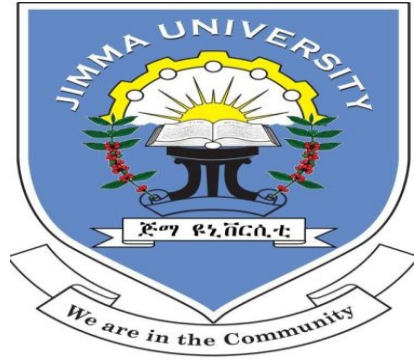




Jimma University
Jimma Institute Of Technology
Faculty Of Civil And Environmental Engineering
(Highway Engineering Stream)

**Stabilization Of Weak Subgrade Soil Using Maize Cob Ash And
Waste Brick Powder: A Case Study at Tarcha Zuriya Woreda**

A Final Research Submitted to School of Graduate Studies in Partial
Fulfillment of the Requirements for Degree of Master of Science in
Civil Engineering (Highway Engineering stream)

By:

Matawork Tarekegn

June, 2023

Jimma, Ethiopia

Jimma University
Jimma Institute Of Technology
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Highway Engineering Stream

**Stabilization of Weak Sub-Grade Soil Using Maize Cob Ash and
Waste brick Powder**

APPROVED BY THE BOARD OF EXAMINERS

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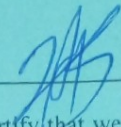
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I, the undersigned, declare that this thesis entitled "Stabilization of Weak Subgrade soil Using Maize cob ash and with waste brick powder ." is my original work, and has not been presented by any other person for an award of a degree in this or any other University, and all sources of material used for theses have been duly acknowledged.

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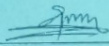
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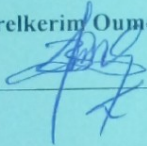
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ABSTRACT

Strength of weak soils present significant geotechnical, road pavements, and structural engineering challenges the world over, with costs associated with expansive behavior estimated to run into several billion dollars of damage annually. Expansive soils are soils that experience significant volume change (swelling and shrinking) associated with changes in water contents. These problems need a wider application of cost-effective and environmentally friendly technology for improving soil properties to be adapted to the current road construction trend. The general objective of this research is to investigate the suitability of MCA and WBP for stabilization of Weak subgrade soils. To achieve the objectives of the research to based on the properties described under field investigations:- Four samples were taken from different locations. A free swell test, high plastic index and low load bearing were carried out for each sample, one sample with high free swell, high plastic index and poor soil values had been selected. Tests were performed on natural and treated soil samples as per the ERA, AASHTO, and ASTM laboratory test standards. The selected soil samples were characterized and were found A-7-5 category of soil based on the AASHTO soil classification system and CH as per the USCS soil classification system which is clay soil of poor engineering properties. The soils were treated with varied proportions of MCA (5%, 10%, 15%, and 20%) and WBP (10%, 20%, 30%, and 40%). and (5%MCA + 40 %WBP, 10%MCA + 30 %WBP, 15%MCA + 20 %WBP, 20%MCA + 10 %WBP) by dry weight of the soil. The laboratory tests carried out to evaluate the stabilized soil sample were; Atterberg limits, free swell, compaction tests, and CBR tests. Analysis of the results shows that improvement on the engineering properties of MCA-WBP stabilized soil. As the MCA-WBP content adds, reduces plasticity index and swelling potential, increase MDD and the OMC is decrease and the CBR strength also increased and the better improvement was found when using 10% MCA and 30%WBP. It can be concluded from the present investigation that weak subgrade soil could properly be treated for use as subgrade material. Adequate strength was achieved with 10% MCA mixed with 30%WBP, and which is found to be the optimum Maize cob ash and waste brick contents for improving the engineering properties of the weak subgrade soils.

Key Words:- Weak subgrade Soil, Maize cob ash, Stabilization, waste brick Powder, Subgrade Strength.

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ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society transportation manual
CAH	Calcium Aluminate Hydrate
CBR	California Bearing Ratio
CC	Centimeter Cube
CCA	Corn cob ash
CEC	Cation exchange capacity
CSH	Calcium silicate hydrate
ERA	Ethiopian Road Authority
FAO	Food and Agricultural Organization
KG	Kilogram
LL	Liquid Limit
MCA	Maize Cob Ash
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PI	Plastic Index
PL	Plastic Limit
SWERME	South West Ethiopian Region Of Metrological Agency
USCS	Unified Soil Classification system
WBP	Waste Brick Powder

CHAPTER ONE

1. INTRODUCTION

1.1. BACKGROUND OF THE STUDY

Worldwide, the availability of natural construction materials within reasonable distance is one of the major factors that have a direct impact on the investment cost of road projects. In areas where natural construction materials are readily available, roads can be constructed on sound economic basis. However, in some regions, natural construction materials are either not available or do not fulfill the quality requirements of road building materials. Infrastructure growth across the world formed demand for construction resources. CO₂ emissions have increased due to the production of cement and inadequate disposal of RHA indications to air pollution and the embankment difficulty. To lessen these problems, RHA was cast off as additive agent of cement in the production of concrete (Kumar & Suresh, 2018).

Civil engineering projects located in areas with soft or weak subgrade soils have traditionally incorporated improvement of soil properties by using various methods. Soil Stabilization is being used for a variety of engineering works, the most common application being in the construction of road and pavements, where the main objective is to increase the strength or stability of soil and to reduce the construction cost by making best use of the locally available materials (Roy, A. 2014)

In Ethiopia like many other countries, lands that are fertile, suitable for agriculture, and covered by forests are highly dominated by expansive soils. In areas with scarce resources of suitable construction materials, it is common to upgrade the available materials using appropriate stabilization techniques and utilize them for the intended construction. In Ethiopia with the increase in population and the scarcity of available land, most of the agricultural lands become construction sites. Such agricultural lands are black cotton soil which has high swelling and shrinkage properties when exposed to changes in moisture content and is poor in shear strength, and this damage is due to moisture fluctuation caused by seasonal variation. One of the weak sub grade soils that not favorable for road construction is expansive soils. Properties of the weak sub grade soil vary from place to place due to topography, climate and content soils etc. Swelling or expansive clays soil is those that contain swelling clay mineral and have high degree of shrink-swell reversibility with change in moisture content. The soil should be treated

or replaced by suitable material to improve its mechanical property before any construction work is undertaken (Melese, 2018).

The road sectors and building foundations in Ethiopia are suffering from the soil that possesses the dual property of high swelling and shrinking. Many damages occur each year on the constructed road pavements and foundation structures over such soil, creating serious problems including increasing the cost of construction and maintenance. The performance of any structure depends on the underlying geotechnical properties of the soil. Soil is the base of any foundation structure, which essentially carries the structural load from beneath and distributes the load effectively. Problematic soils including expansive clay cause major problems in the design, construction, and maintenance of civil structures. To come across the stated problem, it is important to increase the strength of the soil which is crucial for the successful analysis of the structure to be constructed (Kuleno, 2020).

Stabilization is a mechanism in which the natural properties of soil are enhanced by different techniques and an effective method for the improvement of soil properties and pavement systems. Black cotton soil is mainly stabilized using fly ash, lime, and cement. Some agricultural wastes are also used, but they can't be used as a single stabilizing and improve the CBR value and properties of soil using locally available materials to effectively lying of road pavement and increase the strength of the road pavement economically (Sule et al., 2018). Processed waste is less expensive and has better sustainability credentials compared to cement. Moreover, processed wastes are available in sufficient quantities to meet existing demands for sub-grade stabilization and utilized to improve and enhance the geotechnical properties of weak sub-grade materials where possible (Amakye et al., 2021).

To make weak soils useful and meet geotechnical engineering design requirements, researchers focused more on the use of potentially cost-effective materials that are locally available from industrial and agricultural waste to enhance the properties of deficient soils (Firdawek, 2009). Study on soil stabilization method that was used to improve the properties of expansive soil. The most commonly used stabilizers for expansive soils are bitumen, fly ash, lime, and cement, in which the stabilization of expansive soil with lime or cement is more effective.

However, the costs of these stabilizers are very high making them economically unattractive as stabilizing agents (Budhu, 2007). researchers studying in the field of geotechnical engineering, work more on the search for stabilization using cheap and locally available waste materials from industry and agriculture such as maize cob ash, waste ceramic dust powder, Bagasse ash, rice husk ash, coconut husk ash as stabilizing agents for replacement of traditional stabilizers like cement and lime (Ashango & Patra, 2016). Engineering properties of soils are commonly altered when these wastes are blended as admixtures alone or in combination with lime or cement. Moreover, recently, several studies have confirmed the applicability of these wastes as cement replacing material in concrete technology (Firdawek, 2019).

The geotechnical performances of fly ash and rice husk ash (RHA) in stabilizing the weak subgrade were presented. The risks imposed on the environment by possible geotechnical applications of fly ash and rice husk ash (RHA) must be carefully weighed against creating new pollution sources elsewhere. (Dhammika, 2018). Pozzolanas are siliceous and aluminous materials, which itself have little or no cementitious value, but in the presence of water and lime, chemically react with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties and have been used as a cementitious material in construction (Sabzi, 2018).

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 the best utilization of various waste material of construction like waste brick powder and maize cob ash as a soil modifying stabilizing agent. The study focused on investigating the engineering properties of weak soil by the addition and mixing wastage materials, and on determining the minimum amount to obtain the strength of the subgrade materials for road construction. However, environmental damage can occur from the uncontrolled spill of these waste materials in natural habitats, so reusing maize cob ash and waste brick may improve soil properties and also it will reduce their damage to the environment, and reduce cost.

1.2. Statement of the problem

In Ethiopia, the construction of roads is increasing from time to time. Likewise, there is increasing demand for naturally occurring construction materials. Unfortunately, not all the natural materials in the vicinity of road projects are suitable for the proposed

construction because these materials fail to fulfill the standard quality requirements (Deboch, 2018). As stated above expansive soil have great deformation directly related to water content, this deformation is caused by volume change (swelling and shrinking) due to fluctuating moisture condition to makes costly problem with road and other structures.

Understanding the behavior of expansive soil and adopting the appropriate control measures should be great for civil engineers. Instead of borrowing suitable soil from a long distance away, after stabilization with cost-effective and readily available industrial and agricultural waste materials, it is economical to use locally available plastic soil(Fen taw et al., 2021)

The treatment of expansive soil utilizing stabilization like(chemical additives and other additives) enhances the characteristics of expansive soil, the effect of stabilizing agent for stabilization of weak soil, and Stabilization can be performed more economically and more initially than repairs are needed to keep up with an unstable situation, the assessment to guarantee that stabilization technique and environmentally friendly is required(Fondjo et al., 2021)

The cost of construction roads has continually been increasing due to this; it is recommendable to use available waste material and locally available material is beneficial for improving weak soils. One of the development goals of Ethiopia's vision is to provide efficient infrastructure. Alternative locally available materials to lower the cost of construction may achieve this and reduce the wastage of materials for the environment (*Kumar 2015*).

Around Tarcha, zuriya worda weak subgrade soil and different waste materials are available like maize cob ash and waste brick. Therefore, it becomes essential to improve the properties of locally available materials with a stabilizer to the extent that it can be used in the construction of roads and to make the best utilization of various waste materials like maize cob ash and waste brick as soil investigate stabilizing agent.

1.3. Research Question

The research aimed to answer the following research questions:

1. What are the engineering properties of untreat soil?
2. What are the engineering properties of treated soil ?
3. What is the optimum content of maize cob ash and waste brick for stabilization?

1.4.Objective of the study

1.4. General objective

The general objective of this research was to investigate the properties of stabilized weak subgrade soil with maize cob ash and waste brick powder.

1.4.2. Specific objective .

1. To determine engineering properties of untreated soil.
2. To determine the engineering properties of treated soil.
3. To determine the optimum content of maize cob ash and waste brick Powder proportion to stabilize the weak subgrade soil.

1.5. Scope of the study

This study aimed to identification of weak subgrade soil in the selected areas and to investigate the properties of stabilized Weak soil with corresponding percent components of MCA and WBp content. The laboratory works are natural moisture content, specific gravity, Atterberg limit, grain size analysis, free swell, compaction, CBR, and CBR swell of the Tarcha zuriya weak subgrade soil. Tests are conducted and analyzed the results and discussed results compared with ERA Standard Specifications requirements. Finally, a conclusion and recommendation are drawn from this study

1.6. Significance of the Study

This study is to investigate the effect of maize cob ash and waste brick powder on selected weak subgrade soil of Trachea zuriya woreda. The benefit from the studies is cost saving because maize cob ash and waste brick are typically far cheaper than traditional stabilizers such as cement and lime, the study will provide lessons that will help the concerned body to come up with appropriate measures to address problems resulting from loose subgrade soil with cost-effective. Thus, using those materials only possess a significant problem for the local environment and its inhabitants. Therefore, the significance of this research work will introduce maize cob ash and waste brick as new stabilizers in the study area. Using both maize cob ash and waste brick powder for the stabilization of weak subgrade soil is very important because, it is readily available, low costs, and environmentally friendly. Moreover, while utilizing this with the environmental problem like pollution will be reduced.

1.7. Outcome of the study

The outcome at the end of this research served as a new finding which serves as reference material based on its output of all experimental work-study for different governmental or non-governmental organizations as well as other researchers that will satisfy the specification of ERA and AASHTO specification standards.

CHAPTER TWO

2. LITERATURE REVIEW

2.1. Origin of Expansive Soil

The source of expansive soils is related to a combination of conditions and processes that result in the formation of clay minerals having a particular chemical makeup which, when in contact with water, expands. Differences in the conditions and processes could also form other clay minerals, most of which are non-expansive. The conditions or processes, which determine the clay mineralogy; include the composition of the parent material, and the degree of physical, and chemical weathering to which the materials are subjected. The parent materials that can be associated with expansive soil are into two groups. the first group comprises the basic igneous rocks such as basalts, dolerite sills, dykes, and gabbro. In these soils, the feldspar and pyroxene minerals of the parent rocks have decomposed to form montmorillonite, and the second group comprises the sedimentary rocks that contain montmorillonite as a constituent which breaks down physically to form expansive soils (Chen, 1988).

2.2. Distribution of expansive soil

Expansive soils are widespread in the African continent, occurring in South Africa, Ethiopia, Kenya, Mozambique, Morocco, Ghana, Nigeria, etc. In other parts of the world Case of expansive soils has been widely reported in countries like the USA, Australia, India, Spain, Israel, Turkey, Argentina, and Venezuela (Tilahun, 2004) The aerial Coverage of expansive soils in Ethiopia is estimated to be 23.7 million acres (Nebro, 2002)

They are widely spread in the central part of Ethiopia following the major truck roads like Addis Ambo, Addis-Willis, Addis– Debrebirhan, Addis-Gohatsion, and Addis-Modjo are covered by expansive soils. Also, areas like Mekele, Gambella, and southwest Ethiopia are covered by expansive soil.



Figure 2.1:Distributions of expansive soil in Ethiopia (Tilahun, 2004)

Expansive soils can be recognized by using mineralogical identification, indirect index property tests, or direct expansion potential tests. The expansiveness of soil is governed by the type and proportion of clay minerals it contains. Knowing the type and proportion of the clay mineral in the soil indicates the swelling potential (Chen, 1988). However, observed that the extremely small size of clay grains makes the minerals difficult to distinguish in either hand specimen or petrographic microscopy. He also indicated the absence of a single or simple procedure for the identification of clay minerals or their quantification and recommended the application of several methods for even approximate identification and rough quantification (Chen, 1988).

2.2.1 Field Identification

The swell pressure of soil is the external pressure that needs to be placed over a swelling soil to prevent volume increase, while the swell potential of expansive soil is the magnitude of heave of soil for a given final moisture content and loading condition, soil properties, external pressure, and wetting–drying process affect the swell potential and swell pressure of expansive soils(Rao, 2006)

Soils that can show high swelling potential could be identified by field observations, Expansive soil deposits can be recognized in the field through visual inspections. The method is simple and easy to use. Some of the important field identification methods that indicate the potential for the expansiveness of soil are the following:-a color of black and gray, Desiccated surfaces with open or closed fissures, and heave of the ground due to seasonal moisture variation, Wet samples of the soil are sticky causing low traffic ability and Slickenside, (highly polished or shiny fissure surface)

2.2.2. Laboratory Identification

Generally, there are three different methods of identifying expansive soil in the laboratory

1. Mineralogical Identification

This method is used for identifying the mineralogy of clay particles such as characteristic crystal dimensions, characteristic reaction to heat treatment, size, and shape of clay particles, and charge deficiency and surface activity of clay particles. There are different types of techniques, which are used to identify clay mineralogy. The common types of these techniques include X-ray diffraction, differential thermal analysis, and electron microscopy. These properties are a fundamental factor controlling expansive soil behavior (Nelson & Miller, 1992):

2. Direct methods

The swelling pressure and volume changes of soils are measured directly using representative undisturbed samples. The swelling pressure is determined by measuring the pressure needed to prevent the heaving of the sample under the given condition of moisture, density, and confinement. Swelling tests provide complete swelling but due to varying initial conditions of moisture, density, etc., it is difficult to assess the swelling expected in the field. The methods provide quantitative information, which is very useful for engineers to design (Nelson & Miller, 1992).

3. Indirect Methods

In this method, simple soil property tests can be used to evaluate the swelling potential of expansive soils. Such tests are easy to do and should be included as routine tests in the investigation of expansive soils. Such tests may include (Nelson & Miller, 1992):

A1. Atterberg Limits

In this method, the measurements of the Atterberg limits of the soil are performed for the identification of entire soils and provide a wide acceptable means of rating. Particularly

Table 2.1: Relationship between the swelling potential of clays and the plasticity index(Chen, 1988).

Swell potential	Plastic 40-60ty Index	Liquid Limit
Low	0-15	<29
Medium	10-35	29-40
High	19-55	40-60
Very High	55 and above	>60

Based on (Nebro, 2002) it may be true that high swelling soil will manifest high index properties, but the converse is not true.

A2. Method of Daksanamurthy and Raman (1973)

Daksanamurthy and Raman (1973) presented a single index method for identifying expansive soils using only liquid limit as it is shown in the following table

Table 2.2:Expansive soil classification based on the liquid limit (Amer, A., and Mattheus, F.A.,)

Swell potential	Liquid Limit
Low	20<LL<35%
Medium	35<LL<50%
High	50<LL<70%
Very high	LL>70%

B. Free Swell Tests

Soils that have a free swell value as low as 100 percent can cause considerable damage to lightly loaded structures, and soils having a free swell value below 50 percent rarely exhibit considerable volume change even under very light loadings. The free swell test consists of placing a known volume of dry soil passing through sieve No 40 pouring it into a graduated cylinder filled with water and measuring the swelled volume after it has completely settled. The free swell is found as a ratio of change in volume from the dry state to the wet state over the dry volume expressed as a percentage (John et al., 2015). The free swell percentage can be computed using the Equation below from the relationship between the initial and expansion potential as shown in Table 2.2

$$\text{Free swell (\%)} = \frac{V_f - V_i}{V_i} \dots\dots\dots (2.1)$$

Where: VI = initial volume

VF = final volume

Table 2.3: Free swell and degree of expansion relationship (ASTM D 2216)

Free Swell value	Degree of expansion
Free swell <50%	Not expansive
Free swell b/n >50% and <100%	Marginal
Freeswell >100%	Expansive

C. Free Swell Index

The free swell index is also one of the most commonly used simple tests to estimate the swelling Potential of expansive clay. The procedure involves taking two oven-dried soil samples passing through the 425µm sieve, 10cc each was placed separately in two 100-graduated soil Samples. Distilled water was filled with one cylinder and kerosene was in the other cylinder up to the 100ml mark. The final volume of soil is computed after 23 hours to calculate the free swell index. The free swell index is then calculated using the Equation below (Guyer, 2011).

Where:

$$\text{Free Swell Index (\%)} = \frac{V_w - V_k}{V_k} * 100 \dots\dots\dots (2.2)$$

V_w = final volume in water

V_k = final volume in kerosene

Table 2.4: Degree of expansion and differential free swell index (Craig, R.F, 1997)

Free Swell Index (%)	Degree of expansion
Less than 20	Low
20 to 35	Medium
35 to 50	High
Greater than 50	Very high

D. Free Swell Ratio test

For determining the swell property, Sridhar and Prakash proposed the free swell ratio Method of characterizing soil swelling. The free swell ratio can be defined as the ratio

of Sediments volume of 10cc oven-dried soil passing through the 425µm sieve in distilled water to that of the Kerosene Equation.

$$\text{Free swell} = \frac{V_w}{V_k \times 100} \dots\dots\dots (2.3)$$

Where

V_w =final volume in water

V_k = final volume in kerosene

The relation between the degree of expansion and the differential free swell ratio is given in Table 2.

Table 2.5: Classification of Soils based on the free swell ratio (Sridhar and Prakash 2004)

Free Swell Ratio	Soil Expansively	Clay Type
<1	Negligible	Non-Swell
1-1.5	Low	The mixture of non-swelling and swelling
1.5-2	Moderate	Swelling
2-4	High	High
>4	Very high	Swelling

E. Cation Exchange Capacity (CEC)

The CEC is the number of exchangeable cations required to balance the negative charge on the surface of the clay particles. CEC is expressed in mill equivalents per 100 grams of dry clay. CEC is related to clay mineralogy. High CEC values indicate a high surface activity. In general, swell potential increases as the CEC increases. Typical values of CEC for the three basic clay minerals are given in Table 2.4.

Table 2.6: Typical CEC values of basic clay minerals after Mitchell 1976 (Nelson & Miller, 1992)

Clay mineral	CEC(meq/100gm)
Kaolinite	3-15
Illite	10-40
Montmllonite	80-150

The three common types of clay mineral have different expansiveness property.

- Kaolinite => low degree of expansiveness
- Illite => moderate degree of expansiveness

- Montmorillonite => very high degree of expansiveness

Different experimental and empirical methods have been developed to determine swell potential of clayey soils and the measure of volumetric change in various soils on their interaction with water

Table 2.7: Soil Classification Based on Swell Potential, CEC and PI (Yilmaz 2004)

Soil type	cation exchange capacity (Yilmaz 2004)	Plasticity Index PI	Swell Potential (Seed et al., 1962)
Very High swelling	> 55	>35	> 25
High swelling	37 - 55	20-35	5 - 25
Medium swelling	27 - 37	10-25	1.5 - 5
Low swelling	< 27	0-15	< 1.5

The water-clay interaction nature of soil particles is surrounded by water the centers of positive and negative charges of water molecules do not coincide, the molecules behave like dipoles. The negative charge on the surface of the soil particle attracts the positive (hydrogen) end of the water molecules (Budhu, 2000).

2.2. Soil stabilization

Soil stabilization is defined as a chemical, physical, biological, mechanical, or combined technique that maintains or improves the stability of weak soils to achieve engineering goals. The pozzolanic reactions between the stabilizer (binder) and the soil particles improve the quality of the stabilized soft soil

Soil stabilization is a process whereby increased strength and stability of the soil are attained mainly by mechanical or chemical means. The most common improvements attained through stabilization include better soil gradation, reduction of plasticity index or swelling potential, and increase in durability and strength (DEP.T ARMY, 1994)

When unsuitable materials are encountered measures like avoiding the route redesigning the pavement with thicker sections or replacing the poor soil with good-quality materials are practical but increasingly expensive options. With improved technological advances and concern for the reduction of non-renewable resources, improving the properties of soil using chemical additives is gaining increased popularity (Caterpillar, 2006)

2.3. Uses of stabilization

The purposes of soil stabilization may include; increasing the shear strength of an existing ground condition to enhance its load-bearing capacity, achieve a desired improved permeability, and enhance the durability of the soil(Tesema, 2016). Commonly, improvement obtained from soil stabilization can be summarized as researcher; (Guyer, 2011) The purposes of soil stabilization may include; increasing the shear strength of an existing ground condition to enhance its load-bearing capacity, achieve a desired improved permeability, and enhance the durability of the soil(Tesema, 2016).

Quality improvement: the most common modification achieved through stabilization includes reduction of plasticity index or swelling potential and increases in durability and strength of the soil. The studied that stabilization process is also used to provide a working platform for construction operations in wet weather (Guyer, 2011)

Thickness reduction: the strength and stiffness of a soil layer can be improved using a stabilizer to allow a reduction in the design thickness of the stabilized material compared with a stabilized or unbound material. The design thickness can be reduced if the strength, stability, and durability requirement of a base or sub-base course can be decreased; suitability is indicated by further analysis and evaluation (Guyer, 2011)

Expansive soils cause several problems and difficulties for civil engineering infrastructure. a different approach is applied to improve the characteristics of expansive soils. Soil stabilization is one of the construction techniques because it improves the engineering properties of soil such as strength and stability. When the soil strength properties cannot be improved by mechanical stabilization, the chemical admixture technique is used to meet the desired strength. The researcher says Chemical stabilization is the most popular method used to enhance the physical and mechanical properties of problematic soils consisting of soft soil and expansive soil (Niken, et al., 2017)

2.4. Types of soil stabilization

The two most commonly used methods of stabilizing soils are Mechanical stabilization and chemical stabilization. Mechanical stabilization refers to the process of improving the stability and shear strength characteristics of the soil without altering the chemical properties of the soil (Modifying expansive soil by blending with non-expansive soil).

Mechanical stabilization is best suited for coarse-grained soils and clayey soils are more effective if chemically stabilized (Tesema, 2016)

2.4.1. Mechanical soil stabilization

Mechanical stabilization can be defined as a process of enhancing the stability and shear strength characteristics of the soil without altering its chemical properties of the soil. The major methods of mechanical stabilization can be classified into compaction, mixing or blending of two or more gradations, applying geo-reinforcement, and mechanical remediation (Al-chair et al., 2017)

2.4.2. Chemical stabilization

Method of improving the engineering properties of a material by adding chemical substances. Chemical stabilization is used for a wide range of purposes including improving the bearing capacity and strength of pavement layers, drying temporary bypasses during rainy periods, delaying certain chemical reactions that are harmful to road soils or aggregates, drying out where the moisture content is too high for successful compaction, make the soil less permeable soil where necessary, reduce the plasticity of soils used in road construction and thereby reducing the effect of moisture variations, changing clay to a more granular and workable material and reducing swelling and shrinkage (Gautrans., 2004)

(Dallas, 2000) Classify chemical stabilizers into three groups:

- Traditional stabilizers: such as hydrated lime, Portland cement, and Fly ash;
- Non-traditional stabilizers: comprised of suffocated oils, ammonium chloride, Enzymes, polymers, potassium compounds and
- By-product stabilizers: which include cement kiln dust, lime kiln dust, etc.

Over and above these, there are inorganic and hydraulic products that are often clustered into non-traditional soil stabilizers. Perhaps a better term for them would be 'hydraulic soil stabilizers' (Alex, 2010)

2.4.2.1. Chemical stabilization Process

Chemical additives are used as a stabilizer of expansive soils through a different process. Those processes which used to stabilize soils and modify their properties through cation exchange, flocculation, agglomeration, and pozzolanic reactions.

A. Cation Exchange

Negatively charged clay particles adsorb cations of a specific type and amount. The replacement or exchange of cations depends on several factors, primarily the valence of the cation. Higher valence cations easily replace lower valence cations. For ions of the same valence, the size of the hydrated ion becomes essential, the larger the ion, the greater the replacement tendency. If other conditions are kept constant, trivalent cations are held more tightly than divalent and divalent cations are held more tightly than monovalent cations (Mitchell & Soga, 2005). A typical replacement ability series is:

$\text{Na}^+ < \text{Li}^+ < \text{K}^+ < \text{Rb}^+ < \text{Cs}^+ < \text{Mg}^{2+} < \text{Ca}^{2+} < \text{Ba}^{2+} < \text{Cu}^{2+} < \text{Al}^{3+},$

The cation exchange includes an immediate reaction of the clay with the stabilizer in a few minutes of mixing, resulting in soil with improved texture

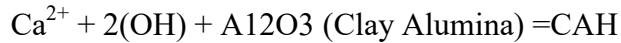
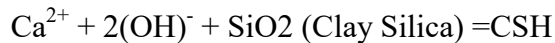
B. Flocculation and Agglomeration

Cation exchange reaction results in the flocculation and agglomeration of the soil particles with the consequent reduction in the amount of clay-sized materials and hence the soil surface area, which inevitably accounts for the reduction in plasticity. Flocculation and agglomeration change the clay texture from that of a plastic, fine-grained material to that of granular soil (Yazici, 2004). Flocculation is the process of clay particles altering their structure from a flat, parallel structure to a more random orientation. Agglomeration is thought to occur as the flocculated clay particles begin to form weak bonds at the edge surface interfaces of the clay particles, because of the deposition of cementitious material at the clay particle interfaces.

C. Pozzolanic Reactions

The pozzolanic reaction is a secondary process of soil stabilization. One prerequisite for the formation of additional cementing materials is the solution of silica and alumina from clay components. The high pH environment of a soil-cement system increases the solubility and reactivity of the silica and alumina present in clay particles. The degree of the crystallinity of the minerals and particle size distribution are some factors influencing solubility. It is postulated that calcium ions combine with silica and alumina dissolved from the clay lattice to form additional cementitious material (C-S-H and C-A-H) (Tasong, 1999).

Calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH) are the two outputs in pozzolanic reactions.



