EVALUATION OF STRENGHT CHARACTERISTICS OF CLAYEY SOIL STABILIZED WITH BAMBOO ASH POWDER AND LIME STABILIZERS

BY

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MAY,2023.

CERTIFICATION

I, **UGOCHUKWU MAC-ANTHONY OTITO**, an undergraduate student in the department of Civil Engineering, Nnamdi Azikiwe University Awka with registration number **2017224009** submitted this Project in partial fulfilment of the requirements for the award of degree of Bachelor of Engineering (B. ENG) in Civil Engineering.

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APPROVAL PAGE

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DEDICATION

This project work is dedicated to GOD Almighty who made it possible that I passed through the school and full of wisdom. May his name be praised forever, Amen. I also dedicate this project to my parents, Mr.&MRS. UGOCHUKWU who made it possible that lacked nothing, I say THANK YOU.

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ABSTRACT

In the past years it has been hard for most civil engineering works when it comes to the use of clayey soil because most clayey soils have low bearing strength and expansive nature and its causes shrinkage and swelling. This thesis describes the evaluation of strength of clayey soil using bamboo ash powder and lime as stablizers. Clay soil was collect from bakasi at unizik and the natural moisture content was determined and it was air dried for one week to allow particial elimination of natural water while lime was bought in onitcha and the bamboo was gotten from bamboo plantation located at Enugu-Agidi and it was air dried for 3days and burnt. The clay was then stabilized by adding lime at intervals 2%, 4%, 6%, 8%, and 10% and also the clay was also stabilized by adding bamboo ash powder at intervals 2.5%, 5%, 7.5%, 10% and 12.5%. The clay soil was now stabilized with mix combination of lime and bamboo ash in the varying percentage used early, a total of 5 mix combination was gotten and it was subjected to Compaction and CBR test which we used to deduce the strength properties of the clay soil when stabilized with bamboo ash and lime, other test we conducted was sieve analysis, specific gravity and Atterberg limit. Results gotten after the sieve analysis of the clay soil after passing 0.075mm sieve is the soil is fine grained soil because of its small particles and its was classified as clay of high plasticity(CH) according to USCS. Results obtained from the composite mixture of clay, lime and bamboo ash powder revealed that the blend of lime and bamboo ash powder increased the specific gravity of natural clay from 2.66 to 2.72 at 4%lime and 5% bamboo ash powder, beyond this point, the specific gravity of the natural clay decreased and it was caused by the high content of bamboo ash powder. Result obtained from Atterberg test for natural clay stabilized with a blend of lime and bamboo ash powder shows that a blend of lime and bamboo ash powder decreased the liquid limit, plastic limit and plasticity index of natural clayey soils. It was also observed that beyond 6% lime and 7.5% bamboo ash powder, the mixture became non plastic. Results obtained from compaction of natural clayey samples stabilized with a blend of lime and bamboo ash powder shows that a blend of lime and bamboo ash powder enhanced the maximum dry unit weight of clay from its natural value of 20.37kN/m^3 to 22.27kN/m^3 at 8% lime and 10% bamboo ash powder. The results obtained therefore, suggest that a blend of lime and bamboo ash powder is more effective in improving the compaction characteristics of natural clayey soils than when lime and bamboo ash powder is used as separate entities.

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INTRODUCTION

1.1 Background of Study

Naturally occurring expansive soils (clayey soils) have been encountered in various regions across the globe (Chen, 2012). Clay soils are generally composed of micro-crystalline particles of a group of minerals ranging from montmorillonite, illite, smectite and vermiculite (Amadi, et al., 2011). Clay soils are variable and complex in nature but due to their relative abundance and low cost, they are frequently utilized in construction works (Roohbakhshan and Kalantari, 2013). These soils are frequently encountered and used as materials for foundation of pavement (flexible and rigid), buildings, slurry walls, landfill liners and other civil engineering infrastructures (Firoozi and Baghini, 2016).

Properties of clayey soils especially strength properties depend significantly on the mineralogical composition and vary with location and depth (Onyelowe, 2016). Clayey soils most times constitute the sub-grade material of pavement (Amadi, et al., 2011). According to Jones and Jefferson, (2012), these volume changes can either be in form of swell or in form of shrinkage and this is why they are occasionally regarded as swell/shrink soils. Clayey soils with higher percentage of clay minerals like montmorillonite, expandable illite, smectite vermiculite, are susceptible to swelling and shrinkage and will ultimately have very low shear strength (Amadi, et al., 2011). The expansive nature and low bearing strength of clay are some of the factors responsible for failures of civil engineering structures (Amadi, et al., 2011). Furthermore, clay soils are generally stiff in dry state but when saturated, they lose their stiffness (Firoozi and Baghini, 2016). This behaviour of clayey soils makes them unsuitable for civil engineering application, hence the need for cost effective improvement of certain undesirable properties (strength properties) of clay samples to meet the necessary geotechnical requirement for use as a construction material in foundation of pavement, buildings, slurry walls, retaining walls, bridges becomes necessary.

Virtually all agricultural activities results to the generation of waste and in most occasions, there are problems associated with the disposal of this waste (Amu and Adetuberu, 2010). In recent years, there has been intensified research towards the use of agricultural by-products in geotechnical engineering. The use of this alternative material will be advantageous in two folds: conservation of natural resources and reduction in the volume of waste generated (Amu and Adetuberu, 2010). The use of bamboo ash powder as an alternative material for improvement of undesirable properties of clayey soil will be explored by the study.

Soil stabilization is a technique introduced many years ago with the main aim of rendering deficient soils capable of meeting the requirements of specific engineering projects (Amadi, et al., 2011). Soil stabilization incorporates the use of additives such as lime, cement and occasionally bamboo ash powder as binder where necessary to reduce swelling and ehane strength properties of soils or may involve an admixture such as coal, fly ash, mainly with the aim of increasing the bulk size of construction materials as well as ensuring waste reduction (Amadi, et al., 2011). Lime stabilization refers to the stabilization of soil by the addition of burned limestone products, either calcium oxide, CaO, or calcium hydroxide, $Ca(OH)_2$ (Roohbakhshan and Kalantari, 2013). Quicklime is the most frequently used lime product for lime stabilization. On the other hand, hydrated lime is used more often than quicklime, although use of quicklime in soil stabilization has gained credence recently (Amadi, et al., 2011). Generally speaking quicklime seems to be a more effective stabilizer of expansive soils than hydrated lime. Furthermore, when quicklime, in slurry form, is added to expansive soils (clayey soils) a higher strength is developed than when lime is added in powder form (Roohbakhshan and Kalantari, 2013). The addition of lime to clay soils produces an improved construction material (Amadi, et al., 2011). Hence soil stabilization with lime has been used in highway, railroad and airport construction to improve bases and sub-bases. Clay stabilized with lime is also used in the construction of embankments, soil exchange in sliding slopes, backfill for bridge abutments and retaining walls and for soil improvement beneath foundation of slab for rigid pavement construction(Amadi, et al., 2011)

In other to reduce problems associated with disposal of agricultural waste, ensure rational use of earth resources and improve the strength properties of clayey soils, this study will therefore evaluate the strength properties of clay stabilized with lime and bamboo ash powder.

1.2 Statement of Problem

Clay soil is one of the commonest materials encountered during at sub-grade level of pavement and foundations of buildings (Amadi, et al., 2011). Properties of clayey soils especially strength properties depend significantly on the mineralogical composition and vary with location and depth (Onyelowe, 2016). Clayey soils with higher percentage of clay minerals like montmorillonite, expandable illite, smectite and vermiculite, are susceptible to swelling and shrinkage and will ultimately have very low shear strength (Amadi, et al., 2011). These soils when encountered in site causes costly damages as a result of their low bearing strength and expansive nature. These high swelling, shrinkage and low shear strength of clay properties makes them unsuitable for civil engineering application, hence there is need to ensure cost effective improvement of certain undesirable properties (strength) of clay soils to make them fit for civil engineering application.

In other to proffer solution to some of the aforementioned problems associated with expansive nature and low strength of clayey soils, this study will therefore assess the strength properties of clay stabilized with bamboo ash powder and lime with respect to engineering/geotechnical requirement for construction.

1.3 Aim and Objectives of Study

The aim of the study is evaluate the strength characteristics of clayey soils stabilized with bamboo ash powder and lime as engineering material while the objectives are:

I. Produce bamboo ash powder and determine its chemical composition.
II. Procure lime used as additive for strength enhancement of clayey soil.
III. Determine the index properties of the natural clayey soils.
IV. Obtain bamboo ash powder and lime and thereafter analyze their effect on geotechnical properties of natural clay soils.
V. Determine the amount of bamboo ash powder and lime required for optimum improvement in strength properties of natural clayey soils.
VI. Draw conclusion and make recommendation based on key findings obtained.

1.4 Significance of Study

Key findings obtained from the experimental study on strength properties of clay stabilized with bamboo ash powder and lime will be significant in the following ways:

1. Serve as a waste treatment process through reduction in the volume of bamboo extract generated.
2. Serve as cost effective means of enhancing undesirable properties of natural clayey soils.

3. Foster economy in construction through reduction in cost of excavation and importation of materials with improved strength properties for replacement of natural clayey soils.
4. Serve as reference or body of knowledge for further investigation and field construction.
5. Serve as an alternative to Portland cement especially in the treatment of clayey soils.

1.5 Scope of Study

The study will focus essentially on investigating the strength properties of clay stabilized with bamboo ash powder and lime. Strength properties of clayey soils stabilized with lime, bamboo ash powder and a blend of lime with bamboo ash powder will be investigated. Bamboo ash powder will be added to the natural clayey samples in a stepped increase of 2.5% to 12.5% by dry weight of clayey sample while lime will be added to the natural clay sample in a stepped increase of 2% to 10% by dry weight of clay. A blend of lime and bamboo ash powder will also be added to clay at similar percentages specified for the individual combinations. Laboratory testing will be used as a tool for the strength evaluation process. The laboratory test to be conducted includes: sieve analysis test, specific gravity test, atterberg limit (liquid and plastic limit) test, compaction and California bearing ratio test. Compaction and California bearing ratio test will be used as basis for making deduction on strength properties of clay stabilized with bamboo ash powder and lime.

CHAPTER TWO

LITERATURE REVIEW

2.1 Overview of Clay Soils

Clayey soils are particles with the diameters of which are less than 0.005 millimeter; they are formed from weathering of rock. Rock in this sense includes soils, ceramic clays, clay shale, mudstones, glacial clays (including great volumes of detrital and transported clays), and deep-sea clays (red clay, blue clay, and blue mud). These are all characterized by the presence of one or more clay minerals, together with varying amounts of organic and detrital materials, among which quartz is predominant (Sherwood, 1993). Clay materials are sticky when wet and hard when dry and are product of weathering (Amadi, et al., 2011).

Clay soil when compared to other soil type has large surface area due to flat and elongated particle shapes (Amadi, et al., 2011). On the other hand, silt materials can be sensitive to small change in moisture and, therefore, may prove difficult during stabilization (Sherwood, 1993). Clayey soils are usually categorized as expansive soils. Other names of these soils are soft soils or fine-grained soils. These types of soils invariably result to critical damage to structures resting on them. Normally, in construction industries, the structures that constructed on clay soils are tend to trigger the soil when exposed to additional load as well as external impact (Firoozi and Baghini, 2016). This deformation could potentially cause significant failure to foundation and structures. Besides, the construction of roadway on the soft soils also encounters the same problem. This is because the soils do not have enough physical properties for construction application.

Naturally occurring expansive soils is found virtually everywhere across the globe. Soils with higher percentage of clay minerals like montmorillonite, expandable illite and vermiculite, are susceptible to swelling and shrinkage. They cause numerous costly damages to the roadways, buildings, bridges and other civil engineering infrastructures. Furthermore, clay soils are generally stiff in dry state but when become saturated, they lose their stiffness. Soft clays are characterized by low compressive strength and excessive compressibility. The reduction in bearing capacity of soft clays results in compressive failure and excessive settlement, leading to severe damage to buildings and foundations (Chen, 1988; Phanikumar and Nagaraju, 2018; Ikeagwani and Nwonu, 2018). Maintenance and rehabilitation costs for the infrastructure on these soils reach billions of naira annually. These problems primarily stem from the presence of montmorillonite clay minerals which are derived from basic and ultra basic igneous rocks; essentially the minerals area by product of the decomposition of these rocks.

These minerals swell when moisture is introduced and shrink when the same moisture is retracted. In the case where the soil undergoes excessive heat, i.e. drought, expansive soils tend to contract and shrink excessively (Firoozi and Baghini, 2016). Clay minerals and cations come in various forms and that it is the relative quantities of each type of these minerals that are important factors contributing to the swell/shrink behavior along with the dry density, soil structure, and loading conditions present. Other researchers added that the arid climate, alkaline environment, and local geology are accountable for the expansive nature of soils.

This section will therefore review relevant literatures on properties of clayey soils, bamboo ash powder, lime and effect of lime and bamboo ash powder on strength properties of clayey soils.

2.2 Review of Geotechnical Properties of Clayey Soils

2.2.1 Particle Size Distribution

Clay has the smallest particle size of any soil type, with individual particles being so small that they can only be viewed by an electron microscope. This allows a large quantity of clay particles to exist in a relatively small space, without the gaps that would normally be present between larger soil particles. This feature plays a large part in clay's smooth texture, because the individual particles are too small to create a rough surface in the clay.

Consequently great importance has also been accorded to particle-size distribution in dealing with clayey soils. Recent studies have revealed that clayey soils are strikingly different from temperate zone soils in terms of genesis and structure. Their concretionary structure as compared to the dispersed temperate zone soils has necessitated modifications to mechanical or grading tests (Remillion, 1997). Consistent reports of variations in the particle-size distribution with methods of pretreatment and testing have been widely reported on clayey soils. Schofield (1957) found out that wet sieving increased the silt and clay fraction from 7 to 20% as compared to the dry sieving. It has been found that sodium hexametaphosphate generally gives better dispersion of the fine fractions. It was also found, for example, that using sodium oxalate on a halloysitic clay from Kenya gave between 20 and 30% clay fraction, while the sodium hexametaphosphate gave as high as between 40 to 50% clay fraction for the same soil (Quinones, 1963).

2.2.2 Plasticity

Textural clayey soils are very variable and may contain all fractions sizes; boulders, cobbles, gravel, sand, silt, and clay as well as concretionary rocks. The interaction of the soil particles at the micro scale is reflected in the atterberg limits of the soil at micro scale level. Knowledge of the atterberg limits may provide the following information:

I. A basis for identification and classification of a given soil texture.
II. Strength and compressibility characteristics swell potential of the soil or the water holding capacity.

Atterberg limit depends on:

I. The clay content**:** plasticity increases with increase in clay content (Piaskowski, 1993).
II. Nature of soil minerals**:** only minerals with sheet-like or plate-like structures exhibit plasticity. This is attributed to the high specific surface areas and hence the increased contact in the shaped particles.
III. Chemical composition of the soil environment**:** the absorptive capacity of the colloidal surface of the actions and water molecules decrease as the ratio of silica to sesquioxides decreases (Baver, 1980).
IV. Nature of exchangeable actions**:** this has a considerable influence upon the soil plasticity (Hough, 1989).

Pre-test preparation, degree of molding and time of mixing, dry and re-wetting, and irreversible changes may affect the plasticity of soil. Drying drives off absorbed water, which is not completely regained, on re-wetting (Fookes, 1997).Studies on the relationship between the natural moisture content, liquid limits and plastic limits of clay have shown that generally the natural moisture contents is less than the plastic limit in normal clayey soils (Vargas, 1993). However, the clay soil from high rain fall areas may have moisture contents as high as the liquid limit (Hirashima, 1979).

2.2.3 Compaction Characteristics

The compaction characteristics of clayey soils are determined by their grading characteristics and plasticity of fines (Firoozi and Baghini, 2016). Most clayey soils contain a mixture of quartz and concretionary coarse particles, which may vary from very hard to very soft (Firoozi and Baghini, 2016). The strength of these particles has major implications in terms of field and laboratory compaction results and their subsequent performance in civil engineering construction projects (Firoozi and Baghini, 2016). Placement variables (moisture content, amount of compaction, and type of compaction efforts) also influence the compaction characteristics. Varying each of these placement variables has an effect on permeability, compressibility, strength and stress-strain characteristics of the soil.

2.2.4 Shear Strength Characteristics

Shear strength is a term used in soil mechanics to describe the magnitude of the shear stress that a soil can sustain. The shear strength of a lateritic soil is a function of the friction and interlocking of particles (soil angle of internal friction) and possibly cementation or bonding at particle contact relative to total and effective stress. Due to cohesion, particulate materials may expand or contract in volume as it is subject to shear strains. If soil expands in volume, the density of particles will decrease and the strength will decrease likewise the shear strength.

The cohesion is attributable to the resultant of inter particle forces which are mainly associated with the clay-size particle of soils and will vary with the particle size and the distance separating them. The angle of internal friction included the effect of interlocking. The interlocking effect is affected to some degree by the shape of particles and the grain–size distribution. The two parameters cohesion (**c**) and angle of friction (ø) depends on the grading, particle shape and void ratio factors of the soil. Cohesion also depends on degree of saturation, while angle of internal friction does not (Gidigasu, 1976).

The shear strength characteristics of lateritic soils have been found to depend significantly on the parent materials, and the degree of weathering which in turn depends on the position of the sample in the soil profile and compositional factors as well as the pretest preparation of the samples (Lohnes, 1988).

2.2.5 Consolidation and Compaction

When a soil mass is subjected to a compressive force, its volume decreases. The property of the soil due to which it decrease in volume occurs under compressive force is known as the compressibility of soil. The compression of soil can occur due to;

1. Compression of solid particles and water in the void
2. Compression and expulsion of air in the void
3. Expulsion of water in the voids

The compression of saturated soil under a steady static pressure is known as consolidation. It is entirely due to expulsion of water from the voids. The consolidation characteristics of clayey soils is generally moderate with the modulus of compressibility ranging between $1 \text{ x } 10^{-3}$ to 1×10^{-2} sq. ft./ton.

