EFFECTS OF FINE QUARRY DUST AND RICE HUSK ASH ON THE ENGINEERING PROPERTIES OF COARSE GRAINED SOIL.

BY

OFOEGBU OBIDI FRANCIS

REG NO: NAU/2017224038

SUBMITTED TO THE

DEPARMENT OF CIVIL ENGINEERING

NNAMDI AZIKIWE UNIVERSITY, AWKA

ANAMBRA STATE, NIGERIA

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IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF DEGREE OF BACHELOR OF ENGINEERING (B.Eng.) IN CIVIL ENGINEERING.

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APPROVAL PAGE

This project on "Effects of fine quarry dust and rice husk ash on the engineering properties of coarse grained soil" by Ofoegbu Obidi Francis with registration number 2017224038 was approved and accepted in partial fulfilment of the requirements for the award of B.Eng. Degree in civil engineering by:

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CERTIFICATION

This is to certify that this project was carried out and completed by Ofoegbu Obidi Francis and has been accepted by the Department of Civil Engineering, Nnamdi Azikiwe University (NAU) Awka, Anambra State.

Ofoegbu Obidi Francis. *(Student)*

Date

DEDICATION

This is project is dedicated to God Almighty of his wisdom, strength and knowledge during the course of this work.

ACKNOWLEDGMENT

I am immensely grateful to God who made it possible for me to start and finish this research work.

But it is worthy of note that this work will not have been complete without the supervision and guidance of my project supervisor Engr. P.N Onodagu who offered objective criticisms and corrections to see this work come to what it is today.

My appreciation also goes to my elder brother Azubuike who had help in sponsoring this work. I am also very grateful to all the staffs in my department firstly to the head of the department Engr.Prof.Ezeagu, Prof. (Mrs.) Nwaiwu, Prof. (Mr.). Nwaiwu, Rev.Dr Nwakire, Prof.Aginam, Engr.Mrs.Nkechi, Engr.Ubani, Engr.Dr.Adinna, Engr.Dr.Okonkwo, Engr.Ezenwamma, Engr.Nwajuaku. A.I, in the department of civil engineering as their teachings has help to facilitate my understanding in this thesis.

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ABSTRACT

This research work involves determining the effects of quarry dust and rice husk on the engineering properties of coarse grained soil. The test was carried out in the civil engineering laboratory Nnamdi Azikiwe University. The soil sample was collected at ifite, Awka Anambra state. The test was carried out on the natural sample and the sample with rice husk ash and quarry dust which include: moisture content, specific gravity, grain size distribution, Atterberng limits, compaction and California bearing ratio(CBR) tests both before and after the adding the quarry dust and rice husk ash at varying percentages and proportions. The results of the natural soil sample and improved soil sample quarry dust and rice husk ash indicate that it can be used to alter and make up for the deficiency in the particle sizes. The results of the optimum moisture content (OMC) in CGS-QD (coarse grained soil and quarry dust) mixtures brought a decrease in optimum moisture content and an increase in the MDUW (maximum dry unit weight) while that of the CGS-RHA mixtures brought a decreases in both the OMC and MDUW. The result obtained from the CGS-QD for the liquid limit brought a slight decrease in the liquid limit state and increased the plasticity from LOW to MEDIUM while that of the CGS-RHA mixtures reduces the plasticity index of the soil to that of NON-PLASTIC. The CBR value obtained from the natural soil sample was 2.5% which suggests that the soil is very weak and hence the need to undergo soil stabilization, at the application of quarry dust at 30% the CBR value of the soil was increased to 5.2% and at the addition of rice husk ash the CBR value was also increased to 4.5%, this suggests that quarry dust and rice husk ash are good stabilizing agents as they enhanced the strength and rigidity of the soil sample.

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CHAPTER ONE

1.1 Background of study

Soil is derived from the breakdown of rock minerals by weathering and /or erosion and it may have suffered some amount of transportation prior to deposition. The type of breakdown process and the amount of transport undergone by sediments influence the nature of the macro and microstructure of the soil which in turn influence its engineering behavior.

The engineering properties of the soil can vary widely over time and space so that their physical properties cannot be defined accurately at all locations for all conditions. Since soils are composed of a mixture of three dissimilar material CGS –soil solids, liquid fluids (usually water) and gaseous fluids (usually air) - the properties are influenced by the interaction of these three phases in the soil mass. The engineering properties' of soil CGS can be significantly influenced by many factors. Some of the factors that influence the behavior of soil are:

- Size ,shape and distribution of soil properties
- Minerology
- Degree of packing of soil particles
- Amount of water in the soil
- Climatic conditions and
- Degree of confinement (i.e depth).

The engineering properties of a soil –such as grain size distribution, plasticity, and shear strength can be assessed by proper laboratory testing.

Coarse grained soil comprises of sand, silt and clay. The importance of engineering properties cannot be over emphases. Generally it is a well-known fact that the stability of any engineering factor depends on the engineering properties of soil and which the structure rest. Hence adequate knowledge of engineering properties of soil before any constructional activity is of utmost importance. It will help the engineer during the design state and also in construction processes to give possible solutions that may arise in the construction and guarantee safety and economy of the proposed work.

This research is focused on the determination of the ***effects of rice husk ash and fine quarry dust on the engineering properties of a coarse grained soil*** **i**n terms of compaction behavior (to obtain the optimum moisture content at the maximum compaction density with respect specified laboratory standard used), index properties, moisture content, particle size distribution, specific gravity, Atterberng limit (liquid limit and plastic limit).

1.2 Statement of problem

Influence of a structural instability on roads and buildings are bound especially in Nigeria there is a no doubt that many that many of these instabilities are related to failure in the underlying soil. Probably there was no adequate knowledge of engineering properties of the soil during the design and construction of some of the structures.

1.3 Aims and objectives

The major aim of this project is to determine the effects of rice husk ash and quarry dust engineering properties of coarse grained soil

The underlying properties objectives are to:

1. Investigate the engineering properties of the coarse grained soil
2. Study the effects of rice husk ash on the engineering properties of the coarse grained soil
3. Study the effects of quarry dust on engineering properties of the coarse grained soil

1.4 Significance of study

This projects is so much important so as to provide information on the engineering properties of coarse grained soil and the effects of the additives i.e. rice husk ash and quarry dust which eventually helps to enhance the vital properties of the soil. It can offer a guide into solving problems of structural instability.

1.5 Scope of study

The scope of the study is centered on the vary effects of rice husk ash and quarry dust on the engineering properties of the coarse grained soil

The study is limited only to the analysis of the soil sample gotten from a farm in Ifite Awka , Anambra state. It involves carrying out various tests like grain size distribution, specific gravity, compaction tests and atterberng limits test.

CHAPTER TWO

LITERATURE REVIEW

2.1 Preamble

Several researches has been conducted to note the effects of the additives on the soil properties of a soils.

The practice of adding additives to the soil in order to improve the properties is known as soil stabilization.

In Nigeria, soil stabilization in road construction is widely spread in some riverine areas of the country such as Calabar in Cross-river State, Lagos State, and etc. (Ola, 1983) emphasis on soil stabilization practice, consequence and advantage in some part of Lagos State Nigeria. On the basis of the area treated, more than half of all current soil stabilization in Nigeria is mechanical stabilization although it reflects much on the federal and state road authorities.

Soil stabilization is any treatment such as compaction, cement stabilization etc. applied to a soil improve all its strength, durability and volume stability and reduce its vulnerability to water penetration (O' Flaherty, 2002).

A soil is said to be stable if the soil is able to withstand the stresses imposed on it. This definition applies irrespective of whether the treatment is applied to the soil in situ or after the soil has been removed and placed in a pavement or embankment.

The effect of stabilization is usually increased when the soil is compacted. Sometimes, compaction alone is insufficient to stabilize the soil. However, without an appropriate stabilizer the effect may not be permanent particularly in the case of increase exposure to water. By the need of stabilization, considerable time, cost and effort can saved (Terzaghi, 1952).

2.2 Soil improvement techniques or stabilization

Soil stabilization is the alternation of the physical or engineering properties of a soil mass and is usually employed when the soil are loose or highly compressible or when they have high permeability or other bad properties that makes them uncultivable for construction process. The process involves blending and mixing materials with a soil to improve the pertinent properties of the soil which includes soil strength, bearing capacity, durability under adverse moisture content, and to improve the volume stability of a soil mass (Omowunmi, 2006).

Soil can be improved either by modification or stabilization, or both. The modification of soil is done by the adding different type of modifiers like (cement, lime, etc.) to a soil in order to change soil’s index properties, though the stabilization of soil is its treatment to improve its strength and durability such that to make it suitable for building. Soil stabilization is modifications or adjustments of the soil CGS properties in order to fulfill the specified engineering requirements. The ways to stabilize the soil are compaction and usage of admixtures. Commonly used stabilizers for altering the properties of soils are Lime and Cement. Recent studies indicates the use of solid waste materials like fly ash and rice husk ash, quarry dust for soil stabilization by means of or devoid of lime or cement .

Soil stabilization aims at improving soil strength and increasing resistance to softening by water through bonding the soil particles together, water proofing the particles or combination of the two (Sherwood, 1993). Usually, the technology provides an alternative provision structural solution to a practical problem. The simplest stabilization processes are compaction and drainage (if water drains out of wet soil it becomes stronger).

The other process is by improving gradation of particle size and further to improvement can be achieved by adding binders to the weak soils (Rogers et al, 1996).

All these methods fall into two broad categories (FM 5-410) namely; -

Mechanical stabilization under this category, soil stabilization can be achieved through physical process by altering the physical nature of native soil particles by either induced vibration or compaction or by incorporating other physical properties such as barriers and nailing. This stabilization method mostly widely used on road construction throughout the world. In mechanical stabilization the lateral displacement under has provided by the natural force of cohesion and internal friction soil. The in the following methods are used

- **Stabilization by Compaction:** This is the artificial increase in weight of a material brought about mechanical means. It is achieved by expelling air from the soil mass thereby decreasing the void ratio.
- **Stabilization by Consolidation:** May be used to increase the soil density and may be achieved by lowering the water table, increasing the inter-granular pressure causing settlement and also possible altering the water content of clay soils to produce adequate strength
- **Thermal and Electric Stabilization:** This is rarely used, it is used highway construction. The process depends primarily on the clay minerals present and the extent to which they have partly or well crystallized structure.
- **Chemical stabilization** under this category, soil stabilization depends mainly on chemical reactions between stabilizer (cementitious material) and soil minerals (pozzolanic materials) to achieve the desired effect. It includes the addition of natural or artificial compound to the soil in order to improve the soil mechanical properties. The method employed includes:
  i. Chloride stabilization ii. Lignin stabilization
  ii. Lime stabilization iv Organic cationic stabilization

Through soil stabilization, unbound materials can be stabilized with cementitious materials. The stabilized soil materials have a higher strength, lower permeability and lower compressibility than the native soil (Keller bronchure 32-01E). The method can be achieved in two ways, namely; (1) in situ stabilization and (2) ex-situ stabilization. Note that, stabilization not necessary a magic wand by which every soil properties can be improved for better (Ingles and Metcalf, 1972). 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 (Ingles and Metcalf, 1972; Sherwood, 1993; EuroSoiCGStab, 2002). For a successful stabilization, a laboratory tests followed by field tests may be required in order to determine the engineering and environmental properties. Laboratory tests although may produce higher strength than corresponding material from the field, but will help to assess the effectiveness of stabilized materials in the field. Results from the laboratory tests, will enhance the knowledge on the choice of binders and amounts (Eurosoilstab, 2002).

2.3 Components of stabilization

Soil stabilization involves the use of stabilizing agents (binder materials) in weak soils to improve its geotechnical properties such as compressibility, strength, permeability and durability. The components of stabilization technology include soils and or soil minerals and stabilizing agent or binders (cementitious materials).

2.3.1 Soils

Most of stabilization has to be undertaken in soft soils (silty, clayey peat or organic soils) in order to achieve desirable engineering properties. According to Sherwood (1993) fine grained granular materials are the easiest to stabilize due to their large surface area in relation to their particle diameter. A clay soil compared to others has a large surface area due to flat and elongated particle shapes. On the other hand, silty materials can be sensitive to small change in moisture and, therefore, may prove difficult during stabilization (Sherwood, 1993). Peat soils and organic soils are rich in water content of up to about 2000%, high porosity and high organic content. The consistency of peat soil can vary from muddy to fibrous, and in most cases, the deposit is shallow, but in worst cases, it can extend to several meters below the surface (Pousette, et al 1999; Cortellazzo and Cola, 1999; Åhnberg and Holm, 1999). Organic soils have high exchange capacity; it can hinder the hydration process by retaining the calcium ions liberated during the hydration of calcium silicate and calcium aluminate in the cement to satisfy the exchange capacity. In such soils, successful stabilization has to depend on the proper selection of binder and amount of binder added (Hebib and Farrell, 1999; Lahtinen and Jyrävä, 1999, Åhnberg et al, 2003.

Soil is formed in a place or deposited by various forces of nature such as glaciers wind, lakes and rivers gradually residually on organically. Soil is the building block of civilization and along with water form two basic ingredient for human survival.

Description of Types of Soil

Type of material	Particles size	Description
Gravel	60mm	Coarse of rock like grantie, marble etc any shape
Sand	2 – 0.6mm	Particle warmly comprising silica or quart 3 lack cohesion in the pressure of water.
Clay	Smaller than 0.002mm (2N)	Results of chemical weathering of rocks forms cohesion in the presence of water and also excessive swelling and shrinkage
Organic matter	Several mm to several cm	Micrograms and fibres resulting from decomposition of plants and soil treaun smells like wet decaying wood.

Table 1

2.3.2 Stabilizing Agents

These are hydraulic (primary binders) or non-hydraulic (secondary binders) materials that when in contact with water or in the presence of pozzolanic minerals reacts with water to form cementitious composite materials. Stabilizers that can be used include:

- Cement
- Fly ash
- Lime
- Blast furnance slag
- Quarry dust
- Rice husk ash(pozzolanas)

Cement

Cement is the oldest binding agent since the invention of soil stabilization technology in 1960's. It may be considered as primary stabilizing agent or hydraulic binder because it can be used alone to bring about the stabilizing action required (Sherwood, 1993; EuroSoilstab, 2002). Cement reaction is not dependent on soil minerals, and the key role is its reaction with water that may be available in any soil (EuroSoils tab, 2002). This can be the reason why cement is used to stabilize a wide range of soils. Numerous types of cement are available in the market; these are ordinary Portland cement, blast furnace cement, sulfate resistant cement and high alumina cement. Usually the choice of cement depends on type of soil to be treated and desired final strength.

Hydration process is a process under which cement reaction takes place. The process starts when cement is mixed with water and other components for a desired application resulting into hardening phenomena. The hardening (setting) of cement will enclose soil as glue, but it will not change the structure of soil (EuroSoilstab, 2002). The hydration reaction is slow proceeding from the surface of the cement grains and the centre of the grains may remain unhydrated (Sherwood, 1993). Cement hydration is a complex process with a complex series of unknown chemical reactions (MacLaren and White, 2003).

However, this process can be affected by

- presence of foreign matters or impurities
- water-cement ratio
- curing temperature
- presence of additives
- specific surface of the mixture

Depending on factor(s) involved, the ultimate effect on setting and gain in strength of cement stabilized soil may vary. Therefore, this should be taken into account during mix design in order to achieve the desired strength. Calcium silicates, C3S and C2S are the two main cementitious properties of ordinary Portland cement responsible for strength development (Al-Tabbaa and Perera, 2005; EuroSoilstab, 2002).

Calcium hydroxide is another hydration product of Portland cement that further reacts with pozzolanic 5 materials available in stabilized soil to produce further cementitious material (Sherwood, 1993). Normally the amount of cement used is small but sufficient to improve the engineering properties of the soil and further improved cat ion exchange of clay. Cement stabilized soils have the following improved properties:

- Decreased cohesiveness (Plasticity)
- Decreased volume expansion or compressibility
- Increased strength (PCA-IS 411, 2003).

Lime

Lime provides an economical way of soil stabilization. Lime modification describes an increase in strength brought by cation exchange capacity rather than cementing effect brought by pozzolanic reaction (Sherwood, 1993). In soil modification, as clay particles flocculates, transforms natural plate like clays particles into needle like interlocking metalline structures. Clay soils turn drier and less susceptible to water content changes (Roger et al, 1993). Lime stabilization may refer to pozzolanic reaction in which pozzolana materials reacts with lime in presence of water to produce cementitious compounds (Sherwood, 1993, Eurosoilstab, 2002). The effect can be brought by either quicklime, CaO or hydrated lime, Ca (OH) 2. Slurry lime also can be used in dry soils conditions where water may be required to achieve effective compaction (Hicks, 2002). 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.

$CaO + H2O \rightarrow Ca\ OH\ 2 + Heat\ kJ\ mol.$

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 (Eurosoilstab, 2002; Sherwood, 1993).