2.4.3. Lime Stabilization

Lime is one of the most popular additives agents used to improve fine-grained soils. Lime, alone or combined with other materials, can be used to treat a variety of soil types. When Soils are treated with lime the construction activity becomes facilitated in three ways. First, a reduction in the liquid limit and an increase in the plastic limit results in a significant reduction in the plasticity index. A decrease in the plasticity index enables higher workability of the treated soil. Second, as the result of the chemical reaction between soil and lime, a decrease in water content occurs. This facilitates the compaction of very wet soils. Additionally, lime addition raises the optimum moisture content but reduces maximum dry density, and finally, a direct increase in strength results in a stable platform that facilitates the mobility of equipment. (Meron, 2013) reasearcher describe a substantial proportion of their strength is retained when they become saturated with water Lime-stabilized material is suitable for use as a capping layer or working platform when the in situ material is excessively wet or weak and removal is not economical. ERA,(2002)

2.4.4. Cement Stabilization

Cement is one of the oldest stabilizing agents since the invention of soil stabilization technology in the 1860s. It could be considered the first stabilizing agent or hydraulic binder because it can be used alone to bring about the stabilizing action required (Sherwood, 1993). Cement reaction is not dependent on soil minerals, and the key role is its reaction with water that may be available in any soil (Euro Soil Stab, 2002). It can be the reason why cement is used to stabilize a wide variety of soils. Many types of cement are accessible in the market; these are ordinary Portland cement, blast furnace cement, sulfate-resistant cement, and high alumina cement. Generally, the choice of cement depends on the type of soil to be stabilized and wanted final strength

2.5. Characteristics of Expansive Soils

The engineering properties of a typical expansive soil (from Regina, Saskatchewan, Canada) were investigated under in situ and compacted conditions. For both sample types, the clay behavior was characterized by its internal structure comprising fissures and aggregates (Aslam et al., 2013). Soils are usually encountered as sub-grade material during any road construction project and these sub-grade soils may vary from highly expansive to non-expansive. Therefore, these sub-grade soils should be sufficiently investigated since the stability and performance of the pavements are greatly influenced by the nature of sub-grade soil as it serves as the foundation for pavements (Chen, 1988).

2.6. General Classification of expansive soil

There are different classification schemes, which use different basics for the purpose. Among these, the Unified Soil Classification System (USCS) and the American Association of State Highway and Transport Officials (AASHTO) method make use of index property for classification and are often used. Soils rated CL or CH by USCS, and A6 or A7 by AASHTO may be considered potentially expansive (Nelson & Miller, 1992).

Table 2.8: Particle size classification (USC) and (AASHTO) (Nelson and Miller, 1992)

Classification system	Grain size (mm)
Unified	Gravel: 75 mm to 4.75 mm Sand: 4.75 mm to 0.075 mm Silt and clay (Fines): < 0.075 mm
Classification system	Grain size (mm)
AASHTO	Gravel: 75 mm to 2.00 mm Sand: 2.00 mm to 0.05 mm Silt : 0.05 mm to 0.002 mm Clay: < 0.002 mm

2.6.1. AASHTO Classification System

According to AASHTO, Particle size analysis and plasticity characteristics are required to classify soil for both coarse-grained and fine-grained from A-1 to A-7 of soil classification.

This classification system required particle size analysis, plasticity index, and liquid limit should be determined to group the soil with similar engineering characteristics (Dr. Arora, 2004).

Table 2.9: AASHTO Soil Classification System (Nelson and Miller, 1992)

Soil Group		Grain size %passing			Liquid limit	Plasticity Index	Mineral type	Sub-grade Range
		#10 Sieve	#40 Sieve	#200 Sieve				
A-1	A-1-a	<50	<30	<15		<6	Stone, gravel sand	Excellent to good
	A-1-b		<50	<25		<6		
A-3			>51	<10		Nonplastic	Fine sand	
A-2	A-2-4			<35	<40	<10	Salty sand Clayey gravel and sand	
	A-2-5			<35	>41	<10		
	A-2-6			<35	<40	>11		
	A-2-7			<35	>41	>11		
A-4				>36	<40	<10	Silty soil	
A-5				>36	>41	<10	Silty soil	Fair to poor
A-6				>36	<40	>11	Clayey soil	
A-7	A-7-5			>36	>41	>11 PI<LL-30	Clayey soil	
	A-7-6			>36	>41	>11 PI>LL-30	Clayey soil	

2.6.2. USCS Classification System

The unified soil classification system (USCS) was first developed by Casagrande in 1948 and modified by the Bureau of reclamation and crop engineers of the USA (Kaminski, 2011). It has also been accepted by the American Society of Testing Materials (ASTM) and most popular classification system for all types of engineering problems involving soils. This method is used to categorize the soil with similar engineering properties, including strength, permeability, and compressibility which specify soil types to achieve a desired performance (Kaminski, 2011).

According to USCS, soils are classified as coarse-grained or fine-grained as follows:

❖ Coarse-grained when soil sample retained on sieve #200 (0.075 mm) more than 50 percent. Coarse-grained soils are further classified as:

Gravels if 50 percent or more of the coarse fraction is retained on the #4 (4.75 mm) sieve

Sands if 50 percent or more of the coarse fraction passes through the #4 (4.75 mm) sieve.

❖ Fine-grained if 50 percent or more of the sample passes the #200 (0.075 mm) sieve. Fine-grained soils are further classified according to whether their liquid limit is less than or greater than 50 percent.

Table 2.10: Unified Soil Classification System (ASTM D2487)

Major Group	Sub-Group	Symbol	Description
Coarse Grained soil (>50% retained #200 sieve)	Gravels (>50% coarse fraction retained on #4 sieve)	GW	Well-graded gravels and gravel-sand mixtures (little or no fines)
		GP	Poorly-graded gravels and gravel-sand mixtures (little or no fines)
		GM	gravels (gravel-sand-silt mixtures)
		GC	clayey gravels (gravel-sand-clay)
	Sands (>=50% coarse fraction pass through #4 sieve)	SW	Well-graded sands and gravelly-sands mixtures (little or no fines)
		SP	Poorly-graded sands and gravelly-sands mixtures (little or no fines)
		SM	Silty- sands (sand-silt mixtures)
		SC	clayey sands (sand-clay mixtures)
Fine-Grained soil (>=50% pass through #200 sieve)	Silts and Clays (with Liquid Limit <50%)	ML	Inorganic silt (very fine sands, silty or clayey sands)
		CL	Inorganic clays of low-to-medium
		OL	organic silts and silty-clay of low
	Silts and Clays (with Liquid Limit >=50%)	MH	Inorganic silts, elastic silts
		CH	Inorganic clay of high plasticity

		OH	Organic clay of medium-to-high
		PT	Peat muck and other organic soil

2.7. Stabilizing Agent For Expansive Soil

Stabilizing agents are manufactured commercial products, industrial or agricultural by-products that, when added to the soil in the proper quantities improve some engineering characteristics of the soil such as strength, texture, workability, and plasticity (Tesfaye, October 2001.). Their research work described the improvement of expansive soil by the addition of lime and cement on black cotton soil from different parts of Addis Ababa. Index properties, compaction characteristics, and swelling pressure of soil-cement and soil-lime were determined using the Atterberg limit test, moisture-density relations, free swell, and swelling pressure tests. The conclusions and findings drawn from the study are;

Expansive soil becomes moderately active to inactive based on the amount of lime and cement added.

The swelling pressure of expansive soil decreases with increasing lime, cement, and molding water content.

4-6% of lime and 9-12% of cement yielded significant improvement in plasticity and swelling properties of expansive soils.

(Nebro, 2002) this research work evaluated lime and liquid stabilizer called Con-Aid for stabilization of potentially expansive subgrade soil on samples collected from Addis-Jimma road which had indicated different pavement damages exacerbated by the presence of expansive soils. The experimental study involved the Atterberg limit test; moisture-density relation, UCS, CBR, and CBR swell. The findings and conclusions of the study can be summarized as follows:

2.8. Innovative Stabilizing Agent

The growing cost of traditional stabilizing agents and the need for the economical utilization of industrial and agricultural wastes for beneficial engineering purposes have enhanced or encouraged an investigation into the stabilizing potential of expansive clay soil. Thus, the possible use of agricultural waste, such as bagasse ash, maize cob ash,

rice husk ash (Rama, et al., 2016), groundnut ash (Sampan, 2007), and industrial wastes, such as molasses, and iron slag have considerably reduced the cost of construction and as well as reduce or eliminate the environmental hazards caused by such waste. These stabilizing agents are preferable due to the following reason:

- ❖ Due to the accessibility and production of innovative stabilizers such as industrial or agricultural wastes are far cheaper than common/traditional stabilizers
- ❖ Innovative stabilizers are environmentally friendly compared with common stabilizers
- ❖ Waste management from the huge factories also can be done economically by using a stabilizer

Therefore, the use of these innovative stabilizers for improving the engineering properties of the Soil is an economical solution for Ethiopia as it is available in large

2.9. Maize Cob Ash and Effect of Maize Cob Ash

Maize is one of the cereal crops which is grown widely throughout the world as well as in Ethiopia and is the largest crop produced yearly. According to the food and agriculture organization (FAO), data, 589 million tons of maize were produced worldwide in the year 2000 describe researcher (Taffessa A, et al., 2011). The United States was the largest maize producer having 43% of world production. Africa produced 7% of the world's maize; South Africa has the highest production of 8.04 million tons. Ethiopia was the fifth largest producer of maize in Africa in the year 2020 with 8600 thousand tons. The maize body consists of three natural parts; the chaff and pith covering and the wood ring which carries grains called maize cob. Maize cob is part of maize on which the grains grow and is often discarded as waste after the grains have been removed, it is the waste product obtained from maize or corn, which is disposed of as useless materials and affects the environment as landfill wastes. Maize cob ash (MCA) is cheaply available locally and is obtained through the burning of waste maize cob in the hole so not as to affect the environment and reduce pollution. Therefore, the use of maize cob ash as an additive for soil stabilization goes a long way in minimizing the cost of soil stabilization and it is an agricultural waste, which contains a high percentage of silica. Moreover, the sum of SiO_2 , Al_2O_3 , and Fe_2O_3 in

the chemical composition of MCA satisfies the requirements of ASTM C618, suggesting its pozzolanic nature.

(Akinwumi & Aidomojie, 2015) in their research, work described expansive soil stabilized with cement-corncob ash (CCA) to ascertain its suitability for use as a pavement layer material. They performed serious laboratory tests varying from zero to 12%, were carried out. The results showed that the addition of CCA to the soil generally reduced its plasticity, swell potential, and permeability; and increased its strength. CCA stabilization, aside from being more economical and environment-friendly than cement stabilization, improved the geotechnical properties of the soil for pavement layer material application. Thus was recommended for use in pavement layers.

(Reddy et al., 2018) had studied on stabilization of expansive soils by using corncob ash, the researcher used various percentages of maize cob ash (4%, 8%, 12%, 16%, 20%) by the weight of dry soil and investigated its effect on, OMC, MDD, CBR, free swell index and consistency of expansive soils. From the result of the investigator, the MDD increased and reached the maximum value of 12% of corncob content by the dry weight of the soil. The UCS and CBR of the soil were also increased and the maximum value was observed at 12% of maize cob ash. Finally, the researcher concluded that using maize cob ash as an additive decreases swelling and increases the strength of expansive soils.

(Sadeeq.J, et al., 2017) In their research work, are investigating the modification of lateritic soil using corncob ash. The researcher understood the effect of corncob ash on the Atterberg limits and compaction characteristics. The percentage of corncob ash used was 0%, 3%, 6%, and 9% by dry weight of natural soils. The researchers understand that the introduction of CCA into the soil increases the liquid limit and plasticity index and thereafter decreased. Increased Atterberg limit is due to the addition of CCA, which introduced more pozzolanic substance into the specimen and decreased due to the agglomeration and flocculation of the clay particles, which is because of exchange ions at the surface of the clay particles. The researchers also concluded that stabilization of lateritic soil with CCA showed a general increase in MDD and a decrease in OMC within increased CCA content.

(Osule, et al.,2017) in their research work, had studied on strength and workability improvement potential of the admixture of corncob ash and cement for stabilizing lateritic soil. The researcher has investigated, the effect of CCA stabilized with cement on the geotechnical properties of soils such as compaction, unconfined compressive strength, and California bearing ratio, by using (0%,3%,6%,9%,12%) and (0%,2.5%,5%) of CCA and cement content respectively. Finally, the researchers investigate that the MDD and OMC of stabilized soil samples increased with increasing the content of CCA and Cement except for 12%CCA at OMC were dropped. The peak MDD was obtained at 12% CCA and 5% cement and the OMC increased up to 9% and then decreased. Moreover, the UCS and CBR were increased with increasing content of maize cob ash and cement contents.

2.10. Waste Brick and Effect of Waste Brick:

Brick is one of the most common building materials, and it is also one of the largest components of waste generated from both construction and demolition. Reuse of this waste would reduce the environmental and social impacts of construction. One potential bulk use of such waste is as a cementing agent for soil stabilization. Burnt brick powder is a waste powder generated from the burning of bricks with the soil covered by surroundings. Due to burning of soil bricks it hardened and at the time of removal the set-up we get the powder form of brick. (Zhang, 2013).

The application of brick waste in the stabilization of soil provides good results when compared to other stabilizers like lime, fly ash, and cement. From an economic point of view, such an application provides a cheaper alternative to conventional materials. Environmentally, using a vast amount of brick waste in soil stabilization and other civil engineering applications is lead to greener construction material and helps in reducing waste in the environment. Solid waste produced from construction and demolition activities amounts to several million tons globally, and one of the prominent wastes is brick waste. Expansive soil treated with brick waste promising results exhibited in decreasing free swell index, liquid limit, plastic limit, plasticity index, soil shrinkage, and swelling potential. The positive reaction of brick waste is due to its effect as a filler material and as a pozzolanic material. It is reacting chemically with components of soil (Al-baidhani & Al-taie, 2019)

(Pardakhe & Ban, 2019) in their research, work described Soil Stabilization by using Waste Material - Brick Dust. The researcher uses various percentages of waste brick dust (10%, 20% & 30%) and the effect of OMC, MDD, and CBR, free swell index consistency of black cotton soil. From the result of investigators, moisture content decreases respectively in Black Cotton Soil. The liquid limit and plastic limit of the soil were decreased by adding waste brick dust to Black Cotton Soil. From the standard Proctor Test, they conclude that the maximum dry density increased and the Optimum moisture content decreases by adding waste brick dust.

(Takele, 2020) Their research justified or described, the effect of mixed gypsum, and crushed waste brick on weak subgrade soil, and the optimum mix improve the soil strength. the evaluation of the engineering properties of soil is improved due to mix gypsum and crushed waste brick. LL and PI decreased from 76.5% to 39.8% and 36.5% to 9%, MDD increased from 1.4 g/cm³ to 1.5 g/cm³ and CBR value increased from 1.5% to 10.7% as the content of gypsum and CBD increases from 0% to 6% G+30% CBD then decreased to as increased the stabilizer.

(Teja et al., 2018) in their research had described the potential of burnt brick dust as stabilizing additive to expansive soil is evaluated for the improving engineering properties of expansive soil. The evaluation involves the determination of the swelling potential, linear shrinkage, atter-berg's limits, & compaction test of expansive soil in its natural state as well as when mixed with varying proportions of burnt brick dust (from 30 to 50%) result shows a considerable reduction in swelling of expansive soil and Maximum dry density of soil is improving and optimum moisture content is decreasing with increasing stabilizing content. For increasing the content of stabilizing agent brick dust atter berg's limit values are also decreasing.

(Prof.A.panddurag, Prof.sharankumar, 2020) In their research, work described effective methods of improving soil properties. The present study is to strengthen, soft subgrade soil by using brick powder and corncob ash. Several tests were carried out on black cotton soil and replaced with brick powder and corn cob ash at various mixing proportions i.e., 25%, 35%, and 45% of brick powder keeping 5% of corn cob ash. results shows reduced swelling potential and improving soil strength.

2.11. Effect of Compaction and CBR characteristics

The effects of compaction energy on the engineering properties of an expansive soil stabilized with the optimum percentage of fly ash-granite dust have been discussed in this paper. Expansive soil stabilized with the optimum percentage of fly ash-granite dust was compacted with five compaction energy levels. Maximum dry density and optimum moisture content corresponding to each energy level were determined. Based on these maximum dry density and optimum moisture content, samples were prepared for unconfined compressive strength, soaked California bearing ratio, hydraulic conductivity, and swelling pressure tests. From the test results it is found that an increase in compaction energy increased the maximum dry density, unconfined compressive strength, soaked California bearing ratio and swelling pressure, and decreased the optimum moisture content(Sabot & Moharana, 2016)

Compaction energy or compaction effort plays an important role in the engineering properties of soil. The soil used in the test is High Plastic Clay or Expansive soil there is a gradual increase in maximum dry density of soil as compaction effort is increased. This increase is a result of the more congested packing of soil particles. The optimum moisture content of soil reduces as the compaction energy increases. This reduction is due to the decrease in void spaces available in the soil mass. As the soil, particles are packed more closely, so the voids available for accumulation of water are less. California bearing ratio (CBR) is a key parameter for the determination of behavior and performance of road subgrade and improvement in California bearing ratio of soil as the compaction energy increases(Khan, 2017)

Pavements are a conglomeration of materials. these materials, their associated properties, and their interactions determine the properties of the resultant pavement. thus, a good understanding of these materials, how they are characterized, and how they perform is fundamental to understanding pavement and the supporting soil beneath the pavement, and its special courses called subgrade. The compacted subgrade is the soil compacted by the controlled movement of heavy compactors. The performance of pavements depends to a large extent on the strength and stiffness of the subgrades. among the various methods of determining the strength of subgrade, the CBR test is very important(Yashas et al., 2016)

Table 2.11: CBR values and the quality of subgrades in pavement design

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

ERA,2002 Manual, and Bowles,1992 the researcher's interest are to use materials with CBRs less than 2 and 0-3 respectively, as a selected stabilization by a method of soil treatment for the construction of road pavement soil during locally available materials.

Table 2.12: CBR values subgrade strength class

Subgrade classes	CBR (%) Range
S1	0-3
S2	3-4
S3	5,6,7
S4	8-14
S5	15-30
S6	>30 ⁺

No allowance for CBRs below 3% has been made because, from both a technical and economic perspective, it would normally be inappropriate to lay a pavement on soils of such poor bearing capacity. Moreover, the measurement of the bearing strength of such soft soils is generally most uncertain and CBRs below 2% are of little significance.

ERA,2013 Manual, The researcher's interest are to use materials with CBRs less than 2 for subgrades with CBRs less than 2, special treatment is required and the CBR value for a samples were above 5% were good to use as good the pavement construction.

CHAPTER THREE

3. METHODOLOGY

3.1. Study Setting/ Area

The study was conducted in the Dawuro zone Tarcha zuriya woreda is one of the Woreda in the zone, found in southwestern Ethiopia. It is geographical location is $7^{\circ} 58'00''$ latitudes and $37^{\circ} 41'00''$ longitudes. Tarcha is the capital of the zone and is found 540 Km from Addis Ababa and 322 Km from the Regional capital southwestern region and 148 km from Jimma. The Woreda is divided into 11 rural Peasant Associations and 7 urban Kebele. The Woreda is classified as one of the surplus productive Woreda in maize. The town is found in an area of an altitude of 1400-2500m above sea level. It lies in the climatic zone locally known as Woyna Daga, and Kola which is considered ideal for agriculture as well as human settlement. In the tarcha zuriya woreda, the average annual temperature is 23°C , the average annual rainfall is 1159.1mm, and the terrain is mostly rolling (source SWERMA,2014)

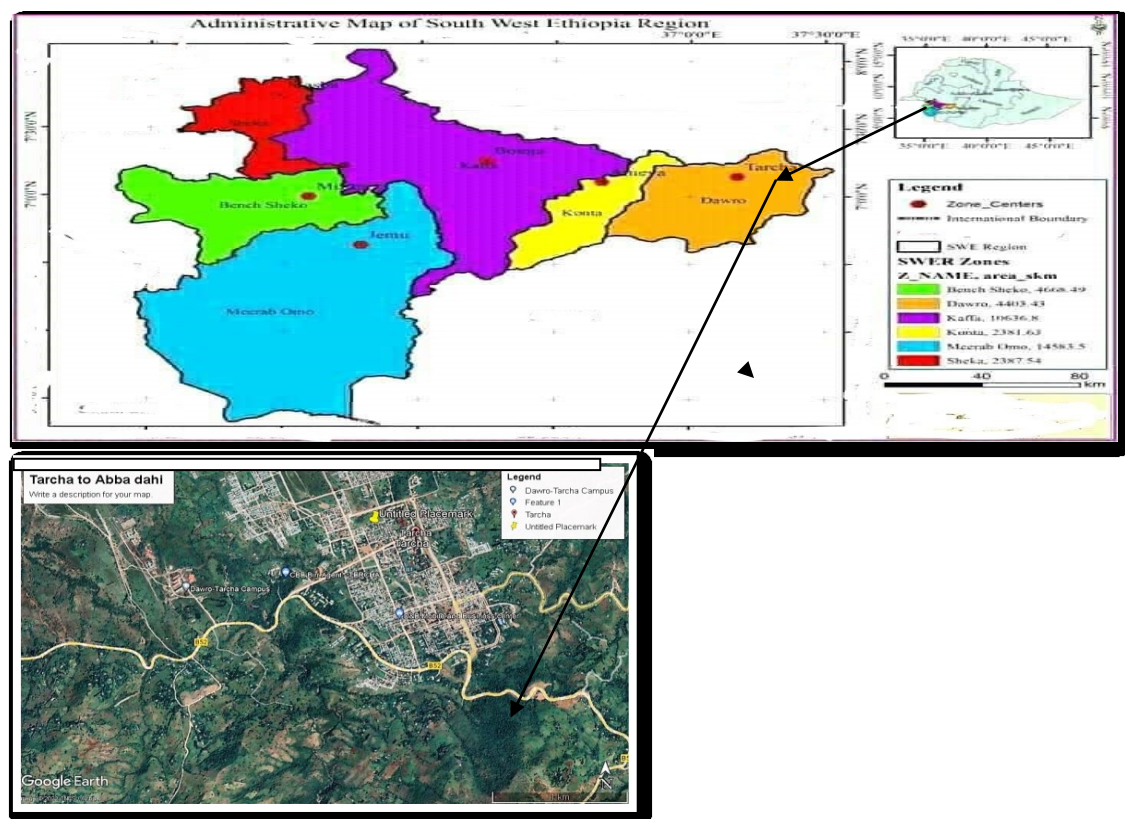


Figure 3.1: Map of the study area (source, Google Earth 2022)

3.2. Materials

The materials, which need to fulfill the research objective, are;

A Weak Subgrade soil.

Expansive soil samples used for this research work are collected from Tarcha zuriya woreda, wara woriya kebele, service safer, Gorka, and lala kebele. The soil is Dark gray-brown and dark black in color respectively. According to ERA, the depth of any pit should in no case be less than 1.5m unless rocks or other material impossible to excavate by hand is encountered. Due to the reference of ERA, the samples were disturbed and collected at a depth of below 1.5m. The full properties of the soil are addressed in the methodology section. a disturbed sample is collected from the test pit at a depth below 1.5m to avoid the inclusion of organic matter.

a.



b.



Figure 3.2:soil sample ,cracking soil(a), digging at 1.5m(b)(source:Matawork T 9/10/2022)

B. Maize cob ash

The maize cob used for this study was obtained from the Dawuro zone, southwest Regional state after the harvest season, the maize cobs were sun-dried. The collected samples were then burnt until they turned to ash by open burning on a metal sheet to protect the ash from the mixing of dust. The burnt maize cob was Grounded after cooling using a manual method and the burnt ash was sieved through 425 μ m. the fine ash was collected and stored in polythene bags to prevent pre-hydration from the atmosphere.

a.



b.



c.

Figure 3.3:maize cob ash, collecting waste(a,b), sun dried(c)(source:Matawork T 10/11/2022)

C.Waste Brick.

Waste brick used for this study from obtained Dawuro zone, Tarcha town with its component burnt brick is a waste brick powder generated from the burning of bricks with the soil covered by surroundings and demolished waste brick used for this study. Due to the burning of soil bricks hardened and at the time of removal of the set-up, we get the powder form of a brick. It has red in color and is fine. It has the great ability to reduce the swelling potential of natural soil

a.





Figure 3.4: Photos of waste bricks(a) and Demolished waste bricks(b)(source:Matawork T 2/11/2022)

3.4. Study design

The research followed the experimental type of study, which began collecting samples.

This study design/frame is the process that guides researchers on how to collect, analyze and interpret observations. Therefore, the objective of the research was achieved by following the methodology outlined (Figure 3.5)

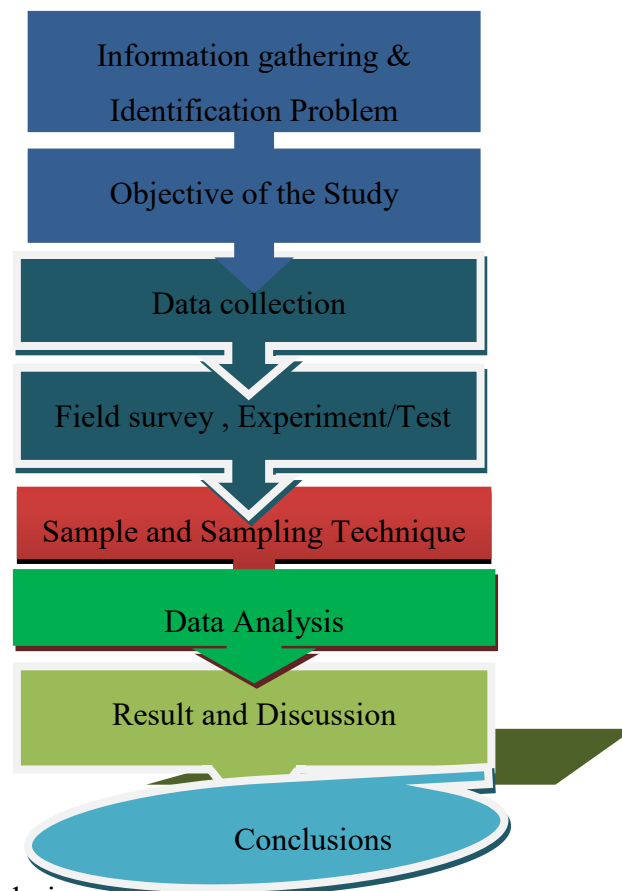


Figure 3.5: Research Frame design

3.5. Study Variables

The study variables are categorized into two. These are dependent variables and independent variables

3.5.1. Dependent variables

Properties of weak sub-grade soil

3.5.2. Independent variables

The independent variable for this research was soil Classification, moisture content, specific gravity, particle size distribution, free swell test, liquid limit, plastic limit, plasticity index, CBR, Compaction test, Dosage Waste Brick Powder, and Maize Cob Ash

3.6. Sampling techniques

The sampling techniques used for this research were purposive sampling. The experimental investigation of the study was executed particularly on the weak subgrade soil sample and experimental sample selection process. For the Free swell test, CBR, Atterberg limit, and Compaction laboratory tests, the samples were dependent on the types of test requirements and standards. The study was comparing the strength of normal sub-grade soils strength through laboratory tests. According to ERA and AASHTO, ASTM the subgrade soils need stabilization if the CBR values of natural subgrade soil samples did not fulfill the requirement of subgrade soils standard (CBR > 3%), and satisfy the standard specifications. Since this study pick out the samples concerning some criteria, which are considered important for the particular study. The testing tools and techniques were applied based on ERA and AASHTO /ASTM manual standards

3.7. Sample size

For this study, four soil samples from different locations, (Wara Wore, Lala Ginji, Gorka, and service safer) which can be well-represented weak soils found in Tarcha zuriya Woreda are purposively selected, and for each sample, the free swell test and attberg limit was conducted on the disturbed wet soil sample. Thus, from four samples, one soil sample (service safer) with high free swell and att-berg limit values was selected, and all tests are performed for it in both natural and stabilized conditions.

3.8. Data Collection Process

Field identification and laboratory tests were conducted to characterize the natural subgrade soil. The field investigation includes the identification of soil based on observation of color, crack, and soil texture. The primary tasks conducted before fieldwork are a literature review, collection of supplementary materials, such as geological and geographical maps of the study area, identifying the availability and functioning of laboratory and field soil sampling apparatus, and site selection.

3.9.1. Field Survey

The field survey was consisting only of looking for where the maize cob ash and brick productions and weak subgrade soil are available. Therefore, the survey helps to get information about the sources of data that the researcher was performing the laboratory tests.

3.9.2. Experimental or laboratory tests

The laboratory tests were conducted for the weak subgradesoil and soil-admixture combination; the tests were conducted according to the ASTM, ERA, and AASHTO standards and recorded in proper format the data becomes the input for the analytical analysis and the result were till as some outputs of the findings and these a stage were all tasks and activities that have been done at the laboratory to examine the effect of using maize cob ash and waste brick powder as weak subgrade soil stabilizing material was conducted. These stages have been categorized into two phases;

Phase One; all activities of preparing the sample to be tested such as air drying, proportioning or mixing of the sample with stabilizers, and Molding the sample for making it suitable for laboratory testing machines belong to the phase one subcategory.

Phase Two; includes laboratory works and procedures that are essential in understanding the properties, and behaviors of untreated and treated weak soils.

3.10. Test procedures

3.10.1. Methods of a Laboratory test procedure

Various laboratory tests are available to identify and study the properties of soils. There are different tests to identify the index and mechanical properties of the soil. These properties are vital to choosing the type of stabilizers as well as the property of stabilizers.

3.10.2. Sample Preparation

Before treatment and testing, the sample was prepared by the method described in AASHTO T87-86. This method involves:

Air drying of samples and/or oven drying at 60°C or less, and breaking up the soil aggregates rubber covered mallet. Then, sieve analysis is performed to separate the dried soils into two groups. The first group involves preparing uniform soil samples passing the #40 (4.25mm) sieve

for Atterberg limits, 2.00mm sieve for specific gravity, 40 (4.25mm) sieve for (free swell, free swell index, and free swell ratio test). And the other # 4 (4.75mm) for compaction and California bearing ratio tests. Then, soil and soil-maize cob ash, soil-waste brick powder, and soil-maize cob ash-waste brick powder are mixed manually to get a uniform mix ratio for each test.

3.10.3. Mixing procedure

The additive was first added to the pulverized, sieved, and air-dried soil sample and dry-mixed thoroughly

When Maize cob ash and Waste brick were applied in combination the soil sample was dry mixed with Maize Cob Ash first, and waste brick and maize cob ash with waste brick was added after that

Finally, wet mixing was done by sprinkling a measured amount of water followed by a thorough mixing until a uniform soil-additive matrix was obtained.



a.



b.



c.

Figure 3.6: sample mixing procedure, (MCA+WB) with soil(a), MCA with soil(b), and WB with soil(c)(source, Matawork T 18/11/2022)

3.10.4. Natural moisture content.

Change in moisture content is the most influential parameter that affects the property of soils. Moisture content is defined as the ratio expressed as a percentage of the mass of water to the mass of soil solids

Natural water content is used to express the consistency of clay soil in its natural state. Consistency is a term used to indicate the degree of firmness of cohesive soils. The consistency of natural cohesive soil deposits is expressed qualitatively by such terms as very soft, soft, stiff, very stiff, and hard. The physical properties of clays greatly differ at different water content.

