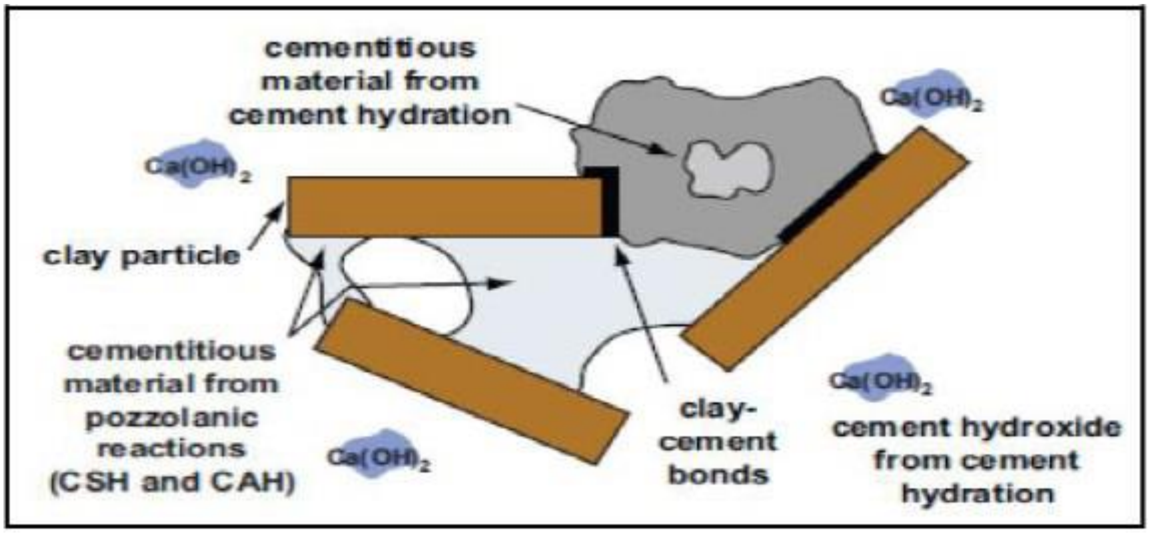


Figure 2:8: Pozzolanic Reactions

B. Soil Stabilization Using Wood ash

Various research works have been undertaken for improving the expansive nature of the black cotton soil using materials like wood ash, as admixture. Wood ash is a locally available material residue powder left after the combustion of wood, such as burning wood in a home fireplace or an industrial power plant. It is generally discarded as waste and dumped outside of house or landfill, which increase the volume of landfill. The experiments on wood ash showed that wood ash have an alkaline nature which clearly defines its potential as raw material resource for the production of acidic and other different types of soil improvers. Therefore as an alternative solution, wood ash can be used as a potential soil stabilizer through the chemical reaction. It is found from the literature that the concentration of heavy metals present in the wood ash is very low [18]. The research by [36] showed that wood ash certainly meets some of these conditions, such as plasticity reduction, swell reduction, improved stability, and substantial strength gain but fails to meet the benefit of continued strength gain with time. These results from research imply that although wood ash provides most of the beneficial effects of lime in soil stabilization, it is unlikely to be a substitute for lime in soil stabilization for engineering construction [36]. Another research [29][11] on wood ash stabilization clay soils found that there is an enhancement of the engineering properties of existing soil in stabilized forms particularly unconfined compressive strength (UCS), shear strength parameters, workability, and compaction and compressibility characteristics.

The study on the composition of wood ash Table 2.6 [18] Showed that wood ash is clearly of an alkaline nature (pH of aqueous extract = 12.61). This clearly defines the high potential of this type of ash as a raw material resource for the production of acidic soil improvers. Other indicators like free moisture content, electrical conductivity, organic matter content and overall nutritional and toxic elements and their mobility in aqueous solutions also confirm that wood ash is suitable for soil improver production [18].

Table 2:7: Chemical composition of Wood ash

Constituents	% in wood ash
CaO	29.80
MgO	5.25
K_2O	9.55
Fe_2O_3	0.95
Na_2O	7.50
SiO_2	25.8
Al_2O_3	14.72
P_2O_5	2.33
TiO_2	0.70
SO_3	0.70
Loss on ignition, LOI	2.70

Source: (Ek. Serafimova, 2011)

C. Behavior of Cement and wood ash blend

According to the research by [1] presented the behavior of wood ash - OPC and conducted chemical analysis of wood ash, bulk density, sieve analysis and specific gravity of wood ash and aggregates, consistency, setting time and slump test of the fresh paste the suitability of the materials for cement replacement. Mix ratio of 1:2:4 and percentage replacement level of 0, 10, 20, 30 and 40 percent's of cement by wood ash were used. 150mm×150mm cubes were cast, cured and crushed at 28 and 60 days to determine their compressive strength. Test result indicates that the wood ash is slightly pozzolanic, water demand increases as the ash content increases and the setting time of the paste increases as the ash content increases. Compressive strength of wood ash OPC concrete it increases with age at curing with optimum replacement of cement by wood ash of 20%. The setting times of wood ash / OPC paste increases as the ash content increases; the 10% and 20% wood ash paste satisfy the recommended standard for ordinary Portland cement paste. 30% and 40% wood ash paste gave higher values of setting times which do not satisfy the standard, The compressive strength of the concrete with 20% wood ash content increased appreciably at 60 days. The optimum replacement level was therefore 20%.

2.7 Previous studies

Chukwuebuka E. & Ogbonnaya I. [16] assessed the combined effect of wood ash a waste product from a bread bakery and lime (calcium oxide) on the geotechnical properties of expansive soils. The geotechnical properties of the soil such as grain size distribution, consistency limits, free swell potential, compaction, and unconfined compressive strength of the natural soil and that of the soil with varying proportion of wood ash and lime was also examined. The results revealed that the natural soil which is classified as highly plastic inorganic soil, on addition of wood ash and lime in the optimum proportion of 78%-18%-4% by weight of soil-wood ash-lime admixture showed reduction in the plasticity index and linear shrinkage, thus improving the workability of the natural soil. There was also reduction in the free swell potential of the natural soil, improvement in the compaction properties of the natural soil, and increase in the shear strength value of the natural soil which drastically improved more after 28 days of curing.

J. James [24] evaluated the strength benefits obtained by amending cement stabilization of an expansive soil by using saw dust ash (SDA), a waste generated in wood milling industries due to burning. The experimental program involved the preparation of cylindrical specimens of size 38 mm x 76mm for evaluating the unconfined compression strength (UCS) of the cement stabilized and amended samples cured for varying periods of 2 hours, 7, 14 and 28 days. Two cement contents of 2% and 6 % by weight of soil were adopted to stabilize the soil. The SDA amended cement stabilized samples adopted SDA contents of 5%, 10% and 20% by weight of soil. The investigation revealed that 5% SDA amendment of cement stabilization can result in up to 26%

increase in early strength and 20% increase in delayed strength. Based on the predicted CBR values, pavement thickness can be reduced up to 8.3%.

Magdi M. E. Zumrawi [28] undertaken an experimental program to study the stabilization of pavement subgrade by fly ash activated with cement. Expansive soil treated with varying percentages of fly ash, 0, 5, 10, 15, and 20 percent combined with 5% cement content were studied. Consistency limits, compaction, California Bearing Ratio, swell potential and swell pressure tests were conducted on treated and untreated soils. The experimental results show that addition of cement-fly ash admixture to the soil has great influence on its properties. It was found that the optimum dosage of fly ash is 15% mixed with 5% cement revealed in significant improvement in strength and durability and reduction in swelling and plasticity properties of the soil.

Bayshakhi Deb Nath [11] has evaluated the extent to which wood ash can improve the fundamental geotechnical properties such as consistency, compaction, UCS, shear strength, and settlement characteristics of untreated and wood ash-based clayey soil. The soil was stabilized through 5%, 7.5%, 10%, and12.5% wood ash content. It is observed that there is an improvement of geotechnical properties of the ash-treated soil. The results obtained showed that. Wood ash reduces the plasticity and maximum dry density of clay, while more water is required for the agglomeration and flocculation of clay particles through cation exchange reaction and coagulation with the consequent reduction in the amount of fines, ash stabilization causes an increase in unconfined compressive strength in the soft clayey soil and 10%wood ash-clay mixture optimizes the results. The larger the ash percentage inserted, the greater the strength is and there is a sharp improvement in the shear strength parameters with the addition of wood ash. The angle of internal friction has witnessed an improvement of about 85% for 10% cement addition, while the cohesion value has an improvement of only about6% for 10% ash addition.

Supanic, K. & Obernberger [42] had analyzed about utilization of wood ash as a stabilizer in road construction. Preliminary laboratory tests were performed using six different wood ash proportions in order to evaluate the chemical and mechanical suitability of the ash samples as a binder in road construction. They inferred that wood ash feature a significant amount of free CaO, which is the relevant reacting component in the soil stabilization process.

Okagbue, C. O. [36] evaluated the efficiency of wood ash for expansive soils stabilization. The evaluation involved the determination of the geotechnical properties of expansive soil in its natural state as well as when mixed with varying proportions of wood ash. The parameters tested included the particle size distribution, specific gravity, Atterberg limits, compaction characteristics, CBR and the compressive strength. The CBR and strength tests were repeated after 28 days of curing of the treated samples. Results showed that the geotechnical parameters of expansive soil are improved substantially by the addition of wood ash; plasticity was reduced by 35% and CBR and strength increased by 23-50% and 49-67%, respectively, depending on the compactive energy used. The highest CBR and strength values were achieved at 10% wood ash. Results also showed that curing improved the strength of the wood ash treated soil.

CHAPTER THREE

3. Research Methods, Materials and Procedures

3.1 Introduction

Expansive subgrade soil in its natural behavior is very weak and does not have enough stability for any type of construction work. Thus to make the subgrade soil stable and to improve its engineering properties stabilization is needed. The main objective of the study is to evaluate the properties of stabilized Expansive soils with cement and wood ash and determining the optimum amount of stabilizing agents that can be used in flexible pavement subgrade constructions. The specific objectives of the study were evaluating the effect of cement and wood ash on the properties of the expansive clay soil with respect to the classification, Atterberg limit, UCS, compaction and CBR tests and also characterizing the engineering properties of untreated and treated specimens. The samples used in this exploration were expansive clay soil, OPC and wood ash. The soil that has been used in the investigations was air-dried and passing through 4.75mm sieve.

3.1.1 Study Area

The study area is found in Addis Ababa, Ethiopia; Addis Ababa is the capital city of Ethiopia and the African Union and is often called the “African capital “due to its historical, diplomatic and political significance for the continent. Located at9.02°00’16.68” N 38044’49.39” E and about 2,440m above sea level, it is the third highest capital in the world order by elevation. It is located in the geographic center of the country. Expansive soil used for the study is black cotton soil obtained from Bole sub city Woroda 02 of Bole –Bole Bulbula road project around bole airport cargo.

3.2 Materials

3.2.1 Expansive soil

Expansive or swelling soil is a highly plastic soil that normally contains montmorillonite and other active clay minerals. Expansive soil is a commonly identified problem which has made scientists concern about the design, protection, and operating of highway and structural systems. Expansive soil used for the study is black cotton soil obtained from Addis Ababa city, Bole – Bole Bulbula road project around Bole airport cargo.

Figure 3:1: Expansive soil used in the study

3.2.2 Cement

Cement is a substance which acts as a binding agent for construction materials which is obtained by burning and crushing the stones containing clay, carbonates of lime and some amount of carbonate of magnesia at very high temperature. The clay content in such stones is about 20 to 40 percent. When Cement mixed with water a chemical reaction called hydration takes place which produces a very hard and strong binding medium with the construction particles. Derba Ordinary Portland Cement (OPC) grade 42.5N type I cement was used in this research.

Raw Materials of Cement Composition: There are three raw materials of Portland cement composition:

Calcareous rocks ($CaCO_3$ > 75% such as limestone, chalk)

Argillaceous rocks ($CaCO_3$ < 40% such as clay and shale)

Argillo-calcareous rocks (40-75% $CaCO_3$ such as clayey limestone, clayey).

Materials from any two of these groups may be used for Portland cement production provided that they must contain in proper form and proportions of lime, silica and alumina and iron oxide. The oxides account for over 90% of the cement.

Ordinary Portland cements are composed of four basic chemical compounds shown with their names, chemical formulas and abbreviations as: Tri-calcium silicate hardens rapidly and responsible for initial set and early strength. Di-calcium silicate hardens slowly and its effect on strength increases occurs at ages beyond one week. Tri-calcium aluminate contributes to strength development in the first few days and the first compound to hydrate. It is the least desirable component because of its high heat generation and it's unstable with soils and water containing moderate to high sulfate concentrations. Tetra-calcium alumino ferrite lowers clinkering temperature. It contributes very little to strength of concrete even though it hydrates very rapidly.

Table 3:1: Main chemical compounds of OPC

Name of Compounds	Chemical Composition	Usual Abbr.	Percentage %
Tricalcium Silicate	3CaO.SiO2	C3S	51
Dicalcium Silicate	2CaO.SiO2	C2S	23
Tricalcium aluminate	3CaO.Al2O3	C3A	8
Tetracalcium alumino ferrite	4CaO.Al2O3Fe2O3	C4AF	9

Storage of Cement: The OPC cement was the main constituent in stabilization of the materials, thus it should be carefully stored to prevent from moisture which affects its properties, for the reason that to prevent early setting and hardening. Cement must be stored in areas where air circulation is less, because the air contains moisture that cause to set early. Cement should be kept in damp proof floor and roof. The cement bags should be stalked in such a way that it facilitates inspection from all the four sides. In one stalk not more than 10 bags should be loaded in vertical space. In general, wooden sleepers should be used in the bottom, which should be at least 20 cm for the floor and it should also be at least 30 cm away from the wall of the stalk room. At the time of using cement, it should be ensured that the cement bags which had arrived first are put to use first. If the cement is more than 6 months old, it should be used only after performing the strength test of cement [20].

Table 3:2: Strength reduction of cement with time with respect to strength of fresh cement

Time elapsed in months	% Reduction in strength	Remaining strength
3 months	20%	80%
6 months	30%	70%
12 months	40%	60%
24 months	50%	50%

(Source: Ethiopian Roads Authority, 2013)

3.2.3 Wood ash

Wood ash is the residue powder left after the combustion of wood, such as burning of wood in home fireplace or industrial power plant. Wood ash used for the study was collected form Addis Ababa Bole sub city woreda 02 from combustion of Eucalyptus (Bahrizaf) wood for house use purpose.

Figure 3:2: Wood ash used in the study

3.3 Research Methods and Experimental Design

This research was designed to answer the research questions and meets its objectives based on the performed laboratory investigation results.

- The first step in the research work was sample collection from the case study. At this stage, sample materials Expansive soil, wood ash and ordinary Portland cement were collected.
- The second step was laboratory investigation of material qualities and the performance of Expansive soil and combined mix design of Expansive soil-cement-wood ash.
- The third step of the research was the analysis and interpretation of the laboratory outcomes.
- The fourth step was the conclusion and recommendation.

Laboratory experiments were conducted with additive amount of cement (2%, 4%, and 6%) and wood ash percentage (5%, 10%, 15% and 20%) in dry weight. In general, the entire research method activities followed was organized on Figure 3.3.

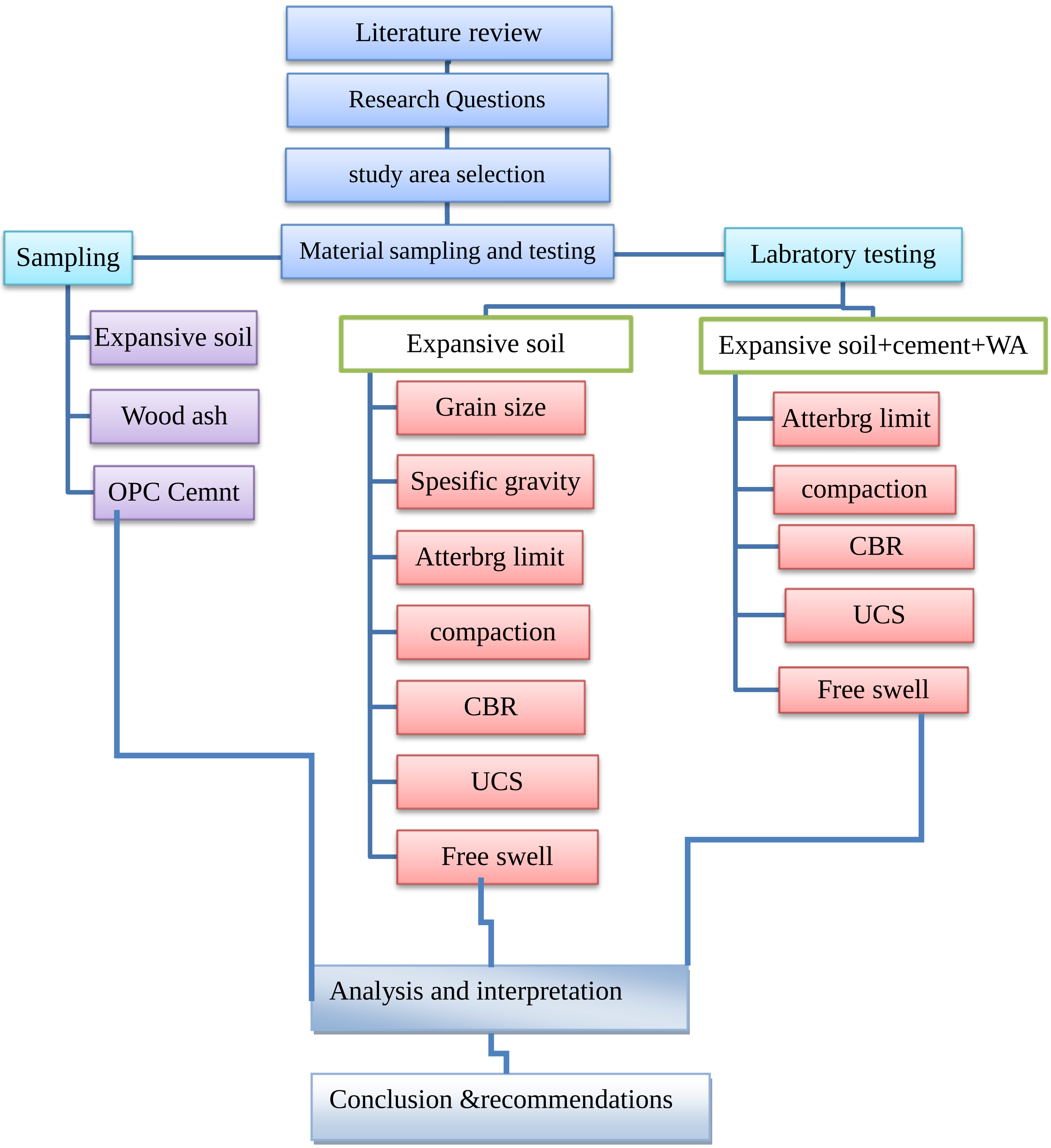


Figure 3:3: Research Methodology and experimental design

3.4 Soil Samples Tests and Laboratory Procedures

Initially the properties of natural soil were determined. The soil was then stabilized with cement and Wood ash. The amount of Wood ash for stabilization is taken in the proportions of (5%, 10%, 15%, and 20%) by dry weight of soil and the amount of cement was (2%, 4% and 6%) by dry weight of soil. To establish engineering property of native/natural soil and treated soil samples the following tests were conducted. Natural moisture content, grain size analysis, Atterberg limits, specific gravity, free swells, modified Proctor, California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS).Mix Proportion Samples of soil, wood ash and cement used for Stabilization are showmen in table 3.3.

Table 3:3: Mix proportions used for stabilization

MIX Proportions	MIX designation
1. Expansive soil	ES
2. Expansive soil + 5% wood ash + 2% cement	2C5WA
3. Expansive soil + 10% wood ash + 2% cement	2C10WA
4. Expansive soil + 15% wood ash + 2% cement	2C15WA
5. Expansive soil + 20% wood ash + 2% cement	2C20WA
6. Expansive soil + 5% wood ash + 4% cement	4C5WA
7. Expansive soil + 10% wood ash + 4% cement	4C10WA
8. Expansive soil + 15% wood ash + 4% cement	4C15WA
9. Expansive soil + 20% wood ash + 4% cement	4C20WA
10. Expansive soil + 5% wood ash + 6% cement	6C5WA

11. Expansive soil + 10% wood ash + 6% cement	6C10WA
12. Expansive soil + 15% wood ash + 6% cement	6C15WA
13. Expansive soil + 20% wood ash + 6% cement	6C20WA

Figure 3:4: Air drying of Native soil

3.4.1 Initial Moisture Content of the Soil

The oven-drying method (ASTM D 2216) was used to determine the moisture contents of the samples. Small, representative specimens obtained from large bulk samples were weighed as received, then oven-dried at 105°C for 24 hours. The sample was then weighed, and the difference in weight was assumed to be the weight of the water driven off during drying. The difference in weight was divided by the weight of the dry soil, giving the water content on a dry weight basis.

3.4.2 Particle Size Analysis of Soil Samples (ASTM D 422)

The sieve and hydrometer analysis tests were conducted to determine the percentage of different grain sizes contained within a soil according to ASTM D 422. This test was performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis was performed to determine the distribution of the coarser, larger sized particles, and the hydrometer method was also used to determine the distribution of the finer particles.

The first analysis is meant for coarse grained soils, particles greater than 75ùm, which can easily pass through a set of sieves. The second analysis is used for fine grained soils, particles smaller than 75um or sieve NO.200. A soil mass may contain the particles of both types of soils, a combined analysis compromising both sieve analysis and sedimentation analysis may be required for such soils.

Particles size smaller than 0.2 micron cannot be determined by the sedimentation method, these can be determined by electron microscope or by X-ray diffraction techniques.

Hydrometer Analysis of Soil Samples: - A hydrometer test has been carried out to determine the particle distribution of finer particles. The apparatuses used in the tests were balance, beaker, thermometer, hydrometer, sedimentation cylinder, control cylinder, timing device and beaker.

Figure 3:5: Hydrometer analysis

3.4.3 Atterberg Limits (ASTM D 4318)

Representative samples of each soil were subjected to Atterberg limits testing to determine the plasticity of the soils. An Atterberg limits device was used to determine the liquid limit of each soil using the material passing through a 425 μm (No. 40) sieve. The liquid limit was determined as the water content, at which a peat of soil in a standard cup and cut by a groove of standard dimensions flowed together at the base of the groove for a distance of 13 mm (1/2 inch) when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second. The plastic limit of each soil was determined by using soil passing through a 425 μm sieve and rolling 3-mm diameter threads of soil until they began to crack. The plasticity index was then computed for each soil based on the liquid and plastic limit obtained. The liquid limit and plasticity index were then used to classify each soil.

Figure 3:6: Liquid limit determination

3.4.4 Specific gravity (ASTM D 854-00)

Values for specific gravity of the soil solids were determined According to ASTM D 854-00 Standard Test for Specific Gravity of Soil Solids by Water Pycnometer Specific gravity was then calculated by taking the ratio of the mass of given volume of solids to the mass of an equal volume of water at 4°C.

Figure 3:7: Specific gravity determination

3.4.5 Moisture- density relationship (AASHTO T-180)

AASHTO T-180 was followed in this test and the test was performed on the soil first air dried and sieved usually through the 4.75-mm (No. 4) sieve, mixed thoroughly with water and then compacted in layers. In the standard compaction test, the soil was compacted in three layers each layer was the compacted with 25 blows of a 4.5 kg (10 1b) hammer falling 45.7 cm (18 in). For each proctor test five runs were conducted by increasing water content 2% of the preceding tests using 8% water content an initial run. These series of determinations were continued until there was either a decrease or no change in the wet unit mass (g/cm3) of the compacted soil.

Figure 3:8: Compaction test

3.4.6 California Bearing Ratio (CBR) Values (AASHTO T-193)

Three point CBR tests were done according to AASHTO T-193 to determine the strength of the sample soil and how it will behave when subjected to loading. An air dried soil and passing through #4 (4.75mm) sieve were mixed at optimum water content and some percentage of maximum dry unit weight, then three specimens from soil were prepared to within ±0.5 percentage point of optimum water content and compacted using 65, 30, and 10 blows in order to prepare specimens having unit weights above and below the desired value. The compacted soil in CBR mold was then soaked in water for four days with 4.5 kg surcharge load applied to it. After the end of soaking period, the CBR mold was removed from water and drained for 15 minutes before CBR penetration. The CBR was performed on a compression Machine and seat the penetration piston on the surface with a 44.5N load and then results were recorded the total load when the piston has penetrated the soil to a depth of 2.54mm and 5.08mm.

3.4.7 CBR Swelling Test Measurements

Expansive soils, especially soils contain montmorillonite clay, increase in volume when their moisture contents increase. These soils are not suitable for roadway constructions. Therefore, to evaluate the potential swell of the samples the following procedures have been accomplished.

Figure 3:9: Sample preparation for CBR & CBR soaking

3.4.8 Free Swell Test

Free swell may be considered as a measure of volume change in expansive clay upon saturation and is one of the most commonly used in the field of geotechnical engineering to estimate the swelling potential .The test was performed by pouring 10cc of oven dry soil passing through sieve NO. 40 (425 μm) in to a 100cc gradated cylinder jar filled with water, notice the swelled volume of the soil after it comes to rest which may take 2-3 days .Then the free swell was calculated using Equation 3.1:,

Figure 3:10: Free swell test

$$FS = \frac{(V-Vo)}{VO} * 100 \text{………………………. (3.1)}$$

Where;

FS= free swell, %

V=soil volume after swelling, cm^3

VO= volume of dry soil, 10 cm^3

3.4.9 Unconfined Compressive Strength (ASTM D2166)

Test specimens were prepared for different cement-wood ash content with standard ASTM D 2166 densities for the respective cement-wood ash content. Test specimens were compacted in a 22.5 cm height by 11 cm diameter cylinder mold using modified proctor compaction. The compaction procedure was using 4.5 kg rammer with a drop of 450 mm, applying 25 blows evenly distributed blows to each of the three equal thick layers. The mass of the specimen, the length of specimen, and diameter of the specimen at mid height were determined and recorded. Having determined the mass and dimension of the specimens, then it was placed in the loading device. A strain rate of 2 percent per minute was used with measurements taken every 20 divisions on deformation until the load values decreased with increasing strain. The specimen was removed from the compression device and a sample for water content determination was taken.

Figure 3:11: UCS test of remolded sample

3.5 Cement and Wood ash Stabilized Expansive Soil Experiments

3.5.1 Atterberg Limits of Cement and Wood ash Stabilized Soil

The tests were executed to define the plastic and liquid limits and plasticity index of Expansive soils mixed with cement and wood ash. The apparatus and procedures that were implemented in the treatment of cement-wood ash stabilized soil were similar to the apparatus and procedures used in section 3.4.3. The trial amounts of cement (2%, 4%, and 6%) and wood ash (5%, 10%, 15% and 20%) was mixed with the expansive soil until they form uniform color. Water was added and mixed thoroughly to a uniform consistency above the PL to determine the LL using Casagrande's cup.

