STABILIZATION OF WEAK SOIL USING MAIZE HUSK ASH

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**ABSTRACT**

This research is aimed at investigating the effect of maize husk ash in stabilizing weak lateritic soil in order to improve its physical and engineering properties. It was achieved through sampling and testing of the weak lateritic soil itself and by the addition of different proportions of maize husk ash. Natural moisture content and specific gravity of the material were obtained as 8.41% and 2.73 respectively. Atterberg Limits revealed the plastic limit the soil is non-plastic. The liquid limit also increased from 15.77, 18, 38, 30 and 42 at 0%, 2%, 4%, 6% and 8% respectively.  The compaction shows that the optimum moisture content (OMC) was observed to have decreased from 10.03% at 0% to 8.69% at 2%, 7.37% at 4%, 9.20% at 6% and increased to 11.89% at 8%. The maximum dry density (MDD) increased from 2%, 4%, 6% and decreased at 8% with maize husk ash content. This increase in MDD agrees with a report relating that cementitious stabilizers generally increase the maximum dry density of soils at optimal amount between 2– 8%. The CBR shows that at 0% the value was 17.26% having gained an increased value from 2% to 6% as 18.13%, 23.29% and 19.10% which is very significant but 16.72% at 8% hence proved irrelevant. As such the stabilized weak soil can be used as a sub-base and in low strength required soil for fill, etc. as specified by the Federal Ministry of Works Specification.

**CHAPTER ONE**

**INTRODUCTION**

**1.1 Background to the Study**

The main intention improvement (i.e., engineered soil) is engineering exacting soil, including its strength (i.e., resistance), hydraulic conductivity and durability against repeating wetting and drying, as well as for ecological revitalization. The most important methods are predictably applied to produce engineered soil: mechanical improvement and chemical treatment. Mechanical development is a development of reinforcing during physical processes such as compaction, drainage, external loading (e.g., surcharge), consolidation, or other means. Chemical treatment involves compound reaction such as hydration or pozzolanic reactions inside the mud to create artificial binding, hydrate (C-S-H) between soil particles (Reddy, *et al.,* 2018).

In Nigeria, lateritic soil are commonly used for road construction. Lateritic soils in its natural state generally have low bearing capacity and low strength, due to high content of clay. When lateritic soil contains a large amount of clay materials, its strength and stability cannot be guaranteed under load especially in the presence of moisture. Technically, soil improvement could either be by modification or stabilization or both. Soil modification is the addition of a modifier (cement, lime etc.) to a soil to change its index properties, while soil stabilization is the treatment of soils to enable improvement in their strength and durability such that they become totally suitable for construction beyond their original classification. Soil stabilization, which refers to a process whereby the physical and/or chemical properties of a soil are modified in order to suit the purpose for which a soil is meant, has been an old practice by workers that utilize soils in construction. Over the years, cement and lime have been the two main materials used for stabilizing soil (Oluremi, 2020).

A number of biomass waste ashes are increasingly being reported as pozzolans. These include Rice Husk Ash, Bamboo Leaf Ash, Palm Fruit Ash, Locust Bean Pod Ash, Cement Cassava Peel Ash, Corn Husk Ash and Maize Husk Ash (Adesanya and Raheem, 2009) Provide references. The use of these waste ashes as pozzolans in concrete works has widely dominated the literature while their application as soil stabilizing agents is scanty and in some cases non-existent in spite of the fact that the mechanism for lime-soil stabilization includes a pozzolanic phase. Of particular instance is the complete absence of work on Corncob Ash as a stabilizing agent for road works. Producing at an annual rate of 9.2million tones, Nigeria ranks 8th among the corn producing countries of the world, with a corresponding generation of waste cobs across the country. Finding further use for biomass wastes will have a salutary effect on the environment, particularly in a developing country like Nigeria, where waste collection tends to be low and these wastes often constitute a menace to the environment. Furthermore, cohesive soils which fall within the band of soils that could be modified abound in Nigeria and are routinely used as sub-base and base course materials in pavement construction, which in some cases do have to be improved or modified with the addition of cement in order to raise the strength parameter to the expected standard. A further advantage to the environment is that partial or total replacement of cement in soil stabilization with a biomass waste will reduce the overall greenhouse gas emission from the construction industry. This is because while emissions from MHA production is carbon neutral, approximately 1tonne of CO2 is generated for every tonne of cement produced (Ahmed, and Yinusa, 2014).

**1.2 Statement of the Problem**

Maize Husk (MH) which is a by-product of maize processing is either for domestic consumption or industrial uses. Indiscriminate disposal of maize husk due to gross underutilization as well as lack of appropriate technology to recycle them is a major challenge, which results in environmental problem. This study considered the use of maize husk ash (MHA) as a stabilizer for lateritic soil, towards the improvement of its strength and durability. The disposal of this enormous waste can constitute pollution of the environment. Thus, many researchers have considered recycling it for various applications.

In light of the cost of using industrially produced materials to stabilize soft soils, especially in Nigeria today, previous research has demonstrated that some agricultural wastes can be used to stabilize soft soils, including pulverized fuel ash, palm kernel shell ash, rice husk ash, seashell powder, sawdust ash, bagasse ash and groundnut shell ash are effective soil modifiers (Emeka, 2018). Large quantities of agricultural waste worldwide isfacing serious problems like handling and disposal. Agriculturalwaste disposal creates a potential negative environmental impactthat causes air pollution, water pollution ultimately affects localecosystems (Emeka 2018).

Therefore, safe disposal of agricultural waste becomesa challenging task.The use of stabilizers from agricultural wastes has the potential of reducing the amount of waste disposal of in landfills/ dump sites, and at the same time reduce the carbon footprints caused by the use of traditional stabilizers like cement, and subsequently the cost of construction.

**1.3 Significance of the Study**

Re-use of these waste products will help to save the environment from pollution and severe ecological problem. It is highly recommended to conduct research on the maize husk and their impact onsoft soil behavior after stabilization. Generally, the maize husk waste is disposed to the landfills or disposal sites where everpresent in the locality and rare studies has not been conveyed yet. Meanwhile, in the present era there is a huge rise in the consumption of sugarcane particularly in Katsina which leaves around high amount maize husk afterwards before consumption. So, a normal yearly production ofthe peel is too much, which is a bulky waste. For the construction industry the soil is one of the most important item which is prepared fills and other engineering projects. Without stabilized soils, some engineering structures such as road ways, rail tracks, buildings, etc. cannot be achieved. However, the high used of chemicals are animportant concern of world environmental professionals. Considering the facts, one of the effective way to reduce the environmental impact is to use mineral admixtures, which will have the possible to cost reduction, energy conservation, and waste emissionminimization.

**1.4 Aim and Objectives**

**1.4.1 Aim**

The aim of this research is to investigate the effect of maize husk ash in stabilized weak soil in order to improve its physical and engineering properties.

**1.4.2 Objectives**

The specific objectives of this work are to investigate the influence of maize husk ash on the geotechnical properties of locally available soil as follows;

1. To obtain adequate maize husk ash under uncontrolled burning of maize husk fiber.
2. To determine physical and engineering properties of the untreated soil samples.
3. To investigate the effect of the maize husk ash on soil density and strength characteristics.

**1.5 Scope and Limitation**

**1.5.1 Scope**

This study emphasizes on changes of geotechnical properties of weak lateritic soil stabilized with maize husk ash in different proportions. The stabilized soil is to be subjected to experimental laboratory tests such as Atterberg limits, compaction test, etc. in order to ascertain the strength, durability and bearing capacity of such samples.

**1.5.2 Limitation**

This study is however limited to only maize husk ash obtained from uncontrolled burning of maize husk fiber available at Hassan Usman Katsina Polytechnic Harvest Field.

**CHAPTER TWO**

**ITERATURE REVIEW**

**2.1 Review of Related**

Soil quality has been described as the balance between soil degradation and soil resilience (Kennedy and Papendick, 1992; Lal, 1998). Soil resilience is the ability of soil to return to a dynamic equilibrium after being disturbed (Blum and Santelises, 1994). Soil resilience is controlled by inherent soil properties governed by the factors affecting soil formation (Blum, 1998). Soil degradation is the short to medium term deterioration of soil caused by land use, soil management, and the soil's susceptibility to soil processes that promote loss of function (Blum, 1998; Lal, 1998). As a soil quality definition is reached, it needs to be flexible to account for the numerous functions that soil may perform.

Soil properties and processes such as moisture retention, water flow, root development, nutrient cycling, and the sustainability of micro and macro organisms are negatively influenced by high bulk density values (Arshad *et al*., 1996; Arshad & Coen, 1992).

Akinwumi and Aidomojie (2015) Studied Effect of Corncob ash on the geotechnical properties of Lateritic soil stabilized with Portland cement. The article aimed at providing experimental insights on the engineering properties of lateritic soil stabilized with cement-corncob ash (CCA) to ascertain its suitability for use as a pavement layer material. Series of specific gravity, consistency limits, compaction, California bearing ratio (CBR) and permeability tests, considering three CCA blends and four CCA contents, varying from 0 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 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.

Yinusa and Apampa (2014) Did an Evaluation of the Influence of Corn Cob Ash on the Strength Parameters of Lateritic Soils. The soil was mixed with CCA in varying percentages of 0%, 1.5%, 3%, 4.5%, 6% and 7.5% and the influence of CCA on the soil was determined for Liquid Limit, Plastic Limit, Compaction Characteristics, CBR and the Unconfined Compression Test. These tests were repeated on laterite CCA-cement mix and laterite-cement mix respectively in order to detect any pozzolanicity in CCA when it combines with Portland cement and to compare results with a known soil stabilizing agent. The result showed a similarity in the compaction characteristics of soil-cement, soil CCA and soil-CCA-cement, in that with increasing addition of binder from 1.5% to 7.5%, Maximum Dry Density progressively declined while the OMC steadily increased. In terms of the strength parameters, the maximum positive impact was observed at 1.5% CCA addition for soil-CCA with a CBR value of 84% and a UCS value of 1.0MN/m2, compared with the control values of 65% and 0.4MN/m2 respectively. For the soil-CCA-cement mix, the strength parameters CBR and UCS continued to increase with increasing binder addition within the tested range for the ratios 1:2 and 1:1 and 2:1 CCA: cement. Significantly, the results from the soil-CCA-cement mix, indicated the pozzolanicity of CCA in that UCS values were higher by at least 14% for the 1:1 ratio, than was attained with the addition of only the corresponding quantity of cement. In the light of the above, it was recommended that CCA can be made commercially available in its pure form or as CCA-cement blends and promoted as a stabilizing agent for soils in pavement construction.