The effect can be explained from Figure 1 for 6 soil at a moisture content of 35% and plastic limit 25%. Addition of 2% lime will change the plastic limit to 40% so that the moisture content of the soil will be 5% below plastic limit instead of 10% above plastic limit (Sherwood, 1993). Sherwood (1993) investigated the decrease in plasticity as brought about in first instance by cation exchange in which cations of sodium and hydrogen are replaced by calcium ions for which the clay mineral has a greater water affinity. Even in soils (e.g. calcareous soils) where, clay may be saturated with calcium ions, addition of lime will increase pH and hence increase the exchange capacity. Like cement, lime when reacts with wet clay minerals result into increased pH which favors solubility of siliceous and aluminous compounds. These compounds react with calcium to form calcium silica and calcium alumina hydrates, a cementitious product similar to those of cement paste. Natural pozzolanas materials containing silica and alumina (e.g. clay minerals, pulverized fly ash, PFA, blast furnace slag) have great potential to react with lime.

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.

FLY ASH

Fly ash is a byproduct of coal fired electric power generation facilities; it has little cementitious properties compared to lime and cement. Most of the fly ashes belong to secondary binders; these binders cannot produce the desired effect on their own. However, in the presence of a small amount of activator, it can react chemically to form cementitious compound that contributes to improved strength of soft soil. Fly ashes are readily available, cheaper and environmental friendly.

There are two main classes of fly ashes; class C and class F (Bhuvaneshwari et al, 2005, FM 5-410). Class C fly ashes are produced from burning subbituminous coal; it has high cementing properties because of high content of free CaO. Class C from lignite has the highest CaO (above 30%) resulting in self-cementing characteristics (FM 5-410). Class F fly ashes are produced by burning anthracite and bituminous coal; it has low self-cementing properties due to limited

Amount of free CaO available for flocculation of clay minerals and thus require addition of activators such as lime or cement. The reduction of swell potential achieved in fly ashes treated soil relates to mechanical bonding rather than ionic exchange with clay minerals (Mackiewicz and Ferguson, 2005). However, soil fly ash stabilization has the following limitations (White, 2005):

Plate 1: A road reclaimer mixes soil with moist conditioned fly ash (Beeghly, 2003).

- Soil to be stabilized shall have less moisture content; therefore, dewatering may be required.

- Soil-fly ash mixture cured below zero and then soaked in water are highly susceptible to slaking and strength loss

- Sulfur contents can form expansive minerals in soil-fly ash mixture, which reduces the long term strength and durability

2.4 QUARRY DUST

Quarry dust also known as stone dust is a kind of solid waste material that is generated from stone crushing industry which is abundantly available. It is estimated that each crusher unit produce 15%-20% stone dust. Disposal of such wastes creates lots of geo-environmental problems such as landfill disposal problems, health and environmental hazards. One of the best way to eliminate these problems is to make use such waste. Stone dust is also a solid waste material that is produced from stone crushing industry, it has been identified that crusher dust exhibits high shear strength and is beneficial as a geotechnical material. Stone dust is a kind of material that has pozzolanic properties and coarser contents in it while other materials like fly ash contains only pozzolanic property and no coarser soil particles. Significant enhancement in the properties of soils is described by different researchers by blending it with stone dust.

Quarry dust exhibits high shear strength which is highly beneficial for its use as a geotechnical material Soosan et al. (2001a). It has a good permeability and variation in water content does not seriously affect its desirable properties.

Quarry dusts are considered as one of the well accepted as well as cost effective ground improvement technique for weak soil deposits. They provide the primary function of reinforcement and drainage, and thus improve the strength and deformation characteristics of weak soil deposits.

Quarry dust is made while blasting, crushing and screening coarse aggregate, quarry dust has rough, sharp and angular properties and as such causes a gain in strength due to better interlock and a concomitant loss in workability.

During the production of aggregates through the crushing process of rocks in rubble crusher units, quarry dust is obtained as a by-product. Quarry dust waste has been used for different activities in the construction industry, such as road construction and building materials. Quarry dust waste act as lightweight aggregates, bricks, tiles, and autoclave blocks. Quarry dust waste also a waste material that is generated from the stone crushing industry.

2.4.1 PROCESS OF CRUSHING

Quartz, granite, limestone, dolomite, sand stone and quartzite are the major rock types that are used by the crushed stone industry, whereas, calcareous marl, marble, shell, and slate are the minor types. Blasting and drilling are the techniques which are used for loosening of rock and crushed stone products. Nature and location of the deposit will affect the techniques that are used for extraction and processing.

The processing operations includes the size classification, screening, crushing and material handling. Generally truck is used to deliver the quarry stone to the processing plant and is dumped in a bin. Large boulders are separated from the finer rocks, which do not require primary crushing, by feeder or screen separators and thus reducing the load on the primary crusher. Initial reduction is performed with the help of jaw, impactor etc.

Belt conveyor is generally used to convey the crushed product, normally 7.5 to 30 centimetres, to a surge pile for temporary storage or is stored as coarse aggregates. Scalping screen is a unit that separates oversized rock from smaller stone, to which stone from the surge pile is conveyed as shown in the Figure.

The undersized material obtained from the scalping screen is considered as a product stream and is transported to storage pile and sold as a base material. Cone crushers are generally used for secondary crushers in which, the stone that is too large to pass through the scalping screen is processed. Cone crusher reduces the material to about 2.5 to 10 centimeters. Tertiary crusher is the crusher which processes the output obtained from the secondary crusher. Cone crushers or other types of impact crushers are used to perform tertiary crushing. The third output is around 0.5 to 2.5 centimeters, is returned to the sizing screen. The process of screening differentiates the gradation sizes. The products are transported to the stock pile areas with the help of trucks such as washing, air separators, and screens. Quarry dust is produced in some of the stone crushing plants.

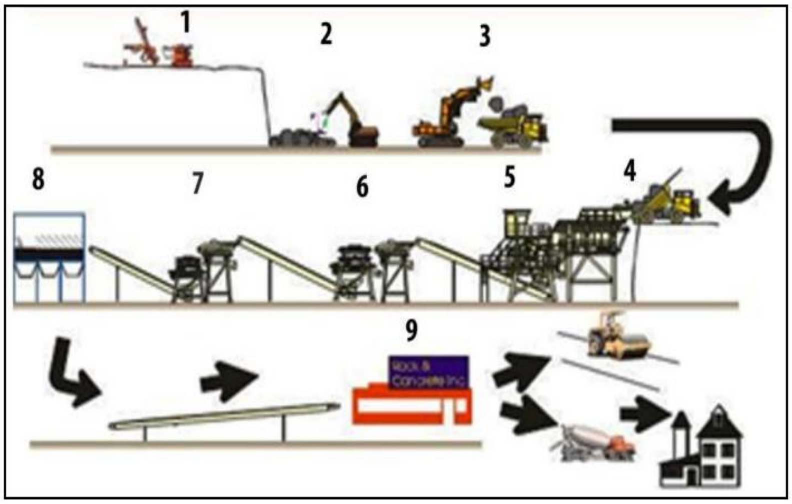


Plate 2 overview of crushing process

2.4.2 COMPONENTS OF QUARRY DUST

They are formed igneous rock with a tough, frozen melt with little texture or layering and they are mostly black, white or grey minerals. Quarry fines may also look like lava. (sinopis pigments website).

Quarry dust consists of dark-grey crushed chips and dust size 0.4mm in size.

The chemical composition of quarry dust consists of (60-90%) of sio2, (2-19%) of Al2O2, (1-7%) of Fe2O3, (3-5%) of Cao, (1-3%) of MgO, (1-2%) of Na2O, (1-3%) of K20, (0-0.5%) of Tio2.

2.4.3 USES OF QUARRY DUST

The use of quarry dust is to ensure economic stabilization of soil and also used under flexible pavements to increase the load carrying capacity of the pavement by distributing the load through a finite thickness pavement Eze-Uzoamaka and Agbo (2010).

The quarry dust can be used to know the compressive and tensile strength of the stabilized soil. The effect of quarry dust on the stability of soil aggregate mix used in a base course and summarizes that the quality of quarry dusts in a soil aggregate mix has a major influence on maximum density, strength, frost resistance and drainage.

Quarry dust is widely used as a setting bed for any of the various types of stone pavers (flagstone pavers such as bluestone being an example) used by homeowners in small projects. It can be smoothen to create a very flat surface and it is strong enough to support the weight of stone pavers which can be quite significant. When used with larger stones it serves as a bonding agent, such as in driveways, cement asphalt for streets and sidewalks.

2.5 RICE HUSK ASH

Rice husk is an agricultural waste obtained from milling of rice. About 108 tons of rice husk is generated annually in the world. In Nigeria, about 2.0 million tons of rice is produced annually. Rice husk ash has so much been utilized in the past for upgrading of soil whereas in Niger state, almost 96,600 tons of rice grains is made/produced in the year of 2000.. In most cases the RHA was used as an admixture and a stabilizing agent (cement or lime) included to increase the cementitious property of the stabilized matrix. In the meantime, the ash has been categorized below pozzolana, with concerning 67-70% silicon oxide and concerning 4.9% and 0.95% Aluminum oxide and iron oxides, this waste product cause a significant environmental problem if it is not disposed well. Rice husks comprise of a high percentage of lignin, and about twenty percent of their outer surfaces are covered with silica (Ndazi et al. [16]; Srivastava et al. [17]).

Rice husk ash (RHA) is one of the abundantly available material in rice grown countries. Rice husks are the hard protective coverings of rice grains which are separated from the grains during milling process rice husk is an abundantly available waste material in all rice producing countries, and it contains about 30%–50% of organic carbon. In the course of a typical milling process, the husks are removed from the raw grain to reveal whole brown rice which upon further milling to remove the bran layer will yield white rice. Rice husk constitutes about 20% of the weight of rice and its composition is as follows: cellulose (50%), lignin (25%–30%), silica (15%–20%), and moisture (10%–15%). Bulk density of rice husk is low and lies in the range 90–150 kg/m3.

Sources of rice husk ash (RHA) will be in the rice growing regions of the world, as for example China, India, and the far-East countries. . RHA is the product of incineration of rice husk. Most of the evaporable components of rice husk are slowly lost during burning and the primary residues are the silicates

The characteristics of the ash are dependent on:

- composition of the rice husks
- burning temperature
- burning time

It has been estimated that for every 100 kg of husks burnt in a boiler for example will yield about 25 kg of RHA. In certain areas, rice husk is used as a fuel for parboiling paddy in rice mills, whereas in some places it is field-burnt as a local fuel.

However, the combustion of rice husks in such cases is far from complete and the partial burning also contributes to air pollution. Under controlled burning conditions, the volatile organic matter in the rice husk consisting of cellulose and lignin are removed and the residual ash is predominantly amorphous silica with a (microporous) cellular structure.

The chemical composition of RHA is significantly dependent on combustion conditions, and the burning temperature must be controlled to keep silica in an amorphous state.

The ash obtained from uncontrolled combustion (as in open-field burning or in industrial furnaces at temperatures greater than 700°C–800°C) will contain significant amounts of cristobalite and tridymite which are nonreactive silica minerals. In order to develop pozzolanic activity, such ashes will be required to be ground to a very fine particle size which is likely to make their use financially unviable. Under controlled combustion (burning temperatures in the range of 500°C–700°C for a period of about 1 hour), amorphous silica is the major constituent of ash whose reactivity is attributed to the presence of this form of silica and to its very large surface area resulting from the microporous structure of ash particles. Although reactivity of a pozzolanic material improves upon increasing its fineness .Mehta and Monteiro (1997) reckon that grinding RHA to a high degree of fineness is not advisable since this material derives its pozzolanic activity from the internal surface area of its microporous particles which is already very high.

The applications of RHA include its use as a pozzolanic in the construction industry, as a filler, additive, abrasive agent, oil adsorbent, sweeping component, and as a suspension agent for porcelain enamels

In the construction industry, RHA can be used as a partial replacement for cement. . According to Chandrasekhar et al. (2006), each application requires specific properties such as reactivity for cement and concrete, chemical purity for synthesizing advanced materials, whiteness, and proper particle size for filler applications and high surface area and porosity for use as an adsorbent and catalyst, If used as a supplementary cementitious material in concrete, for example, RHA particles may have a high water demand due to their porous microstructure, this can be controlled by intergrinding the RHA particles with clinker during the process of cement manufacture so as to breakdown the porous structure and thereby reduce water demand. If intergrinding is not possible, then RHA may be used by blending it with cement at site. RHA in the blended cement will fix free lime released by clinker silicates during their hydration.

RHA is grayish-black in color due to unburned carbon. At burning temperatures of 550–800 °C, amorphous silica is formed, while crystalline silica is produced at higher temperatures. The specific gravity of RHA varies from 2.11 to 2.27; it is highly porous and light weight, with a very high specific surface area.

Typically, RHA is used in the form of ground RHA, having typical particle sizes generally less than 10 ìm; natural RHA (NRHA) has larger sizes of approximately 100 ìm.

RHA could be a terribly fine material. The typical particle size of rice-husk ash ranges from 3 to 75 µm. Physical properties numerous researches were applied to maximize the reactivity of RHA by burning in several conditions. The assembly of reactive RHA depends upon factors corresponding to temperature, acid treatment, and hours of exposure. . Completely burnt RHA is gray to white in color and partially burnt RHA is blackish.

Due to its highly micro porous structure, the specific surface area of RHA determined by Brunauer-Emmett-Teller (BET) using nitrogen adsorption method can range from 20 m2/g to as high as 270 m2/g, comparatively, silica fume has specific surface area of 18–23 m2/g (Singh et al., 2018). Level of carbon detected in RHA before thermal treatment averaged 18.60% and after thermal treatment the level of carbon content decreased considerably to 0.14% when thermally treated at 700°C for 6 h.

RHA has a high silica (SiO_2) content of about 85%–90%. RHA has its cellular structure before grinding while very irregular-shaped particles of ground RHA and there are pores among the cellular surface. RHA mainly comprises of amorphous silica with a very little quantity of crystalline phase, and thus the silica is accountable for the pozzolanic activity in the mortar or concrete (Sharmin et al., 2017).

The chemical reaction between RHA and cement creates a secondary Calcium-Silicate-Hydrate (C--S--H) gel which signifies the pozzolanic property of RHA particles,

According to Givi et al. (2010), the high percentage of silica content in RHA makes it a potential pozzolanic compound. Thus, for the pozzolanic activity, RHA can be used as a partial replacement of Portland cement while producing concrete

The chemical reactions of the pozzolanic material start when the di-calcium silicate (C_2S) and tri-calcium silicate (C_3S) compounds of cement come into contact with water during the hydration process

Rice husk is difficult to ignite and does not burn easily with an open flame, unless air is blown through the husk. Also, it has a high average calorific value of 3410 kcal/kg. Therefore, it is a good, renewable energy source. Rice husk can be used as an alternative energy source, i.e. as the fuel in the boiler of a rice-milling kiln to generate electricity where the heating value of the husk ranges from 12.6 MJ/kg (Xu et al., 2012) to 13.34–16.20 MJ/kg (Mansaray & Ghaly, 1997) to 15.7 MJ/kg, of which 18.8% is carbon, 62.8% is volatile materials, and 9.3% is moisture content (Ekasilp, Soponronnarit, & Therdyothin, 1995; Thorburn, 1982), and even up to 17 MJ/kg (Ferraro, Nanni, Vempati, & Matta, 2010).

ADVANTAGES OF RICE HUSK ASH

- Rice husk ash provides good compressive strength to the concrete.
- It is a by-product ;hence it helps in cutting down the environmental pollution

- The high silica content makes it a good supplementary cementitous material or pozzolanic admixture
- The density of concrete containing rice husk ash is similar to the normal weight concrete; hence, it can also be used for the general-purpose application too.
- The impervious microstructure of rice husk ash concrete provides better resistance to the sulphate attack, chloride ingress, carbonation etc.
- Rice hull concrete has good shrinkage property and increases the durability of concrete.

DISADVANTAGES OF RICE HUSK ASH

By the use of rise husk ash, concrete progressively becomes unworkable. Hence water-reducing admixtures should be used to obtain workable concrete for the ease of placement and compaction of concrete.

2.5.1 USES OF RICE HUSK ASH

The main uses of rice husk ash are as follows:

- Rice husk ash is used to make high-performance concrete.
- Rice husk ash is used as an insulator

Rice husk ash is also used in:

- In a ceramic glaze
- In roofing shingles
- In waterproofing chemicals
- In oil spill absorbents
- In specialty paints
- Flame retardants
- In insecticides and bio fertilizers etc.

2.5.2 CHEMICAL AND PHYSICAL PROPERTIES OF RICE HUSK ASH (RHA)

According to 'Siddika, Ayesha & Mamun, Md & Ali, Md.' (Published in Innovative Infrastructure Solutions), the chemical properties of rice husk ash are as follows:

CHEMICAL PROPERTIES:

SiO_2: 78–86

Al_2O_3: 1–2.0

Fe_2O_3: 16–1.85

CaO: 55–4.81

MgO: 35–4.5

SO_3: 24–1.18

Na_2O: 1–1.14

K_2O: 54–3.68

Loss in ignition: 4–8.55

2.5.3 PHYSICAL PROPERTIES:

The physical properties of rice husk ash vary depending on the temperature of burning and grinding of the rice husk.