The moisture content test is carried out in a laboratory as per the procedures of AASHTO T 265 or ASTM D 2216. The test is conducted under ASTM D2216.

3.10.5. Specific gravity.

Specific gravity is a measure of the heaviness of the material and is used in the determination of hydrometer analysis. It was determined by ASTM D 854 using a 50 ml density bottle, which is used for fine-grained soils. The disturbed samples were used for the determination of specific gravity.

Test procedure

The empty pycnometer was cleaned, dried, and weighed (W_1) and a small quantity of dry soil passing a 2 mm sieve was placed in it, the mass of the pycnometer and the dry soil (W_2) was weighed, and water was added up to half of the pycnometer and agitated for 15 minutes to remove air bubble, fill water and mass of pycnometer, soil, and water recorded (W_3). After all air bubbles have been removed, the pycnometer was filled with

de-aired water and weighed (W_4). Accordingly, the specific gravity (G_s) of the soil mass is determined from (Equation 3.1), and the average specific gravity of three trials taken and determined at 20°C using (Equation 3.2). Then after it is reported at a temperature of 20°C . The above procedures were repeated for other samples. Different soils have different specific gravity (Table 3.1).

$$G_s = \frac{(W_2 - W_1)}{((W_4 - W_1) - (W_3 - W_2))} \text{-----} 3.1$$

$$G_{s\ 20^\circ\text{C}} = K \times G_s \text{-----} 3.2$$

Where K is the coefficient of specific gravity at a temperature

Table 3.1: General Ranges of G_s for Various Soils (Das, 2002)

Soil Type	Range of G_s
Sand	2.63 - 2.67
Silt	2.65 - 2.7
Clay and silty clay or inorganic clay soil	2.67 - 2.9
Organic	less than 2

3.10.6. Atterberg limits (AASHTO T89/90/ASTM D 4318-98).

Plasticity indicates the materials' clay reactivity (active clay content) and affinity to water (moisture content), which will affect the variability in strength development and the possible need for increasing the stabilizer content. The series of tests for determining the boundaries known as Atterberg limits between the physical states of soil and each boundary or limit is defined by the water content that produces a specified consistency. The Swedish scientist, Atterberg, established the four states of soil consistency which are called the liquid, the plastic, the semi-solid, and the solid states. He also proposed a series of tests for determining the boundaries known as Atterberg limits between the physical states of soil. The physical and mechanical behavior of fine-grained soils is linked to four distinct states: – solid, semi-solid, plastic, and liquid in order of increasing water content.

A. Liquid limit

The test procedure conducted for the determination of liquid limit for both untreated and treated soil samples was in according with the standard test procedure of AASHTO T89, the test is performed by the following process

300 g of air-dried soil sample passing through sieve no 40 (aperture $425\mu\text{m}$) was obtained and soaked at least for 16 hr, to ensure that the soil grains had absorbed water and softened through

Then the mixed soil paste was placed on the Casagrande cup and grooved with a standard grooving tool

Then the cup is lifted and dropped by turning the crank until the two parts of the soil come into contact at the bottom of the groove.

The moisture content at which the soil suspension passes from no strength to a very small strength. The handle is next turned at a rate of about two revolutions per second and the number of blows necessary to close the groove along the bottom for a distance of 13 mm is counted. The water content of the soil taken near the closed groove is found, the water content of the soil in the cup is altered and the tests are repeated.

At least three tests should be carried out by adjusting the water contents in such a way that the number of blows required to close the groove may fall within the range of 10 to 40 blows.

The final flow curve plot water content corresponding to 25 blows is termed as liquid limit



a.



b.



Figure 3.7: Liquid limit test, Apparatus arrangements(a), grooving soil with standard(b), blows(c)(source: matawork T 2/2/2023)

B. Plastic limit

The plastic limit test was conducted according to the standard test procedure of AASHTO T90, and the test is performed by the following process.

A portion of the natural soil and the soil with maize cob ash, waste brick, and maize cob ash with waste brick used for the liquid limit test is retained for the determination of the plastic limit.

The plastic limit is used to determine the lowest moisture content at which the soil behaves plastically. It is carried out only on the soil fraction passing #40 (0.425 mm) sieve and is usually performed in conjunction with the liquid limit test and the moisture content at which the sample when it is rolled into a thread, starts to crumble rather than distort plastically. The soil is rolled on a glass plate with the hand until it is about 3 mm in diameter and this procedure of mixing and rolling is repeated till the soil shows signs of plastic limit.

C. PLASTIC INDEX (PI)

This plastic index provides a measure of soil plasticity, which is the amount of water that must be added to change soil from its plastic limit to its liquid limit. The plasticity index is defined as:

$$PI = LL - PL \text{ -----(3.3)}$$

The plastic index is used frequently to predict the soil's relative ability to undergo volume changes (if expansive soil or not).

3.10.7. Particle Size Distribution

The test includes the determination of the particle size distribution in the natural soil. The tests are conducted in accordance with (ASTM D422-63, 1998) (Standard Practice for Particle-Size Analysis of Soils) testing procedures.

Approximately, 1000gm of dry natural soil sample is put in a tray and soak it with water treated with a dispersing agent for 18 hours. Transfer the slurry into a 4.75mm sieve and wash it with a jet of water, then Dry it in an oven, Transfer the material passing through 4.75mm to a 75- μ m sieve, and wash it with a jet of water. Collect the material retained on the the75- μ m sieve and dry it in an oven, take the material passed the 75- μ m sieve for hydrometer analysis then performed it to measure the amount of silt and clay size particles. Finally, determine the percentage of sand, silt, and clay-sized particles in the specimens.

3.10.8. Soil Classification (AASHTO-145)

A. Wet sieve analysis

A wet sieve analysis test was carried out for the natural soil sample, and the test is conducted by the following procedure

- 1000gm of the natural soil is taken and washed on a sieve size of 75 μ m
- After removed the soil sample, the retained soil on a sieve size of 75 μ m was taken and oven-dried over the night
- Recording the weight of the dried soil sample
- Cleaning all the sieves and assembling them in the ascending order of sieve numbers (#4 sieves at the top and #200 sieves at the bottom) Place the pan below #200 sieves. Carefully pour the dried soil sample into the top sieve and place the cap over it.
- Place the sieve stack in the mechanical shaker and shake it, after 10 minutes Remove the stack from the shaker and finally record the weight of the soil retained on each sieve.



a.

b.

Figure 3.8: photos of soil classification(source :matawork T 2/4/2023)

B. Classification of soils based on the Unified Soil Classification System (USCS)

The bases for the USCS is that coarse-grained soils can be classified according to their grain size distribution, whereas the engineering behavior of fine-grained soil is primarily related to their plasticity.

Coarse-grained soils are those having 50% or more materials retained on sieve No 200. Fine-grained soils are those having more than 50% passing through sieve No 200. USCS uses symbols for the particle size groups. These symbols and their representations are G-gravel, S-sand, M-silt, and C-clay. These are combined with other symbols expressing gradation characteristics. „W“ for well-graded „P „ for poorly graded and plasticity characteristics „H“ for high plastic and „L“ for low plastic and the symbol „O“ indicates the presence of organic material.

The information provided in the plasticity chart is of great value and is the basis for the classification of fine-grained soils in the Unified Soil Classification System. The plasticity index, the numerical difference between liquid limit and plastic limit, represents the range in water content through which soil is in the plastic state. A high numerical value of the plasticity index is an indication of the presence of a high percentage of clay in the soil sample. This implies that the plasticity values increase with the responding increase in clay content. The important feature of this chart is the empirical A-line that is given by the equation $PI = 0.73(LL - 20)$. An A-line separates the inorganic clays from the inorganic silts. Inorganic clay values lie above the A-line, and values for inorganic silts lie below the A-line. Organic silts plot in the same region

below the A-line and with LL ranging from 30 to 50 as the inorganic silts of medium compressibility. Organic clays plot in the same region as inorganic silts of high compressibility but below the A-line and LL greater than 50. A line called the U-line lies above the A-line. The U-line is approximately the upper limit of the relationship of the plasticity index to the liquid limit for any currently known soil. The equation for the U-line can be given as $PI = 0.91(LL - 8)$ (Das, B.M., 1997)

C. Classification of soils using the AASHTO classification system

The AASHTO classification system, also called public roads administration (ERA) classification, is based on grain size distribution, liquid limit, and plasticity index. This system is generally used by highway engineers for the classification of sub-grade soils for highway pavement. According to this system, the soil is classified into seven major groups, A-1 through A-7. Group A-1, A-2, and A-3 are granular materials with 35% or less passing through a No.200 sieve. Soils with more than 35% passing through a No.200 sieve are classified under groups A-4, A-5, A-6, and A-7. These soils are mostly silt and clay-type material. Under this classification system, a characteristic called group index (G.I) is used to describe the performance of soil when used as a highway subgrade material.

$$GI = (F-35) [0.2 + 0.005(LL-40)] + 0.01(F-15)(PI-10) \dots\dots\dots (3.4)$$

Where

F=percentage by mass passing American sieve no 200(size 0.075mm), expressed as a whole number

LL=liquid limit (%), expressed as a whole number P I=plasticity index (%), expressed as a whole number

While calculating GI from the above equation, if any term in parenthesis becomes negative, it is dropped, and not given a negative value. The maximum values of (F-35) and (F-15) are taken as 40 and that of (LL-40) and (PI-10) Murthy, (1994). The method of classification used in this study was the AASHTO M-145 System

3.10.11. Free Swell test

Before sampling, visual site investigations and information from residents and construction firms were collected to consider the different soil types and to take representative samples evenly in the tarcha zuriya woreda surrounding area.

Accordingly, four sampling areas were selected from different places in Tarcha zuriya soil where selected and to identify whether it is highly expansive or not, free swell tests were conducted on each sample by following the methods of Holtz and Gibbs (1996). which are supposed to represent types of the expansive soils found in the Tarcha zuriya woreda from preliminary investigation tests.

Free swell < 50%, Not Expansive

Free swell between 50-100%, Marginal

Free swell > 100, Expansive

Then the free swell values of four samples from a different place in Tarcha zuriya woreda soil and their classification according to U.S.B.R methods are shown in the following table

A. Free Swell Index

The test includes the determination of the free swell index of the natural soil and the soil-maize cob ash, soil waste brick, and soil-maize cob ash-waste brick mixture. The tests are conducted in accordance with IS 2720 (Part 40) 1977 testing procedure. Samples of oven-dried soil 10cc each, passing through a 425-micron sieve are taken. One is put in a 100 cubic centimeters graduated glass cylinder containing kerosene. The other sample is put in a similar cylinder containing distilled water. Both the samples are left undisturbed for 24 hours and then their volumes are noted. The free swell index is determined using Equation (2.2).

B. Free Swell Ratio Test

In this study, the recommended procedure of Sridhar and Prakash is adopted. 10gm oven-dried soil passing through 425 microns is added to 100ml of distilled water in a jar and another 10gm of the same sample is added to 100ml of Kerosene. After 24 hours, sediment volumes of samples are measured to determine the free swell ratio. The free swell ratio is the ratio of the change in volume in water to the change in volume in kerosene after 24 hours. The free swell ratio is determined using Equation (2.3).

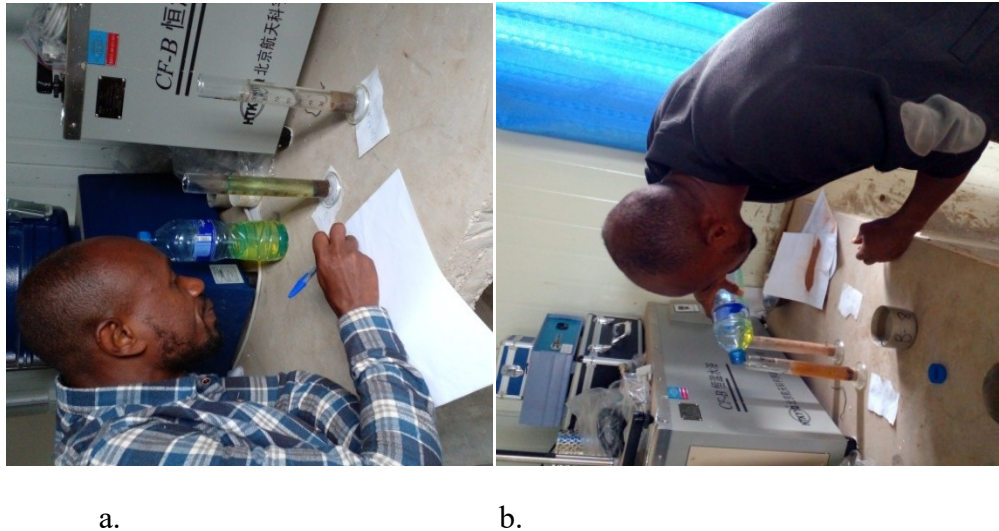


Figure 3.9: Free swelling test, samples before peak swell(a), sample after swelling(b)

3.10.10. Compaction (AASHTO T180/ASTM D1557)

The determination of the relationship between water content and density of soils is used in determining the compaction of the material. The purpose of compaction is to arrange the particles in such a way as to achieve the highest possible density for the layer with minimum voids.

The analysis was carried out for natural and stabilized soil; the test is conducted by AASHTO 180 testing procedures

- 5 kg of air-dried soil samples passing through sieve size #'s were obtained (for stabilized soil the percentage of MCA, WB, and Addition of WB and MCA are converted into kg and subtracted from the weight of soil)
- Mixing the prepared soil sample by varying the different amounts of water
- The mixed sample is then compacted into the 2124 cubic centimeters mold (of mass m1); in five layers of approximately equal mass with each layer receiving 56 blows. The blows are uniformly distributed over the surface of each layer
- The compacted sample is leveled off at the top of the mold with a straight edge.
- At least four small representative samples are then taken from the compacted soil for the determination of moisture content (W)

The test includes the determination of the maximum dry density and the optimum moisture content for the natural soil and the soil-maize cob ash, soil-waste brick, and soil-maize cob ash mixed with waste brick mixture. laboratory compaction test, the soil at a known water content is placed in a specified manner in a mold of given dimensions

and subjected to a compaction effort of controlled magnitude after which the resulting unit weight of the soil is determined. The procedure is repeated at various water contents until a relation between water content and unit weight of the soil is established and elastic modules improved but also the ingress of water is reduced or eliminated.



Figure 3.10: photos of the compaction test(source:Matawork T 2/12/2022)

3.10.12. CBR test (AASHTO-193/ASIM D-1883)

Is the most common strength test conducted on soils and other unbounded road-making to measure the resistance to penetration of plunger materials? It is developed to measure the resistance of a material to penetration of a standard plunger under controlled density and moisture content. The specimen will be compacted in a standard CBR mold at OMC. The sample mold will be soaked in water with a surcharge load for four days and then a load will be applied with a plunger to take penetration record. A three-point CBR test at 10, 30, and 65 blows was conducted according to AASHTO T193, and the CBR values at 95% MDD were determined. The CBR test in this study is conducted by taking air-dried samples that pass through sieve No.4 (4.75mm) for naturally expansive soil, different percent of waste brick (10%, 20%, 30%&40%), different percent maize cob ash (5%, 10%, 15%. &20%),) and different percent of a combination of waste brick and maize cob ash (5%, 10%, 15%,20% blend with 40%, 30%, 20%,10%) their opposite direction, added to natural soil then compacted with their corresponding density and Optimum Moisture content as per AASHTO 180 standard. For each sample take 5kg of natural soil only and the mixture of soil with different percent-off additives

at their respective optimum moisture content compacted by 56 blows for five layers. The compacted samples of the CBR mold are soaked for 96 hours in a water bath to the anticipated worst case on the field.

The CBR test measures the shearing resistance of soil under controlled moisture and density conditions. This value is computed by taking the ratio of load required at the depth of penetration (2.54mm and 5.08mm) into a soil specimen compacted at a given OMC and MDD corresponds to the load required to obtain the same depth of penetration on a standard sample of crushed stone.

Finally, the CBR value will be reported with the percentage

Standard of CBR Determination

$$\text{CBR at 2.54} = \frac{\text{load @2.54}}{\text{standard (13.2KN)}} * 100 \text{-----(3.5)}$$

$$\text{CBR at 5.08} = \frac{\text{load @5.08}}{\text{standard (20KN)}} * 100 \text{-----(3.6)}$$

$$\text{CBR} = \frac{\text{Corrected load value}}{\text{Standard load}} * 100 \text{-----(3.7)}$$

CBR Swell: - Road pavement structural design is usually based on 4-days soaked CBR values, to simulate the anticipated “worst-case” soil condition on the field. It was measured by placing the tripod with the dial indicator on the top of the soaked CBR mold. The compacted soil samples of the CBR mold are soaked for 96 hours in a water bath to get the CBR swell of the soil. The initial dial reading of the soil of the dial indicator on the soaked CBR of mold is taken just after soaking the sample. At the end of 96 hours, the final dial reading of the dial indicator is taken hence the swell percentage of the initial sample length is 116.43mm.

At the end of 96 hours, make a final daily reading on the soaked specimens and calculate the swell as percentages of the initial sample length.

$$\text{CBR Swell}\% = \frac{\text{change in length in mm during soaking}}{116.43\text{mm}} * 100 \text{-----(3.8)}$$

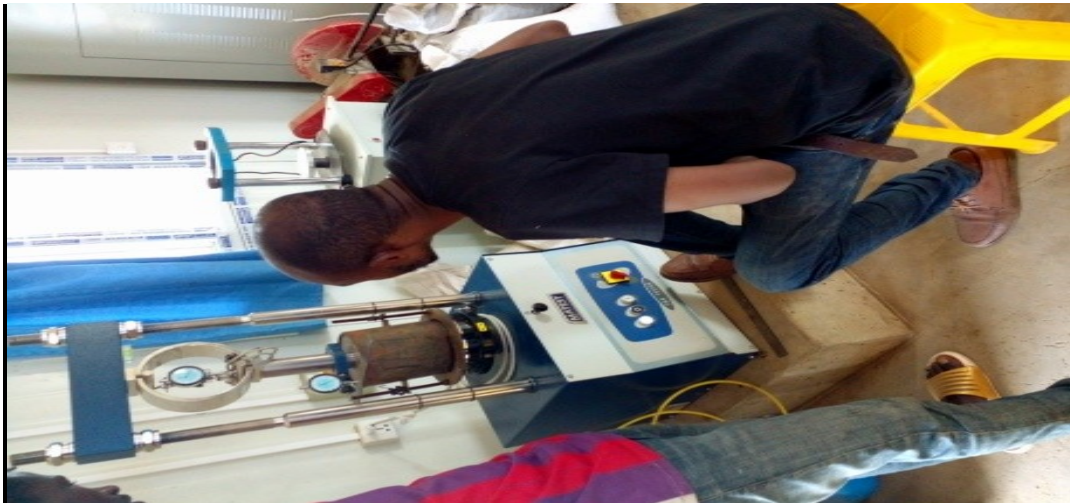
or

$$\text{CBR Swell}\% = \frac{\text{Measure the length before soaked} - \text{Measure the length after soaked}}{116.43\text{mm}} * 100 \text{--(3.9)}$$

a.



b.



c.

Figure 3.11: CBR test ,Sample soaking (a),swell reading(b),penetration reading(c)(source:matawork T 6/12/2022)

CHAPTER FOUR

4. RESULTS AND DISCUSSIONS

4.1. Introduction

Engineering properties of weak subgrade soil investigation involved the evaluation of both natural and stabilized soil samples separately by performing the following tests; soil classification, specific gravity, Atterberg limits (Liquid limit, plastic limit, plasticity index) conducted from 24hr cured sample, free swell test, free swell index, free swell ratio, moisture density relationship (compaction), take 4 days cured sample for CBR and CBR swell and the summary of the results of the test showed that of maize cob ash, waste brick powder, and maize cob ash mixed with waste brick powder improved the engineering properties of weak subgrade soil in general.

4.2. All Properties of Natural Soils

The laboratory test results conducted for the determination of properties of the natural soil before mixing with maize cob ash and maize cob ash with waste brick powder are presented in Table 4.1. For this study, test soil sample 3 was taken for treatment because it is weaker than test samples 1,2, and 4. Generally, for test pits - 3, the soil is dark black in color, as shown in Figure 4.1, on the grain analysis distribution curve almost 96.1% of the soil is passing through No. 200 sieves; it exhibits a liquid limit of 88.4, a plastic limit of 32%, and plasticity index of 56.4%. According to Daksan and Rama (1973), the Liquid limit of less than 35% is low, between 35% and 50% *medium*, between 50% and 70% *high*, and greater than 70% *very high*. Therefore, the value for the soil under consideration is 88.4%, which is *very high*. Accordingly, the soil falls under the A-7-5 soil class based on AASHTO, and based on the USCS soil classification system the soil is CH (high plastic clay). Soils under this class are generally classified as a material of poor engineering property to be used as subgrade material. The results of swelling characteristics also indicate that the soil is highly expansive clay with a free swell of about 102.5%, a free swell index of 84.2, and a free swell ratio of 1.842. The soil has a maximum dry density of 1.381 g/cm³, and optimum moisture content of 28.2% the soil also possesses, a soaked CBR of 1.0%, and a CBR swell of 7.85% respectively.

Table 4.1: Geotechnical Properties of untreated soil of test pit-3

Parameter	Laboratory Result	
Percentage of Passing through No.200	96.1%	
Sand	3.9	
Silty	35.2	
Clay	60.9	
LL	88.4 %	
PL	32 %	
PI	56.4 %	
Natural moisture content	36.173 %	
Soil classification	USCS	CH
	AASHTO	A-7-5 (64)
Free swell	102.5 %	
Free swell index	84.2 %	
Free swell ratio	1.842%	
Specific gravity	2.736	
Compaction	MDD	1.381 g/cm ³
	OMC (%)	28.2 %
CBR	1%	
CBR Swell	7.85%	

4.2.1. Natural Moisture content.

Table 4.2:Natural Moisture content.

S/No	Sample	Moisture content (w%)
1	S1	36.173

Those soils have high moisture content. They contain nearly equal water and soil.

4.2.2. Grain size analysis

Sieve Analysis was carried out to determine the percentage of different grain sizes contained within a soil sample and used in the classification of the soil.

Accordingly, the Wet sieve analysis was employed to determine the Grain Size Distribution of soil samples in accordance with the ASTM D422 Test Method for Particle-Size Analysis of Soils. In addition, Hydrometer Analysis was conducted for Samples to know the silt and clay fractions. The lab result for the Wet sieve and hydrometer test shows that the sample sub-grade soil contains (3.9%) sand, (35.2%) silt particles and (60.92%) clay particles. The Grain Size Distribution for the soil Samples were presented in Figure 4.1

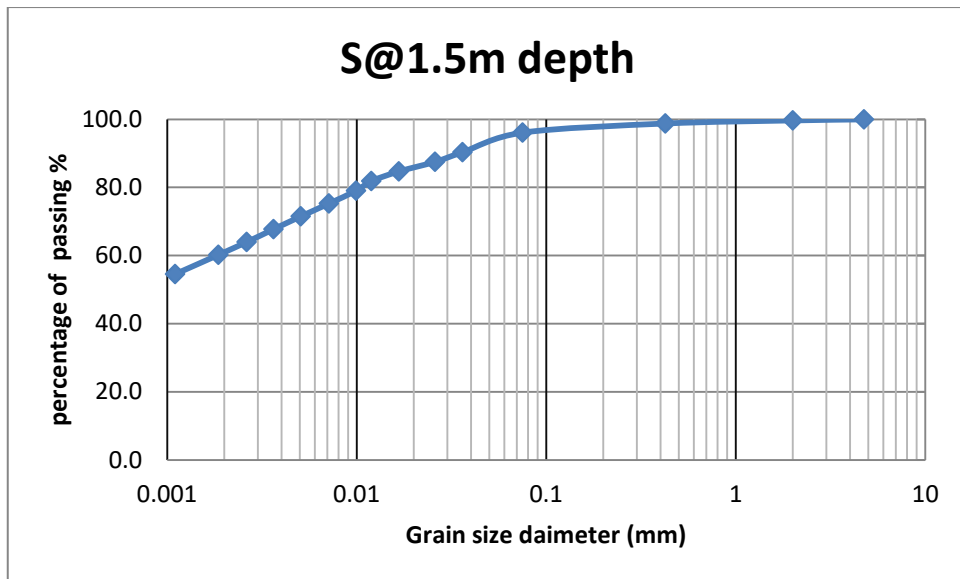


Figure 4.1:-Particle Size Distribution curve of sample

4.2.3. Soil classification

After completion Atterberg limit and sieve analysis, the sample soil was classified as CH as per the USCS system in Table 4.5 and Figure 4.2, and A-7-5 as per the AASHTO classification system in Table 4.6 and Figure 4.3. This indicated that the sub-grade soil was clay, a highly expansive material, Weak sub-grade and it required treatment to be used as a sub-grade material. Accordingly, the soils fall under the A-7-5 soil class based on AASHTO soil classification system and CH soil class based on USCS soil classification system. Soils under this class are generally classified as a material of poor engineering property to be used as a sub-grade material. Therefore, the soil requires initial modification and/or stabilization to improve its workability and engineering property

Table 4.3: Classification of soils based on the USCS classification system

Sample location	Sample depth	Amount of grain size			LL %	PL	PI%	USCS
		sand	Silty	clay				
S	1.5m below	3.9	35.2	60.9	88.4	32	56.4	CH

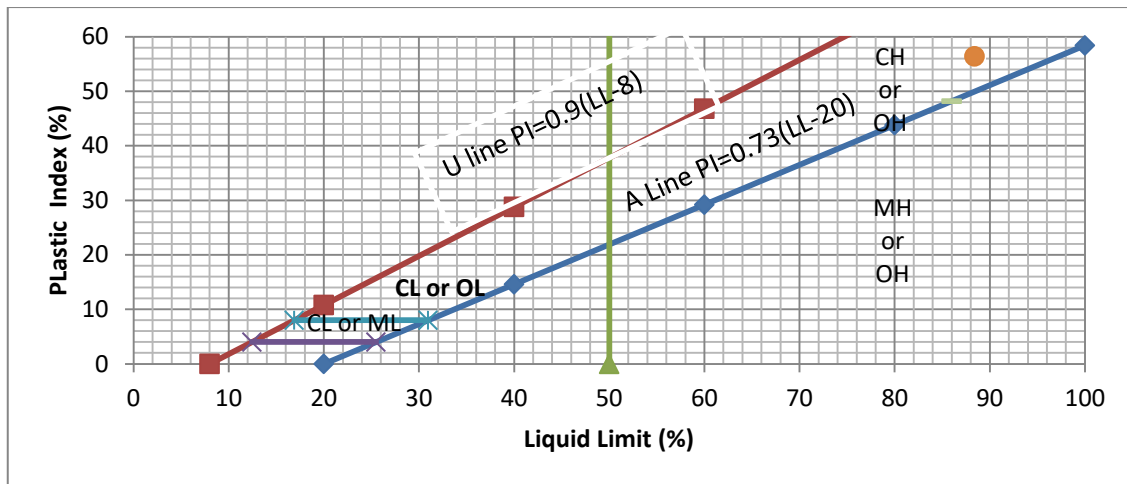


Figure 4.2: Plasticity chart of natural soil samples according to USCS

Table 4.4: Classification of soils based on the AASHTO classification system

Sample location	%pass No.200(75μm)	LL %	PL%	PI%	GI	Soil Group	Material type
S	96.1	88.4		56.4	64	A-7-5	clay

The soils sampled are classified as A-7-5(64) according to AASHTO soil classification and CH for sample according to USCS.

Percentage pass 75μm is 96.1% > 35%; PL=32; LL=88.4; PI=LL-PL=>88.4-32=56.4 when we compare this $PI \leq LL - 30$ this means $56.4 < 58.4$ and $PL > 30$ (i.e. $32 > 30$) from this the soil type is **A-7-5**.

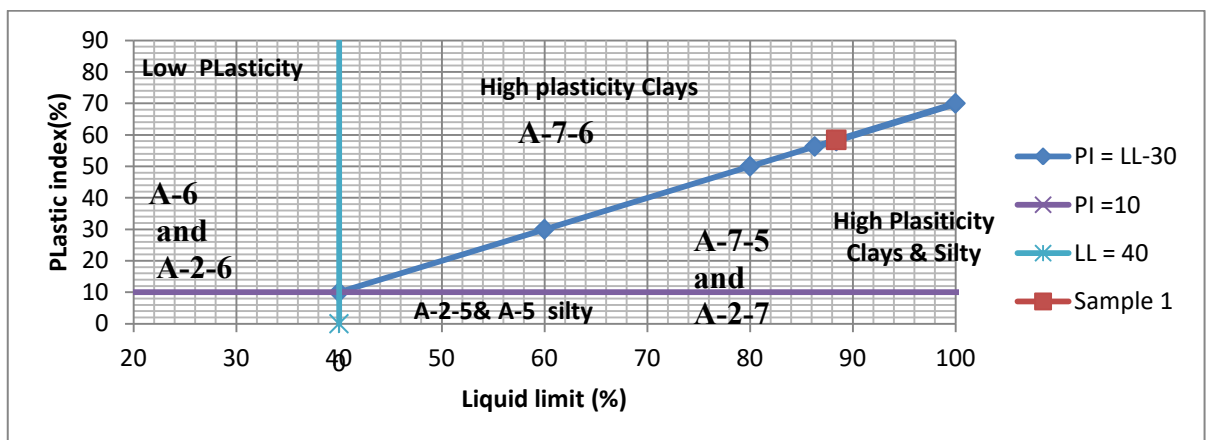


Figure 4.3: Plasticity chart of Natural soil samples according to AASHTO

4.2.4. The specific gravity of untreated soil.

Specific gravity is used to calculate parameters such as void ratio, porosity, and degree of saturation. It is the ratio of the mass of a unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. The specific gravity test results of the Sample indicate 2.739 respectively). The soil is inorganic soil according to Das, foundation analysis and design and Arora, soil mechanics, and silty clay and clay's specific gravity range is 2.67 to 2.75 and 2.68 to 2.90 respectively. The test results summary is shown in Appendix A 0.3

4.2.5. Free swelling characteristics of Untreated soil.

Free swelling tests (free swell, free swell index, and free swell ratio) are simple laboratory tests as discussed in chapter two, that are used to determine the expansion and swelling nature of weak soil as well as after the addition of MCA and WB stabilizers.