Apampa *et al*., (2015) studied Modelling of Compaction Curves for Corn Cob Ash-Cement Stabilized Lateritic Soils. The study proposed a model for predicting the dry density of lateritic soils stabilized with corn cob ash (CCA) and blended cement – CCA. Lateritic soil was first stabilized with CCA at 1.5, 3.0, 4.5 and 6% of the weight of soil and then stabilized with the same proportions as replacement for cement. Dry density, specific gravity, maximum degree of saturation and moisture content were determined for each stabilized soil specimen, following standard procedure. Polynomial equations containing alpha and beta parameters for CCA and blended CCA-cement were developed. Experimental values were correlated with the values predicted from the Mat lab curve fitting tool, and the Solverfunction of Microsoft Excel 2010. The correlation coefficient (R2) of 0.86 was obtained indicating that the model could be accepted in predicting the maximum dry density of CCA stabilized soils to facilitate quick decision making in road works.

**2.2 Soil**

The term ‘Soil’ has different meanings in different scientific fields. It has originated from the Latin word Solum. To an agricultural scientist, it means ‘‘the loose material on the earth’s crust consisting of disintegrated rock with an admixture of organic matter, which supports plant life’’. To a geologist, it means the disintegrated rock material which has not been transported from the place of origin. But, to a civil engineer, the term ‘soil’ means, the loose unconsolidated inorganic material on the earth’s crust produced by the disintegration of rocks, overlying hard rock with or without organic matter. Foundations of all structures have to be placed on or in such soil, which is the primary reason for our interest as Civil Engineers in its engineering behaviour. Soil may remain at the place of its origin or it may be transported by various natural agencies. It is said to be ‘residual’ in the earlier situation and ‘transported’ in the latter. ‘‘Soil mechanics’’ is the study of the engineering behaviour of soil when it is used either as a construction material or as a foundation material. This is a relatively young discipline of civil engineering, systematised in its modern form by Karl Von Terzaghi, who is rightly regarded as the ‘‘Father of Modern Soil Mechanics’’. An understanding of the principles of mechanics is essential to the study of soil mechanics.

A knowledge and application of the principles of other basic sciences such as physics and chemistry would also be helpful in the understanding of soil behaviour. Further, laboratory and field research have contributed in no small measure to the development of soil mechanics as a discipline. The application of the principles of soil mechanics to the design and construction of foundations for various structures is known as ‘‘Foundation Engineering’’. ‘‘Geotechnical Engineering’’ may be considered to include both soil mechanics and foundation engineering. In fact, according to Terzaghi, it is difficult to draw a distinct line of demarcation between soil mechanics and foundation engineering; the latter starts where the former ends. Until recently, a civil engineer has been using the term ‘soil’ in its broadest sense to include even the underlying bedrock in dealing with foundations. However, of late, it is well recognised that the sturdy of the engineering behaviour of rock material distinctly falls in the realm of ‘rock mechanics’, research into which is gaining impetus the world over.

**2.2.1 Development of Soil Mechanics**

The use of soil for engineering purposes dates back to prehistoric times. Soil was used not only for foundations but also as construction material for embankments. The knowledge was empirical in nature and was based on trial and error, and experience. The hanging gardens of Babylon were supported by huge retaining walls, the construction of which should have required some knowledge, though empirical, of earth pressures. The large public buildings, harbours, aqueducts, bridges, roads and sanitary works of Romans certainly indicate some knowledge of the engineering behaviour of soil. This has been evident from the writings of Vitruvius, the Roman Engineer in the first century B.C., Mansar and Viswakarma, in India, wrote books on ‘construction science’ during the medieval period. The Leaning Tower of Pisa, Italy, built between 1174 and 1350 A.D., is a glaring example of a lack of sufficient knowledge of the behaviour of compressible soil, in those days.

Coulomb, a French Engineer, published his wedge theory of earth pressure in 1776, which is the first major contribution to the scientific study of soil behaviour. He was the first to introduce the concept of shearing resistance of the soil as composed of the two components cohesion and internal friction. Poncelet, Culmann and Rebhann were the other men who extended the work of Coulomb.

D’ Arcy and Stokes were notable for their laws for the flow of water through soil and settlement of a solid particle in liquid medium, respectively. These laws are still valid and play an important role in soil mechanics.

Rankine gave his theory of earth pressure in 1857; he did not consider cohesion, although he knew of its existence.

Boussinesq, in 1885, gave his theory of stress distribution in an elastic medium under a point load on the surface.

Mohr, in 1871, gave a graphical representation of the state of stress at a point, called ‘Mohr’s Circle of Stress’. This has an extensive application in the strength theories applicable to soil.

Atterberg, a Swedish soil scientist, gave in 1911 the concept of ‘consistency limits’ for a soil. This made possible the understanding of the physical properties of soil.

The Swedish method of slices for slope stability analysis was developed by Fellenius in 1926. He was the chairman of the Swedish Geotechnical Commission.

Prandtl gave his theory of plastic equilibrium in 1920 which became the basis for the development of various theories of bearing capacity.

Terzaghi gave his theory of consolidation in 1923 which became an important development in soil mechanics. He also published, in 1925, the first treatise on Soil Mechanics, a term coined by him. (Erd bau mechanik, in German). Thus, he is regarded as the Father of modern soil mechanics’. Later on, R.R. Proctor and A. Casagrande and a host of others were responsible for the development of the subject as a full-fledged discipline.

Fifteen International Conferences have been held till now under the auspices of the international Society of Soil Mechanics and Foundation engineering at Harvard (Massachusetts, U.S.A.) 1936, Rotterdam (The Netherlands) 1948, Zurich (Switzerland) 1953, London (U.K.) 1957, Paris (France) 1961, Montreal (Canada) 1965, Mexico city (Mexico) 1969, Moscow (U.S.S.R) 1973, Tokyo (Japan) 1977, Stockholm (Sweden) 1981, San Francisco (U.S.A.) 1985, and Rio de Janeiro (Brazil) 1989. The thirteenth was held in New Delhi in 1994, the fourteenth in Hamburg, Germany, in 1997, and the fifteenth in Istanbul, Turkey in 2001. The sixteenth is proposed to be held in Osaka, Japan, in 2005. These conferences have given a big boost to research in the field of Soil Mechanics and Foundation Engineering.

**2.2.2 Fields of Application of Soil Mechanics**

The knowledge of soil mechanics has application in many fields of Civil Engineering.

**2.2.2.1 Foundations**

The loads from any structure have to be ultimately transmitted to a soil through the foundation for the structure. Thus, the foundation is an important part of a structure, the type and details of which can be decided upon only with the knowledge and application of the principles of soil mechanics. (Craig, 2004).

**2.2.2.2 Underground and Earth-retaining Structures**

Underground structures such as drainage structures, pipe lines, and tunnels and earth-retaining structures such as retaining walls and bulkheads can be designed and constructed only by using the principles of soil mechanics and the concept of ‘soil-structure interaction’ (Craig, 2004).

**2.2.2.3 Pavement Design**

Pavement Design may consist of the design of flexible or rigid pavements. Flexible pavements depend more on the subgrade soil for transmitting the traffic loads. Problems peculiar to the design of pavements are the effect of repetitive loading, swelling and shrinkage of sub-soil and frost action. Consideration of these and other factors in the efficient design of a pavement is a must and one cannot do without the knowledge of soil mechanics (Craig, 2004).

**2.2.2.4 Excavations, Embankments and Dams**

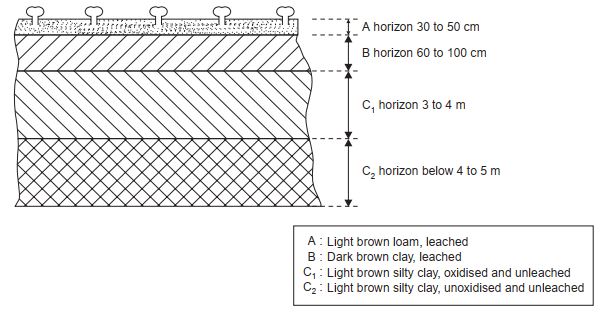
Excavations require the knowledge of slope stability analysis; deep excavations may need temporary supports ‘timbering’ or ‘bracing’, the design of which requires knowledge of soil mechanics. Likewise the construction of embankments and earth dams where soil itself is used as the construction material, requires a thorough knowledge of the engineering behaviour of soil especially in the presence of water. Knowledge of slope stability, effects of seepage, consolidation and consequent settlement as well as compaction characteristics for achieving maximum unit weight of the soil in-situ, is absolutely essential for efficient design and construction of embankments and earth dams. The knowledge of soil mechanics, assuming the soil to be an ideal material elastic, isotropic, and homogeneous material coupled with the experimental determination of soil properties, is helpful in predicting the behaviour of soil in the field. Soil being a particulate and heterogeneous material, does not lend itself to simple analysis. Further, the difficulty is enhanced by the fact that soil strata vary in extent as well as in depth even in a small area. Although knowledge of soil mechanics is a prerequisite to be a successful foundation engineer. It is difficult to draw a distinguishing line between Soil Mechanics and Foundation Engineering; the later starts where the former ends (Craig, 2004).