Colour: Grey

Specific Gravity: 0.5-2.53

Specific Surface Area: 40-100 m^2/g

Bulk Density: 200-300 kg/m^3.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Collection and Preparation of sample

1 **Coarse grained soil**

The coarse grained soil sample was obtained from a farm in ifite awka anambra state,it was collected with the aid of shovel. The soil sample passed the physical tests in identifying a coarse grained soil, which are:

I. Dry –clods moderately difficult to break; somewhat gritty.
II. Moist- neither very gritty nor very smooth; forms a ball; stains fingers.
Coarse grained soil consists of a sand, silt and clay. The proportions of the sand, silt and clay was determined by test of particle size distribution. The natural moisture content of soil sample was obtained upon arrival.

2 **Quarry dust:** The quarry dust was ordered for from ebony state, the quarry dust serve as one the additives in the various tests that carried out.

3 **Rice husk ash**: The rice husk a byproduct from the milling of rice was gotten from store involved in de-stoning of rice and thereafter subjected to burning until the volatile matter in the rice husk consisting of cellulose and lignin are removed and the residual ash was obtained which was used as an additives for the various tests that was carried out.

3.2 METHODS

Civil Engineering Laboratory Testing.

The research demands a series of Civil Engineering Laboratory Test that would aid in understanding the effect of quarry dust and rice husk ash on certain properties of the coarse grained soil. The test were carried out in accordance with BS 1377 (1990) Standard.

The list of the civil engineering tests to be carried out are:

- Sieve Analysis(grain size distribution)
- Atterberg's Limit Test
- Compaction Test
- CBR test (California Bearing Ratio)
- Natural Moisture Content Test
- Specific Gravity Test

3.2.1 Grain Size Distribution (Sieve Analysis)

Sieve analysis is a practice used in civil engineering to assess the particle size distribution of a granular material by allowing the material to pass through a series of sieves of sieves of progressively smaller mesh size and weighing the amount of materials that is stopped by each sieve as fraction of the whole mass.

The grain size analysis is widely used in classification of soils. The data obtained from the grain distribution curve is used in the design of filters for earth dams and to determine the suitability of soil for road construction, air field etc.

Information obtained from grain size analysis can be used to predict soil water movement although permeability test are more generally used. Soil gradation is very important to geotechnical engineering; it is an indication of other engineering properties such as shear strength, compressibility and hydraulic conductivity. In a design, the gradation of the in-situ- soil help in the selection of filler material for the construction of highway embankment and it also control the design and ground water drainage of site. A poorly graded soil (one with predominantly one-sized particle) will have better drainage property than the well graded soil (soil with varieties of particle sizes) because of the relatively higher magnitude of void present. A well graded can be easily compacted more than a poorly graded soil. However most Engineering project may have graduation requirement that must be satisfied before the soil is to be used is accepted for construction work.

Soil type		Particle Size (mm)
Clay		<0.002
Silt		0.002-0.075
Sand	Fine	0.075-0.42
	Medium	0.42-2.0
	Coarse	2.0-4.75
Gravel		4.75-75

Plate 3.1 Ranges for grain Sizes of different Soil type.

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.
9. 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 3.3 Apparatus for Particle Size Distribution Test (Sieve shaker).

STEP BY STEP PROCEDURES:

1. The set of sieves to be used are thoroughly clean with the aid of a brush.
2. 500g of the coarse grained sample was weigh.
3. The weighed soil sample is poured 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 24hrs.
5. The sample is removed from the oven and the weight is determined.
6. The set of sieves is arranged in order and the placed on the sieve shaker and the sample is emptied into to the set of sieves.
7. The sieve shaker is disconnected from the power supplied and the weight in each sieves is recorded.

8. The percentage retained, percentage finer and cumulative % finer is calculated.
9. The graph of sieve Cumulative percentage finer against sieve sizes was plotted.
10. D10, D30 and D60 was determined 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 the density of a substance to the density of substance to the density of a reference substance. It is a dimensionless quantity that is commonly used to compare the densities of compare the densities of different materials.

The specific gravity of a substance is determined by diving its density by the density of the reference substance. The reference substance is typically water for liquids and solids and or hydrogen for gases.

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

STEP BY STEP PROCEDURES:

1. The density bottles is clean and weigh and recorded.
2. Weigh and record the weight of the empty clean and dry density bottle say (M_1)
3. 10g of the coarse grained soil was passed through 425um sieve and place inside the density bottle, it is then weigh and recorded as thus: weight of density bottle +dry soil + stopper say (M_2).
4. Distilled water is added to fill about half to three-fourth of the density bottle, and then soak 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. The density bottles stirred gently 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. It is allow to settle 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. The density bottle is empty and cleaned 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. The procedures was repeated for another trail again and the average of the specific gravity was taken.

The Procedure for Computation of result obtained are 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.

3.2.3 ATTERBERG LIMIT TEST.

The atterberg limits are a set of standardized tests used to determine the properties of fine-grained soils, such as clay and silts.

These limits provide important information about the soil's behavior, including its moisture content and its ability to change from a liquid state to a plastic or solid state.

The plasticity of soils can be identified practically by rolling a known weight of wet soil into a 3mm uniform diameter thread and the following inferences based on the observation are made and they are as follows:

1. **NON-PLASTIC:** If the wet soil cannot be rolled into thread.
2. **SLIGHTLY PLASTIC:** If the wet soil can be rolled into thread but crumbles easily under application of little pressure.
3. **PLASTIC:** If the wet soil can be rolled into 3mm thread but crumbles under intense application of pressure and cannot be reformed.

4. **VERY PLASTIC:** If the wet soil can be rolled into 3mm diameter thread but crumbles under intense application of pressure and can be reformed.

There are three primary atterberg limits:

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

Liquid Limit Test

The liquid limit is the moisture content at which a soil transitions from a liquid –like state to a plastic state.

Its determines by performing a test called the casagrande method which involves repeatedly grooving and closing a soil sample until the groove closes at a specified rate.

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.

STEP BY STEP PROCEDURES:

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 soil.

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.

STEP BY STEP PROCEDURES:

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

Where W_1 = Weight of empty tins.

W_2 = Weight of tin plus wet soil

W3 = Weight of tin plus oven-dried soil

1.2.4 NATURAL MOISTURE CONTENT

Aim

This test is aimed at determining the natural moisture content of the lateritic soil sample

Apparatus

Weighing balance, Containers, Electric oven, Dessicator

Procedures

An empty container will be weighed to 0.01g on the electronic weighing balance and filled with representative specimen of the soil sample. The container with the soil will then be weighed and recorded. The weight of the empty and filled containers will be taken as W1 and W2 respectively. Afterwards, oven drying for about 24 hours will be done at 105 to 110^0C. Then, the container will be placed in a desiccators to cool and then reweighed to give W3. The moisture content of the soil will be calculated as a percentage of the dry weight using the equation below:

M = (W2 – W3) (W3 – W1) x 100%

1.2.5 COMPACTION TEST

Aim

- To determine the maximum dry density and optimum content of the soil sample when compacted.
- To determine the rate of soil permeability of the soil sample.

The compaction test is a laboratory test performed to determine the relationship between the moisture content and the dry density of soil for a specified compaction energy. Compaction energy is the amount of mechanical energy that is applied to the soil mass. It involves the densification of soils by mechanical means thereby increasing the dry density of the soil by driving out aridity 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 civil engineering practice soil compaction is essential for the following reasons:

1. Increasing the bearing strength of foundation
2. Provide stability to slope and foundation.
3. Prevention of undesirable settlement of structures
4. Reduction of water seepage from structure

The compaction methods to be adopted for this research are:

1. British Standard Light for the natural samples of the coarse grained soil.
2. British Standard Light for the coarse grained and quarry dust, coarse grained and rice husk ash, coarse grained soil with quarry dust and rice husk ash mixtures respectively.

Details of British Standard Compaction Process

Table 3.2 Details of Compaction Mould.

Type	Diameter (mm)	Height (mm)	Volume(cm^3)
British Standard	105	115.5	1000

Table 3.3 Details of Compaction Procedure.

Type of test	Mould (cm^3)	Rammer(kg)	Drop (mm)	No of layers	Blow per layer
BS light	1000	2.5	300	3	27

The mechanical energy applied in each type of British Standard in term of work done is given as follows:

British Standard Light

Mechanical energy = $\frac{\boldsymbol{Weight\ of\ rammer \times no\ of\ layers \times no\ of\ blows \times height\ of\ drop}}{\boldsymbol{Volume\ of\ mould}}$

$=\frac{\boldsymbol{2.5g \times 3layers \times 27blows \times 300mm}}{\boldsymbol{1000}}$ **= 60.75kgm =60.75× 9.81Nm=596j**

Work done per unit volume of soil =$\frac{\boldsymbol{596}}{\boldsymbol{1000}}$ **=596kj/m³**

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 too also 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.

STEP BY STEP PROCEDURES:.

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) = $\frac{Wt\ of\ wet\ soil\ (kg)}{Vol\ of\ mould\ (m3)} = \frac{W2-W1}{W4}$

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

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

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

3.2.6 CALIFORNIA BEARING RATIO (CBR BS 1377: PART 4, 1990)

Aim

To determine the shearing strength of the soil.

Apparatus

1 CBR TEST APPARATUS: Consisting of loading machine with capacity of at least 5000 kg and equipped with a movable head or base which enables Plunger of 50 mm dia. to penetrate into the specimen at a rate of 1.25 mm/ minute.

Plate 3.8

2 CBR Mould with Base Plate, Stay Rod and Wing Nut:

Plate 3.9

4 Cylindrical mould: Inside dia. 150mm and height 175mm with a detachable perforated base plate of 235mm dia. and 10mm thickness. Net capacity - 2250 ml; conforming to IS-9669:1980 (Reaffirmed-2016).

5 Collar A detachable extension collar of 60 mm height.

6 Spacer Disc 148 mm in diameter and 47.7 mm in height along with handle.

7 Weights One annular metal weight and several slotted weights weighing 2.5 kg each, 147 mm in diameter, with a central whole 53 mm in diameter.

8 Compaction Rammer Weight - 4.89 kg with a drop 450 mm.

Plate 4.0

TEST PROCEDURES:

1. Remoulded specimen: The test material should pass 19 mm IS sieve and retained on 4.75 mm IS sieve. The dry density for a remoulding shall be either the field density or the value of the maximum dry density estimated by the compaction test (Heavy Compaction Test as per IS 2720 (Part-8) - 1983, for Railway Formation). The water content used for compaction shall be the optimum water content or the field moisture as the case may be.

2. Dynamic Compaction: A representative sample of the soil weighing approximately 4.5 kg or more for fine grained soil and 5.5 kg or more for granular soil shall be taken and mixed thoroughly with water. If the soil is to be compacted to the maximum dry density at the optimum moisture content, the exact mass of the soil required shall be taken and the necessary quantity of water added so that the water content of the soil sample is equal to the determined optimum moisture content.

3. Fix the extension collar and the base plate to the mould. Insert the spacer disc over the base. Place the filter paper on the top of the spacer disc.

4. Apply Lubricating Oil to the inner side of the mould. Compact the mix soil in the mould using heavy compaction. I.e. compact the soil in 5 layers with 55 blows to each layer by the 4.89 kg rammer.

5. Remove the extension collar and trim the compacted soil carefully at the level of top of mould, by means of a straight edge. Any holes developed on the surface of the compacted soil by removal of the coarse material, shall be patched with the smaller size material. Remove the perforated base plate, Spacer disc and filter paper and record the mass of the mould and compacted soil specimen. Place a disc of coarse filter paper on the perforated base plate, invert the mould and compacted soil and clamp the perforated base plate to the mould with the compacted soil in contact with the filter paper.

6. Place a filter paper over the specimen and place perforated plate on the compacted soil specimen in the mould. Put annular weights to produce a surcharge equal to weight of base material and pavement, to the nearest 2.5 kg.

The un-soaked method of CBR was used for the experiment:

For the un-soaked mould of soil, sufficient slotted weight not less than 4.5kg will be placed on the sample to simulate the overburden pressure.

The specimen will be placed in the compression machine and the piston of the machine will be made to seat through the surface and internal body of the trimmed soil sample using a seating load not greater than 4.5kg penetration readings and the corresponding load reading will be taken and recorded on the data sheet.

Calculations:

1. If the initial portion of the curve is concave upwards, apply correction by drawing a tangent to the curve at the point of greatest slope and shift the origin. Find and record the correct load reading corresponding to each penetration. C.B.R. = (PT/PS) X 100 where PT = Corrected test load corresponding to the chosen penetration from the load penetration curve. PS = Standard load for the same penetration taken from the table above.

2. C.B.R. of specimen at 2.5 mm penetration =

3. C.B.R. of specimen at 5.0 mm penetration =

4. The C.B.R. values are usually calculated for penetration of 2.5 mm and 5 mm. Generally the C.B.R. value at 2.5 mm will be greater than at 5 mm and in such a case/the former shall be taken

as C.B.R. for design purpose. If C.B.R. for 5 mm exceeds that for 2.5 mm, the test should be repeated. If identical results follow, the C.B.R. corresponding to 5 mm penetration should be taken for design.

5 Graph

Draw graph between Loads versus Penetration.

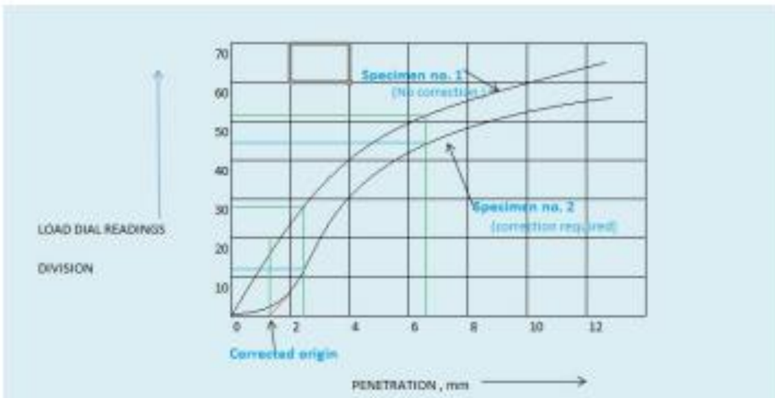


Plate 5.0

3.3 Soil classification system

Two commonly used systems for Classifying soils based on ***particle distribution*** and ***Atterberg limits:***

1. AASHTO System: American Association of State Highway and Transportation Official.

2. USCS: Unified Soil Classification System

1 AASHTO SYSTEM:

AASHTO system of soil classification was developed by Hogentogler and Terzaghi in 1929 as the Public Road Administration classification system. It has undergone several revisions, with the present version proposed by the Committee on Classification of Material for Subgrades and

Granular Type Roads of the Highway Research Board in 1945 (ASTM designation D-3282; AASHTO method M145). • The system is based on the following three soil properties:

1. Particle-size distribution (AASHTO T-11 and AASHTO T-27 test)

2. Liquid Limit (AASHTO T-89 test).

3. Plasticity Index (AASHTO T-90 test)

Key Elements:

1. Grain Size: • Gravel: Fraction passing 75mm sieve and retained on #10 (2mm) US sieve • Sand: Fraction passing #10 sieve and retained #200 sieve • Silt and Clay: Fraction passing #200 sieve

2. Plasticity: • Term silty is applied when fine fractions have a PI < 10 • Term clayey is applied when fine fractions have PI > 11

3. Groups: (see Tables) • Soils are classified into eight groups, A-1 through A-8. • The major groups A-1, A-2, and A-3 represent the coarse grained soils. • The A-4, A-5, A-6, and A-7 represent fine grained soils. • The A-8 are identified by visual inspection

The ranges of the LL and PI for groups A-2 , A-4, A-5 , A-6 and A-7:

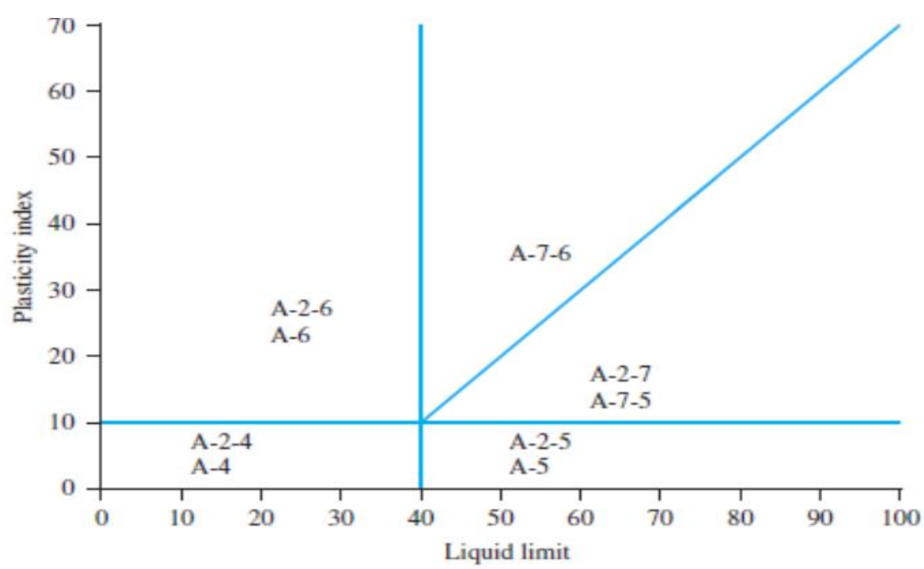


Fig 3.0

2 Unified Soil Classification System, USCS

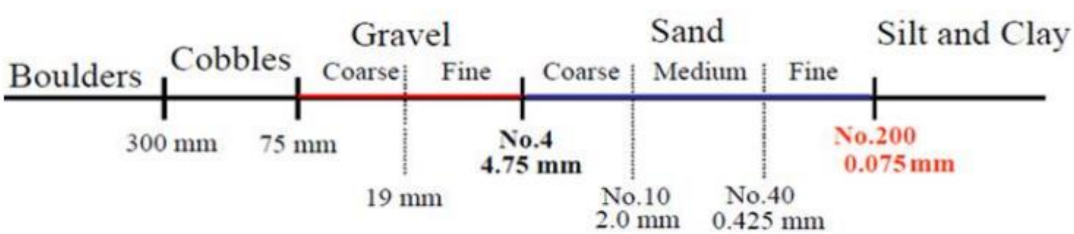
Origin:

The Unified Soil Classification system was first developed by Professor A. Casagrande in 1942 for the purpose of airfield construction during World War II. Afterwards, it was expended and revised in cooperation with the U.S. Bureau of Reclamation and the U.S. Army Corps of Engineers so that it applies not only to airfields but also to embankments, dams, foundations, and other engineering features. In 1969 the American Society for Testing and Materials (ASTM) adopted the USCS as a standard method for classification for engineering purposes (ASTM Test Designation D-2487). The USCS is based on the recognition of the type and predominance of the constituents considering grain-size, gradation, plasticity and compressibility. It classifies soils into four major categories:

1. Coarse-grained 2. Fine-grained 3. Organic soils 4. Peat

Procedures for Classification:

- From sieve analysis and the grain-size distribution curve determine the percent passing as the following:



- First, Find % passing # 200.
- If 5% or more of the soil passes the # 200 sieve, then conduct Atterberg Limits (LL & PL).
- If the soil is fine-grained (≥50% passes #200) follow the guidelines for fine grained soils.

SYMBOLS

Soil symbols:

- G: Gravel
- S: Sand
- M: Silt
- C: Clay
- O: Organic
- Pt: Peat

Liquid limit symbols:

- H: High LL (LL>50)
- L: Low LL (LL<50)

Gradation symbols:

- W: Well-graded
- P: Poorly-graded

Well – graded soil

$1 < C_c < 3$ and $C_u \geq 4$

(for gravels)

$1 < C_c < 3$ and $C_u \geq 6$

(for sands)

GROUP SYMBOLS

- The group symbols for coarse-grained gravelly soils are: GW, GP, GM, GC, GC-GM, GW-GM, GW-GC, GP-GM, and GP-GC.
- The group symbols for fine-grained soils are: CL, ML, OL, CH, MH, OH, CL-ML, and Pt.

Example:

SW, Well-graded sand

SC, Clayey sand

SM, Silty sand,

MH, Elastic silt

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Classification and Symbols

COARSE-GRAINED SOILS
(more than 50% of material is larger than No. 200 sieve size.)

	Clean Gravels (Less than 5% fines)	
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)	
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
	Clean Sands (Less than 5% fines)	
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	SW	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)	
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures

FINE-GRAINED SOILS
(50% or more of material is smaller than No. 200 sieve size.)

SILTS AND CLAYS Liquid limit less than 50%	ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
	CH	Inorganic clays of high plasticity, fat clays
	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

Table3.4

Laboratory Classification Criteria

GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limits above "A" line with P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for GW	
SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
SC	Atterberg limits above "A" line with P.I. greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent .... GW, GP, SW, SP
More than 12 percent .... GM, GC, SM, SC
5 to 12 percent .... Borderline cases requiring dual symbols

Table3.5

Unified Soil Classification System, USCS

Criteria for assigning group symbols				Group symbol
Coarse-grained soils More than 50% of retained on No. 200 sieve	**Gravels** More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[a]	$C_u \geq 4$ and $1 \leq C_c \leq 3$[c]	GW
			$C_u < 4$ and/or $1 > C_c > 3$[c]	GP
		Gravels with Fines More than 12% fines[a,d]	$PI < 4$ or plots below "A" line (Figure 5.3)	GM
			$PI > 7$ and plots on or above "A" line (Figure 5.3)	GC
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[b]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[c]	SW
			$C_u < 6$ and/or $1 > C_c > 3$[c]	SP
		Sands with Fines More than 12% fines[b,d]	$PI < 4$ or plots below "A" line (Figure 5.3)	SM
			$PI > 7$ and plots on or above "A" line (Figure 5.3)	SC
Fine-grained soils 50% or more passes No. 200 sieve	**Silts and clays** Liquid limit less than 50	Inorganic	$PI > 7$ and plots on or above "A" line (Figure 5.3)[e]	CL
			$PI < 4$ or plots below "A" line (Figure 5.3)[e]	ML
		Organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$; see Figure 5.3; OL zone	OL
	Silts and clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line (Figure 5.3)	CH
			PI plots below "A" line (Figure 5.3)	MH
		Organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$; see Figure 5.3; OH zone	OH
Highly Organic Soils	Primarily organic matter, dark in color, and organic odor			Pt

[a] Gravels with 5 to 12% fine require dual symbols: GW-GM, GW-GC, GP-GM, GP-GC.

[b] Sands with 5 to 12% fines require dual symbols: SW-SM, SW-SC, SP-SM, SP-SC.

[c] $C_u = \frac{D_{60}}{D_{10}}$; $C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}}$

[d] If $4 \leq PI \leq 7$ and plots in the hatched area , use dual symbol GC-GM or SC-SM.

[e] If $4 \leq PI \leq 7$ and plots in the hatched area, use dual symbol CL-ML.

Table 3.6

Unified Soil Classification System, USCS

Plasticity Chart :

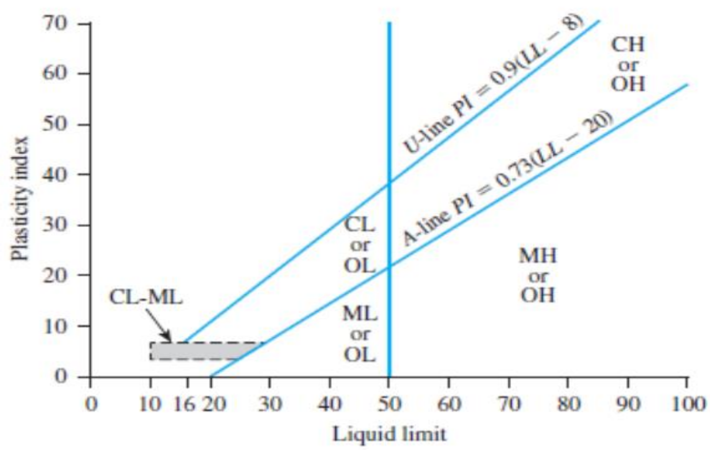


Fig 3.1

CHAPTER FOUR

RESULTS AND DISCUSSION

During the course of the research work, certain results were obtained which was used to classify the Soil Samples. This result which is referred to as Index Properties of the Soil Sample is shown in Table 4.0.

Properties	**Coarse grained soil**
Specific Gravity (Gs)	2.3
Liquid Limit (%)	20.2
Plastic Limit (%)	15.9
Plasticity Index (%)	4.3
Plasticity	Low
% Passing Sieve No 10 (2mm)	100
% Passing Sieve No 200 (0.075mm)	12.7
American Association of State Highway and Transportation Officials (AASHTO).	A-2-4
Unified Soil Classification System (USCS).	SC

Table 4.1

Index properties of Coarse grained soil and Quarry dust

Properties	**90%CGS +10%QD**	**80%CGS +20%QD**	**70%CGS+ 30%QD**
Specific Gravity (Gs)	2.39	2.48	2.6
Liquid Limit (%)	20.0	19.6	19.8
Plastic Limit (%)	15.2	14.5	13.8
Plasticity Index (%)	4.8	5.1	6.0
Plasticity	LOW	LOW	MEDIUM

Table 4.2

Index properties of coarse grained soil (CGS) and Rice husk ash (RHA).

Properties	**90%CGS +10%RHA**	**80%CGS+ 20%RHA**	**70%CGS +30%RHA**
Specific Gravity (Gs)	2.05	1.81	1.57
Liquid Limit (%)	24.8	29.4	34.0
Plastic Limit (%)	0	0	0
Plasticity Index (%)	0	0	0
Plasticity	NON-PLASTIC	NON-PLASTIC	NON-PLASTIC

Table 4.3

Compaction characteristics of coarse grained soil and quarry dust

Properties	**100%CGS +0%QD**	**90%CGS+ 10%QD**	**80%CGS+ 20%QD**	**70%CGS +30%QD**
Maximum Dry Unit Weight (kN/m^3) (BSL)	19.1	19.9	20.78	21.7
Optimum Moisture Content (%)(BSL)	15.6	15.1	14.6	14.1

Compaction characteristics of coarse grained soil (CGS) and Rice husk ash (RHA)

Properties	**100%CGS + %RHA**	**90%CGS +10%RHA**	**80%CGS +20%RHA**	**70%CGS +30%RHA**
Maximum Dry Unit Weight (kN/m^3) (BSL)	19.1	15.9	12.7	11.1
Optimum Moisture Content (%)(BSL)	15.6	11.5	9.0	7.4

4.1 Analysis of sample and specimen

4.2.1 Particle Size Distribution Test (Sieve Analysis Test).

Figure 4.1 is the semi logarithmic plot of the particle size distribution of the coarse grained soil sample. From the graph of Figure 4.1, the percentage passing Sieve No 200 (0.075mm) is 25.651, while that passing sieve No 10 (2mm) is 99.88. According to AASHTO Classification System, the coarse grained soil is classified as A-2-4. According to USCS Classification System, the coarse grained soil is classified as is classified as sand with clay sized particles (SC). According to the Federal Ministry of work Standard Specification for roads and bridges (1997) for a sample to be used for road construction the percentage by weight passing the sieve No 200 shall be less than but not greater than 35%. Sequel to the above the sample under review is found to be good because the percentage by weight passing sieve no 200 for the soil do not exceed 35%.

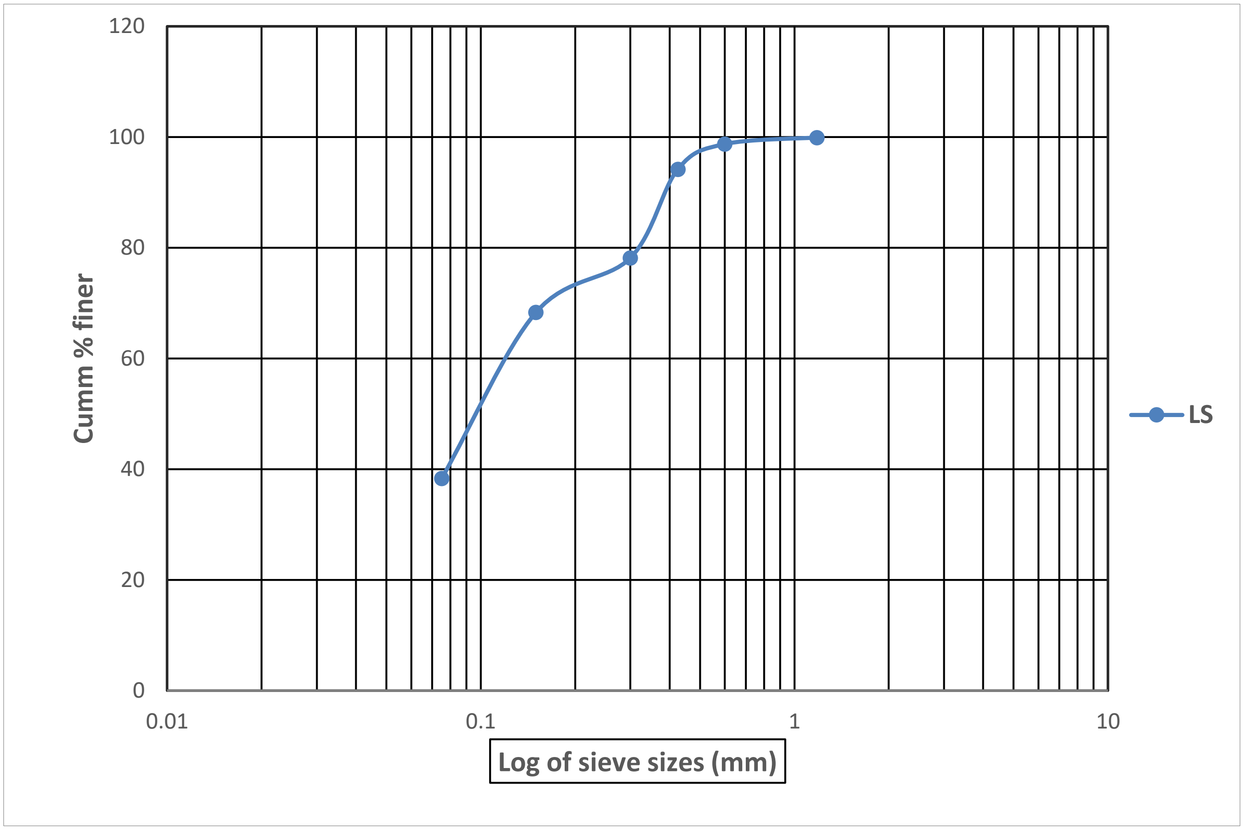


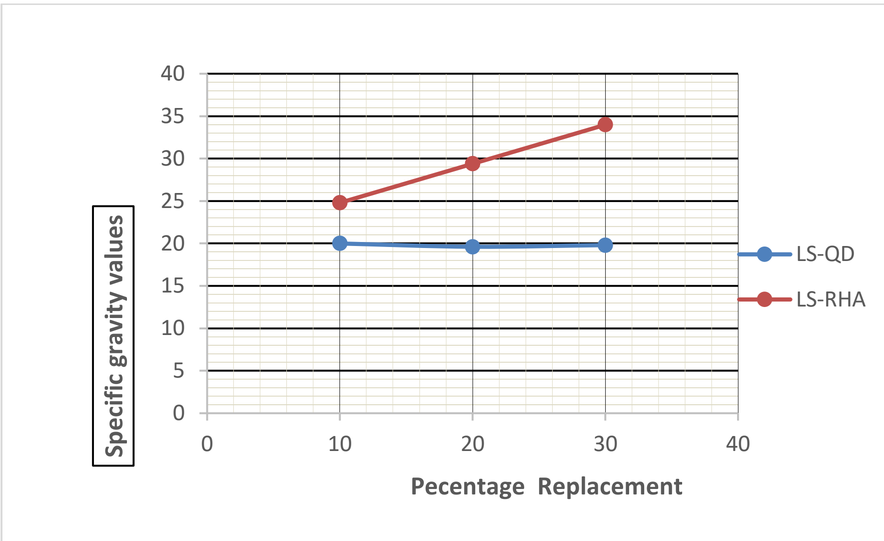
Figure 4.1 particle size distribution curve

4.2 Specific Gravity Test

The variation of the specific gravity of the soil sample with quarry dust and rice husk ash is shown in the table 4.1 to 4.2. The specific gravity of the coarse grained soil sample increased initially from 2.3 to 2.39 at the addition of 10%QD and increased to 2.6 at 30% addition of quarry dust.

It can be adduced that the coarse grained soil sample has a good relationship with the quarry dust.

The specific gravity of the CGS-RHA mixtures decreased from 2.6 to 2.05 at 10% addition of rice husk ash (RHA) and subsequently to 1.57 at 30% addition of rice husk ash (RHA). It can said that rice husk ash has a poor relationship with the coarse grained soil sample, this is as a result of the low specific gravity of the rice husk ash. The range of specific gravity from for all the mixtures suggests the presence of clay or silt which actually can be of advantage at the sub-grade and sub-base level of road construction. According to the Federal Ministry of work Standard Specification for roads and bridges (1997) a good sub-grade material should have specific gravity value ranging from 2.5 to 2.75.