Table 4.5: Free Swell Test Result of Untreated Soil

Depth	Vi		Vf		FS	FSI	FSR
	water	kerosene	water	kerosene			
1.5m	10	10	20.25	11.2	102.5	84.2	1.842

The conducted test result showed above the Free Swell indicated that the soil Samples have problems with civil Engineering structures built on them Means a Free Swell of the sample is 102.5 and high expansive soil.

4.2.6. Compaction test of Untreated Soil

The Dry density and Optimum moisture content were determined by the modified proctor test as per AASHTO T-180. MDD and OMC for the sample of natural expansive soil sample were 1.381, and 28.2% as shown in Figure 4.4 below. The optimum moisture content is the moisture content corresponding to the maximum dry density of soils obtained from the compaction curve. The optimum moisture content obtained from this compaction test is used as input data to prepare the CBR specimen to be tested for the soaked condition CBR determination.

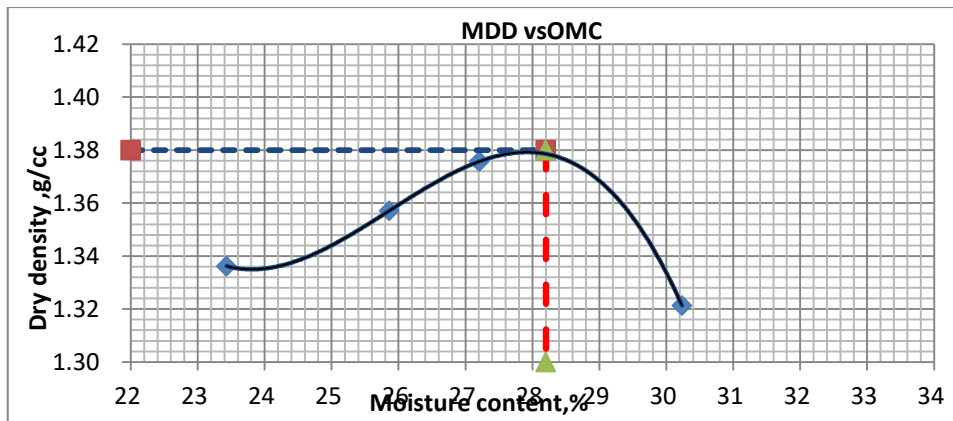


Figure 4.4: Compaction Characteristics of weak subgrade Soil For Samples

4.2.7. California Bearing Ratio (CBR) And CBR Swell of Untreated soil

The CBR tests were conducted according to AASTO T193. The CBR test samples were prepared with optimum moisture and maximum dry density with CBR mold for natural soil as well as stabilizer-soil mix.

Samples were soaked for 96 hours, and CBR and CBR swell was determined as per ERA& ASTM standards. The sample had CBR and CBR swell (1%, and 7.85%) respectively. Based on CBR value, the material was classified as poor materials. The test result showed that the natural soil has a low CBR value. This does not satisfy the minimum requirements as subgrade material. According to ERA standard specification when the soil CBR value of less than 3% is found, there should be special treatment. CBR test result for natural soil is shown below.

Table 4.6: CBR Result of Untreated soil.

Depth	CBR (%)	CBR swell (%)	ERA Requirement	Remark
S@1.5m	1	7.85	< 3%	Not satisfy

4.3. Chemical properties of maize cob ash (MCA)

Previews researchers Nigesu.G studies chemical compositions of Maize Cob Ash Using X-ray Fluorescence analysis is as test result indicates the combined percent composition of main oxides (SiO₂, Al₂O₃ & Fe₂O₃) was 73.75% which is above the minimum of (70%) specified by ASTM (C618) which is acceptable as a good Pozzolana is presented in the below table

Table 4.7: Chemical composition of MCA

Oxide composition	Test Result in (%)	ASTM(C618) requirement in	Remark
-------------------	--------------------	---------------------------	--------

		(%)	
SiO ₂	65.09	35.2min	Satisfied
Al ₂ O ₃	5.84		
Fe ₂ O ₃	2.82		
CaO	1.67		
MgO	0.65	5 max	Satisfied
SiO ₂ + Al ₂ O ₃ + Fe ₂ O ₃	73.5%	70 min	Satisfied

The chemical composition of maize cob ash test result indicates the combined percent composition of main oxides (SiO₂ + Al₂O₃ + Fe₂O₃) was 73.75% which is above the minimum of (70%) specified by ASTM (C618). which is acceptable as a good Pozzolana with specific gravity 2.29.

4.4. The laboratory test result of Expansive soil treated with MCA and WB

The effect of maize cob ash and waste brick on the properties of the expansive soil was also evaluated by conducting, Atterberg limits, free swelling tests, Modified compaction tests, and CBR tests.

4.4.1. Atterberg limits test results of weak subgrade treated by MCA and WB

The Liquid limit and Plastic index of soil indicate the water amounts certain in the materials of expansive behavior of soil that was being observed and to evaluate the changes of liquid limits, plastic limits, and plasticity index with the addition of Maize cob ash and waste brick with to the selected soil samples. To achieve this objective, liquid limit, and plastic limit tests were conducted on maize cob ash and waste brick-soil mixtures according to the consistency test of AASHTO T89 and T90, respectively. Details of the Atterberg limit test results are shown in Appendix A upto A4.4

Table 4.8: Atterberg limit test results of MCA-WB to treat Weak soils

S/NO	MCA (%)	WB (%)	LL (%)	PL (%)	PI (%)	PI (%) decrease	ERA (2013) Requirement of PI in (%)	Remark
1	0	0	88.4	32	56.4	0		Not
2	5	0	76	36	40	29.07	< 30%	Not
3	10	0	70	37	33	41.5		Not
4	15	0	64	39.5	24.5	57		Satisfied
5	20	0	60	40	20	64.46		Satisfied

6	0	10	78	33	45	20.21	< 30%	Not
7	0	20	72.4	35.2	37	34.39		Not
8	0	30	68	41	27	52.12		Satisfied
9	0	40	62.5	40.1	22.4	60.3		Satisfied
10	5	40	63	43.7	19.3	65.8	< 30%	Satisfied
11	10	30	58	44	14	75.177		Satisfied
12	15	20	59	41	18	68.08		Satisfied
13	20	10	63.7	43	21	62.76		Satisfied

4.4.1.1. Effect Of MCA On Atter-Bergs Limits

The Atterberg Limit result for various percentages of MCA was shown in Figure 4.5 given below. The potential usage of expansive soils can easily be determined from Atterberg limits. The higher values of LL and PI indicate that the soil is highly expansive. The Liquid Limit decreased from 88.4 to 60, the Plastic Limit increased from 32 to 40, the PI decreased from 56.4 to 20, and the plasticity index decreased by 64.46%.

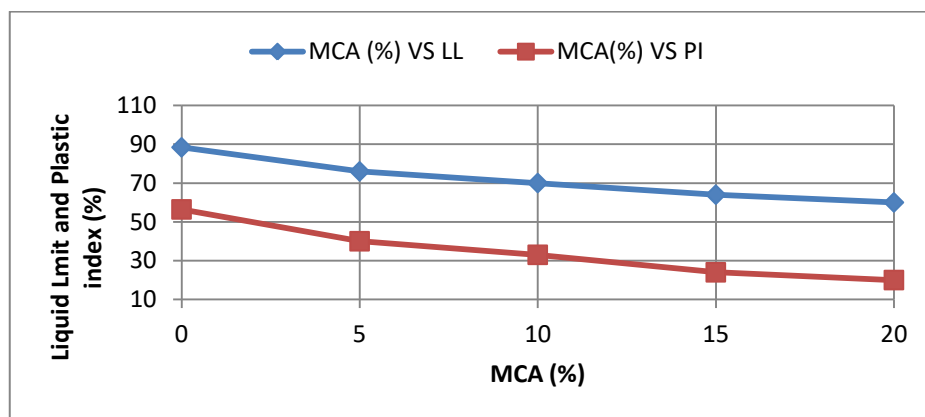


Figure 4.5: Variation of plasticity index with the addition of different MCA

As shown in the figure and table, the addition of maize cob ash causes a decrease in the liquid limit, which in turn causes a reduction in the plastic index and indicates the reduction of the swelling potential of weak soils. The degree of expansion of natural soil is very high before treatment with MCA, but it is changed to a degree of expansion after treatment with MCA. In general, this is clearly shown by the fact that the plasticity index of treated soil decreased with increasing additive quantity. These effects are due to the partial replacement of plastic soil particles with maize cob ash which is a non plastic material, and due to the reduced surface activities, and flocculation and

agglomeration of clay particles caused by cation exchange may be the other. In addition, the formation of adsorbed layer and coating of soil by maize cob ash particles (no sufficient time for pozzolanic reaction) is also another cause.

4.4.1.2. Effect Of WB On Atter-Bergs Limits

The Atterberg Limit result for various percentages of WB was shown in Figure 4.6 given below. The potential usage of weak soils can easily be determined from Atterberg limits. The higher values of LL and PI indicate that the soil is highly expansive soil. The Liquid Limit decreased from 88.4 to 62.5, the Plastic Limit increased from 32 to 40, the PI decreased from 56.4 to 22.5, and the plasticity index decreased by 60.1%.

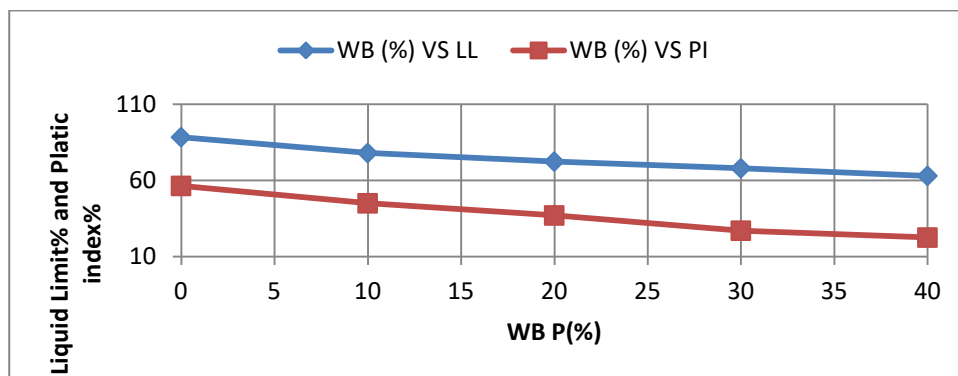


Figure 4.6: Variation of LL, and PI with the addition of different WB

Variation of liquid limit and plastic limit for weak subgrade soil for different waste brick powder content is shown in Figure 4.9. It can be seen that the liquid limit decreased with an increase in waste brick powder content while the plastic limit and plasticity index increased initially and decreased gradually as waste brick powder content increased. As a result when the percentage of additive agent increased plasticity index of the treated weak soil significantly decrease.

4.4.1.3. Effect Of Additions Of MCA And WB On Atter-Bergs Limits

The effect of an addition of MCA and WB on the expansive soil was shown in Figure 4.7 given below. The addition of various percentages of WB and MCA from 5%MCA+40%WBP upto 20%MCA+10%WBP, a decrease in liquid limit, and an increase in plastic limit, can decrease the plasticity index of the soil. As a result, the strength of the soil increased, since the plasticity index of the soil decreased.

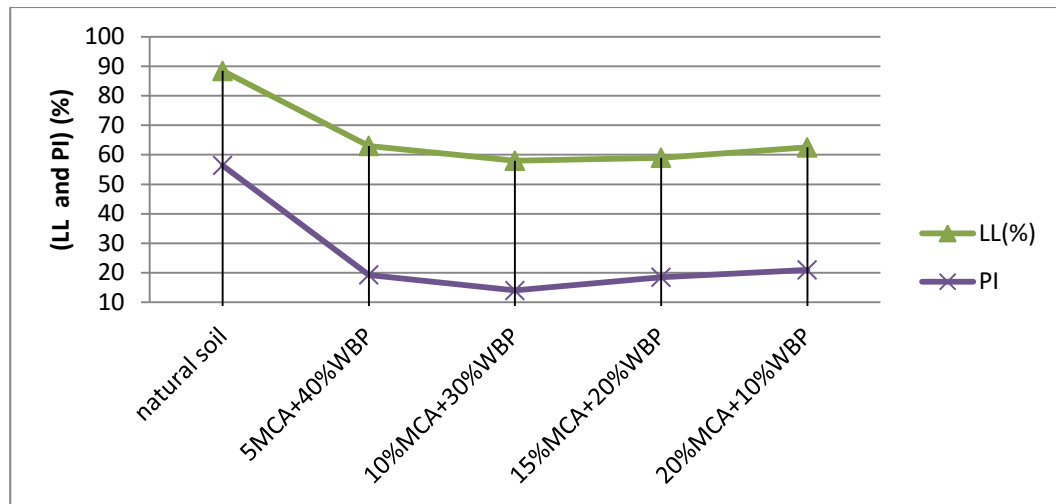


Figure 4.7: Variation of LL, and PI with the addition of different(MCA+WB)

The general trend observed was that with increase in the amount of MCA-WB, PI reduced from a neat value of 56.4% to a value of 14% after an improvement with 10% MCA+30%WB. Hence MC-WB has great impact in reduction of PI. According to ERA manual (2013), the subgrade soil with $PI < 30\%$ and $LL < 60\%$ are suitable subgrade materials. The significant reduction of liquid limit and plasticity index is indicative of improvement. This clearly shows the fact that the plasticity index of treated soil decreased with increasing additive quantity until it reaches the optimum contents. These effects are due to the partial replacement of plastic soil particles with MCA which is non-plastic material and WBP which dries the wet soils. The flocculation and agglomeration properties of soils with materials reduce the plasticity of weak soils and increase soil shear strength.

4.4.2. Free swell test results for weak sub-grade soil treated by MCA and WB

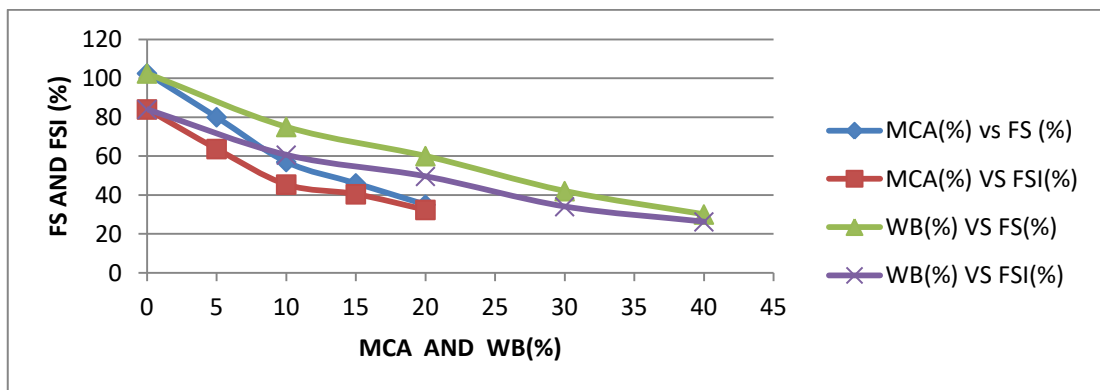
Table 4.9: Free swell test results for weak subgrade soil treated by MCA and WB

S/N O	MCA (%)	WB (%)	FS (%)	FS% decrease	FSI (%)	FSR (%)	IS1270 requirement	
1	0	0	102.5	0	84	1.84	FSI<50 Low expansive	Not Satisfied
2	5	0	80	21.95	63.64	1.64		Not Satisfied
3	10	0	57	44.39	45.37	1.454		Satisfied
4	15	0	46	55.21	40.4	1.4		Satisfied
5	20	0	35	64.85	32.35	1.32		Satisfied
6	0	10	75	26.83	60.55	1.606	FSI<50	Not Satisfied
7	0	20	60	41.46	49.53	1.495		Satisfied
8	0	30	42	59	34	1.34		Satisfied

9	0	40	30	70.7	26.21	1.26	Low expansive	Satisfied
10	5	40	28	72.6	24.3	1.24	FSI<50 Low	Satisfied
11	10	30	22	78.53	22	1.22		Satisfied
12	15	20	31	69.7	27.8	1.278		Satisfied
13	20	10	35	65.9	29.8	1.3		Satisfied

A. Free swell

The effect of maize cob ash on the free swell of the soil under investigation is shown in Figure 4.6, the reduction in free swell is directly proportional to the quantity of maize cob ash, waste brick, and combination of maize cob ash with waste brick. The highest reduction in free swell is attained when the weak subgrade soil is treated with maize cob ash, waste brick, and a combined result which are decreased 64.85%, 70.7%, and 78.53%. An above percent reduction compared to untreated soil, the reduction in free swell is directly proportional to the quantity of maize cob ash, and waste brick. According to the degree of expansion of ASTM D2216 requirement of the test result of maize cob ash (10%,15%, and 20%), the test result of waste brick (20%,30%, and 40%) and the combined test result are decreased. The result obtained guaranteed, the soil treated using the combinations of MCA with WBP shows a low degree of expansion as compared to untreated soil



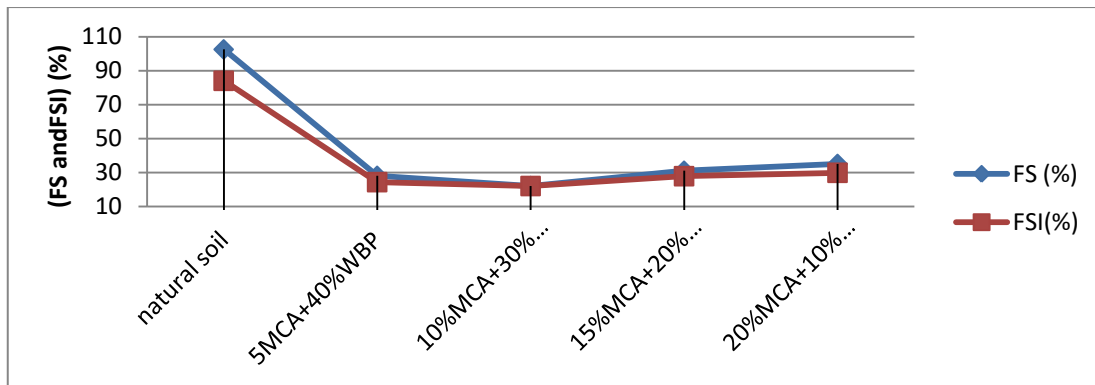


Figure 4.8: Change in the FS, and FSI with varying percentages of MCA and WB

B. Free swell Index

The effects of waste brick on the free swell index of the expansive soil are shown in Figure 4.10. The free swell index value decreased from 84% up to 32.1%, 26.6%, and 22% with an increased amount of maize cob ash, waste brick, and combined percentages requirement of the test result of maize cob ash and waste bricks is satisfied? According to researcher (Craig, R.F, 1997) the sample has FSI the problematic soil values >50% which is categorized as a problematic soil for constructions. I observed that addition of MCA-WB with soil resulted in decreases FSI of the soil. With varies percentages of MCA-WB treatment on soil, FS values becomes below 50%. Generally, as observed from soil samples had significantly changed from high swell property to low swell property, thus may be due to chemical reaction or Cation exchange between the soil- (MCA + WB) mix. Therefore, regards to free swell index test results (MCA and WB) are used as stabilizing additive agents for subgrade road construction material

C. Free swell ratio

The free swell ratio decreases from 1.84 up to 1.32, 1.26, 1.22 as maize cob ash, waste brick content, and combined test increase. From the analysis of test results the free swell ratio of expansive soil swell classification based on the free swell ratio of chapter two in table 2.3, the soil is considered as high swelling soil. According to this classification, the free swell ratio value decreased up to 1.22 with the addition of maize cob ash mix with waste brick, and this makes the soil low soil expansivity or clay type is nonswelling. According to (Sridhar and Prakash) the classification of FSR of soil expansiveness is between 1-1.5 is low, and it makes the soil is nonswelling in clay type.

From the above three swelling characteristics (free swell, free swell index and free swell ratio) the decrease in swelling are:-MCA and WB decreases swell potential of

volume that is previously held by expansive clay minerals and by non plastic and dries the wet soil particles together. The reason may be due to cation exchange in the MCA-WB –soil matrix

4.4.3.The compaction test of treated by MCA-WBP.

A . Maximum dry density and Optimum moisture content

This test was conducted to determine maximum dry density at optimum moisture content and to study the behavior of expansive soil when it blended with different percentages of maize cob ash. Results of maximum dry density (MDD) and optimum moisture content (OMC) for expansive soil stabilized with maize cob ash and waste brick are listed in Table 4.11 and clearly explained in Fig. 4.9,4.10 and 4.11 that the maximum dry density and OMC are at the percentage of maize cob ash and waste brick. The detailed test result is in Appendix (A5 to A7.4)

Table 4.10. Compaction test results for weak soil treated by MCA and WB

S/NO	MCA (%)	WB (%)	MDD (g/cc)	MDD(%) increase	OMC (%)	OMC(%) decrease
1	0	0	1.381	0	28.2	0
2	5	0	1.42	2.82	27.5	2.48
3	10	0	1.432	3.69	24.	14.89
4	15	0	1.423	3.54	24.4	13.47
5	20	0	1.42	2.82	24.8	12.056
6	0	10	1.4	1.37	26.7	5.32
7	0	20	1.42	2.82	25	11.35
8	0	30	1.41	2.099	24.2	14.18
9	0	40	1.4	1.37	24.4	13.47
10	5	40	1.427	3.33	24	14.89
11	10	30	1.44	4.27	22.7	19.503
12	15	20	1.431	3.62	23.7	15.96
13	20	10	1.42	2.85	24	14.89

4.4.3.1. Effect Of Maize Cob Ash On Compaction Tests

From the compaction test result, it is found that MDD increases with the addition of MCA up to 10% and then decreases. The effect of the addition of various percentages of MCA on compaction was shown in the figure below. From Figure 4.9 we can understand the addition of MCA various percentages increase the MDD and decreases the OMC. With an addition of 10 %MCA, MDD increased from 1.381 to 1.432 which shows an (3.69%) improvement. The addition of MCA increased the dry density of weak soil since it is used as filler to fill voids and the oxide composition of MCA

interferes with an absorbed capacity of weak soil, thus show an increase in the maximum dry density. The OMC decreased from 28.2 % to 24% which implies an improvement of 14.89%

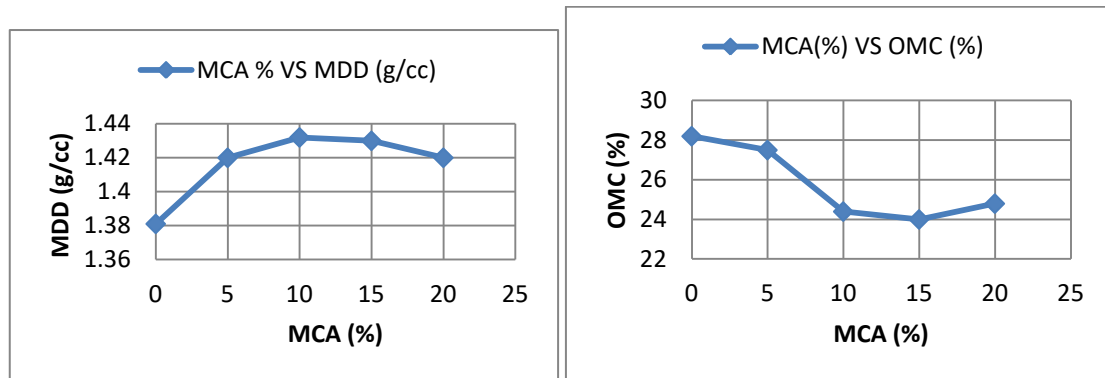


Figure 4.9: Variation of MDD and OMC with the addition of different MCA.

Figure 4.9 shows that as the percentage of MCA content increases the maximum dry density increase and optimum moisture content decrease. The maximum increase in MDD is by addition of 10% MCA due to increase in the density of modified natural soil mix it leads to have strength.

4.4.3.2. Effect Of Waste Brick On Compaction Tests

From the compaction test result like that in the case of WB the MDD increase by the addition WB up to 30% and then the MDD decrease. the effect of the addition of various percentages of MCA on compaction was shown in the figure below. From Figure 4.10 we can understand the addition of WB various percentages increases the MDD and decreases the OMC. With an addition of 30%WB, MDD increased from 1.381 to 1.42 which shows an improvement. The addition of WB increased the dry density of expansive soil since it is used as a filler to fill voids and thus an increase in the maximum dry density. The OMC decreased from 28.2 % to 24.2% which implies an improvement of 14.18% also if we compare those treated soil samples by the duration of time the water content is decreased

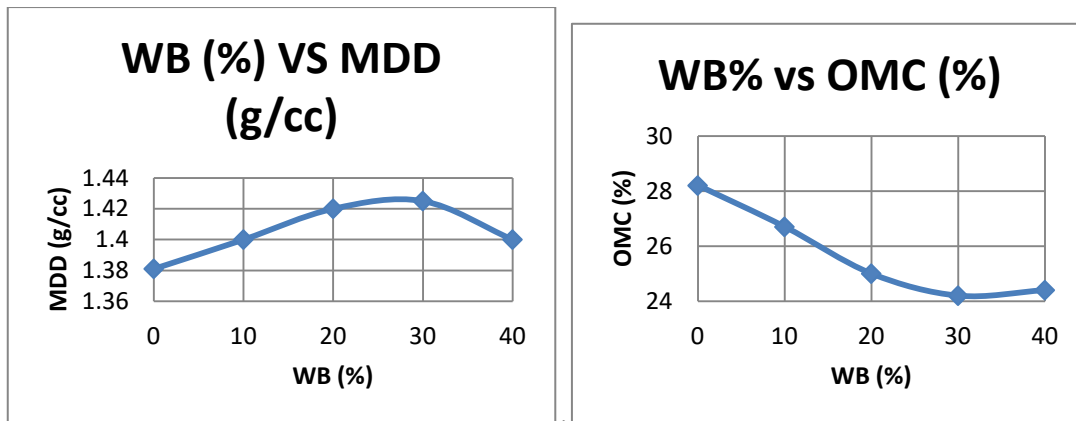


Figure 4.10: Variation of MDD and OMC with addition of different WB

Figure 4.1 shows that as the percentage of waste brick content increases the maximum dry density increase and optimum moisture content decrease. Soil modified with WB by 10%, 20%, 30%, and 40% the value of OMC found to be decreased by 5.32%, 11.35%, 14.18%, and 13.47% respectively. The maximum increase in MDD is by addition of 30% WB due to increase in the density of modified natural soil mix it leads to have strength.

4.4.3.3. The Effect Of The Addition Of MCA - WB On The Compaction Tests.

The addition of MCA and WB up to 30%WB+10% MCA increases MDD and decreases the OMC in the compaction test. Improvement with a blend of 40% waste brick + 5% MCA gave a value of 1.427g/cm³, A addition of 30% waste brick + 10% MCA gave a value of 1.44 g/cm³, Addition of 20% waste brick + 15%MCA gave a value of 1.431 g/cm³, Addition of 10%waste brick + 20%MCA gave a value of 1.42 g/cm³.

As can be seen from the below a figure, the MDD and OMC show a increase in the treatment of weak soil with MCA-WB additive agents. Hence, the MDD of natural soil increases from 1.381 g/cm³ to 1.44 g/cm³ and OMC decreases from 28.2% to 22.7 when the two materials were used in combination. As the researcher state the addition of agricultural waste and waste material increases the MDD, and decreases the OMC during the compaction test. The increment of MDD comes from the filling property of MCA and WB material in the voids spaces of the soil. Thus the materials used to improve the soil were found to facilitate the closer packing of the soil particles and thus an increase in the maximum dry density. The reduction in repulsive force in clay particles due to minimization of layer thickness enables the weak soil to compact with small

water content and results increase the density of soil as well as the shear strength of weak soil. The water solubility of the soil is decreased after treating the soil with the stabilizing agent, which results in a decrease in OMC.

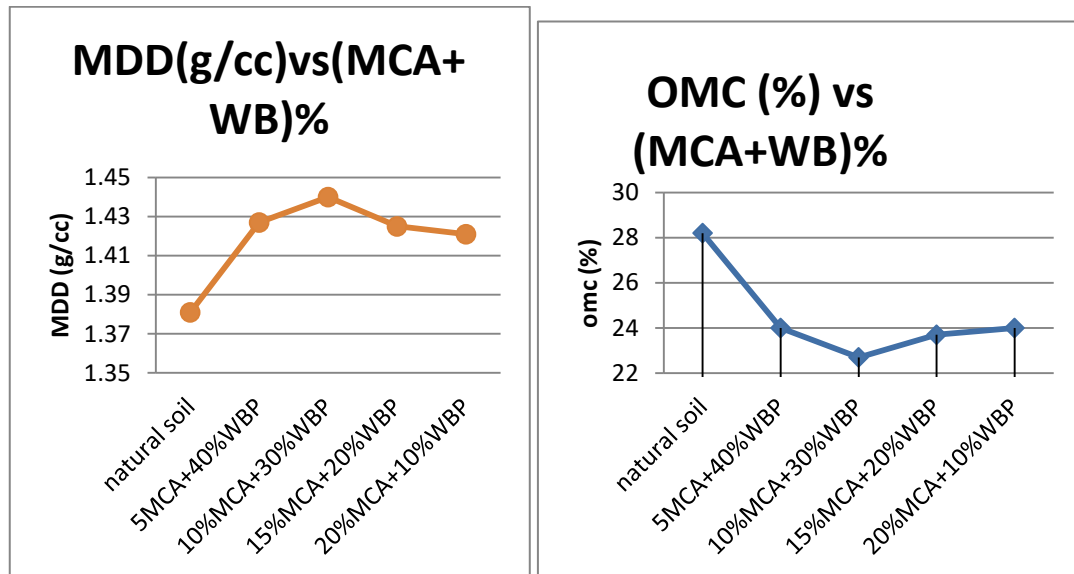


Figure 4.11: Variation of MDD and OMC with addition of different (MCA+ WB) content

Figure 4.11 shows that as the percentage of MCA-WB increases the maximum dry density increase and optimum moisture content decrease. The maximum increase in MDD is by addition of 10%MCA+30%WB due to increase in the density of modified natural expansive soil mix it leads to have more strength. When MCA-WB is added to the soil, aggregation and agglomeration take place, which lead to more non plastic formation and reduction in clay colloidal content, which reduces the moisture uptakes of the mix. This is because of the frictional resistance from waste brick in addition to cohesion from weak soil and maize cob ash gives the filling of void spaces properties to the soil.