**2.2.3 Soil Formation**

Soil is formed by the process of ‘Weathering’ of rocks, that is, disintegration and decomposition of rocks and minerals at or near the earth’s surface through the actions of natural or mechanical and chemical agents into smaller and smaller grains. The factors of weathering may be atmospheric, such as changes in temperature and pressure; erosion and transportation by wind, water and glaciers; chemical action such as crystal growth, oxidation, hydration, carbonation and leaching by water, especially rainwater, with time. Obviously, soils formed by mechanical weathering (that is, disintegration of rocks by the action of wind, water and glaciers) bear a similarity in certain properties to the minerals in the parent rock, since chemical changes which could destroy their identity do not take place. It is to be noted that 95% of the earth’s crust consists of igneous rocks, and only the remaining 5% consists of sedimentary and metamorphic rocks. However, sedimentary rocks are present on 80% of the earth’s surface area. Feldspars are the minerals abundantly present (60%) in igneous rocks. Amphiboles and pyroxenes, quartz and micas come next in that order. Rocks are altered more by the process of chemical weathering than by mechanical weathering. In chemical weathering some minerals disappear partially or fully, and new compounds are formed. The intensity of weathering depends upon the presence of water and temperature and the dissolved materials in water. Carbonic acid and oxygen are the most effective dissolved materials found in water which cause the weathering of rocks. Chemical weathering has the maximum intensity in humid and tropical climates (Smith and Ian, 2008).

A deposit of soil material, resulting from one or more of the geological processes described earlier, is subjected to further physical and chemical changes which are brought about by the climate and other factors prevalent subsequently. Vegetation starts to develop and rainfall begins the processes of leaching and eluviation of the surface of the soil material. Gradually, with the passage of geological time profound changes take place in the character of the soil. These changes bring about the development of ‘soil profile’. Thus, the soil profile is a natural succession of zones or strata below the ground surface and represents the alterations in the original soil material which have been brought about by weathering processes. It may extend to different depths at different places and each stratum may have varying thickness.

Generally, three distinct strata or horizons occur in a natural soil-profile; this number may increase to five or more in soils which are very old or in which the weathering processes have been unusually intense. From top to bottom these horizons are designated as the A-horizon, the B-horizon and the C-horizon. The A-horizon is rich in humus and organic plant residue. This is usually eluviated and leached; that is, the ultrafine colloidal material and the soluble mineral salts are washed out of this horizon by percolating water. It is dark in colour and its thickness may range from a few centimetres to half a metre. This horizon often exhibits many undesirable engineering characteristics and is of value only to agricultural soil scientists. The B-horizon is sometimes referred to as the zone of accumulation. The material which has migrated from the A-horizon by leaching and eluviation gets deposited in this zone. There is a distinct difference of colour between this zone and the dark top soil of the A-horizon. This soil is very much chemically active at the surface and contains unstable fine-grained material. Thus, this is important in highway and airfield construction work and light structures such as single storey residential buildings, in which the foundations are located near the ground surface. The thickness of B-horizon may range from 0.50 to 0.75m. The material in the C-horizon is in the same physical and chemical state as it was first deposited by water, wind or ice in the geological cycle. The thickness of this horizon may range from a few centimetres to more than 30 m. The upper region of this horizon is often oxidized to a considerable extent. It is from this horizon that the bulk of the material is often borrowed for the construction of large soil structures such as earth dams. Each of these horizons may consist of sub-horizons with distinctive physical and chemical characteristics and may be designated as A1, A2, B1, B2, etc. The transition between horizons and sub-horizons may not be sharp but gradual. At a certain place, one or more horizons may be missing in the soil profile for special reasons (Smith and Ian, 2008).



**Fig. 2.1: Typical Soil Profile**

**2.2.4 Residual and Transported Soils**

Soils which are formed by weathering of rocks may remain in position at the place of region. In that case these are ‘Residual Soils’. These may get transported from the place of origin by various agencies such as wind, water, ice, gravity, etc. In this case these are termed ‘‘Transported soil’’. Residual soils differ very much from transported soils in their characteristics and engineering behaviour. The degree of disintegration may vary greatly throughout a residual soil mass and hence, only a gradual transition into rock is to be expected. An important characteristic of these soils is that the sizes of grains are not definite because of the partially disintegrated condition. The grains may break into smaller grains with the application of a little pressure (Smith and Ian, 2008).

The residual soil profile may be divided into three zones: (i) the upper zone in which there is a high degree of weathering and removal of material; (ii) the intermediate zone in which there is some degree of weathering in the top portion and some deposition in the bottom portion; and (iii) the partially weathered zone where there is the transition from the weathered material to the unweathered parent rock. Residual soils tend to be more abundant in humid and warm zones where conditions are favourable to chemical weathering of rocks and have sufficient vegetation to keep the products of weathering from being easily transported as sediments. Residual soils have not received much attention from geotechnical engineers because these are located primarily in undeveloped areas. In some zones in South India, sedimentary soil deposits range from 8 to 15 m in thickness. Transported soils may also be referred to as ‘Sedimentary’ soils since the sediments, formed by weathering of rocks, will be transported by agencies such as wind and water to places far away from the place of origin and get deposited when favourable conditions like a decrease of velocity occur. A high degree of alteration of particle shape, size, and texture as also sorting of the grains occurs during transportation and deposition. A large range of grain sizes and a high degree of smoothness and fineness of individual grains are the typical characteristics of such soils. Transported soils may be further subdivided, depending upon the transporting agency and the place of deposition, as under:

1. Alluvial soils. Soils transported by rivers and streams: Sedimentary clays.
2. Aeoline soils. Soils transported by wind: loess.
3. Glacial soils. Soils transported by glaciers: Glacial till.
4. Lacustrine soils. Soils deposited in lake beds: Lacustrine silts and lacustrine clays.
5. Marine soils. Soils deposited in sea beds: Marine silts and marine clays.

Broad classification of soils may be:

1. Coarse-grained soils, with average grain-size greater than 0.075 mm, e.g., gravels and sands.
2. Fine-grained soils, with average grain-size less than 0.075 mm, e.g., silts and clays.

These exhibit different properties and behaviour but certain general conclusions are possible even with this categorization. For example, fine-grained soils exhibit the property of ‘cohesion’ bonding caused by inter molecular attraction while coarse-grained soils do not; thus, the former may be said to be cohesive and the latter non-cohesive or cohesionless (Smith and Ian, 2008).

**2.2.5 Some Commonly Used Soil Designations**

The following are some commonly used soil designations, their definitions and basic properties:

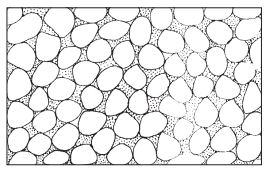
1. Bentonite. Decomposed volcanic ash containing a high percentage of clay mineral  
   montmorillonite. It exhibits high degree of shrinkage and swelling.
2. Black cotton soil. Black soil containing a high percentage of montmorillonite and colloidal material; exhibits high degree of shrinkage and swelling. The name is derived from the fact that cotton grows well in the black soil.
3. Boulder clay. Glacial clay containing all sizes of rock fragments from boulders down to finely pulverized clay materials. It is also known as ‘Glacial till’.
4. Caliche. Soil conglomerate of gravel, sand and clay cemented by calcium carbonate.
5. Hard pan. Densely cemented soil which remains hard when wet. Boulder clays or glacial tills may also be called hard-pan— very difficult to penetrate or excavate.
6. Laterite. Deep brown soil of cellular structure, easy to excavate but gets hardened on exposure to air owing to the formation of hydrated iron oxides.
7. Loam. Mixture of sand, silt and clay size particles approximately in equal proportions; sometimes contains organic matter.
8. Loess. Uniform wind-blown yellowish brown silt or silty clay; exhibits cohesion in the dry condition, which is lost on wetting. Near vertical cuts can be made in the dry condition.
9. Marl. Mixtures of calcareous sands or clays or loam; clay content not more than 75% and lime content not less than 15%.
10. Moorum. Gravel mixed with red clay.
11. Top-soil. Surface material which supports plant life.
12. Varved clay. Clay and silt of glacial origin, essentially a lacustrine deposit; varve is a term of Swedish origin meaning thin layer. Thicker silt varves of summer alternate with thinner clay varves of winter (Verruijt, 2008).

**2.2.6 Structure of Soils**

The ‘structure’ of a soil may be defined as the manner of arrangement and state of aggregation of soil grains. In a broader sense, consideration of mineralogical composition, electrical properties, orientation and shape of soil grains, nature and properties of soil water and the interaction of soil water and soil grains, also may be included in the study of soil structure, which is typical for transported or sediments soils. Structural composition of sedimented soils influences, many of their important engineering properties such as permeability, compressibility and shear strength. Hence, a study of the structure of soils is important. The following types of structure are commonly studied:

1. Single-grained structure
2. Honey-comb structure
3. Flocculent structure (Venkatramaiah, 2006).

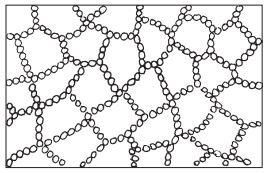
**2.2.6.1 Single-grained Structure**Single-grained structure is characteristic of coarse grained soils, with a particle size greater than 0.02mm. Gravitational forces predominate the surface forces and hence grain to grain contact results. The deposition may occur in a loose state, with large voids or in a sense state, with less of voids (Venkatramaiah, 2006).



**Fig. 2.2: Single-Grained Structure**

**2.2.6.2 Honey-comb Structure**

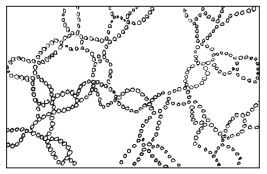
This structure can occur only in fine-grained soils, especially in silt and rock flour. Due to the relatively smaller size of grains, besides gravitational forces, inter-particle surface forces also play an important role in the process of settling down. Miniature arches are formed, which bridge over relatively large void spaces. This results in the formation of a honey-comb structure, each cell of a honey-comb being made up of numerous individual soil grains. The structure has a large void space and may carry high loads without a significant volume change. The structure can be broken down by external disturbances (Venkatramaiah, 2006).