3.5.2 Moisture-Density Relations of Cement –Wood ash Stabilized Soil

These methods determine the relationship between the moisture content and the density of cement-wood ash stabilized soil when compacted according to AASHTO T-180. The soil samples that were prepared was passed sieve number 4 (4.75mm opening) in order to avoid oversized natural particles.

Moisture-Density Relation procedures of Cement-Wood ash Stabilized Clay Soils

I. Add the required amount cement-wood ash component to the soil (2%, 4%, and 6%) cement with wood ash (5%, 10%, 15% and 20%) and mix the cement-wood ash and Expansive soil thoroughly to a uniform color.

II. To complete absorption waiting the cement-wood ash and Expansive soil mixture 5 to 10 min then compact the mixture similarly using the procedures in (section 3.4.5).

3.5.3 California Bearing Ratio of Cement and Wood ash Stabilized Soil

The tests were carried out to determine the stabilized strength and swelling potential of cement-wood ash and Expansive soil using similar procedures as in section 3.4.6. The quantities of stabilizers used were cement (2%, 4%, and 6%) with wood ash (5%, 10%, 15% and 20%) prepared in dry weight.

3.5.4 Unconfined Compressive Strength of Cement and Wood ash Stabilized Soil

The purpose of this test is to determine the UCS of cement-wood ash and expansive soil which is used to estimate the unconsolidated undrained shear strength of the expansive soil within the stabilizers under unconfined conditions using similar procedure of section 3.4.9. According to the ASTM 2166-98a standard, the UCS (qu) is defined as the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test. The amount of cement (2%, 4%, and 6%) with wood ash percentage of (5%, 10 %, 15% and 20%)were prepared in dry weight and the required quantity of water also determined from moisture-density relations of cement-wood ash stabilized expansive soil. The stabilized prepared specimens also cured for seven days.

CHAPTER FOUR

4. Results and Discussion

4.1 Introduction

This chapter presents test results, discussion and analysis of all experimental work that was performed on native soil and treated/stabilized soils with cement-wood ash mixtures. Generally, the effect of each stabilizer mix on Atterberg limits, swelling, moisture-density relation, CBR values and UCS values were established by varying percent of cement-wood ash stabilizer and compared with native soil/untreated soil engineering property.

4.2 Experimental Result of Native Soil Sample Material

4.2.1 Moisture Content of Native Soil Sample

The in-situ soil moisture content of soil sample which were collected from study area was determined by measuring the weight of the sample after collection and again also by measuring the sample after oven dry. Thus the moisture content was determined by the following equation

$$MC = (WW - WS)/(WS - WP) * 100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.\ 1)$$

MC= Moisture content in percent

WW=weight of weight soil +pan

WS=Weight of dry soil+ pan

WP=Weight of pan

Take an average value of test I&II to find the NMC Thus NMC= 58%

4.2.2 Specific Gravity of Native Soil Sample

The specific gravity of native soil samples was determined by means of a Pycnometer method executed according to the procedures in section 3.4.4.

The calculation of specific gravity of soil solids was preceded by equation 4.2.

$$GS = \frac{M2-M1}{((M4-M1)-(M3-M2))} * 1000 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. (4.2)$$

Where;

M1 is the mass of density bottle

M2 is the mass of bottle and dry soil

M3 is the mass of bottle, soil and water

M4 is the mass of bottle full of water only

Gs= 2.75

4.2.3 Sieve Analysis Result of Native Soil Samples

Sieve analysis test on native soil samples have been executed according to the procedures in section 3.4.2. The sieve analysis was performed to determine the distribution of the larger sized particles and then hydrometer method was used to determine the distribution of the finer particles.

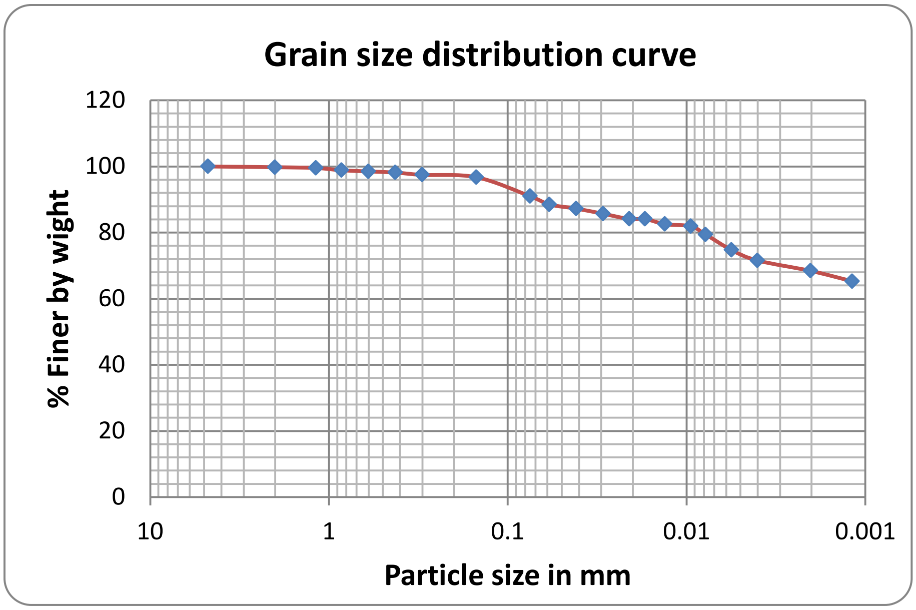


Figure 4:1: Grain Size Distribution Curve for native soil sample

4.2.4 Atterberg Limits of Native Soil sample

Atterberg limits of native soil samples were determined using ASTM as explained in section 3.4.3 and results are tabulated in table 4.3.

$$PI = LL - PL \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.\ 3)$$

Where,

PI = plasticity index

LL = liquid limit

PL = plasticity limit

The suitability of fine grained soils for pavement construction was also determined using Group Index (GI). A value of GI equal to zero shows a good sub grade material whereas a GI of 20 or more shows a poor subgrade material.

$$GI = (F - 35) * \big(0.2 + 0.05 * (LL - 40)\big) + 0.01 * (F200 - 15) * (PI - 10) \ldots\ldots \quad (4.4)$$

Where:

F200= Percentage by mass passing 75µm sieve (sieve no.200), specified as whole number

LL= Liquid limit (%) specified as whole number

PI = Plastic Index (%) specified as whole number

Table 4:1: Atterberg limit and Group index value of native soil sample

Atterberg Limit of Soil Samples	
LL	88
PL	36
PI	52
GI	20

4.2.5 Soil Classification of native soil sample

A. Using AASHTO Classification System

The classification of trail soils was executed based on AASHTO M145. The main interest is the GI which is a function of LL, PI and the amount of material passing the 0.075mm. In well compacted and good drainage, the load bearing ability of the sample soil may be assumed an inverse ratio to its group index that is a group index of zero indicates a good subgrade material and a group index of 20 or more indicates a poor subgrade material. Referring section 4.2.3 grain size analysis it has been obtained that percentage passing sieve No.200 =96%. From this result the soil is grouped under [A-4]-[A-7]. Further classification is done using the Plastic index and liquid limit, the soil is under A-7. But there are two groups of soil under **A-7**. Further classification is again performed using the following chart LL=88%, using the chart equation for determining IP=88-30 =58%, such that the soil is A-7-5(20).

General classification		Granular materials (35% or less of total sample passing 75 micron (No. 200 sieve)							Silty-clay materials (More than 35% of total sample passing 75 micron (No. 200 sieve)			
Group classification		A-1		A-3	A-2				A-4	A-5	A-6	A-7
		A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5, A-7-6
Sieve analysis, percent passing	2 mm (No 10)	50 max										
	0.425 mm (No. 40)	30 max	50 max	51 min								
	75 micron (No. 200)	15 max	25 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 (0.425 mm)	LL				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
	PI	6 max		NP	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min*
Usual significant constituent materials		Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
Group Index**		0		0	0		4 max		8 max	12 max	16 max	20 max

Classification procedure:
With required test data available, proceed from left to right on chart; correct group will be found by process of elimination. The first group from left into which the test data will fit is the correct classification.
*Plasticity Index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity Index of A-7-6 subgroup is greater than LL minus 30.
**See group index formula below. Group index should be shown in parentheses after group symbol as: A-2-6(3), A-4(5), A-7-5(17), etc.

GI = (F-35)[0.2+0.005(LL-40)] + 0.01(F-15) (PI-10)

Figure 4:2: AASHTO soil classification chart using LL AND PI

B. Using USCS Classification Systems

According to USCS soil classification the native soil was classifies as follows

For **LL >50**, the soil is classified as silts and clays.

Referring to figure 4.3

$$PI = 0.73(LL - 20) \quad \text{……………………} \quad (4.\ 5)$$

PI=0.73(88-20) =**49.64**, the native soil have PI =52 which plots above A line thus the soil is classified as **CH.**

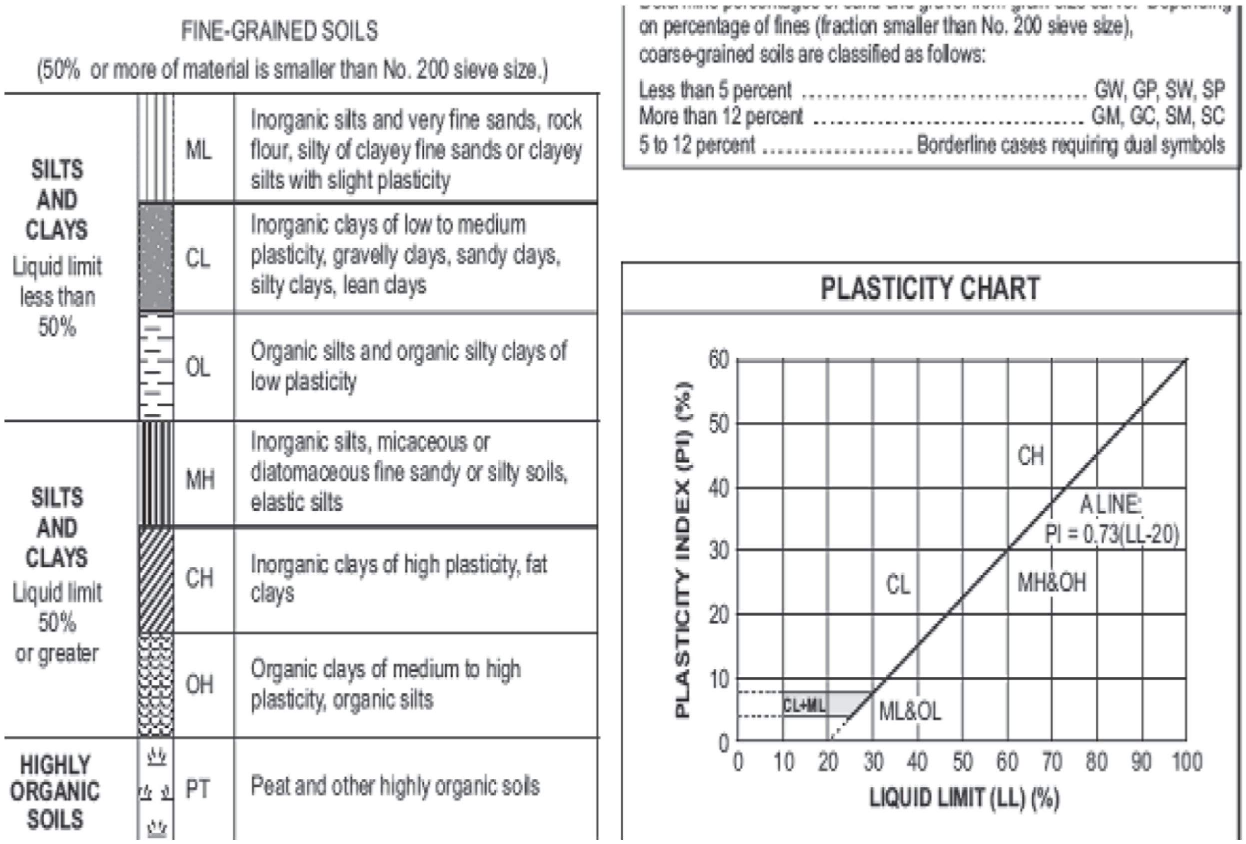


Figure 4:3: Unified soil classification

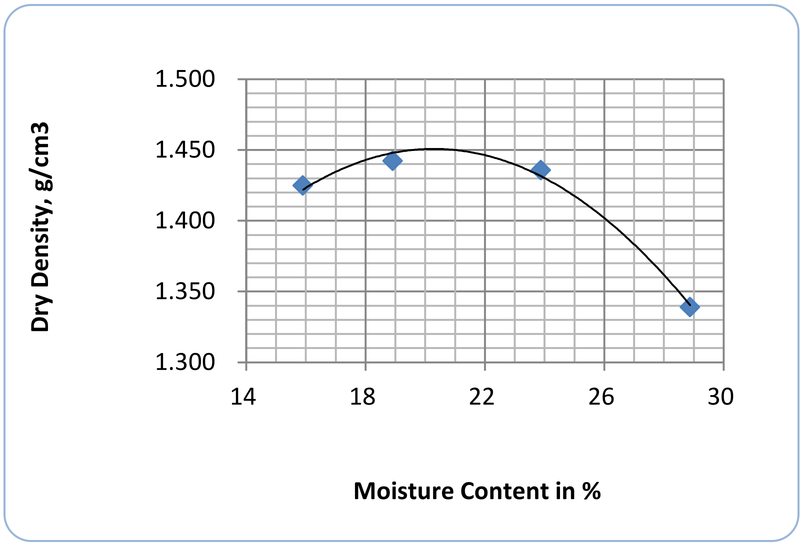
4.2.6 Moisture-Density Relations of Soil

The moisture density relation test was executed on native soil sample using AASHTO T-180 according to the procedures in section 3.4.5. Then calculate the moisture content of each trail by taking to oven from the compacted specimens and determine the wet density (g/cm^3) of the compacted native soil sample by dividing the wet mass by the volume of the mold used. Finally, determine the dry density (ρd) using the wet density and the obtained water content from compacted specimens, then draw dry density with moisture content to get the OMC.

$$\rho d = \frac{\rho}{(1+\omega)} \qquad \text{(4. 6)}$$

Where: ω = moisture content in percent divided by 100,

ρ = wet density in g/cm3



MDD=1.45gm/cm^{3}OMC=21%

Figure 4:4: Compaction of native soil sample

4.2.7 California Bearing Ratio (CBR) of Soil samples

This test method was executed to estimate the potential strength of subgrade soil samples which are used in pavement constructions. CBR estimates the strength of cohesive materials having maximum particle sizes less than 4.75mm. This test method provides for the determination of the CBR of materials at OMC from a specified compaction test and a specified dry unit mass.

4.2.8 CBR Swelling Test Measurements

Since clay soil has swelling behavior when became wet, the CBR swelling measurement of the clay subgrade was executed according to the procedures in section 3.4.8 and the results were calculated as:

$$\text{Change Percent swell} = \frac{(\text{Hf}-\text{Hi})}{\text{Ho}} * 100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots.. (4.\ 7)$$

Where: Hi = Initial dial gauge reading in mm

Hf = final dial gauge reading in mm

H0 = initial height of sample in mm, use 116.43

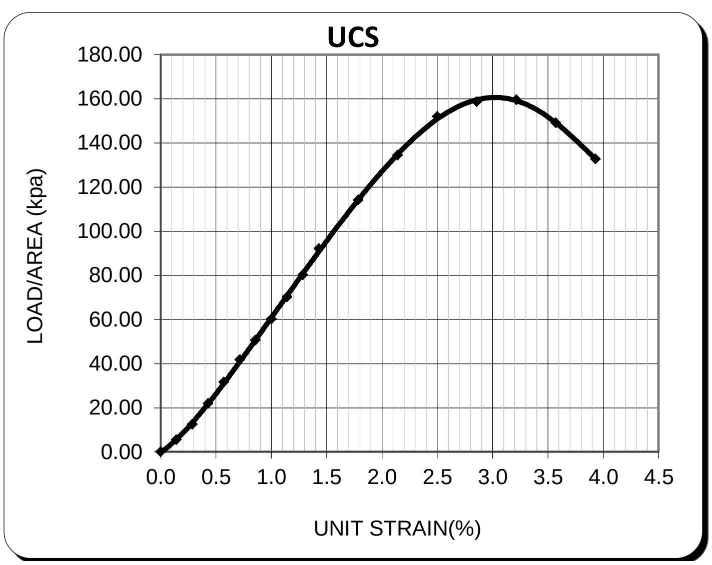
Table 4:2: CBR and CBR swell of native soil

No of blows	CBR	CBR Swell %	MDD	MDD 95%	CBR at 95%of MDD
10	1	18.73	1.45	1.38	1
30	1	17.07			
65	1	13.03			

4.2.9 Unconfined Compressive Strength of Soils

Unconfined compressive strength tests were conducted for the native sample soil and result is given in Figure 4.5 and the un-drained shear strengths were calculated as one half of the UCS values as shown in Equation 4.14. The un-drained shear strength (Su) of clays is commonly determined from unconfined compression test. The un-drained shear strength (Su) of a cohesive soil is equal to one-half the unconfined compression strength (qu) when the soil is under the f=0 condition (f=the angle of internal friction). The most critical condition in the soil usually occurs immediately after construction, which represent un-drained conditions, when the un-drained shear strength is basically equal to the cohesion (C). This is expressed as:

$$Su = C = qu/2 \text{ ……………………………………….. } (4.\ 8)$$



UCS=161kpa

SU=C=Qu/2=80.5kpa

Figure 4:5: UCS of Expansive soil

The comparisons above between ERA design manual and laboratory results of the soil shows that, the soil sample do not full fill the requirements as a sub-grade and are determined to be unsuitable for sub-grade in road construction. Therefore, the sub-grade soil should be treated with appropriate improving methods before use as road sub grade.

4.3 Cement and Wood ash Stabilized Expansive Soil Experimental Analysis and Discussion.

4.3.1 Effect of Cement and wood ash on Atterberg Limits

The Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI) of soil treated with cement and wood ash combination were determined and plotted against cement and wood ash content (Table 4.5 and Figure 4.6). PI of the samples decreased with increasing stabilizers percentages. The PI of native soil has decreased from 52% to (50%, 47% and 40%) when mixed with 2C5WA, 4C5WA and 6C5WA respectively. Similarly the PI has decreased to (48%, 42% and 38%), (47%, 44%and 41%) and (46%, 45% and 38%) when mixed with (2C10WA, 4C10WA and 6C10WA), (2C15WA, 4C15WA and 6C15WA) and (2C20WA, 4C20WA and 6C20WA) respectively. The maximum decrease in PI was observed from 52% to 38% when mixed with 6%cement and 20% wood ash.

These effects are due to the partial replacement of plastic soil particles with cement and wood ash which are non plastic materials and flocculation and agglomeration of clay particles caused by cation exchange may be the other cause.

Table 4:3: Index property of cement blended with wood ash stabilized soil

CEMENT%	WOOD ASH%	LL %	PL %	PI %
2%	5%	87	37	50
2%	10%	83	35	48
2%	15%	86	39	47
2%	20%	88	42	46
4%	5%	85	38	47
4%	10%	80	38	42
4%	15%	90	46	44
4%	20%	88	43	45
6%	5%	75	35	40
6%	10%	79	41	38
6%	15%	86	45	41
6%	20%	85	47	38

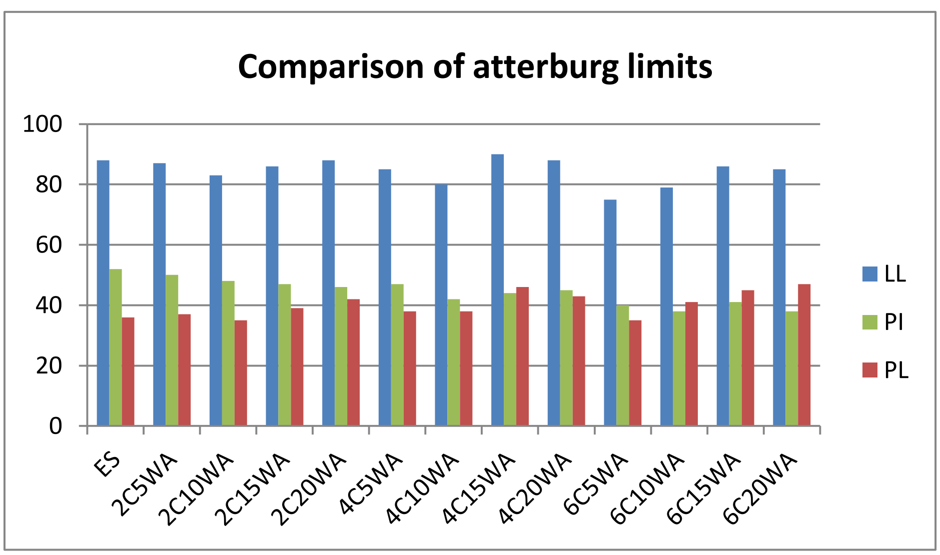


Figure 4:6: Effect of cement and wood ash on Atterburg limits

4.3.2 Moisture-Density Relations of Cement and wood ash stabilized native Soil

This section presents the compaction characteristic curves determined for sample soil used in the experimental work. Modified Proctor tests were performed on the raw subgrade soil and treated/stabilized native soil. The standards AASHTO T-180 compaction curves for the untreated soil, 0% and addition of stabilizers of 2%, 4% and 6% cement with wood ash 5%, 10%, 15% and 20% are shown below.

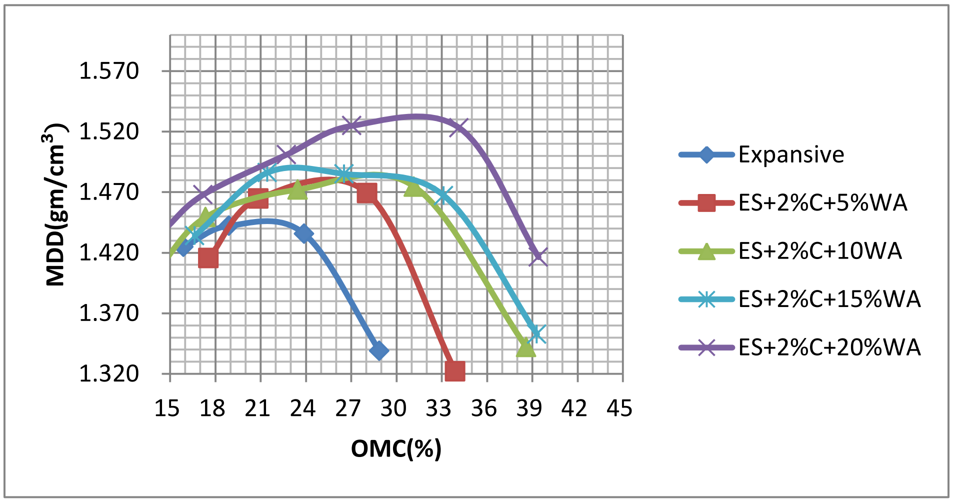


Figure 4:7: The effect of wood ash on compaction at 2% of cement

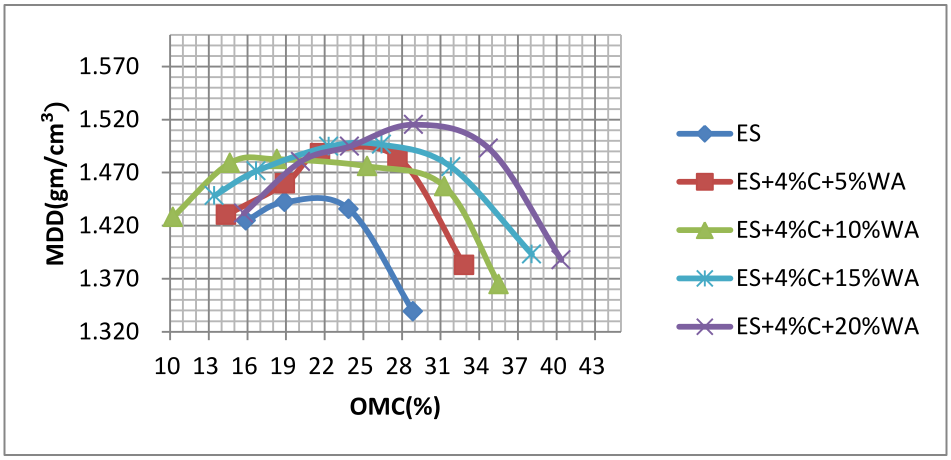


Figure 4:8: The effect of wood ash on compaction at 4% of cement

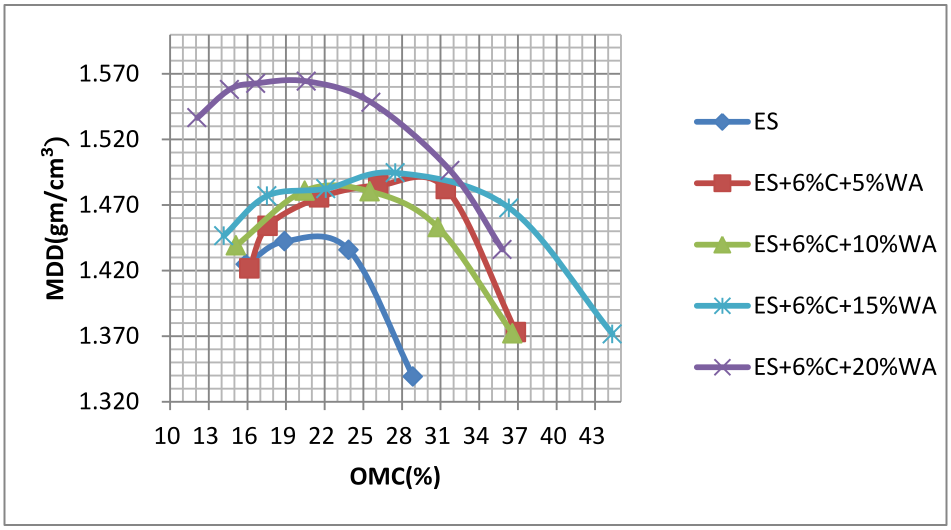


Figure 4:9: The effect of wood ash on compaction at 6% of cement

The combined effect of wood ash and cement on the maximum dry density of expansive soil was increasing. As the percentage of both additives increased maximum dry density increased, the maximum increase occurred at 6C20WA combination from 1.45g/cm^3 to 1.56 g/cm^3. The minimum reduction was observed at 2C5WA from1.45g/cm^3 to 1.49g/cm^3. Optimum moisture content of expansive soil for the combination of wood ash and cement additives had an increasing effect. The maximum increment occurred at 6C20WA from 21% to 30%. The minimum increment was observed at 2C5WA from 21% to 24%.

4.3.4 CBR of Cement and Wood ash with Expansive Soil

CBR test of cement and wood ash stabilized Expansive soils with an addition of2%, 4% and 6% cement with wood ash 5%,10%,15% and 20% executed according to the procedures in section 3.4.7. The CBR results were increased significantly as the stabilizers continent were increased as shown in table 4.6.

The CBR of native soil has increased from 1% to (2.6%, 3.1% and 5.7%) when mixed with 2C5WA, 4C5WA and 6C5WA respectively. Similarly it has increased to (4.2%, 4.5% and 4.7%), (5%, 5.9% and 13.8%) and (8.7%, 20.1% and 24.3%) when mixed with (2C10WA, 4C10WA and 6C10WA), (2C15WA, 4C15WA and 6C15WA) and (2C20WA, 4C20WA and 6C20WA) respectively. The maximum increase in CBR was observed from 1% to 24.3% when mixed with 6%cement and 20% wood ash.