4.4.4. CBR and CBR swell results for weak subgrade soil treated by MCA and WB

CBR test was conducted using the AASHTO T193 procedure and conducted to determine the CBR value of the soil samples. The results of the soaked CBR test for weak soils and soils mixed with varying percentages of MCA and WB. CBR test for this study was conducted by taking air dried samples which pass through sieve No.4 (4.75mm) for natural soil mixed with different replacement proportions of MCA and WB and applied modified compaction. A three-point CBR test at 10, 30, and 65 blows

was conducted according to AASHTO T193, the CBR values at 95% MDD were determined, and soaked for 96 hours for samples.

CBR swells were also determined after four days soaked for each replacement with varying percentages added to the soil. The proof reading ring load graphs are above the natural soil, which indicated each proposed replacement percentage used to improve the engineering properties of sub-grade soil as seen in Figures 4.12,4.13,4.14 below. The detailed test result is Appendix (A8 upto A10.4)

Table 4.11: CBR and CBR swell results for weak soil treated by MCA and WB

S/NO	MCA (%)	WB (%)	CBR	CBR Swell	ERA CBR Req(2013) subgrade	Remark
1	0	0	1	7.85	S1	Not satisfy
2	5	0	2.2	7	S1	Not satisfy
3	10	0	3.31	4	S2	Satisfy
4	15	0	3.05	5	S2	Satisfy
5	20	0	2.6	3.21	S1	Not satisfy
6	0	10	1.6	7.14	S1	Not satisfy
7	0	20	3.2	5.7	S2	Satisfy
8	0	30	3.5	5	S2	satisfy
9	0	40	3	3.18	S2	Not satisfy
10	5	40	5.3	1.83	S3	satisfied
11	10	30	10.5	1.2	S4	satisfied
12	15	20	8.3	1.3	S4	satisfied
13	20	10	7.1	1.55	S3	satisfied

4.4.4.1. Effect of maize cob ash on CBR and CBR swell of the soils

The California bearing ratio value slightly increased varying percentage of MCA, and soaked stabilizing expansive soil with MCA. CBR value was significantly increased by According to the laboratory test result the CBR value for all proposed percentages of 5 up to 20% of MCA and were 2.2%, 3.31%, 3.05% & 2.6% as shown in Table 4.12 and Figure 4.12 respectively. According to ERA 2013 design manual, subgrade class soil improved S1 to S2 initially value of 1% to 3.31%. The results of treated soil samples show an increase of soaked CBR and when treating the weak soil using stabilizing

agent peak CBR values were recorded at 3.31% CBR at 10% MCA, which satisfied ERA sub-grade specification.

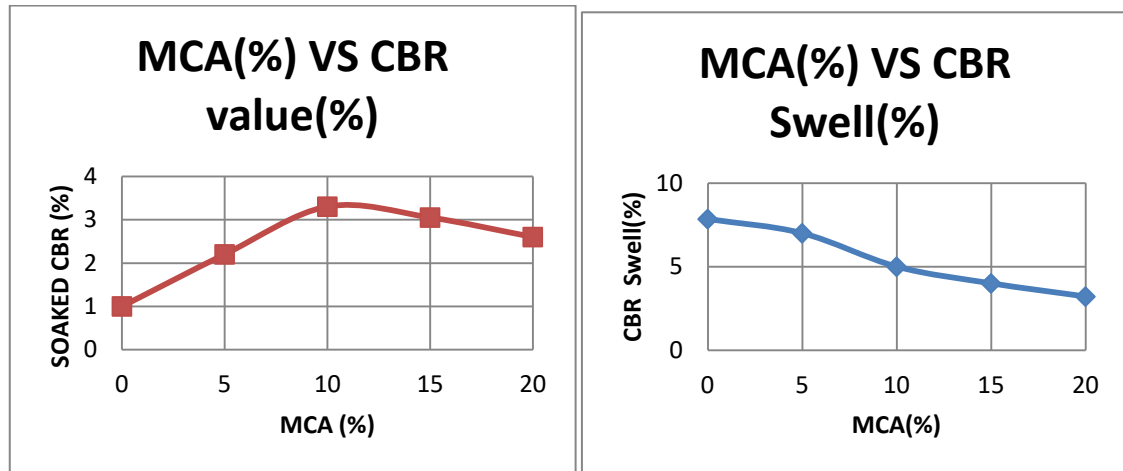


Figure 4.12: Variation of CBR and CBR Swell with addition of different MCA.

The CBR swell values also decreased from 7.85% to 3.21%, 7%, 4.0%, 5%, and 3.21 for an addition of 5%, 10%, 15%, and 20% of MCA with expansive soils

4.4.4.2. Effect of WB on CBR and CBR swell of the soils

The effect of waste brick on the CBR values of the soil is shown in figure 4.13. When the weak soil was treated with 10% to 40% of waste brick. The CBR value of natural soil was increased from 1% to 3.5%. According to ERA 2013 design manual, subgrade class soil improved S1 to S2 initially value of 1% to 3.5%. It was observed that there is a significant improvement in CBR value with increase in waste brick content. CBR values from WB stabilization started reducing when 30% WB content was reached. And also a number blows increases at a constant mix of weak soil with waste brick stabilizer agent the value of cbr increases. A better compaction improves intermolecular attraction which fills voids and dries of wet soils enhances the strength of the subgrade soil.

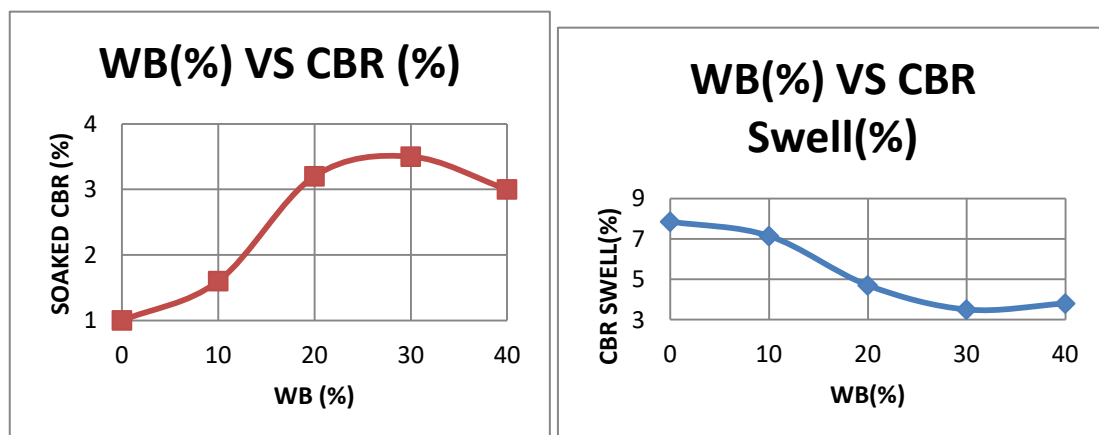


Figure 4.13: Variation of CBR and CBR Swell with addition of different WB.

As shown in Figure 13 shown above, the CBR value increased as the amount of 30% WB increased and then decreased, while the CBR Swell test decreased. The natural soil has a CBR value of 1%, which is a poor subgrade material. However, the addition of WB by 10%, 20%, and 30% increases the CBR value by 1.6%, 3.1%, 3.5, and then decreases by 3.05%. The CBR swell values also decreased from 7.85% to 3.18%, 7.14%, 5.7%, 5 and 3.18% for an addition of 10%, 20%, 30%, and 40% of WB with expansive soils. This result agrees with the findings of (Pardakhe & Ban, 2019 and Al-bandhani & Al-taie, 2019). The swell potential of natural soil was reduced by 3.18% at the WB content. During the mix of WB with natural soils, is probably reduce surface activity result of form good agglomeration and reduce the change in the volume of weak soils.

4.4.4.3. The Effect of Combined Addition of MCA and WB on CBR and CBR swell of the soils

The soaked CBR value was significantly increased by treating weak soils with addition varying content of MCA and WB. The CBR value increased from 1% to 10.5% for addition of MCA with WB. The CBR value increased by 10.5% at addition of 10% of MCA with 30%WB. The effect of varying content MCA with a varying percentage of WB on CBR value is shown in Figure 4.14.

According to ERA 2013 design manual, a subgrade strength class S4 should be attain between 8-14% of CBR value to design pavement structure. When the 10%MCA+30%WB amount used and with four days of curing time, it can achieve the sufficient strength per the Ethiopian code requirement. As it shown clearly after soaking four days of curing time the CBR value progressively improved with 10%MCA+30%WB from inside of the factor the initially value of 1% to 10.5%.

The results of the CBR tests soils in Table 4.11 show that sample replace with the combination of maize cob ash and waste brick powder have CBR value of greater than 5%. Based on ERA specification, these sample indicates as fair sub grade materials. The percent swell results <2% which is an indication of less expansiveness of the soil which is a fair subgrade material.

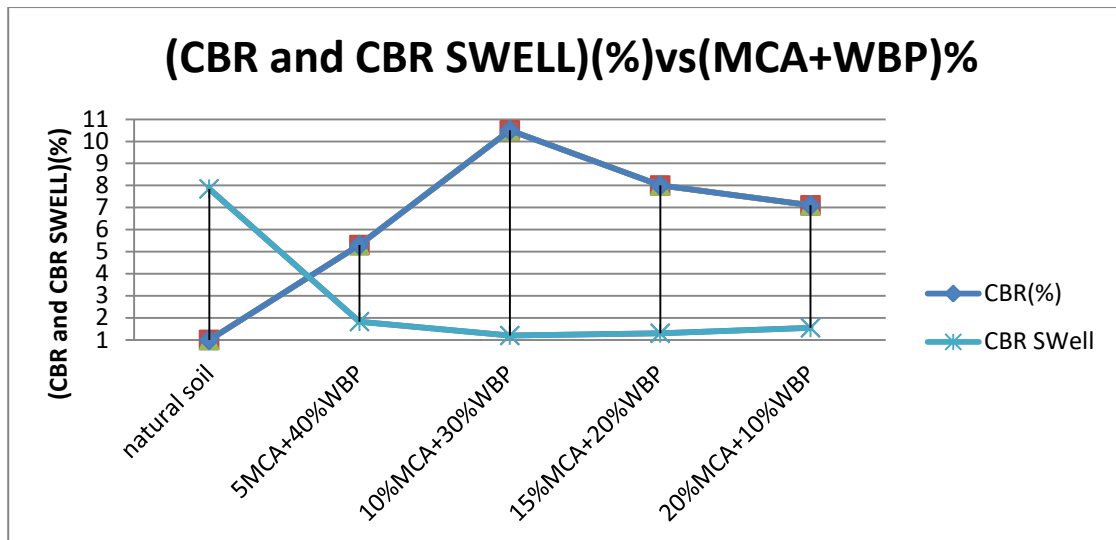


Figure 4.14: Variation of CBR and CBR Swell with addition of different (MCA+WB content)

The addition of MCA and WB together led to a more increase of the CBR value, but MCA or WB used alone is lower than a combination. This shows that the load bearing capacity of the sample increased considerably with MCA and WB treatment and curing has a significant effect on the CBR values. The combination of MCA and WB can strongly improve the strength of expansive soils. The improvements in the CBR values of samples satisfy the minimum requirements that qualify them as road construction materials and showed that the soil sample was effectively stabilized by MCA in combination with WB. Depending on the result an Addition of a minimum all combination of MCA-WB on natural soil satisfied ERA standard specification for subgrade construction

The CBR swell was also significantly decreased due to the addition of MCA with WB. The effect of the addition of MCA and WB on the CBR swell for the soil-MCA and WB mixtures are shown in Table 4.9 (above). The result shows a decrease in the CBR swell from 7.85% to 1.2%, with the combination addition of the 10% MCA and 30% WB. So the minimum CBR swell value was obtained at combination of 10% MCA and 30% WB and further addition of MCA and WB causes an increase in the CBR swell of the weak soils for samples. ERA (2013) Manual recommends that sub-grade soil should have CBR swell of less than 2 percent so the stabilized soil satisfies this standard and it is possible to use the treated soil for sub-grade material in road construction. This is also due to the replacement of some space by maize cob ash and waste brick powder that is previously held by expansive clay minerals. Generally, the CBR value started to increase when it effectively achieved combination of weak subgrade soil with

percentages of 30% waste brick and 10% maize cob ash mix were satisfied the quality and the strength .

4.5. Optimum content of maize cob ash and waste brick

From among the experiment percentage of maize cob ash (5%,10%,15%,20%), the replacement of MCA give, the maximum of CBR. CBR for 10% replacement of MCA was obtained as 3.3% and experiment percentage of waste brick (10%,20%,30%,40%), the replacement of 30%WB give, the maximum of CBR. CBR for 30% replacement of WB was obtained as 3.5%. The maximum dry density (MDD) value increases until it reaches the optimum. A combination of the experiment percentage of addition of 40%WB+5%MCA,30%WB+10%MCA,20%WB+15MCA,10%WB+20%MCA),the maximum dry density (MDD) value increases until it reaches the optimum. At percent of (10%MCA+30%WB) values of MDD and CBR were achieved. When the optimum (10%MCA+30%WB) is added to this soil, the CBR value is maximum and the CBR was obtained as 10.5

CHAPTER FIVE

5. CONCLUSION AND RECOMMENDATIONS

5.1. Conclusions.

The objective of this study is to investigate achieved on the properties of weak subgrade soils with maize cob ash and waste brick powder stabilization. stabilizing soils with maize cob ash and waste brick is now a cost-effective method of converting poor-quality soil into a strong impermeable medium. The laboratory tests conducted for this study were moisture content, specific gravity, grain size analysis, Atterberg limits, Compaction test, free swell test, California Bearing Ratio, and CBR swell tests. The test procedures were based on AASHTO, ERA, and ASTM laboratory test standards.

- The subgrade soils considered for this study have a very low load-bearing capacity and high swelling potential, which makes the soils unsuitable for subgrade construction without improvement.
- The improvement on engineering properties of weak subgrade soil to use as subgrade materials, were observed plastic index, Freeswell test ,MDD and OMC ,CBR value and CBR swell potential.
- The weak soil used in this study has liquid limit (LL= 88.4 %), plastic index(PI= 56.4%) and more than 35 % passing #200 sieve .Thus, as per AASHTO soil classification system, the soil categorized as an A-7-5, and CH in the USCS soil classification system, and the group index 64. Based on the AASHTO an A-7-5 with rating of Fair-to Poor to be used as a sub-grade material
- As far as the index properties of natural subgrade soils were studied, the four soils were weak soils, and one sample with high free swell, high plastic index and poor soil values had been selected.
- The additives mixed ratio (MCA,WB,MCA-WB) and10%MCA,30%WB and Combination of 10%MCA+30%WB decreases the LL and PI for untreated soil samples. The LL decreases with slight changes for soil samples from control value 88.4% to 60%, 88.4% to 62.5% and 88.4% to 58%,and for soil samples, respectively. The PL increases with stabilization of additives of the Mix-ratio from control value 32% to

40%, 32% to 40.1% and 32 to 44% for a soil sample respectively. The PI decreased from 56.4% to 20%, 56.4% to 22.4% and 56.4% to 14% soil samples, respectively. Also, in the MCA-WB mixed ratio, the liquid limit and plastic index are decreases when increasing the stabilizing agent. The stabilizers (MCA, WB) alone is significantly change the PI, and combination of MCA and WB is significantly changing both LL and PI. This means that a treated soil samples fulfill the ERA specification according to PI

- The free swell, free swell index, and free swell ratio of the stabilized samples decreased with increasing maize cob ash, waste brick, and a combination of waste brick and maize cob ash content.
- The addition of MCA and WB together led to a more increase of the maximum dry density and decrease in optimum moisture content compared to the addition of MCA and WB separately.
- Stabilization of natural subgrade soil with maize cob ash and waste brick increased the CBR values of weak soil. Problematic soil modified with MCA by 5%, 10%, 15%, and 20%. The value maximum increment in CBR value is achieved of 10% MCA, the CBR value increased from 1% to 3.3% and expansive soil modified with WB by 10%, 20%, 30%, and 40%. The value maximum increment in CBR value is achieved of 30% MCA, the CBR value increased from 1% to 3.5% and combined percentage of MCA, and WB, maximum increment in CBR value is achieved a combination of 10% MCA and 30% WB, the CBR value increased from 1% to 10.5% respectively for sample. Optimum content in CBR is by the addition of percentage 10% MCA and 30% WB, a combination that satisfies the ERA standard of requirements. Hence, combination of MCA and WB can actively improve the CBR strength of the weak soil.
- The CBR and CBR swell value for the Sample were above 5% and less than 2%, so they were good to use as a subgrade soil for road construction.

Distinguish of the results obtained with some specifications showed that the improvements obtained from this stabilization, and it was satisfied ERA Standard Specification for sub-grade materials.

Combining local available materials and waste material like (maize cob ash and Waste brick) can efficiently improve the engineering properties of soils, and help increase the environmental resources available for construction projects. In addition to this, it will reduce pollution and increase the production rate.

Generally, natural weak subgrade soil and treated natural soil properties were computed based on laboratory test results. Each observed value was presented according to the requirements of ERA standard specifications. Based on the laboratory result the natural soils was not satisfy requirements of ERA standard specifications for subgrade material. However, Replacement with 10% maize cob ash, and 30% waste brick, the mix proportion satisfied ERA standard specifications more effectively for sub-grade material. Finally, we concluded that these locally available materials and waste materials have a significant effect on improving the plasticity and swell potential, as well as the strength of the weak soils.

5.2. Recommendations

Based on the findings of this research, the following recommendations are forwarded:

- As investigated in this research work maize cob ash or waste brick is not an effective stand-alone stabilizer for highly plastic soils. Therefore, it is recommended to study the more effective potential use of combination as an admixture stabilizer.
- MCA and WBP investigated in the present study can be used as a soil stabilizing material in combination with economic and environmental benefits. Therefore, concerned bodies like industries and agriculture should be made aware of this potential soil stabilizing material and promote its standardized production and usage
- Further study should be done using MCA-WBP as stabilizer materials on different type of soils

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APPENDIX

Appendix 0.1. moisture content

Sample	sample 1	
Moisture Content: Container No.	A-3	A-10
Weight Of Wet Soil + Container(Gm)(W1)	236.30	243.20
Weight Of Dry Soil + Container(Gm)(W2)	180.60	185.20
Weight Of Container(Gm)(W3)	26.00	25.50
Weight Of Moisture(W4=W1-W3)	55.70	58.00
Weight Of Dry Soil(W5=W2_ W3)	154.60	159.70
Moisture Content (W%=W4/W5)	36.028	36.32
Average Moisture Content(W%)	36.173	

Appendix A 0.2: Specific gravity of soil in samples

The relative degree of temp K@20°C=0.9973	Sample		
Specimen number	A3	A1	A
Mass of pycnometer (W1)	28.8	28.13	27.7
Mass of pycnometer+Oven dry soil (W2)	38.8	38.13	37.7
Mass of pycnometer+Oven dry soil+water (W3)	84.1	83.6	84.27
Mass of oven-dry soil (WS=W2-W1)	10	10	10
Mass of pycnometer +water (W4)	77.72	77.23	77.94
Mass of water (A=W4-W1)	48.92	49.1	50.24
Mass of pycnometer over and above dry soil (B=W3-W2)	45.3	45.47	46.57
Mass of water in the same volumetric soil (C=A-B)	3.62	3.63	3.67
The specific gravity of soil temp (Gs@24°C (WS/C)	2.7624	2.7548	2.7248
Specific Gravity of soil temp K(GS at 20°C)	2.755	2.747	2.717
Average K (Gs at 20°C)	2.739		

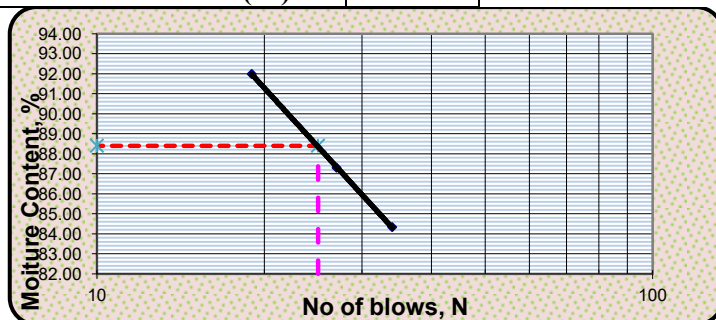
Appendix 3

SAMPLE (1) "SOIL CLASSIFICATION, (AASHTO M- 145 and ASTM D3282)			
Soil Classification,(Group Index)		64	A-7-5 (64)
Wt of Before Washing, (gm)			1000
Sieve sizes (mm)	Wt Retained (gm)	% Retained	Cum.% Pass
4.75	0	0.0	100
2	3	0.3	99.7
0.425	9	0.9	98.8
0.075	26.7	2.7	96.1
pan	961.3	96.1	
Sieve sizes (mm)	Cum.% Pass		
4.75	100.0		
2	99.7		
0.425	98.4		
0.075	96.1		
0.036139453	90.37		
0.02588	87.55		
0.016672	84.72		
0.01193	81.90		
0.009911	79.07		
0.007127	75.30		
0.005062	71.54		
0.003636	67.77		
0.002624	64.00		
0.002	60.92		
0.001861	60.24		
0.0011	54.59		

Appendix A1:-Natural soil sample.

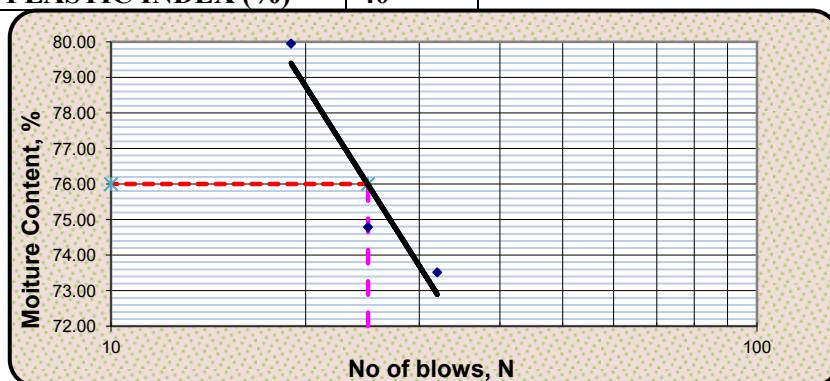
Test method of Atterberge Limit AASHTO T-89 and AASHTO T-90							
LIQUID LIMIT					PLASTIC LIMIT		
Container No.	42	73	16		66	15	Avg
Wt of wet soil + container, gm	31.010	29.370	27.84		12.26	12.050	
Wt of dry soil + container, gm	21.430	20.160	19.01		11.60	11.47	

Wt of water	9.58	9.21	8.83		0.66	0.58	
Wt of container	10.070	9.610	9.410		9.580	9.610	
Wt of dry soil, gm	11.36	10.55	9.60		2.02	1.86	
Water content, %	84.33	87.30	91.98	87.9	32.67	31.18	32
No. of blows	34	27	19				
LIQUID LIMIT (%)	88.4						
PLASTIC LIMIT (%)	32						
PLASTIC INDEX (%)	56.4						



Appendix A2:-Atterberg Limit Test Analysis of 5% Maize Cob ash

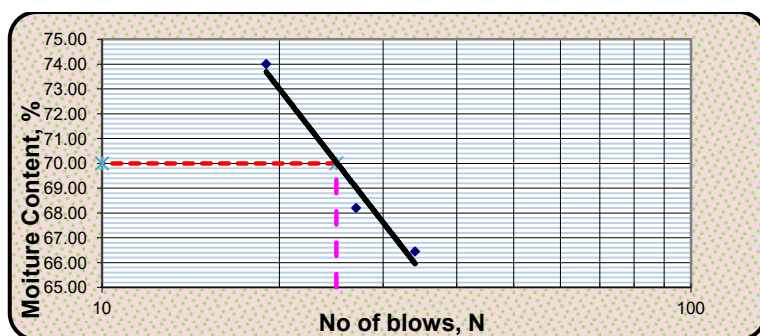
<i>Test method of Atterberge Limit AASHTO T-89 and AASHTO T-90</i>							
LIQUID LIMIT				PLASTIC LIMIT			
Container No.	42	66	34		F	G	Avg
Wt of wet soil + container, gm	30.580	30.370	32.640		12.680	12.600	
Wt of dry soil + container, gm	21.890	21.470	22.630		11.940	11.870	
Wt of water	8.69	8.90	10.01		0.74	0.73	
Wt of container	10.070	9.570	10.110		9.790	9.880	
Wt of dry soil, gm	11.82	11.90	12.52		2.15	1.99	
Water content, %	73.52	74.79	79.95	76.1	34.42	36.68	35.6
LIQUID LIMIT (%)	76						
PLASTIC LIMIT (%)	36						
PLASTIC INDEX (%)	40						



Appendix A2.1:-Atterberg Limit Test Analysis of 10% Maize Cob ash

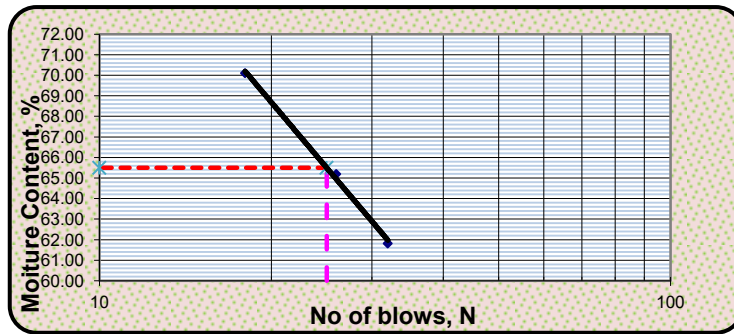
<i>Test method of Atterberge Limit AASHTO T-89 and AASHTO T-90</i>							
LIQUID LIMIT				PLASTIC LIMIT			
Container No.	G	73	21		22	34	Avg

Wt of wet soil + container, gm	34.010	32.650	31.830		12.420	12.780	
Wt of dry soil + container, gm	24.380	23.180	22.630		11.750	12.040	
Wt of water	9.63	9.47	9.20		0.67	0.74	
Wt of container	9.890	9.610	10.010		9.860	10.110	
Wt of dry soil, gm	14.49	13.57	12.62		1.89	1.93	
Water content, %	66.46	69.79	72.90	69.7	35.45	38.34	31.9
No. of blows	34	27	19				
LIQUID LIMIT (%)	70						
PLASTIC LIMIT (%)	36.9						
PLASTIC INDEX (%)	33						



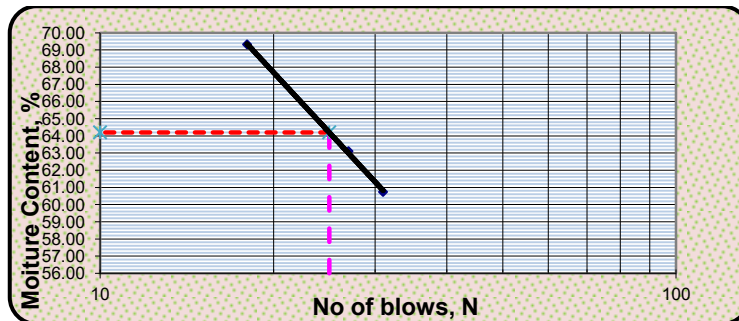
Appendix A2.2:-Atterberg Limit Test Analysis of 15% Maize Cob ash

<i>Test method of Atterberge Limit AASHTO T-89 and AASHTO T-90</i>							
LIQUID LIMIT					PLASTIC LIMIT		
Container No.	73	9	F		13	4	Avg
Wt of wet soil + container, gm	30.29	31.53	30.71		11.97	11.67	
Wt of dry soil + container, gm	22.39	22.97	22.08		11.32	11.01	
Wt of water	7.90	8.56	8.63		0.65	0.66	
Wt of container	9.610	9.840	9.770		9.670	9.340	
Wt of dry soil, gm	12.78	13.13	12.31		1.65	1.67	
Water content, %	61.82	65.19	70.11	65.7	39.39	39.52	39.5
No. of blows	32	26	18				
LIQUID LIMIT (%)	65.7						
PLASTIC LIMIT (%)	39.5						
PLASTIC INDEX (%)	26						



Appendix A2.3:-Atterberg Limit Test Analysis of 20% Maize Cob ash

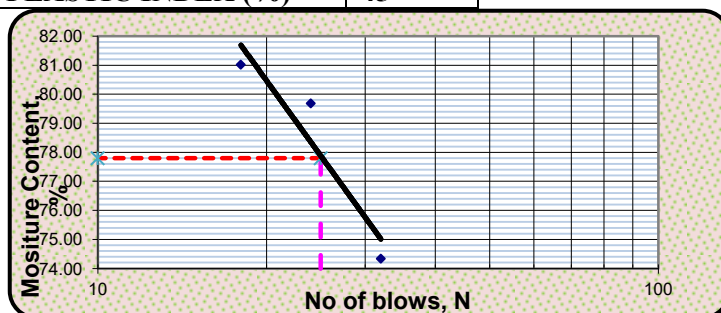
<i>Test method of Atterberge Limit AASHTO T-89 and AASHTO T-90</i>							
LIQUID LIMIT				PLASTIC LIMIT			
Container No.	34	42	66		33	4	Avg
Wt of wet soil + container, gm	33.39	32.44	31.69		12.32	11.88	
Wt of dry soil + container, gm	24.59	23.77	22.63		11.55	11.1	
Wt of water	8.80	8.67	9.06		0.77	0.76	
Wt of container	10.10	10.03	9.560		9.500	9.33	
Wt of dry soil, gm	14.49	13.74	13.07		2.05	1.79	
Water content, %	60.73	63.10	69.32	64.4	37.56	42.46	40.0
No. of blows	31	26	18				
LIQUID LIMIT (%)	64						
PLASTIC LIMIT (%)	40						
PLASTIC INDEX (%)	24						



Appendix A3.1:-Atterberg Limit Test Analysis of 10% Waste brick

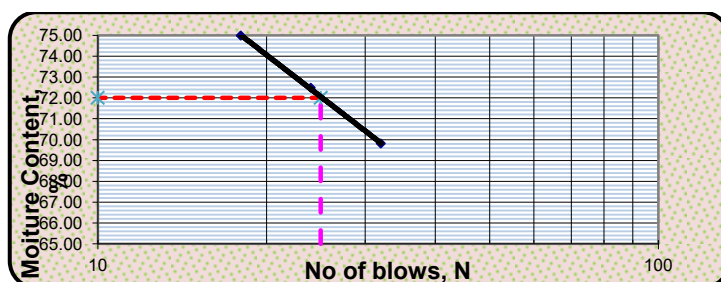
<i>Test method of Atterberge Limit AASHTO T-89 and AASHTO T-90</i>							
LIQUID LIMIT				PLASTIC LIMIT			
Container No.	F	G	66		34	42	Avg
Wt of wet soil + container, gm	31.64	33.050	32.54		12.73	12.69	
Wt of dry soil + container, gm	22.31	22.770	22.25		12.07	12.029	
Wt of water	9.33	10.28	10.29		0.66	0.66	
Wt of container	9.760	9.870	9.550		10.09	10.04	
Wt of dry soil, gm	12.55	12.90	12.70		1.98	1.989	
Water content, %	74.34	79.69	81.02	78.4	33.33	33.23	33.3

No. of blows	32	24	18	
LIQUID LIMIT (%)	78			
PLASTIC LIMIT (%)	33			
PLASTIC INDEX (%)	45			



Appendix A3.2:-Atterberg Limit Test Analysis of 20% Waste brick.