2.2.6 Specific Gravity

The available data indicate that specific gravities vary not only with the textural soil groups but also within different fractions. In the first place, clayey soils have been found to have very high specific gravities of between 2.6 to 3.0 (Amadi, et al., 2011). For the same soil, gravel fractions were found to have higher specific gravities than fine fractions due to the concentration of iron oxide in the gravel fraction while alumina is concentrated in the silt and clay fractions (Amadi, et al., 2011). The average of the two values can be assumed to be more representative of the specific gravity for the whole soil.

2.2.7 Permeability Characteristics

One of the problems with clay soil is its slow permeability resulting in a very large water retention capacity (Amadi, et al., 2011). Because the soil particles are small and close together, it takes water much longer to move through clay soil than it does with other soil types (Amadi, et al., 2011). Clayey soils experiences significant volume change with change in moisture content. The minerals present in clayey soils are an index of the permeability of clayey soils (Amadi, et al., 2011). Clayey soils containing substantial amount of minerals such as illite, smectite are like to be less permeable than clay containing infinitesimal amount of minerals, therefore, the permeability properties of clayey soils is largely influenced by the its mineralogical composition (Amadi, et al., 2011).

2.3 Review of Chemical Properties of Clayey Soils

2.3.1 Ion Exchange

Depending on deficiency in the positive or negative charge balance of mineral structures, clay minerals are able to absorb certain cations and anions and retain them around the outside of the structural unit in an exchangeable state, generally without affecting the basic silicate structure (Amadi, et al., 2011). These absorbed ions are easily exchanged by other ions. The exchange reaction differs from simple sorption because clayey soils have quantitative relationship between reacting ions (Firoozi and Baghini, 2016). Exchange capacities vary with particle size, perfection of crystallinity, and nature of the absorbed ion; hence, a range of values exists for a given mineral rather than a single specific capacity (Amadi, et al., 2011). With certain clay minerals such as, allophane, and to some extent kaolinite that have hydroxyls at the surfaces of their structures, exchange capacities of such clay minerals vary with the pH (index of acidity or alkalinity) of the medium, which greatly affects dissociation of the hydroxyls (Amadi, et al., 2011).

2.3.2 Clay Mineral – Water Interactions

Clay materials contain water in several forms. The water may be held in pores and may be removed by drying under ambient conditions (Sherwood, 1993). Water also may be absorbed on the surface of clay mineral structures and in smectites, vermiculites, hydrated halloysite, and sepiolite (Amadi, et al., 2011). This water may occur in interlayer positions or within structural channels (Amadi, et al., 2011). The water absorbed between layers or in structural channels may further be divided into zeolitic and bound waters (Amadi, et al., 2011). The latter is bound to exchangeable cations or directly to the clay mineral surfaces. Both forms of water may be removed by heating to temperatures on the order of 100°–200° C and in most cases, except for hydrated halloysite which are are regained readily at ordinary temperatures. It is generally agreed that the bound water has a structure other than that of liquid water; its structure is most likely that of ice (Firoozi and Baghini, 2016). As the thickness of the absorbed water increases outward from the surface and extends beyond the bound water, the nature of the water changes either abruptly or gradually to that of liquid water (Sherwood, 1993). Ions and molecules absorbed on the clay mineral surface exert a major influence on the thickness of the adsorbed water layers and on the nature of this water (Firoozi and Baghini, 2016). The non liquid water may extend out from the clay mineral surfaces and Hydroxyl ions are driven off by heating clay minerals to temperatures of 400°–700° C. The rate of loss of the hydroxyls and the energy required for their removal are specific properties of the various clay minerals. This dehydroxylation process results in the oxidation of Fe^{2+} to Fe^{3+} in ferrous-iron-bearing clay minerals (Amadi, et al., 2011).

The water-retention capacity of clay minerals is generally proportional to their surface area (Amadi, et al., 2011). As the water content increases, clays become plastic and then change to a near-liquid state (Firoozi and Baghini, 2016). The amounts of water required for the two states are defined by the plastic and liquid limits, which vary with the kind of exchangeable cations and the salt concentration in the absorbed water (Amadi, et al., 2011). The plasticity index (PI), which is the difference between the liquid limi and plastic limit of the difference between the two limits, gives a measure for the rheological (flowage) properties of clays (Amadi, et al., 2011). A good example is a comparison of the plasticity index of montmorillonite with that of allophane. The former (montmorillonite) is considerably greater than of the latter (allophone), indicating that montmorillonite has a prominent plastic nature (Amadi, et al., 2011). Such rheological properties of clay minerals have great impact on building foundations, highway construction, chemical engineering, and soil structure in agricultural practices.

2.3.3 Hydraulic Conductivity

Hydraulic conductivity of clayey soils is mostly affected by bulk density and swelling pressure (Amadi, et al., 2011). High density and low electrolyte content of the clay mineral give rise to a very low conductivity for sodium smectite (Sherwood, 2013). On the other hand, the conductivity of calcium smectite is slightly higher because of its low densities (Amadi, et al., 2011). Hydraulic conductivity is depended on density at fluid saturation for different clay minerals. If the hydraulic gradient is high, the particles can also move and this affects the hydraulic conductivity. Thus, particle sand aggregates, that are set free, can be transported by flowing pore water to narrow parts of the pore spaces and cause clogging (Hicks, 2002).

2.3.4 Organic Contents

Clay contains very little organic material (Amadi, et al., 2011). It is a conventional practice to add amendment to plant grown on clayey soils (Amadi, et al., 2011). Without added organic material, clayey soils typically lacks the nutrients and micronutrients essential for plant growth and photosynthesis (Firoozi and Baghini, 2016). Clayey soil containing substantial amount of organic matter may be alkaline in nature resulting in the need for additional amendments to balance pH of the soil. It's important to test clay containing substantial amount of organic matter in other to determine the PH and ascertain whether there is a deficiency in important nutrients such as nitrogen, phosphorus and potassium (Amadi, et al., 2011).

2.4 Stabilization

Soil stabilization is not new, but man has sought to accomplish it by various means almost since the first roads were built, but it is only in recent years that scientific methods has been applied to soil stabilization (Olugbenga and Adetuberu, 2013). Soil stabilization maybe defined as the process of blending and mixing materials with a soil so as to improve certain properties of the soil. A stabilized material may be considered as a combination of binder soil and aggregates preferably obtained at or near the site of stabilization manipulated and treated with or without admixtures, and compacted so that it will remain in its compacted state without detrimental change in shape or volume under applied force or exposure to weather.

Stabilization signifies improvement in both strength and durability which are related to performance. Increase in strength may be expressed quantitatively in terms of compressive strength, shearing strength, or some measure of bearing value or load deflection to indicate the load bearing quality (Olugbenga and Adetuberu, 2013). Stabilization is a method of processing available materials for the production of low cost roads and other civil engineering projects. In this type of project, design and construction, emphasis is definitely placed on the effective utilization of local materials, with a view to decrease construction cost. In some areas, naturally occurring aggregates and soil aggregate combinations exists which requires minimum processing for successful stabilization (Olugbenga and Adetuberu, 2013). While in other places, the natural soils are of unfavorable character and require modification through the use of suitable components such as gravels, crushed stones, geosynthetics, natural fibers or clay binder. While in other areas, admixtures like bituminous materials, lime or Portland cement must be used for effective stabilization. The type and degree of stabilization is dependent on the availability and cost of the required materials.

2.4.1 Methods of Soil Stabilization

In road construction projects, soil or gravelly material is used as the road main body in pavement layers. To have required strength against tensile stresses and strains spectrum, the soil used for constructing pavement should have special specification. Through soil stabilization, unbound materials can be stabilized with cementitious materials (cement, lime, fly ash, bitumen or combination of these). The stabilized soil materials have a higher strength, lower permeability and lower compressibility than the native soil (Keller, 2014). The method can be achieved in two ways, namely:

1. In-situ soil stabilization
2. Ex-situ soil stabilization

Stabilization is not necessary a magic wand by which every soil properties can be improved for better. The decision to technological usage depends on which soil properties have to be modified. The chief properties of soil which are of interest to engineers are volume stability, strength, compressibility, permeability and durability (Altabba and Evans, 2015). Some stabilization technique includes mechanical and chemical stabilization.

2.4.1.1 Mechanical Stabilization

Mechanical Stabilization is the process of improving the properties of the soil by changing its gradation moisture (Onyelowe and Chibuzor, 2012). This process includes soil compaction and densification by application of mechanical energy using various sorts of rollers, rammers, vibration techniques and sometime blasting. The stability of the soil in this method relies on the inherent properties of the soil material moisture (Onyelowe and Chibuzor, 2012). Two or more types of natural soils are mixed to obtain a composite material which is superior to any of its components. Mechanical stabilization is accomplished by mixing or blending soils of two or more gradations to obtain a material meeting the required specification.

2.4.1.2 Chemical Stabilization

In order to improve the properties of expansive soil, a combination of chemical stabilizers such as cement, fly ash, and lime with chloride or individually can be used. About replacing soil particles to meet more stable soil structure, there are two main methods which include increasing the particle size by cementation to produce an increment in shear strength, reduction in plasticity index, and reduction in expansion potential and improving the compaction and physical properties of the soil by using absorption and chemical binding of moisture (Onyelowe and Chibuzor, 2012).

2.4.1.3 Types of Additives used in Soil Stabilization

There are many additives that have been used to improve the engineering properties of expansive soil. These additives can be classified as waste materials such as dust, agricultural wastes, synthetic wastes, and organic wastes to enhance the economic cost.

Table 2.1: Additives Employed in Soil Stabilization (Salem, 2018).

Industrial Solid Waste	**Agricultural Solid Waste**	**Domestic Solid Waste**	**Mineral Solid Waste**
Fly Ash	Rice Husk Ash	Incinerator Ash	Quarry Dust, Stone Dust or Chipping Dust
Cement Kiln Ash	Bagasse Ash	Waste Tire	Marble Dust
Silica Fume	Groundnut Shell Ash	Egg Shell Powder	Limestone Dust
Copper Slag	Plantain Peel Ash	Grain Storage Dust	Granite Dust
Granulated Blast Furnace Slag	Banana Leaf Ash	Glass Cullet	Mine Tailings
Phosphogypsum	Concob Ash		Baryte
Ceramic Dust	Guinea Corn Ash		
Brick Dust	Bamboo Ash Powder		

2.5 Bamboo Ash Powder (BAP)

Bamboo is one of the oldest building materials used by mankind (Abdulatif et al., 2014). The bamboo stem has been made into different products ranging from domestic household products to industrial applications, examples are found in food containers, skewers, chopsticks, handicrafts, toys, furniture, flooring, pulp and paper, boats, charcoal, musical instruments and weapons. In Asia, bamboo is quite common for bridges, scaffolding and housing, but it is usually a temporary exterior structural material, while in many overpopulated region of the tropics, certain bamboos supply the one suitable material that is sufficiently cheap and plentiful to meet the extensive need for economical housing (Amadi, et al., 2011). Bamboo shoots are important sources of food and delicacy in Asia and in addition to its more common applications, bamboo has other uses from skyscraper, scaffolding and phonograph needles to slide rules, skins of airplanes and diesel fuels (Farelly, 1984). Extracts from various parts of the plant have been used for skin ointment, medicine for asthma, eyewash, potions for lovers and poisons for rivals. Bamboo ashes are used to polish jewels and manufacture electrical batteries. (Ernesto et al., 2013). Bamboo has been used in bicycles, dirigibles, windmills, scales, retaining walls, ropes, cables and filament in the first light bulb. Indeed, bamboo has many applications beyond imagination. Its uses are broad and plentiful (Lee, et al., 2013). Massive plantation of bamboo provides an increasingly important source of raw material for pulp and paper industry in China (Hammett et al., 2015).

The chemical composition of bamboo is similar to that of wood (Yusoff, et al., 2016). The main constituents of bamboo are cellulose, hemicelluloses and lignin, which amount to over 90% of the total mass. The minor constituents of bamboo are resins, tannins, waxes and inorganic salts. Compared with wood, however, bamboo has higher alkaline extractives, ash and silica contents. Yusoff et al (2016) studied the chemical composition of one, two, and three year old bamboo (Gigantochloa scortechinii). The results indicated that the holocellulose content did not vary much among different ages of bamboo. Alpha-cellulose, lignin, extractives, pentosan ash and silica content increased with increasing age of bamboo. Bamboo contains other organic composition in addition to cellulose and lignin. It contains about 2-6% starch, 2% deoxidized saccharide, 2-4% fat, and 0.8-6% protein. The carbohydrate content of bamboo plays an important role in its durability and service life. Durability of bamboo against mould, fungal and borers attack is strongly associated with its chemical composition. Bamboo is known to be susceptible to fungal and insect attack. The natural durability of bamboo varies between 1 and 36 months depending on the species and climatic condition. The presence of large amounts of starch makes bamboo highly susceptible to attack by staining fungi and powder-post beetles (Mathew and Nair, 2014). The ash content of bamboo is made up of inorganic minerals, primarily silica, calcium, and potassium. Manganese and magnesium are two other common minerals. Silica content is the highest in the epidermis, with very little in the nodes. Higher ash content in some bamboo species can adversely affect the processing machinery. Bamboo fired in an open atmosphere and then heated at 600ºC for 2h in a furnace was found to be an amorphous material containing amorphous silica.

Table 2.0: Chemical Composition of Bamboo Ash Powder (Olugbenga and Adetuberu, 2010).

	SiO2	AL2O3	Fe2O3	CaO	MgO	K2O	Na2O	TiO2	SO3	LOI
BAP	**75.9**	**4.13**	**1.22**	**7.47**	**1.85**	**5.62**	**0.21**	**0.20**	**1.06**	**-**

2.5.1 Review of Past Works on Stabilization of Clayey Soils Using Bamboo Ash Powder

Various studies have been conducted in the area of soil stabilization using bamboo ash powder. Ayobami, et al., (2018) conducted a research on sustainability in road construction using bamboo ash powder to improve the index properties of clayey soils. The clayey soils was stabilized with increasing percentage of bamboo ash powder at 0%, 2%, 4%, 6%, 8%, 10%, and 12%. The index properties, Compaction and CBR, of the soil samples with bamboo ash powder were evaluated. Response Surface Analysis was used to model the mathematical relationship between the atterberg limit and the CBR of the bamboo ash powder stabilized soil sample. The plasticity index of the un-stabilized soil sample was 14.01 upon the addition of 16% bamboo ash powder, the plasticity index reduced to 10.73 which showed an improvement in the soil index properties. The CBR increased from 26.38% to 30.2% at 0% and 8% respectively which signifies an improved strength. From the Response Surface Analysis, the highest plasticity index achievable with bamboo ash powder stabilization is 27.18. The model equation showed that the plasticity index and plasticity limit have a positive relationship with the CBR.

In another study, Olugbenga and Adetuberu, (2010) investigated the stabilization of clayey soils using bamboo ash powder for highway construction. The results showed that the addition of bamboo ash powder improved the strengths of the samples, Optimum moisture contents reduced to 20.20, 19.60 and 9.32% at 8, 4 and 6% bamboo ash powder additions in the samples. Maximum Dry Unit Weight (MDUW) increased to 1400, 1676 and 1941 kg/m^3 respectively at 8%, 2% and 4% bamboo ash powder additions in the samples. The un-soaked CBR and shear strength of the samples increased. It was therefore concluded that bamboo ash powder has a good potential for stabilizing lateritic soils in highway construction.

Lorliam, et al., (2013) conducted an experimental study on effect of bamboo ash powder on cement stabilization of Markudi shale for use as flexible pavement construction material. Classification test, compaction, consistency, California bearing ratio and unconfined compressive strength tests, were conducted on Makurdi shale specimen treated with cement and bamboo leaf ash in combined incremental order of 2% up to 14% cement, and 4% up to 20% bamboo ash powder by dry weight of soil sample respectively. Results obtained suggested that Makurdi shale can be classified as an A-7-6, CH and high swell potential soil by the AASHTO, USCS and NBRRI classification systems respectively. The plasticity index (PI) decreased from 39.4% for untreated Makurdi shale to 4.7% at 14% cement + 20% bamboo ash powder. Maximum dry unit weight (MDUW) of untreated shale increased from 1.49Mg/m^3 to a peak value of 1.80Mg/m^3 at 0% BAP + 14% cement. While, the optimum moisture content (OMC) of shale increased from 14.5% to 33.1% at 14% cement content + 20% bamboo ash powder. Maximum soaked CBR and 7 day UCS values of 80% and 1783.9kN/m^2 was obtained at 14% cement content + 20% bamboo ash powder. Based on the results of the different tests, the use of Makurdi shale treated with 14% cement content + 20 % bamboo ash powder were recommended for use as sub-base materials in flexible pavement.

Amu and Babjide, (2011) conducted a study on effect of bamboo ash powder on lime stabilized clayey soils for highway construction. Three samples of clayey soils designated as A, B and C used in the study were collected from different locations in Ile-Ife, Nigeria. Preliminary tests such as the natural moisture content, specific gravity, grain size analysis and Atterberg's limits were performed on the natural samples and when stabilized with optimum percentages of lime and bamboo ash powder. Engineering tests such as compaction, California Bearing Ratio (CBR) and undrained traixial were conducted. Bamboo ash powder was added to the natural clayey soil at 2, 4 and 6% by weight of the clay sample. The strength test results suggest that bamboo ash powder improved the strengths of all the clayey modified lime samples. It was therefore deduced that bamboo ash powder will further increase the strength of clayey modified lime samples for highway construction since it was found as an effective complement for lime in soil stabilization.