Therefore, according the results the subgrade soil class changed in its strength from S1 which are consider as poor subgrade to S5 which is good enough as subgrade material according to ERA design manual classification [20].

Table 4:4 Sub-grade Strength Classes (ERA, 2013)

Class	CBR Range (%)
S1	<3
S2	3,4
S3	5,6,7
S4	8-14
S5	15-30
S6	>30

4.3.5 CBR swelling of cement and wood ash stabilized soil

Tests of CBR-swelling for cement and wood ash stabilized clay soils have been carried out using similar procedures in section 3.4.8 in order to determine its swelling capacity. Therefore, the swelling potential of soils mixed with cement-wood ash treated soils is as shown in table blow.

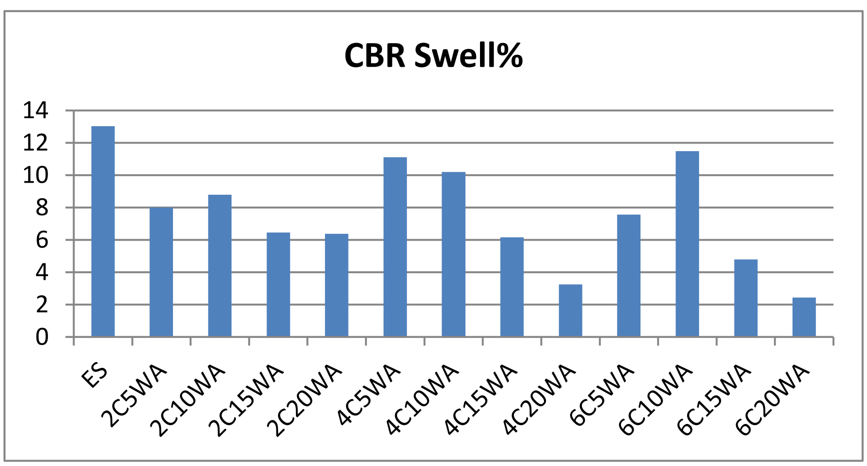


Figure 4:10: The effect of cement and wood ash on CBR swells

Table 4:5: Result of CBR and CBR swell of cement and wood ash stabilized Expansive soil

CEMENT%	WOOD ASH%	CBR %	CBR SWELL %
2%	5%	2.6	8.01
2%	10%	4.2	8.79
2%	15%	5.0	6.46
2%	20%	8.7	6.38
4%	5%	3.1	11.11
4%	10%	4.5	10.19
4%	15%	5.9	6.17
4%	20%	20.1	3.25
6%	5%	5.7	7.56
6%	10%	4.5	11.49
6%	15%	13.8	4.8
6%	20%	24.3	2.3

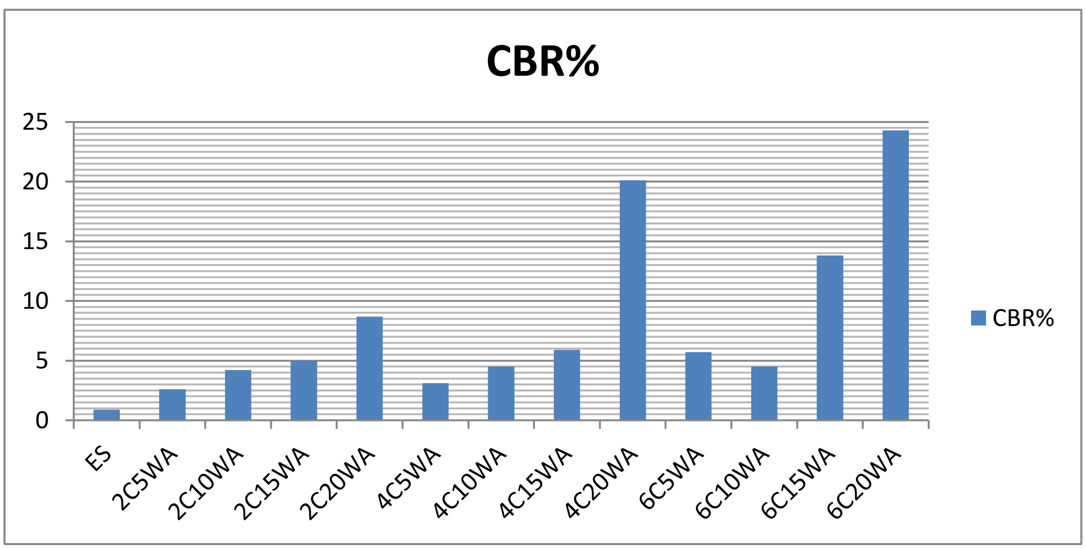


Figure 4:11: Effect of cement and wood ash on CBR values

The CBR swell value of native soil decreased from 13.03% to 2.43% when stabilized with 6% of cement with 20% of wood ash. These reduced swell characteristics are generally attributed to decreased affinity for water of the calcium saturated clay and the formation of a cementious matrix that resists volumetric expansion [40].

4.3.6 Unconfined Compressive Strength of Cement and wood ash Stabilized Soils

This section presents the UCS values determined for sample soils used in the experimental work. UCS test was performed on treated/stabilized native soil specimens which were mixed at OMC and MDD from results in section (4.3.2) following procedures listed in ASTM D-2166-98 as briefly discussed in section (3.4.5). The stress strain curves obtained from treated /stabilized soils with cement and wood ash with different percentages are shown in Figure 4.12. The cement-wood ash-Expansive soil mixture was cured for seven days. The UCS value for the expansive soil has increased from 161kpa to 450kpa when treated with 6% of cement and 20% of wood ash as shown in Figure 4.12, strength of the expansive clay soil increased with higher cement-wood ash content combinations. Typical stress-strain curves of soil specimens with different cement-wood ash content as presented in Figure 4.12 clearly indicated that the stress-strain curves shift towards the left hand side as the strain at failure decreased with the addition of stabilizers and the failure mode of the treated material exhibited a brittle type of failure mode, this is due to hardening of the cured clay particles with time and this agree with works of [12].

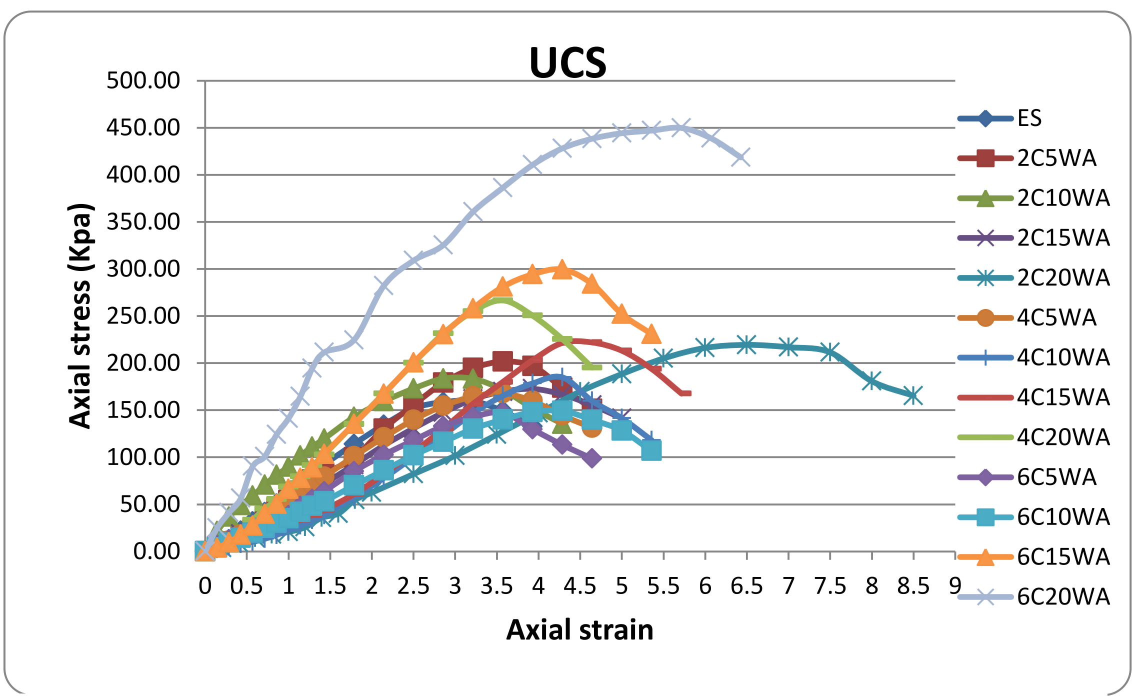


Figure 4:12: The effect of cement and wood ash on UCS

CHAPTER FIVE

5. Conclusions and Recommendations

5.1 Conclusions

From the laboratory observed experimental results of the study the following conclusions are drawn:

- The engineering property of the studied native Expansive soil revealed that it is not suitable to use as a sub-grade material unless its undesirable properties are remedied.
- The index properties of the native expansive soil were significantly influenced by the addition of cement-wood ash additives. There was a marked decrease in liquid limit and an increase in plastic limit with the addition of both cement-wood ash mixtures, consequently reducing the plasticity index of native soil from 52% to 38% when stabilized with 6% of cement and 20% of wood ash.
- The addition of cement-wood ash causes increase in maximum dry density and optimum moisture content.
- The addition of cement-wood ash causes an increase in the CBR and decreases CBR swelling values. In the CBR test it is found that the addition of Cement-wood ash in the percentage 6% cement and 20% wood ash and 4 %, cement and 20% wood ash to the expansive soil improves the CBR values of native expansive subgrade soils from 1% to 24.3% and 20.1 % respectively while CBR swelling value of native soil decreases from 13.08% to 2.43% and 3.25% respectively.
- The addition of cement-Wood ash additive improves the UCS values of native expansive soil from 161kpa to 450kpa and 300kpa when treated with 6% cement and 20% wood ash and 6%, cement and 15% wood ash and cured for seven days respectively.
- The addition of cement-wood ash has significantly improved strength and eliminated swelling potential of the native expansive soil. Based on the investigation the addition of 6% cement and 20% wood ash additive can effectively stabilize this particular soil type by eliminating the swell and significantly increasing CBR and UCS values. Hence 6% cement and 20% wood ash can be taken as optimum cement-Wood ash stabilizer content.

- Since wood ash is regarded as a waste material and it is cheap, using it as a stabilizing material for expansive soils will reduce the cost of carrying out engineering constructions on expansive soils and also reduce the environmental problems associated with indiscriminate disposal of wood ash.

5.2 Recommendations and Future Works

Based on the results of the study the following recommendations are made:

- Laboratory tests were done for the expansive soil, cement-wood ash mixture such as compaction, CBR, UCS and Atterberg limits has a positive trend but field tests such as DCP will have effect on the obtained result so further study shall be conducted.
- The composition of wood ash is influenced by the type of firewood used and the process of burning conditions at the fire place, wood ash from Bahrzaf firewood was used in this research and it was not compared with other types of firewood. Therefore further investigations should be made on the type of most effective firewood or wood ash type.
- Effect of curing period on soils treated with cement and wood ash combination shall be studied.
- Finally mechanism of soil stabilization with cement and wood ash combination is relatively new concept and literatures are scanty in the area, therefore further investigations are required.

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Appendix

GRAIN SIZE ANALYSIS (WET SIEVE HYDROMETER ANALYSIS)

The Hydrometer analysis was performed by applying meniscus correction (i.e. +1) to the actual hydrometer reading. The equivalent particle diameter was calculated by using the following formula:

$$D = K\sqrt{L/t} \text{ ………………………… ……….. (1)}$$

Where: t is in minutes and D is given in mm.

K = constant depending on the temperature of the suspension and the specific gravity of the soil particles.

L= distance from the surface of the suspension to the level at which the density of the suspension is being measured, cm

T= interval of time from beginning of sedimentation to the taking of the reading, min,

Temperature correction (CT,)

CT was executed since the actual temperature of the test is not 20^{0}C; therefore the correction is approximated as

$$CT = -4.85 + 0.25T \text{…………………………….…………….. (2)}$$

Where T: temperature of test in ^{0}C (for T between 15 and 28^{0}C)

The corrected hydrometer reading is determined by

$$RC = R(Actual) - zero\ correction + CT \text{ …………………………. (3)}$$

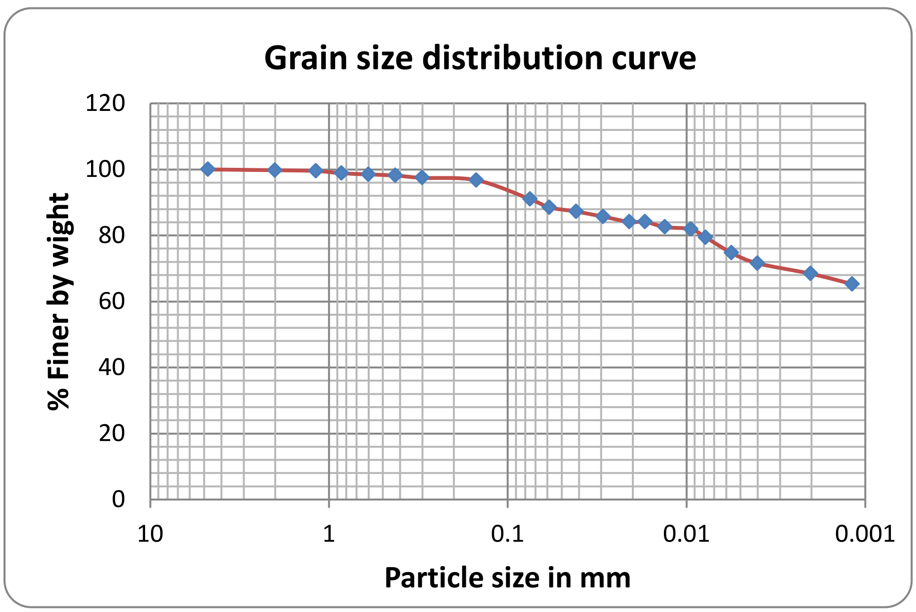
Then using the corrected Hydrometer, the percent finer will be calculated as

$$P = \text{Rc} * \text{aWs} * 100 \text{ ……………………………………… (4)}$$

GRAIN SIZE ANALYSIS (WET SIEVE HYDROMETER ANALYSIS)

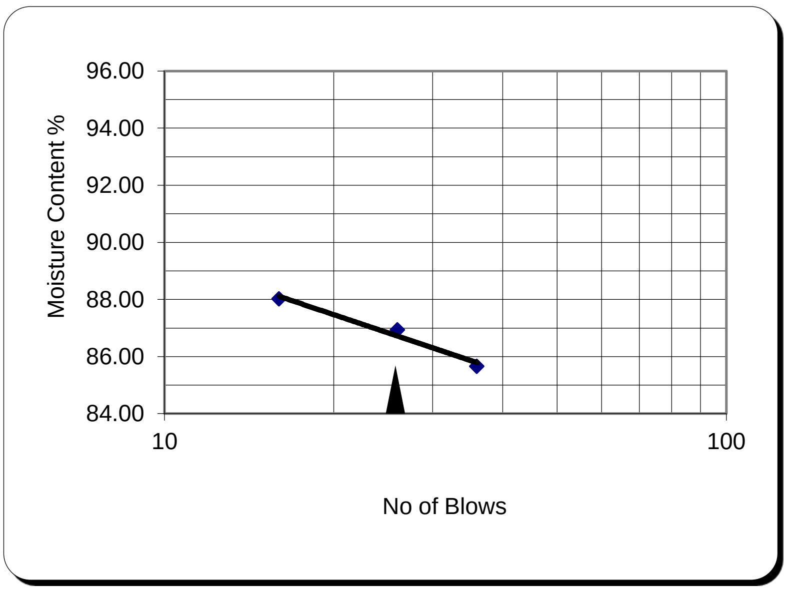
CUMULATIVE % RETAINED	%PASS
0	100
0.2	100
0.5	100
1.2	99
1.5	98
1.9	98
2.5	97
3.2	97
3.6	91

Diameter of soil particle **D**(mm)	Soil in suspension, **P(%of soil finer)**
0.058	88.50
0.041	87.24
0.029	85.68
0.021	84.11
0.017	84.11
0.013	82.54
0.009	81.91
0.008	79.40
0.006	74.69
0.004	71.55
0.002	68.42
0.001	65.28



Expansive soil+2% cement+5% Wood Ash

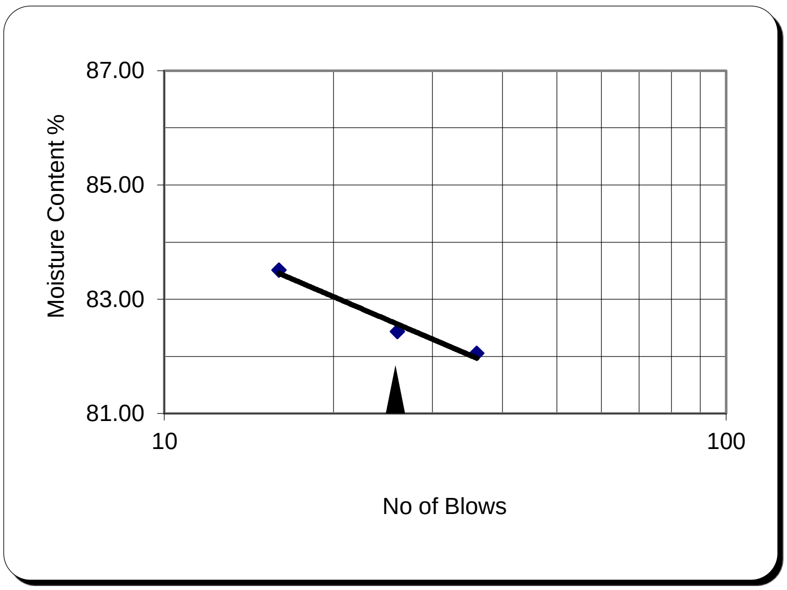
	LIQUID LIMIT			PLASTIC LIMIT		
	1	2	3	1	2	
No.of blows	36	26	16			
Tare	I3	A75	H4	P5	A1	
Wt. tare + wet soil	34.71	37.03	36.57	21.04	21.36	
Wt tare + dry soil	25.99	27.65	26.28	19.49	19.83	
Wt. water	8.72	9.38	10.29	1.55	1.53	
Wt. of tare	15.81	16.86	14.59	15.30	15.59	
Wt. of dry soil	10.18	10.79	11.69	4.19	4.24	
No.of blows N	36	26	16			
Moisture content	85.66	86.93	88.02	36.99	36.08	
				AVERAGE PLASTIC LIMIT		37



LIQUID LIMIT (LL%)
87
PLASTIC LIMIT (PL %)
37
PLASTICITY INDEX (P.I)
50

Expansive soil+2% cement+10% Wood Ash

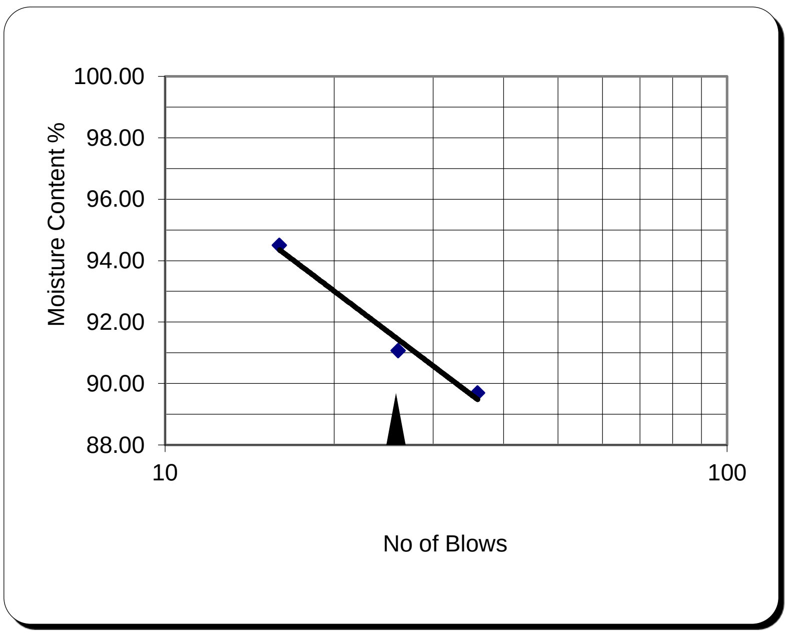
	LIQUID LIMIT			PLASTIC LIMIT		
	1	2	3	1	2	
No. of blows	36	26	16			
Tare nº	06	05	A1	X12	Y4	
Wt. tare + wet soil	34.44	34.63	33.06	20.83	21.45	
Wt tare + dry soil	25.48	25.48	25.11	19.26	20.04	
Wt. water	8.96	9.15	7.95	1.57	1.41	
Wt. of tare	14.56	14.38	15.59	14.84	16.02	
Wt. of dry soil	10.92	11.10	9.52	4.42	4.02	
No.of blows	36	26	16			
Moisture content	82.05	82.43	83.51	35.52	35.07	
				AVERAGE PLASTIC LIMIT		35



LIQUID LIMIT (LL%)
83
PLASTIC LIMIT (PL %)
35
PLASTICITY INDEX (P.I)
48

Expansive soil+2% cement+15% Wood Ash

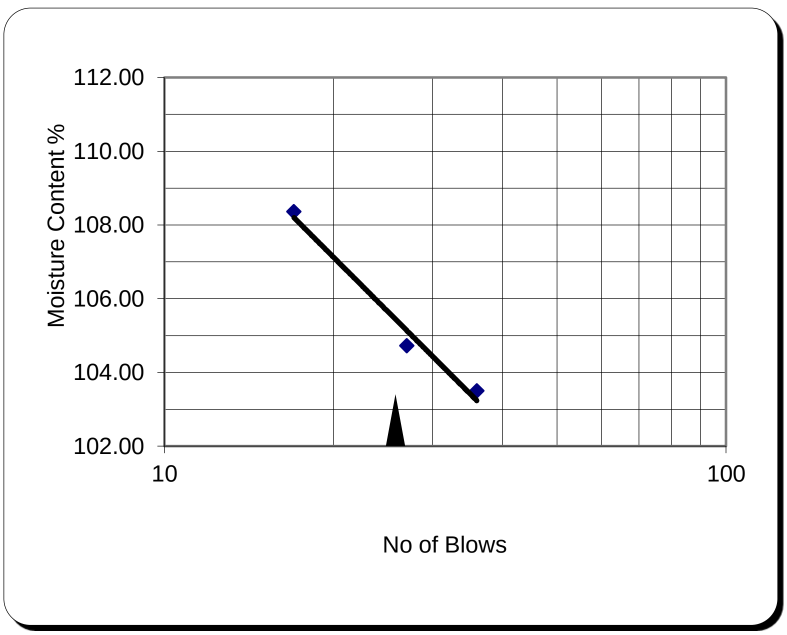
	LIQUID LIMIT			PLASTIC LIMIT		
	1	2	3	1	2	
No.of blows	36	26	16			
Tare	Y2	D5	C6	O5	N2	
Wt. tare + wet soil	31.00	33.22	36.32	21.17	22.31	
Wt tare + dry soil	22.73	24.86	27.22	19.30	20.23	
Wt. water	8.27	8.36	9.10	1.87	2.08	
Wt. of tare	13.51	15.68	17.59	14.38	14.82	
Wt. of dry soil	9.22	9.18	9.63	4.92	5.41	
No.of blows	36	26	16			
Moisture content	84.70	86.07	90.50	39.01	39.45	
				AVERAGE PLASTIC LIMIT		39



LIQUID LIMIT (LL%)
86
PLASTIC LIMIT (PL %)
39
PLASTICITY INDEX (P.I)
47

Expansive soil+2% cement+20% Wood Ash

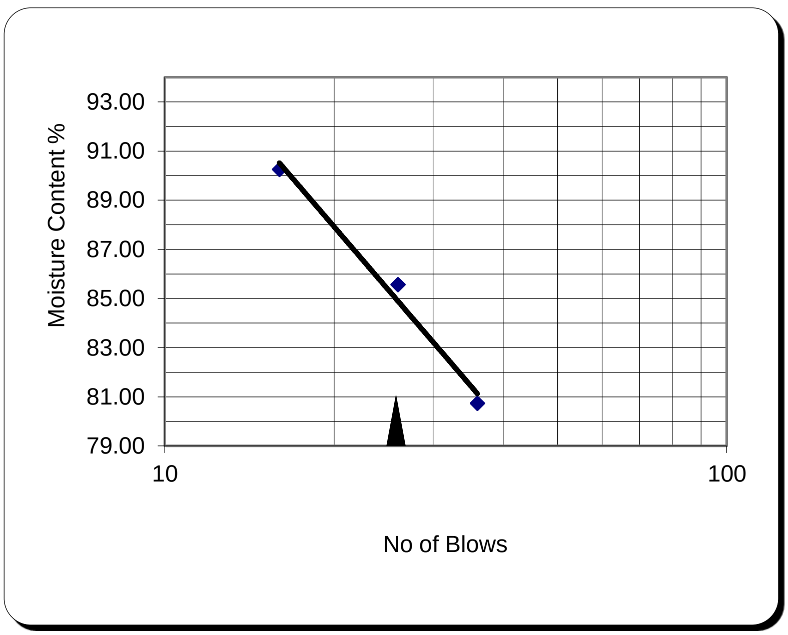
	LIQUID LIMIT			PLASTIC LIMIT		
	1	2	3	1	2	
No.of blows	36	27	17			
Tare	Y4	E2	A2	C1	F2	
Wt. tare + wet soil	35.23	34.71	32.92	20.23	20.40	
Wt tare + dry soil	25.46	26.28	23.46	18.67	18.67	
Wt. water	9.77	8.43	9.46	1.56	1.73	
Wt. of tare	16.02	18.23	14.73	15.17	14.73	
Wt. of dry soil	9.44	8.05	8.73	3.50	3.94	
No.of blows	36	27	17			
Moisture content	87.50	88.72	92.36	42.57	41.91	
				AVERAGE PLASTIC LIMIT		42