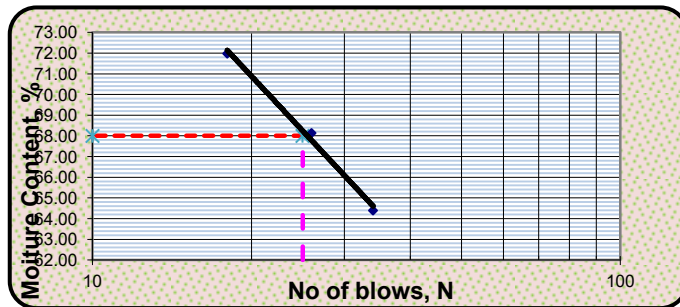
<i>Test method of Atterberge Limit AASHTO T-89 and AASHTO T-90</i>							
LIQUID LIMIT				PLASTIC LIMIT			
Container No.							Avg
Wt of wet soil + container, gm	33.840	32.190	33.89		12.11	11.73	
Wt of dry soil + container, gm	23.990	22.920	23.67		11.51	11.21	
Wt of water	9.85	9.27	10.22		0.60	0.52	
Wt of container	9.880	10.130	10.04		9.870	9.670	
Wt of dry soil, gm	14.11	12.79	13.63		1.64	1.54	
Water content, %	69.81	72.48	74.98	72.4	36.59	33.77	35.2
No. of blows	32	24	18				
LIQUID LIMIT (%)	72						
PLASTIC LIMIT (%)	35						
PLASTIC INDEX (%)	42						



Appendix A3.3:-Atterberg Limit Test Analysis of 30% Waste brick

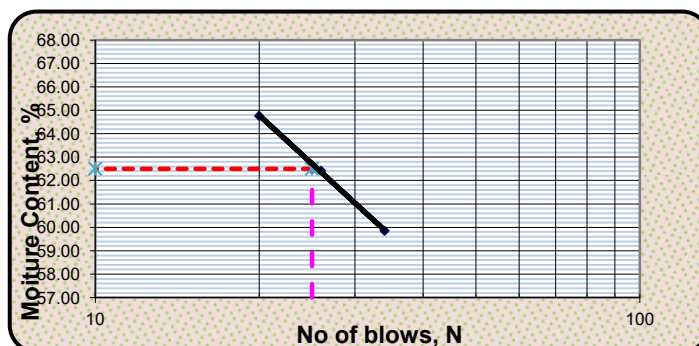
<i>Test method of Atterberge Limit AASHTO T-89 and AASHTO T-90</i>							
LIQUID LIMIT				PLASTIC LIMIT			
Container No.	4	33	66		42	F	Avg
Wt of wet soil + container, gm	30.6	33.08	30.34		11.92	11.77	
Wt of dry soil + container, gm	22.280	23.520	21.76		11.33	11.28	
Wt of water	8.32	9.56	8.58		0.59	0.49	

Wt of container	9.360	9.490	9.840		9.860	10.09	
Wt of dry soil, gm	12.92	14.03	11.92		1.47	1.19	
Water content, %	64.40	68.14	71.98	68.2	40.14	41.18	40.7
No. of blows	34	26	18				
LIQUID LIMIT (%)	68						
PLASTIC LIMIT (%)	41						
PLASTIC INDEX (%)	27						



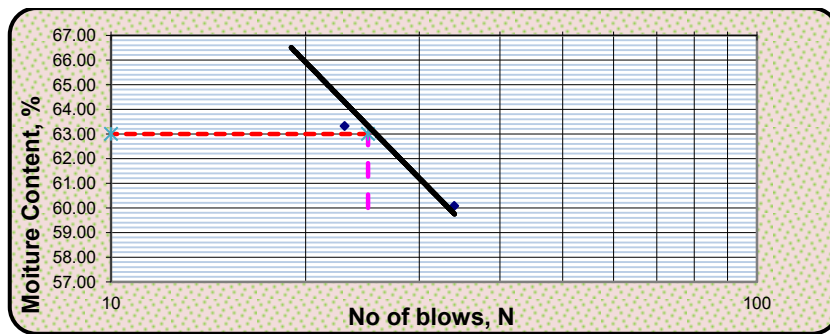
Appendix A3.4:-Atterberg Limit Test Analysis of 40% Waste brick

<i>Test method of Atterberge Limit AASHTO T-89 and AASHTO T-90</i>							
LIQUID LIMIT				PLASTIC LIMIT			
Container No.	F	15	17		22	21	Avg
Wt of wet soil + container, gm	32.890	34.94	33.60		11.97	12.24	
Wt of dry soil + container, gm	24.200	25.21	24.34		11.38	11.59	
Wt of water	8.69	9.73	9.26		0.59	0.65	
Wt of container	9.680	9.620	10.04		9.870	10.01	
Wt of dry soil, gm	14.52	15.59	14.30		1.51	1.58	
Water content, %	59.85	62.41	64.76	62.3	39.07	41.14	40.1
No. of blows	34	26	20				
LIQUID LIMIT (%)	62.5						
PLASTIC LIMIT (%)	40.1						
PLASTIC INDEX (%)	22.4						



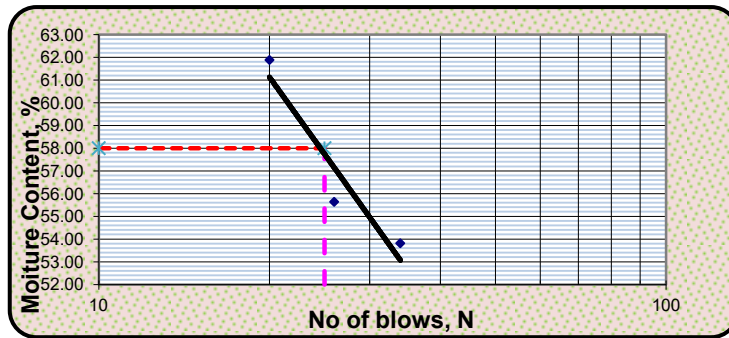
Appendix A4.1:-Atterberg Limit Test Analysis of 5% MCA+40% WB

<i>Test method of Atterberge Limit AASHTO T-89 and AASHTO T-90</i>							
LIQUID LIMIT					PLASTIC LIMIT		
Container No.	66	F	42		41	22	Avg
Wt of wet soil + container, gm	34.31	34.030	34.97		15.37	14.90	
Wt of dry soil + container, gm	24.950	24.610	24.97		13.75	13.37	
Wt of water	9.53	9.42	10.00		1.60	1.53	
Wt of container	9.370	9.780	10.080		10.050	9.850	
Wt of dry soil, gm	15.58	14.83	14.89		3.70	3.52	
Water content, %	60.08	63.52	67.16	63.5	43.38	43.47	43.7
No. of blows	34	24	19				
LIQUID LIMIT (%)	63						
PLASTIC LIMIT (%)	43.7						
PLASTIC INDEX (%)	19						



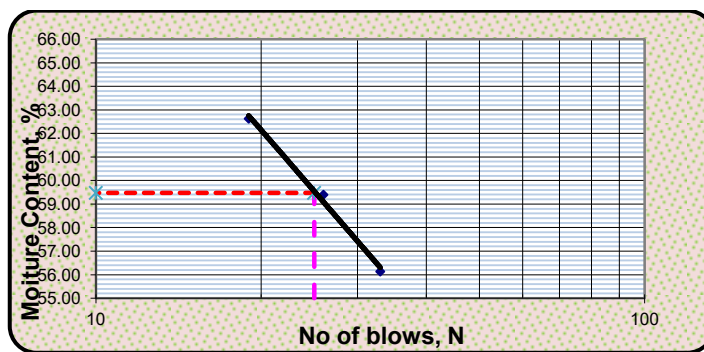
Appendix A4.2:-Atterberg Limit Test Analysis of 10% MCA+30% WB

<i>Test method of Atterberge Limit AASHTO T-89 and AASHTO T-90</i>							
LIQUID LIMIT					PLASTIC LIMIT		
Container No.	4	Z	15		F	42	Avg
Wt of wet soil + container, gm	31.540	33.030	33.18		11.84	11.93	
Wt of dry soil + container, gm	23.780	24.740	24.17		11.22	11.37	
Wt of water	7.76	8.29	9.01		0.62	0.56	
Wt of container	9.360	9.840	9.610		9.800	10.09	
Wt of dry soil, gm	14.42	14.90	14.56		1.42	1.28	
Water content, %	53.81	55.64	61.88	57.1	43.66	43.75	43.7
No. of blows	34	26	20				
LIQUID LIMIT (%)	58						
PLASTIC LIMIT (%)	44						
PLASTIC INDEX (%)	14						



Appendix A4.3:-Atterberg Limit Test Analysis of 15% MCA+20% WB

<i>Test method of Atterberg Limit AASHTO T-89 and AASHTO T-90</i>							
LIQUID LIMIT					PLASTIC LIMIT		
Container No.	41	22	66		42	F	Avg
Wt of wet soil + container, gm	33.360	33.880	33.03		12.71	12.18	
Wt of dry soil + container, gm	24.980	24.930	24.00		11.95	11.47	
Wt of water	8.38	8.95	9.03		0.76	0.71	
Wt of container	10.050	9.860	9.580		10.07	9.790	
Wt of dry soil, gm	14.93	15.07	14.42		1.88	1.68	
Water content, %	56.13	59.39	62.62	59	40.43	42.26	41.3
No. of blows	33	26	19				
LIQUID LIMIT (%)	59						
PLASTIC LIMIT (%)	41						
PLASTIC INDEX (%)	18						



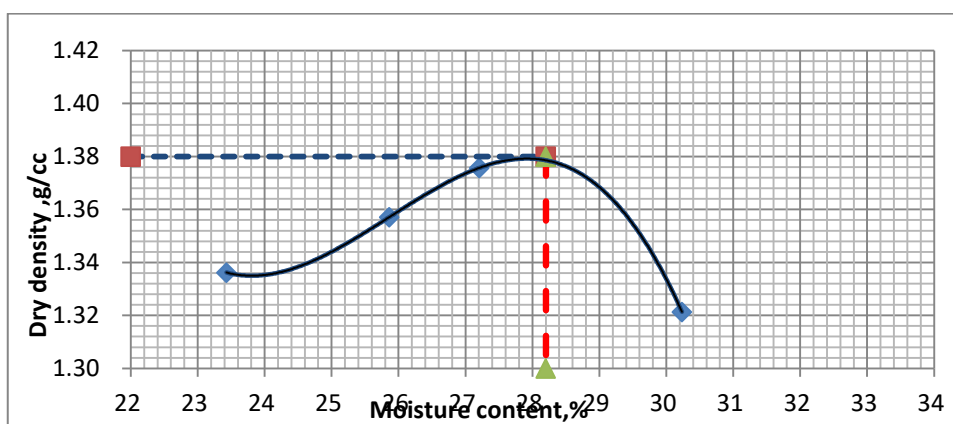
Appendix A4.4:-Atterberg Limit Test Analysis of 20% MCA+10% WB

<i>Test method of Atterberg Limit AASHTO T-89 and AASHTO T-90</i>							
LIQUID LIMIT					PLASTIC LIMIT		
Container No.	15	21	22		16	73	Avg
Wt of wet soil + container, gm	33.670	33.04	33.02		12.93	12.62	
Wt of dry soil + container, gm	24.710	24.06	23.84		11.87	11.72	
Wt of water	8.96	8.98	9.18		1.06	0.90	
Wt of container	9.80	10.01	9.86		9.410	9.610	
Wt of dry soil, gm	14.91	14.05	13.98		2.46	2.11	

Water content, %	60.09	63.91	65.67	63.	43.09	42.65	42.9
No. of blows	33	26	19				
LIQUID LIMIT (%)	63.7						
PLASTIC LIMIT (%)	42.9						
PLASTIC INDEX (%)	21						

APPENDIX A5:-COMPACTION TEST ANALYSIS OF NATURAL SOIL

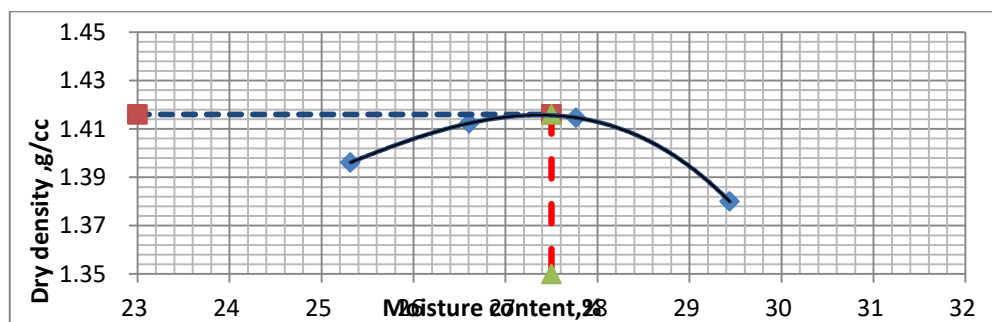
Dry Density/Moisture Content Relationship (Aashto T 180 Method B Or D)					
Blow Of Layer	56	No.layer	5	Rammer weight=4.54kg	
Mold daimeter	152mm	Volume of mold (Cm ³)=2124			
Percent Water Added		10%	14%	18%	22%
Weight Of Wet Soil + Mold	(gm)	9791.0	9916.0	10005.0	9943.0
Weight Of Mold	(gm)	6288.0			
Weight Of Wet Soil	(gm)	3503.0	3628.0	3717.0	3655.0
Wet Density	(gm/cm3)	1.65	1.71	1.75	1.72
Moisture Content : Container No.		A-7	B-2	A-2	A-3
Weight Of Wet Soil + Container	(gm)	207.60	215.80	216.30	209.50
Weight Of Dry Soil + Container	(gm)	172.90	176.80	175.15	166.90
Weight Of Container	(gm)	24.80	26.00	23.90	26.00
Weight Of Moisture	(gm)	34.70	39.00	41.15	42.60
Weight Of Dry Soil	(gm)	148.10	150.80	151.25	140.90
Moisture Content	(%)	23.4	25.9	27.2	30.2
Dry Density	(gm/cm3)	1.34	1.36	1.38	1.32
MDD	1.381gm/cc				
OMC	28.2%				



Appendix A5.1: Compaction Test Analysis Of 5% Maize Cob Ash

Dry Density/Moisture Content Relationship (Aashto T 180 Method B Or D)
--

Blow Of Layer	56	No.layer	5	Rammer weight=4.54kg	
Mold daimeter	152mm		Volume of mold (Cm ³)=2124		
Percent Water Added		10%	14%	18%	22%
Weight Of Wet Soil + Mold	(gm)	10004.0	10086.0	10127.0	10082.0
Weight Of Mold	(gm)	6288.0			
Weight Of Wet Soil	(gm)	3716.0	3798.0	3839.0	3794.0
Wet Density	(gm/cm3)	1.75	1.79	1.81	1.79
Moisture Content : Container No.		F-3	C-4	B-7	C_3
Weight Of Wet Soil + Container	(gm)	215.90	206.30	223.30	212.90
Weight Of Dry Soil + Container	(gm)	177.60	168.20	180.40	170.60
Weight Of Container	(gm)	26.30	25.00	25.90	26.90
Weight Of Moisture	(gm)	38.30	38.10	42.90	42.30
Weight Of Dry Soil	(gm)	151.30	143.20	154.50	143.70
Moisture Content	(%)	25.3	26.6	27.8	29.4
Dry Density	(gm/cm3)	1.40	1.41	1.41	1.38
MDD	1.42gm/cc				
OMC	27.5%				



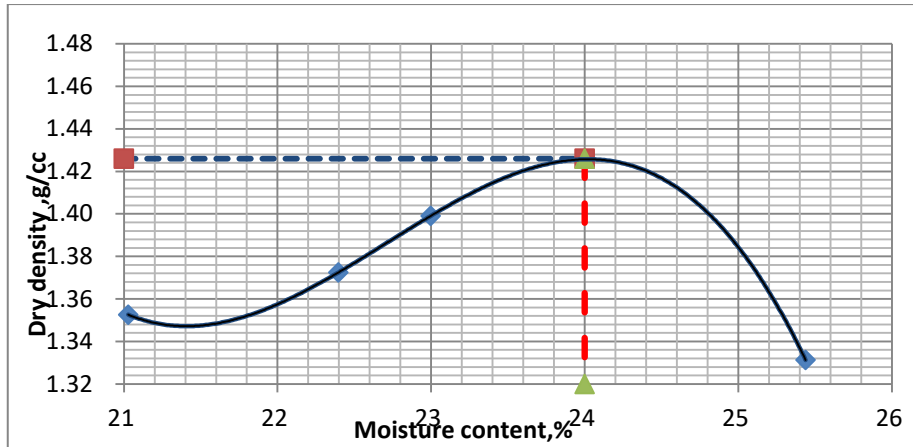
Appendix A5.2:Compaction Test Analysis Of 10% Maize Cob Ash

Dry Density/Moisture Content Relationship (Aashto T 180 Method B Or D)					
Blow Of Layer	56	No.layer	5	Rammer weight=4.54kg	
Mold daimeter	152mm		Volume of mold (Cm ³)=2124		
Percent Water Added		10%	14%	18%	22%
Weight Of Wet Soil + Mold	(gm)	9852.0	9969.0	10070.0	9806.0
Weight Of Mold	(gm)	6888			
Weight Of Wet Soil	(gm)	3564.0	3681.0	3782.0	3518.0
Wet Density	(gm/cm3)	1.68	1.73	1.78	1.66

Moisture Content : Container No.		B-2	B-6	A-5	E-1
Weight Of Wet Soil + Container	(gm)	227.30	229.80	221.30	229.00
Weight Of Dry Soil + Container	(gm)	190.60	190.80	182.65	188.00
Weight Of Container	(gm)	25.40	24.40	26.40	26.90
Weight Of Moisture	(gm)	36.70	39.00	38.65	41.00
Weight Of Dry Soil	(gm)	165.20	166.40	156.25	161.10
Moisture Content	(%)	22.2	23.4	24.7	25.5
Dry Density	(gm/cm ³)	1.35	1.37	1.40	1.43
MDD	1.432gm/cc				
OMC	24%				

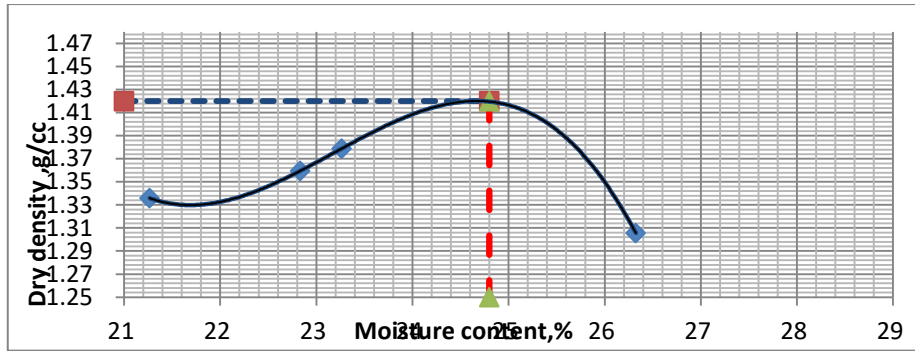
Appendix A5.3: Compaction Test Analysis Of 15% Maize Cob Ash

Dry Density/Moisture Content Relationship (Aashto T 180 Method B Or D)					
Blow Of Layer	56	No.layer	5	Rammer weight=4.54kg	
Mold daimeter	152mm		Volume of mold (Cm³)=2124		
Percent Water Added		10%	14%	18%	22%
Weight Of Wet Soil + Mold	(gm)	9763.0	9854.0	9941.0	9833.0
Weight Of Mold	(gm)	6286.0			
Weight Of Wet Soil	(gm)	3477.0	3568.0	3655.0	3547.0
Wet Density	(gm/cm3)	1.64	1.68	1.72	1.67
Moisture Content : Container No.		C-2	D-2	F-3	C_4
Weight Of Wet Soil + Container	(gm)	210.90	202.63	216.30	205.90
Weight Of Dry Soil + Container	(gm)	178.60	170.20	180.70	169.60
Weight Of Container	(gm)	25.00	25.40	25.90	26.90
Weight Of Moisture	(gm)	32.30	32.43	35.60	36.30
Weight Of Dry Soil	(gm)	153.60	144.80	154.80	142.70
Moisture Content	(%)	21.0	22.4	23.0	25.4
Dry Density	(gm/cm3)	1.35	1.37	1.40	1.33
MDD	1.423 gm/cc				
OMC	24 %				



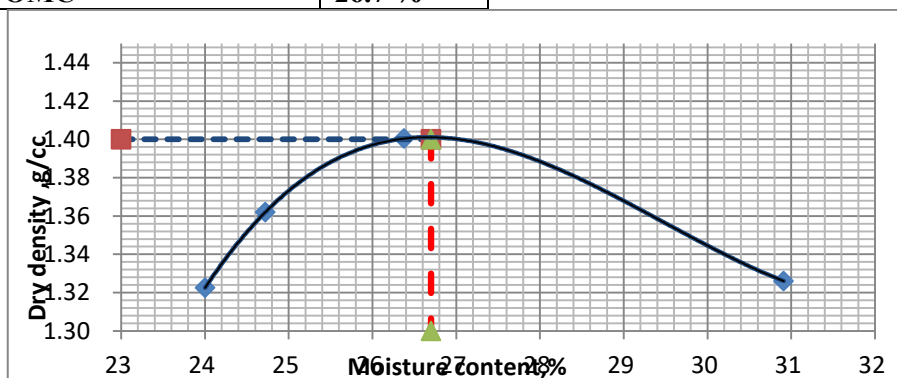
APPendix A5.4 : Compaction Test Analysis Of 20% Maize Cob Ash

Dry Density/Moisture Content Relationship (Aashto T 180 Method B Or D)						
Blow Of Layer	56	No.layer	5	Rammer weight=4.54kg		
Mold daimeter	152mm		Volume of mold (Cm ³)=2124			
Percent Water Added		10%	14%	18%	22%	
Weight Of Wet Soil + Mold	(gm)	9679.0	9789.0	9966.0	9789.0	
Weight Of Mold	(gm)	6286.0				
Weight Of Wet Soil	(gm)	3440.9	3547.0	3610.0	3503.0	
Wet Density	(gm/cm3)	1.62	1.67	1.70	1.65	
Moisture Content : Container No.		A-2	B-2	33	D-4	
Weight Of Wet Soil + Container	(gm)	210.90	203.63	214.00	205.90	
Weight Of Dry Soil + Container	(gm)	178.30	170.50	178.50	168.60	
Weight Of Container	(gm)	25.00	25.40	25.90	26.90	
Weight Of Moisture	(gm)	32.60	33.13	35.50	37.30	
Weight Of Dry Soil	(gm)	153.30	145.10	152.60	141.70	
Moisture Content	(%)		21.3	22.8	23.3	26.3
Dry Density	(gm/cm3)		1.34	1.36	1.38	1.31
MDD	1.42 gm/cc					
OMC	24.8 %					



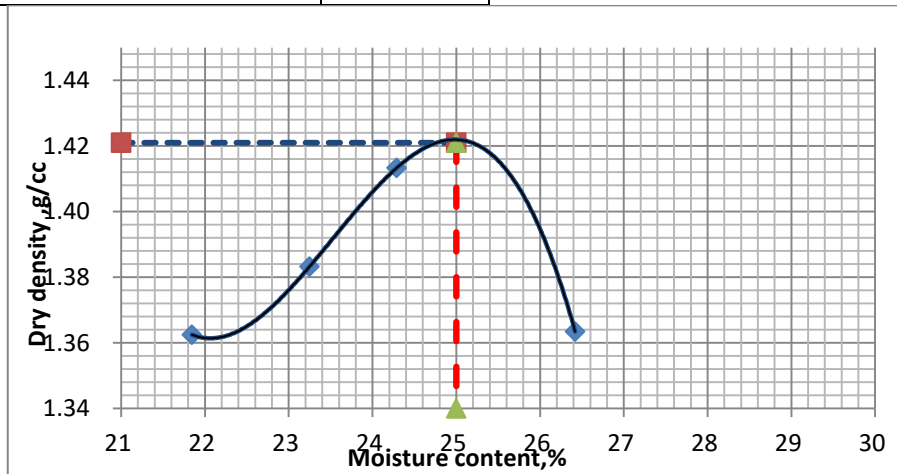
Appendix A6.1 : Compaction Test Analysis of 10% Waste brick

Dry Density/Moisture Content Relationship (Aashto T 180 Method B Or D)					
Blow Of Layer	56	No.layer	5	Rammer weight=4.54kg	
Mold daimeter	152mm		Volume of mold (Cm ³)=2124		
Weight Of Wet Soil + Mold	(gm)	9776.0	9896.0	10047.0	9975.0
Weight Of Mold	(gm)	6288.0			
Weight Of Wet Soil	(gm)	3488.0	3608.0	3759.0	3687.0
Wet Density	(gm/cm3)	1.64	1.70	1.77	1.74
Moisture Content : Container No.		C-5	A-2	B-17	B-27
Weight Of Wet Soil + Container	(gm)	219.00	214.00	221.00	206.00
Weight Of Dry Soil + Container	(gm)	181.70	176.30	180.30	163.50
Weight Of Container	(gm)	26.30	23.80	26.00	26.00
Weight Of Moisture	(gm)	37.30	37.70	40.70	42.50
Weight Of Dry Soil	(gm)	155.40	152.50	154.30	137.50
Moisture Content	(%)	24.0	24.7	26.4	30.9
Dry Density	(gm/cm3)	1.32	1.36	1.40	1.33
MDD	1.4 gm/cc				
OMC	26.7 %				



Appendix A6.2 : Compaction Test Analysis of 20% Waste brick

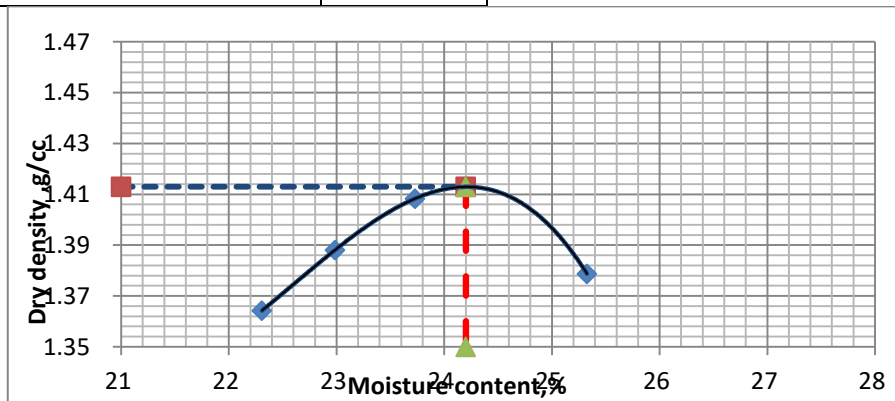
Dry Density/Moisture Content Relationship (Aashto T 180 Method B Or D)					
Blow Of Layer	56	No.layer	5	Rammer weight=4.54kg	
Mold daimeter	152mm		Volume of mold (Cm ³)=2124		
Weight Of Wet Soil + Mold	(gm)	9814.	9909.	10019.0	9949.0
Weight Of Mold	(gm)	6288.0			
Weight Of Wet Soil	(gm)	3526.0	3621.0	3731.0	3661.0
Wet Density	(gm/cm3)	1.66	1.70	1.76	1.72
Moisture Content : Container No.		A-8	B-7	A-5	18
Weight Of Wet Soil + Container	(gm)	238.00	208.50	223.10	213.10
Weight Of Dry Soil + Container	(gm)	200.10	174.00	184.70	174.00
Weight Of Container	(gm)	26.60	25.60	26.60	26.00
Weight Of Moisture	(gm)	37.90	34.50	38.40	39.10
Weight Of Dry Soil	(gm)	173.50	148.40	158.10	148.00
Moisture Content	(%)	21.8	23.2	24.3	26.4
Dry Density	(gm/cm3)	1.36	1.38	1.41	1.36
MDD	1.42 gm/cc				
OMC	25 %				



Appendix 6.3 : Compaction Test Analysis of 30% Waste brick

Dry Density/Moisture Content Relationship (Aashto T 180 Method B Or D)					
Blow Of Layer	56	No.layer	5	Rammer weight=4.54kg	
Mold daimeter	152mm		Volume of mold (Cm ³)=2124		
Weight Of Wet Soil + Mold	(gm)	9832.0	9914.0	9989.0	9958.0
Weight Of Mold	(gm)	6288.0			