Alam, et al., (2021) conducted an experimental study on improvement of bearing capacity of clayey soils using bamboo ash powder. Bamboo ash powder was employed in the study as a stabilizer for clay. Bamboo ash powder was added to clayey soils in a stepped increase of 5%, 10%, and 15%. The California Bearing Ratio (CBR) test was conducted on the natural clayey samples and clay modified bamboo charcoal powder to determine the bearing capacity of the soil. The results showed an increase in the CBR value of the clayey sample at each percentage addition of bamboo ash powder. The highest CBR value was recorded at of 15% addition of bamboo ash powder to the natural clayey samples. The increase in the CBR value of the soil suggest that there is an improvement in the bearing capacity of the clay due to the addition of bamboo ash powder

2.6 Lime

Lime is the high-temperature product obtained from calcinations of limestone. Although limestone deposits are found in every state, only a minute fraction is sufficiently pure for industrial lime production Dallas and Jon, (2001). Requirement for classification as limestone is that the rock must contain at least 50 percent calcium carbonate. Dolomite or dolomite limestone is pronounced when the rock contains 30 to 45 percent magnesium carbonate. Lime can also be manufactured from aragonite, chalk, coral, marble, and sea shells. The Standard Industry Classification (SIC) code for lime manufacturing is 3274. The six-digit Source Classification Code (SCC) for lime manufacturing is 3-05-016.

Lime is produced in several kinds of kilns by one of the following reactions:

$\mathrm{CaCO3} + \mathrm{heat}\ 6\ \mathrm{CO2} + \mathrm{CaO}$ (high calcium lime).

(2.0)

$\mathrm{CaCO3.MgCO3} + \mathrm{heat}\ 6\ \mathrm{2CO2} + \mathrm{CaO.MgO}$ (dolomitic lime).

(2.1)

Quicklime is the most commonly used lime; the followings are the advantages of quicklime over hydrated lime (Rogers et al, 1996). - Higher available free lime content per unit mass - denser than hydrated lime (less storage space is required) and less dust - generates heat which accelerate strength gain and large reduction in moisture content according to the reaction equation below

$\mathrm{CaO} + \mathrm{H_2O} \rightarrow \mathrm{Ca\,(OH)_2} + \mathrm{Heat\ (65kJ/\,mol)}$

(2.2)

Quicklime when mixed with wet soils, immediately takes up to 32% of its own weight of water from the surrounding soil to form hydrated lime; the generated heat accompanied by this reaction will further cause loss of water due to evaporation which in turn results into increased plastic limit of soil i.e. drying out and absorption (Sherwood, 1993).

In some lime plants, the resulting lime is reacted with water to form hydrated lime. The fundamental processes in the manufacture of lime are: (1) quarrying raw limestone; (2) preparing limestone for the kilns by crushing and sizing; (3) calcining limestone; (4) processing the lime further by hydrating; and (5) miscellaneous transfer, storage, and handling operations Dallas and Jon, (2001). It is worthy of note that some operations shown may not be performed in all plants. The heart of a lime plant is the kiln. The commonest type of kiln is the rotary kiln, accounting for about 90 percent of all lime production in the United States. This kiln is a long, cylindrical, slightly inclined, refractory-lined furnace, through which the limestone and hot combustion gases pass counter currently. Coal, oil, and natural gas may all be fired in rotary kilns. Product coolers and kiln feed pre-heaters of several types are frequently used to recover heat from the hot lime product and hot exhaust gases respectively.

2.6.1 Importance of Lime Stabilization

Lime stabilization enhances engineering properties of soils, such as improved strength, higher resistance to fracture, fatigue, and permanent deformation, enhanced resilient properties, reduction in swelling; and resistance to the harmful effects of moisture. The most considerable improvements in these properties are observed in moderately to highly plastic clays. Lime stabilizations technology is mostly widely used in geotechnical and environmental applications. Some of applications include encapsulation of contaminants, rendering of backfill (e.g. wet cohesive soil), highway capping, slope stabilization and foundation improvement such as in use of lime pile or lime-stabilized soil columns (Ingles and Metcalf, 1972). However, presence of sulphur and organic materials may inhibit the lime stabilization process. Sulphate (e.g. gypsum) will react with lime and swell, which may have effect on soil strength.

Al-Kiki, et al., (2013) acknowledged that over the time, the properties of treated soil affect the strength gain. Soil pH, organic content, the quantity of exchangeable sodium, clay mineralogy, natural drainage, weathering conditions, extractable iron, carbonates and silica-alumina ratio are some of the properties which influence the gain in strength. The stabilization of acidic soil using lime, resulted in lower compressive strength than that of alkaline soil. Broderick and Daniel, (2013) reported that the lime and cement stabilized soils are less vulnerable to attack by organic chemicals in comparison to untreated soils Haraguchi, et al., (2010). Haraguchi, et al., (2010) investigated the variation of the engineering properties of freshly cement-stabilized decomposed granite soil cured in water and in 0.2N acid solutions, and indicated that the CBR obtained from the specimens cured in the 0.2N acid solution was lower than that cured in water (Little, 1999). The strong alkaline conditions were able to release silica and alumina from the clay mineral and eventually react with lime to form new cementation products. The success of the lime treatment process is highly dependent on the available lime content, curing time, soil type, soil pH and clay minerals Eadesand and Grim, (1966).

2.6.2 Review of Past Works on Stabilization of Clayey Soils Using Lime

Roohbakhshan and Kalantari, (2013) investigated the stabilization of clayey soils with lime and waste stone powder. The clay samples in natural state and when mixed with varying percentages of lime and waste stone powder were used for the experimental study. Laboratory testing ranging from atterberg limits tests, grain size analysis, standard Proctor compaction tests, unconfined compression tests and California bearing ratio tests were conducted. The results show significant reduction in plasticity and changed the optimum moisture content and maximum dry density of clayey soil with increasing percentage content of waste stone powder and lime. The results of the unconfined compressive strength (UCS) and California bearing ratio (CBR) tests indicate that at the different curing times, the addition of waste stone powder and lime caused an increase in the value of UCS up to 6% waste stone powder content and 7% lime content, and increase in the value of CBR to 6% waste stone powder content and 9% lime content, thereafter, the values of UCS and CBR decreased.

Bhardwaj and Sharma, (2020) in another study investigated the effect of industrial waste and lime on strength characteristics of clayey soils. In the first part of the study, the optimum percentages of materials (molasses, lime) have been found out by conducting differential free swell (DFS) and consistency limit tests on clayey soil by adding various admixtures. The second and third part of the study investigates the compaction behaviour and unconfined compressive strength (UCS) of clayey soil on addition of optimum amount of various materials alone and in combination with each other. Finally, the micro-structural behaviour of addition of optimum percentages of lime, waste foundry sands (WFS) and molasses using Scanning electron microscopic technique were ascertained. The laboratory results revealed that the addition of optimum content of lime along with WFS and molasses reduced differential free swell and plasticity index and increased maximum dry density and UCS values of the natural clay samples. The microstructural behaviour showed that the presence of lime and molasses filled the voids present in the soil and the addition of WFS helped in providing compact structure, thus improving the strength characteristics of the natural clay samples.

A study conducted by Amadi, et al., (2011) on stabilization of expansive soil derived from Enugu shale using lime, Portland cement and coal fly ash. Engineering performance of treated soils were evaluated using Atterberg limits, California bearing ratio and Compaction test. Results obtained revealed that there was a remarkable improvement in dry unit weight and strength of the clayey soil.

2.7 Summary of Research Findings

Researchers	Percentages of Bamboo Ash Powder Used	Percentages of Lime Used	Effect on Compaction Properties of Clayey Samples	Effect on Strength of Clayey Samples
Bhardwaj and Sharma, (2020)		4%, 8%, and 12% by weight of clay sample	Improved the maximum dry unit weight of the clay sample	Increased the unconfined compressive strength of the clay sample
Roohbakhshan and Kalantari, (2013)	-	3%, 5%, 7% and 9% by weight of clayey samples	The maximum dry unit weight of the clay samples increased at a reduced optimum moisture content	Increased the California Bearing Ratio and Unconfined Compressive Strength of the natural Clay Sample
Ayobami, et al., (2018)	2%, 4%, 6%, 8%, 10% and 12% by weight of clayey soils	-	Increased the maximum dry unit weight of the clay samples at a reduced optimum moisture content	Increased the California Bearing Ratio and Unconfined Compressive Strength of the natural Clay Sample

Amu and Babjide, (2011)	2%, 4% and 6% by weight of clayey samples		Increased the maximum dry unit weight of the clay samples at a reduced optimum moisture content	Increased the California Bearing Ratio of the natural clay samples

The current will builds on gaps identified and extensively study the effect of bamboo ash powder and lime on strength properties of clayey soils. Clayey soils used for the experimental study will be partially admixed with bamboo ash powder in a stepped increase of 2.5, 5, 7.5, 10 and 12.5% while the lime will be added to clayey sample in a stepped increase of 2%, 4%, 6%, 8% and 10% by dry weight of the clayey samples. A blend of lime and bamboo ash powder will be added to the natural clayey soils at similar percentages specified from the individual combinations.

CHAPTER THREE

MATERIALS AND METHODS

This section presents the materials and methods used to accomplish the research goal. Relevant standards were employed to ascertain how the materials collected be analyzed and also the various laboratory tests to be conducted. All Tests such as sieve analysis test, specific gravity, atterberg limit test (liquid and plastic limit), compaction and California bearing ratio test were carried out at Nnamdi Azikiwe University Civil Engineering Laboratory located inside the school campus.

3.1 Collection and Preparation of Materials

3.1.1 Clayey Samples

Natural clayey samples used for the experimental study designated as NC was collected at Bakasi close to a Catholic Church situated in Nnamdi Azikiwe University Campus. The clay sample collected was disturbed samples. This soil sample was collected in two empty cement bags, marked indicating the sampling depth, soil description, sampling date and conveyed to geotechnical laboratory of Department of Civil Engineering Nnamdi Azikiwe University. After conveyance, the natural moisture content of the clay sample was determined and was thereafter air-dried in a corrugated roofing sheet for one week to allow partial elimination of natural water which may affect analysis. After drying, the lumps in the samples were slightly pulverized with minimal pressure in order not to damage the individual particles, the samples were passed through sieve No 4 (4.75mm) and the materials passing through the sieve were stored in cement bags in a safe location preparatory for laboratory testing.

3.1.2 Lime

Lime used in the experimental study was designated as LM was purchased from a chemical vendor at Onitsha Market and conveyed to geotechnical laboratory of Department of Civil Engineering Nnamdi Azikiwe University. Upon arrival, confirmatory test were conducted on the lime sample, the lime was thereafter stored in a safe location preparatory for various laboratory testing.

3.1.3 Bamboo Ash Powder

Bamboo ash powder designated as BAP was obtained locally from a bamboo plantation located at Enugu-Agidi in Anambra State. The bamboo leaf was collected in large quantities with the aid of a cutlass and a large bowl. This bamboo leaf was thereafter conveyed to Syndey in Enugu State where it was air-dried and burnt at a controlled temperature. Chemical composition of the ash was also determined. The ash was collected in sacks and conveyed to the soil mechanics research laboratory in Nnamdi Azikiwe University. The ash was then passed through the BS No 200 sieve (75µm) to meet the requirements of ASTM class N pozzolans (ASTM D4318-10e1) as reported by Head in (1994).

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3.1.4 Mix Proportion

Natural clayey soils obtained at Bakasi located in Nnamdi Azikiwe Unversity Awka Anambra State, Nigeria was stabilized by adding lime by weight at intervals of 2%, 4%, 6% , 8% and 10% by dry weight of clay while bamboo ash powder was added at intervals of 2.5%, 5%, 7.5%, 10% and 12.5% by dry weight of clay sample. The clayey soil was additionally stabilized with a blend of lime and bamboo ash powder at varying percentages used earlier for individual combinations cumulating to a total of five (5) mix combinations. The choice on selection of percentages for lime and bamboo ash powder was justified by the relative cost of the materials. The specimen (natural clay samples stabilized with lime and bamboo ash powder) will be subjected to compaction and California bearing ratio test. Tables 3.1-3.3 present the percentage replacement of natural clay samples using lime and bamboo ash powder.

Table 3.1 Mix proportion for Clay and Lime

Mix No	Mix Proportion
1	Control (100% Clay + 0% Lime)
2	98% Clay + 2% Lime
3	96% Clay + 4% Lime
4	94% Clay + 6% Lime
5	92% Clay + 8% Lime
6	90% Clay + 10% Lime

Table 3.2 Mix proportion for Clay and Bamboo Ash Powder

Mix No	Mix Proportion
1	Control (100% Clay + 0% Bamboo Ash Powder)
2	97.5% Clay + 2.5% Bamboo Ash Powder
3	95% Clay + 5% Bamboo Ash Powder
4	92.5% Clay + 7.5% Bamboo Ash Powder
5	90% Clay + 10% Bamboo Ash Powder
6	87.5% Clay + 12.5% Bamboo Ash Powder

Table 3.3: Mix Proportion for Natural Clay, Lime and Bamboo Ash Powder

Mix No	Mix Proportion
1	Control (100% Clay + 0% Lime + 0% Bamboo Ash Powder
2	2% Lime + 2.5% Bamboo Ash Powder
3	4% Lime + 5% Bamboo Ash Powder
4	6% Lime + 7.5% Bamboo Ash Powder
8	8% Lime + 10% Bamboo Ash Powder
10	10% Lime + 12.5% Bamboo Ash powder

3.2 Laboratory Investigations

This section presents the experimental procedure and laboratory tests that were adopted for the project work. The tests was conducted for the natural clayey samples and clayey samples stabilized with bamboo ash powder and lime. The test includes: sieve analysis test, specific gravity, atterberg limit (liquid and plastic limit), compaction and California bearing ratio test. The aforementioned tests were carried out at Nnamdi Azikiwe University Civil Engineering Laboratory located inside the school campus. Below is a description of test procedures and apparatus:

3.3.1 Particle Size Distribution (Sieve Analysis)

Sieve analysis is a procedure used to assess the particle size distribution of a granular material (sand, gravel). The size distribution is often of critical importance to the behaviour of the material during use. Sieve analysis can performed on any type of non-organic or organic granular material including sand, crushed rock, clay, granite, feldspar and a wide range of manufactured powders, grains and seed down to minimum size depending on the exact method. The standard grain size analysis test determines the relative proportion of different grain sizes as they are distributed among certain size ranges.

The apparatus needed for this experiment is listed below:

1. Stack of sieves including pan and cover.
2. Mechanical sieve shaker.
3. Weighing balance of 0.01g sensitivity.
4. Hand brush
5. Mortar and pestle (Used for crushing if the sample is conglomerated or lumped)
6. Thermostatically controlled Oven (With temperature of about $80^{O}C$-$110^{O}C$).
7. Masking tape for identification of sample.
8. Exercise book and pen for recording of result.

The calculation for attaining Coefficient of uniformity and Coefficient of curvature are outlined below.

Percentage retained (%) $=\frac{mass\ of\ soil\ retained\ in\ the\ sieve(g)}{total\ mass\ of\ soil\ sample(g)} \times 100$

Cumulative percentage retained = $\sum Percentage\ retained$ (%)

Cumulative Percentage Finer (%) = 100-Cummulative percentage retained.

Coefficient of Curvature $=\frac{D60}{D10}$

Coefficient of Uniformity =$\frac{(D30)2}{D10\times D60}$

Where

D10= particle size such that 10% of the soil is finer than the size

D30= particle size such that 30% of the soil is finer than the size.

D60= particle size such that 60% of the soil is finer than the size**.**

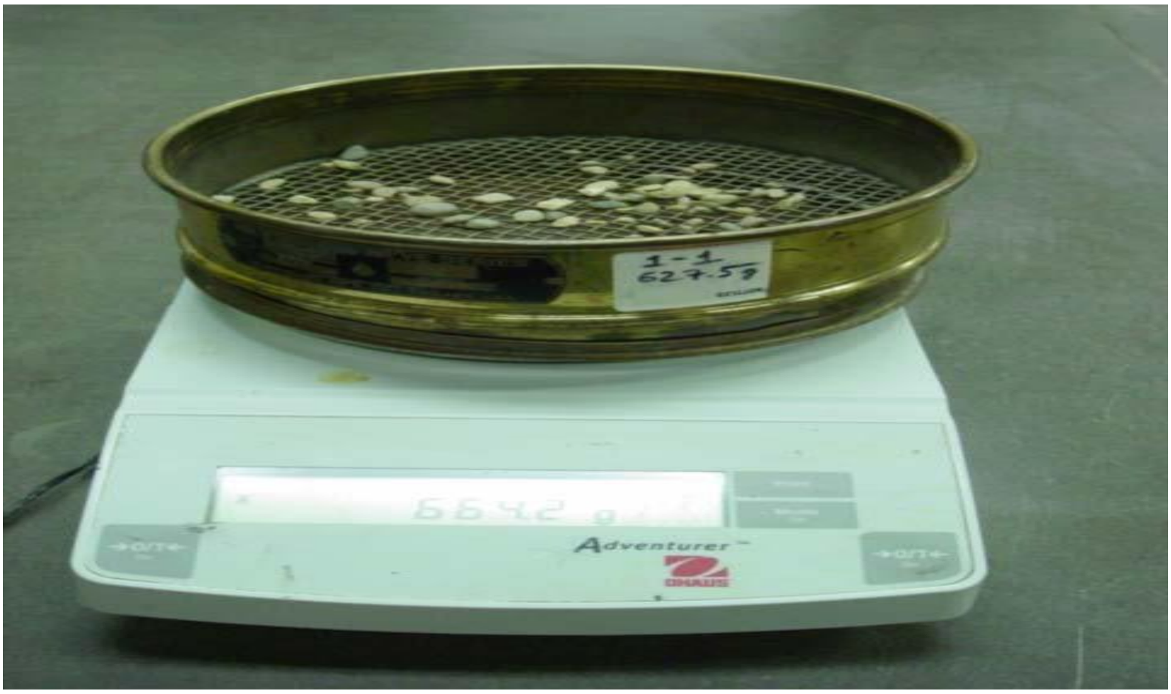

Plate 3.3 Apparatus for Particle Size Distribution Test (Sieve Analysis).

Plate 3.4 Apparatus for Particle Size Distribution Test (Sieve Analysis).