LIQUID LIMIT (LL%)
88
PLASTIC LIMIT (PL %)
42
PLASTICITY INDEX (P.I)
46

Expansive soil+4% cement+5% Wood Ash

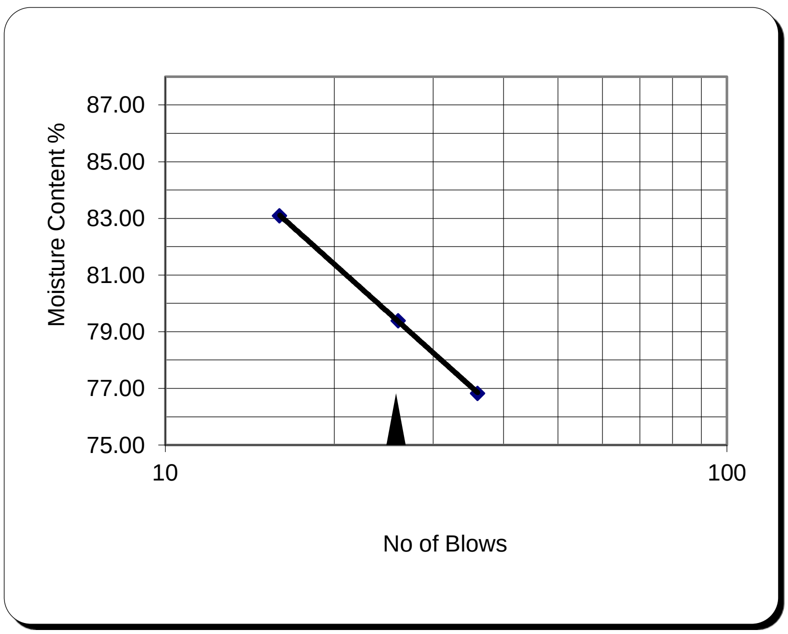
	LIQUID LIMIT			PLASTIC LIMIT		
	1	2	3	1	2	
No.of blows	36	26	16			
Tare	C1	Q4	G2	O3	O6	
Wt. tare + wet soil	34.02	35.54	30.95	21.29	20.19	
Wt tare + dry soil	25.60	26.89	23.83	19.68	18.65	
Wt. water	8.42	8.65	7.12	1.61	1.54	
Wt. of tare	15.17	16.78	15.94	15.51	14.56	
Wt. of dry soil	10.43	10.11	7.89	4.17	4.09	
No.of blows	36	26	16			
Moisture content	80.73	85.56	90.24	38.61	37.65	
				AVERAGE PLASTIC LIMIT		38



LIQUID LIMIT (LL%)
85
PLASTIC LIMIT (PL %)
38
PLASTICITY INDEX (P.I)
47

Expansive soil+4% cement+10% Wood Ash

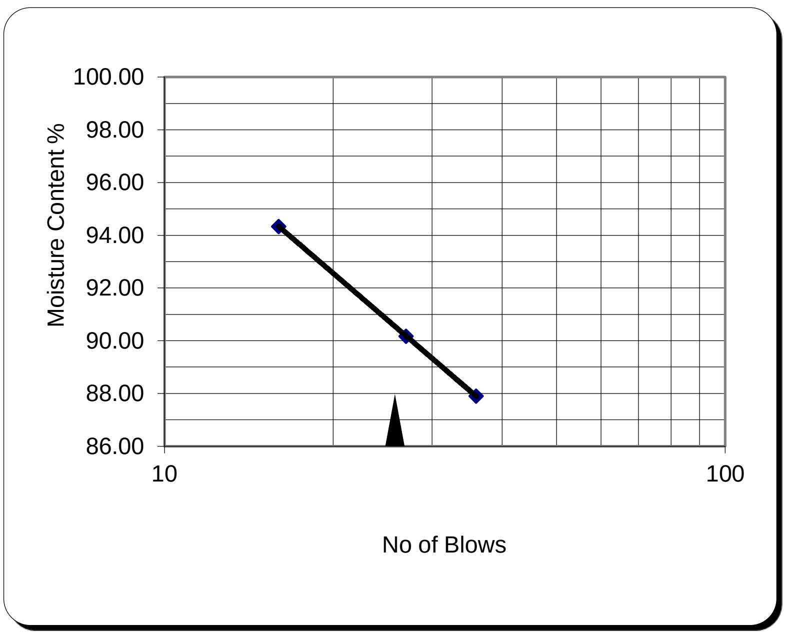
	LIQUID LIMIT			PLASTIC LIMIT		
	1	2	3	1	2	
No.of blows	36	26	16			
Tare	Z6	Q6	J3	T1	K3	
Wt. tare + wet soil	34.42	34.27	36.19	23.94	21.62	
Wt tare + dry soil	26.33	25.64	27.05	22.26	20.22	
Wt. water	8.09	8.63	9.14	1.68	1.40	
Wt. of tare	15.80	14.77	16.05	17.88	16.58	
Wt. of dry soil	10.53	10.87	11.00	4.38	3.64	
No.of blows	36	26	16			
Moisture content %	76.83	79.39	83.09	38.36	38.46	
				AVERAGE PLASTIC LIMIT		38



LIQUID LIMIT (LL%)
80
PLASTIC LIMIT (PL %)
38
PLASTICITY INDEX (P.I)
42

Expansive soil+4% cement+15% Wood Ash

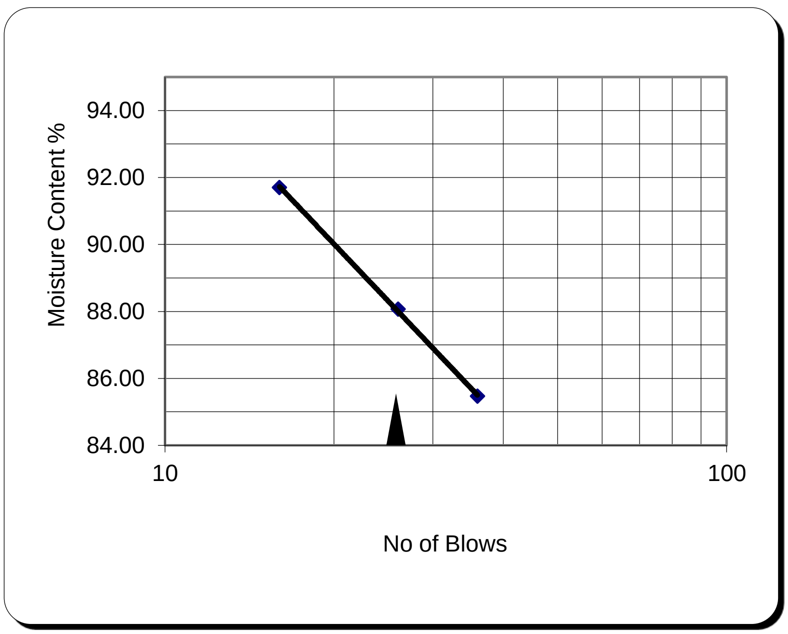
	LIQUID LIMIT			PLASTIC LIMIT		
	1	2	3	1	2	
No.of blows	36	27	16			
Tare	C3	G3	A1	L1	U3	
Wt. tare + wet soil	39.77	36.04	39.68	21.99	22.23	
Wt tare + dry soil	29.61	26.14	29.03	20.00	20.29	
Wt. water	10.16	9.90	10.65	1.99	1.94	
Wt. of tare	18.05	15.16	17.74	15.59	16.11	
Wt. of dry soil	11.56	10.98	11.29	4.41	4.18	
No.of blows	36	27	16			
Moisture content	87.89	90.16	94.33	45.12	46.41	
				AVERAGE PLASTIC LIMIT		46



LIQUID LIMIT (LL%)
90
PLASTIC LIMIT (PL %)
46
PLASTICITY INDEX (P.I)
44

Expansive soil+4% cement+20% Wood Ash

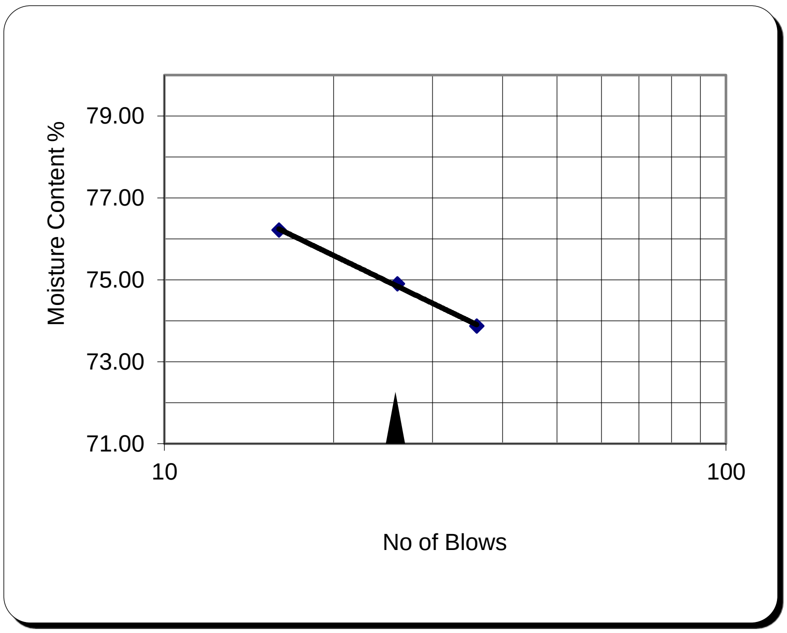
	LIQUID LIMIT			PLASTIC LIMIT		
	1	2	3	1	2	
No.of blows	36	26	16			
Tare	O2	I6	Q3	M2	E3	
Wt. tare + wet soil	35.75	34.23	34.97	24.07	23.24	
Wt tare + dry soil	27.40	26.04	25.25	21.89	20.96	
Wt. water	8.35	8.19	9.72	2.18	2.28	
Wt. of tare	17.63	16.74	14.65	16.72	15.66	
Wt. of dry soil	9.77	9.30	10.60	5.17	5.30	
No.of blows	36	26	16			
Moisture content	85.47	88.06	91.70	42.17	43.02	
				AVERAGE PLASTIC LIMIT		43



LIQUID LIMIT (LL%)
88
PLASTIC LIMIT (PL %)
43
PLASTICITY INDEX (P.I)
45

Expansive soil+6% cement+5% Wood Ash

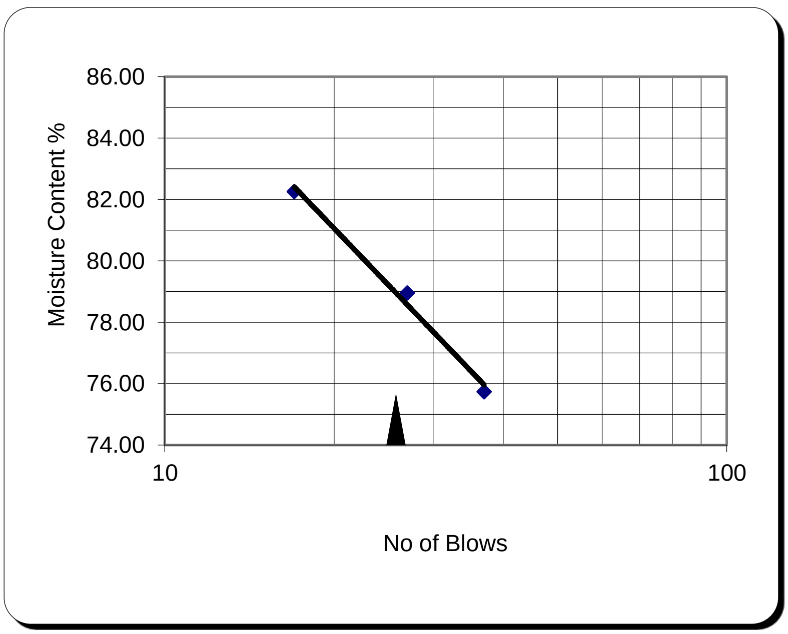
	LIQUID LIMIT			PLASTIC LIMIT		
	1	2	3	1	2	
No.of blows	36	26	16			
Tare	C1	O3	H4	L6	A2	
Wt. tare + wet soil	39.39	34.26	34.89	20.63	20.73	
Wt tare + dry soil	29.10	26.23	26.11	19.15	19.17	
Wt. water	10.29	8.03	8.78	1.48	1.56	
Wt. of tare	15.17	15.51	14.59	14.83	14.73	
Wt. of dry soil	13.93	10.72	11.52	4.32	4.44	
No.of blows	36	26	16			
Moisture content	73.87	74.91	76.22	34.26	35.14	
				AVERAGE PLASTIC LIMIT		35



LIQUID LIMIT (LL%)
75
PLASTIC LIMIT (PL %)
35
PLASTICITY INDEX (P.I)
40

Expansive soil+6% cement+10% Wood Ash

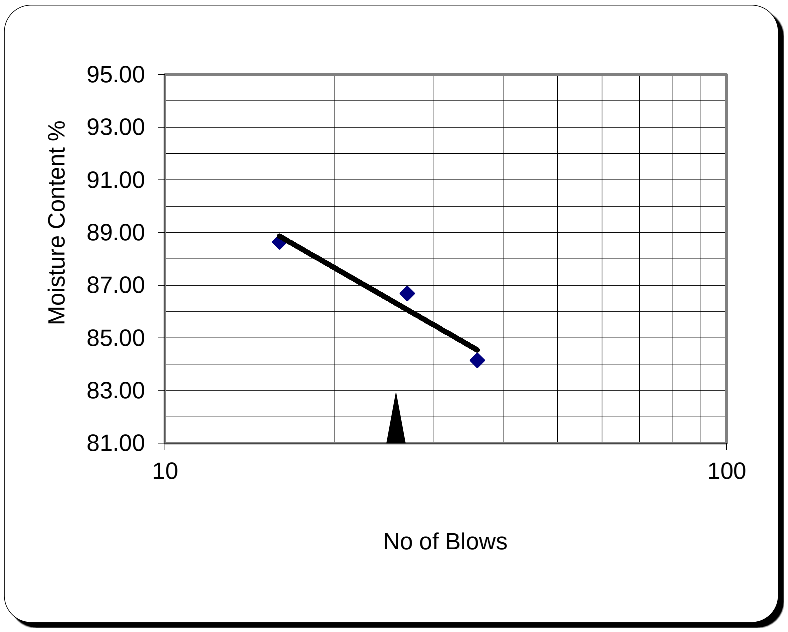
	LIQUID LIMIT			PLASTIC LIMIT		
	1	2	3	1	2	
No.of blows	37	27	17			
Tare	Y6	P5	N5	F2	X6	
Wt. tare + wet soil	34.74	35.11	34.09	19.90	21.99	
Wt tare + dry soil	26.06	26.37	25.42	18.41	20.40	
Wt. water	8.68	8.74	8.67	1.49	1.59	
Wt. of tare	14.60	15.30	14.88	14.73	16.48	
Wt. of dry soil	11.46	11.07	10.54	3.68	3.92	
No.of blows	37	27	17			
Moisture content	75.74	78.95	82.26	40.49	40.56	
				AVERAGE PLASTIC LIMIT		41



LIQUID LIMIT (LL%)
79
PLASTIC LIMIT (PL %)
41
PLASTICITY INDEX (P.I)
38

Expansive soil+6% cement+15% Wood Ash

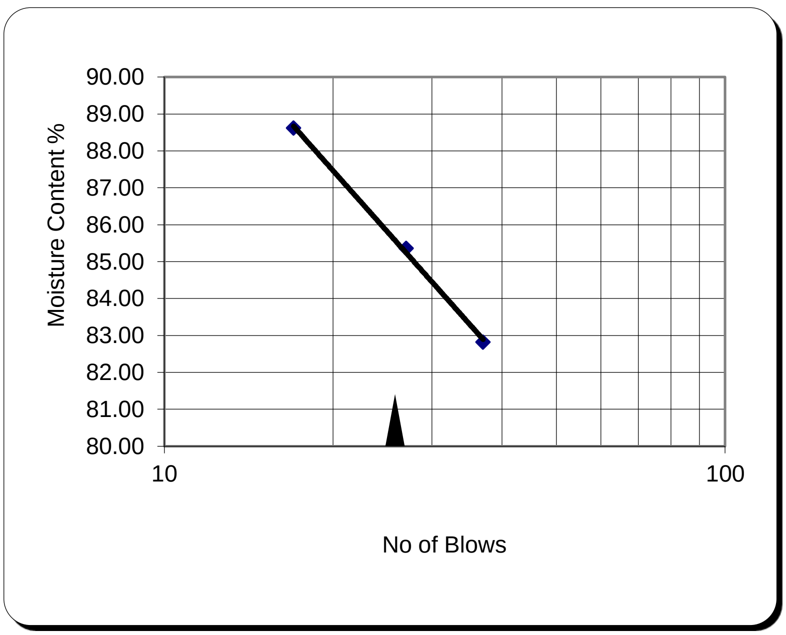
	LIQUID LIMIT			PLASTIC LIMIT		
	1	2	3	1	2	
No.of blows	36	27	16			
Tare	T1	T3	A	I3	E55	
Wt. tare + wet soil	37.37	35.81	36.43	22.53	22.89	
Wt tare + dry soil	27.71	26.57	26.75	20.44	20.80	
Wt. water	9.66	9.24	9.68	2.09	2.09	
Wt. of tare	16.23	15.91	15.83	15.81	16.21	
Wt. of dry soil	11.48	10.66	10.92	4.63	4.59	
No.of blows	36	27	16			
Moisture content	84.15	86.68	88.64	45.14	45.53	
				AVERAGE PLASTIC LIMIT		45



LIQUID LIMIT (LL%)
86
PLASTIC LIMIT (PL %)
45
PLASTICITY INDEX (P.I)
41

Expansive soil+6% cement+20% Wood Ash

	LIQUID LIMIT			PLASTIC LIMIT		
	1	2	3	1	2	
No.of blows	37	27	17			
Tare	G2	A9	H6	O4	E40	
Wt. tare + wet soil	36.80	35.98	39.24	21.66	22.23	
Wt tare + dry soil	27.35	26.42	27.87	19.92	20.49	
Wt. water	9.45	9.56	11.37	1.74	1.74	
Wt. of tare	15.94	15.22	15.04	16.18	16.90	
Wt. of dry soil	11.41	11.20	12.83	3.74	3.59	
No.of blows	37	27	17			
Moisture content	82.82	85.36	88.62	46.52	48.47	
				AVERAGE PLASTIC LIMIT		47



LIQUID LIMIT (LL%)
85
PLASTIC LIMIT (PL %)
47
PLASTICITY INDEX (P.I)
38

Compaction

Expansive soil

A	Mold	No.	1	2	3	4
B	Wt. of Mold + Wet Soil	Grams	4800	4860	4920	4870
C	Wt. of Mold	Grams	3241	3241	3241	3241
D	Wt. Wet Soil	Grams	1559	1619	1679	1629
E	Volume of Mold	cu.cm.	944	944	944	944
F	Wet Density	gr/cu.cm.	1.65	1.72	1.78	1.73

G	Container	No.	A1	C	A2	D1
H	Wt. Cont + Wet soil	Grams	249.92	260.93	285.69	243.74
I	Wt. Cont + Dry soil	Grams	223.23	229.34	240.83	200.86
J	Weight of Water	grams	26.69	31.59	44.86	42.88
K	Weight of Container	grams	55.46	62.34	53.02	52.37
L	Weight of Dry Soil	grams	167.77	167.00	187.81	148.49

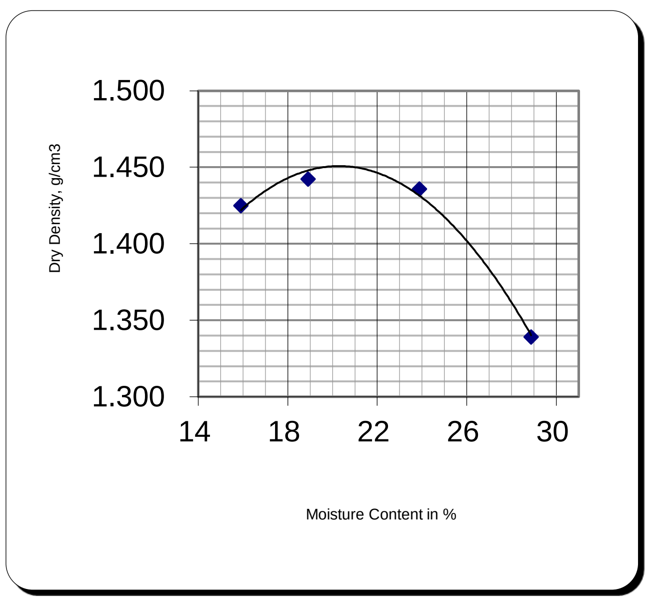
M	Moisture Content	%	15.9	18.9	23.9	28.9
N	Dry Density	gr/cu.cm.	1.425	1.442	1.436	1.339

Maximum Dry Density :

MDD = 1.450 gm/cc

Optimum Moisture Content :

OMC = 21.0 %



Expansive soil+2% cement+5% Wood Ash

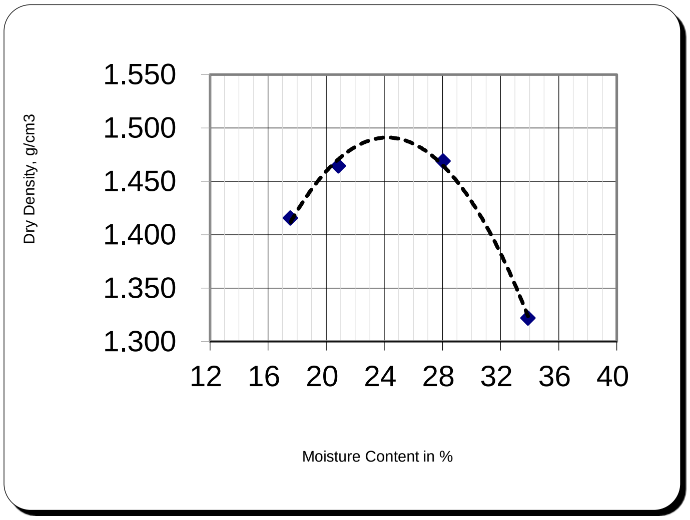
A	Mold	No.	1	2	3	4
B	Wt. of Mold + Wet Soil	grams	4745	4845	4950	4845
C	Wt. of Mold	grams	3174	3174	3174	3174
D	Wt. Wet Soil	grams	1571	1671	1776	1671
E	Volume of Mold	cu.cm.	944	944	944	944
F	Wet Density	gr/cu.cm.	1.66	1.77	1.88	1.77
G	Container	No.	A5	A16	A8	L6
H	Wt. Cont + Wet soil	grams	331.94	206.19	273.22	263.76
I	Wt. Cont + Dry soil	grams	293.86	179.18	230.49	216.06
J	Weight of Water	grams	38.08	27.01	42.73	47.70
K	Weight of Container	grams	76.99	49.72	78.23	75.33
L	Weight of Dry Soil	grams	216.87	129.46	152.26	140.73
M	Moisture Content	%	17.6	20.9	28.1	33.9
N	Dry Density	gr/cu.cm.	1.416	1.465	1.469	1.322

Maximum Dry Density:

MDD = 1.490 Gm/cc

Optimum Moisture Content :

OMC = 25.0 %



Expansive soil+2% cement+10% Wood Ash

A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	grams	4500	4600	4710	4820	4750
C	Wt. of Mold	grams	2994	2994	2994	2994	2994
D	Wt. Wet Soil	grams	1506	1606	1716	1826	1756
E	Volume of Mold	cu.cm.	944	944	944	944	944
F	Wet Density	gr/cu.cm.	1.60	1.70	1.82	1.93	1.86

G	Container	No.	T	A5	W	A1	X
H	Wt. Cont + Wet soil	grams	319.31	220.98	310.63	376.48	268.65
I	Wt. Cont + Dry soil	grams	290.17	195.62	258.67	297.29	204.48
J	Weight of Water	grams	29.14	25.36	51.96	79.19	64.17
K	Weight of Container	grams	80.17	49.62	37.36	43.28	38.24
L	Weight of Dry Soil	grams	210.00	146.00	221.31	254.01	166.24

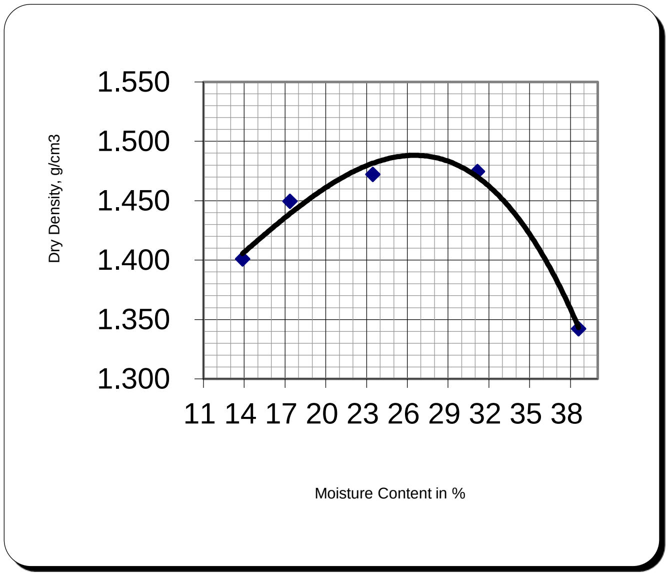
M	Moisture Content	%	13.9	17.4	23.5	31.2	38.6
N	Dry Density	gr/cu.cm.	1.401	1.449	1.472	1.475	1.342

Maximum Dry Density :

MDD = 1.490 gm/cc

Optimum Moisture Content :

OMC = 27.0 %



Expansive soil+2% cement+15% Wood Ash

A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	Grams	4820	4945	5015	5085	5020
C	Wt. of Mold	Grams	3241	3241	3241	3241	3241
D	Wt. Wet Soil	Grams	1579	1704	1774	1844	1779
E	Volume of Mold	cu.cm.	944	944	944	944	944
F	Wet Density	gr/cu.cm.	1.67	1.81	1.88	1.95	1.88

G	Container	No.	B6	A3	A13	A1	Y5
H	Wt. Cont + Wet soil	Grams	353.77	313.10	260.81	244.97	226.60
I	Wt. Cont + Dry soil	Grams	314.29	266.26	214.02	197.20	177.45
J	Weight of Water	Grams	39.48	46.84	46.79	47.77	49.15
K	Weight of Container	Grams	77.00	47.98	37.72	53.04	52.43
L	Weight of Dry Soil	Grams	237.29	218.28	176.30	144.16	125.02

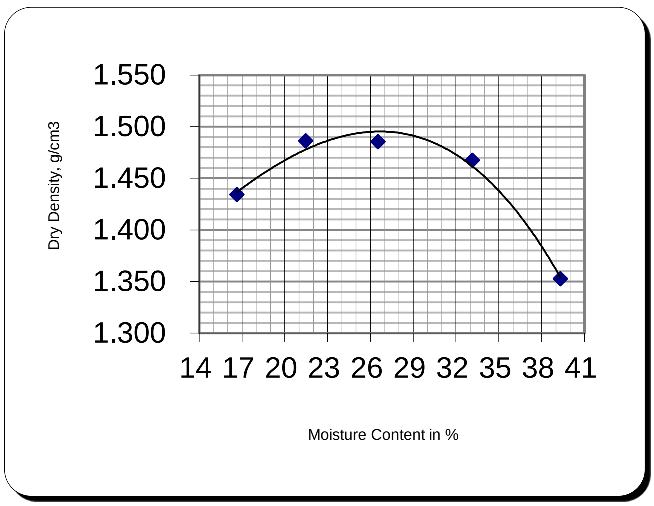
M	Moisture Content	%	16.6	21.5	26.5	33.1	39.3
N	Dry Density	gr/cu.cm.	1.434	1.486	1.485	1.467	1.353

Maximum Dry Density :

MDD = 1.495 gm/cc

Optimum Moisture Content :

OMC = 28.0 %



Expansive soil+2% cement+20% Wood Ash

A	Mold	No.	1	2	3	4	5	6
B	Wt. of Mold + Wet Soil	grams	4800	4865	4980	5070	5170	5105
C	Wt. of Mold	grams	3241	3241	3241	3241	3241	3241
D	Wt. Wet Soil	grams	1559	1624	1739	1829	1929	1864
E	Volume of Mold	cu.cm.	944	944	944	944	944	944
F	Wet Density	gr/cu.cm.	1.65	1.72	1.84	1.94	2.04	1.97