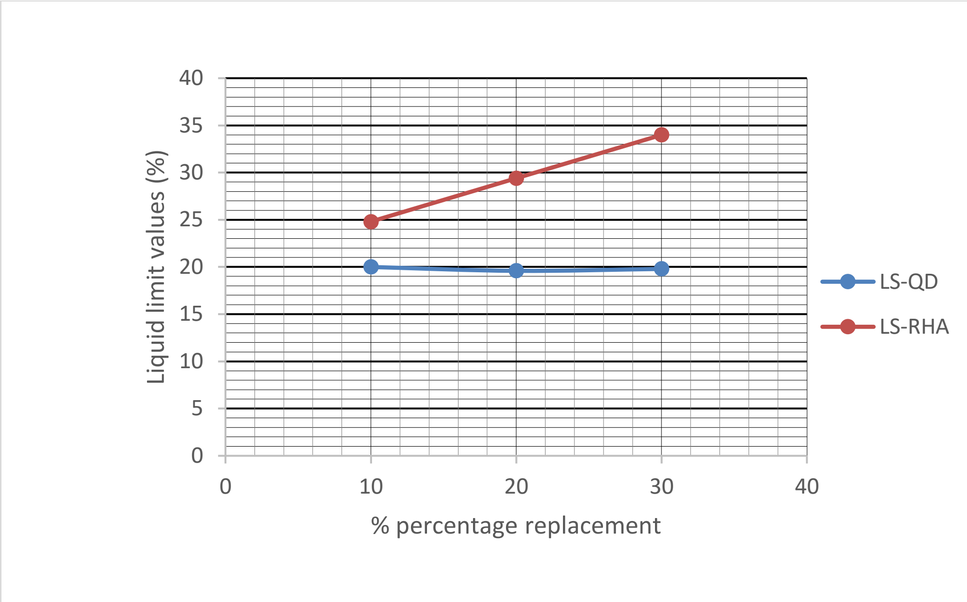


Graphs showing the specific gravity values of CGS-QD, CGS-RHA against %replacement

4.3 Liquid limit Test

The variation of the liquid limit test of the CGS-QD mixtures decreased from 20.2 to 19.8 at 30% addition of quarry dust the most probable reason for this may be due to the mechanical stabilization and addition of non-plastic material this result mirrors the works of (Dr. M.S Dixit 2016), while that of CGS-RHA mixtures increased significantly from 20.2 to 34.0 at the addition 30% of RHA this can be attributed to the high water holding capacity of rice husk ash.

According to the federal ministry of works and housing (1997) for road works recommended liquid limits of 50% maximum for sub-base and base materials. The above studies mixtures shows that with the increased percentage of quarry dust makes the soil more suitable for road works and the increased percentage of rice husk ash makes it unfit since the liquid limit increases and will eventually exceeds the standard specification.



Graphs showing the liquid limit values of CGS-QD, CGS-RHA mixtures against % replacement.

4.4 Plastic limit

Plastic limit is a characteristics of fine-grained soils, which refers to the moisture content at which a soil transitions from a plastic state to a semi-solid state. The plastic limit of the coarse grained soil reduces from 15.9 to 13.8 at 30% addition of quarry dust this can be as a result of the fact that quarry dust consists of fine particles to grain sized particles, when mixed with soil it changes the particle size distribution of the soil. Fine particles tend to fill the void spaces between larger soil particles reducing the overall porosity and permeability of the soil. Quarry dust particles also absorb water from surrounding soil, causing a decrease in the available water content for the soil, this reduction in the moisture content can lead to a decrease in the plastic limit of the soil, the reduction in the plastic limit can be attributed to the cementitous properties of quarry dust which its exhibit when hydrated.

The plastic limit of the coarse grained soil reduces to zero a 30% addition of rice husk ash this can be as a result of the fine particles of rice husk ash which fill the void spaces between soil particles resulting, in a denser soil structure. This densification can reduce the plasticity of the soil.

4.5 Plasticity index

The plasticity index of a soil is the numerical difference between the Liquid limit and the plastic limit. Plasticity index are indices of the quantity of clay sized particles and their mineralogical composition. Typically, higher liquid limits and plasticity indices values are linked with soils having a significant amount of clay particles or particles having higher surface activity. The plasticity index results obtained from the two composite mixtures (CGS-QD, CGS-RHA), indicate the plasticity increased significantly at the addition of quarry dust from low to medium and decreased at the addition of rice husk ash from low to becoming non-plastic.

According to the federal ministry of works standard specification for roads and bridges which states that the plasticity index of sub-grade materials should not exceed 12% and the above mixtures satisfies this requirements.

4.6 Compaction test

1. **MDUW**: The variation of the MDUW of the CGS-QD, CGS-RHA mixtures is shown in the fig below. Result recorded for the BSL for CGS-QD mixtures shows that the MDUW increased significantly from 19.1 to 21.7 at 30% addition of quarry dust. This can be attributed to the angular shape and interlocking nature of the particles allow for better compaction when compared to rounded particles like sand. This results in a higher dry unit weight because the particles can fit together more closely, the increase in dry unit weight can also be attributed to the rough surface texture which provides additional interlocking and frictional resistance between particles during compaction, promoting densification and increasing the dry unit weight.

   Result recorded for the BSL for the CGS-RHA decreased significantly from 19.1 to 11.1 at the addition of 30% addition of rice husk ash, this can be attributed to the light weight nature of rice husk ash when added to the soil it increases the overall volume without significantly increasing the weight, the shape and size of rice husk ash particles also contribute to the decreased in dry unit weight. The particles are generally small and irregularly shaped allows them to occupy more space within the mixture leading to lower density

2. **OMC:** The variation of the OMC of CGS-QD, CGS-RHA mixtures is shown in the fig below.

   Result recorded for the BSL for the CGS-QD decreased from 15.6 to 14.1% at the addition of 30% quarry dust, the reason for this can be attributed to the particle size distribution of the dust, and the fine quarry dust particles used fill the voids between soil particles reducing the pore space for water. The can lead to a decreases in the OMC as less moisture is required to achieve maximum compaction., the angular shape of the quarry dust enhances the interlocking ability. When mixed the soil the angular dust particles interlocking and reduce the void ratio which eventually leads to a decrease in the OMC, the fine particles of the quarry dust possess some cohesive properties. These fine particles can contribute to the cementation of soil particles, increasing cohesion within the soil matrix. The increased cohesion results to the decreased in the OMC as less moisture is required to achieve compaction.

   Result recorded for the BSL for the CGS-RHA decreased from 15.6 to 7.4% at the 30% addition of the rice husk ash this can be attributed to the pozzolanic properties meaning it can reacts with lime in the presence of moisture to form cementitious compounds, this reaction can lead to the formation of additional bonds and increased cohesion within the soil matrix. As a result the OMC of the soil may decrease, the addition of RHA can improve the overall compatibility of soils by enhancing particles interlocking and reducing the void ratio. This improvement in compaction efficiency can result in higher dry densities at lower moisture contents leading to a decrease in the OMC, the fine particles of the rice husk ash fill the voids within the soil structure, by occupying these voids RHA reduces the available pore spaces for water to occupy. This can lead to a decreased in the OMC as less water is needed to achieve maximum compaction.

4.7 California bearing ratio (CBR):

TABLE 1

			DIAL GUAGE READING		COREECTED TEST LOAD (KN)	
Proving Ring Factor	Standard Load For Same Penetration(KN)	Penetration Of Plunger (mm)	TOP	BOTTOM	TOP	BOTTOM
a	b	c	d	e	d x a	e x a
0.04465		0.00	0	0	0	0
0.04465		0.50	2.0	2.1	0.00893	0.093765
0.04465		1.0	2.5	2.6	0.111625	0.11609
0.04465		1.5	4.0	4.1	0.1786	0.183065
0.04465		2.00	5.0	5.6	0.22325	0.25004
0.04465	13.2	2.50	6.50	6.8	0.290225	0.30362
0.04465		3.00	7.0	7.3	0.31255	0.325945
0.04465		3.50	7.5	7.8	0.334875	0.34827
0.04465		4.00	8.0	8.2	0.3572	0.36613
0.04465		4.50	11.0	11.2	0.49115	0.50008
0.04465	20	5.00	12.0	12.2	0.5358	0.54473
0.04465		5.50	12.50	12.8	0.558125	0.57152
0.04465		6.00	13.50	13.6	0.58045	0.60724
0.04465		6.50	14.00	14.20	0.6251	0.63403
0.04465		7.00	15.00	15.30	0.66975	0.683145
0.04465		7.50	16.00	16.30	0.7144	0.727795

The above table shows the readings of the CBR of the coarse grained soil sample the evaluation of the CBR is calculated as follows:

CBR calculations:

Top:

$CBR_{2.5}$= 0.290225/13.2 X 100 =2.19%

$CBR_{5.0\ =}$0.5358/20 X 100 =2.67%

AVEARGE = (2.19 + 2.67) /2 =2.43%

Bottom:

$CBR_{2.5}$ =0.30362/13.2 X 100 =2.3%

$CBR_{5.0\ =}$0.54473/20 X 100 =2.723%

AVERAGE = (2.3+ 2.723) / 2 =2.5%

From the calculation the CBR for the soil sample is 2.5 % (the highest value is selected) this shows that the strength of the soil is very weak hence it cannot be used for subgrade, hence the need for the soil to undergo stabilization.

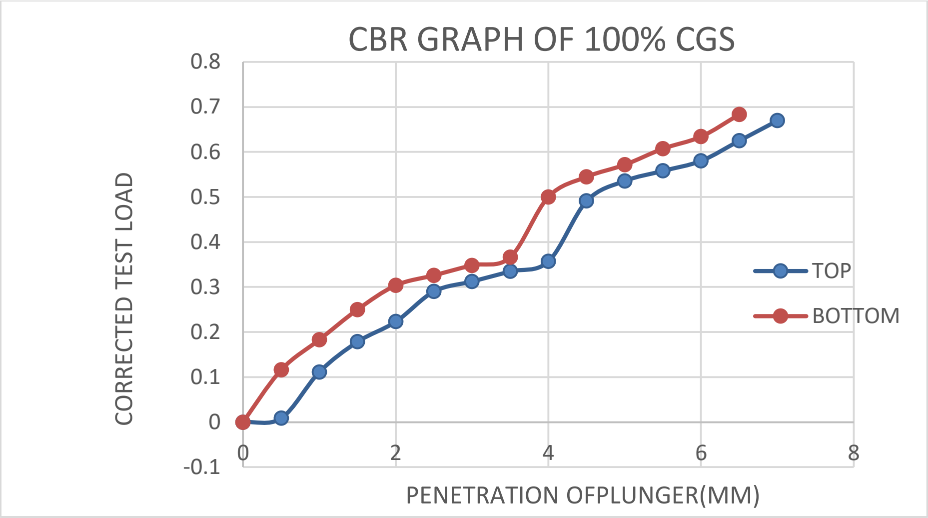


TABLE 2

			DIAL GUAGE READING		CORECTED TEST LOAD (KN)	
Proving Ring Factor	Standard Load For Same Penetration(KN)	Penetration Of Plunger (mm)	TOP	BOTTOM	TOP	BOTTOM
a	b	c	d	e	d x a	e x a
0.04465		0.00	0	0	0	0
0.04465		0.50	2.0	2.1	0.0893	0.093765
0.04465		1.0	4.2	4.3	0.18753	0.191995
0.04465		1.5	6.5	6.6	0.290225	0.29469
0.04465		2.00	8.8	8.9	0.39292	0.397385
0.04465	13.2	2.50	10.30	10.40	0.459895	0.46436
0.04465		3.00	12.2	12.30	0.54473	0.549195
0.04465		3.50	13.2	13.40	0.58938	0.59831
0.04465		4.00	14.30	14.50	0.638495	0.647425

0.04465		4.50	15.50	15.80	0.692075	0.70547
0.04465	20	5.00	16.50	16.40	0.736725	0.73226
0.04465		5.50	17.00	17.10	0.75905	0.763515
0.04465		6.00	17.50	17.60	0.781375	0.78584
0.04465		6.50	18.10	18.30	0.808165	0.817095
0.04465		7.00	18.50	18.60	0.826025	0.83049
0.04465		7.50	19.10	19.20	0.852815	0.85728

TABLE 2

The above table shows the CBR readings of coarse grained soil sample and 10% addition of quarry dust.

CBR calculation

Top:

$CBR_{2.5}$= 0.459895/13.2 X 100 =3.48%

$CBR_{5.0\ =}$0.736725/20 X 100 =3.68%

AVEARGE = (3.48 + 3.68) /2 =3.58%

Bottom:

$CBR_{2.5}$ =0.46436/13.2 X 100 =3.52%

$CBR_{5.0\ =}$0.73226/20 X 100 =3.66%

AVERAGE = (3.52+3.66) / 2 =3.6%

The CBR of CGS-QD mixtures at 10% addition obtained was 3.68% this shows that addition of the quarry dust increases the strength of the soil.

The result above reveals that at the addition of quarry dust at 10% the CBR value is increase this implies the strength of the is increase; this can be attributed to the angular and rough texture of quarry dust particles enhances the frictional resistance between soil particles. This interlocking effects improves the shear strength and stability of the soil, thereby increasing the CBR value.

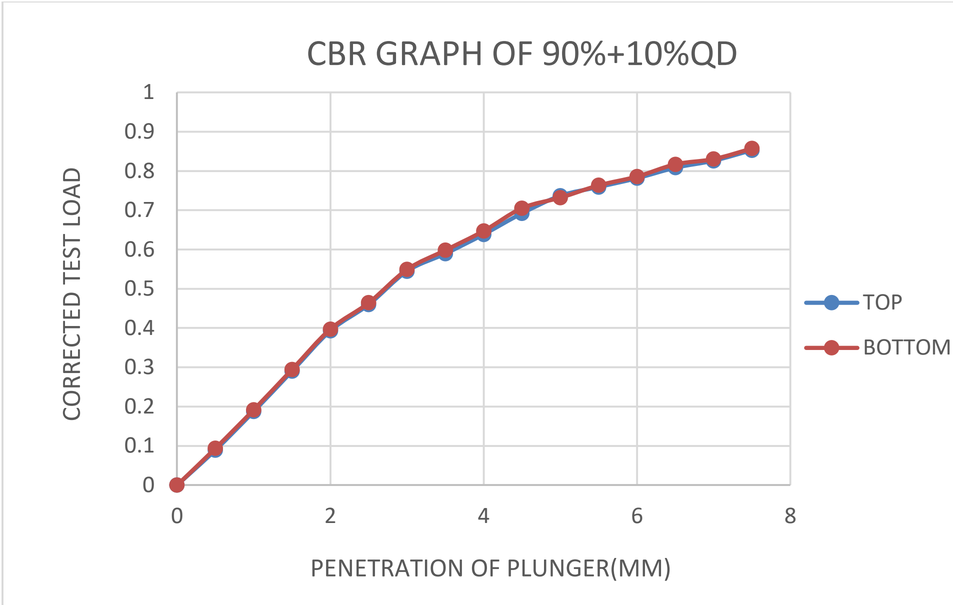


TABLE 3

			DIAL GUAGE READING		COREECTED TEST LOAD (KN)	
Proving Ring Factor	Standard Load For Same Penetration(KN)	Penetration Of Plunger (mm)	TOP	BOTTOM	TOP	BOTTOM
a	b	c	d	e	d x a	e x a
0.04465		0.00	0	0	0	0
0.04465		0.50	1.5	1.60	0.066975	0.07144
0.04465		1.0	3.0	3.1	0.13395	0.138415
0.04465		1.5	5.0	5.1	0.22325	0.227715
0.04465		2.00	6.0	6.1	0.2679	0.272365
0.04465	13.2	2.50	9.50	9.60	0.424175	0.42864
0.04465		3.00	10.50	10.70	0.468825	0.477755
0.04465		3.50	11.00	11.20	0.49115	0.50008
0.04465		4.00	12.00	12.20	0.5358	0.54473

0.04465		4.50	13.50	13.60	0.602775	0.60724
0.04465	20	5.00	14.80	14.70	0.66082	0.656355
0.04465		5.50	16.10	16.20	0.718865	0.72333
0.04465		6.00	17.00	17.20	0.75905	0.76798
0.04465		6.50	18.00	18.20	0.8037	0.81263
0.04465		7.00	19.00	19.1	0.84835	0.852815
0.04465		7.50	19.20	19.30	0.85728	0.861745

The above shows the CBR readings of the coarse grained soil sample with the addition of 10% of rice husk ash.

CBR calculation

Top:

$CBR_{2.5}$= 0.424175/13.2 X 100 =3.21%

$CBR_{5.0}$ =0.66082/20 X 100 =3.3%

AVEARGE = (3.21 + 3.3) /2 =3.255%

Bottom:

$CBR_{2.5}$ =0.42864/13.2 X 100 =3.247%

$CBR_{5.0}$ =0.656355/20 X 100 =3.28%

AVERAGE = (3.247+3.28) / 2 =3.26%

The CBR value for the 10% addition of rice husk ash which is 3.2% speaks of an increment from 2.5%, the suggests rice husk ash helps to increase the CBR value which implies the strength of the soil is increased this can be attributed to the fact that rice husk ash fills the voids and this improves the compaction and density of the soil thereby enhancing its load-bearing capacity and CBR value.