Test Procedure

1. Clean properly the stack of sieves to be used for the experiment using hand brush.
2. Weigh about 500g of air-dried soil sample on a weighing balance.
3. Pour the weighed soil sample into 75um sieve and wash under a steady supply of water until clear water start coming out from the sieve after passing through the soil sample.
4. After washing pour the washed soil sample into a pre-weighed plate and dry it inside the thermostatically controlled oven at a controlled temperature of 80-110^{O}C for 16-24hrs.
5. Remove the sample from the oven and determine it weight (net weight) by deducting the weight of plate from the weight of plate and soil.
6. Arrange the stacks of sieve in the ascending order, place in a mechanical sieve shaker, and thereafter pour the sample and connect the shaker for about 10-15 minute.
7. Disconnect the sieve shaker and determine the mass retained on each of the sieve sizes.
8. Determine the percentage retained, Cumulative percentage retained and Cumulative percentage finer.
9. Plot the graph of sieve Cumulative percentage finer against sieve sizes.
10. Determine D10, D30 and D60 from the plotted graph.

11. Determine the Coefficient of Curvature and Coefficient of Uniformity and classify the soil using the American Association of State Highway and Transportation Official (AASHTO) and Unified Soil Classification System (USCS) respectively.

3.2.2 Specific Gravity Test

Specific gravity is the ratio of mass of unit volume of soil at a stated temperature to mass of equal volume of gas-free distilled water at the same temperature (Krishna, 2002). Also as defined by (Braja, 2006), Specific gravity can be defined as the ratio of unit weight of a material to unit weight of water. The specific gravity of soil solids is often needed for various calculations in soil mechanics. It can be determined accurately in the soil laboratory.

The apparatus employed for this experiment includes:

1. Density bottle of 50ml capacity and a stopper.
2. Desiccator containing anhydrous silica gel.
3. Thermostatically controlled oven with temperature of about 80-110^{O}C.
4. Weighing balance of 0.01g sensitivity.
5. Mantle heater.
6. Plastic wash bottle.
7. Distilled water.
8. Funnel
9. Thin glass rod for stirring.
10. 425um Sieve.
11. Dry piece of cloth for cleaning.
12. Masking tape for identification of sample.
13. Exercise book and pen for recording of result.

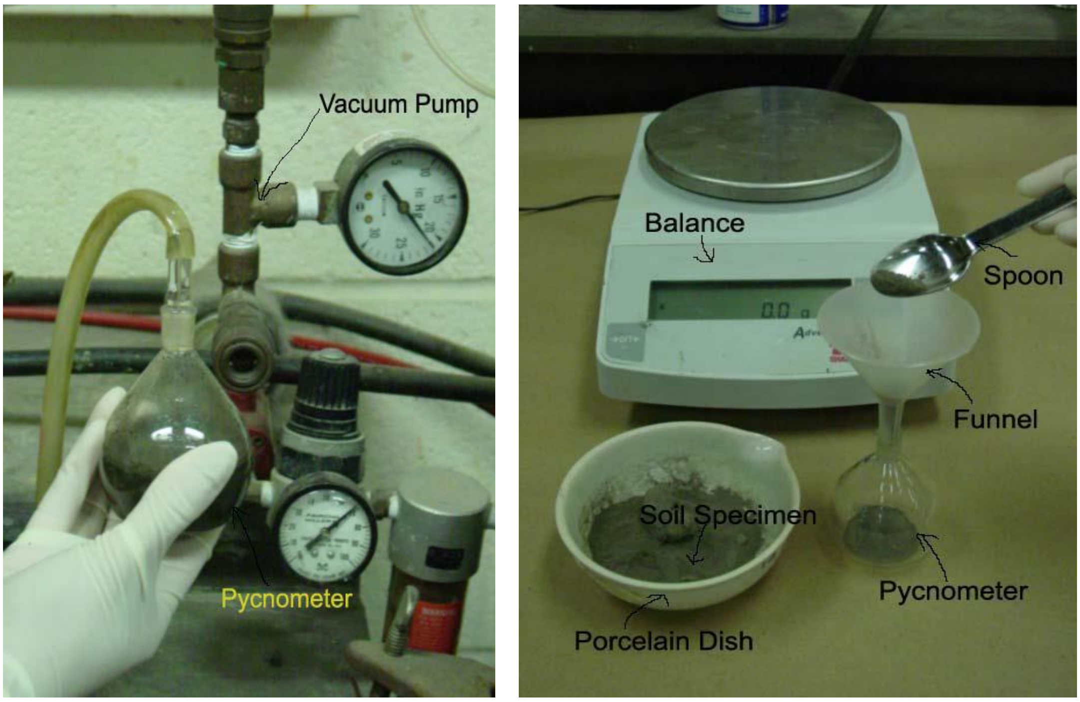

Plate 3.5 Apparatus used for Specific Gravity Test.

Test Procedure

1. Firstly clean the density bottle properly and rinse it with distilled water, oven- dry the clean density bottle with stopper, then cool it in a desiccator so as to remove any moisture present.
2. Weigh and record the weight of the empty clean and dry density bottle say (M_1)
3. Place 10-15g of soil passing through 425um sieve inside the density bottle, weigh and record the weight of density bottle +dry soil + stopper say (M_2).
4. Add distilled water to fill about half to three-fourth of the density bottle, soak the sample for 24hrs (The time stated is to enable complete settlement of the soil particle which is evident when clear water appears above the submerged soil).
5. Gently stir the density bottle using thin glass rod and thereafter connect to a mantle heater to de-air the sample, do not allow the sample to boil over.
6. After agitation, allow to cool at room temperature and fill it with distilled water up to the specified mark (at lower meniscus level), clean the exterior surface of the density bottle with a clean dry cloth and determine the weight of the density bottle + stopper +soil filled with water say (M_3).
7. Empty the density bottle clean and rinse with distilled water, then fill it with distilled water up to the same mark. Clean the exterior surface of the density bottle with a clean dry cloth and determine the weight of the density bottle filled with distilled water + stopper say (M_4).

8. Repeat the procedure for two more trials and take the average specific gravity value obtained from the total no of trial, the variation in the specific gravity result obtained for each trial must not exceed 2%, otherwise repeat the experiment.

The Procedure for Computation of result obtained is as follows:

Specific gravity (G_S) = $\frac{(M2-M1)}{(M2-M1)-(M3-M4)}$

Where M_1= weight of density bottle + stopper

M_2= Weight of density bottle + air-dried soil + stopper.

M_3= Weight of density bottle filled with water + wet soil + stopper.

M_4= Weight of density bottle filled with water + stopper

3.2.3 Atterberg Limit Test.

The atterberg limit is a limit characterized by visible transition of soil (especially fine grained soils) from liquid-plastic-semi-solid-solid state consequent upon the variation of moisture content. This test was developed by Albert Atterberg a Swedish agricultural scientist in 1911. This test is divided into three limits namely:

1. Liquid Limit (LL)
2. Plastic Limit (PL)
3. Shrinkage Limit

3.2.3.1 Liquid Limit Test

It is the water content at which the soil has a small shear strength that it flows to close a groove of standard width when jarred in a specified manner. It is the minimum water content at which the soil tends to flow like a liquid. When a soil is mixed with an excessive amount of water, it will be in a liquid state and flow like a viscous liquid. When the viscous liquid dries gradually due to loss of moisture it will pass into a plastic state. With further loss of moisture, the soil will pass into a semi-solid state. With even further reduction of moisture, the soil will pass into a solid state. The moisture content (%) at which a cohesive soil will pass from liquid state to plastic state is referred to as the liquid limit of the soil.

In order to study the liquid limit of the soil Casagrande test was conducted. Liquid limit is generally determined by the mechanical method using Casagrande apparatus or the standard liquid limit test apparatus. With respect to this method, the liquid limit is defined as the moisture content at which 25 blows or drop in standard liquid limit apparatus will just close a groove of standardized dimension cut into sample by a grooving tool at a specified amount (Aroja, et al 2017).

The apparatus used for liquid limit determination is outlined below:

1. Liquid limit device (Cassagrande type)
2. Grooving tool
3. Moisture content tins
4. Porcelain evaporating dish
5. Spatula or pellet knife
6. Thermostatically controlled oven
7. Weighing balance sensitive to 0.01g
8. Plastic wash bottle containing distilled water
9. Paper towels
10. Masking tape for identification of tin.
11. Exercise book and pen for recording of data
12. 425um Sieve
13. Airtight container

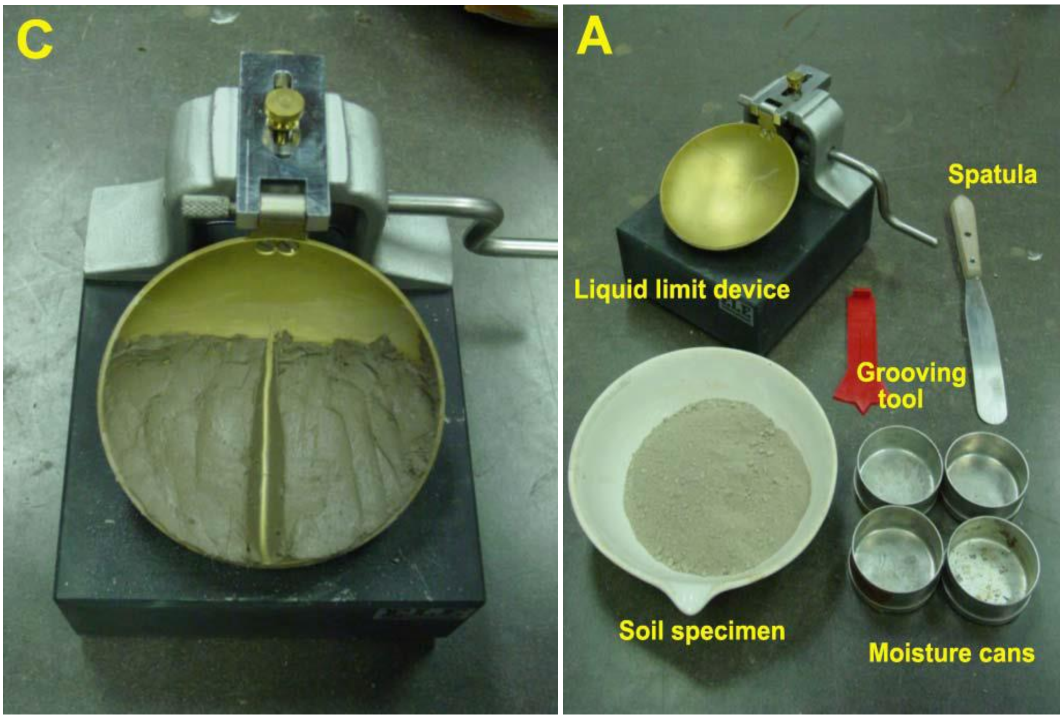

Plate 3.6 Apparatus for Atterberg Limit Test.

Test Procedure

1. Prepare the sample by weighing about 150g of soil passing through 425um sieve, mix the sample with distilled water in a glass plate mixing with pellet knife, remove any coarse particle by hand and mix to form a thick homogenous paste, place the mixed soil in an airtight container and leave to mature for 24hrs.
2. Determine the mass of four moisture content tins say (W_1)
3. Place the matured sample on an evaporating dish and add little water using the plastic squeeze bottle, mix the soil properly to ensure uniform distribution of moisture.
4. Place a portion of the paste (mixed soil) on the liquid limit device and level the mixture so as to obtain a maximum depth of 1cm.
5. Using the grooving tool, cut a groove along the symmetrical axis of the cup holding the tool perpendicular to the cup.
6. Turn the crank or rotate the handle of the liquid limit device at the rate of 2 revolution per second and count the no of blows required to close the groove at a distance of 13mm. Closing of the groove should be as a result of plastic flow of the soil and not by sliding, if sliding occurs repeat the test.

7. Take about 10g of soil in the closed groove and put in the moisture content tins for moisture content determination, weigh the sample say (W_2)
8. Remove the rest of the soil in the cup and use paper towel to clean the cassagrande cup.
9. Alter the water content of the soil and the repeat the process to get the no of blows in the range of 15-40 blows.
10. Plot the graph of moisture content against the log of no of blows, the moisture content corresponding to 25 blows on the abscissa gives the value of the liquid limit.

The Procedure employed for the Computation of the Result obtained is as Follows:

Moisture content = $\frac{Weight\ of\ water}{weight\ of\ dry\ soil}\times 100 = \frac{W2-W3}{W3-W1} \times 100$

Where W_1 = Weight of empty tin.

W_2 = Weight of tin + wet soil.

W_3 = Weight of tin + oven-dried

3.3.3.2 Plastic Limit Test

The plastic limit of a soil is the moisture content expressed as a percentage of the weight of oven-dried soil at the boundary between the plastic and the semi-solid state of consistency. It is the moisture content at which a soil will just begin to crumble when rolled into a uniform 3mm diameter thread using a glass plate or other recommended surface for rolling. Soil used for Atterberg limit test can be classified based on the plasticity index of the soil. The plasticity index is the amount of water required to change a soil from its plastic limit to liquid limit, in other word it is the numerical difference between the liquid limit and the plastic limit of soil. Table 3.2 is used to classify soil based on the ranges of it plasticity index.

Table: 3.1 Plasticity Ratings for Fine grained Soil (Braja, M.Das, 2002).

Plasticity Index	Plasticity
0	**Non-Plasticity**
<7	**Low Plasticity**
7-17	**Medium Plasticity**
17-35	**High Plasticity**

>35	**Very High Plasticity**

1. The apparatus used for this experiment includes:
2. A smooth glass plate about 300mm square and 10mm thick.
3. A palette knife or spatula
4. A short length of 3mm metal rod
5. Moisture content tins
6. Plastic squeeze bottle
7. Weighing balance with 0.01g sensitivity
8. Veneer caliper
9. Masking tape for tin identification
10. Exercise book and pen for recording of result.

Test Procedure

1. Prepare the sample by the method described in the liquid limit using the sample passing 425um sieve.
2. Identify and weigh the empty moisture content tins say (W1).
3. Take about 20g of the prepared soil paste on a porcelain evaporating dish, add water from the plastic squeeze bottle and mix thoroughly until the paste is plastic enough to be rolled into a ball.
4. Take a portion of the ball and roll it on a glass plate with the palm of the hand into a thread of uniform diameter throughout its length by rolling forward and backward.
5. Continue rolling and remolding until the thread just start to crack at a distance of 3mm.
6. Collect the small crumbed pieces, place in a moisture content tin and weigh say (W2).
7. Place the tin in the oven at a constant temperature of 80-110^{O}C for a period of 16-24hrs.
8. After 24hrs, remove the tin from the oven and determine the weight of the dry soil plus the tin say (W_3).
9. Repeat the test for at least two trials and take the average plastic limit value for all the trials.

The Computation for Plastic Limit is as follows:

Plastic limit = $\frac{Weight\ of\ water}{Weight\ of\ oven\text{-}dried\ soil} \times 100 = \frac{W2-W3}{W3-W1} \times 100$

Where W_1 = Weight of empty tins.

W_2 = Weight of tin plus wet soil

W3 = Weight of tin plus oven-dried soil

3.2.4 Compaction Test

Compaction is the process of increasing the bulk density of the soil by driving out air. It involves the densification of soils by mechanical means thereby increasing the dry density of the soil. According to (Shruthi, 2017) Compaction of soil is the process by which the soil solid are packed more closely together by mechanical means, thus increasing it dry density. It could also be stated as the process of packing the soil particle more closely together usually by tamping, rolling or other mechanical means, thus increasing the dry density of the soil. It is achieved through the reduction of the volume of air void in the soil with little or no reduction in water content. The process must not be confused with consolidation in which water is squeezed out under the action of steady static load. Consolidation is a natural process and result in dense packing of the soil. In his test we did 27blows.

The apparatus used for the test are as follows:

1. Compaction mould with a detachable base plate and removable extension collar.
2. Metal rammer (either 2.5kg or 4.5kg)
3. Measuring Cylinder 200ml or 500ml
4. Large Metal tray (600mm×600mm ×600mm)
5. Balance up to 10kg readable to 1g
6. Small tools such as palette knife, steel straight edge about 300mm long.
7. Drying oven temperature of 105-110^{O}C
8. Apparatus for moisture content determination

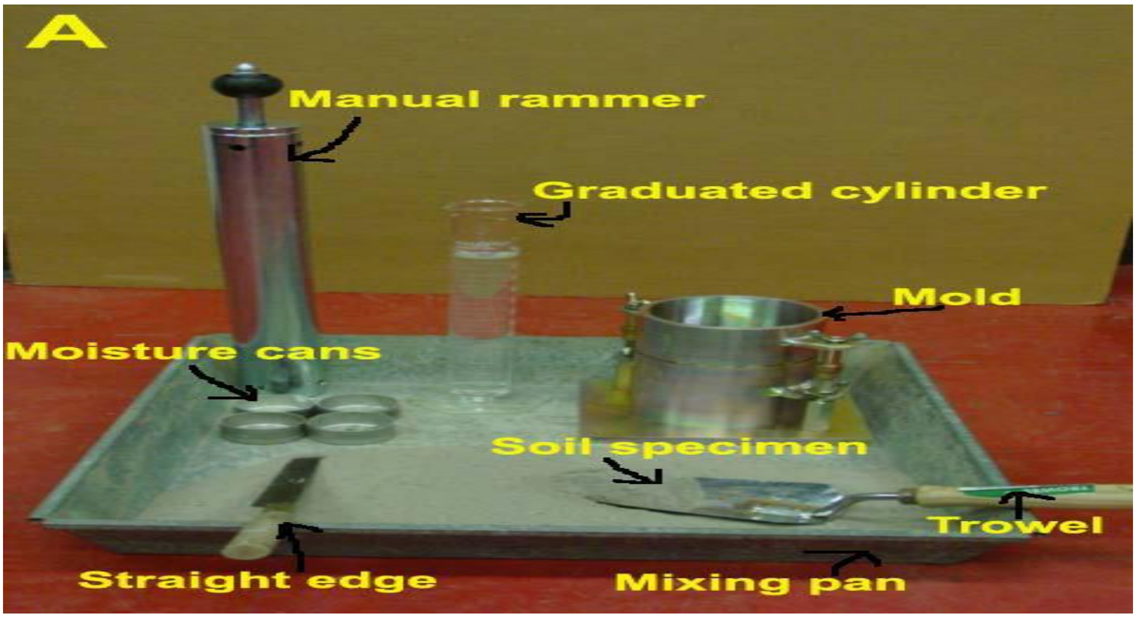

Plate 3.7 Apparatus employed for Compaction Test.