**Fig. 2.3: Honey-Comb Structure**

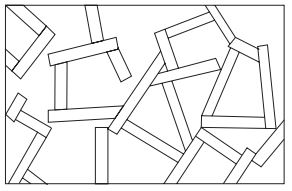
**2.2.6.3 Flocculent Structure**

This structure is characteristic of fine-grained soils such as clays. Inter-particle forces play a predominant role in the deposition. Mutual repulsion of the particles may be eliminated by means of an appropriate chemical; this will result in grains coming closer together to form a ‘floc’. Formation of flocs is ‘flocculation’. But the flocs tend to settle in a honeycomb structure, in which in place of each grain, a floc occurs. (Venkatramaiah, 2006).



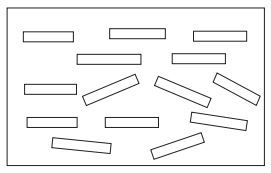
**Fig. 2.4: Flocculent Structure**

Thus, grains grouping around void spaces larger than the grain-size are flocs and flocs grouping around void spaces larger than even the flocs result in the formation of a ‘flocculent’ structure. Very fine particles or particles of colloidal size (< 0.001 mm) may be in a flocculated or dispersed state. The flaky particles are oriented edge-to-edge or edge-to-face with respect to one another in the case of a flocculated structure. Flaky particles of clay minerals tend to from a card house structure when flocculated (Venkatramaiah, 2006).



**Fig. 2.5: Card-House Structure of Flaky Particles**

When inter-particle repulsive forces are brought back into play either by remoulding or by the transportation process, a more parallel arrangement or reorientation of the particles occurs. This means more face-to-face contacts occur for the flaky particles when these are in a dispersed state. In practice, mixed structures occur, especially in typical marine soils (Venkatramaiah, 2006).



**Fig. 2.6: Dispersed Structure**

**2.2.7 Texture of Soils**

The term ‘Texture’ refers to the appearance of the surface of a material, such as a fabric. It is used in a similar sense with regard to soils. Texture of a soil is reflected largely by the particle size, shape, and gradation. The concept of texture of a soil has found some use in the classification of soils to be dealt with later (Venkatramaiah, 2006).

**2.3 Soil Stabilization**

Soil Stabilisation in the broadest sense, refers to the procedures employed with a view to altering one or more properties of a soil so as to improve its engineering performance. Soil Stabilisation is only one of several techniques available to the geotechnical engineer and its choice for any situation should be made only after a comparison with other techniques indicates it to be the best solution to the problem. It is a well-known fact that, every structure must rest upon soil or be made of soil. It would be ideal to find a soil at a particular site to be satisfactory for the intended use as it exists in nature, but unfortunately, such a thing is of rare occurrence (Budhu, 2019).

Although certain techniques of stabilisation are of a relatively recent origin, the art itself is very old. The original objective of soil stabilisation, was, as the name implies, to increase the strength or stability of soil. However, techniques have now been developed to alter almost every engineering property of soil. The primary aim may be to alter the strength and/or to reduce its sensitivity to moisture changes. The most common application of soil stabilisation is the strengthening of the soil components of highway and airfield pavements (Craig, 2004).

**2.3.1 Classification of the Methods of Stabilisation**

A completely consistent classification of soil stabilisation techniques is difficult. Classifications may be based on the treatment given to soil, on additives used, or on the process involved. Broadly speaking, soil stabilisation procedures may be brought under the following two heads:

1. Stabilization without additives
2. Stabilisation with additives

Stabilization without additives may be ‘mechanical’ rearrangement of particles through compaction or addition or removal of soil particles. It may be by ‘drainage’ drainage may be achieved by the addition of external load, by pumping, by electro-osmosis, or by application of a thermal gradient heating or cooling. Stabilisation with additives may be cement stabilisation (that is, soil cement), bitumen stabilisation, or chemical stabilisation (with fly ash, lime, calcium or sodium chloride, sodium silicate, dispersants, physico-chemical alteration involving ion-exchange in clay-minerals or injection stabilisation by grouting with soil, cement or chemicals). The appropriate method for a given situation must be chosen by the geotechincal engineer based on his experience and knowledge. Comparative laboratory tests followed by limited field tests, should be used to select the most economical method that will serve the particular problem on hand. Field-performance data may help in solving similar problems which arise in future. It must be remembered, however, that soil stabilisation is not always the best solution to a problem (Craig, 2004).

**2.3.1.1 Stabilization of Soil without Additives**

Some kind of treatment is given to the soil in this approach; no additives are used. The treatment may involve a mechanical process like compaction and a change of gradation by addition or removal of soil particles or processes for drainage of soil (Craig, 2004).

**2.3.1.1.1 Mechanical Stabilization**

‘Mechanical stabilisation’ means improving the soil properties by rearrangement of particles and densification by compaction, or by changing the gradation through addition or removal of soil particles (Craig, 2004).

1. **Rearrangement of Particles (Compaction):** The process of densification of a soil or ‘compaction’, as it is called, is the oldest and most important method. In addition to being used alone, compaction constitutes an essential part of  
   a number of other methods of soil stabilisation. The important variables involved in compaction are the moisture content, compactive effort or energy and the type of compaction. The most desirable combination of the placement variables depends upon the nature of the soil and the desired properties. Fine-grained soils are more sensitive to placement conditions than coarse-grained soils (Craig, 2004).
2. **Change of Gradation (Addition or Removal of Soil Particles):** The engineering behaviour of a soil depends upon (among other things) the grain-size distribution and the composition of the particles. The properties may be significantly altered by adding soil of some selected grain-sizes, and, or by removing some selected fraction of the soil. In other words, this approach consists in manipulating the soil fractions to obtain a suitable grading, which involves mixing coarse material or gravel (called ‘aggregate’), sand, silt and clay in proper proportions so that the mixture when compacted attains maximum density and strength. It may involve blending of two or more naturally available soils in suitable proportions to achieve the desired engineering properties for the mixture after necessary compaction. Soil materials can be divided into two fractions, the granular fraction or the ‘aggregate’, retained on a 75-micron I.S. Sieve, and the fine soil fraction or the ‘binder’, passing this sieve. The aggregate provides strength by internal friction and hardness or incompressibility, white the binder provides cohesion or binding property, water-retention capacity or imperviousness and also acts as a filler for the voids of the aggregate. The relative amounts of aggregate and binder determine the physical properties of the compacted stabilized soil. The optimum amount of binder is reached when the compacted binder fills the voids without destroying all the grain-to-grain contacts of coarse particles. Increase in the binder beyond this limit results in a reduction of internal friction, a slight increase in cohesion and greater compressibility. Determination of the optimum amount of binder is an important component of the design of the mechanically stabilized mixture. Mechanical stabilisation of this type has been largely used in the construction of lowcost roads. Guide specifications have been developed based on past experience, separately for base courses and surface courses. (Craig, 2004).

**2.3.1.1.2 Stabilization by Drainage**

Generally speaking, the strength of a soil generally decreases with an increase in pore water and in the pore water pressure. Addition of water to a clay causes a reduction of cohesion by increasing the electric repulsion between particles. The strength of a saturated soil depends directly on the effective or intergranular stress. For a given total stress, an increase in pore water pressure results in a decrease of effective stress and consequent decrease in strength. Thus, drainage of a soil is likely to result in an increase in strength which is one of the primary objectives of soil stabilisation. The methods used for drainage for this purpose are:

1. Application of external load to the soil mass,
2. Drainage of pore water by gravity and/ or pumping, using well-points, sand-drains, etc.,
3. Application of an electrical gradient or electro-osmosis; and,
4. Application of a thermal gradient (Craig, 2004).

**2.3.1.2 Stabilization of Soil with Additives**

Stabilisation of soil with some kind of additive is very common. The mode and degree of alternation necessary depend on the nature of the soil and its deficiencies. If additional strength is required in the case of cohesionless soil, a cementing or a binding agent may be added and if the soil is cohesive, the strength can be increased by making it moisture-resistant, altering the absorbed water films, increasing cohesion with a cement agent and adding internal friction. Compressibility of a clay soil can be reduced by cementing the grains with a rigid material or by altering the forces of the adsorbed water films on the clay minerals. Swelling and shrinkage may also be reduced by cementing, altering the water adsorbing capacity of the clay mineral and by making it moisture-resistant. Permeability of a cohessionless soil may be reduced by filling the voids with an impervious material or by preventing flocculation by altering the structure of the adsorbed water on the clay mineral; it may be increased by removing the fines or modifying the structure to an aggregated one. A satisfactory additive for soil stabilisation must provide the desired qualities and, in addition, must meet the following requirements: Compactibility with the soil material, permanency, easy handling and processing, and low cost. Many additives have been employed but with varying degrees of success. No material has been found to meet all the requirements, and most of the materials are expensive (Smith and Ian, 2008).

**2.3.1.2.1 Additives Stabilization**

The various additives used fall under the following categories:

1. **Cementing Materials**: Increase in strength of the soil is achieved by the cementing  
   action of the additive. Portland cement, line, fly-ash and sodium silicate are examples of such additives.
2. **Water-Proofers**: Bituminous materials prevent absorption of moisture. These may be used if the natural moisture content of the soil is adequate for providing the necessary strength. Some resins also fall in this category, but are very expensive.
3. **Water-Retainers**: Calcium chloride and sodium chloride are examples of this category.
4. **Water-Repellents or Retarders**: Certain organic compounds such as stearates and silicones tend to get absorbed by the clay particles in preference to water. Thus, they tend to keep off water from the soil.
5. Modifiers and other miscellaneous agents: Certain additives tend to decrease  
   the plasticity index and modify the plasticity characteristics. Lignin and  
   lignin-derivatives are used as dispersing agents for clays (Smith and Ian, 2008).