G	Container	No.	M	M2	A	A10	M	N
H	Wt. Cont + Wet soil	grams	251.99	267.45	270.65	335.52	311.89	237.34
I	Wt. Cont + Dry soil	grams	223.21	236.91	229.31	273.17	243.21	179.21
J	Weight of Water	grams	28.78	30.54	41.34	62.35	68.68	58.13
K	Weight of Container	grams	27.28	59.34	47.41	42.78	42.17	31.73
L	Weight of Dry Soil	grams	195.93	177.57	181.90	230.39	201.04	147.48

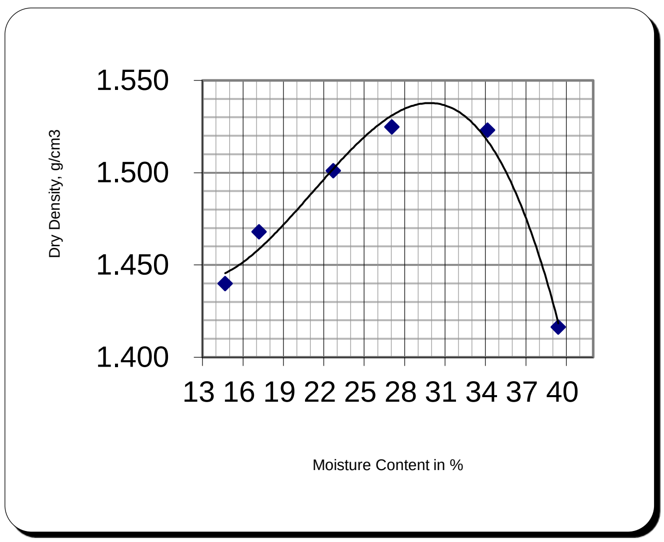
M	Moisture Content	%	14.7	17.2	22.7	27.1	34.2	39.4
N	Dry Density	gr/cu.cm.	1.440	1.468	1.501	1.525	1.523	1.416

Maximum Dry Density :

MDD= 1.539 gm/cc

Optimum Moisture Content :

OMC= 30.0 %



Expansive soil+4% cement+5% Wood Ash

A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	grams	4785	4880	4950	5030	4975
C	Wt. of Mold	grams	3241	3241	3241	3241	3241
D	Wt. Wet Soil	grams	1544	1639	1709	1789	1734
E	Volume of Mold	cu.cm.	944	944	944	944	944
F	Wet Density	gr/cu.cm.	1.64	1.74	1.81	1.90	1.84

G	Container	No.	G	A1	X	L	A2
H	Wt. Cont + Wet soil	grams	285.28	266.58	269.83	296.96	236.81
I	Wt. Cont + Dry soil	grams	256.94	232.80	232.40	242.74	190.66
J	Weight of Water	grams	28.34	33.78	37.43	54.22	46.15
K	Weight of Container	grams	59.68	54.46	59.58	46.87	50.11
L	Weight of Dry Soil	grams	197.26	178.34	172.82	195.87	140.55

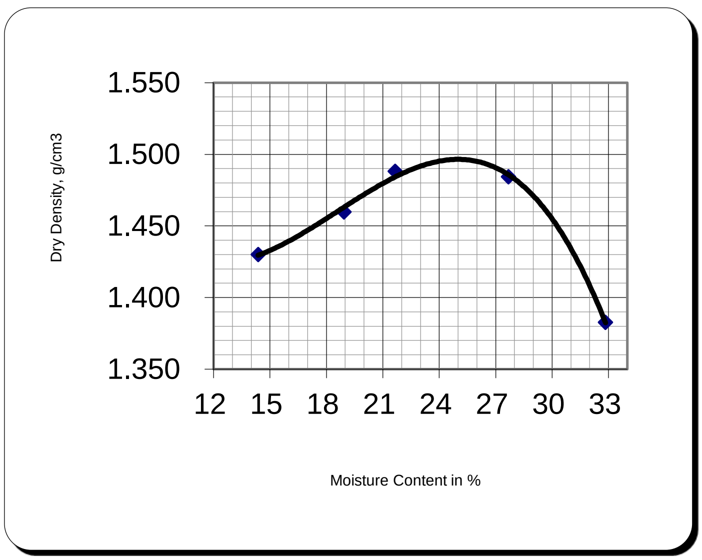
M	Moisture Content	%	14.4	18.9	21.7	27.7	32.8
N	Dry Density	gr/cu.cm.	1.430	1.460	1.488	1.484	1.383

Maximum Dry Density :

MDD = 1.499 gm/cc

Optimum Moisture Content :

OMC = 26.0 %



Expansive soil+4% cement+10% Wood Ash

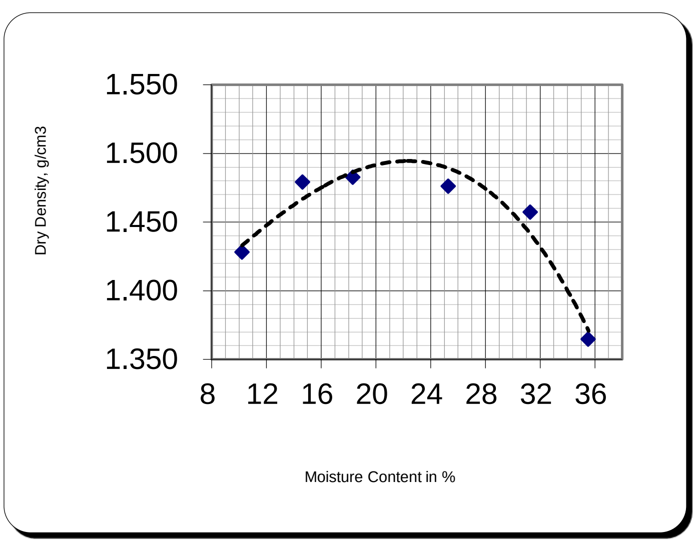
A	Mold	No.	1	2	3	4	5	6
B	Wt. of Mold + Wet Soil	grams	4480	4595	4650	4740	4800	4740
C	Wt. of Mold	grams	2994	2994	2994	2994	2994	2994
D	Wt. Wet Soil	grams	1486	1601	1656	1746	1806	1746
E	Volume of Mold	cu.cm.	944	944	944	944	944	944
F	Wet Density	gr/cu.cm.	1.57	1.70	1.75	1.85	1.91	1.85
G	Container	No.	N	G4	A3	B	GE	E9
H	Wt. Cont + Wet soil	grams	225.00	204.00	200.00	225.00	185.00	225.00
I	Wt. Cont + Dry soil	grams	208.25	184.24	173.24	186.41	146.43	180.32
J	Weight of Water	grams	16.75	19.76	26.76	38.59	38.57	44.68
K	Weight of Container	grams	44.43	49.45	27.16	33.94	23.18	54.59
L	Weight of Dry Soil	grams	163.82	134.79	146.08	152.47	123.25	125.73
M	Moisture Content	%	10.2	14.7	18.3	25.3	31.3	35.5
N	Dry Density	gr/cu.cm.	1.428	1.479	1.483	1.476	1.457	1.365

Maximum Dry Density :

MDD = 1.495 gm/cc

Optimum Moisture Content :

OMC = 22.0 %



Expansive soil+4% cement+15% Wood Ash

A	Mold	No.	1	2	3	4	5	6
B	Wt. of Mold + Wet Soil	grams	4545	4615	4720	4780	4830	4810
C	Wt. of Mold	grams	2994	2994	2994	2994	2994	2994
D	Wt. Wet Soil	grams	1551	1621	1726	1786	1836	1816
E	Volume of Mold	cu.cm.	944	944	944	944	944	944
F	Wet Density	gr/cu.cm.	1.64	1.72	1.83	1.89	1.94	1.92

G	Container	No.	GE	A1	J	J	N	A
H	Wt. Cont + Wet soil	grams	219.14	263.00	250.03	314.41	258.66	266.41
I	Wt. Cont + Dry soil	grams	195.98	230.74	209.90	258.43	205.03	201.01
J	Weight of Water	grams	23.16	32.26	40.13	55.98	53.63	65.40
K	Weight of Container	grams	23.74	37.54	30.20	46.63	36.50	29.33
L	Weight of Dry Soil	grams	172.24	193.20	179.70	211.80	168.53	171.68

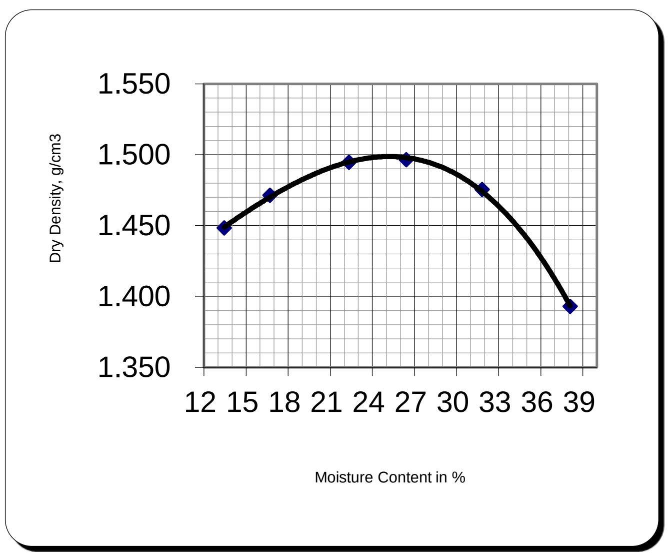
M	Moisture Content	%	13.4	16.7	22.3	26.4	31.8	38.1
N	Dry Density	gr/cu.cm.	1.448	1.471	1.495	1.496	1.475	1.393

Maximum Dry Density :

MDD= 1.500 gm/cc

Optimum Moisture Content :

OMC= 27.0 %



Expansive soil+4% cement+20% Wood Ash

A	Mold	No.	1	2	3	4	5	6
B	Wt. of Mold + Wet Soil	grams	4805	4920	4990	5085	5140	5080
C	Wt. of Mold	grams	3241	3241	3241	3241	3241	3241
D	Wt. Wet Soil	grams	1564	1679	1749	1844	1899	1839
E	Volume of Mold	cu.cm.	944	944	944	944	944	944
F	Wet Density	gr/cu.cm.	1.66	1.78	1.85	1.95	2.01	1.95

G	Container	No.	C7	N5	G	P	G4	J
H	Wt. Cont + Wet soil	grams	261.87	254.23	312.18	232.92	271.58	328.00
I	Wt. Cont + Dry soil	grams	230.68	216.99	260.43	193.63	217.35	255.31
J	Weight of Water	grams	31.19	37.24	51.75	39.29	54.23	72.69
K	Weight of Container	grams	32.38	32.14	44.39	57.77	61.26	75.31
L	Weight of Dry Soil	grams	198.30	184.85	216.04	135.86	156.09	180.00

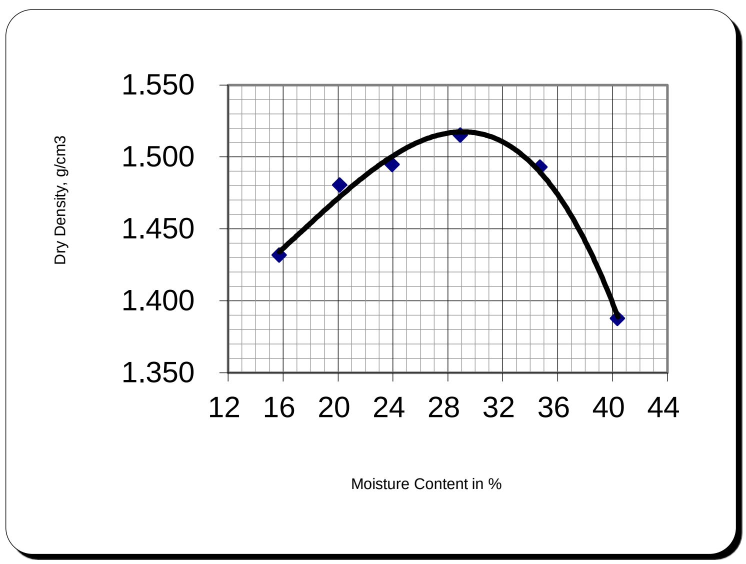
M	Moisture Content	%	15.7	20.1	24.0	28.9	34.7	40.4
N	Dry Density	gr/cu.cm.	1.432	1.480	1.495	1.515	1.493	1.388

Maximum Dry Density :

MDD= 1.520 gm/cc

Optimum Moisture Content :

OMC= 30.0 %



Expansive soil+6% cement+5% Wood Ash

A	Mold	No.	1	2	3	4	5	6
B	Wt. of Mold + Wet Soil	Grams	4800	4855	4935	5010	5080	5015
C	Wt. of Mold	Grams	3241	3241	3241	3241	3241	3241
D	Wt. Wet Soil	Grams	1559	1614	1694	1769	1839	1774
E	Volume of Mold	cu.cm.	944	944	944	944	944	944
F	Wet Density	gr/cu.cm.	1.65	1.71	1.79	1.87	1.95	1.88

G	Container	No.	T	J	G9	G	A	G
H	Wt. Cont + Wet soil	Grams	285.99	288.02	287.77	273.97	240.99	233.89
I	Wt. Cont + Dry soil	Grams	258.25	250.59	246.81	230.21	191.29	178.77
J	Weight of Water	Grams	27.74	37.43	40.96	43.76	49.70	55.12
K	Weight of Container	Grams	57.90	32.11	57.19	63.15	33.21	29.29
L	Weight of Dry Soil	Grams	200.35	218.48	189.62	167.06	158.08	149.48

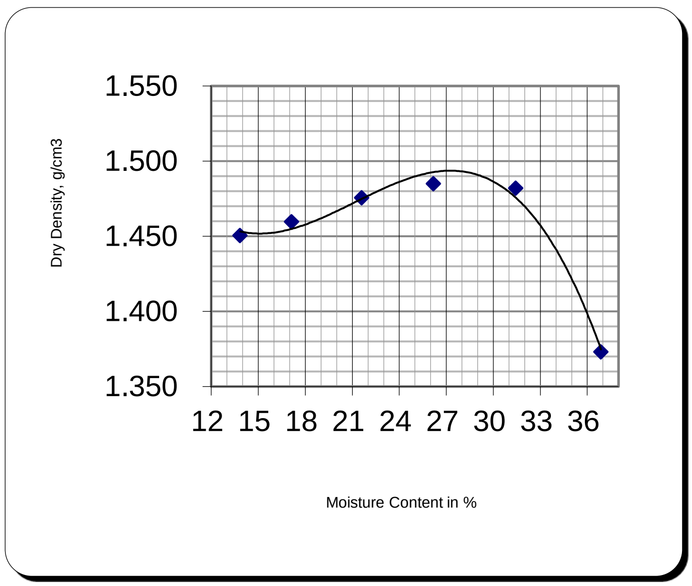
M	Moisture Content	%	13.8	17.1	21.6	26.2	31.4	36.9
N	Dry Density	gr/cu.cm.	1.451	1.460	1.476	1.485	1.482	1.373

Maximum Dry Density :

MDD= 1.493 gm/cc

Optimum Moisture Content :

OMC= 28.0 %



Expansive soil+6% cement+10% Wood Ash

A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	Grams	4805	4925	4995	5035	5010
C	Wt. of Mold	Grams	3241	3241	3241	3241	3241
D	Wt. Wet Soil	Grams	1564	1684	1754	1794	1769
E	Volume of Mold	cu.cm.	944	944	944	944	944
F	Wet Density	gr/cu.cm.	1.66	1.78	1.86	1.90	1.87

G	Container	No.	A2	E	M	A1	S
H	Wt. Cont + Wet soil	Grams	251.01	334.59	280.03	257.74	248.72
I	Wt. Cont + Dry soil	Grams	225.15	285.90	229.60	204.87	191.86
J	Weight of Water	Grams	25.86	48.69	50.43	52.87	56.86
K	Weight of Container	Grams	54.26	47.93	31.94	33.20	36.46
L	Weight of Dry Soil	Grams	170.89	237.97	197.66	171.67	155.40

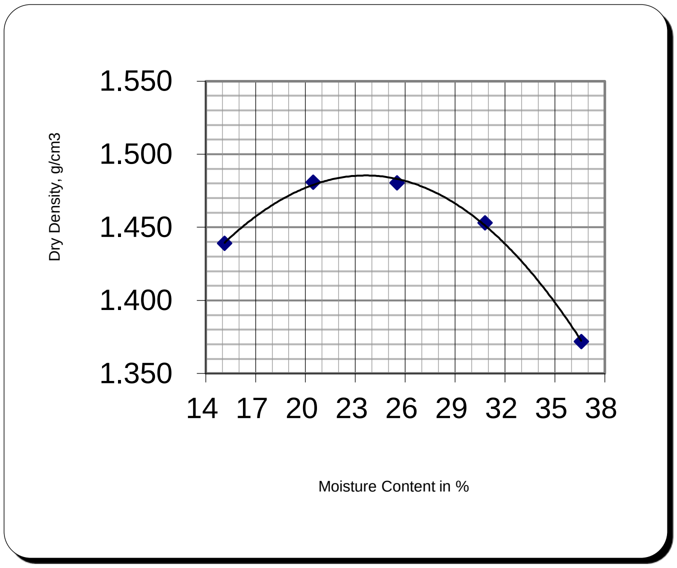
M	Moisture Content	%	15.1	20.5	25.5	30.8	36.6
N	Dry Density	gr/cu.cm.	1.439	1.481	1.480	1.453	1.372

Maximum Dry Density :

MDD= 1.486 gm/cc

Optimum Moisture Content :

OMC= 24.3 %



Expansive soil+6% cement+15% Wood Ash

A	Mold	No.	1	2	3	4	5	6
B	Wt. of Mold + Wet Soil	Grams	4800	4880	4950	5040	5130	5110
C	Wt. of Mold	Grams	3241	3241	3241	3241	3241	3241
D	Wt. Wet Soil	Grams	1559	1639	1709	1799	1889	1869
E	Volume of Mold	cu.cm.	944	944	944	944	944	944
F	Wet Density	gr/cu.cm.	1.65	1.74	1.81	1.91	2.00	1.98

G	Container	No.	EX	V	A	A1	Y3	W
H	Wt. Cont + Wet soil	Grams	290.19	318.22	325.63	310.46	238.19	240.08
I	Wt. Cont + Dry soil	Grams	260.59	277.45	281.13	251.85	183.31	182.94
J	Weight of Water	Grams	29.60	40.77	44.50	58.61	54.88	57.14
K	Weight of Container	Grams	51.75	45.11	79.96	38.78	32.27	54.10
L	Weight of Dry Soil	Grams	208.84	232.34	201.17	213.07	151.04	128.84

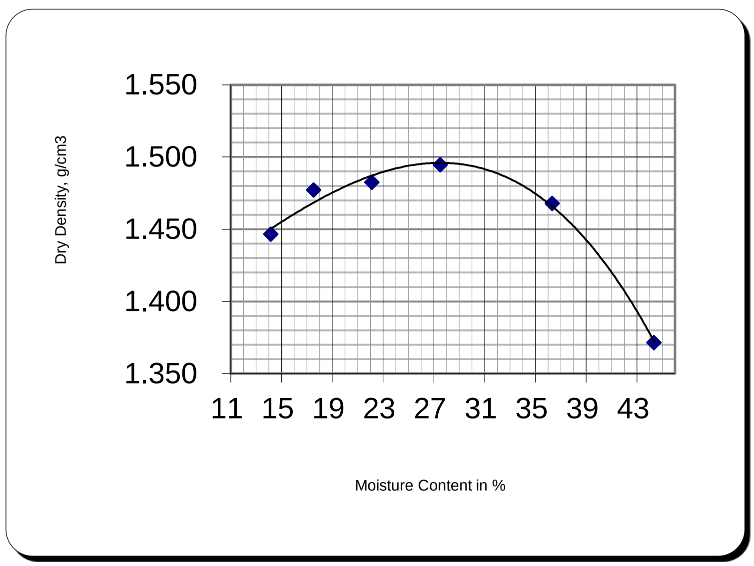
M	Moisture Content	%	14.2	17.5	22.1	27.5	36.3	44.3
N	Dry Density	gr/cu.cm.	1.446	1.477	1.482	1.495	1.468	1.372

Maximum Dry Density :

MDD= 1.500 gm/cc

Optimum Moisture Content :

OMC= 29.0 %



Expansive soil+6% cement+20% Wood Ash

A	Mold	No.	1	2	3	4	5	6
B	Wt. of Mold + Wet Soil	grams	4800	4860	4895	4955	5010	5035
C	Wt. of Mold	grams	3174	3174	3174	3174	3174	3174
D	Wt. Wet Soil	grams	1626	1686	1721	1781	1836	1861
E	Volume of Mold	cu.cm.	944	944	944	944	944	944
F	Wet Density	gr/cu.cm.	1.72	1.79	1.82	1.89	1.94	1.97

G	Container	No.	A2	A	E2	E	X5	F1
H	Wt. Cont + Wet soil	grams	278.11	316.38	267.51	304.20	268.39	214.39
I	Wt. Cont + Dry soil	grams	252.76	279.82	236.51	261.91	224.49	174.35
J	Weight of Water	grams	25.35	36.56	31.00	42.29	43.90	40.04
K	Weight of Container	grams	43.39	29.85	50.42	56.60	53.13	48.23
L	Weight of Dry Soil	grams	209.37	249.97	186.09	205.31	171.36	126.12

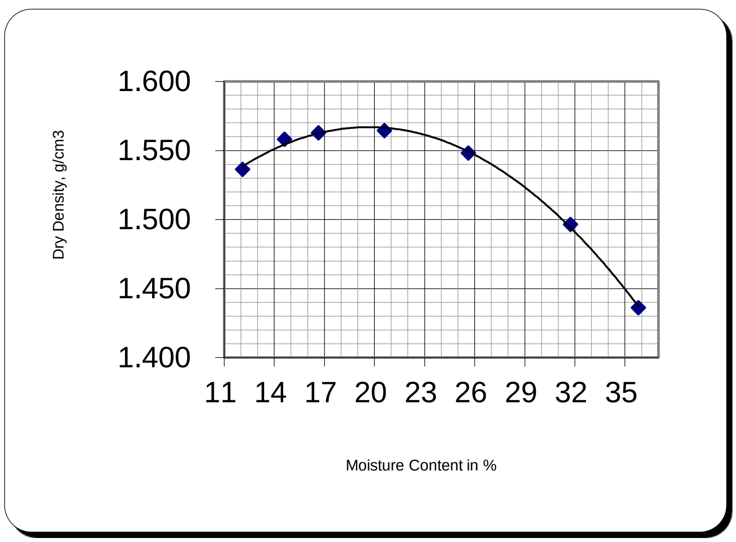
M	Moisture Content	%	12.1	14.6	16.7	20.6	25.6	31.7
N	Dry Density	gr/cu.cm.	1.536	1.558	1.563	1.564	1.548	1.496

Maximum Dry Density :

MDD= 1.568 gm/cc

Optimum Moisture Content :

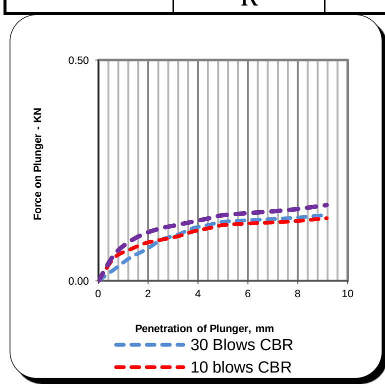
OMC= 20.0 %

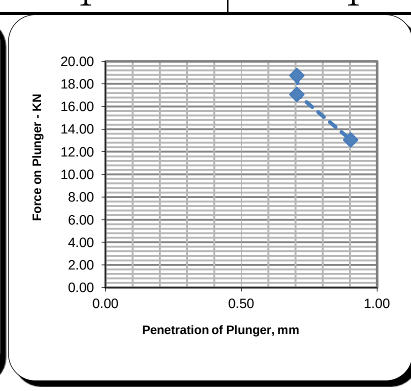


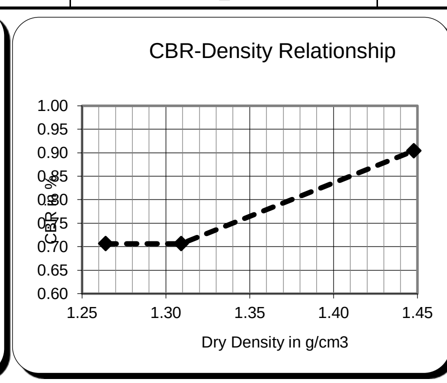
Soaked CBR

Expansive soil

Ring Factor:	0.00373							
10 blows 5 Layers			30 blows, 5 Layers			65 blows, 5 Layers		
PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE	
PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD
0	0	0.00	0	0	0.00	0	0	0.00
0.64	14	0.05	0.64	7	0.03	0.64	16	0.06
1.27	19	0.07	1.27	14	0.05	1.27	24	0.09
1.91	23	0.09	1.91	19	0.07	1.91	29	0.11
2.54	25	0.09	2.54	27	0.10	2.54	32	0.12
3.18	27	0.10	3.18	28	0.10	3.18	34	0.13
3.81	30	0.11	3.81	32	0.12	3.81	36	0.13
4.45	32	0.12	4.45	34	0.13	4.45	38	0.14
5.08	34	0.13	5.08	36	0.13	5.08	40	0.15
7.62	36	0.13	7.62	38	0.14	7.62	43	0.16
9.16	38	0.14	9.16	40	0.15	9.16	46	0.17
SWELL	Dial	swell %	SWELL	Dial	swell%	SWELL	Dial	swll %
	2.01			2.01			2.01	
	17.19	18.73		21.90	17.07		23.83	13.03
CBR %	PEN.	2.5mm	5.08mm	2.5mm	5.08mm	2.5mm	5.08mm	
	TOP							
	BOTTOM	1	1	1	1	1	1	
	RECO.CBR	1		1		1		