Test Procedure.

1. Check to see if the mould, extension collar and base plate are clean and dry. Measure the dimension and weigh to the nearest 1kg check if the rammer falls freely.
2. Grease the internal surface of the mould.
3. Attach the extension collar to the mould.
4. Weigh about 3kg of the soil sample on a weighing balance
5. Add about 4% water to the soil sample, mixing it thoroughly and separating the soil into three layers for British Standard Light and five layers for British Standard Heavy.
6. Pour the wet soil into the mould and compact by applying the required no of blow using either a 2.5kg or 4.5kg rammer falling freely from a height of 300mm. The blow must be distributed uniformly over the surface of the mould.
7. After completion of the compaction operation remove the extension collar and level carefully the top of the mould by means of a straight edge.
8. Weigh the mould with the compacted soil to the nearest 1kg, record the weight as W_2.
9. Determine the moisture content of the representative sample of the specimen; record the moisture content as M.

10. Repeat the procedure for 8%, 12%, 16% and 20% of water to be added and record the value obtained.
11. Plot the graph of dry density against moisture content and determine the maximum dry density (MDD) of the soil at the corresponding optimum moisture content (OMC).

The Computation of the result obtained is as follows:

Determination of Dry Density (P_d).

Wt of mould (kg) = W_1

Wt of mould + wet soil (kg) = W_2

Wt of wet soil (kg) = W_2-W_1

Volume of mould (M^3) = W_4

Bulk Density (kg/m^3) = $\frac{\text{Wt of wet soil (kg)}}{\text{Vol of mould (m3)}} = \frac{W2-W1}{W4}$

Moisture Content (%) = $\frac{\text{mosture content (top)+ moisture content (bottom)}}{2}$

Dry Density (kg/m^3) = $\frac{\text{Bulk density}}{\text{1+moisture content (\%)}} = \frac{Pb}{1+w/100}$

Determination of Moisture Content (w) for top and bottom respectively.

Wt of tin (kg) = W_1

Wt of tin + wet soil = W_2

Wet of wet soil (kg) = W_3 = W_2-W_1

Wt of tin + dry soil (kg) = W_4

Wt of dry soil (kg) = W_5= W_4-W_1

Wt of water (kg) =W_6 = W_3-W_5

Moisture Content (%) = $\frac{\text{Wt of water}}{\text{Wt of dry soil}} \times 100 = \frac{W6}{W5} \times 100$

3.2.5 California Bearing Ratio Test

The California bearing ratio test was originally developed by the California division of highway in 1938, for the design of highway thickness. The test is used for evaluating the suitability of materials used in sub-grade, sub-base and base course respectively. The test result has been correlated with the thickness of various materials required for flexible pavement construction. The test may be conducted on a prepared specimen in a mould or on the soil in-situ condition.

In the test the load required to push a plunger into a soil specimen at a controlled rate is measured, then the load on the plunger at a certain depth is recorded as a percentage of a standardized load. The load necessary to push a plunger to a certain depth into the soil is expressed as a percentage of the load required to force the same plunger to the same depth into a standard sample of compacted crush stone. The construction of highway pavement requires a California Bearing Ratio value for 2.5mm and 5mm penetration respectively, with that of 2.5mm penetration being comparatively higher than that of 5mm penetration. The Federal Ministry of work Standard Specification for roads and bridges (1997) state that road construction material should have a CBR value of 10%, 20% and 80% for use as sub-grade, sub-base and base course respectively. The material to be used for the test will be subjected to 48 hours soaking in other to ascertain it behavior under worst condition (flooding as a result of intense rainfall).

Table 3.1 Standard load adopted for different penetration on a standard material with CBR value of 100%.

Penetration of plunger (mm)	Standard Load (kg)
2	1150
2.5	1320
4	1760
5	2000
6	2220
7.5	2630
8	2650
10	3180
12.5	3600

The apparatus used for the test are outlined below:

1. A cylindrical corrosion resistant mould 152mm×127mm having a diameter of 150-152mm with a detachable base plate and a removable extension collar.
2. A compressive device for static compaction of applying a force of at least 300KN

3. Metal plugs 150mm $\pm$ 0,5mm and 50mm thick.
4. Metal rammer 2.5kg or 4.5kg.
5. Dial gauge of 0.01g sensitivity.
6. Soaking tank.
7. A steel rod of about 16mm diameter and 600mm long and a straight edge of 300mm steel stripe and 3mm thick with one beveled edge.
8. Weighing balance of 25kg accuracy and a spatula.
9. Filter paper
10. Apparatus for moisture content determination.
11. Masking tape used for identification of moisture content tins.
12. Exercise book and pen for recording.

Plate 3.8 California Bearing Ratio (CBR) Test Machine.

Test Procedure

The methods used for California Bearing Ratio Test are:

1. Compression with tamping.
2. Recompaction with known maximum dry unit weight (MDUW) and optimum moisture content (OMC).
3. For this course of study the method for recompacted sample with known maximum dry unit weight (MDUW) and optimum moisture content (OMC) is to be adopted and the procedure is outlined below:
4. Carry out Compaction test using 6kg of soil sample, varying the moisture content at a particular percentage say 4%, determine the maximum dry density and optimum moisture content.
5. Clean properly and grease the internal surface of the CBR mould.
6. Weigh 6kg of soil mixing with the optimum moisture content determined from compaction test.
7. Divide the soil into 5 equal layer (CBR Heavy) and seal in an airtight container until requested for use.
8. Stand the mould assembly in a solid base, place the first soil portion and compact using 4.5kg rammer for 62 even blows.
9. Repeat using the remaining four portion of soil in turn so that the level of the soil is not more than 6mm above the top of the mould body.
10. Remove the collar and trim the soil flush with mould with the scrapper or knife edge.
11. Weigh the mould, soil and base plate to the nearest kg.

Preparation for Soaking

Soil may soften when load is placed on it due to flooding or increase in moisture content. Soaking of the sample is done primarily to determine the strength (load bearing strength) of the soil under worst condition (rainy season).Below are the list of apparatus used for CBR Soaking:

1. Perforated base plate fitted to CBR mould in place of normal base plate.
2. Perforated swell plate with an adjustable stem to provide a sealing for the stem of the dial gauge.
3. Tripod mounting to support dial gauge
4. Soaking tank

5. Annular Surcharge discs with internal diameter of 52-54mm and external diameter of 145mm to 150mm.
6. Petroleum jelly.
7. The Soaking procedures are enumerated as follows:
8. Remove the base plate and replace with perforated base plate.
9. Fit the collar to the other end of the mould, pack the screw thread with petroleum jelly to make it water tight.
10. Place the mould assembly in soaking, place the filter paper in the sample, the perforated swell plate, and then annular surcharge disc.
11. Mount dial gauge on top of the extension collar, secure the dial gauge in place and adjust the stem in the perforated base plate to give zero.
12. Fill the immersion tank with water just below the extension collar. Start the timer when water has just covered the base plate.
13. Record the time taken for water to appear at the top of the sample if it does occur within two days. Flood the top of the sample and leave to soak for a day.
14. Plot the swelling against elapsed time or square root of time. Flattening curve indicates that swelling is complete.
15. Take off the dial gauge and its support; remove the mould assembly and leave to drain for 15min.
16. Remove the Surcharge discs, perforated plate and collar, then fit the other base plate.
17. Weigh the sample + mould + base plate if density is required after soaking is completed.
18. If the sample has swollen, trim it to the level of the mould and reweigh
19. Test the sample by adjusting the dial gauge to start at zero and take the reading at interval of 0.5mm for every 30seconds till 7mm penetration.
20. Record the load at penetration of 0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5 and 7.0mm and express this force as percentage of the standard load.
21. Calculate the CBR for 2.5 and 5mm penetration; repeat the same procedure for top and bottom, the higher CBR value will be used as the CBR for the material.
22. Plot the graph of force (KN) against penetration (mm).
23. The normal curve is convex upward, but if the initial part is concave upward applies the necessary correction to the curve.

Mathematically it is expressed as $\frac{\textbf{test load}}{\textbf{standard load}} \times \mathbf{100}$

Where

Test load = dial gauge reading × proof ring constant

$\textbf{CBR}_{\textbf{2.5mm}} = \frac{\textbf{test load}}{\textbf{13.2}} \textbf{ x } \mathbf{100}$ $\textbf{CBR}_{\textbf{5mm}} = \frac{\textbf{test load}}{\textbf{20}} \textbf{ x } \mathbf{100}$

CHAPTER FOUR

RESULTS AND DISCUSSION

During the course of the experimentation phase of the study, certain results were obtained which was valuable in evaluating the effect of lime and bamboo ash powder on strength properties of natural clayey natural clayey soils. These findings are presented in the Table below.

4.1 Results

Table 4.0: Index Properties of Natural Clay Stabilized with Lime

Percentage Replacement/ Properties	NC + 0%LM	NC + 2%LM	NC + 4%LM	NC + 6%LM	NC + 8%LM	NC + 10%LM
Percentage Passing Sieve No 200 (0.075mm)	43.7	–	–	–	–	–
AASHTO Soil Classification System	A-7-6	–	–	–	–	–
Unified Soil Classification System	CH	–	—	–	–	–
Specific Gravity	2.66	2.7	2.72	2.75	2.76	2.79
Liquid Limit (%)	41.2	42.8	38.4	28.2	-	-
Plastic Limit (%)	12.82	15.83	17.57	20.46	-	-
Plasticity Index (%)	28.38	25.27	20.83	7.74	-	-

Table 4.1: Index Properties of Natural Clay Stabilized with Bamboo Ash Powder

Percentage Replacement/ Properties	NC + 0%BAP	NC + 2.5%BAP	NC + 5%BAP	NC + 7.5%BAP	NC + 10%BAP	NC + 12.5%BAP
Percentage Passing Sieve No 200 (0.075mm)	43.7	–	–	–	–	–
AASHTO Soil Classification System	A-7-6	–	–	–	–	–
Unified Soil Classification System	CH	–	—	–	–	–
Specific Gravity	2.66	2.66	2.62	2.58	2.55	2.49
Liquid Limit (%)	41.2	38.8	37.2	-	-	-
Plastic Limit (%)	12.82	15.12	13.05	-	-	-
Plasticity Index (%)	28.38	23.68	24.15	-	-	-

Table 4.2: Index Properties of Natural Clay Stabilized with Lime and Bamboo Ash Powder

Percentage Replacement/ Properties	NC +0%LM 0%BAP	NC +2%LM+ 2.5%BAP	NC +4%LM+ 5%BAP	NC +6%LM+ 7.5%BAP	NC +8%LM + 10%BAP	NC + 10%LM 12.5%BAP
Percentage Passing Sieve No 200 (0.075mm)	43.7	–	–	–	–	–
AASHTO Soil Classification System	A-7-6	–	–	–	–	–
Unified Soil Classification System	CH	–	—	–	–	–
Specific Gravity	2.66	2.71	2.72	2.62	2.62	2.58
Liquid Limit (%)	41.2	35.8	30.4	29.2	-	-
Plastic Limit (%)	12.82	18.81	16.97	14.84	-	-
Plasticity Index (%)	28.38	16.99	13.43	14.36	-	-

Table 4.3: Compaction Characteristics of Natural Clay Stabilized with Lime

Percentages Replacement/Properties	NC + 0%LM	NC + 2%LM	NC + 4%LM	NC + 6%LM	NC + 8%LM	NC + 10%LM
Maximum Dry Unit Weight (kN/m^3)	21.7	21.2	20.3	21.7	22.7	20.7

Optimum Moisture Content (%)	**14.6**	**15.5**	**17.8**	**10.1**	**17.4**	**14.4**

Table 4.4: Compaction Characteristics of Natural Clay Stabilized with Bamboo Ash Powder

Percentages Replacement/Properties	**NC + 0%BAP**	**NC + 2.5%BAP**	**NC + 5%BAP**	**NC + 7.5%BAP**	**NC + 10%BAP**	**NC + 12.5%BAP**
Maximum Dry Unit Weight (kN/m^3)	**21.7**	**20.28**	**21.23**	**22.2**	**22.7**	**21.8**
Optimum Moisture Content (%)	**14.6**	**14.6**	**14.2**	**13.9**	**13.8**	**18.7**

Table 4.5: CBR and Compaction Characteristics of Natural Clay Stabilized with Lime and Bamboo Ash Powder

Percentages Replacement/Properties	**NC + 0%LM+ 0%BAP**	**NC + 2%LM + 2.5%BAP**	**NC + 4%LM+ 5%BAP**	**NC + 6%LM + 7.5%BAP**	**NC + 8%LM + 10%BAP**	**NC + 10%LM + 12.5%BAP**
Maximum Dry Unit Weight (kN/m^3)	**21.7**	**20.2**	**21.3**	**21.7**	**22.2**	**21.7**
Optimum Moisture Content (%)	**14.6**	**17.78**	**17.08**	**12.94**	**16.9**	**15.1**
Soaked CBR Values (%)	**10.61**	**13.60**	**15.20**	**10.60**	**9.80**	**8.30**
Un-soaked CBR Values (%)	**14.4**	**18.9**	**19.70**	**20.5**	**23.5**	**15.9**

4.2 Discussion on Research Findings

4.2.1 Sieve Analysis Test

Figure 4.0 is a semi-logarithmic plot of the particle size distribution of the natural clayey soils. The percentage passing sieve No 200 (0.075mm) was 43.7 and as a result, the soil sample was classified as CH (clay of high plasticity) according to Unified Soil Classification System and A-7-6 according to AASHTO Soil Classification System. The shape parameter (D10) cannot be determined from the particle size distribution chart and as a result, the gradation of the clayey samples could not be ascertained.

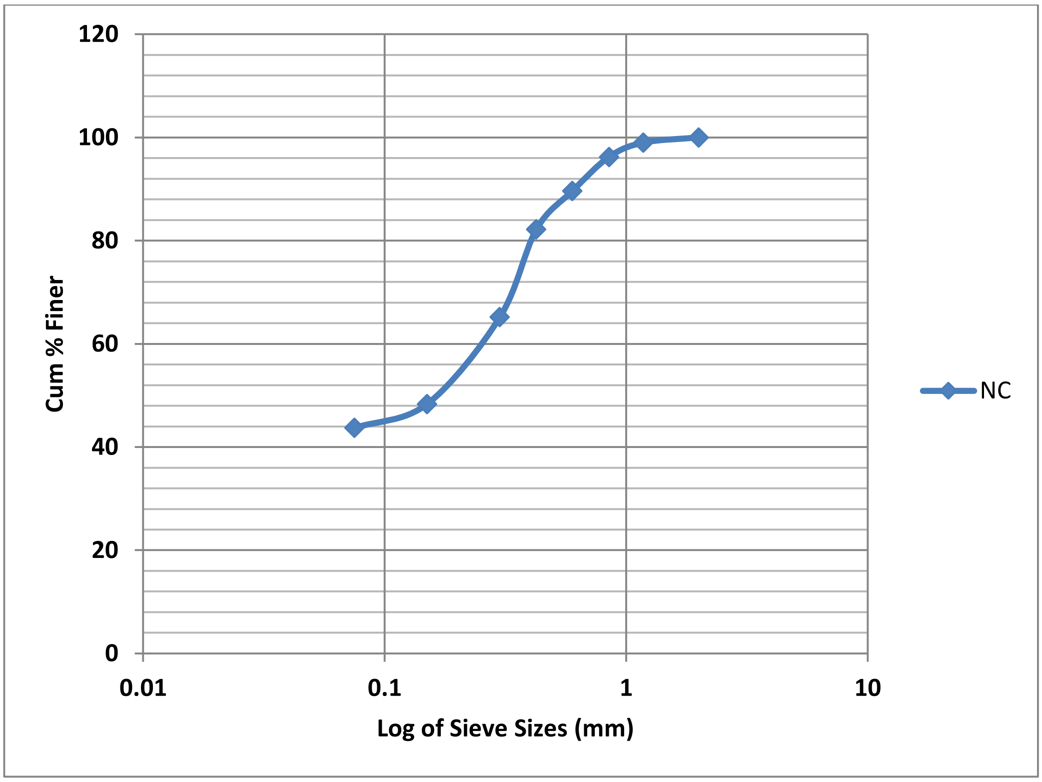

Figure 4.0: Particle Size Distribution Curve for the Natural Clayey Sample

4.2 Specific Gravity

Figure 4.1-4.3 shows the specific gravity results for natural clayey soils stabilized with lime, bamboo ash powder and a blend of lime and bamboo ash powder. Results obtained from a mixture of clay and lime indicates that addition of lime to clay increased the specific gravity of clayey soils from its natural value of 2.66 to 2.79. The consistent increase in the specific gravity values of clay on addition of lime could be attributed to the high specific gravity value of lime (2.71). Findings obtained from a blend of natural clay with bamboo ash powder shows that bamboo ash powder decreased the specific gravity of the natural clayey soils from its natural value of 2.66 to 2.49 beyond 2.5% addition of bamboo ash powder to clay. At 2.5% addition of bamboo ash powder to clay, the specific gravity of the mixture remained constant with that of the natural clayey sample. The decline in specific gravity of the natural clay could be attributed to the low specific gravity value of bamboo ash powder (2.48). Results obtained from the composite mixture of clay, lime and bamboo ash powder revealed that the blend of lime and bamboo ash powder increased the specific gravity of natural clay from 2.66 to 2.72 at 4%lime and 5% bamboo ash powder, beyond this point, the specific gravity of the natural clay decreased. The latter decline in specific gravity could be attributed to the high content of bamboo ash powder which depressed the specific gravity of the mixture.