**2.3.1.2.2 Cement Stabilization**

Portland cement is one of the most widely used additives for soil stabilisation. A mixture of soil and cement is called ‘‘soil-cement’’. If a small percentage of cement is added primarily to reduce the plasticity of fat soils, the mixture is said to be a ‘‘cement-modified soil’’. If the soil cement has enough water which facilitates pouring it as mortar, it is said to be a ‘plastic soil cement’’. It is used in canal linings. The chemical reactions of cement with the silicious soil in the presence of water are believed to be responsible for the cementing action. Many of the grains of the coarse fraction get cemented together, but the proportion of clay particles cemented is small. Almost any inorganic soil can be successfully stabilized with cement; organic matter may interfere with the cement hydration. Soil-cement has been widely used for low-cost pavements for highways and airfields, and as bases for heavy traffic. Generally, it is not recommended as a wearing coarse in view of its low resistance to abrasion (Craig, 2004).

**2.3.1.2.3 Bitumen Stabilization**

Bituminous materials such as asphalts and tars have been used for soil stabilisation. This method is better suited to granular soils and dry climates. ‘Bitumens’ are nonaqueous system of hydrocarbons which are completely soluble in ‘Carbon disulphide’. ‘Asphalts’ are natural materials or refined petroleum products, which are bitumens. ‘Tars’ are bituminous condensates produced by the destructive distillation of organic materials such as coal, oil, lignite and wood. Most bitumen stabilisation has been with asphalt. Asphalt is usually too viscous to be incorporated directly with soil. Hence, it is either heated or emulsified or cut back with a solvent like gasoline, to make it adequately fluid. Tars are not emulsified but are heated or cut back prior to application. Soil-asphalt is used mostly for base courses of roads with light traffic. Bitumen stabilises soil by one or both of two mechanisms: (i) binding soil particles together, and (ii) making the soil water-proof and thus protecting it from the deleterious effects of water. Obviously, the first mechanism occurs in cohesionless soils, and the second in cohesive soils, which are sensitive to water. Asphalt coats the surfaces of soil particles and protects them from water. If also plugs the voids in the soil, inhibiting a flow of pore water. Bitumen stabilisation may produce one of the following:

1. Sand-bitumen
2. Soil-bitumen
3. Water-proof mechanical stabilization
4. Oiled earth (Craig, 2004).

**2.3.1.3 Chemical Stabilization**

Chemical stabilisation refers to that in which the primary additive is a chemical. The use of chemicals as secondary additives to increase the effectiveness of cement and of asphalt has been mentioned earlier. Lime and salt have found wide use in the field. Some chemicals are used for stabilizing the moisture in the soil and some for cementation of particles. Certain aggregates and dispersants have also been used (Craig, 2004).

**2.3.1.3.1 Lime Stabilization**

Lime is produced from natural limestone. The hydrated limes, called ‘slaked lines’, are the commonly used form for stabilisation. In addition to being used alone, lime is also used in the following admixtures, for soil stabilisation:

1. Lime-fly ash (4 to 8% of hydrated lime and 8 to 20% of fly-ash)
2. Lime-Portland cement
3. Lime-bitumen

The use of lime as a soil stabilizer dates back to Romans, who used it in the construction of the ‘Appian way’ in Rome. This road has given excellent service and is maintained as a traffic artery even today. There are two types of chemical reactions that occur when lime is added to wet soil. The first is the alteration of the nature of the adsorbed layer through ion exchange of calcium for the ion naturally carried by the soil, or a change in the double layer on the soil colloids. The second is the cementing action or pozzolanic action which requires a much longer time. This is considered to be a reaction between the calcium with the available reactive alumina or silica from the soil (Craig, 2004).

Lime has the following effects on soil properties: Lime generally increases the plasticity index of low-plasticity soil and decreases that of highly plastic soils; in the latter case, lime tends to make the soil friable and more easily handled in the field. It increases the optimum moisture content and decreases the maximum compacted density; however, there will be an increase in strength. About 2 to 8% of lime may be required for coarse-grained soils, and 5 to 10% for cohesive soils. Certain sodium compounds (e.g., sodium hydroxide and sodium sulphate), as secondary additives, improve the strength of soil stabilized with lime. Lime may be applied in the dry or as a slurry. Better penetration is obtained when it is used as a slurry. The construction of lime-stabilized soil is very much similar to that of soilcement. The important difference is that, in this case, no time limitation may be placed on the operations, since the lime-soil reactions are slow. Care should be taken, however, to prevent the carbonation of lime. Lime stabilisation has been used for bases of pavements (Craig, 2004).

**2.3.1.3.2 Salt Stabilization**

Calcium chloride and sodium chloride have been used for soil stabilisation. Calcium chloride is hygroscopic and deliquescent. It absorbs moisture from the atmosphere and retains it. It also acts as a soil flocculant. The action of sodium chloride is similar. The effect of salt on soil arises from colloidal reactions and the alteration of the characteristics of soil water. Salt lowers the vapour pressure of pore water and also the freezing point; the frost heave will be reduced because of the latter phenomenon. The main disadvantage is that the beneficial effects of salt are lost, if the soil gets leached (Craig, 2004).

**2.3.1.3.3 Lignin and Chrome-Lignin Stabilization**

Lignin is one of the major constituents of wood and is obtained as a by-product during the manufacture of paper from wood. Lignin, both in powder form and in the form of sulphite liquor, has been used as an additive to soil for many years. A concentrated solution, partly neutralized with calcium base, known as Lignosol, has also been used. The stabilizing effects of lignin are not permanent since it is soluble in water; hence periodic applications may be required. In an attempt to improve the action of lignin, the ‘Chromelignin process’’ was developed. The addition of sodium bichromate or potassium bichromate to the sulphite waste results in the formation of an insoluble gel. If the lignin is not neutralized, it is acid and acts as a soil aggregate; when neutralized as with Lignisol, it acts as a dispersant. Chrome lignin imparts considerable strength to soils as a cementing agent (Craig, 2004).

**2.3.1.3.4 Stabilizers with Water-Proofers**

It is well known that cohesive soils possess considerable strength when they are dry. When they have access to water, they imbibe it and lose strength. Water-proofers, i.e., chemicals which prevent the deleterious effects of water on soils, are useful in such cases. Siliconates, amines and quaternary ammonium salts fall in this category. Water-proofers do not increase the strength, but help the soil retain its strength even in the presence of water (Verruijtm, 2018).

**2.3.1.3.5 Stabilisation with Natural and Synthetic Resins**

Certain natural as well as synthetic resins, which are obtained by polymerization of organic monomers, have also been used for soil stabilisation. They act primarily as water-proofers. Vinol resin and Rosin, both of which are obtained from pinetrees, are the commonly used natural resins. Aniline-furfural, polyvinyl alcohol (PVA), and calcium acrylate are commonly used synthetic resins. Asphalt and lignin, which are also resinous materials, have already been discussed separately (Verruijtm, 2018).

**2.3.1.3.6 Aggregants and Dispersants**

Aggregants and dispersants are chemicals which bring about modest changes in the properties of soil containing fine grains. These materials function by altering the electrical forces between the soil particles of colloidal size, but provide no cementing action. They affect the plasticity, permeability and strength of the soil treated. Low treatment levels are adequate for the purpose. Aggregants increase the net electrical attraction between adjacent fine-grained soil particles and tend to flocculate the soil mass. Inorganic salts such as calcium chloride and ferric chloride, and polymers such as Krilium are important examples. Change in adsorbed water layers, ion-exchange phenomena and increase in ion concentration are the possible mechanisms by which the aggregants work. Dispersants are chemicals which increase the electrical repulsion between adjacent fine grained soil particles, reduce the cohesion between them, and tend to cause them to disperse. Phosphates, sulphonates and versanates are the most common dispersants, which tend to decrease the permeability. Ion exchange and anion adsorption are the possible mechanisms by which the dispersants work (Verruijtm, 2018).

**2.3.1.3.7 Miscellaneous Chemical Stabilizers**

Sodium silicate can be used as a primary stabilizer as well as a secondary additive to conventional stabilizers such as cement. Injection is the usual process by which this is used. Phosphoric acid also has been used to some extent. Molasses, tung oil, sodium carbonate, paraffin and hydrofluoric acid are some miscellaneous chemicals which have been considered but have not received any extensive application (Verruijtm, 2018).

**2.3.1.4 Injection Stabilization**

Injection of the stabilizing agent into the soil is called ‘Grouting’. This process makes it possible to improve the properties of natural soil and rock formations, without excavation, processing, and recompaction. Grouting may have one of the two objectives; to improve strength properties or to reduce permeability. This is achieved by filling cracks, fissures and cavities in the rock and the voids in soil with a stabilizer this is initially in a liquid state or in suspension and which subsequently solidifies or precipitates. Injection is a very common technique in the oil industry; petroleum engineers frequently use this method for sealing or operating wells. Injection techniques, unfortunately, are rather complex. The selection of proper grout material and appropriate technique can normally be made best only after field exploration and testing. The results of the injection process are rather difficult to assess. Grouting must be called an art rather than a science (Verruijtm, 2018).

**2.4 Maize Husk Ash**

This is obtained from uncontrolled burning of maize husk. The husks of maize were sun dried to make them easily combustible and then burnt inside the iron drum at 700°C for90 minutes inside a galvanized drum to prevent contamination with soil materials. The maize husk ash wassieved through sieve No. 200 to obtain a material of fineness similar to cement. The presence and compositions of the following major and minor compounds such as silica (SiO) 64.90%,aluminum oxide (ALO) 10.79%, ferric oxide (FeO) 4.75%, calcium oxide (CaO) 10.24%, magnesium oxide (MgO) 2.08%, Sulphur trioxide(SO) 2.53%, sodium oxide (NaO) 0.43%, potassium oxide (KO) 4.23% and phosphorus oxide (P,O,) 0.05% were determined in study conducted by (Oluremi *et al*., 2018).

**CHAPTER THREE**

**METHODOLOGY**

## 3.1 Materials

## 3.1.1 Soils

The soil sample used for the study was a weak laterite soil obtained after removing top a depth of 0.5m before the soil sample was collected by disturbed sampling method and sealed in bags to avoid loss of moisture during transportation.

## 3.1.2 Water

Clean water from the tap at the Civil Engineering Laboratory of Hassan Usman Katsina Polytechnic was used for conducting all laboratory tests.