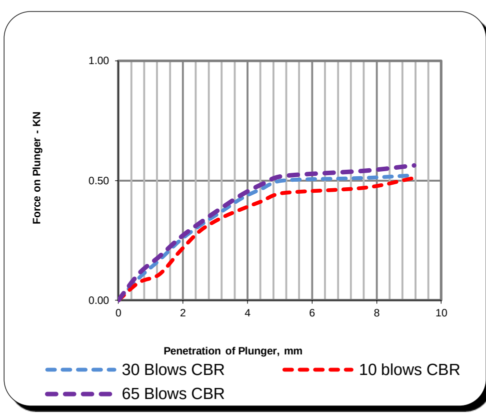


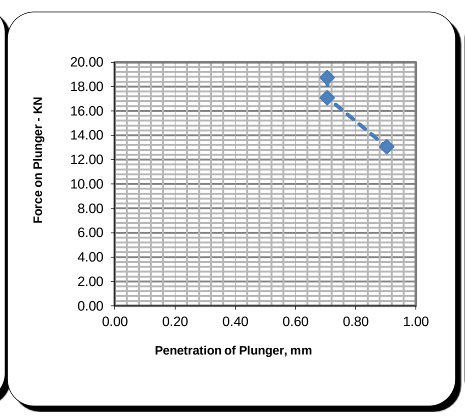


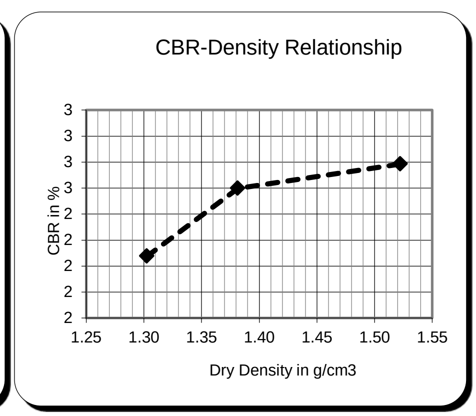


Expansive soil+2% cement+5% Wood Ash

Ring Factor: 0.00373								
10 blows 5 Layers			30 blows, 5 Layers			65 blows, 5 Layers		
PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE	
PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD
0	0	0.00	0	0	0.00	0	0	0.00
0.64	20	0.07	0.64	25	0.09	0.64	30	0.11
1.27	29	0.11	1.27	45	0.17	1.27	49	0.18
1.91	55	0.21	1.91	67	0.25	1.91	70	0.26
2.54	78	0.29	2.54	84	0.31	2.54	87	0.32
3.18	92	0.34	3.18	99	0.37	3.18	103	0.38
3.81	102	0.38	3.81	114	0.43	3.81	118	0.44
4.45	111	0.41	4.45	125	0.47	4.45	130	0.48
5.08	120	0.45	5.08	134	0.50	5.08	139	0.52
7.62	126	0.47	7.62	137	0.51	7.62	145	0.54
9.16	137	0.51	9.16	140	0.52	9.16	151	0.56
SWELL	Dial	swell%	SWELL	Dial	swell%	SWELL	Dial	swll%
	2.01			2.01			2.01	
	20.96	16.27		15.27	11.38		11.34	8.01
CBR %	PEN.	2.5mm	5.08mm	2.5mm	5.08mm	2.5mm	5.08mm	
	TOP							
	BOTTOM	2.2	2.2	2.4	2.5	2.5	2.6	
	RECO.CBR	2.2		2.5		2.6		

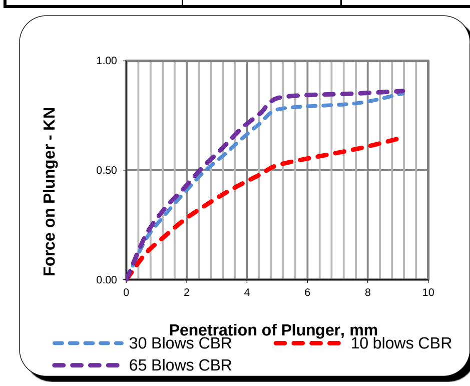


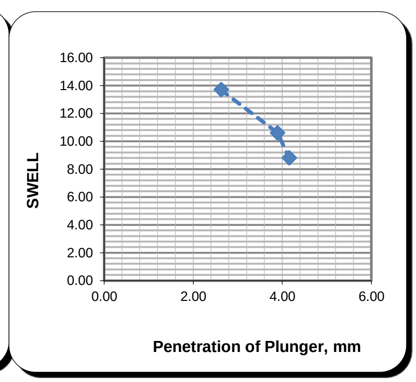


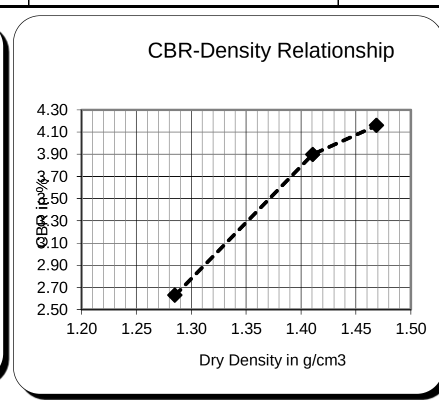


Expansive soil+2% cement+10% Wood Ash

Ring Factor:	0.00373							
10 blows 5 Layers			30 blows, 5 Layers			65 blows, 5 Layers		
PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE	
PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD
0	0	0.00	0	0	0.00	0	0	0.00
0.64	32	0.12	0.64	49	0.18	0.64	54	0.20
1.27	53	0.20	1.27	79	0.29	1.27	87	0.32
1.91	73	0.27	1.91	106	0.40	1.91	112	0.42
2.54	89	0.33	2.54	131	0.49	2.54	138	0.51
3.18	104	0.39	3.18	151	0.56	3.18	161	0.60
3.81	117	0.44	3.81	172	0.64	3.81	185	0.69
4.45	129	0.48	4.45	192	0.72	4.45	204	0.76
5.08	141	0.53	5.08	209	0.78	5.08	223	0.83
7.62	160	0.60	7.62	216	0.81	7.62	228	0.85
9.16	174	0.65	9.16	228	0.85	9.16	231	0.86
SWELL	Dial	swell%	SWELL	Dial	swell%	SWELL	Dial	swll%
	2.01			2.01			2.01	
	17.95	13.68		14.35	10.59		12.25	8.79
CBR %	PEN.	2.5mm	5.08mm	2.5mm	5.08mm	2.5mm	5.08mm	
	TOP							
	BOTTOM	2.5	2.6	3.7	3.9	3.9	4.2	
	RECO.CBR	2.6		3.9		4.2		

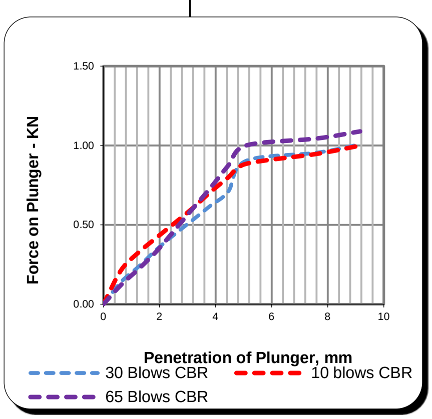


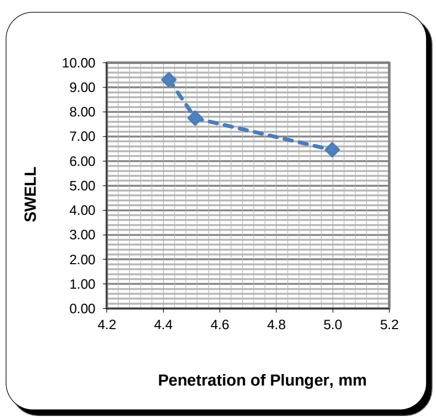


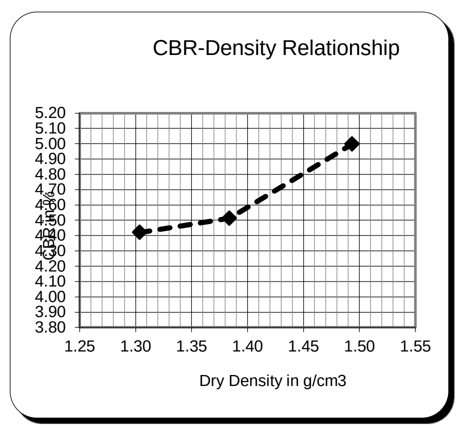


Expansive soil+2% cement+15% Wood Ash

Ring Factor:	0.00373							
10 blows 5 Layers			30 blows, 5 Layers			65 blows, 5 Layers		
PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE	
PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD
0	0	0.00	0	0	0.00	0	0	0.00
0.64	58	0.22	0.64	38	0.14	0.64	33	0.12
1.27	88	0.33	1.27	64	0.24	1.27	60	0.22
1.91	113	0.42	1.91	93	0.35	1.91	90	0.34
2.54	137	0.51	2.54	118	0.44	2.54	125	0.47
3.18	162	0.60	3.18	142	0.53	3.18	161	0.60
3.81	189	0.70	3.81	166	0.62	3.81	196	0.73
4.45	214	0.80	4.45	190	0.71	4.45	234	0.87
5.08	237	0.88	5.08	242	0.90	5.08	268	1.00
7.62	254	0.95	7.62	256	0.95	7.62	280	1.04
9.16	268	1.00	9.16	270	1.01	9.16	292	1.09
SWELL	Dial	swell%	SWELL	Dial	swell%	SWELL	Dial	swll%
	2.01			2.01			2.01	
	12.86	9.31		11.03	7.74		9.53	6.45
CBR %	PEN.	2.5mm	5.08mm	2.5mm	5.08mm	2.5mm	5.08mm	
	TOP							
	BOTTOM	3.9	4.4	3.3	4.5	3.5	5.0	
	RECO.CBR	4.4		4.5		5.0		

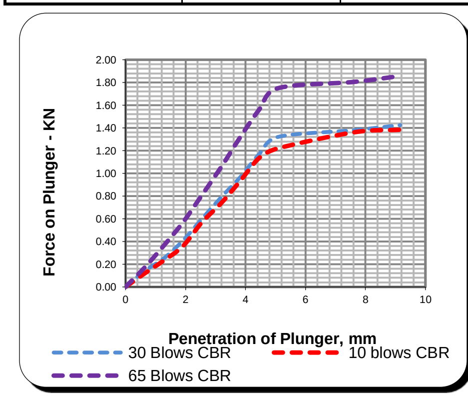


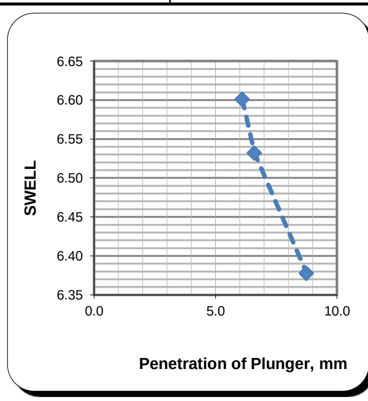


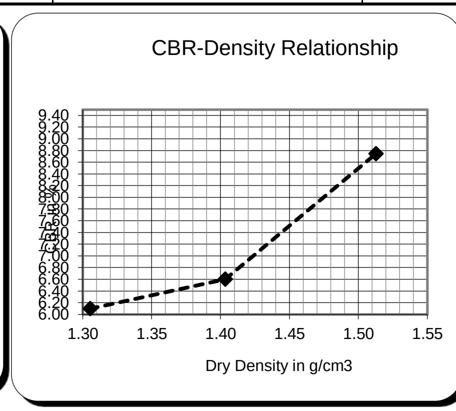


Expansive soil+2% cement+20% Wood Ash

Ring Factor:	0.00373							
10 blows 5 Layers			30 blows, 5 Layers			65 blows, 5 Layers		
PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE	
PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD
0	0	0.00	0	0	0.00	0	0	0.00
0.64	32	0.12	0.64	38	0.14	0.64	46	0.17
1.27	62	0.23	1.27	67	0.25	1.27	97	0.36
1.91	96	0.36	1.91	109	0.41	1.91	152	0.57
2.54	152	0.57	2.54	160	0.60	2.54	216	0.81
3.18	197	0.73	3.18	211	0.79	3.18	282	1.05
3.81	249	0.93	3.81	258	0.96	3.81	352	1.31
4.45	304	1.13	4.45	313	1.17	4.45	418	1.56
5.08	327	1.22	5.08	354	1.32	5.08	469	1.75
7.62	365	1.36	7.62	370	1.38	7.62	484	1.81
9.16	371	1.38	9.16	382	1.42	9.16	498	1.86
SWELL	Dial	swell%	SWELL	Dial	swell%	SWELL	Dial	swll%
	2.01			2.01			2.01	
	9.70	6.60		9.62	6.53		9.44	6.38
CBR %	PEN.	2.5mm	5.08mm	2.5mm	5.08mm	2.5mm	5.08mm	
	TOP							
	BOTTOM	4.3	6.1	4.5	6.6	6.1	8.7	
	RECO.CBR	6.1		6.6		8.7		

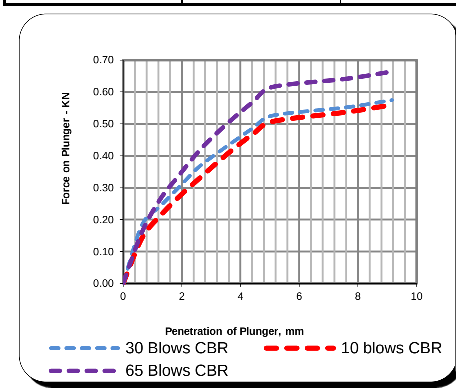


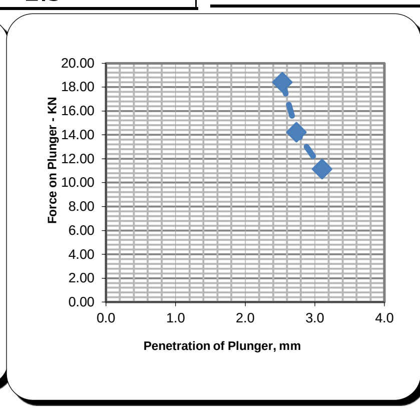


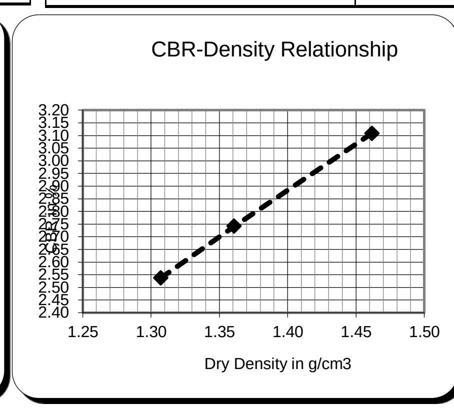


Expansive soil+4% cement+5% Wood Ash

Ring Factor: 0.00373								
10 blows 5 Layers			30 blows, 5 Layers			65 blows, 5 Layers		
PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE	
PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD
0	0	0.00	0	0	0.00	0	0	0.00
0.64	38	0.14	0.64	49	0.18	0.64	43	0.16
1.27	57	0.21	1.27	65	0.24	1.27	71	0.26
1.91	73	0.27	1.91	81	0.30	1.91	91	0.34
2.54	87	0.32	2.54	97	0.36	2.54	110	0.41
3.18	101	0.38	3.18	109	0.41	3.18	126	0.47
3.81	114	0.43	3.81	120	0.45	3.81	140	0.52
4.45	126	0.47	4.45	131	0.49	4.45	153	0.57
5.08	136	0.51	5.08	141	0.53	5.08	165	0.62
7.62	144	0.54	7.62	148	0.55	7.62	172	0.64
9.16	150	0.56	9.16	154	0.57	9.16	178	0.66
SWELL	Dial	swell%	SWELL	Dial	swell%	SWELL	Dial	swll%
	2.01			2.01			2.01	
	23.42	18.38		18.52	14.17		14.95	11.11
CBR %	PEN.	2.5mm	5.08mm	2.5mm	5.08mm	2.5mm	5.08mm	
	TOP							
	BOTTOM	2.5	2.5	2.7	2.6	3.1	3.1	
	RECO.CBR	2.5		2.7		3.1		

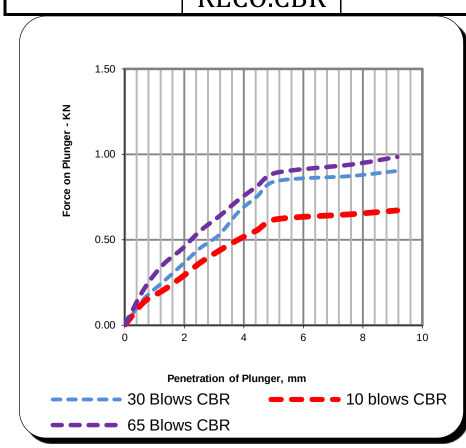


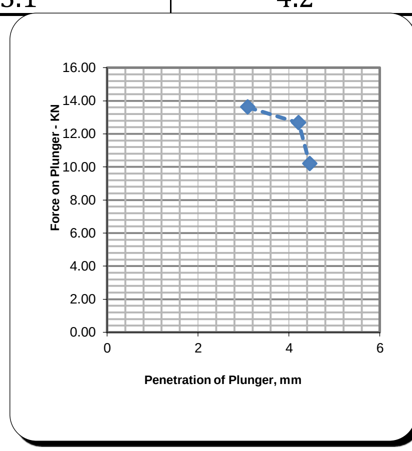


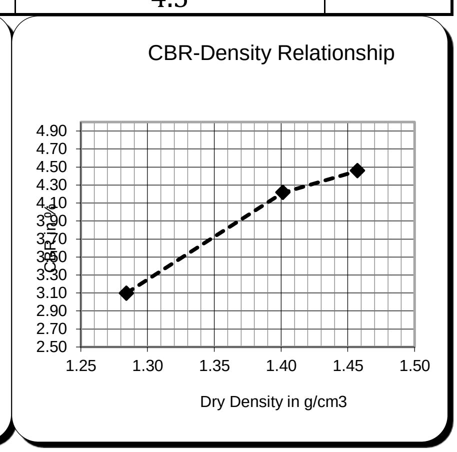


Expansive soil+4% cement+10% Wood Ash

Ring Factor: 0.00373								
10 blows 5 Layers			30 blows, 5 Layers			65 blows, 5 Layers		
PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE	
PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD
0	0	0.00	0	0	0.00	0	0	0.00
0.64	36	0.13	0.64	41	0.15	0.64	56	0.21
1.27	54	0.20	1.27	67	0.25	1.27	94	0.35
1.91	75	0.28	1.91	94	0.35	1.91	119	0.44
2.54	98	0.37	2.54	121	0.45	2.54	148	0.55
3.18	117	0.44	3.18	142	0.53	3.18	171	0.64
3.81	134	0.50	3.81	177	0.66	3.81	196	0.73
4.45	149	0.56	4.45	202	0.75	4.45	218	0.81
5.08	166	0.62	5.08	226	0.84	5.08	239	0.89
7.62	174	0.65	7.62	234	0.87	7.62	252	0.94
9.16	180	0.67	9.16	242	0.90	9.16	264	0.98
SWELL	Dial	swell%	SWELL	Dial	swell%	SWELL	Dial	swll%
	2.01			2.01			2.01	
	17.86	13.61		16.76	12.66		13.88	10.19
CBR %	PEN.	2.5mm	5.08mm	2.5mm	5.08mm	2.5mm	5.08mm	
	TOP							
	BOTTOM	2.8	3.1	3.4	4.2	4.2	4.5	
	RECO.CBR	3.1		4.2		4.5		

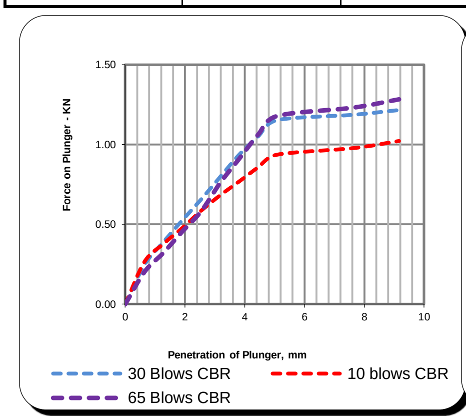


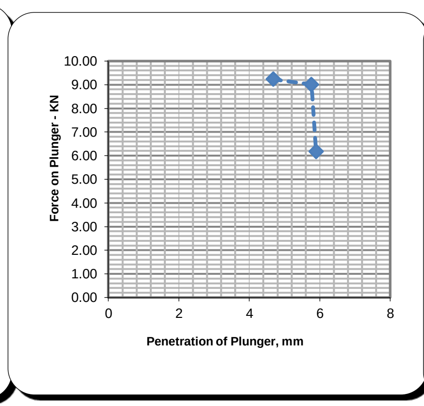


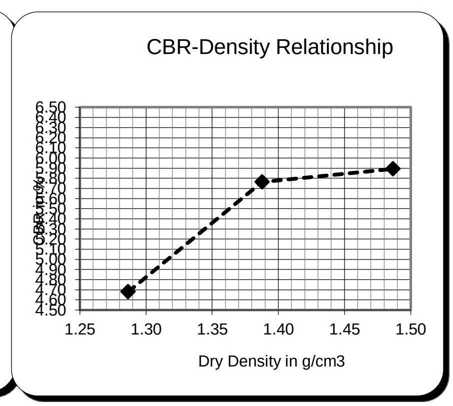


Expansive soil+4% cement+15% Wood Ash

Ring Factor:	0.00373							
10 blows 5 Layers			30 blows, 5 Layers			65 blows, 5 Layers		
PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE	
PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD
0	0	0.00	0	0	0.00	0	0	0.00
0.64	71	0.26	0.64	66	0.25	0.64	54	0.20
1.27	101	0.38	1.27	104	0.39	1.27	86	0.32
1.91	128	0.48	1.91	141	0.53	1.91	122	0.46
2.54	157	0.59	2.54	176	0.66	2.54	156	0.58
3.18	183	0.68	3.18	214	0.80	3.18	204	0.76
3.81	206	0.77	3.81	251	0.94	3.81	244	0.91
4.45	230	0.86	4.45	282	1.05	4.45	285	1.06
5.08	251	0.94	5.08	309	1.15	5.08	316	1.18
7.62	262	0.98	7.62	318	1.19	7.62	330	1.23
9.16	274	1.02	9.16	326	1.22	9.16	344	1.28
SWELL	Dial	swell%	SWELL	Dial	swell%	SWELL	Dial	swll%
	2.01			2.01			2.01	
	12.76	9.23		12.50	9.00		9.20	6.17
CBR %	PEN.	2.5mm	5.08mm	2.5mm	5.08mm	2.5mm	5.08mm	
	TOP							
	BOTTOM	4.4	4.7	5.0	5.8	4.4	5.9	
	RECO.CBR	4.7		5.8		5.9		

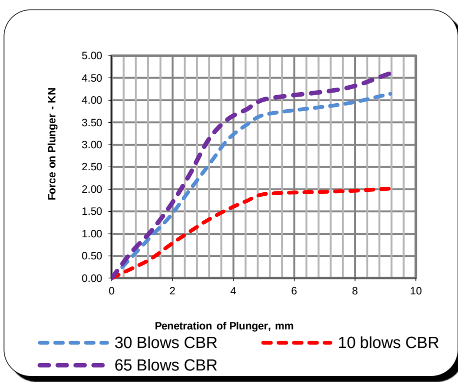


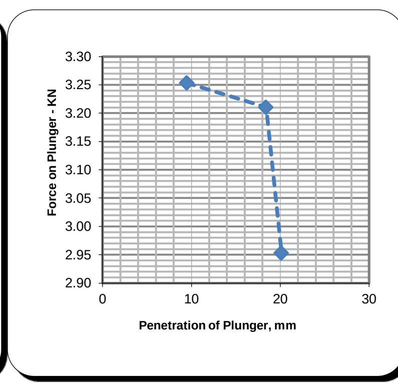


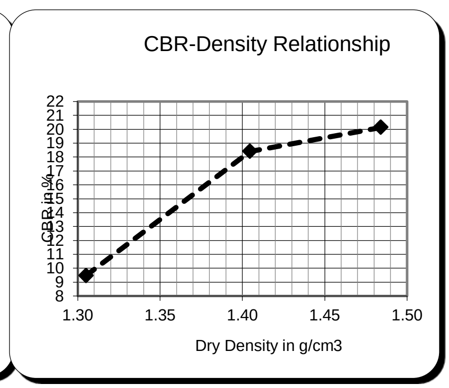


Expansive soil+4% cement+20% Wood Ash

Ring Factor:	0.00373		0.11509			0.11509		
10 blows 5 Layers			30 blows, 5 Layers			65 blows, 5 Layers		
PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE	
PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD
0	0	0.00	0	0	0.00	0	0	0.00
0.64	56	0.21	0.64	4	0.46	0.64	5	0.58
1.27	114	0.43	1.27	8	0.92	1.27	9	1.04
1.91	199	0.74	1.91	12	1.38	1.91	14	1.61
2.54	277	1.03	2.54	17	1.96	2.54	20	2.30
3.18	352	1.31	3.18	22	2.53	3.18	27	3.11
3.81	414	1.54	3.81	27	3.11	3.81	31	3.57
4.45	466	1.74	4.45	30	3.45	4.45	33	3.80
5.08	508	1.89	5.08	32	3.68	5.08	35	4.03
7.62	524	1.95	7.62	34	3.91	7.62	37	4.26
9.16	540	2.01	9.16	36	4.14	9.16	40	4.60
SWELL	Dial	swell%	SWELL	Dial	swell%	SWELL	Dial	swll%
	2.01			2.01			2.01	
	5.80	3.25		5.75	3.21		5.45	2.95
CBR %	PEN.	2.5mm	5.08mm	2.5mm	5.08mm	2.5mm	5.08mm	
	TOP							
	BOTTOM	7.8	9.5	14.8	18.4	17.4	20.1	
	RECO.CBR	9.5		18.4		20.1		

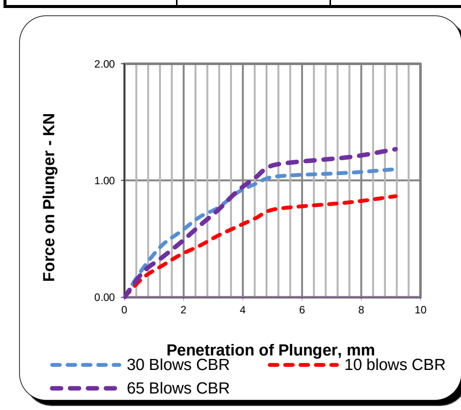


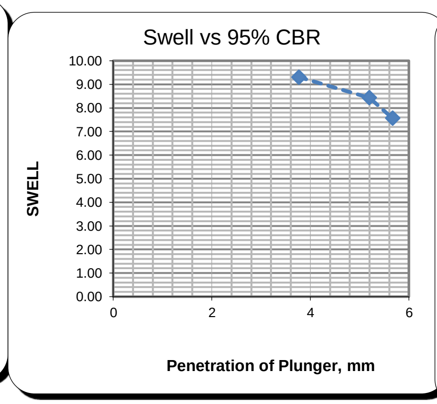


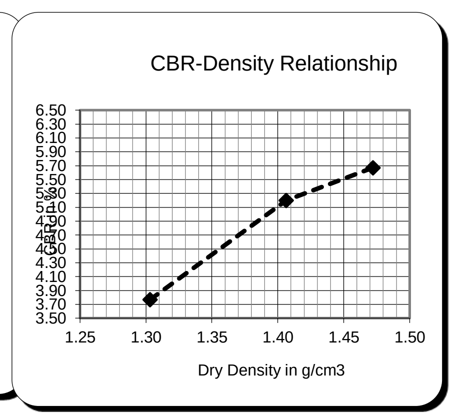


Expansive soil+6% cement+5% Wood Ash

Ring Factor: 0.00373								
10 blows 5 Layers			30 blows, 5 Layers			65 blows, 5 Layers		
PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE	
PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD
0	0	0.00	0	0	0.00	0	0	0.00
0.64	45	0.17	0.64	67	0.25	0.64	58	0.22
1.27	72	0.27	1.27	119	0.44	1.27	91	0.34
1.91	98	0.37	1.91	151	0.56	1.91	126	0.47
2.54	118	0.44	2.54	184	0.69	2.54	164	0.61
3.18	142	0.53	3.18	206	0.77	3.18	201	0.75
3.81	162	0.60	3.81	240	0.90	3.81	243	0.91
4.45	182	0.68	4.45	261	0.97	4.45	276	1.03
5.08	202	0.75	5.08	277	1.03	5.08	304	1.13
7.62	218	0.81	7.62	286	1.07	7.62	322	1.20
9.16	232	0.87	9.16	294	1.10	9.16	340	1.27
SWELL	Dial	swell %	SWELL	Dial	swell%	SWELL	Dial	swll%
	2.01			2.01			2.01	
	12.85	9.30		11.83	8.43		10.82	7.56
CBR %	PEN.	2.5mm	5.08mm	2.5m m	5.08m m	2.5mm	5.08m m	
	TOP							
	BOTTOM	3.3	3.8	5.2	5.2	4.6	5.7	
	RECO.CB R	3.8		5.2		5.7		