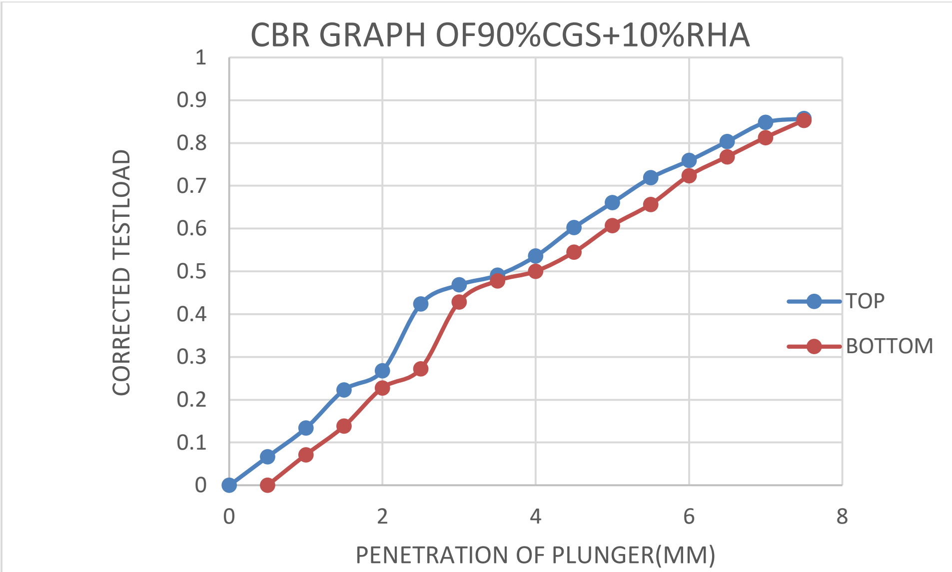


TABLE 4

			DIAL GUAGE READING		COREECTED TEST LOAD (KN)	
Proving Ring Factor	Standard Load For Same Penetration(KN)	Penetration Of Plunger (mm)	TOP	BOTTOM	TOP	BOTTOM
a	b	c	d	e	d x a	e x a
0.04465		0.00	0	0	0	0
0.04465		0.50	3.0	3.10	0.13395	0.138415
0.04465		1.0	6.10	6.20	0.272365	0.27683
0.04465		1.5	9.0	9.10	0.40185	0.406315
0.04465		2.00	12.30	12.40	0.549195	0.55366
0.04465	13.2	2.50	15.00	15.10	0.66975	0.674215

0.04465		3.00	16.10	16.20	0.718865	0.72333
0.04465		3.50	17.00	17.00	0.75905	0.75905
0.04465		4.00	18.00	18.10	0.8037	0.808165
0.04465		4.50	20.1	20.20	0.897465	0.90193
0.04465	20	5.00	23.3	23.40	1.040345	1.04481
0.04465		5.50	25.00	25.10	1.11625	1.120715
0.04465		6.00	25.50	25.60	1.138575	1.14304
0.04465		6.50	26.00	26.10	1.1609	1.165365
0.04465		7.00	27.00	26.60	1.20555	1.18769
0.04465		7.50	28.00	27.0	1.2502	1.20555

The above reading shows the CBR readings of the coarse grained soil sample with 30% addition quarry dust the evaluation of the CBR is as follows:

CBR calculation

Top:

$CBR_{2.5}$=0.66975/13.2 X 100 =5.07%

$CBR_{5.0}$ =1.040345/20 X 100 =5.20%

AVEARGE = (5.07+ 5.20) /2 =5.135%

Bottom:

$CBR_{2.5}$ =0.674215/13.2 X 100 =5.107%

$CBR_{5.0}$ =1.04481/20 X 100 =5.22%

AVERAGE = (5.22+ 5.107) / 2 =5.16%

From the calculation the CBR value increases from 2.5% to 5.16% at 30% addition this shows that the quarry dust increases the strength of the soil this can be attributed to the angular and rough texture of quarry dust particles enhances the frictional resistance between soil particles.

This interlocking effects improves the shear strength and stability of the soil, thereby increasing the CBR value.

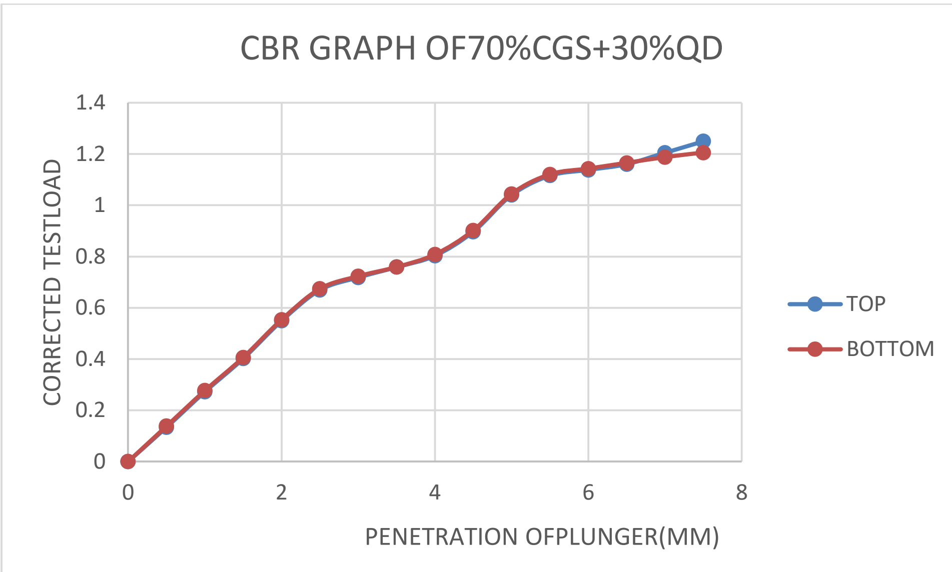


TABLE 5

			DIAL GUAGE READING		COREECTED TEST LOAD (KN)	
Proving Ring Factor	Standard Load For Same Penetration(KN)	Penetration Of Plunger (mm)	TOP	BOTTOM	TOP	BOTTOM
a	b	c	d	e	d x a	e x a
0.04465		0.00	0.0	0	0	0
0.04465		0.50	2.0	2.10	0.0893	0.093765
0.04465		1.0	6.0	6.10	0.2679	0.272365
0.04465		1.5	9.2	9.30	0.41078	0.415245

0.04465		2.00	11.50	11.70	0.513475	0.522405
0.04465	13.2	2.50	13.30	13.50	0.593845	0.602775
0.04465		3.00	15.50	15.60	0.692075	0.69654
0.04465		3.50	16.80	16.90	0.75012	0.754585
0.04465		4.00	17.20	17.30	0.76798	0.772445
0.04465		4.50	18.20	18.30	0.81263	0.817095
0.04465	20	5.00	20.20	20.30	0.90193	0.906395
0.04465		5.50	21.10	21.20	0.942115	0.94658
0.04465		6.00	21.80	21.60	0.97337	0.96444
0.04465		6.50	22.00	22.00	0.9823	0.9823
0.04465		7.00	22.20	22.20	0.99123	0.99123
0.04465		7.50	22.40	22.60	1.00016	1.00909

The above table shows the CBR readings of the coarse grained soil sample with 30% addition of rice husk ash, the evaluation of the CBR of the mixtures is as follows:

CBR calculation

Top:

$CBR_{2.5}$= 0.593845/13.2 X 100 =4.49%

$CBR_{5.0}$ =0.90193/20 X 100 =4.5%

AVEARGE = (4.49 + 4.5) /2 =4.49%

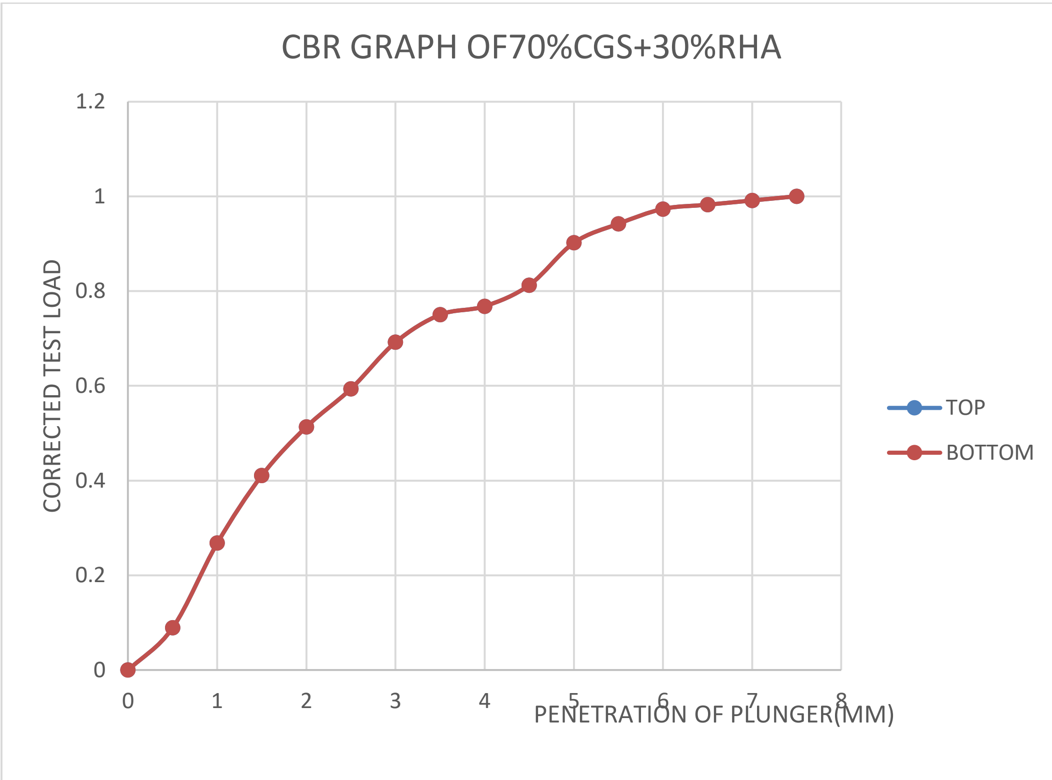
Bottom:

$CBR_{2.5}$ =0.602775/13.2 X 100 =4.56%

$CBR_{5.0}$ =0.906395/20 X 100 =4.53%

AVERAGE = (4.56+4.53) / 2 =4.545%

From the above calculation the CBR value is 4.56% which again enhances the strength of the soil this tells is a good stabilizing agent to improve the strength of soils,


CBR GRAPH OF70%CGS+30%RHA
CORRECTED TEST LOAD
1.2
1
0.8
0.6
0.4
0.2
0
0
1
2
3
4
5
6
7
8
PENETRATION OF PLUNGER(MM)
TOP
BOTTOM

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATION

The geotechnical properties of the coarse grained soil sample and the soil sample with rice husk ash, and the quarry dust have been carried out in compliance with the BS 1377 (1990) and ASTM D 3080 (2001) and from the result obtained the following deduction can be made:

1 According to AASHTO Classification System the coarse grained soil is classified as **A-2-4** and according to the USCS the soil is classifies as sand with clay sized particles (SC).

2 The specific gravity of the coarse grained soil sample increases from 2.3 to 2.6 at the 30% addition of quarry dust this shows quarry dust as a good relationship with the soil sample, while at the addition of rice husk ash at 10% the specific gravity of the sample drop to 2.05 and further decrease to 1.57 at 30%, the shows that rice husk ash has a poor relationship with the coarse grained soil sample.

3 The variation of the liquid limits of the CGS-QD suggests that the liquid limit of the natural soil sample decreases from 20.2 to 19.8 at 30% addition of the coarse grained soil sample, while for the CGS-RHA mixtures the liquid limit increase significantly from 20.2 to 34.0.

4 The plastic limit of the CGS-QD mixtures decreases from 15.9 to 13.8%,while that of CGS-RHA mixtures reduces to Zero at 30% addition of RHA, this can be attributed to the formation of cementitiuos compounds when it reacts with lime in the presence of moisture and this reaction reduces the plastic limit of the soil.

5 The plasticity index of the soil results obtained from the natural sample is low in plasticity and the CGS-QD mixtures at 30% addition of quarry dust suggests an increase in the plasticity index of the soil from LOW to MEDIUM and that of the CGS-RHA mixtures brought the plasticity index of the soil to be NON-PLASTIC.

6 Results obtained from the BSL for the CGS-QD mixtures shows the MDUW of the soil sample increased significantly from 19.1 to 21.7 at 30% addition of quarry dust. This can be attributed to

the angular shape and interlocking nature of the particles allow for better compaction when compared to rounded particles like sand.

Result recorded for the BSL for the CGS-RHA decreased significantly from 19.1 to 11.1 at the addition of 30% addition of rice husk ash, this can be attributed to the light weight nature of rice husk ash when added to the soil it increases the overall volume without significantly increasing the weight, the shape and size of rice husk ash particles also contribute to the decreased in dry unit weight.

7 Result recorded for the BSL for the CGS-QD shows a decreased in the OMC from 15.6 to 14.1% at the addition of 30% quarry dust, the reason for this can be attributed to the particle size distribution of the dust, and the fine quarry dust particles used fill the voids between soil particles reducing the pore space for water. The can lead to a decreases in the OMC as less moisture is required to achieve maximum compaction., the angular shape of the quarry dust enhances the interlocking ability.

9 The CBR value of the natural sample obtained was 2.5% this suggests the strength of the soil is weak and has poor bearing capacity and may not be suitable for supporting heavy loads without additional reinforcement or stabilization. The CBR value obtained for CGS-QD mixtures at 10% addition was 3.68% and 3.2% for CGS-RHA at 10% addition this suggests that at the addition of quarry dust and rice husk ash the strength of the soil is enhanced this is further shown when 30% of quarry dust and rice husk ash was added the CBR value obtained for the CGS-QD mixture was 5.16% and the CGS-RHA mixtures was 4.56%.

5.2 Recommendation

1 The coarse grained soil sample when stabilized with quarry dust can serve as a subgrade material which can withstand moderate loads.

2 The use of rice husk ash and quarry dust can serve as good stabilizing agent in increasing the strength of soil.

REFERENCES

AASHTO M145-91 (2003). Standard specifications for Classification of soils and soil-Aggregates Mixtures for Highway construction purposes.

.Abebaw, Z. (2005). Basic Engineering Properties of Lateritic Soils Found in Nejo-Mendi Road Construction Area, Welega. Pp. 1-11.

Adekoya, J.A. (1987). Preliminary Geotechnical and Geotechnical Study of Lateritic Weathering Profiles Derived From Banded Gneiss in the Ibadan Area, Southwest Nigeria Proceedings of the 9th Regional Conference For Africa on Soil Mechanics and Foundation Engineering, Lagos Nigeria Pp: 21-30.

Aginam, CH. Nwakire, C. and Nwajuaku, A.I (2015). Engineering Properties of Lateritic Soil From Anambra Central Zone Nigeria. Amsterdam, Newyork. Blue Eye Intelligence Engineering and Sciences Publication, ISSN:2231-2307, Volume-4, Issue-6.

Albrecht, B. A. and Benson, C. H. (2001) Effect of desiccation on compacted natural clay. J. Geotech & Geoenvir.Engrg., ASCE, 127(1):67 – 75.

Amadi, A. A. and Eberemu, A. O. Delineation of compaction criteria for acceptable hydraulic conductivity of lateritic soil-bentonite mixtures designed as land_ll liners. Journal of Environmental Earth Sciences, Vol. 67, No.4, 2012, pp 999{1006.

Amadi, A. A. and Osinubi, K. J. Assessment of Bentonite Inuence on Hydraulic Conductivity of Lateritic Soil. International Journal of Engineering Research in Africa, Vol. 3, 2010, pp 84{93. 19.

American Society For Testing and Material (2001). Standard Method For Direct Shear Test of Soil under Consolidated Drained Condition (ASTM D3080). ASTM International PA, USA.

Aroja, M. Nagaraj, K. Prashant, Rajeshwari, S. (2017) "Soil Stabilization Using Geosynthetic Material, Visvesvaraya Technological University Belgium (Pp.9).

ASTM (2004). Standard Test Method For Liquid Limit, Plastic Limit and Plasticity index of Soils. D 4318-00, Annual Book of ASTM Standards, Section 4 Construction Soil and Rock (I), Baltimore, Md U.S.A Volume 04,08.

Atkinson, J. (2002). Particle Size Analysis, University of the West England, John City University.

Braja, M. (2002). Soil Mechanics Laboratory Manual 6th-Edition, New York, Oxford University Press.