## 3.2 Methods

The geotechnical properties of the natural soil and the soil with maize husk at a varying percentages of coir were determined in accordance with BS 1377 (1990) to ascertain the degree of alteration involved.

The following laboratory tests were conducted:

* Natural moisture content test
* Specific gravity test
* Particle size distribution (Sieve analysis)
* Atterberg’s limits test
* Compaction test
* CBR

## 3.2.1 Natural Moisture Content Determination

Two moisture cans were weighed empty and recorded. A representative specimen of the soil sample was collected to the cans. The cans with the sample were weighed and kept inside an oven for 24 hours at a temperature of 100°C after which the weight of the oven dried sample was noted and recorded. The weights of the dried soil and the water in the soil were obtained by difference in weight.

Natural moisture content = % = - (Eqn. 3.1)

Where; M1 = Weight of empty can

M2 = Weight of can + wet soil

M3 = Weight of can + dry soil

## 3.2.2 Specific Gravity

A clean dry pychometer (density bottle) of 250ml was weighed (164g) after which it was filled with distilled water. The pychometer with the water was weighed again. The bottle was then emptied and dried. 120g of the oven dried sample of soil was introduced into the bottle. Distilled water was then carefully poured into the bottle and stirred with a glass rod during the process to allow trapped air to be released. Finally, the bottle was filled to the rim with water after which the outside was dried and then weighed.

The specific gravity (Gs) was thus calculated from the following

Specific gravity, (Gs) = (Eqn. 3.2)

Where; W1 = Weight of empty pycnometer

W2 = Weight of pycnometer + dry soil

W3 = Weight of pycnometer + soil + water

W4 =Weight of pycnometer + water

## 3.2.3 Particle Size Distribution (Sieve Analysis)

The method employed for determination of particle size distribution is the dry sieve analysis. A process known as sieve washing was carried out on about 300g of the black cotton soil sample. This process involves soaking and washing out the clay content from the soil then oven-drying the remaining soil sample for 24 hours. The dried soil was weighed again before poring it through a stack of sieves already arranged in descending order of the aperture size (i.e. 5.00mm, 3.35mm, 2.0mm, 1.18mm, 850µm, 600µm, 425µm, 300µm, 150µm, 75µm and the base pan) and vibrated for 5 minutes. After shaking, each sieve together with its content was then weighed. The weight of empty sieve which had earlier been weighed was then subtracted from the combined weights of soils retained and the sieve, which gave the actual weight of the soil sample retained in each of the sieves. The sum of these retained weights was checked against the original soil.

## 3.2.4 Atterberg’s Limit Determination

Albert Atterberg developed a series of tests to evaluate the relationship between moisture content and soil consistency. This series include three separate tests namely: liquid limit test, plastic limit test and the shrinkage limit test.

3.2.4.1 Liquid Limit Test (Cone Penetrometer Method)

About 200g of air dried soil passing through the 425µm sieve was mixed with distilled water to a putty-like consistency on a flat surface, filled in a cup and levelled with spatula. The height of the penetrometer cone was adjusted to 0.00mm reading, then the penetrometer brass cup well compacted with the soil sample was placed under the cone. The knob was adjusted until there was a slight contact between the tip of the penetrometer cone and the surface of the soil sample in the brass cup. The knob was pressed so as to allow the penetrometer penetrate into the soil sample.

The penetrometer reading was taken and noted. A representative specimen of the soil sample from the brass cup was taken and placed in a labelled moisture can. The weight of the can and the wet sample was also noted and recorded. It was then kept in an oven for 24 hours in order to determine its moisture content. The test was repeated five times but each time with increasing water content at varied proportion. The liquid test was carried out for the natural soil and then on maize husk ash with laterite soil.

3.2.4.2 Plastic Limit

About 150g of the soil sample passing through sieve 425µm was prepared in the same manner as in liquid limit. It was thoroughly mixed with distilled water on a glass plate until it was plastic enough to be rolled into a ball. The ball of the soil was then rolled between the hand and glass plate. The rolling continues until a thread of about 3mm in diameter was obtained, a stage at which the thread crumbled.

The portion of the crumbled soil were then gathered and placed in a moisture can which was kept in the oven for 24 hours for moisture content determination.

Plasticity index is given by:

Plasticity index = Liquid limit - plastic limit - - - - (Eqn. 3.3)

## 3.2.5 Compaction Test

An empty mould of diameter 10.2cm and volume 940cm3 was weighed with the base but without the collar. 3000g representative sample of the soil to be compacted was weighed and poured into the tray with the lumps broken so as to be reduced to smaller sizes. Water was added to the soil (1st trial) and mixed thoroughly after which it was packed into the mould in three layers. Using the British Standard Light compactive effort, each layer was given an evenly distributed 25blows from a 2.5kg rammer. After the compaction of the last layer, the surface of the compacted soil was ensured to be slightly above the top rim of the mould then the collar was removed and the excess soil trimmed off to be even with the top of the mould. The mould with the compacted soil was weighed and soil sample was taken from the top and bottom into the moisture cans for moisture content determination after 24 hours of drying in the oven. The entire procedure was repeated until the weight of the compacted soil and the mould began to drop (i.e. the soil has failed) with each trial having additional quantity of water mixed thoroughly with the soil.

The procedure was followed in like manner when the soil was stabilized with varying quantity of maize husk ash at a predetermined optimum amount maize husk ash in order to obtain the optimum moisture content.

**3.2.6 California Bearing Ratio**

A ¾ in (19 mm) sieve is used to sieve the soil specimen. If all material passes through the sieve, we can use all of it for the test. But some of the material might be retained in the sieve. In that situation have to replace the retained amount with an equal amount of the materials which pass ¾ in the sieve and retained on the #4 sieve. After sieving, make 3 sample specimens each containing 6.8 kg (15 lb). Specimen 1, 2, 3 will be compacted with about 10, 30 & 56 blows respectively. This will provide variations in the percentage of maximum dry density. Sufficient amounts of water shall be mixed with specimens to maintain optimum water content. The mould shall be attached to the base plate with the extension collar. Then the weight shall be measured. Then a spacer disk shall be placed into the mould with a filter paper on top of the spacer disk. The mould shall be filled with soil in 3 layers. For example: for specimen 1, we have to provide 10 blows per layer with the rammer for the compaction. The water content of the material shall be determined before and after the compaction procedure. Then the extension collar shall be removed and the top of the mold shall be trimmed with a straightedge to smoothen the surface. The other two specimens shall be compacted following the same procedures mentioned above. Remove spacer disk, base plate. Then, the weight of Mould plus compacted soil shall be measured. Then invert the mould and soil and attach the base plate to the mould with a coarse filter paper. Provide numbering of all your equations, e.g. (3.1), (3.2) etc.

**CHAPTER FOUR**

**RESULTS AND DISCUSSIONS**

**4.1 Introduction**

This chapter presents the results of the tests conducted and the discussion of the results.

**4.2 Natural Moisture Content**

**Table 4.1: Natural Moisture Content of Weak Soil**

|  |  |  |  |
| --- | --- | --- | --- |
| **Can Number** | **1** | **2** | **3** |
| Weight of Can (g) | 10.19 | 10.32 | 10.38 |
| Weight of Can + Wet Soil (g) | 20.99 | 23.80 | 24.35 |
| Weight of Can + Dry Soil (g) | 20.02 | 22.83 | 23.36 |
| Weight of Moisture (g) | 0.97 | 0.97 | 0.99 |
| Weight of Dry Soil (g) | 9.83 | 12.51 | 12.98 |
| Moisture Content (%) | 9.86 | 7.75 | 7.62 |
| **Average Moisture Content (%)** | **8.41** | | |

Results obtained from the experiment shows the average moisture content of the weak soil in its natural state is 8.41%

**4.3 Specific Gravity**

**Table 4.2: Specific Gravity of Weak Soil**

|  |  |  |
| --- | --- | --- |
| **Number of Trial** | **A** | **B** |
| Jar + Plate, M1(g) | 805.4 | 804.2 |
| Jar + Sample + Plate, W2 (g) | 1104.9 | 1103.9 |
| Jar + Sample + Water + Plate, M3 (g) | 2275.4 | 2275.2 |
| Jar + Water + Plate, M4(g) | 2085.4 | 2085.2 |
| Specific Gravity | 2.72 | 2.73 |
| **Average Specific Gravity** | **2.73** | |

The results obtained from the experiment shows that the average specific gravity is 2.73.

**4.4 Particle Size Distribution**

The results of the particle size distribution analysis is shown in table and figure 4.1.

**Table 4.3: Particle Size Distribution of Weak Soil**

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| **Sieve Sizes (mm/µm)** | **Weight of Soil Retained (g)** | **Percentage Retained (%)** | **Cumulative Percentage Retained (%)** | **Percentage Passing (%)** |
| 20 | 0 | 0 | 0 | 100 |
| 3.35 | 2.0 | 0.2 | 0.2 | 99.80 |
| 2.36 | 0.9 | 0.09 | 0.29 | 99.71 |
| 1.18 | 5.8 | 0.58 | 0.87 | 99.13 |
| 600µm | 96.2 | 9.62 | 10.49 | 89.53 |
| 425µm | 299.8 | 29.98 | 40.47 | 59.53 |
| 300µm | 218.4 | 21.84 | 62.31 | 37.69 |
| 212µm | 91.8 | 9.18 | 71.49 | 28.51 |
| 150µm | 48.6 | 4.86 | 76.35 | 23.65 |
| 78µm | 10.6 | 1.06 | 77.41 | 22.59 |
| 63µm | 0 | 0 | 77.41 | 22.59 |
| Pan | 0 | 0 | 77.41 | 22.59 |

**Fig 4.1: Particle Size Distribution Curve for Unstabilized Weak Soil**

The graph of the particle size distribution shows that 37.69% is fine sand fraction, and a little fraction of medium sand, coarse sand and fine gravel. The fact that over 60% of the soil passed through sieve 2.36mm shows that is a fairly graded sandy-clay soil.