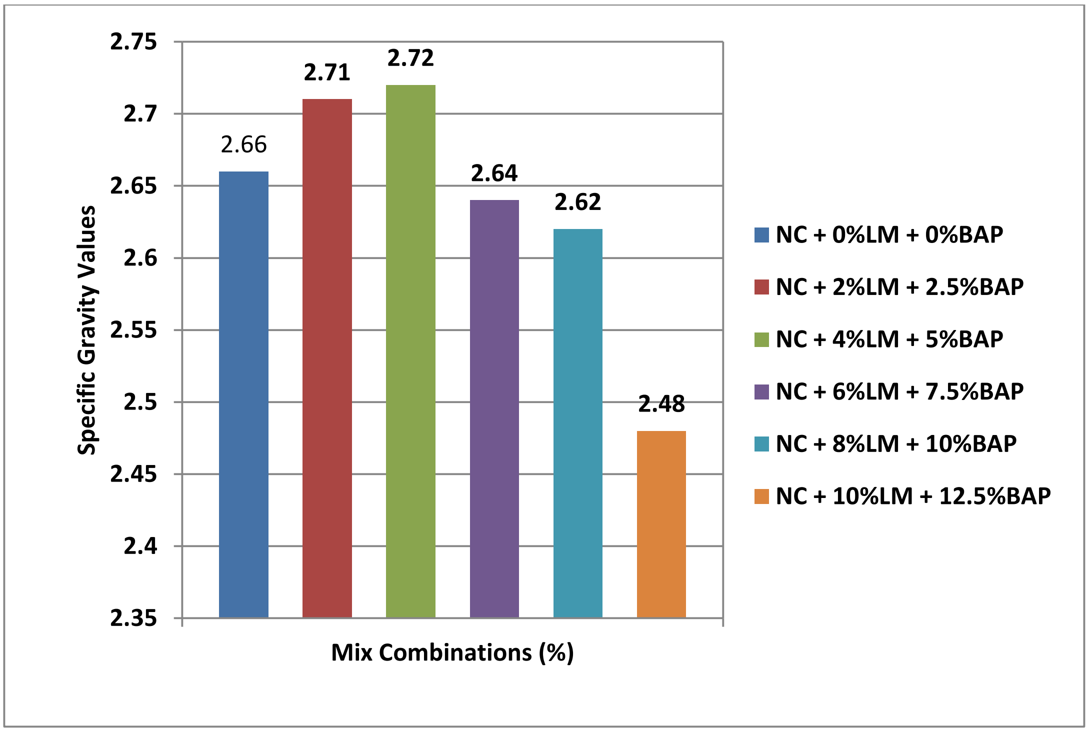

Figure 4.1: Specific Gravity of Natural Clay Stabilized with Lime and Bamboo Ash Powder

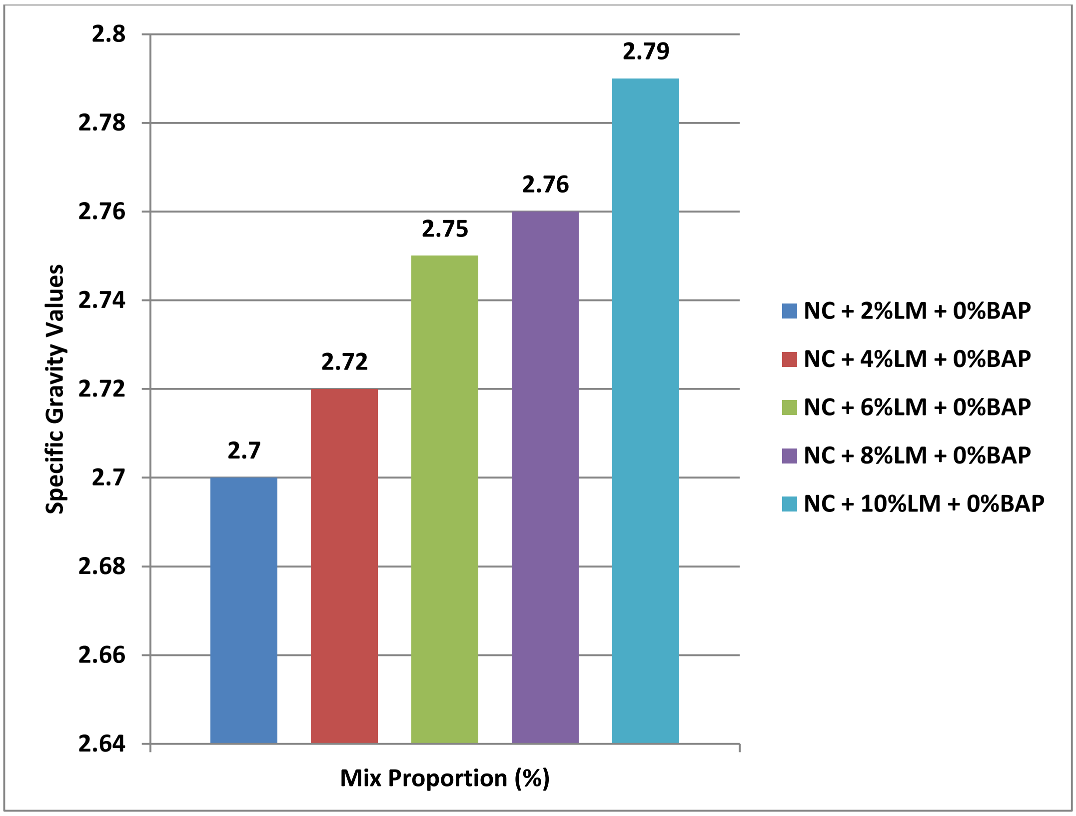

Figure 4.2: Specific Gravity of Natural Clay Stabilized with Lime

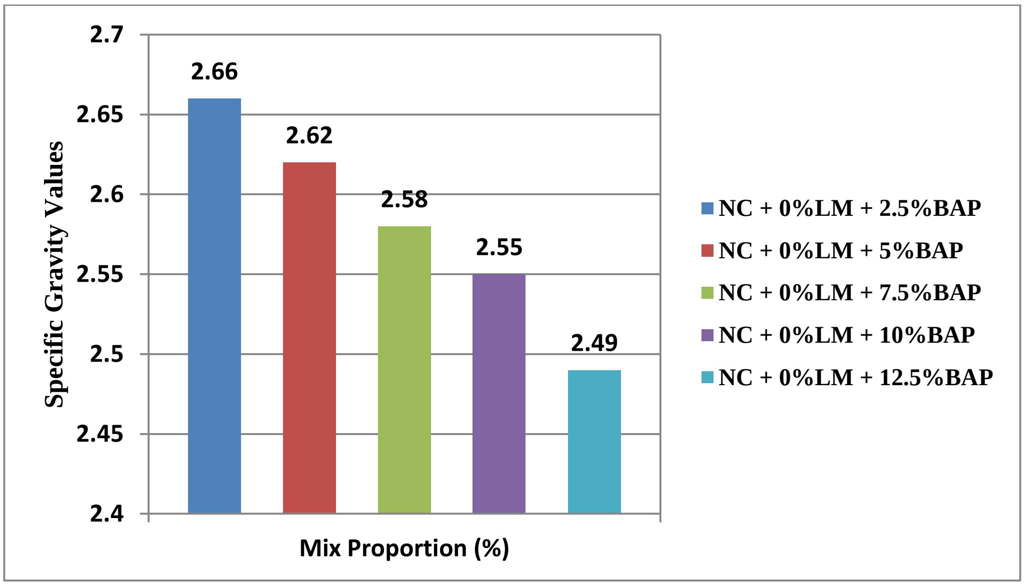

Figure 4.3: Specific Gravity of Natural Clay Stabilized with Bamboo Ash Powder

4.3 Atterberg Limit

Atterberg limit shows the state of transition of the soil sample from liquid to semi-solid and solid state. The liquid, plastic and plasticity index is an index of the amount of fines present within a soil samples. Results obtained from a mixture of clay with bamboo ash powder revealed that bamboo ash powder depressed the liquid, plastic and plasticity index of clay with plasticity index increasing marginally. It was observed that beyond 5% addition of bamboo ash powder to clay, the mixture became non plastic. Results obtained from a blend of clay with lime contradict that obtained for natural clay stabilized with bamboo ash powder as the liquid limit; plastic limit and plasticity index were found to increase. Result obtained for natural clay stabilized with a blend of lime and bamboo ash powder shows that a blend of lime and bamboo ash powder decreased the liquid limit, plastic limit and plasticity index of natural clayey soils. Although, a slight deviation in plastic limit and plasticity index were observed this could be attributable to lapses in the experimentation process. It was also observed that beyond 6% lime and 7.5% bamboo ash powder, the mixture became non plastic.

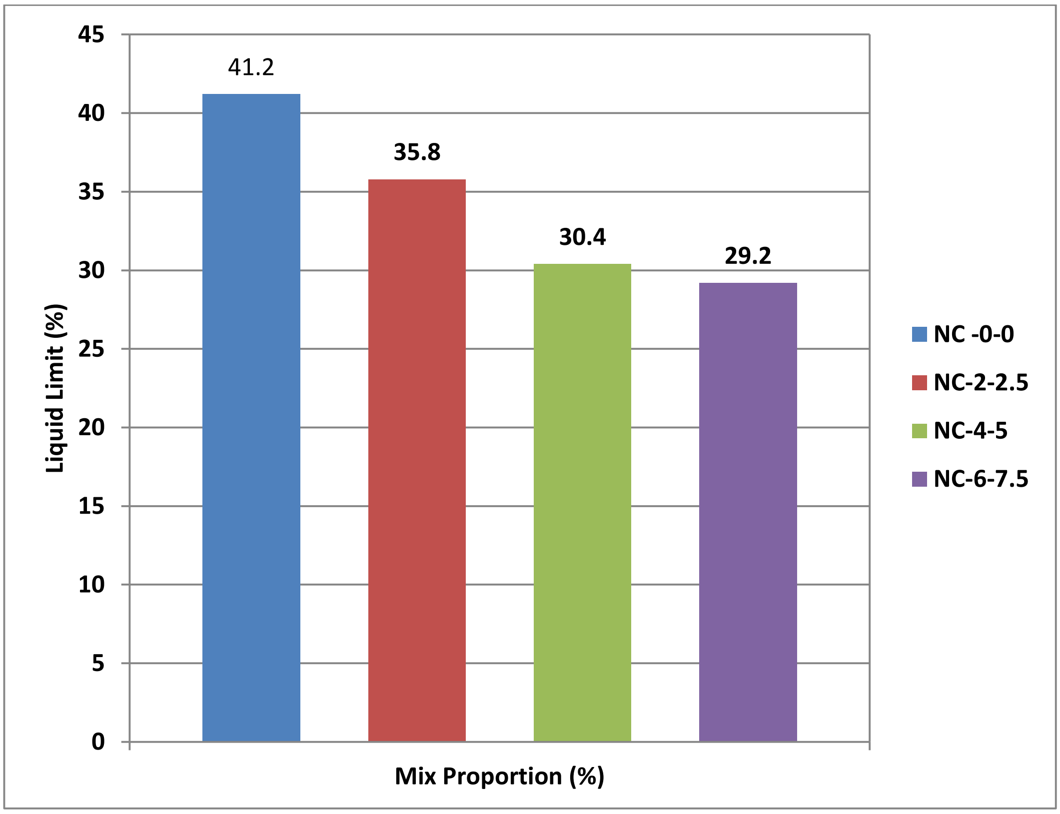

Figure 4.4: Liquid Limit of Natural Clay Stabilized with Lime and Bamboo Ash Powder

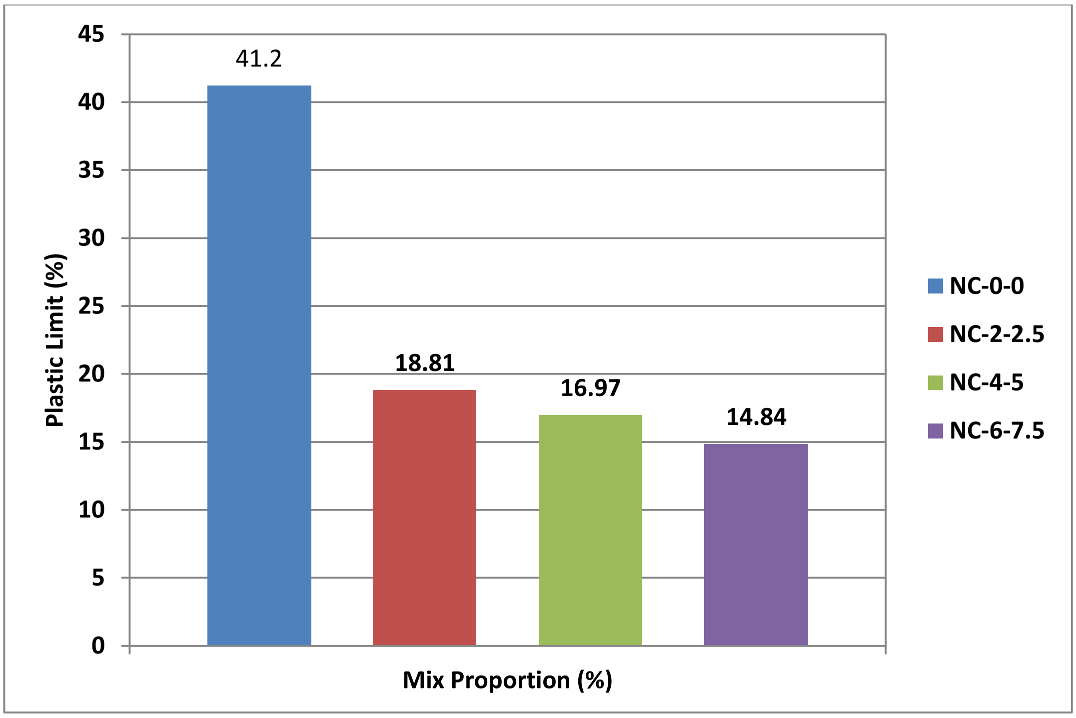

Figure 4.5: Plastic Limit of Natural Clay Stabilized with Lime and Bamboo Ash Powder

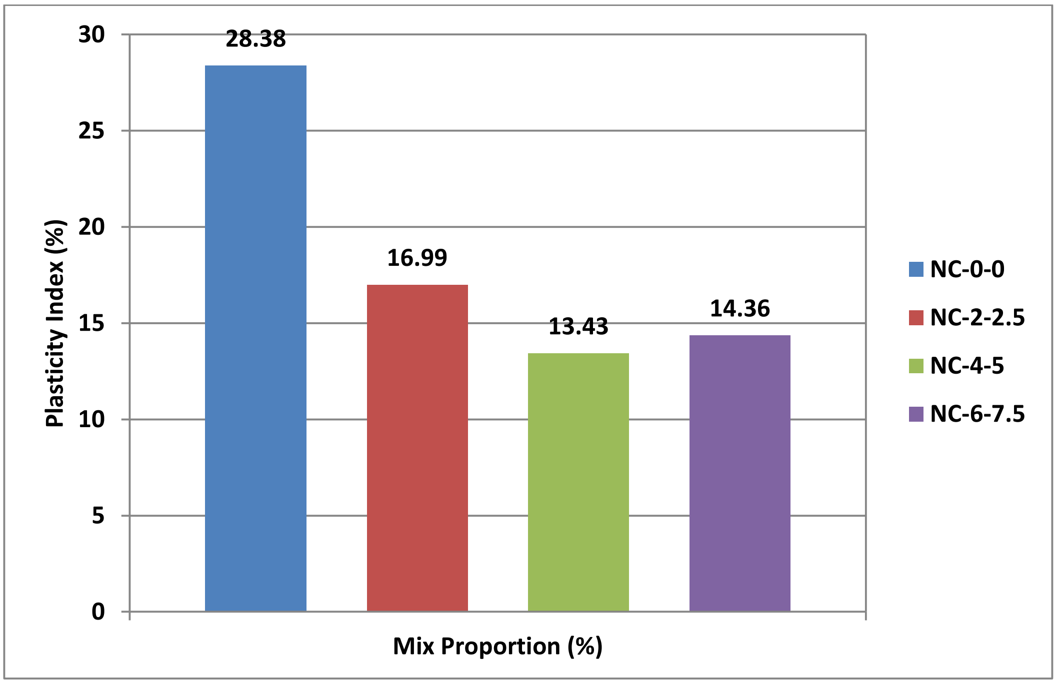

Figure 4.6: Plasticity Index of Natural Clay Stabilized with Lime and Bamboo Ash Powder