Braja, M. (2006) "Specific Gravity" In Principle of Geotechnical Engineering, 7th Edition, Cengage Learning (Pp.34)

Lukiantchuki, J. A. and Esquivel, E. R. Hydrauli Conductivity and Shear strength Behaviour of compacted lateritic soil bentonite mixtures used for sanitary land_ll liners. Soils & Rocks, Vol., 33, 2010, p69{79.

Agbede, O.A. (1992). Characteristics of tropical Red soils as foundation materials, Nigerian Journal of Science, Vol. 26 pp 237-242.

Alkaya, D. and Esener, A.B., 2011. Usability of sand-bentonite-cement mixture in the construction of unpermeable layer. Scientific research and essays.

Alther, G., 1982. The role of bentonite in soil sealing applications. Bulletin of the association of Engineering Geologists

Amadi, A. N., Eze, C. J., Igwe, C. O., Okunlola, I. A. and Okoye, N. O., (2012). Architect's and Geologist's view on the causes of building failures in NigeriaModern Applied Science,6 (6), 3138.

Benson, C. H. Zhai, H. Wang, X. (1994) Estimating hydraulic conductivity of compacted clay liners. J. Geotech.Engrg. ASCE, 2: 366 – 387.

BRRI/Lyon Associates. Laterites and lateritic soils and other problem soils of Africa. An engineering study for USAID. AID/csd-2164, Baltimore, Md,1971.

Balogun, L. A. (1984). Influence of geological origin on the geotechnical properties of lime-stabilized laterites. Proc., 8th Regional Conf. for Africa on Soil Mechanics and Foundation Engineering International Society for Soil Mechanics and Foundation Engineering,
1, 355-362.

Charman, J. H. Laterite in Road Pavements. Construction Industry Research and Information Association (CIRCA), Special Publication 47, Westminster,London, 1988.

D'Appolonia, D.J., 1980. Soil bentonite slurry trench cutoffs journal of the Geotechnical Ebgineering Divisions.

Das, B.M. (1998). Principles of Geotechnical Engineering, 4th Edition,PWS publishing company,USA

De. Graft-Johnson, J.W.S. and Bhatia, H.S. (1969). Engineering properties of lateritic soils

General Report, Specific Session on Engineering Properties of lateritic soils, Seventh International Conference on Soil Mechanics and Foundation Engineering Mexico City, Vol. 1:117-128.

Dixon, D.A., M.N., and Thomas, A.W., 1985. A study of the compaction properties of potential clay-sand buffer mixtures for the use in nuclear fuel waste disposal, Engineering Geology.

Etim, R. K., Eberemu, A. O. and Osinubi, K. J. (2014). Effect of lime – iron ore tailing blend on the expansive behavior of black cotton soil. Proceedings 2014 Nigerian Engineering Conference, Theme: Engineering andTechnology for Economic Transformation. Ahmadu Bello University Zaria. 15th – 18th Sept. 2014, Zaria,Kaduna State. CD ROM of Presentation. Section 6. Pg. 864 – 872.

Eberemu, A. O. (2011). Desiccation Induced Shrinkage of Compacted Tropical Clay Treated with Rice Husk Ash.International Journal of Engineering Research in Africa, Vol. 6 (2011) pp 45-64.doi:10.4028/www.scientific.net/JERA.6.45

Etim, R.K., Eberemu, A.O., and Osinubi, K.J. (2017). Stabilization of black cotton soil with lime iron ore tailingsadmixture. Transportation Geotechnics. ECGSevier, 10: 85 – 95.

Federal Ministry of Works and housing, (1997) “General Specifications For roads and bridges” Volume II, Federal highway department Lagos Nigeria. Pp: 145.

Frempong, E. M., and Yanful, E. K. Interaction between three tropical soils and municipal solid waste land_ll leachate. J. Geotech and Goenv. Engrg.ASCE, Vol. 134. No.3, 2008, pp 379{396

Gidigasu, M.D. Laterite Soil Engineering, ECGSevier Scientific Publishing Co. New York, 1976.

.

Gleason, M. H., Daniel, D. E., and Eykholt, G. R.Calcium and Sodium Bentonite for Hydraulic containment Applications, J. Geotech. And Geoenviron.Engrg., ASCE, Vol. 123, No. 5, 1997, pp 438{445

Grim, R., Clay Mineralogy. 2nd Ed. McGraw-Hill, New York, 1968.

Grim, R. and Guven, N. Bentonite: Geology, Mineralogy, Properties and Uses. ECGSevier Science Publishing, N.Y, 1978.

Howell, J.L., Sckelford, C. D., Amer, N. H., and Stern, R. T. Geotechnical testing journal, GTJODJ, Vol. 20.

Hamilton, R. (1964). Microscopic Studies of Lateritic Formation. A Jungerius (Editor), ECGSerier

Amsterdam. Pp: 269-278.

Heimdahl, T.C. and Drescher, A. (1999), Elastic Anisotropy of tire shreds, Journal of

Geotechnical and environmental engineering, ASCE, 125(5), 383-389.

Hirashima, K.B. 1979 Highway experience with thixotropic volcanic clay. Proe. High. Resistance Board, Vol. 28:481494.

Hough, B.K. (1989).Compressibility as the basis for soil bearing value. Journal of oil mechanics and foundation division, vol.85, No SM4, pp.11-39

Jiregna, D. (2008). Detail Investigation on Index Properties of Lateritic Soils. The Case of Nedjo-Mendi-Assosa. Pp: 5-14.

Lohnes, R.A. Fish, R.O and Demirel, T (1971). Geotechnical properties of selected Puerto Rican soiCGS in relation to climate and parent rock Bulletin of the Geological Society, America, 82:2617-2624.

Loughnan, F.C. (1969). Chemical Weathering of the Silicate minerals, American Elsevier Publ. Co., Inc., New York: 154 p.

Lohnes, R. A. Fish R O and Demirel, T. (1971).Geotechnical Properties of selected Puerto Rican soils in relation to climate and parent rock. Bulletin of the Geological Society,America,82:2617-2624

Mitchell, J. K. Fundamentals of Soil Behaviour. 2nd Ed. John Wiley & Sons, Inc., New York, 1993.

Maigien, R. Review of Research on Laterites. Natural Resources Research IV, United Nations Educational Scientific And Cultural Organisation, Paris, 1966.

Madu, R.M. (1976). An Investigation into the geotechnical and engineering properties of some laterites of Eastern Nigeria. Engineering Geology, 11: 101-125. https://doi.org/10.1016/0013-7952(77)90022-9.

Moses, G. Peter, O. F. O. and Osinubi, K. J. (2016) Desiccation-induced volumetric shrinkage of compacted metakaolin-treated black cotton soil for a hydraulic barriers system. Slovak Journal of Civil Engineering, Vol.24, 2016, No. 1, 1 – 5. DOI: 10.1515/sjce-2016-0001

Moses, G., Etim, R.K., Sani, J. E., Nwude, M. (2018) Desiccation effect of compacted tropical black clay treatedwith concrete waste. Leonardo Electronic Journal of Practices and Technologies, Issue 33: p. 69-88

Nazile Ural (September 12th 2018). The Importance of Clay in Geotechnical Engineering, Current Topics in the Utilization of Clay in Industrial and Medical Applications.

Nwankwoala, H.O. and Amadi, A.N. (2013), “Geotechnical Investigation of Sub-soil and Rock Characteristics in parts of ShiroroMuya-Chanchaga Area of Niger State, Nigeria. International Journal of Earth Sciences and Engineering, 6(1), 8 – 17.

Ogunsanwo, O. (1986). Basic Index Properties, Minerology and Microstructure of an Amphibiote derived Laterite Soils. Bulletin 1AEG, 33, 19-25.

Ogunsanwo, O. (1992). CBR and Shear Strengths of Compacted Lateritic Soils From Southwestern Nigeria Quarterly Journal, Engineering Geology Volume 22: 317-328.

Ogunsanwo, O. (1989). Some geotechnical properties of two Lateritic Soils compacted at different energies. Engineering Geology, 26,261-269 https://doi.org/10.1016/0013-7952(89)90013-6.

Ojo, G.P., Igbokwe, U.G., Nwozor, K.K. and Egbuachor, C.J. 2016.Geotechnical Properties of Lateritic Overburden Materials on theCharnockite and Gneiss Complexes in Ipele-Owo Area, Southwestern Nigeria American Journal of Engineering Research (AJER)
Vol. 5, Issue-9, pp-53-59.

Omotoso, (2010). An Investigation into the geotechnical and engineering properties of some laterites of Eastern Nigeria Journal of Nigeria Mining Geology and Metallurgical Society.C.C.

Oke, S,A.,Okeke, O.E., Amadi, A.N. & Onoduku, U.S. (2009), Geotechnical Properties of the Subsoil for Designing ShallowFoundation in some selected parts of Chanchaga area, Minna, Nigeria", Journal of Environmental Science, 1(1), 45 – 54.

Osinubi, K. J. and Nwaiwu, C. M. O. Design of Compacted Lateritic soils liners and cover. J. Geotech Geoenviron. Eng., ASCE, Vol., 132, No. 2, 2006, p203 {213.

Osinubi, K. J., Eberemu, A. O. and Amadi, A. A.Compatibility of compacted lateritic soil bagasse ash mixtures with municipal solid waste leachate. International Journal of Environment and Waste Management, Vol. 10, No. 4, 2012, pp 365{376.

Osinubi, K. J. and Amadi, A. A. Hydraulic performance of compacted lateritic soil bentonite mixture permeated with municipal solid waste land_ll leachate.Transportation Research Board (TRB) 88th Annualmeeting CD-ROM, 11-15th Jan., 2009, Washington DC, USA. Subject: Geology and Earth materials, session AF {40{Physico-chemical and Biological pro-
cesses in Soils committee, Paper # 09-0620, pp 1{18

Osinubi, K. J. and Nwaiwu, C. M. O. Desiccation induced shrinkage in compacted lateritic soil. Journal of Geotechnical and Geological Engineering, 2008, pp1513 {1529.

Osinubi, K. J., Eberemu, A. O. and Adzegah, B. Effect of Fines Content on the Engineering Properties of Reconstituted Lateritic Soils in Waste Containment Application Nigerian Journal of Technology, Vol. 31,No.3, 2012, pp 277{287

.Rowe, R. K.; Quigley, R. M. and Booker, J. R. ClayeBarrier Systems for Waste DisposaFacilities. E &FN Spon, London, UK, 1995.

Rahardjo, H, Aung , K.K, Leong, E.C and Rezam, R.B. (2004). Characteristics of Residual Soils in Singapore as Formed by Weathering. Journal of Engineering Geology, 73: 157-169.

Rowe, K. (2000). Geotechnical and Geo-environmental Engineering Handbook:Kluver Academic Publishers.

Shackelford C. D. and Nelson, J. D. Geoenvironmen-tal Design Considerations Tailings Dams. Proc. Int.Symp. On Seismic and Environmental Aspects of Dam Design: Earth, Concrete and Tailing Dams,Vol.1, Santiago, Chile, Oct. 14{18, 1996, pp 131{187.

Shackelford, C. D., Benson, C. H., Katsumi, T., Edil,T. B., Lin, L. Evaluating the Hydraulic Conductiv-ity of GCCGS Permeated with Non-Standard Liquids.Geotextiles and Geomembranes, Vol. 18, 2000, pp 137{161

APPENDICES

SPECIFIC GRAVITY TEST

Table A1. Specific Gravity Result for 100% COARSE GRAINED SOIL

Determinants	Trial 1	Trial 2
Wt of density bottle, W_1 (g).	24.87	26.17
Wt of bottle + dry soil, W_2 (g).	34.87	36.14
Wt of bottle + soil + water, W_3 (g).	84.97	84.92
Wt of bottle + water, W_4 (g).	79.36	79.34

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 100% CGS

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.87-24.87)}{(79.36-24.87)-(84.97-34.87)}$ **= 2.30**

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.14-26.17)}{(36.14-26.17)-(84.92-79.34)}$ **= 2.27**

SPECIFIC GRAVITY= $\frac{(GS1+GS2)}{2} = \frac{(2.30+2.27)}{2}$ **= 2.30**

Table A2. Specific Gravity Result for 90%CGS +10%QD

Determinants	Trial 1	Trial 2
Wt of density bottle, W_1 (g).	23.10	24.87
Wt of bottle + dry soil, W_2 (g).	33.10	34.87
Wt of bottle + soil + water, W_3 (g).	84.01	84.20
Wt of bottle + water, W_4 (g).	78.2	78.39

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 90% CGS +10%

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(33.10-23.10)}{(33.10-23.10)-(84.01-78.20)}$ **= 2.39**

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.87-24.87)}{(34.87-24.87)-(84,20-78.39)}$ **= 2.387**

SPECIFIC GRAVITY= $\frac{(GS1+GS2)}{2} = \frac{(2.39+2.387)}{2}$ **= 2.39**

Table A3. Specific Gravity Result for 80%CGS +20%QD

Determinants	**Trial 1**	**Trial 2**
Wt of density bottle, W_1 (g).	24.87	26.14
Wt of bottle + dry soil, W_2 (g).	34.80	36.14
Wt of bottle + soil + water, W_3 (g).	84.50	84.60
Wt of bottle + water, W_4 (g).	78.50	78.50

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 80% CGS +20%

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.80-24.87)}{(34.80-24.87)-(84.60-78.40)}$ **= 2.48**

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.14-26.14)}{(36.14-26.14)-(84,60-78.40)}$ **= 2.50**

SPECIFIC GRAVITY= $\frac{(GS1+GS2)}{2} = \frac{(2.48+2.50)}{2}$ **= 2.49**

Table A4. Specific Gravity Result for 70%CGS +30%QD

Determinants	**Trial 1**	**Trial 2**
Wt of density bottle, W_1 (g).	23.50	24.60
Wt of bottle + dry soil, W_2 (g).	33.50	34.60
Wt of bottle + soil + water, W_3 (g).	85.16	85.20
Wt of bottle + water, W_4 (g).	79.0	79.0

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 70% CGS +30%

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(33.50-23.50)}{(33.50-23.50)-(85.16-79.0)}$ **= 2.60**

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.60-24.60)}{(34.60-24.60)-(85.20-79.0)}$ **= 2.63**

SPECIFIC GRAVITY= $\frac{(GS1+GS2)}{2} = \frac{(2.60+2.63)}{2}$ **= 2.60**

Table A5. Specific Gravity Result for 90%CGS +10%RHA

Determinants	**Trial 1**	**Trial 2**
Wt of density bottle, W_1 (g).	26.14	24.87
Wt of bottle + dry soil, W_2 (g).	36.15	34.87
Wt of bottle + soil + water, W_3 (g).	80.0	80.10
Wt of bottle + water, W_4 (g).	74.86	74.89

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 90% CGS +10%RHA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.15-26.15)}{(36.15-26.15)-(80.50-74.86)}$ **= 2.05**

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.87-24.87)}{(34.87-24.87)-(80.10-74.89)}$ **= 2.09**

SPECIFIC GRAVITY= $\frac{(GS1+GS2)}{2} = \frac{(2.05+2.09)}{2}$ **= 2.05**

Table A6. Specific Gravity Result for 80%CGS +20%RHA

Determinants	**Trial 1**	**Trial 2**
Wt of density bottle, W_1 (g).	26.14	24.87
Wt of bottle + dry soil, W_2 (g).	36.15	34.87
Wt of bottle + soil + water, W_3 (g).	82.00	82.10
Wt of bottle + water, W_4 (g).	77.51	77.61

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 80% CGS +20%RHA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.15-26.15)}{(36.15-26.15)-(82.00-77.51)}$ **= 1.814**

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.87-24.87)}{(34.87-24.87)-(82.10-77.61)}$ **= 1.80**

SPECIFIC GRAVITY= $\frac{(GS1+GS2)}{2} = \frac{(1.814+1.80)}{2}$ **= 1.81**

Table A7. Specific Gravity Result for 70%CGS +30%RHA

Determinants	**Trial 1**	**Trial 2**
Wt of density bottle, W_1 (g).	26.14	24.87
Wt of bottle + dry soil, W_2 (g).	36.15	34.87
Wt of bottle + soil + water, W_3 (g).	83.00	82.50
Wt of bottle + water, W_4 (g).	79.34	78.84

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 70% CGS +30%RHA

Trial 1 (G_{S1}) $= \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.15-26.15)}{(36.15-26.15)-(83.00-79.34)} = \mathbf{1.58}$

Trial 2 (G_{S2}) $= \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.87-24.87)}{(34.87-24.87)-(82.50-78.84)} = \mathbf{1.57}$