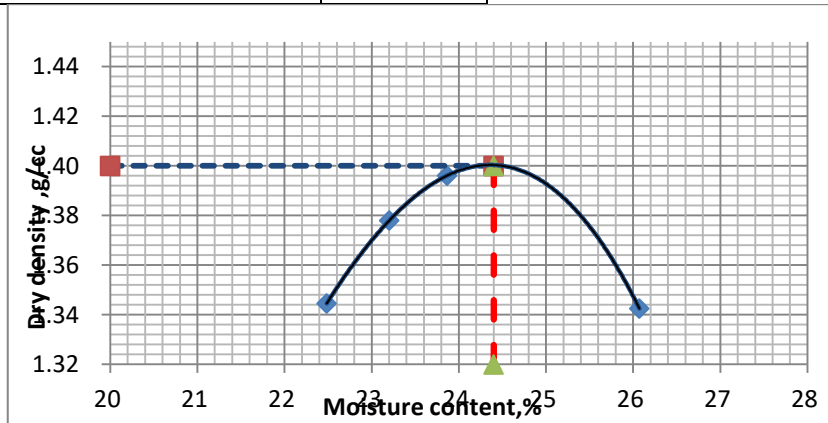
Weight Of Wet Soil	(gm)	3544.0	3626.0	3701.0	3670.0
Wet Density	(gm/cm ³)	1.67	1.71	1.74	1.73
Moisture Content : Container No.		A-7	B-6	C-4	F-3
Weight Of Wet Soil + Container	(gm)	217.80	223.00	229.00	218.30
Weight Of Dry Soil + Container	(gm)	182.60	185.90	189.80	179.40
Weight Of Container	(gm)	24.80	24.50	24.60	25.80
Weight Of Moisture	(gm)	35.20	37.10	39.20	38.90
Weight Of Dry Soil	(gm)	157.80	161.40	165.20	153.60
Moisture Content	(%)	22.3	23.0	23.7	25.3
Dry Density	(gm/cm ³)	1.36	1.39	1.41	1.38
MDD	1.41gm/cc				
OMC	24.2%				



Appendix A6.4: Compaction Test Analysis of 40% Waste brick

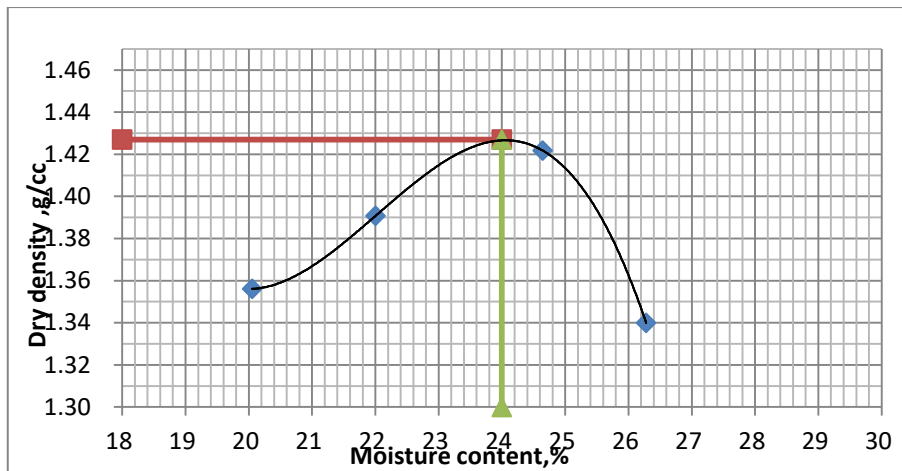
Dry Density/Moisture Content Relationship (Aashto T 180 Method B Or D)					
Blow Of Layer	56	No.layer	5	Rammer weight=4.54kg	
Mold daimeter	152mm		Volume of mold (Cm ³)=2124		
Weight Of Wet Soil + Mold	(gm)	9786.0	9894.0	9961.0	9883.0
Weight Of Mold	(gm)	6288.0			
Weight Of Wet Soil	(gm)	3498.0	3606.0	3673.0	3595.0
Wet Density	(gm/cm3)	1.65	1.70	1.73	1.69
Moisture Content : Container No.		C-5	A-7	A-1	K_
Weight Of Wet Soil + Container	(gm)	201.90	213.30	203.10	204.80
Weight Of Dry Soil + Container	(gm)	169.50	177.80	168.90	167.70
Weight Of Container	(gm)	25.40	24.80	25.60	25.40
Weight Of Moisture	(gm)	32.40	35.50	34.20	37.10

Weight Of Dry Soil	(gm)	144.10	153.00	143.30	142.30
Moisture Content	(%)	22.5	23.2	23.9	26.1
Dry Density	(gm/cm ³)	1.34	1.38	1.40	1.34
MDD	1.4gm/cc				
OMC	24%				



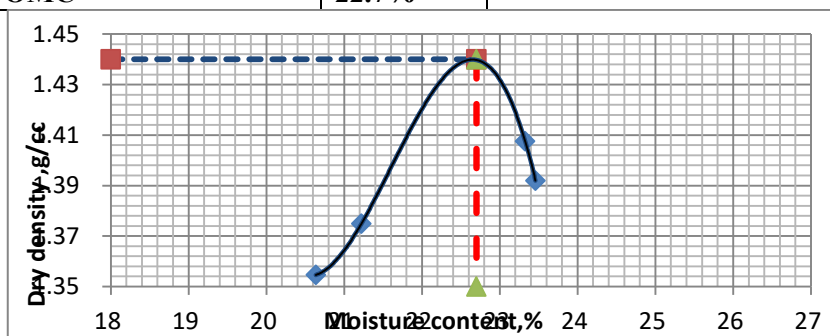
Appendix A7.1: Compaction test Analysis Of 5%Mca+40%Wb

Dry Density/Moisture Content Relationship (Aashto T 180 Method B Or D)					
Blow Of Layer	56	No.layer	5	Rammer weight=4.54kg	
Mold daimeter	152mm		Volume of mold (Cm ³)=2124		
Weight Of Wet Soil + Mold	(gm)	9740.0	9886.0	10046.0	9876.0
Weight Of Mold	(gm)	6282.0			
Weight Of Wet Soil	(gm)	3458.0	3604.0	3764.0	3594.0
Wet Density	(gm/cm3)	1.63	1.70	1.77	1.69
Moisture Content : Container No.		k-1	B-7	B-8	A_2
Weight Of Wet Soil + Container	(gm)	245.20	218.50	193.20	208.90
Weight Of Dry Soil + Container	(gm)	208.50	183.80	160.20	170.40
Weight Of Container	(gm)	25.50	26.10	26.30	23.90
Weight Of Moisture	(gm)	36.70	34.70	33.00	38.50
Weight Of Dry Soil	(gm)	183.00	157.70	133.90	146.50
Moisture Content	(%)	20.1	22.0	24.6	26.3
Dry Density	(gm/cm3)	1.36	1.39	1.42	1.34
MDD	1.427gm/cc				
OMC	24%				



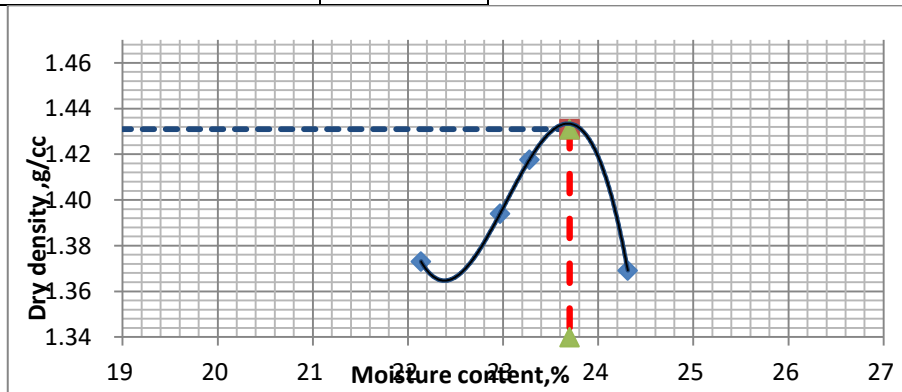
Appendix A7.2: Compaction test Analysis Of 10%Mca+30%Wb

Dry Density/Moisture Content Relationship (Aashto T 180 Method B Or D)					
Blow Of Layer	56	No.layer	5	Rammer weight=4.54kg	
Mold daimeter	152mm		Volume of mold (Cm ³)=2124		
Weight Of Wet Soil + Mold	(gm)	9753.0	9822.0	9969.0	9932.0
Weight Of Mold	(gm)	6282.0			
Weight Of Wet Soil	(gm)	3471.0	3540.0	3687.0	3650.0
Wet Density	(gm/cm3)	1.63	1.67	1.74	1.72
Moisture Content : Container No.		C-2	D-2	A-11	C-1
Weight Of Wet Soil + Container	(gm)	206.60	184.00	193.00	204.00
Weight Of Dry Soil + Container	(gm)	175.70	156.40	162.30	168.50
Weight Of Container	(gm)	25.00	25.40	24.90	25.70
Weight Of Moisture	(gm)	30.90	27.60	30.70	35.50
Weight Of Dry Soil	(gm)	150.70	131.00	137.40	142.80
Moisture Content	(%)	20.5	21.1	22.3	24.9
Dry Density	(gm/cm3)	1.36	1.38	1.42	1.38
MDD	1.44gm/cc				
OMC	22.7%				



Appendix A7.3: Compaction test Analysis Of 15%Mca+20%Wb

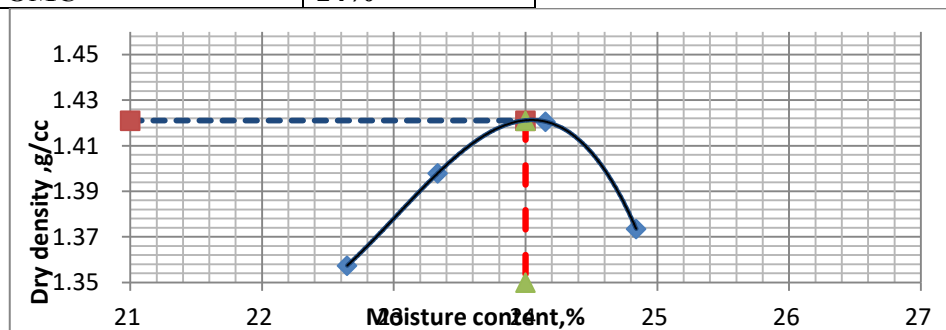
Dry Density/Moisture Content Relationship (Aashto T 180 Method B Or D)					
Blow Of Layer	56	No.layer	5	Rammer weight=4.54kg	
Mold daimeter	152mm		Volume of mold (Cm³)=2124		
Percent Water Added		10%	14%	18%	22%
Weight Of Wet Soil + Mold	(gm)	9842.0	9921.0	9992.0	9895.0
Weight Of Mold	(gm)	6280.0			
Weight Of Wet Soil	(gm)	3562.0	3641.0	3712.0	3615.0
Wet Density	(gm/cm3)	1.68	1.71	1.75	1.70
Moisture Content : Container No.		E-1	B-2	A-2	02
Weight Of Wet Soil + Container	(gm)	233.20	226.70	217.50	220.20
Weight Of Dry Soil + Container	(gm)	195.70	189.10	181.00	182.30
Weight Of Container	(gm)	26.30	25.40	24.20	26.40
Weight Of Moisture	(gm)	37.50	37.60	36.70	37.90
Weight Of Dry Soil	(gm)	169.40	163.70	156.80	155.90
Moisture Content	(%)	22.1	23.0	23.4	24.3
Dry Density	(gm/cm3)	1.37	1.39	1.42	1.37
MDD	1.431gm/cc				
OMC	23.7%				



Appendix A7.4: Compaction test Analysis Of 20%Mca+10%Wb (S1)

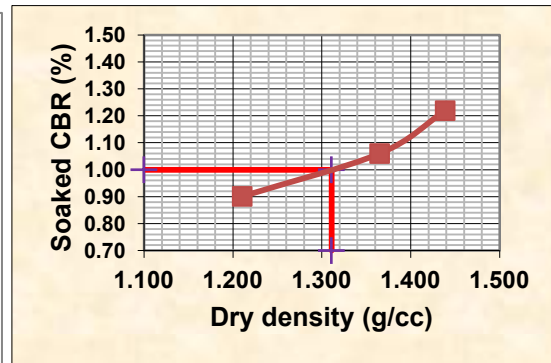
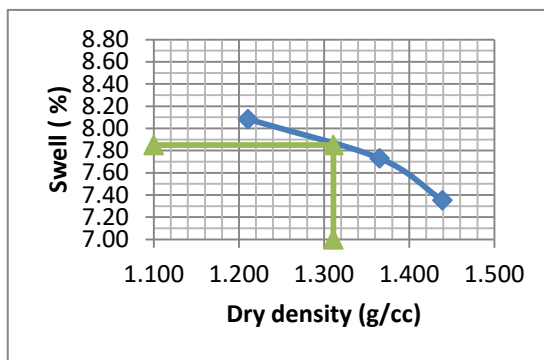
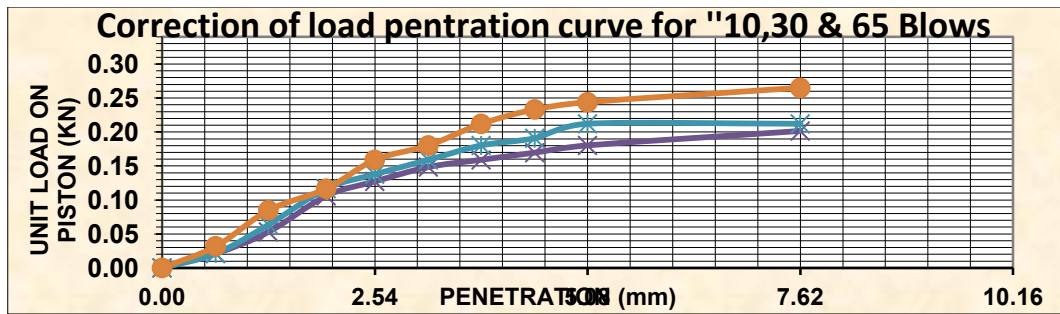
Dry Density/Moisture Content Relationship (Aashto T 180 Method B Or D)					
Blow Of Layer	56	No.layer	5	Rammer weight=4.54kg	
Mold daimeter	152mm		Volume of mold (Cm ³)=2124		
Percent Water Added			10%	14%	18% 22%
Weight Of Wet Soil + Mold	(gm)		9816.0	9942.0	10026.0 9922.0
Weight Of Mold	(gm)		6280.0		

Weight Of Wet Soil	(gm)	3536.0	3662.0	3746.0	3642.0
Wet Density	(gm/cm ³)	1.66	1.72	1.76	1.71
Moisture Content : Container No.		A-7	A-11	C-3	D-1
Weight Of Wet Soil + Container	(gm)	195.40	210.60	213.20	220.40
Weight Of Dry Soil + Container	(gm)	163.90	175.60	176.90	181.70
Weight Of Container	(gm)	24.80	25.60	26.60	25.90
Weight Of Moisture	(gm)	31.50	35.00	36.30	38.70
Weight Of Dry Soil	(gm)	139.10	150.00	150.30	155.80
Moisture Content	(%)	22.6	23.3	24.2	24.8
Dry Density	(gm/cm ³)	1.36	1.40	1.42	1.37
MDD	1.42 gm/cc				
OMC	24%				



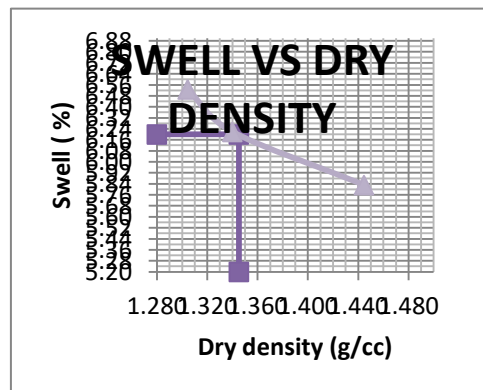
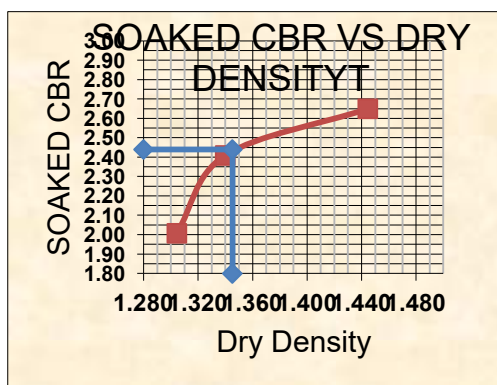
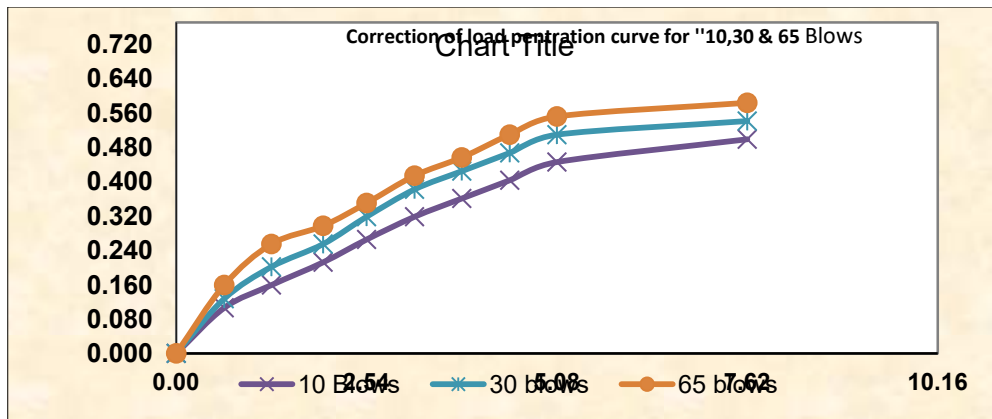
Appendix A8: CBR Test Analysis of Natural soil

California Bearing Ratio - Data Sheet (TEST METHOD AASHTO T-193)									
CBR Ring,(KN) :-			30			Socking date:-			
Surcharge Weight, (Kg) :-			4.50			Testing date:-			
Ring Factor (KN/div) :-			0.106			Time:-			
Blows	Cor. Load (Kn)		CBR (%)		Swell %	Dry Density Vs Socked C.B.R.			
	2.54m m	5.08m m	2.54m m	5.08 mm		No BLOWS	10	30	65
10	0.0096	0.009	0.96	0.90	8.082	Swell	8.082	7.730	7.352
30	0.0104	0.0106	1.04	1.06	7.730	Dry density	1.211	1.365	1.439
65	0.0120	0.0122	1.20	1.22	7.352	Soaked CBR	0.90	1.06	1.22
MODIFIED PROCTOR : T 180 - D									
MDD (g/cc) :			1.380						
OMC (%) :			28.20						
95% of MDD(g/cc)			1.311						
CBR % at 95% of MDD			1.0						
Swell % at 95 of MDD			7.85						



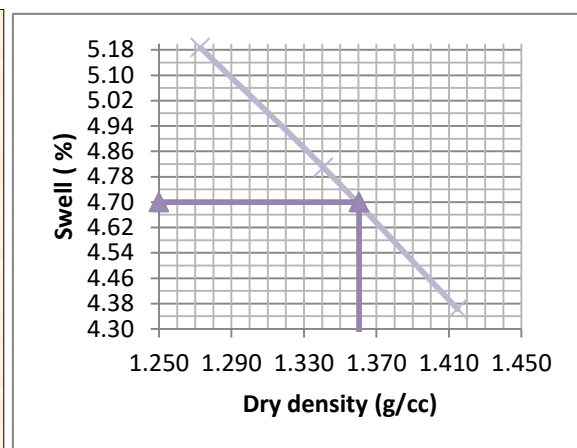
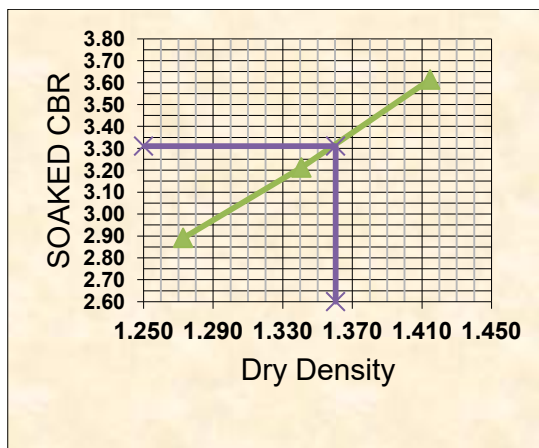
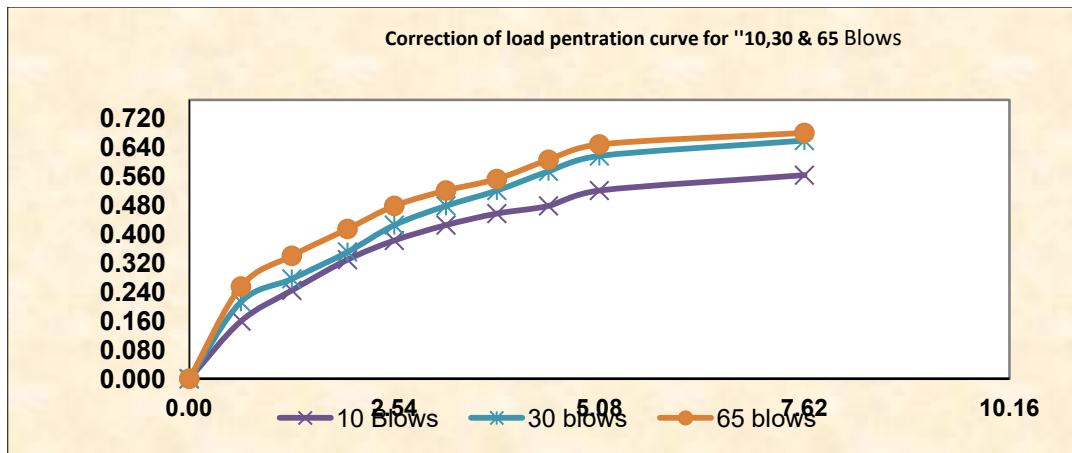
Appendix A8.1:- CBR Test Analysis Of 5%Mca

California Bearing Ratio - Data Sheet (TEST METHOD AASHTO T-193)									
CBR Ring,(KN) :-		30		Socking date:-		9/10/14			
Surcharge Weight, (Kg) :-		4.50		Testing date:-		13/10/14			
Ring Factor (KN/div) :-		0.106		Time:-		10:10am			
Blows	Cor. Load (Kn)		CBR (%)		Swell %	Dry Density Vs Socked C.B.R.			
	2.54m m	5.08m m	2.54m m	5.08 mm		No _____ blow BLOWS	10	30	65
10	0.0201	0.0223	2.01	2.23	6.528	Swell, (%)	6.528	6.218	5.832
30	0.0241	0.0254	2.41	2.54	6.218	DryDensity,(g/ cm3)	1.304	1.340	1.445
65	0.0265	0.0276	2.65	2.76	5.832	SockedCbr, (%)	2.01	2.41	2.65
MODIFIED PROCTOR : T 180 - D									
MDD (g/cc) :		1.416							
OMC (%) :		27.50							
95% of MDD(g/cc)		1.345							
CBR % at 95% of MDD		2.4							
Swell % at 95 of MDD		6.2							



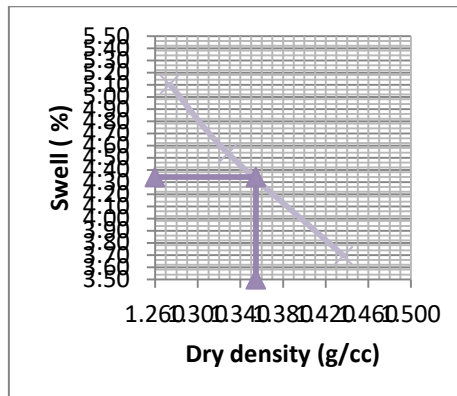
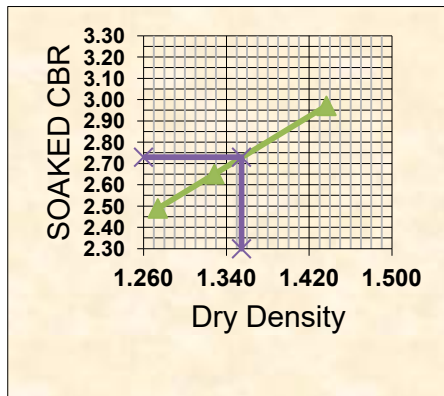
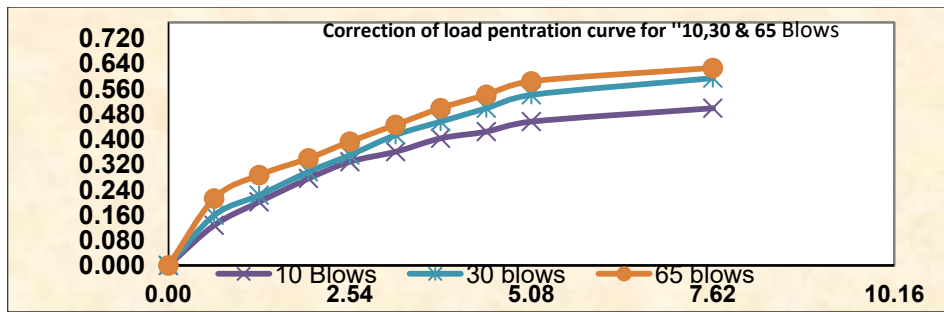
Appendix 8.2:- Car Test Analysis Of 10%Mca

California Bearing Ratio - Data Sheet (TEST METHOD AASHTO T-193)									
CBR Ring,(KN) :-			30			Socking date:-		9/10/14	
Surcharge Weight, (Kg) :-			4.50			Testing date:-		13/10/14	
Ring Factor (KN/div) :-			0.106			Time:-		10:10am	
Blows	Cor. Load (Kn)		CBR (%)		Swell %	Dry Density Vs Socked C.B.R.			
	2.54m m	5.08m m	2.54m m	5.08 mm		No BLOWS	10	30	65
10	0.0289	0.026	2.89	2.60	5.188	Swell, (%)	5.188	4.810	4.363
30	0.0321	0.0307	3.21	3.07	4.810	DryDensity,(g/ cm3)	1.273	1.341	1.415
65	0.0361	0.0323	3.61	3.23	4.363	SockedCbr, (%)	2.89	3.21	3.61
MODIFIED PROCTOR : T 180 - D									
MDD (g/cc) :			1.432						
OMC (%) :			24.50						
95% of MDD(g/cc)			1.360						
CBR % at 95% of MDD			3.3						
Swell % at 95 of MDD			4.7						



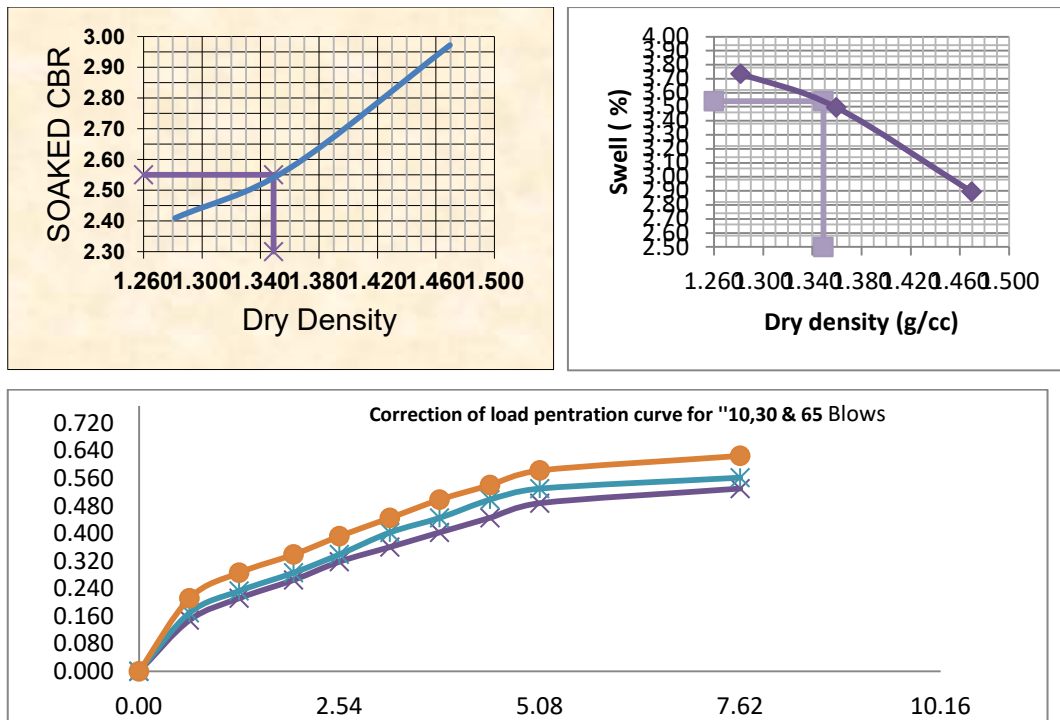
Appendix 8.3:- Car Test Analysis Of 15%Mca

California Bearing Ratio - Data Sheet (TEST METHOD AASHTO T-193)									
CBR Ring,(KN) :-		30		Socking date:-		9/10/14			
Surcharge Weight, (Kg) :-		4.50		Testing date:-		13/10/14			
Ring Factor (KN/div) :-		0.106		Time:-		10:10am			
Blows	Cor. Load (Kn)		CBR (%)		Swell %	Dry Density Vs Socked C.B.R.			
	2.54m m	5.08m m	2.54m m	5.08 mm		No BLOWS	10	30	65
10	0.0289	0.026	2.89	2.60	5.188	Swell, (%)	5.188	4.810	4.363
30	0.0321	0.0307	3.21	3.07	4.810	DryDensity,(g/ cm3)	1.273	1.341	1.415
65	0.0361	0.0323	3.61	3.23	4.363	SockedCbr, (%)	2.89	3.21	3.61
MODIFIED PROCTOR : T 180 - D									
MDD (g/cc) :		1.432							
OMC (%) :		24.50							
95% of MDD(g/cc)		1.360							
CBR % at 95% of MDD		3.3							
Swell % at 95 of MDD		4.7							