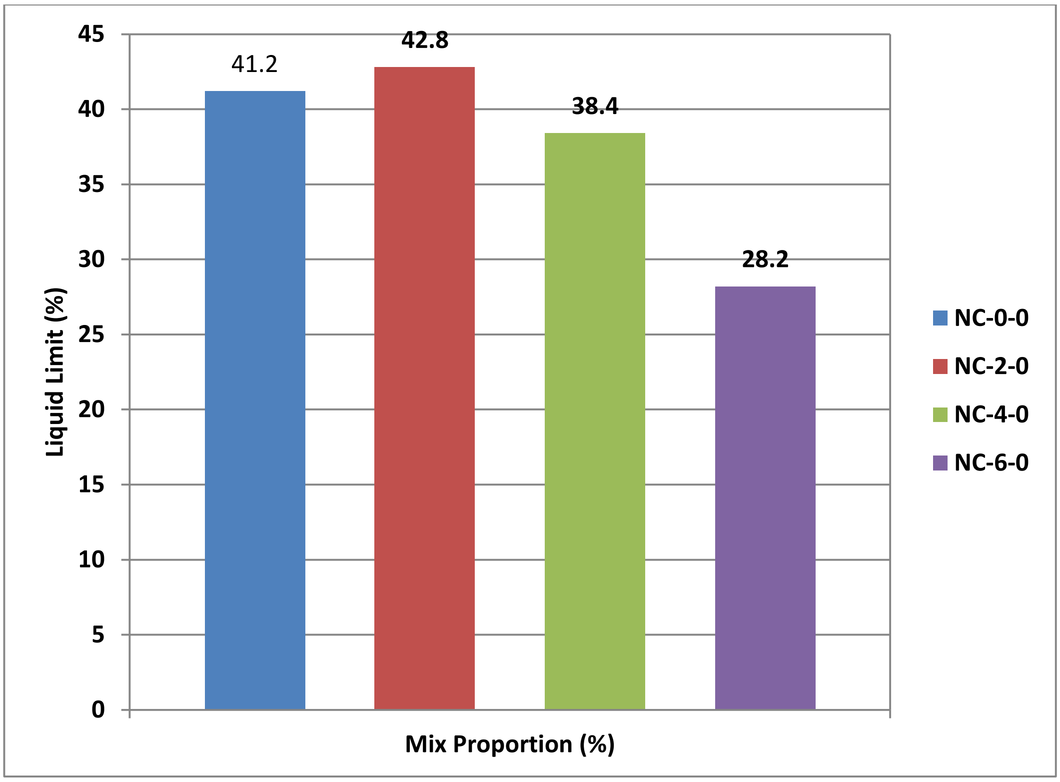

Figure 4.7: Liquid Limit of Natural Clay Stabilized with Lime

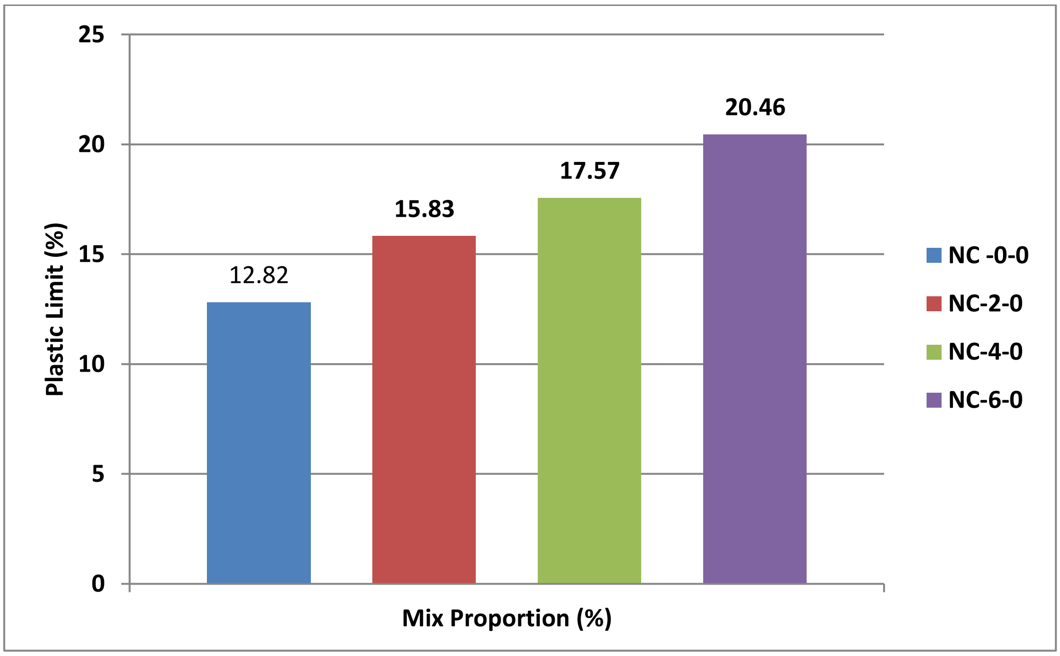

Figure 4.8: Plastic Limit of Natural Clay Stabilized with Lime

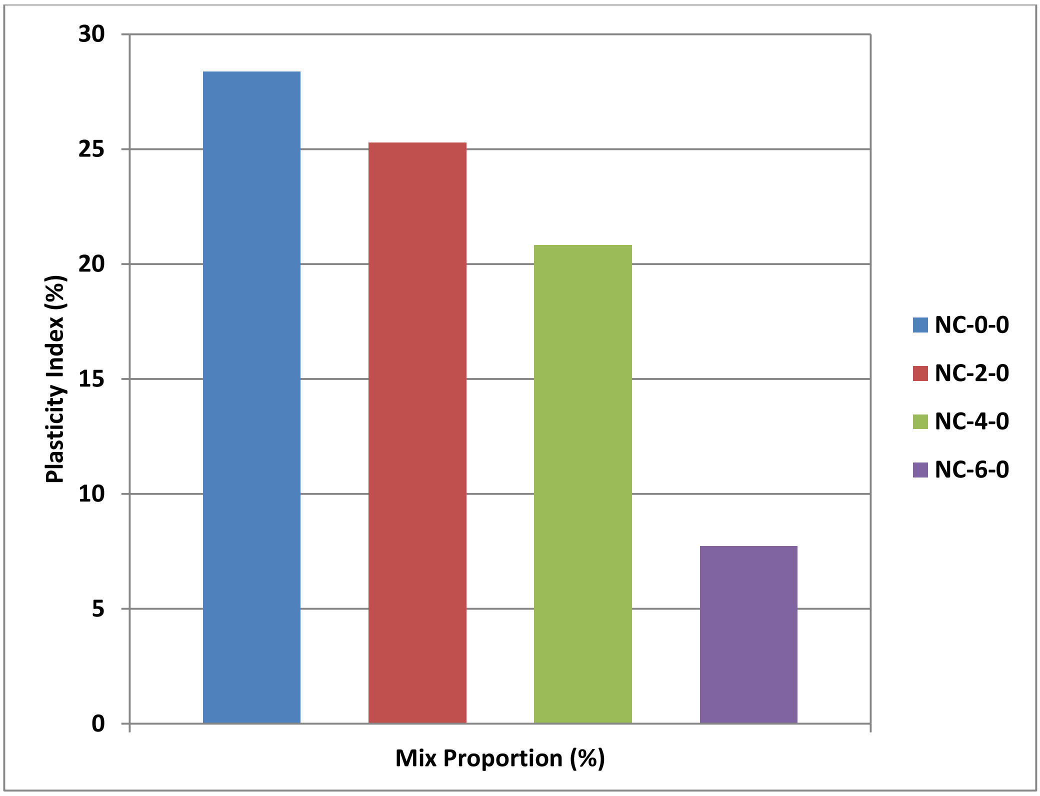

Figure 4.9: Plasticity Index of Natural Clay Stabilized with Lime

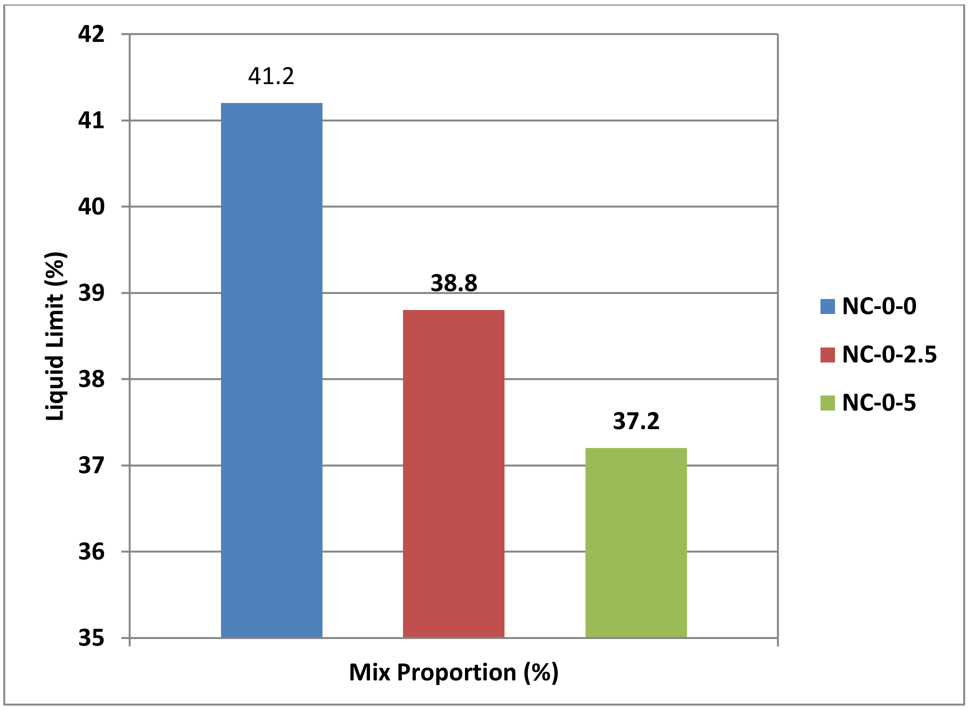

Figure 4.10: Liquid Limit of Natural Clay Stabilized with Bamboo Ash Powder

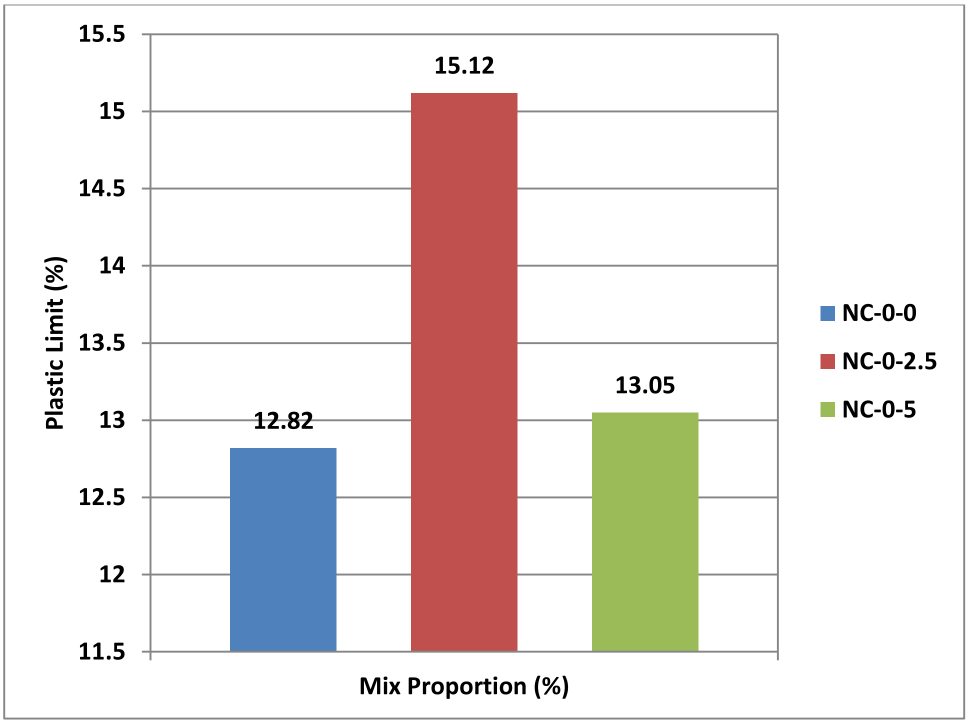

Figure 4.11: Plastic Limit of Natural Clay Stabilized with Bamboo Ash Powder

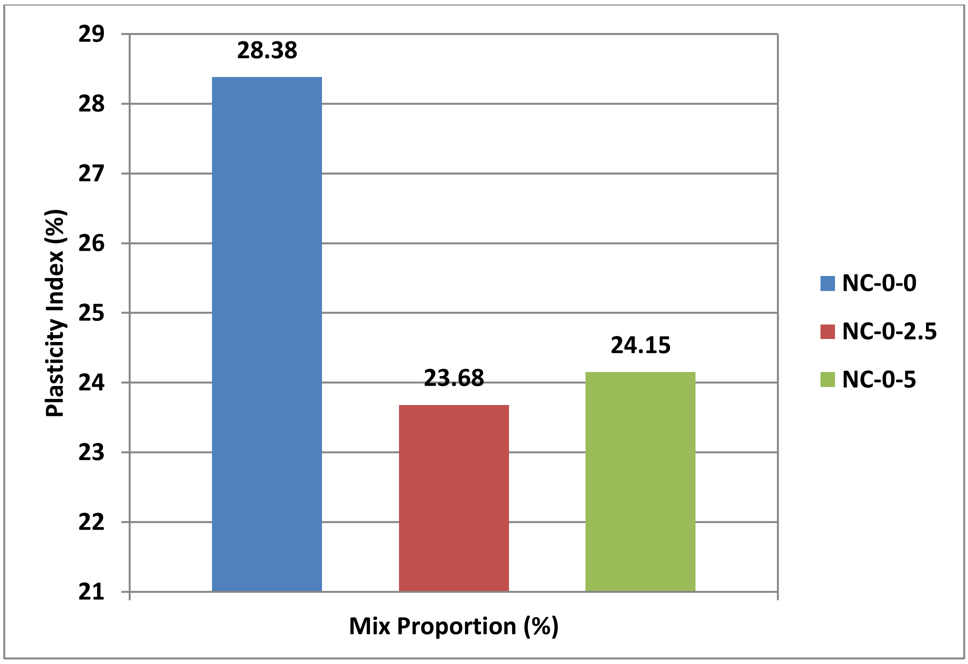

Figure 4.12: Plasticity Index of Natural Clay Stabilized with Bamboo Ash Powder

4.4 Compaction Test

Figure 4.13-4.18 depicts the relationship between the maximum dry unit weight and optimum moisture content of natural clayey soils stabilized with lime and bamboo ash powder. Results obtained from a mixture of clay and lime revealed that the maximum dry unit weight of clay decreased on consistent addition of lime to clay from 21.7kN/m^3 to 20.37kN/m^3 at 4% lime content, beyond 4% lime content, the maximum dry unit weight of the mixture increased. Although a fall in maximum dry unit weight was observed at 10% lime content. The optimum moisture content of the mixture was found to increase. Results obtained from a mixture of clay with bamboo ash powder revealed that bamboo ash powder enhanced the maximum dry unit weight of clay from 2.5% addition of bamboo ash powder to 10% addition of bamboo ash powder to clay, beyond 10% bamboo ash powder, a fall in maximum dry unit weight was observed. Similar results were obtained for optimum moisture content. Results obtained from natural clayey samples stabilized with a blend of lime and bamboo ash powder shows that a blend of lime and bamboo ash powder enhanced the maximum dry unit weight of clay from its natural value of 20.37kN/m^3 to 22.27kN/m^3 at 8% lime and 10% bamboo ash powder. The results obtained therefore, suggest that a blend of lime and bamboo ash powder is more effective in improving the compaction characteristics of natural clayey soils than when lime and bamboo ash powder is used as separate entities.

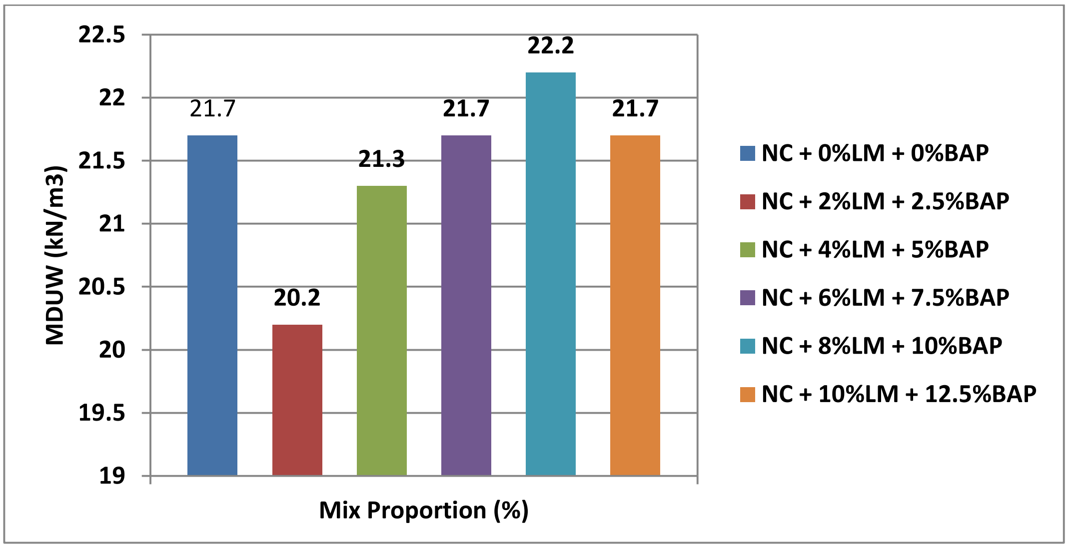

Figure 4.13: Maximum Dry Unit Weight of Natural Clay Stabilized with a Blend of Lime and Bamboo Ash Powder