**4.5 Atterberg Limits**

Atterberg limit test was carried out to determine the liquid limit, plastic limit and plasticity index of the plain weak soil and various percentages of stabilized weak soil. From the results obtained, it was observed that the plastic limit the soil is non-plastic. The liquid limit also increased from 38, 30, 18 and 15.77 at 0%, 2%, 4%, 6% and 8% respectively.

This is as a result of the increased of the clay sized fractions in the soil, thereby increasing the ability of the soil to attract and absorb water.

Liquid Limit = *15.77%* Plastic Limit = *Non-plastic* Plasticity Index = *0%*

**Fig. 4.2: Atterberg Limit for Untreated Weak Soil**

According a research conducted by Hamisu (2022) the result average Atterberg limit was 23.4%, which is far less than 15.77% this signifies that the soil used in this research is a weak soil.

Liquid Limit = *39%* Plastic Limit = *Non-plastic* Plasticity Index = 0*%*

**Fig 4.4: Atterberg Limit for Soft Soil with 4% Maize Husk Ash**

Hamisu's research (2022) showed an average atterberg limit of 23.4%, which is much lower than 39%. This is an improvement, and it also indicates that adding 4% maize husk ash improved the soil.

Liquid Limit = *30%* Plastic Limit = *Non-plastic* Plasticity Index = *0%*

**Fig 4.5: Atterberg Limit for Soft Soil with 6% Maize Husk Ash**

The average atterberg limit for the soil, reported by Hamisu (2022), was 23.4%, which is low compared to 30%, depicting that the 6% maize husk ash added helped improve the soil.

Liquid Limit = *38.55%* Plastic Limit = *Non-plastic* Plasticity Index = *0%*

**Fig 4.6: Atterberg Limit for Soft Soil with 8% Maize Husk Ash**

Hamisu (2022) determined that the average atterberg limit for the soil to be23.4%, which is lower than 38.55%, indicating that the weak soil had been improved with 8% maize husk ash.

**4.6 Compaction Characteristics**

**Table 4.9: Compaction Data for Unstablized Weak Soil**

|  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- |
| **No. of Trials** | **1** | | **2** | | **3** | | **4** | |
| Wt. of Mould + Wet Soil (g) | 5690 | | 5830 | | 5770 | | 5681 | |
| Wt. of Mould (g) | 3760 | | 3760 | | 3760 | | 3760 | |
| Wt. of Wet Soil | 1930 | | 2070 | | 2010 | | 1921 | |
| Bulk Density (g/cm3) | 1.930 | | 2.070 | | 2.010 | | 1.921 | |
| Can No. | A1 | C3 | T3 | A3 | E2 | B3 | J1 | W8 |
| Wt. of Empty Can (g) | 10.19 | 10.32 | 10.38 | 7.31 | 10.37 | 10.27 | 10.31 | 10.37 |
| Wt. of Can + Wet Soil (g) | 26.99 | 27.80 | 28.35 | 27.77 | 36.77 | 29.33 | 28.20 | 27.53 |
| Wt. of Can + Dry Soil (g) | 26.13 | 26.87 | 26.86 | 25.74 | 33.37 | 27.00 | 26.10 | 25.16 |
| Wt. of Moisture (g) | 0.86 | 0.93 | 1.49 | 2.03 | 3.35 | 2.33 | 2.10 | 2.37 |
| Wt. of Dry Soil (g) | 15.94 | 16.55 | 16.48 | 18.43 | 23.00 | 16.73 | 15.79 | 14.79 |
| Moisture Content (%) | 5.40 | 5.61 | 9.04 | 11.01 | 14.6 | 13.92 | 13.30 | 16.02 |
| Average MC (%) | 5.51 | | 10.03 | | 14.26 | | 14.66 | |
| Dry Density (g/cm3) | 1.83 | | 1.88 | | 1.75 | | 1.62 | |

MDD = *1.88g/cm3* OMC = *10.03%*

**Fig 4.7: Dry Density and Moisture Content Curve for Unstabilized Soft Soil**

According to research conducted by Enzenwa *et al*., (2022) which indicated that the average MDD and OMC to 1.60 and 16% respectively when compared to the result obtained above it can be deduced that the soil is not suitable for construction.

**Table 4.10: Compaction Data for 2% Stabilized Weak Soil with Maize Husk Ash**

|  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| **No. of Trials** | **1** | | **2** | | **3** | | **4** | | **5** | |
| **Wt. of Mould + Wet Soil (g)** | 5685 | | 5750 | | 5855 | | 5885 | | 5750 | |
| **Wt. of Mould (g)** | 3760 | | 3760 | | 3760 | | 3760 | | 3760 | |
| **Wt. of Wet Soil** | 1925 | | 1990 | | 2095 | | 2125 | | 1990 | |
| **Bulk Density (g/cm3)** | 1.925 | | 1.990 | | 2.095 | | 2.125 | | 1.990 | |
| **Can No.** | V1 | N1 | M1 | C2 | Z1 | Z2 | J2 | U1 | Y1 | W2 |
| **Wt. of Empty Can (g)** | 14.66 | 14.93 | 14.07 | 14.66 | 14.54 | 14.30 | 14.65 | 13.83 | 14.89 | 14.34 |
| **Wt. of Can + Wet Soil (g)** | 34.70 | 33.11 | 26.25 | 25.05 | 36.58 | 35.44 | 32.22 | 28.48 | 37.54 | 35.31 |
| **Wt. of Can + Dry Soil (g)** | 33.82 | 32.23 | 25.46 | 24.42 | 35.12 | 33.80 | 30.50 | 26.98 | 34.06 | 34.71 |
| **Wt. of Moisture (g)** | 0.88 | 0.88 | 0.79 | 0.63 | 1.46 | 1.94 | 1.72 | 1.48 | 3.48 | 3.60 |
| **Wt. of Dry Soil (g)** | 19.16 | 17.30 | 11.39 | 9.76 | 20.58 | 18.85 | 16.20 | 13.15 | 19.17 | 20.37 |
| **Moisture Content (%)** | 4.59 | 5.10 | 6.94 | 6.45 | 7.09 | 10.29 | 10.62 | 11.25 | 18.13 | 17.67 |
| **Average MC (%)** | 4.85 | | 6.69 | | 8.69 | | 10.84 | | 17.90 | |
| **Dry Density (g/cm3)** | 1.84 | | 1.86 | | 1.93 | | 1.91 | | 1.69 | |

MDD = *1.93g/cm3* OMC = *8.69%*

MDD = *1.93g/cm3* OMC = *8.69%*

**Fig 4.8: Dry Density and Moisture Content Curve for 2% Stabilized Soft Soil**

In accordance with the findings of Enzenwa *et al*., (2022), the average MDD and OMC were 1.60 and 16% respectively, indicating that the stabilized soil is not suitable for construction.

**Table 4.11: Compaction Data 4% Stabilized Weak Soil with Maize Husk Ash**

|  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| **No. of Trials** | **1** | | **2** | | **3** | | **4** | | **5** | |
| **Wt. of Mould + Wet Soil (g)** | 5555 | | 5660 | | 5795 | | 5880 | | 5745 | |
| **Wt. of Mould (g)** | 3760 | | 3760 | | 3760 | | 3760 | | 3760 | |
| **Wt. of Wet Soil** | 1795 | | 1900 | | 2035 | | 2120 | | 1985 | |
| **Bulk Density (g/cm3)** | 1.795 | | 1.900 | | 2.035 | | 2.120 | | 1.985 | |
| **Can No.** | A1 | A2 | B1 | B2 | C1 | C2 | A7 | A8 | C3 | A3 |
| **Wt. of Empty Can (g)** | 6.73 | 6.93 | 10.67 | 14.12 | 6.76 | 6.91 | 12.67 | 12.75 | 10.29 | 10.34 |
| **Wt. of Can + Wet Soil (g)** | 14.72 | 13.92 | 18.90 | 15.55 | 20.01 | 23.49 | 25.52 | 22.41 | 28.74 | 35.24 |
| **Wt. of Can + Dry Soil (g)** | 14.44 | 13.67 | 18.50 | 13.94 | 19.15 | 22.29 | 24.43 | 22.04 | 24.75 | 32.10 |
| **Wt. of Moisture (g)** | 0.28 | 0.25 | 0.40 | 1.61 | 0.86 | 1.20 | 1.09 | 0.37 | 3.99 | 3.14 |
| **Wt. of Dry Soil (g)** | 7.71 | 6.74 | 7.83 | 6.82 | 12.39 | 15.38 | 11.76 | 9.29 | 14.46 | 21.76 |
| **Moisture Content (%)** | 3.36 | 3.71 | 5.12 | 23.1 | 6.94 | 7.80 | 9.27 | 3.98 | 27.59 | 14.43 |
| **Average MC (%)** | 3.36 | | 6.63 | | 7.37 | | 14.12 | | 21.01 | |
| **Dry Density (g/cm3)** | 1.74 | | 1.78 | | 1.89 | | 1.85 | | 1.64 | |

MDD = *1.89g/cm3* OMC = *7.37%*

MDD = *1.89g/cm3* OMC = *7.37%*

**Fig 4.9: Dry Density and Moisture Content Curve for 4% Stabilized Soft Soil**

The MDD and OMC averages were 1.60 and 16%, respectively, according to Enzenwa *et al*., (2022), indicating that the stabilized weak soil in this research cannot be used for construction.