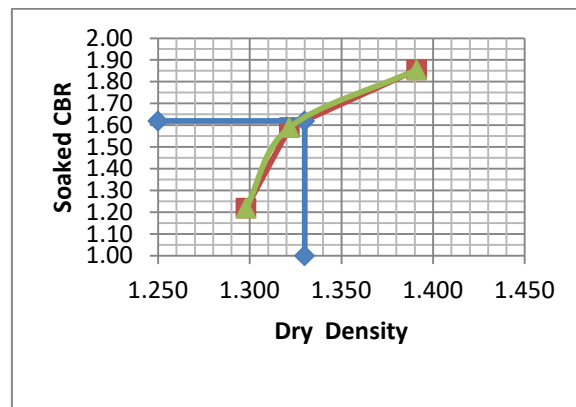
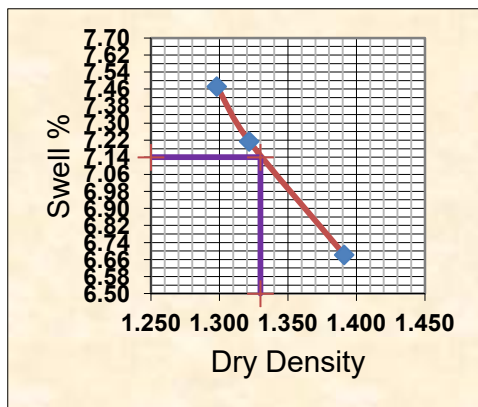
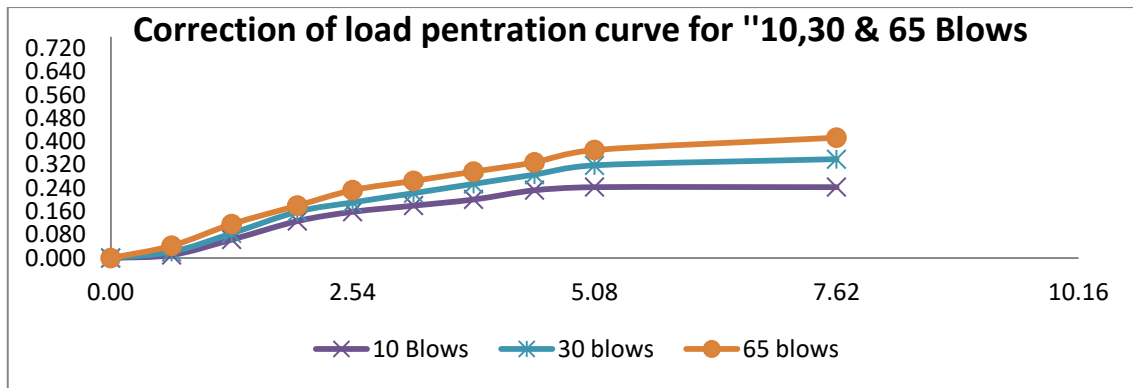
APPENDIX 8.4:-CBR TEST ANALYSIS OF 1 20%MCA

California Bearing Ratio - Data Sheet (TEST METHOD AASHTO T-193)									
CBR Ring, (KN) :-		30		Socking date:-		9/10/14			
Surcharge Weight, (Kg) :-		4.50		Testing date:-		13/10/14			
Ring Factor (KN/div) :-		0.106		Time:-		10:10am			
Blows	Cor. Load (Kn)		CBR (%)		Swell %	Dry Density Vs Socked C.B.R.			
	2.54m m	5.08m m	2.54m m	5.08 mm		No BLOWS	10	30	65
10	0.0241	0.0244	2.41	2.44	3.736	Swell, (%)	3.736	3.496	2.894
30	0.0257	0.0265	2.57	2.65	3.496	DryDensity, (g/ cm ³)	1.282	1.359	1.470
65	0.0297	0.0297	2.97	2.97	2.894	Socked Cbr, (%)	2.41	2.57	2.97
MODIFIED PROCTOR : T 180 - D									
MDD (g/cc) :		1.420							
OMC (%) :		24.80							
95% of MDD(g/cc)		1.349							
CBR % at 95% of MDD		2.6							
Swell % at 95 of MDD		3.5							



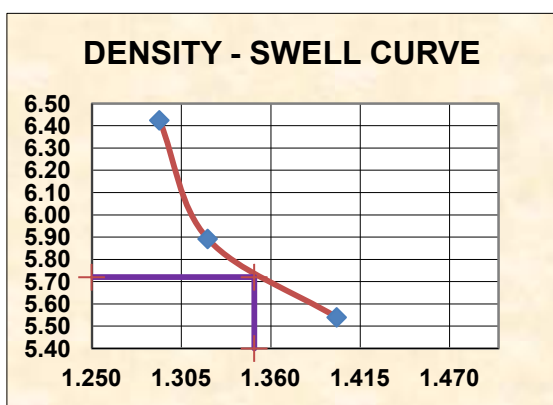
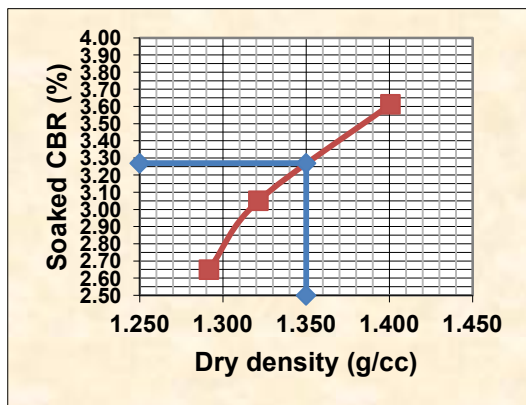
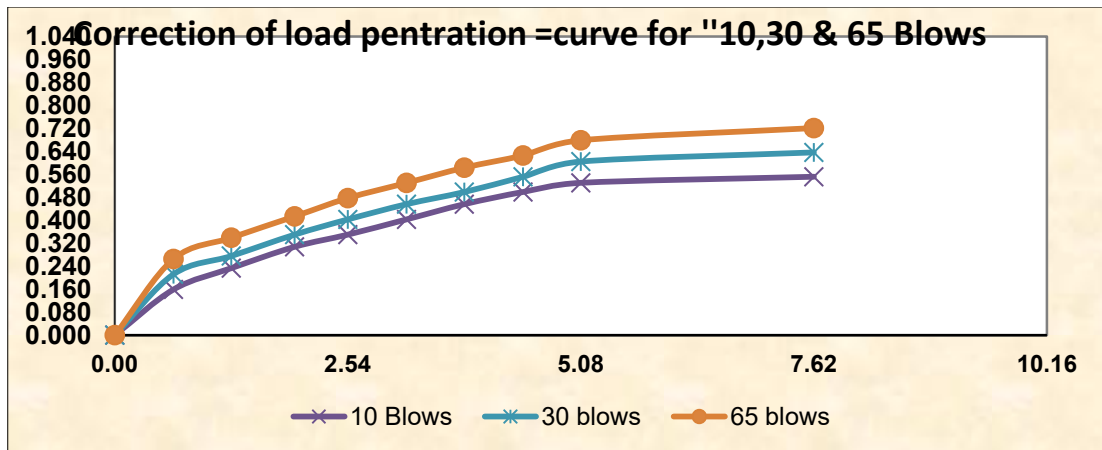
Appendix A9.1: CBR Test Analysis Of 10%Wb

California Bearing Ratio - Data Sheet (TEST METHOD AASHTO T-193)									
CBR Ring,(KN) :-		30		Socking date:-					
Surcharge Weight, (Kg) :-		4.50		Testing date:-					
Ring Factor (KN/div) :-		0.106		Time:-					
Blows	Cor. Load (Kn)		CBR (%)		Swell %	Dry Density Vs Socked C.B.R.			
	2.54m m	5.08m m	2.54m m	5.08 mm		No BLOWS	10	30	65
10	0.0120	0.0122	1.20	1.22	7.472	Swell, (%)	7.472	7.215	6.682
30	0.0145	0.0159	1.45	1.59	7.215	DryDensity,(g/ cm3)	1.298	1.322	1.391
65	0.0177	0.0186	1.77	1.86	6.682	Socked Cbr, (%)	1.22	1.59	1.86
MODIFIED PROCTOR : T 180 - D									
MDD (g/cc) :		1.400							
OMC (%) :		26.70							
95% of MDD(g/cc)		1.330							
CBR % at 95% of MDD		1.6							
Swell % at 95 of MDD		7.14							



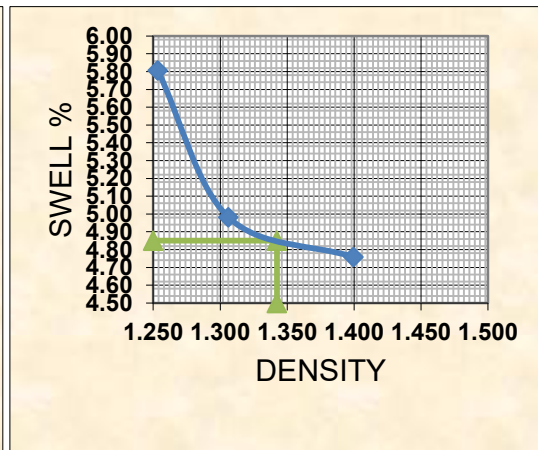
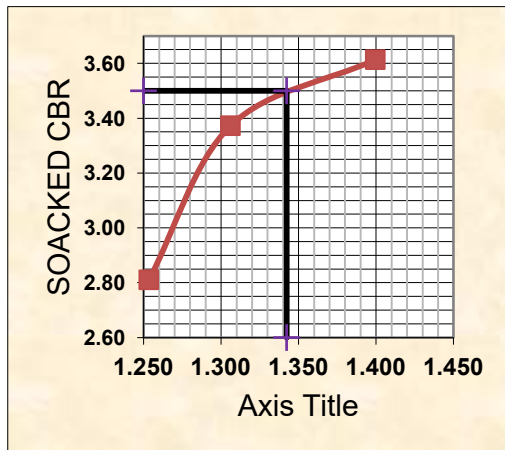
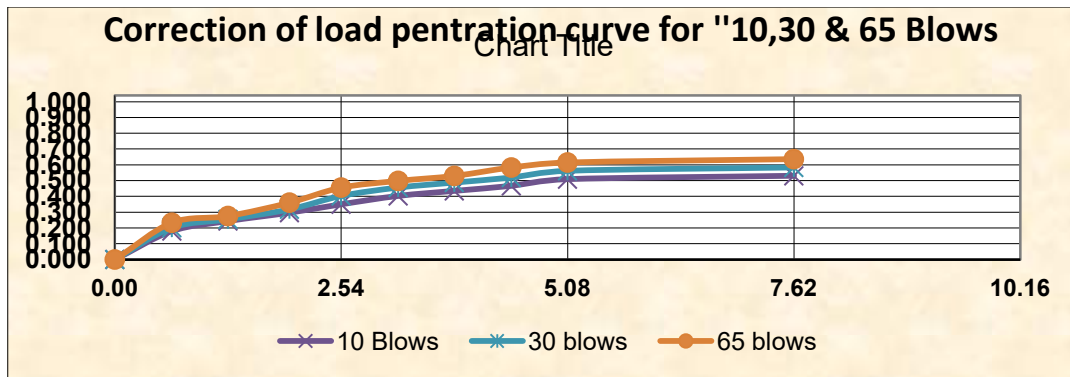
Appendix A9.2: Car Test Analysis Of 20%Wb

California Bearing Ratio - Data Sheet (TEST METHOD AASHTO T-193)									
CBR Ring,(KN) :-			30			Socking date:-			
Surcharge Weight, (Kg) :-			4.50			Testing date:-			
Ring Factor (KN/div) :-			0.106			Time:-			
Blows	Cor. Load (Kn)		CBR (%)		Swell %	Dry Density Vs Socked C.B.R.			
	2.54m m	5.08m m	2.54m m	5.08 mm		No BLOWS	10	30	65
10	0.0265	0.0265	2.65	2.65	6.424	Swell, (%)	6.424	5.892	5.540
30	0.0305	0.0302	3.05	3.02	5.892	DryDensity,(g/ cm3)	1.292	1.321	1.401
65	0.0361	0.0339	3.61	3.39	5.540	SockedCbr, (%)	2.65	3.05	3.61
MODIFIED PROCTOR : T 180 - D									
MDD (g/cc) :			1.421						
OMC (%) :			25%						
95% of MDD(g/cc)			1.35						
CBR % at 95% of MDD			3.3						
Swell % at 95 of MDD			5.7						



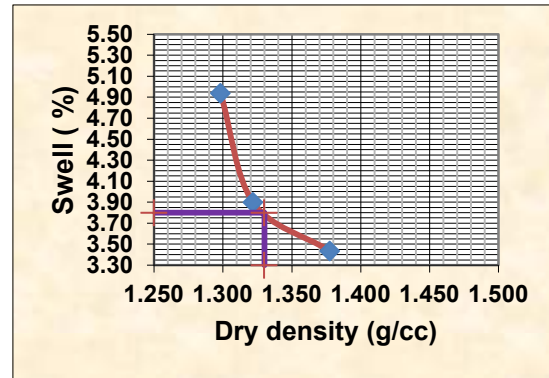
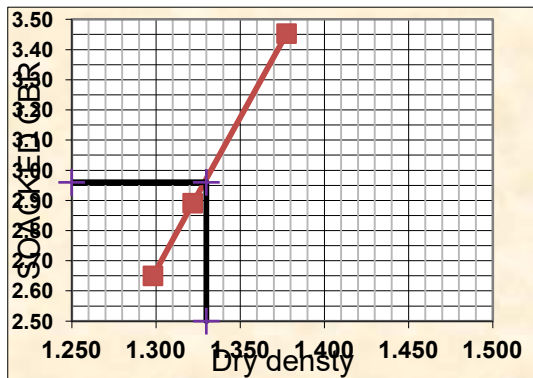
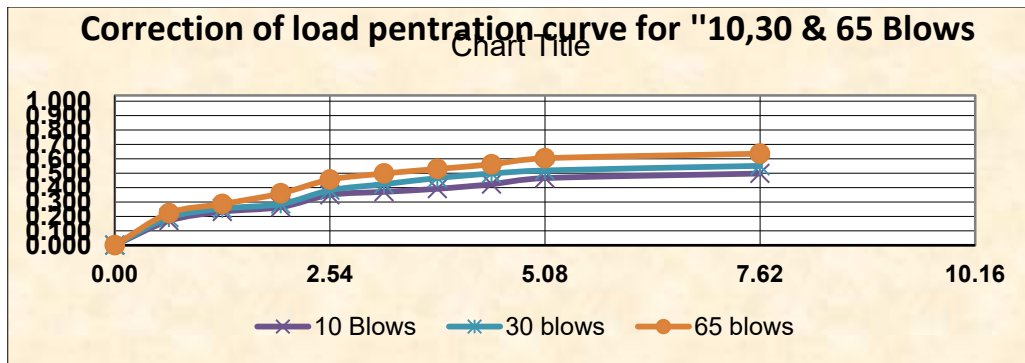
Appendix A9.3: CBR Test Analysis Of 30%Wb

California Bearing Ratio - Data Sheet (TEST METHOD AASHTO T-193)									
CBR Ring,(KN) :-			30			Socking date:-			
Surcharge Weight, (Kg) :-			4.50			Testing date:-			
Ring Factor (KN/div) :-			0.106			Time:-			
Blows	Cor. Load (Kn)		CBR (%)		Swell %	Dry Density Vs Socked C.B.R.			
	2.54m m	5.08m m	2.54m m	5.08 mm		No BLOWS	10	30	65
10	0.0281	0.0254	2.81	2.54	5.806	Swell, (%)	5.806	4.982	4.758
30	0.0337	0.0286	3.37	2.86	4.982	DryDensity,(g/ cm3)	1.253	1.306	1.400
65	0.0361	0.0313	3.61	3.13	4.758	SockedCbr, (%)	2.81	3.37	3.61
MODIFIED PROCTOR : T 180 - D									
MDD (g/cc) :			1.413						
OMC (%) :			24.20						
95% of MDD(g/cc)			1.342						
CBR % at 95% of MDD			3.5						
Swell % at 95 of MDD			4.9						



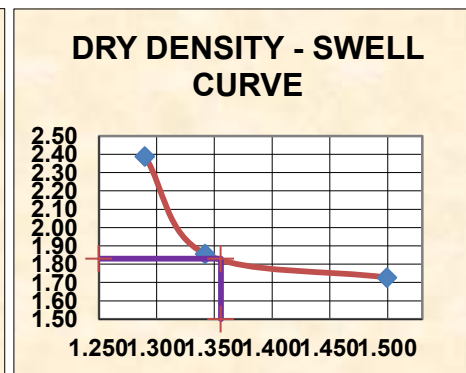
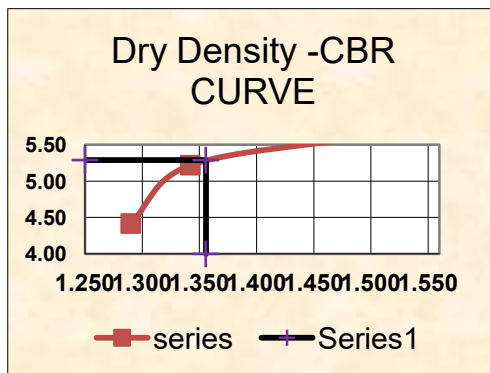
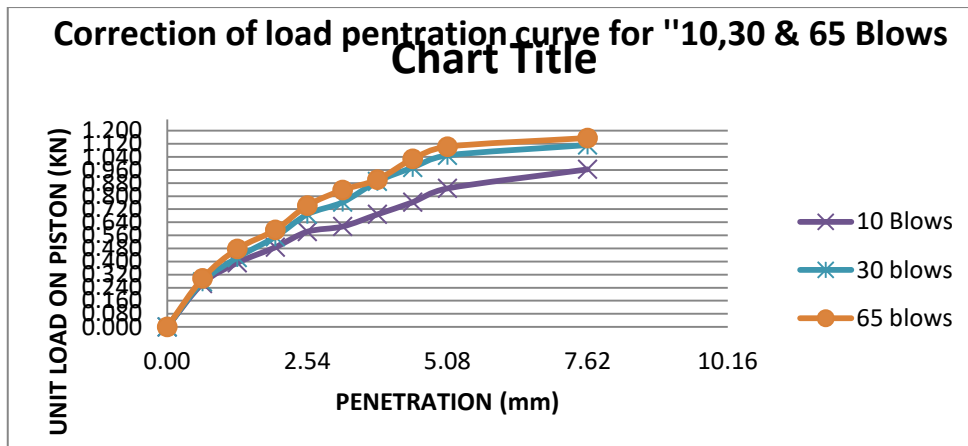
Appendix A9.4: Car Test Analysis Of 40%Wb

California Bearing Ratio - Data Sheet (TEST METHOD AASHTO T-193)									
CBR Ring,(KN) :-			30			Socking date:-			
Surcharge Weight, (Kg) :-			4.50			Testing date:-			
Ring Factor (KN/div) :-			0.106			Time:-			
Blows	Cor. Load (Kn)		CBR (%)		Swell %	Dry Density Vs Socked C.B.R.			
	2.54m m	5.08m m	2.54m m	5.08 mm		No BLOWS	10	30	65
10	0.0265	0.0233	2.65	2.33	4.939	Swell, (%)	4.939	3.899	3.436
30	0.0289	0.0260	2.89	2.60	3.899	DryDensity,(g/ cm3)	1.298	1.322	1.378
65	0.0345	0.0302	3.45	3.02	3.436	SockedCbr, (%)	2.65	2.89	3.45
MODIFIED PROCTOR : T 180 - D									
MDD (g/cc) :			1.400						
OMC (%) :			24.40						
95% of MDD(g/cc)			1.330						
CBR % at 95% of MDD			3.0						
Swell % at 95 of MDD			3.8						



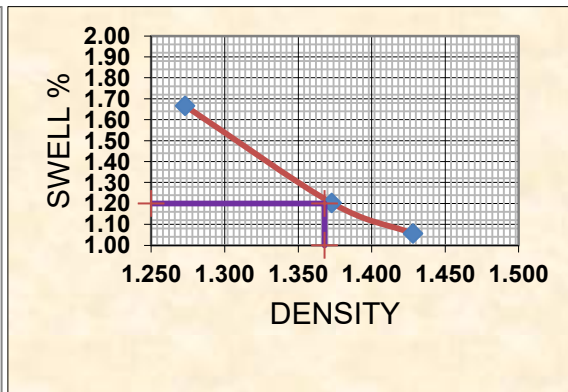
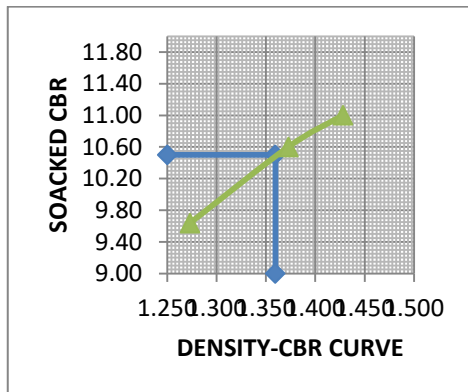
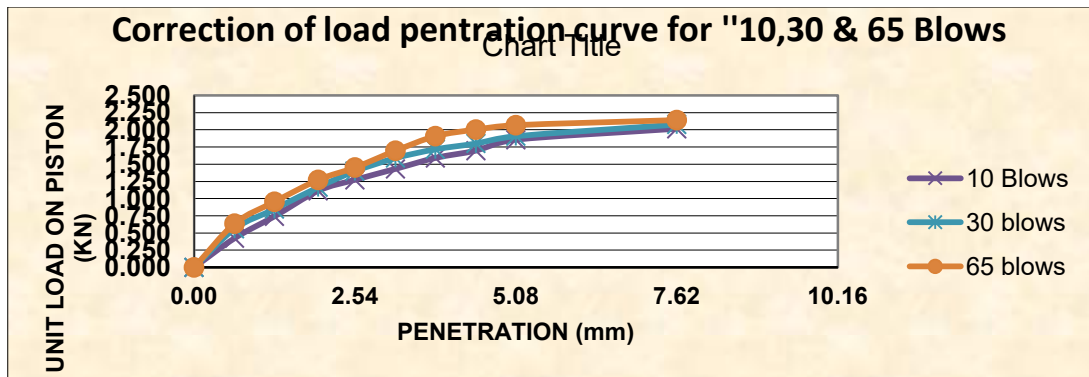
APPENDIX 10.1:-CBR TEST ANALYSIS OF 1 5%MCA +40%WB

California Bearing Ratio - Data Sheet (TEST METHOD AASHTO T-193)									
CBR Ring,(KN) :-		30		Socking date:-		18/10/14			
Surcharge Weight, (Kg) :-		4.50		Testing date:-		22/10/14			
Ring Factor (KN/div) :-		0.106		Time:-		11:00 AM			
Blows	Cor. Load (Kn)		CBR (%)		Swell %	Dry Density Vs Socked C.B.R.			
	2.54m m	5.08m m	2.54m m	5.08 mm		No OF BLOWS	10	30	65
10	0.0442	0.0424	4.42	4.24	2.388	Swell, (%)	2.388	1.855	1.76
30	0.0522	0.0525	5.22	5.25	1.855	DryDensity,(g/ cm3)	1.290	1.342	1.500
65	0.0562	0.0551	5.62	5.51	1.76	SockedCbr, (%)	4.42	5.22	5.62
MODIFIED PROCTOR : T 180 - D									
MDD (g/cc) :		1.427							
OMC (%) :		24.00							
95% of MDD(g/cc)		1.356							
CBR % at 95% of MDD		5.3							
Swell % at 95 of MDD		1.84							



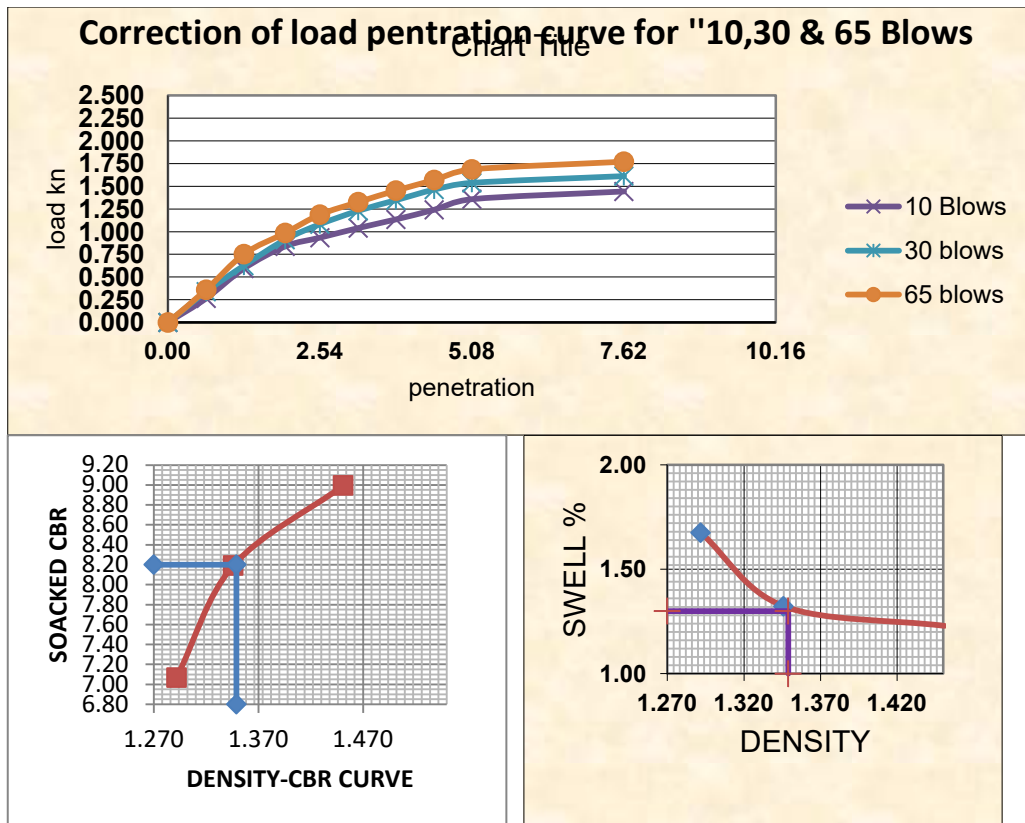
APPENDIX 10.2-CBR TEST ANALYSIS OF 10%MCA +30%WB

California Bearing Ratio - Data Sheet (TEST METHOD AASHTO T-193)									
CBR Ring,(KN) :-		30		Socking date:-		18/10/14			
Surcharge Weight, (Kg) :-		4.50		Testing date:-		22/10/14			
Ring Factor (KN/div) :-		0.106		Time:-		11:00 AM			
Blows	Cor. Load (Kn)		CBR (%)		Swell %	Dry Density Vs Socked C.B.R.			
	2.54m m	5.08m m	2.54m m	5.08 mm		No OF BLOWS	10	30	65
10	0.0964	0.0928	9.64	9.28	1.666	Swell, (%)	1.666	1.202	1.056
30	0.1060	0.0954	10.60	9.54	1.202	DryDensity,(g/cm ³)	1.273	1.373	1.428
65	0.1100	0.1034	11.00	10.34	1.056	SockedCbr, (%)	9.64	10.60	11.00
MODIFIED PROCTOR : T 180 - D									
MDD (g/cc) :			1.44						
OMC (%) :			22.7						
95% of MDD(g/cc)			1.368						
CBR % at 95% of MDD			10.5						
Swell % at 95 of MDD			1.2						



APPENDIX 10.3:-CBR TEST ANALYSIS OF 15%MCA +20%WB

California Bearing Ratio - Data Sheet (TEST METHOD AASHTO T-193)									
CBR Ring,(KN) :-			30			Socking date:-		18/10/14	
Surcharge Weight, (Kg) :-			4.50			Testing date:-		22/10/14	
Ring Factor (KN/div) :-			0.106			Time:-		11:00 AM	
Blows	Cor. Load (Kn)		CBR (%)		Swell %	Dry Density Vs Socked C.B.R.			
	2.54m m	5.08m m	2.54m m	5.08 mm		No BLOWS	10	30	65
10	0.0707	0.0678	7.07	6.78	1.675	Swell, (%)	1.675	1.323	1.288
30	0.0819	0.0769	8.19	7.69	1.323	DryDensity,(g/ cm3)	1.292	1.346	1.451
65	0.0899	0.0843	8.99	8.43	1.288	SockedCbr, (%)	7.07	8.19	8.99
MODIFIED PROCTOR : T 180 - D									
MDD (g/cc) :			1.420						
OMC (%) :			23.70						
95% of MDD(g/cc)			1.349						
CBR % at 95% of MDD			8.						
Swell % at 95 of MDD			1.3						



APPNDIX A10.4:-CBR TEST ANALYSIS OF 1 20%MCA +10%WB

California Bearing Ratio - Data Sheet (TEST METHOD AASHTO T-193)									
CBR Ring,(KN) :-		30		Socking date:-		18/10/14			
Surcharge Weight, (Kg) :-		4.50		Testing date:-		22/10/14			
Ring Factor (KN/div) :-		0.106		Time:-		11:00 AM			
Blows	Cor. Load (Kn)		CBR (%)		Swell %	Dry Density Vs Socked C.B.R.			
	2.54m m	5.08m m	2.54m m	5.08 mm		No BLOWS	10	30	65
10	0.0634	0.0610	6.34	6.10	2.259	Swell, (%)	2.259	1.615	1.486
30	0.0691	0.0663	6.91	6.63	1.615	DryDensity,(g/ cm3)	1.320	1.334	1.431
65	0.0755	0.0710	7.55	7.10	1.486	SockedCbr, (%)	6.34	6.91	7.55
MODIFIED PROCTOR : T 180 - D									
MDD (g/cc) :		1.421							
OMC (%) :		24.00							
95% of MDD(g/cc)		1.350							
CBR % at 95% of MDD		7.1							
Swell % at 95 of MDD		1.55							