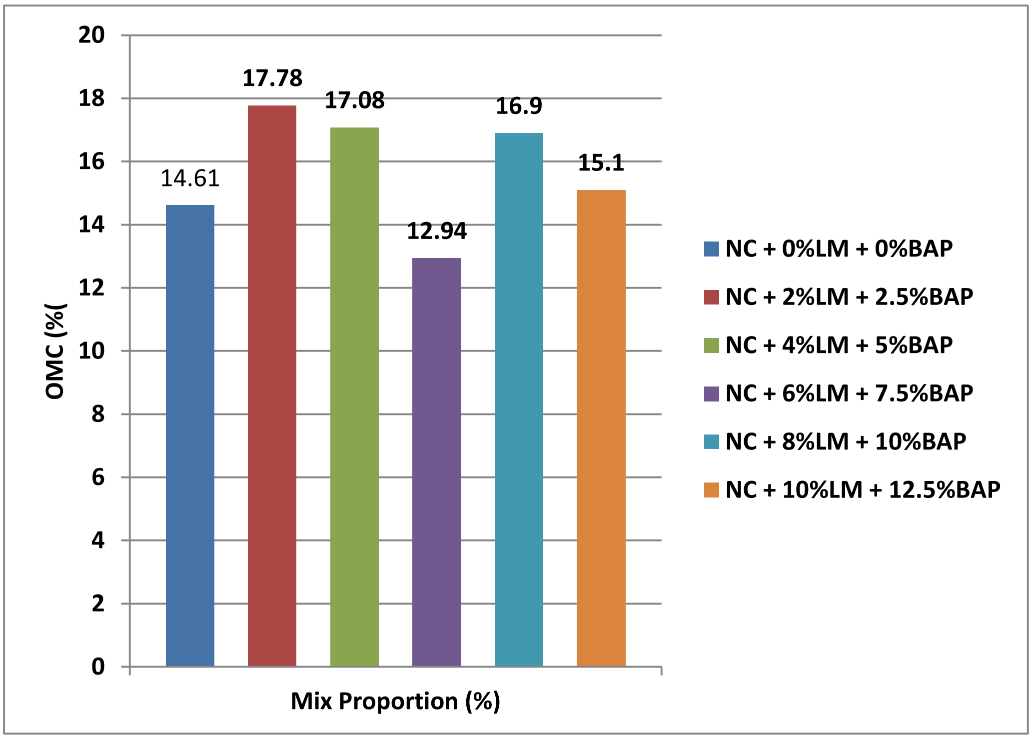

Figure 4.14: Optimum Moisture Content of Natural Clay Stabilized with a Blend of Lime and Bamboo Ash Powder

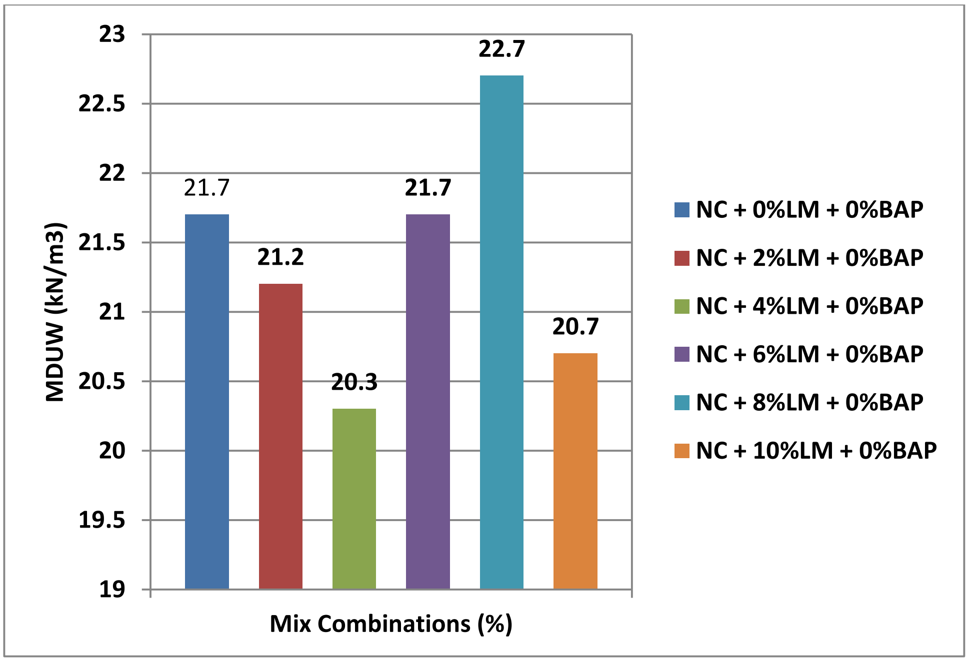

Figure 4.15: Maximum Dry Unit Weight of Natural Clay Stabilized with Lime

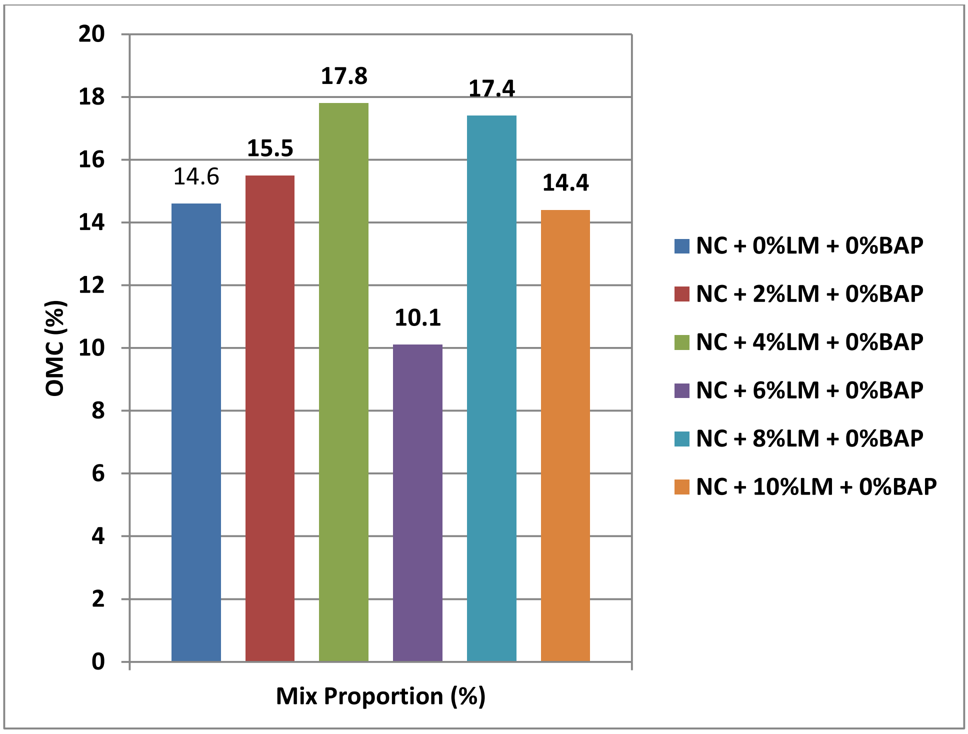

Figure 4.16: Optimum Moisture Content of Natural Clay Stabilized with Lime

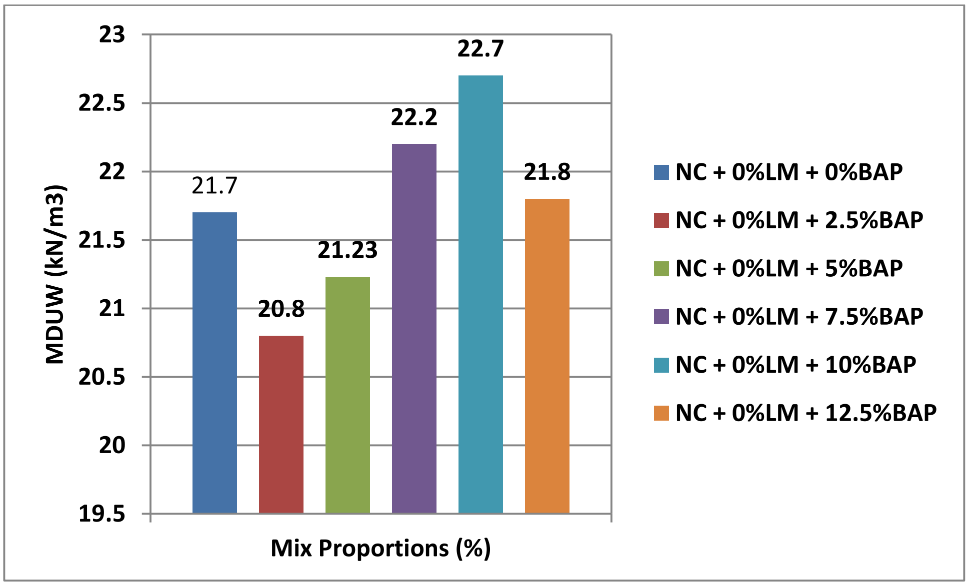

Figure 4.17: Maximum Dry Unit Weight of Natural Clay Stabilized with Bamboo Ash Powder

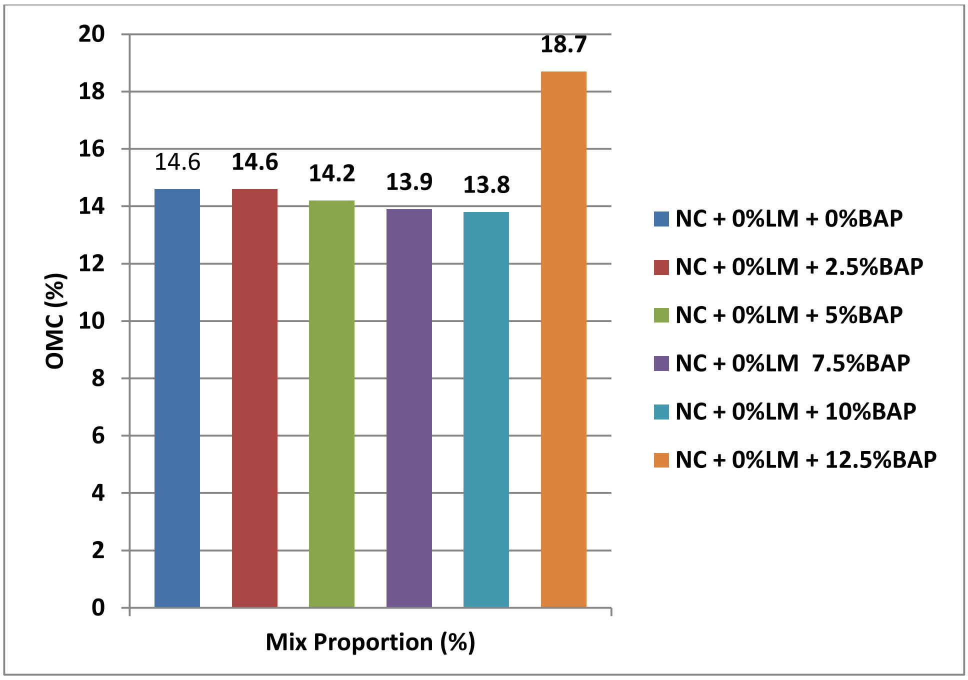

Figure 4.18: Optimum Moisture Content of Natural Clay Stabilized with Bamboo Ash Powder

4.5 California Bearing Ratio Test

Table 4.5 shows the soaked and un-soaked CBR characteristics of natural clay and natural clay stabilized with a blend of lime and bamboo ash powder. Results obtained for the soaked CBR samples revealed that the CBR of the natural clayey sample increased from 10.61% to 15.2% at 4% lime and 5% bamboo ash powder, beyond this point, the soaked CBR of the mixture decreased. Results obtained for the un-soaked CBR suggest that the un-soaked CBR of clay increased from its natural value of 14.4% to 23.5% at 8% lime and 10% bamboo ash powder beyond this point, the un-soaked CBR of the mixture decreased. Comparative assessment of the effect of lime and bamboo ash powder on soaked and un-soaked CBR values of natural clayey soils shows that a blend of lime and bamboo ash powder was effective in improving the CBR characteristics of the un-soaked clayey samples than the soaked clayey samples. Works indicative of these findings are the works of Asma and Dariuz, (2013) and Kassim, (2009).

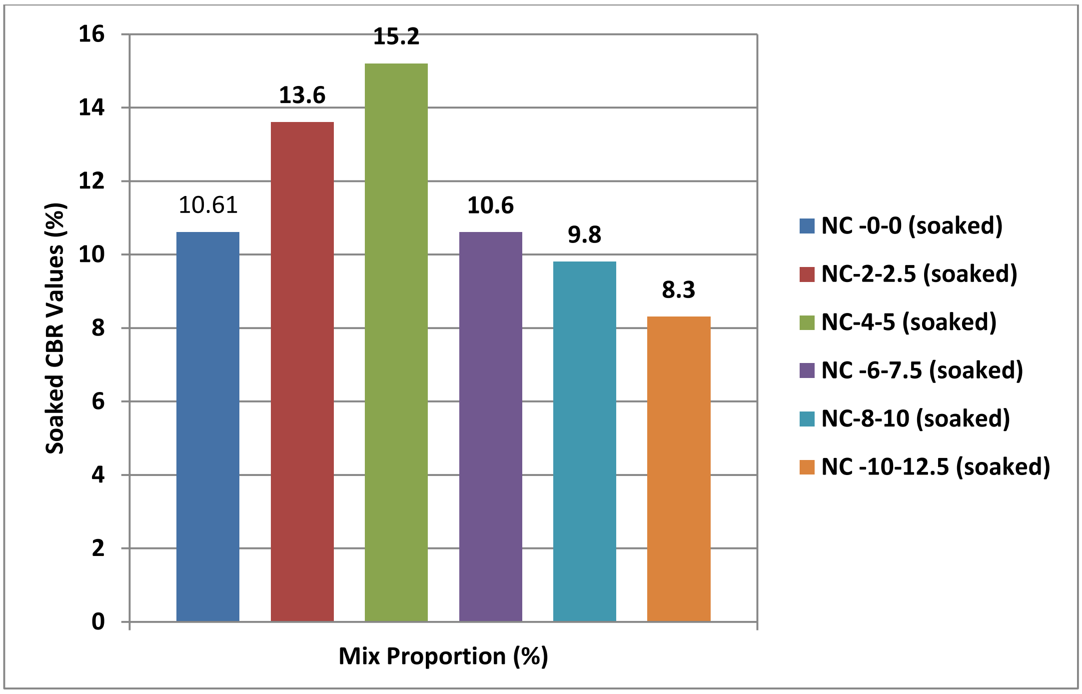

Figure 4.19: Soaked CBR Values of Natural Clay Stabilized with Lime and Bamboo Ash Powder

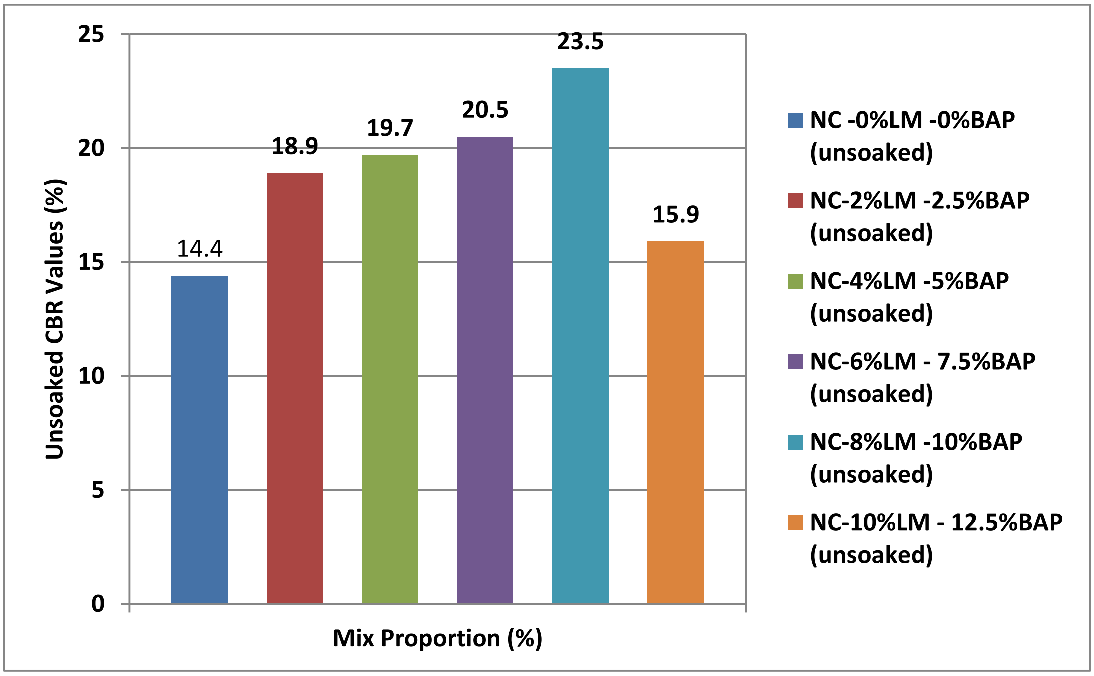

Figure 4.20: Un-Soaked CBR Values of Natural Clay Stabilized with Lime and Bamboo Ash Powder

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The following conclusion in the light of the findings obtained on investigation into the strength properties of clay stabilized with lime and bamboo ash powder can be drawn:

1. Evaluation of the particle size distribution of clay shows that the percentage passing sieve No 200 (0.075mm) was 43.7 and as a results, the clay sample was classified as CH (clay of high plasticity) according to Unified Soil Classification System and A-7-6 according to AASHTO Soil Classification System.
2. The specific gravity of the natural clayey soil decreased on addition of bamboo ash powder to clay but increased on addition of lime to clay.However, a slight increase in specific gravity of clay was observed from addition of bamboo ash powder and lime to clay.
3. The liquid limit, plastic limit and plasticity index of clay decreased on addition of bamboo ash powder, lime and a blend of lime and bamboo ash powder to clay.
4. The maximum dry unit weight of clay decreased slightly on addition of lime to clay but increased on addition of bamboo ash powder to clay. However, for a blend of bamboo ash powder and lime, the maximum dry unit weight of clay was found to increase.
5. The soaked CBR of the natural clayey soil increased from 0% lime and bamboo ash powder to 4% lime and 5% bamboo ash powder while the un-soaked CBR of natural clayey sample improved from 0% lime and bamboo ash powder to 8% lime and bamboo ash powder.
6. The blend of bamboo ash powder and lime added to natural clay was adjudged as an effective modifier for strength enhancement of clayey soils.

5.2 Recommendation

From the findings obtained on strength properties of clay stabilized with lime and bamboo ash powder, the following recommendation can be made:

1 Natural clay stabilized with a blend of lime and bamboo ash powder can only be effective at 4% lime and 5% bamboo ash powder as beyond this point, the strength properties of the natural clay may be undermined.

2 The recommendation 1 should be subjected to further investigation so as to note the additional blend of additives to be used so as to a ensure cost effective enhancement of strength properties of natural clayey soils.

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