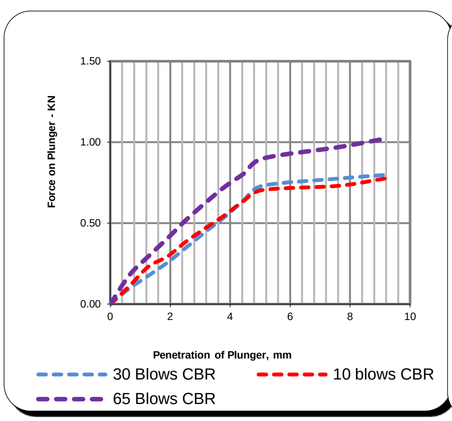


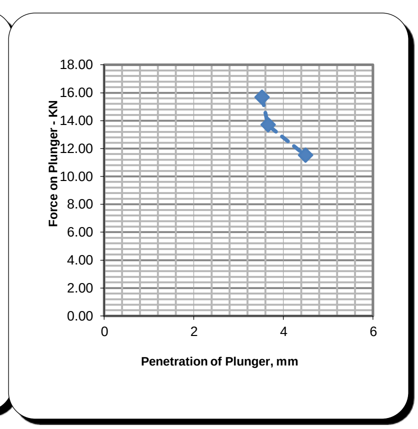


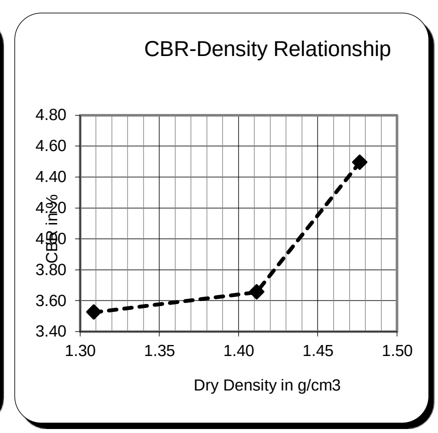


Expansive soil+6% cement+10% Wood Ash

Ring Factor:	0.00373							
10 blows 5 Layers			30 blows, 5 Layers			65 blows, 5 Layers		
PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE	
PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD
0	0	0.00	0	0	0.00	0	0	0.00
0.64	30	0.11	0.64	26	0.10	0.64	48	0.18
1.27	62	0.23	1.27	47	0.18	1.27	79	0.29
1.91	79	0.29	1.91	69	0.26	1.91	109	0.41
2.54	104	0.39	2.54	94	0.35	2.54	140	0.52
3.18	126	0.47	3.18	120	0.45	3.18	168	0.63
3.81	147	0.55	3.81	144	0.54	3.81	194	0.72
4.45	171	0.64	4.45	172	0.64	4.45	216	0.81
5.08	189	0.70	5.08	196	0.73	5.08	241	0.90
7.62	196	0.73	7.62	208	0.78	7.62	260	0.97
9.16	208	0.78	9.16	214	0.80	9.16	274	1.02
SWELL	Dial	swell %	SWELL	Dial	swell%	SWELL	Dial	swll %
	2.01			2.01			2.01	
	20.25	15.66		17.95	13.68		15.40	11.49
CBR %	PEN.	2.5mm	5.08mm	2.5mm	5.08mm	2.5mm	5.08mm	
	TOP							
	BOTTOM	2.9	3.5	2.7	3.7	4.0	4.5	
	RECO.CBR	3.5		3.7		4.5		

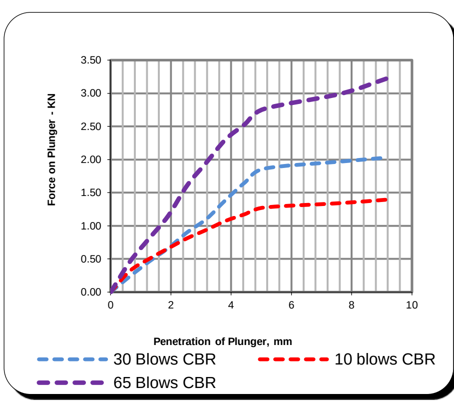


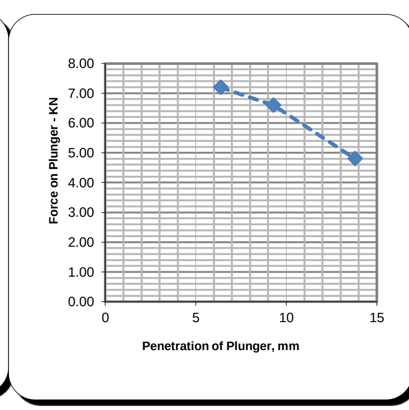


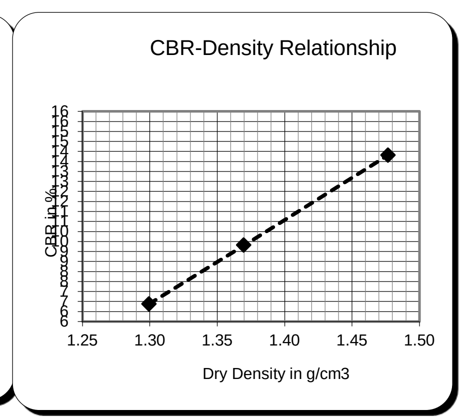


Expansive soil+6% cement+15% Wood Ash

Ring Factor: 0.00373			0.00373			0.11509		
10 blows 5 Layers			30 blows, 5 Layers			65 blows, 5 Layers		
PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE	
PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD
0	0	0.00	0	0	0.00	0	0	0.00
0.64	86	0.32	0.64	64	0.24	0.64	4	0.46
1.27	134	0.50	1.27	123	0.46	1.27	7	0.81
1.91	177	0.66	1.91	178	0.66	1.91	10	1.15
2.54	218	0.81	2.54	242	0.90	2.54	14	1.61
3.18	252	0.94	3.18	296	1.10	3.18	17	1.96
3.81	288	1.07	3.81	372	1.39	3.81	20	2.30
4.45	315	1.17	4.45	443	1.65	4.45	22	2.53
5.08	342	1.28	5.08	499	1.86	5.08	24	2.76
7.62	360	1.34	7.62	528	1.97	7.62	26	2.99
9.16	374	1.40	9.16	544	2.03	9.16	28	3.22
SWELL	Dial	swell%	SWELL	Dial	swell%	SWELL	Dial	swll %
	2.01			2.01			2.01	
	10.39	7.19		9.69	6.59		7.60	4.80
CBR %	PEN.	2.5mm	5.08mm	2.5mm	5.08mm	2.5mm	5.08mm	
	TOP							
	BOTTOM	6.2	6.4	6.8	9.3	12.2	13.8	
	RECO.CBR	6.4		9.3		13.8		

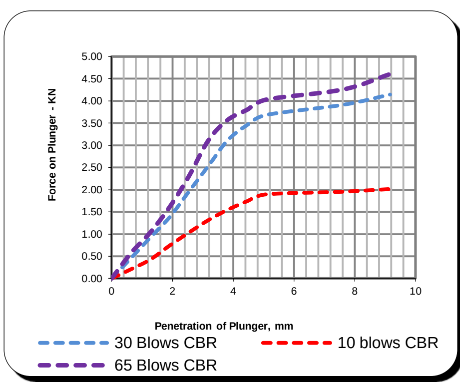


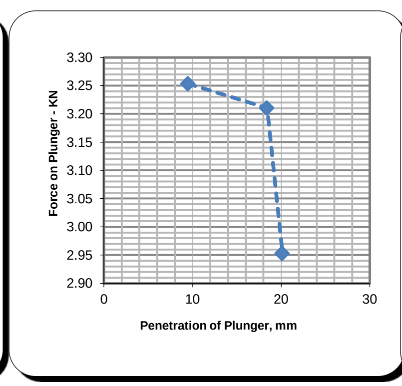


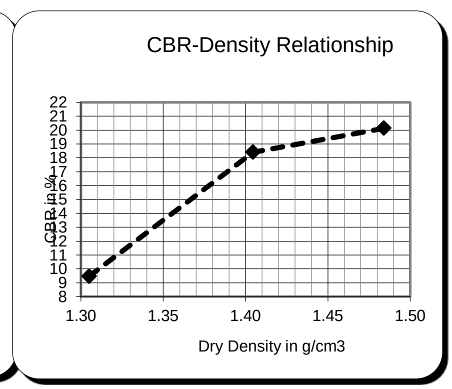


Expansive soil+6% cement+20% Wood Ash

Ring Factor:	0.00373		0.11509			0.11509		
10 blows 5 Layers			30 blows, 5 Layers			65 blows, 5 Layers		
PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE		PLUNGER	BOTTOM FACE	
PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD	PENT.(mm)	DIAL	LOAD
0	0	0.00	0	0	0.00	0	0	0.00
0.64	56	0.21	0.64	4	0.46	0.64	5	0.58
1.27	114	0.43	1.27	8	0.92	1.27	9	1.04
1.91	199	0.74	1.91	12	1.38	1.91	14	1.61
2.54	277	1.03	2.54	17	1.96	2.54	27	3.11
3.18	352	1.31	3.18	22	2.53	3.18	29	3.34
3.81	414	1.54	3.81	27	3.11	3.81	35	4.03
4.45	486	1.81	4.45	30	3.45	4.45	38	4.37
5.08	528	1.97	5.08	34	3.91	5.08	42	4.87
7.62	544	2.03	7.62	36	4.14	7.62	44	5.06
9.16	560	2.09	9.16	38	4.37	9.16	46	5.29
SWELL	Dial	swell%	SWELL	Dial	swell%	SWELL	Dial	swll %
	2.01			2.01			2.01	
	5.40	2.91		5.35	2.87		4.84	2.43
CBR %	PEN.	2.5mm	5.08mm	2.5mm	5.08mm	2.5mm	5.08mm	
	TOP							
	BOTTOM	7.8	9.8	14.8	19.6	23.5	24.3	
	RECO.CBR	9.8		19.6		24.3		



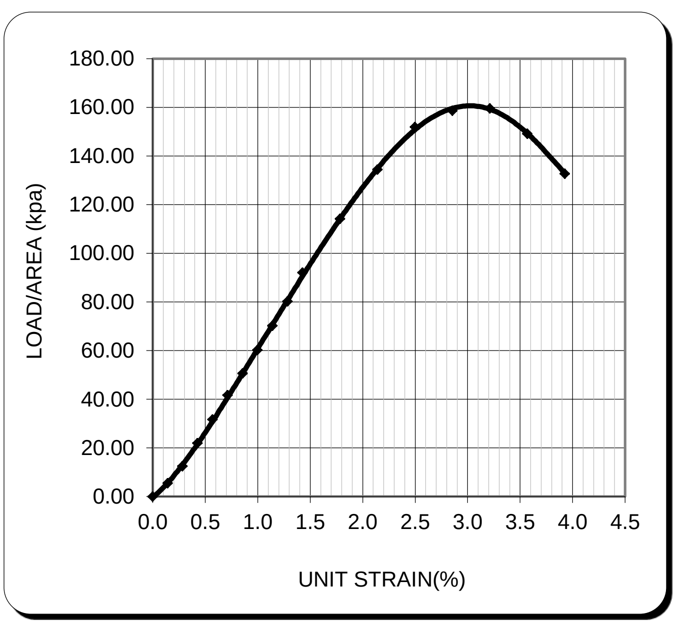




UCS

Expansive soil

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL (mm)	Strain (E)	% Strain	Corrected Area A'	Load (KN)	Stress (kpa)
0	0	0.0	0.0	0.0	9503.3	0.0	0.0
20	14	0.2	0.0014	0.1429	9516.9	5.222	5.5
40	32	0.4	0.0029	0.2857	9530.5	11.936	12.5
60	56	0.6	0.0043	0.4286	9544.2	20.888	21.9
80	81	0.8	0.0057	0.5714	9557.9	30.213	31.6
100	107	1.0	0.0071	0.7143	9571.7	39.911	41.7
120	130	1.2	0.0086	0.8571	9585.5	48.490	50.6
140	155	1.4	0.0100	1.0000	9599.3	57.815	60.2
160	181	1.6	0.0114	1.1429	9613.2	67.513	70.2
180	207	1.8	0.0129	1.2857	9627.1	77.211	80.2
200	238	2.0	0.0143	1.4286	9641.0	88.774	92.1
250	296	2.5	0.0179	1.7857	9676.1	110.408	114.1
300	350	3.0	0.0214	2.1429	9711.4	130.550	134.4
350	397	3.5	0.0250	2.5000	9747.0	148.081	151.9
400	416	4.0	0.0286	2.8571	9782.8	155.168	158.6
450	420	4.5	0.0321	3.2143	9818.9	156.660	159.5
500	394	5.0	0.0357	3.5714	9855.3	146.962	149.1
550	352	5.5	0.0393	3.9286	9891.9	131.296	132.7

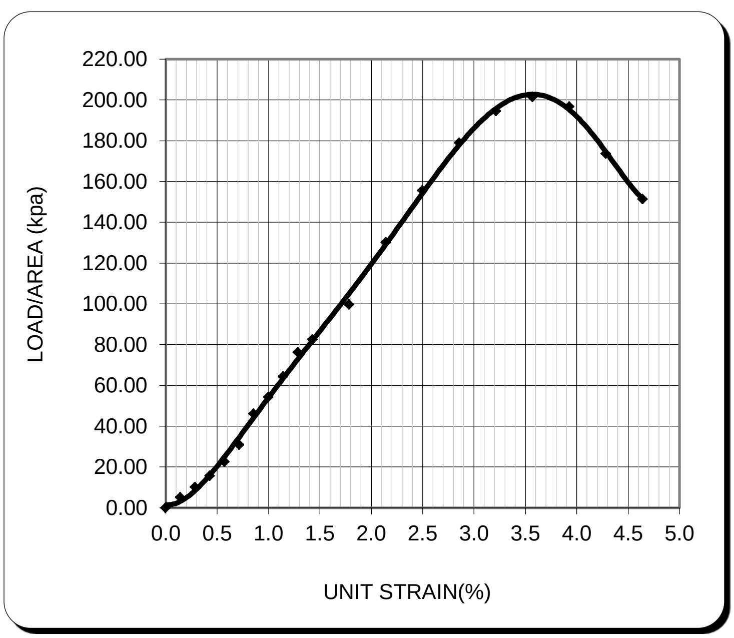


UCS=161kpa

CU=qu/2=80.5

Expansive soil+2% cement+5% Wood Ash

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL (mm)	Strain (E)	% Strain	Corrected Area A'	Load (KN)	Stress (kpa)
0	0	0.0	0.0	0.0	8011.8	0.0	0.0
20	11	0.2	0.0014	0.1429	8023.3	4.103	5.1
40	22	0.4	0.0029	0.2857	8034.8	8.206	10.2
60	34	0.6	0.0043	0.4286	8046.3	12.682	15.8
80	49	0.8	0.0057	0.5714	8057.9	18.277	22.7
100	67	1.0	0.0071	0.7143	8069.5	24.991	31.0
120	100	1.2	0.0086	0.8571	8081.1	37.300	46.2
140	118	1.4	0.0100	1.0000	8092.8	44.014	54.4
160	140	1.6	0.0114	1.1429	8104.5	52.220	64.4
180	166	1.8	0.0129	1.2857	8116.2	61.918	76.3
200	180	2.0	0.0143	1.4286	8128.0	67.140	82.6
250	218	2.5	0.0179	1.7857	8157.5	81.314	99.7
300	286	3.0	0.0214	2.1429	8187.3	106.678	130.3
350	343	3.5	0.0250	2.5000	8217.3	127.939	155.7
400	396	4.0	0.0286	2.8571	8247.5	147.708	179.1
450	432	4.5	0.0321	3.2143	8277.9	161.136	194.7
500	449	5.0	0.0357	3.5714	8308.6	167.477	201.6
550	440	5.5	0.0393	3.9286	8339.5	164.120	196.8
600	390	6.0	0.0429	4.2857	8370.6	145.470	173.8
650	341	6.5	0.0464	4.6429	8401.9	127.193	151.4

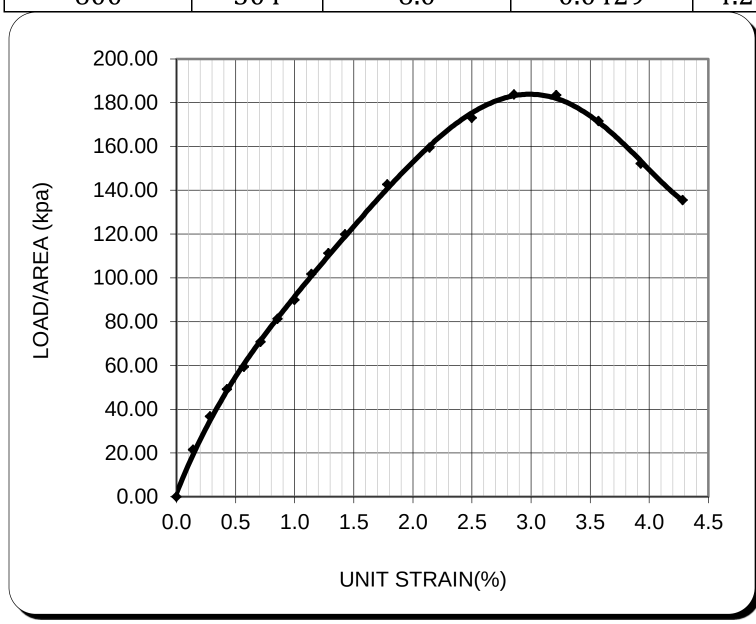


UCS=202kpa

CU=qu/2=101kpa

Expansive soil+2% cement+10% Wood Ash

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL (mm)	Strain (E)	% Strain	Corrected Area A'	Load (KN)	Stress (kpa)
0	0	0.0	0.0	0.0	8011.8	0.0	0.0
20	46	0.2	0.0014	0.1429	8023.3	17.158	21.4
40	79	0.4	0.0029	0.2857	8034.8	29.467	36.7
60	106	0.6	0.0043	0.4286	8046.3	39.538	49.1
80	128	0.8	0.0057	0.5714	8057.9	47.744	59.3
100	153	1.0	0.0071	0.7143	8069.5	57.069	70.7
120	176	1.2	0.0086	0.8571	8081.1	65.648	81.2
140	195	1.4	0.0100	1.0000	8092.8	72.735	89.9
160	221	1.6	0.0114	1.1429	8104.5	82.433	101.7
180	242	1.8	0.0129	1.2857	8116.2	90.266	111.2
200	261	2.0	0.0143	1.4286	8128.0	97.353	119.8
250	312	2.5	0.0179	1.7857	8157.5	116.376	142.7
300	350	3.0	0.0214	2.1429	8187.3	130.550	159.5
350	381	3.5	0.0250	2.5000	8217.3	142.113	172.9
400	406	4.0	0.0286	2.8571	8247.5	151.438	183.6
450	407	4.5	0.0321	3.2143	8277.9	151.811	183.4
500	382	5.0	0.0357	3.5714	8308.6	142.486	171.5
550	340	5.5	0.0393	3.9286	8339.5	126.820	152.1
600	304	6.0	0.0429	4.2857	8370.6	113.392	135.5

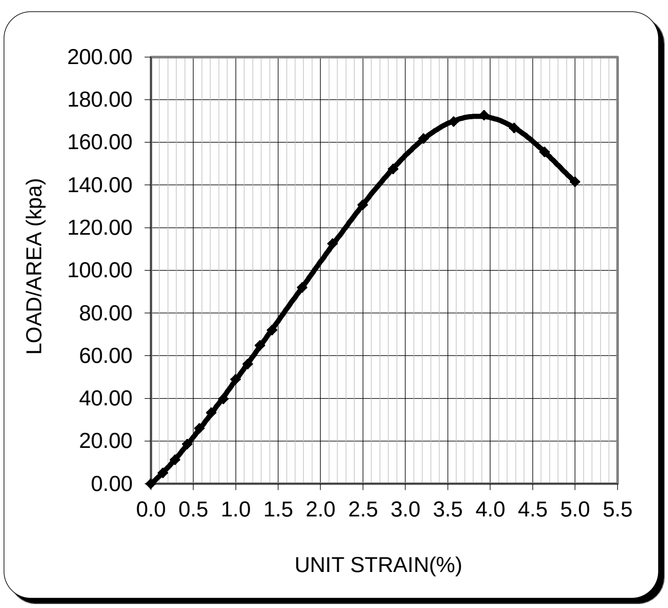


UCS=184kpa

CU=qu/2=92kpa

Expansive soil+2% cement+15% Wood Ash

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL (mm)	Strain (E)	% Strain	Corrected Area A'	Load (KN)	Stress (kpa)
0	0	0.0	0.0	0.0	8011.8	0.0	0.0
20	11	0.2	0.0014	0.1429	8023.3	4.103	5.1
40	24	0.4	0.0029	0.2857	8034.8	8.952	11.1
60	40	0.6	0.0043	0.4286	8046.3	14.920	18.5
80	56	0.8	0.0057	0.5714	8057.9	20.888	25.9
100	72	1.0	0.0071	0.7143	8069.5	26.856	33.3
120	86	1.2	0.0086	0.8571	8081.1	32.078	39.7
140	106	1.4	0.0100	1.0000	8092.8	39.538	48.9
160	122	1.6	0.0114	1.1429	8104.5	45.506	56.1
180	141	1.8	0.0129	1.2857	8116.2	52.593	64.8
200	157	2.0	0.0143	1.4286	8128.0	58.561	72.0
250	201	2.5	0.0179	1.7857	8157.5	74.973	91.9
300	247	3.0	0.0214	2.1429	8187.3	92.131	112.5
350	288	3.5	0.0250	2.5000	8217.3	107.424	130.7
400	326	4.0	0.0286	2.8571	8247.5	121.598	147.4
450	359	4.5	0.0321	3.2143	8277.9	133.907	161.8
500	378	5.0	0.0357	3.5714	8308.6	140.994	169.7
550	386	5.5	0.0393	3.9286	8339.5	143.978	172.6
600	374	6.0	0.0429	4.2857	8370.6	139.502	166.7
650	350	6.5	0.0464	4.6429	8401.9	130.550	155.4
700	320	7.0	0.05000	5.00000	8433.5	119.360	141.5

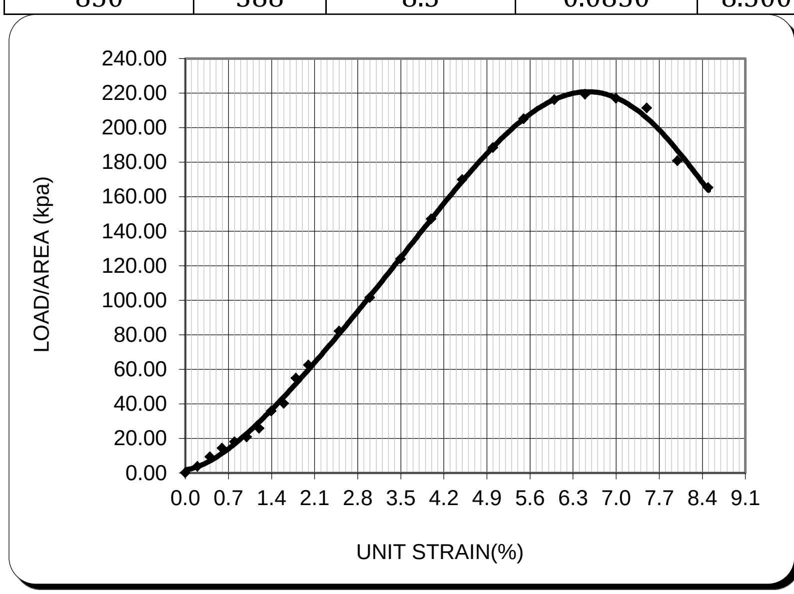


UCS=172kpa

CU=qu/2=86kpa

Expansive soil+2% cement+20% Wood Ash

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL (mm)	Strain (E)	% Strain	Corrected Area A'	Load (KN)	Stress (kpa)
0	0	0.0	0.0	0.0	8011.8	0.0	0.0
20	8	0.2	0.0020	0.2000	8027.9	2.984	3.7
40	20	0.4	0.0040	0.4000	8044.0	7.460	9.3
60	31	0.6	0.0060	0.6000	8060.2	11.563	14.3
80	39	0.8	0.0080	0.8000	8076.5	14.547	18.0
100	45	1.0	0.0100	1.0000	8092.8	16.785	20.7
120	56	1.2	0.0120	1.2000	8109.2	20.888	25.8
140	78	1.4	0.0140	1.4000	8125.6	29.094	35.8
160	88	1.6	0.0160	1.6000	8142.1	32.824	40.3
180	120	1.8	0.0180	1.8000	8158.7	44.760	54.9
200	137	2.0	0.0200	2.0000	8175.4	51.101	62.5
250	181	2.5	0.0250	2.5000	8217.3	67.513	82.2
300	225	3.0	0.0300	3.0000	8259.6	83.925	101.6
350	276	3.5	0.0350	3.5000	8302.4	102.948	124.0
400	329	4.0	0.0400	4.0000	8345.7	122.717	147.0
450	382	4.5	0.0450	4.5000	8389.4	142.486	169.8
500	426	5.0	0.0500	5.0000	8433.5	158.898	188.4
550	466	5.5	0.0550	5.5000	8478.1	173.818	205.0
600	494	6.0	0.0600	6.0000	8523.2	184.262	216.2
650	504	6.5	0.0650	6.5000	8568.8	187.992	219.4
700	501	7.0	0.07000	7.00000	8614.9	186.873	216.9
750	491	7.5	0.0750	7.5000	8661.5	183.143	211.4
800	422	8.0	0.0800	8.0000	8708.5	157.406	180.7
850	388	8.5	0.0850	8.5000	8756.1	144.724	165.3