**Table 4.12: Compaction Data 6% Stabilized Weak Soil with Maize Husk Ash**

|  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| **No. of Trials** | **1** | | **2** | | **3** | | **4** | | **5** | |
| **Wt. of Mould + Wet Soil (g)** | 5590 | | 5765 | | 5860 | | 5870 | | 5810 | |
| **Wt. of Mould (g)** | 3760 | | 3760 | | 3760 | | 3760 | | 3760 | |
| **Wt. of Wet Soil** | 1830 | | 2005 | | 2100 | | 2110 | | 2050 | |
| **Bulk Density (g/cm3)** | 1.830 | | 2.005 | | 2.100 | | 2.110 | | 2.050 | |
| **Can No.** | A1 | A2 | B2 | B1 | C1 | C2 | A7 | A11 | C3 | A3 |
| **Wt. of Empty Can (g)** | 6.80 | 6.94 | 7.12 | 10.68 | 6.76 | 6.91 | 12.70 | 12.78 | 10.28 | 10.33 |
| **Wt. of Can + Wet SOIL (g)** | 21.52 | 24.31 | 20.65 | 21.53 | 16.70 | 19.66 | 26.31 | 30.64 | 33.82 | 30.20 |
| **Wt. of Can + Dry Soil (g)** | 20.87 | 23.43 | 19.77 | 20.79 | 15.89 | 18.55 | 24.94 | 28.80 | 31.03 | 27.77 |
| **Wt. of Moisture (g)** | 0.64 | 0.88 | 0.88 | 0.74 | 0.81 | 1.11 | 1.37 | 1.84 | 2.79 | 2.43 |
| **Wt. of Dry Soil (g)** | 14.00 | 16.49 | 12.65 | 10.11 | 9.13 | 11.64 | 12.24 | 16.62 | 20.75 | 17.44 |
| **Moisture Content (%)** | 4.57 | 5.33 | 6.95 | 7.34 | 8.87 | 9.83 | 11.19 | 11.49 | 13.45 | 13.93 |
| **Average MC (%)** | 4.95 | | 7.15 | | 9.20 | | 11.34 | | 13.69 | |
| **Dry Density (g/cm3)** | 1.74 | | 1.87 | | 1.92 | | 1.89 | | 1.80 | |

MDD = *1.92g/cm3* OMC = *9.20%*

MDD = 1.92g/cm3 OMC = 9.20%

**Fig 4.10: Dry Density and Moisture Content Curve for 6% Stabilized Soft Soil**

When compared to the results obtained above it can be concluded that the stabilized soil is not suitable for construction based on the MDD and OMC results 1.6 and 16% respectively obtained by Enzenwa *et al*., (2022).

**Table 4.13: Compaction Data 8% Stabilized Weak Soil with Maize Husk Ash**

|  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| **No. of Trials** | **1** | | **2** | | **3** | | **4** | | **5** | |
| **Wt. of Mould + Wet Soil (g)** | 5570 | | 5770 | | 5840 | | 5690 | | 5580 | |
| **Wt. of Mould (g)** | 3760 | | 3760 | | 3760 | | 3760 | | 3760 | |
| **Wt. of Wet Soil** | 1810 | | 2010 | | 2080 | | 1930 | | 1820 | |
| **Bulk Density (g/cm3)** | 1.810 | | 2.010 | | 2.080 | | 1.930 | | 1.820 | |
| **Can No.** | Y2 | Z2 | V2 | C2 | W1 | W2 | Z1 | Z3 | Y3 | Y8 |
| **Wt. of Empty Can (g)** | 14.30 | 14.66 | 14.67 | 14.65 | 14.06 | 14.32 | 14.56 | 14.94 | 14.06 | 14.16 |
| **Wt. of Can + Wet SOIL (g)** | 23.95 | 26.27 | 25.76 | 26.11 | 30.27 | 31.50 | 30.66 | 40.59 | 40.66 | 41.40 |
| **Wt. of Can + Dry Soil (g)** | 23.45 | 25.61 | 24.94 | 24.08 | 28.61 | 29.61 | 28.61 | 37.31 | 37.40 | 38.16 |
| **Wt. of Moisture (g)** | 0.50 | 0.66 | 0.82 | 1.03 | 1.66 | 1.89 | 2.02 | 3.28 | 3.26 | 3.24 |
| **Wt. of Dry Soil (g)** | 9.15 | 10.95 | 10.27 | 10.43 | 14.55 | 15.29 | 14.03 | 22.37 | 23.34 | 24.00 |
| **Moisture Content (%)** | 5.46 | 6.03 | 7.98 | 9.87 | 11.41 | 12.36 | 14.59 | 14.66 | 13.97 | 13.50 |
| **Average MC (%)** | 5.75 | | 8.93 | | 11.89 | | 13.63 | | 14.63 | |
| **Dry Density (g/cm3)** | 1.71 | | 1.85 | | 1.86 | | 1.73 | | 1.68 | |

MDD = *1.86g/cm3* OMC = *11.89%*

MDD = 1.86g/cm3 OMC = 11.89%

**Fig 4.11: Dry Density and Moisture Content Curve for 8% Stabilized Weak Soil**

The compaction test results shown in table 4.13 and Figure 4.11. The optimum moisture content (OMC) was observed to have decreased from 10.03% at 0% to 8.69% at 2%, 7.37% at 4%, 9.20% at 6% and an increase to 11.89% at 8%. The decrease and increase in optimum moisture content with in maize husk ash content could have been as a result of decreasing and increasing demand for water by various soil and maize husk ash mineral particles to undergo hydration reaction. On the other hand, the maximum dry density (MDD) increased from 2%, 4%, 6% and decreased at 8% with maize husk ash content. This increase in MDD agrees with report relating that cementitious stabilizers generally increase the maximum dry density of soils at optimal amount between 2 – 8%.

**4.6 California Bearing Ratio**

**Table 4.14: California Bearing Ratio**

|  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
|  | 0% | | 2% | | 4% | | 6% | | 8% | |
| Pen. | Top | Bottom | Top | Bottom | Top | Bottom | Top | Bottom | Top | Bottom |
| 0.5 | 30 | 35 | 35 | 39 | 45 | 47 | 38 | 38 | 33 | 36 |
| 1.0 | 65 | 78 | 47 | 50 | 77 | 83 | 56 | 57 | 45 | 49 |
| 1.5 | 90 | 101 | 59 | 65 | 89 | 92 | 71 | 73 | 57 | 60 |
| 2.0 | 105 | 117 | 68 | 78 | 101 | 105 | 82 | 85 | 66 | 69 |
| 2.5 | 114 | 122 | 80 | 97 | 115 | 118 | 98 | 101 | 78 | 80 |
| 3.0 | 125 | 138 | 96 | 107 | 125 | 129 | 103 | 107 | 94 | 101 |
| 3.5 | 137 | 150 | 103 | 118 | 135 | 140 | 112 | 113 | 99 | 110 |
| 4.0 | 148 | 161 | 112 | 129 | 147 | 150 | 121 | 125 | 110 | 119 |
| 4.5 | 159 | 172 | 122 | 137 | 158 | 163 | 132 | 135 | 120 | 125 |
| 5.0 | 165 | 185 | 134 | 150 | 171 | 181 | 139 | 142 | 132 | 138 |
| 5.5 | 180 | 198 | 145 | 161 | 182 | 197 | 145 | 147 | 143 | 147 |
| 6.0 | 189 | 202 | 153 | 169 | 199 | 203 | 152 | 161 | 152 | 162 |
| 6.5 | 192 | 219 | 162 | 173 | 201 | 218 | 160 | 169 | 160 | 171 |
| 7.0 | 205 | 225 | 169 | 187 | 225 | 230 | 171 | 172 | 167 | 182 |
| 7.5 | 212 | 239 | 172 | 199 | 229 | 243 | 183 | 182 | 170 | 193 |
| 8.0 | 220 | 250 | 179 | 201 | 238 | 254 | 192 | 189 | 177 | 199 |
| 8.5 | 232 | 262 | 181 | 209 | 250 | 268 | 201 | 195 | 182 | 203 |
| 9.0 | 242 | 272 | 183 | 210 | 261 | 273 | 205 | 203 | 187 | 212 |
| 9.5 | 30 | 35 | 187 | 218 | 271 | 293 | 209 | 209 | 192 | 223 |
| 10 | 65 | 78 | 195 | 220 | 287 | 303 | 213 | 215 | 201 | 229 |

**Fig 4.12: CBR @ 0%, 2%, 4%, 6% & 8%**

Fig. 4.12 shows the CBRvalue in relation to unstabilized i.e. 0% and stabilized specimens. However, the unstabilized weak soil had the highest CBR value of 265, when treated with 2% Maize Husk Ash there was a decrease as seen in the figure above, the weak soil shows an increase with the highest CBR value of 287 when 4% of Maize Husk Ash, and thereafter the CBR decreased with the further addition of 6% and 8% maize husk ash respectively. The highest CBR value is 287%, which is obtained at an inclusion of 4% Maize Husk Ash into the weak soil, but further additional Maize Husk Ash tends to decrease the CBR value. This behavior may be attributed to the compaction resistance of the Maize Husk Ash and the fact that the Maize Husk Ash had a lower specific gravity than soils. Nataraj and McManis (1997) noted that the interaction between the soil and the fiber reinforcement controlled the response of the soil/fiber mixture to compaction.

**CHAPTER FIVE**

**CONCLUSIONS AND RECOMMENDATIONS**

**5.1 Introduction**

This chapter contains a summary of the conclusions developed based on the results of this research study.

**5.2 Conclusions**

From the research conducted, the following conclusions can be drawn.

* The compaction results show that the optimum moisture content (OMC) was observed to have decreased from 10.03% at 0% to 8.69% at 2%, 7.37% at 4%, 9.20% at 6% and an increase to 11.89% at 8%.
* The maximum dry density (MDD) increased from 2%, 4%, 6% and decreased at 8% with maize husk ash content. This increase in MDD agrees with report relating that cementitious stabilizers generally increase the maximum dry density of soils at optimal amount between 2 – 8%. The strength of the weak soil was improved to 1.93g/cm3, 1.89g/cm3, 1.92g/cm3 and 1.86g/cm3 respectively, hence strength performance suggests that stabilization reactions were relatively permanent.
* As such the stabilized weak soil can be used as sub-base and in low strength required soil for fill, etc.

**5.2 Recommendations**

* From the results of the tests conducted, it is recommended that 2%, 4% and 6% of maize husk ash by weight of the soil be considered for use, as it causes significant improvement on the weak soil properties.
* Further research study should be carried out to determine the effectiveness of stabilization of weak soil, using maize husk ash at varying percentages greater than 8% and at longer curing period, e.g., 3, 7, and 28 days.

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