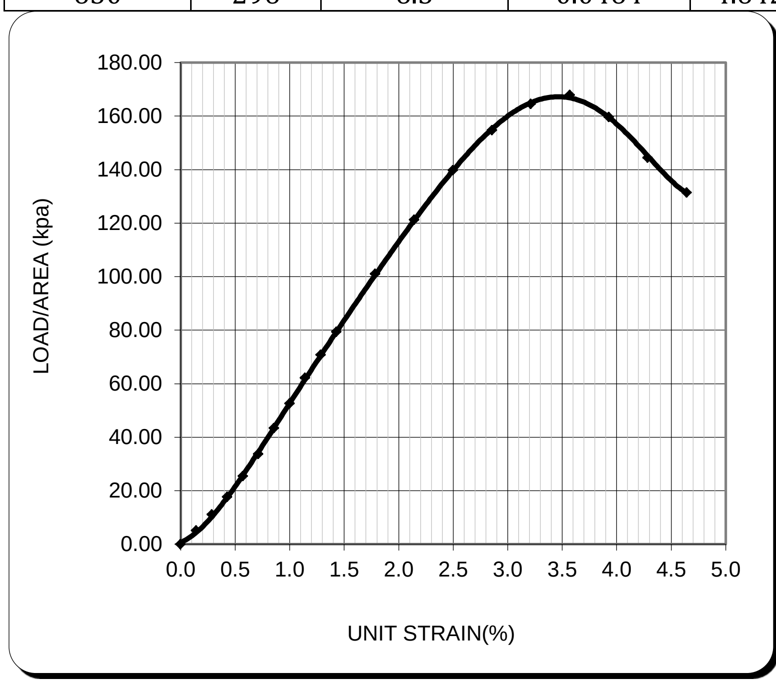


UCS=220kpa

CU=110kpa

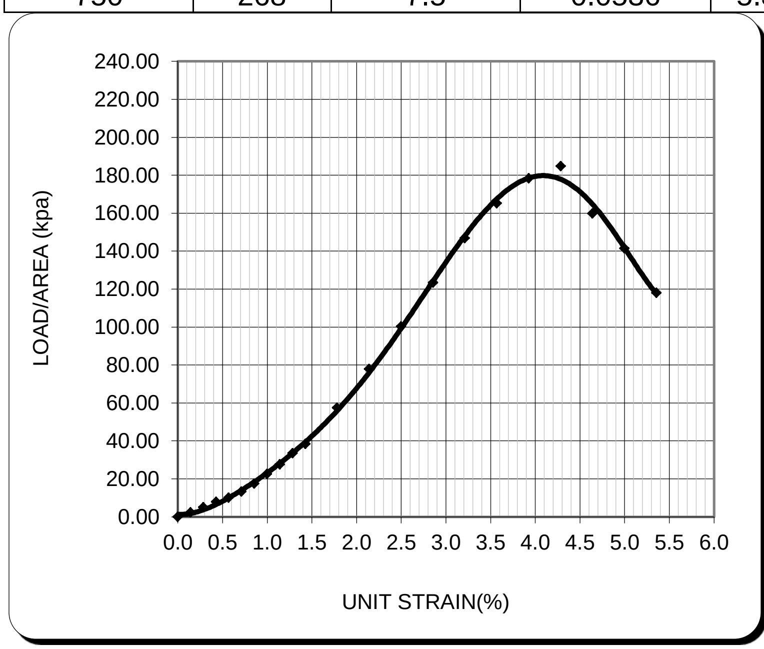
Expansive soil+4% cement+5% Wood Ash

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL (mm)	Strain (E)	% Strain	Corrected Area A'	Load (KN)	Stress (kpa)
0	0	0.0	0.0	0.0	8011.8	0.0	0.0
20	11	0.2	0.0014	0.1429	8023.3	4.103	5.1
40	24	0.4	0.0029	0.2857	8034.8	8.952	11.1
60	38	0.6	0.0043	0.4286	8046.3	14.174	17.6
80	55	0.8	0.0057	0.5714	8057.9	20.515	25.5
100	73	1.0	0.0071	0.7143	8069.5	27.229	33.7
120	94	1.2	0.0086	0.8571	8081.1	35.062	43.4
140	114	1.4	0.0100	1.0000	8092.8	42.522	52.5
160	135	1.6	0.0114	1.1429	8104.5	50.355	62.1
180	154	1.8	0.0129	1.2857	8116.2	57.442	70.8
200	173	2.0	0.0143	1.4286	8128.0	64.529	79.4
250	221	2.5	0.0179	1.7857	8157.5	82.433	101.1
300	266	3.0	0.0214	2.1429	8187.3	99.218	121.2
350	308	3.5	0.0250	2.5000	8217.3	114.884	139.8
400	342	4.0	0.0286	2.8571	8247.5	127.566	154.7
450	365	4.5	0.0321	3.2143	8277.9	136.145	164.5
500	374	5.0	0.0357	3.5714	8308.6	139.502	167.9
550	357	5.5	0.0393	3.9286	8339.5	133.161	159.7
600	324	6.0	0.0429	4.2857	8370.6	120.852	144.4
650	296	6.5	0.0464	4.6429	8401.9	110.408	131.4



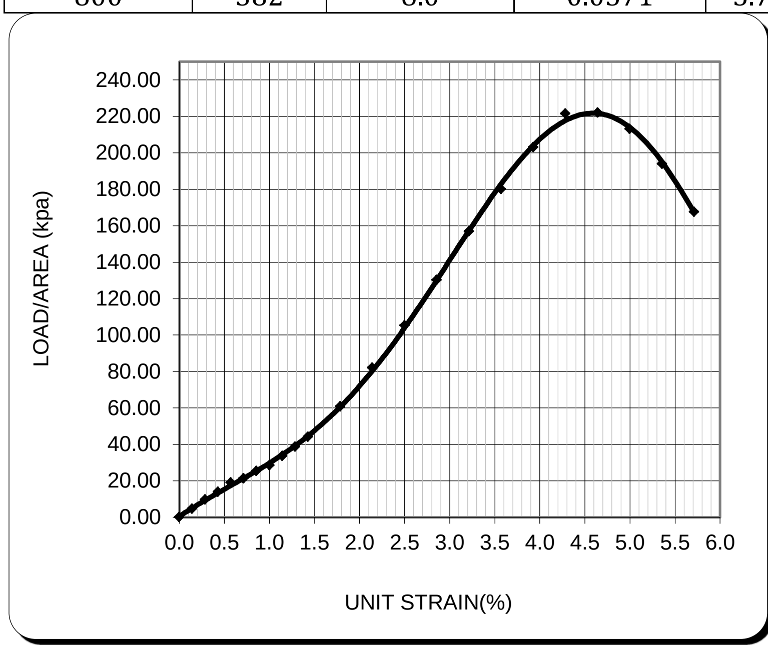
Expansive soil+4% cement+10% Wood Ash

Deformation Dial Reading	Load Dial Reading	Sample Deformation ∆L (mm)	Strain (*E*)	% Strain	Corrected Area A'	Load (KN)	Stress (kpa)
0	0	0.0	0.0	0.0	8011.8	0.0	0.0
20	5	0.2	0.0014	0.1429	8023.3	1.865	2.3
40	11	0.4	0.0029	0.2857	8034.8	4.103	5.1
60	17	0.6	0.0043	0.4286	8046.3	6.341	7.9
80	22	0.8	0.0057	0.5714	8057.9	8.206	10.2
100	29	1.0	0.0071	0.7143	8069.5	10.817	13.4
120	38	1.2	0.0086	0.8571	8081.1	14.174	17.5
140	49	1.4	0.0100	1.0000	8092.8	18.277	22.6
160	60	1.6	0.0114	1.1429	8104.5	22.380	27.6
180	73	1.8	0.0129	1.2857	8116.2	27.229	33.5
200	84	2.0	0.0143	1.4286	8128.0	31.332	38.5
250	126	2.5	0.0179	1.7857	8157.5	46.998	57.6
300	171	3.0	0.0214	2.1429	8187.3	63.783	77.9
350	221	3.5	0.0250	2.5000	8217.3	82.433	100.3
400	273	4.0	0.0286	2.8571	8247.5	101.829	123.5
450	326	4.5	0.0321	3.2143	8277.9	121.598	146.9
500	368	5.0	0.0357	3.5714	8308.6	137.264	165.2
550	399	5.5	0.0393	3.9286	8339.5	148.827	178.5
600	415	6.0	0.0429	4.2857	8370.6	154.795	184.9
650	360	6.5	0.0464	4.6429	8401.9	134.280	159.8
700	320	7.0	0.05000	5.00000	8433.5	119.360	141.5
750	268	7.5	0.0536	5.3571	8465.3	99.964	118.1



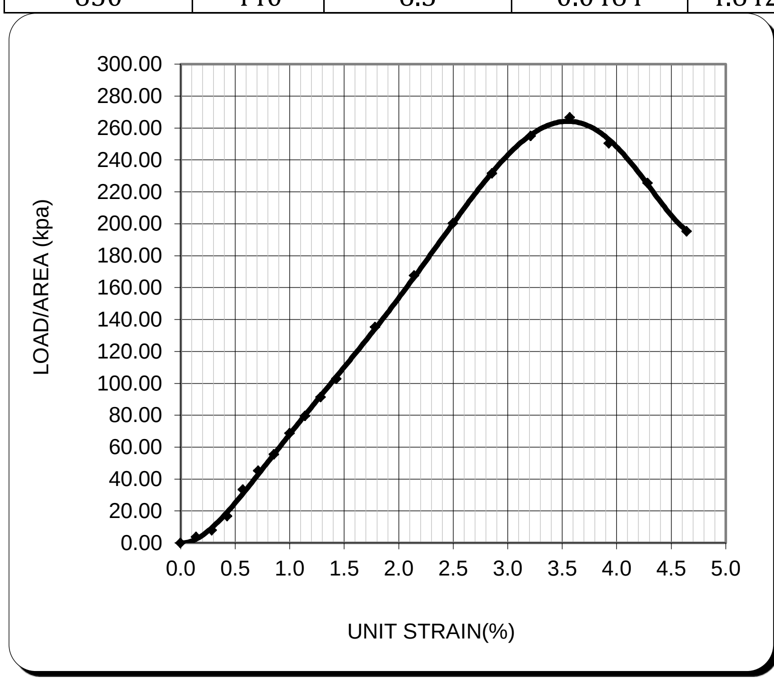
Expansive soil+4% cement+15% Wood Ash

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL (mm)	Strain (E)	% Strain	Corrected Area A'	Load (KN)	Stress (kpa)
0	0	0.0	0.0	0.0	8011.8	0.0	0.0
20	10	0.2	0.0014	0.1429	8023.3	3.730	4.6
40	21	0.4	0.0029	0.2857	8034.8	7.833	9.7
60	30	0.6	0.0043	0.4286	8046.3	11.190	13.9
80	41	0.8	0.0057	0.5714	8057.9	15.293	19.0
100	46	1.0	0.0071	0.7143	8069.5	17.158	21.3
120	55	1.2	0.0086	0.8571	8081.1	20.515	25.4
140	62	1.4	0.0100	1.0000	8092.8	23.126	28.6
160	73	1.6	0.0114	1.1429	8104.5	27.229	33.6
180	84	1.8	0.0129	1.2857	8116.2	31.332	38.6
200	96	2.0	0.0143	1.4286	8128.0	35.808	44.1
250	133	2.5	0.0179	1.7857	8157.5	49.609	60.8
300	180	3.0	0.0214	2.1429	8187.3	67.140	82.0
350	232	3.5	0.0250	2.5000	8217.3	86.536	105.3
400	288	4.0	0.0286	2.8571	8247.5	107.424	130.3
450	348	4.5	0.0321	3.2143	8277.9	129.804	156.8
500	401	5.0	0.0357	3.5714	8308.6	149.573	180.0
550	454	5.5	0.0393	3.9286	8339.5	169.342	203.1
600	497	6.0	0.0429	4.2857	8370.6	185.381	221.5
650	500	6.5	0.0464	4.6429	8401.9	186.500	222.0
700	482	7.0	0.05000	5.00000	8433.5	179.786	213.2
750	440	7.5	0.0536	5.3571	8465.3	164.120	193.9
800	382	8.0	0.0571	5.7143	8497.4	142.486	167.7



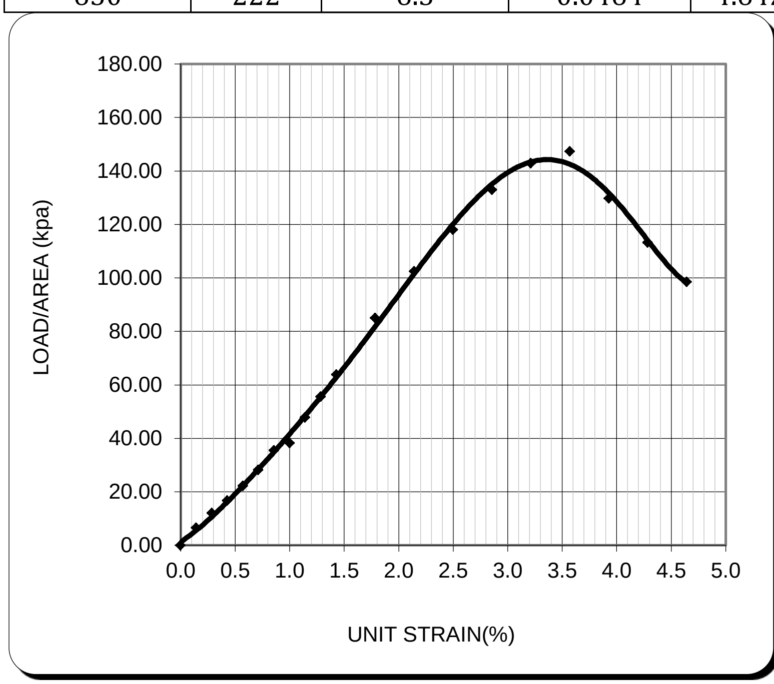
Expansive soil+4% cement+20% Wood Ash

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL (mm)	Strain (E)	% Strain	Corrected Area A'	Load (KN)	Stress (kpa)
0	0	0.0	0.0	0.0	8011.8	0.0	0.0
20	8	0.2	0.0014	0.1429	8023.3	2.984	3.7
40	17	0.4	0.0029	0.2857	8034.8	6.341	7.9
60	36	0.6	0.0043	0.4286	8046.3	13.428	16.7
80	72	0.8	0.0057	0.5714	8057.9	26.856	33.3
100	98	1.0	0.0071	0.7143	8069.5	36.554	45.3
120	120	1.2	0.0086	0.8571	8081.1	44.760	55.4
140	149	1.4	0.0100	1.0000	8092.8	55.577	68.7
160	173	1.6	0.0114	1.1429	8104.5	64.529	79.6
180	199	1.8	0.0129	1.2857	8116.2	74.227	91.5
200	224	2.0	0.0143	1.4286	8128.0	83.552	102.8
250	296	2.5	0.0179	1.7857	8157.5	110.408	135.3
300	368	3.0	0.0214	2.1429	8187.3	137.264	167.7
350	441	3.5	0.0250	2.5000	8217.3	164.493	200.2
400	512	4.0	0.0286	2.8571	8247.5	190.976	231.6
450	566	4.5	0.0321	3.2143	8277.9	211.118	255.0
500	594	5.0	0.0357	3.5714	8308.6	221.562	266.7
550	560	5.5	0.0393	3.9286	8339.5	208.880	250.5
600	506	6.0	0.0429	4.2857	8370.6	188.738	225.5
650	440	6.5	0.0464	4.6429	8401.9	164.120	195.3



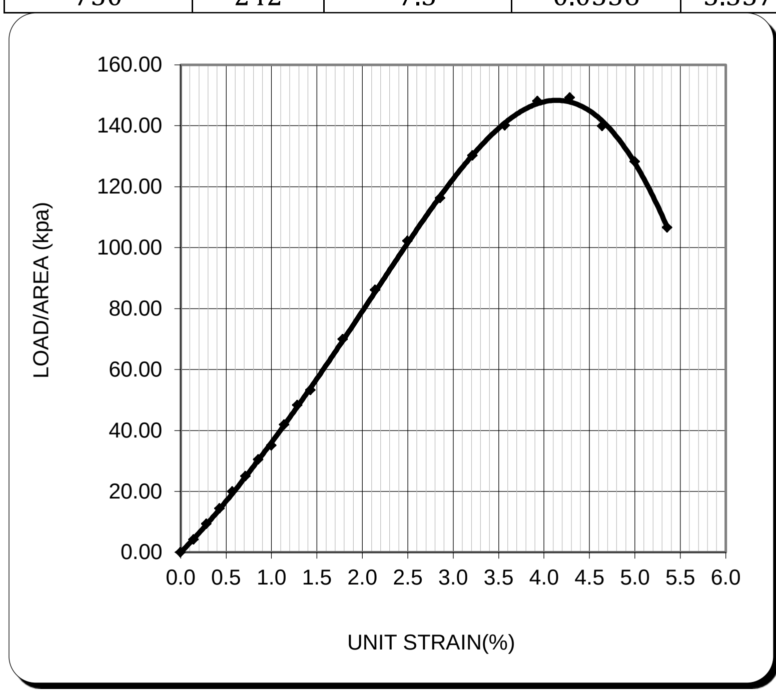
Expansive soil+6% cement+5% Wood Ash

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL (mm)	Strain (E)	% Strain	Corrected Area A'	Load (KN)	Stress (kpa)
0	0	0.0	0.0	0.0	8011.8	0.0	0.0
20	14	0.2	0.0014	0.1429	8023.3	5.222	6.5
40	26	0.4	0.0029	0.2857	8034.8	9.698	12.1
60	36	0.6	0.0043	0.4286	8046.3	13.428	16.7
80	48	0.8	0.0057	0.5714	8057.9	17.904	22.2
100	61	1.0	0.0071	0.7143	8069.5	22.753	28.2
120	77	1.2	0.0086	0.8571	8081.1	28.721	35.5
140	83	1.4	0.0100	1.0000	8092.8	30.959	38.3
160	104	1.6	0.0114	1.1429	8104.5	38.792	47.9
180	121	1.8	0.0129	1.2857	8116.2	45.133	55.6
200	139	2.0	0.0143	1.4286	8128.0	51.847	63.8
250	186	2.5	0.0179	1.7857	8157.5	69.378	85.0
300	225	3.0	0.0214	2.1429	8187.3	83.925	102.5
350	260	3.5	0.0250	2.5000	8217.3	96.980	118.0
400	294	4.0	0.0286	2.8571	8247.5	109.662	133.0
450	317	4.5	0.0321	3.2143	8277.9	118.241	142.8
500	328	5.0	0.0357	3.5714	8308.6	122.344	147.3
550	290	5.5	0.0393	3.9286	8339.5	108.170	129.7
600	254	6.0	0.0429	4.2857	8370.6	94.742	113.2
650	222	6.5	0.0464	4.6429	8401.9	82.806	98.6



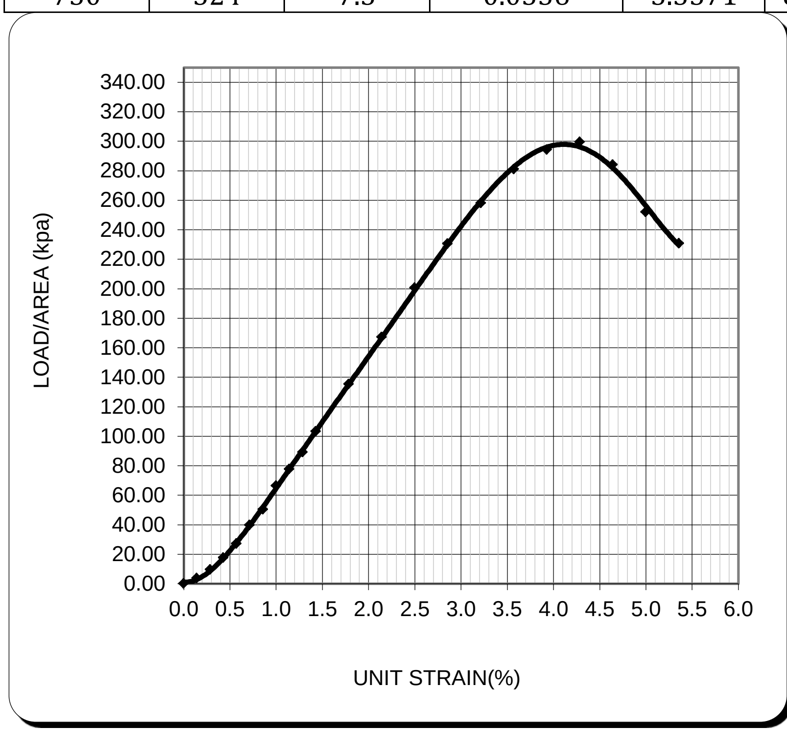
Expansive soil+6% cement+10% Wood Ash

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL (mm)	Strain (E)	% Strain	Corrected Area A'	Load (KN)	Stress (kpa)
0	0	0.0	0.0	0.0	8011.8	0.0	0.0
20	9	0.2	0.0014	0.1429	8023.3	3.357	4.2
40	20	0.4	0.0029	0.2857	8034.8	7.460	9.3
60	31	0.6	0.0043	0.4286	8046.3	11.563	14.4
80	43	0.8	0.0057	0.5714	8057.9	16.039	19.9
100	54	1.0	0.0071	0.7143	8069.5	20.142	25.0
120	66	1.2	0.0086	0.8571	8081.1	24.618	30.5
140	76	1.4	0.0100	1.0000	8092.8	28.348	35.0
160	91	1.6	0.0114	1.1429	8104.5	33.943	41.9
180	105	1.8	0.0129	1.2857	8116.2	39.165	48.3
200	116	2.0	0.0143	1.4286	8128.0	43.268	53.2
250	153	2.5	0.0179	1.7857	8157.5	57.069	70.0
300	189	3.0	0.0214	2.1429	8187.3	70.497	86.1
350	225	3.5	0.0250	2.5000	8217.3	83.925	102.1
400	257	4.0	0.0286	2.8571	8247.5	95.861	116.2
450	289	4.5	0.0321	3.2143	8277.9	107.797	130.2
500	312	5.0	0.0357	3.5714	8308.6	116.376	140.1
550	331	5.5	0.0393	3.9286	8339.5	123.463	148.0
600	335	6.0	0.0429	4.2857	8370.6	124.955	149.3
650	315	6.5	0.0464	4.6429	8401.9	117.495	139.8
700	290	7.0	0.05000	5.00000	8433.5	108.170	128.3
750	242	7.5	0.0536	5.3571	8465.3	90.266	106.6



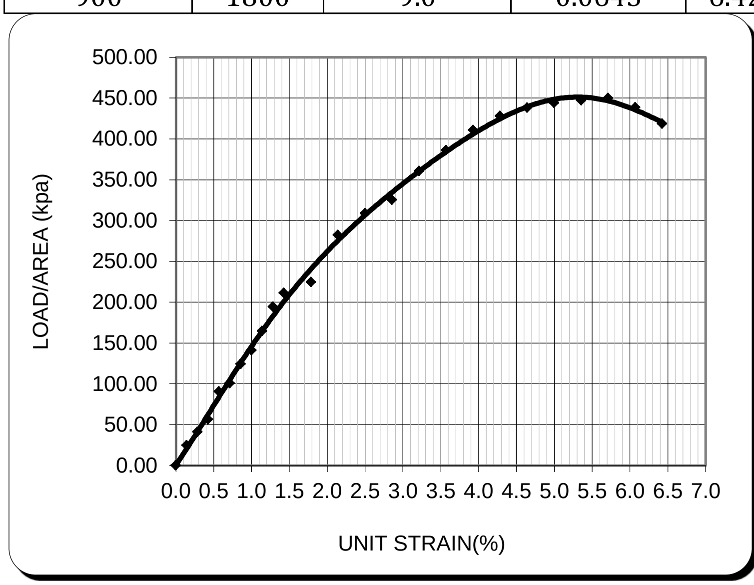
Expansive soil+6% cement+15% Wood Ash

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL (mm)	Strain (*E*)	% Strain	Corrected Area A'	Load (KN)	Stress (kpa)
0	0	0.0	0.0	0.0	8011.8	0.0	0.0
20	8	0.2	0.0014	0.1429	8023.3	2.984	3.7
40	21	0.4	0.0029	0.2857	8034.8	7.833	9.7
60	38	0.6	0.0043	0.4286	8046.3	14.174	17.6
80	59	0.8	0.0057	0.5714	8057.9	22.007	27.3
100	86	1.0	0.0071	0.7143	8069.5	32.078	39.8
120	109	1.2	0.0086	0.8571	8081.1	40.657	50.3
140	144	1.4	0.0100	1.0000	8092.8	53.712	66.4
160	169	1.6	0.0114	1.1429	8104.5	63.037	77.8
180	194	1.8	0.0129	1.2857	8116.2	72.362	89.2
200	225	2.0	0.0143	1.4286	8128.0	83.925	103.3
250	296	2.5	0.0179	1.7857	8157.5	110.408	135.3
300	367	3.0	0.0214	2.1429	8187.3	136.891	167.2
350	442	3.5	0.0250	2.5000	8217.3	164.866	200.6
400	510	4.0	0.0286	2.8571	8247.5	190.230	230.7
450	573	4.5	0.0321	3.2143	8277.9	213.729	258.2
500	626	5.0	0.0357	3.5714	8308.6	233.498	281.0
550	658	5.5	0.0393	3.9286	8339.5	245.434	294.3
600	672	6.0	0.0429	4.2857	8370.6	250.656	299.4
650	640	6.5	0.0464	4.6429	8401.9	238.720	284.1
700	570	7.0	0.05000	5.00000	8433.5	212.610	252.1
750	524	7.5	0.0536	5.3571	8465.3	195.452	230.9



Expansive soil+6% cement+20% Wood Ash

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL (mm)	Strain (E)	% Strain	Corrected Area A'	Load (KN)	Stress (kpa)
0	0	0.0	0.0	0.0	8011.8	0.0	0.0
20	100	0.2	0.0014	0.1429	8023.3	37.300	24.8
40	166	0.4	0.0029	0.2857	8034.8	61.918	41.1
60	228	0.6	0.0043	0.4286	8046.3	85.044	56.4
80	366	0.8	0.0057	0.5714	8057.9	136.518	90.4
100	408	1.0	0.0071	0.7143	8069.5	152.184	100.7
120	504	1.2	0.0086	0.8571	8081.1	187.992	124.2
140	574	1.4	0.0100	1.0000	8092.8	214.102	141.2
160	670	1.6	0.0114	1.1429	8104.5	249.910	164.6
180	793	1.8	0.0129	1.2857	8116.2	295.789	194.5
200	863	2.0	0.0143	1.4286	8128.0	321.899	211.5
250	920	2.5	0.0179	1.7857	8157.5	343.160	224.5
300	1160	3.0	0.0214	2.1429	8187.3	432.680	282.1
350	1275	3.5	0.0250	2.5000	8217.3	475.575	308.9
400	1348	4.0	0.0286	2.8571	8247.5	502.804	325.4
450	1500	4.5	0.0321	3.2143	8277.9	559.500	360.7
500	1612	5.0	0.0357	3.5714	8308.6	601.276	386.2
550	1720	5.5	0.0393	3.9286	8339.5	641.560	410.6
600	1800	6.0	0.0429	4.2857	8370.6	671.400	428.1
650	1850	6.5	0.0464	4.6429	8401.9	690.050	438.3
700	1882	7.0	0.05000	5.00000	8433.5	701.986	444.2
750	1901	7.5	0.0536	5.3571	8465.3	709.073	447
800	1920	8.0	0.0571	5.7143	8497.4	716.160	449.8
850	1880	8.5	0.0607	6.0714	8529.7	701.240	438.8
900	1800	9.0	0.0643	6.4286	8562.3	671.400	418.5



THE END!