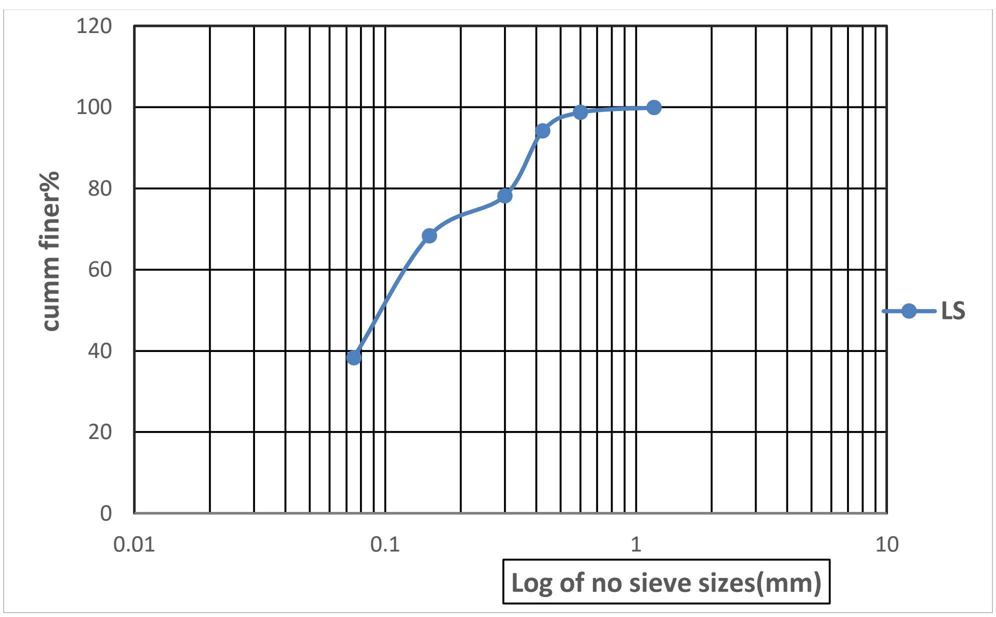
SPECIFIC GRAVITY$= \frac{(GS1+GS2)}{2} = \frac{(1.58+1.57)}{2} = \mathbf{1.57}$

APPENDIX B

SIEVE ANALYSIS RESULT

Table B1 Sieve Analysis Result for the coarse grained soil (CGS)

Sieve Sizes (mm)	Mass Ret (g)	% Mass Retained	Cum % Retained	Cum % Finer
2	0.58	0.1169	0.1169	99.88
1.18	5.77	1.154	1.2709	98.70
0.6	22.895	4.579	5.8499	94.15
0.425	80.0	16.00	21.849	78.151
0.3	49.20	9.84	31.689	68.311
0.15	150	30.0	61.689	38.311
0.075	63.3	12.66	74.349	25.651
Tray	127.50	25.50	99.849	0.151
Total	500			



APPENDIX C

LIQUID LIMIT RESULT

Table C1 Liquid Limit Result for coarse grained soil sample (CGS) 100%

BLOWS	**11**	**30**	**42**	**17**
Wt of empty tin (g)	19.80	18.30	18.70	19.0
Wt of tin + wet soil (g)	42.50	34.30	38.20	36.9
Wt of wet soil (g)	22.70	16.00	19.50	17.90
Wt of tin +dry soil (g)	38.0	31.90	35.30	33.80
Wt of dry soil (g)	25	13.60	16.60	14.80
Wt of water (g)	2.30	2.40	2.90	31.1
Moisture Content (g)	24.70	17.70	17.50	21.0

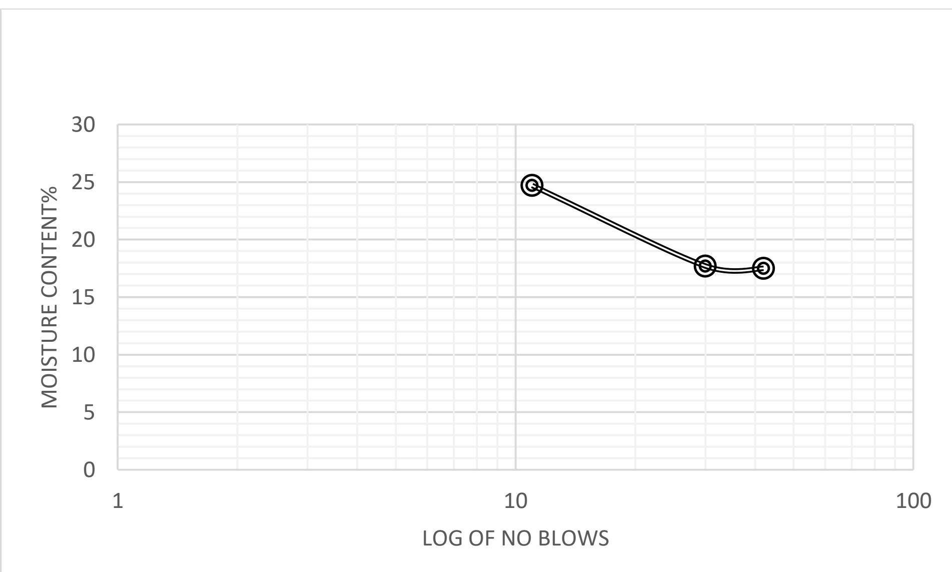


Table C1 Liquid Limit Result for coarse grained soil sample (CGS) 90%+10%QD

BLOWS	13	18	29	44
Wt of empty tin (g)	17.00	16.00	15.00	17.50
Wt of tin + wet soil (g)	29.0	28.50	27.0	29.30
Wt of wet soil (g)	12	12.50	12.00	11.80
Wt of tin +dry soil (g)	26.70	26.50	25.0	27.50
Wt of dry soil (g)	9.70	7.5	10	10.00
Wt of water (g)	2.30	5.0	2.0	1.8
Moisture Content (g)	23.70	19.0	20.0	18.00

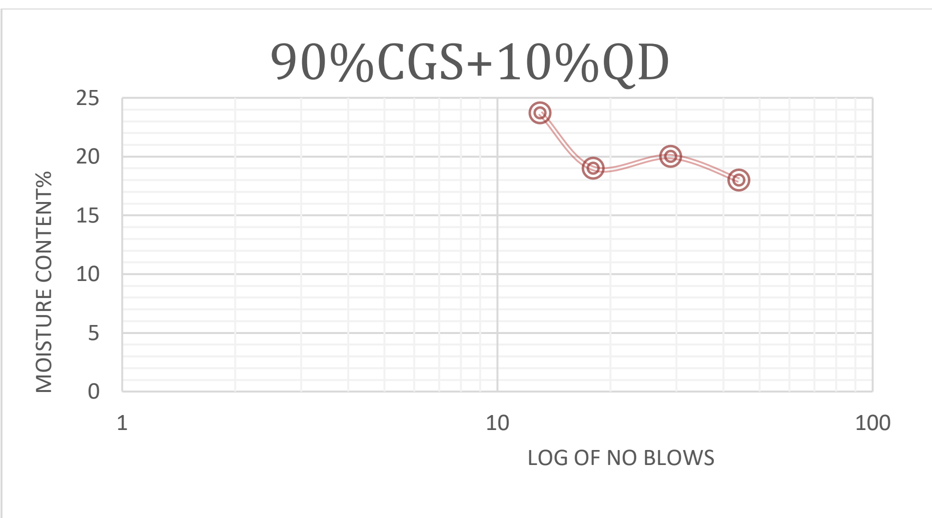


Table C3 Liquid Limit Result for coarse grained soi006C sample (CGS) 80%+20%QD

BLOWS	11	19	27	42
Wt of empty tin (g)	15.96	16.6	16.90	15.84
Wt of tin + wet soil (g)	28.0	28.60	29.0	27.60
Wt of wet soil (g)	12.04	12.50	12.00	11.80
Wt of tin +dry soil (g)	25.96	26.62	26.92	25.84
Wt of dry soil (g)	10.00	10.02	10	10
Wt of water (g)	2.04	1.98	2.08	1.76
Moisture Content (g)	20.40	19.80	20.80	17.60

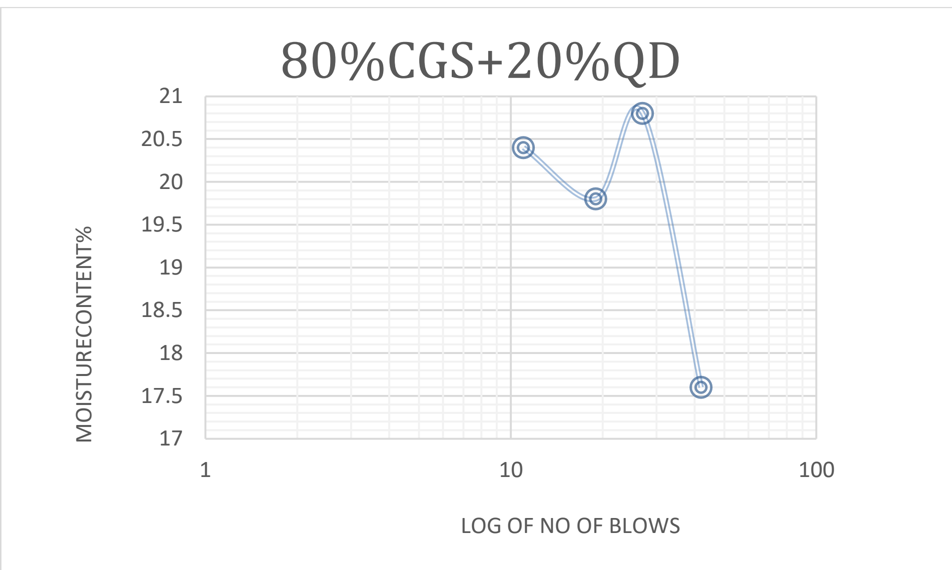


Table C3 Liquid Limit Result for coarse grained soil sample (CGS) 70%+30%QD

BLOWS	12	18	30	46
Wt of empty tin (g)	16.0	16.6	15.90	16.40
Wt of tin + wet soil (g)	28.8	30.60	28.0	29.50
Wt of wet soil (g)	12.80	14.00	12.10	13.10
Wt of tin +dry soil (g)	26.4	28.20	26.10	27.60
Wt of dry soil (g)	10.40	10.02	10.20	11.20
Wt of water (g)	2.31	2.070	1.850	1.70
Moisture Content (g)	23.10	20.7	18.50	17.00

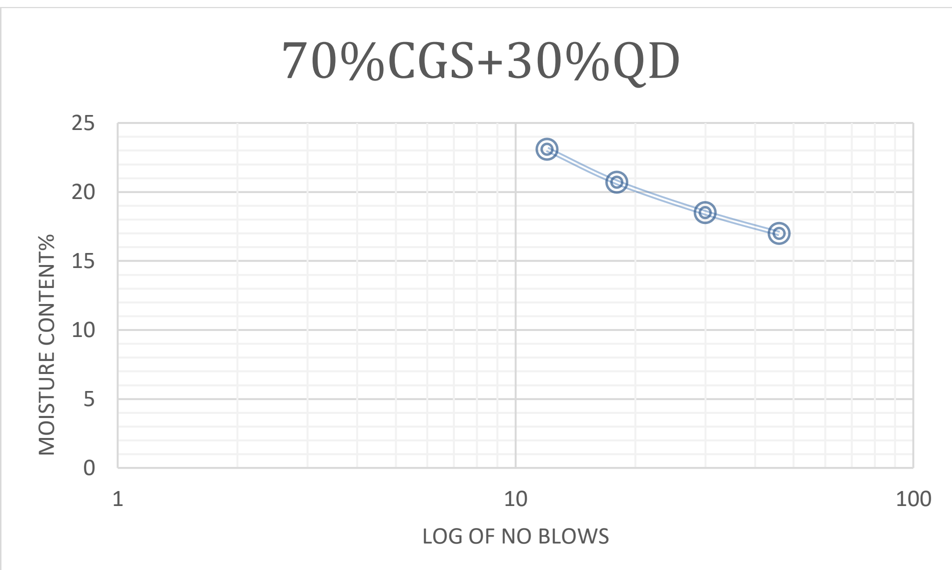


Table C4 Liquid Limit Result for coarse grained soil sample (CGS) 90%+10%RHA

BLOWS	25	17	20	10
Wt of empty tin (g)	16.6	15.45	15.2	16.92
Wt of tin + wet soil (g)	29.0	28.0	27.60	29.50
Wt of wet soil (g)	12.40	12.55	12.40	13.10
Wt of tin +dry soil (g)	26.60	25.45	25.22	27.60
Wt of dry soil (g)	10.40	10.00	10.02	11.20
Wt of water (g)	2.40	2.52	2.38	1.70
Moisture Content (g)	24.0	25.20	23.80	25.80

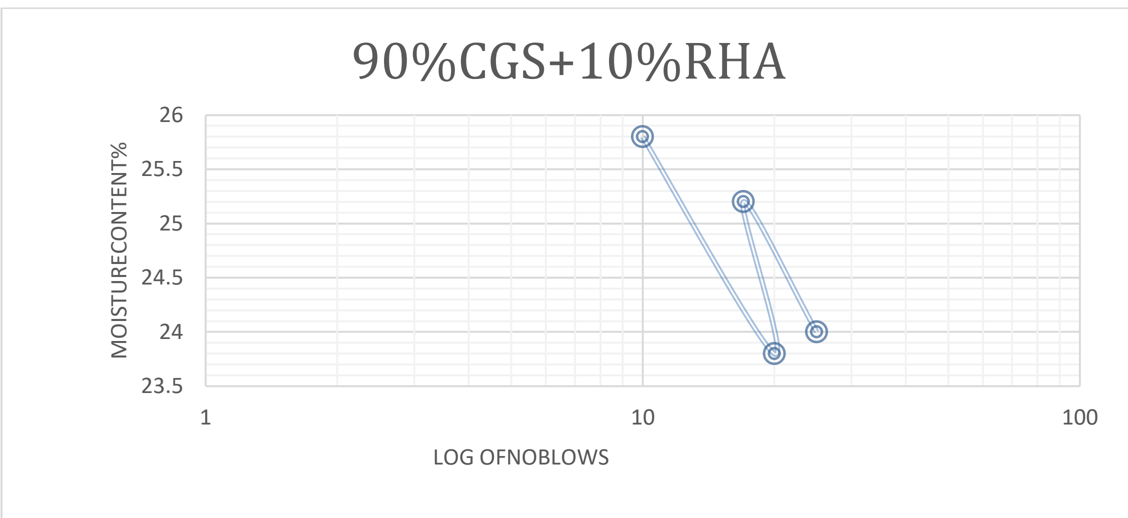


Table C5 Liquid Limit Result for coarse grained soil sample (CGS) 80%+20%RHA

BLOWS	17	11	21	30
Wt of empty tin (g)	15.1	16.02	15.76	16.70
Wt of tin + wet soil (g)	28.0	29.0	28.60	29.70
Wt of wet soil (g)	13.0	12.58	13.24	10.90
Wt of tin +dry soil (g)	25.10	26.02	25.76	26.66
Wt of dry soil (g)	10.0	10.00	10.10	9.96
Wt of water (g)	2.90	2.98	2.84	3.04
Moisture Content (g)	29.0	29.80	28.40	30.40

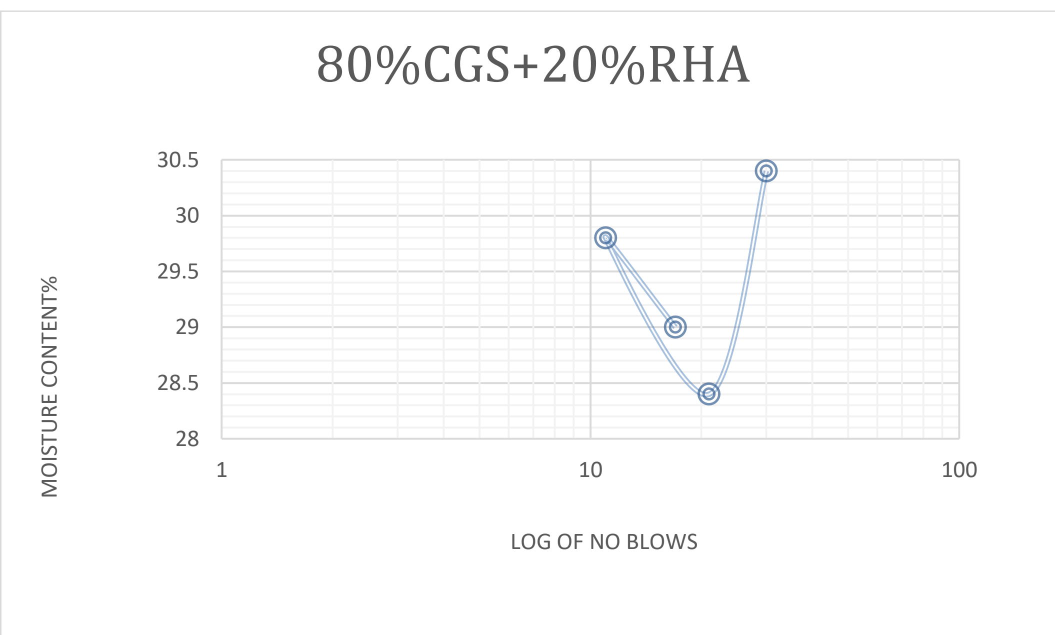
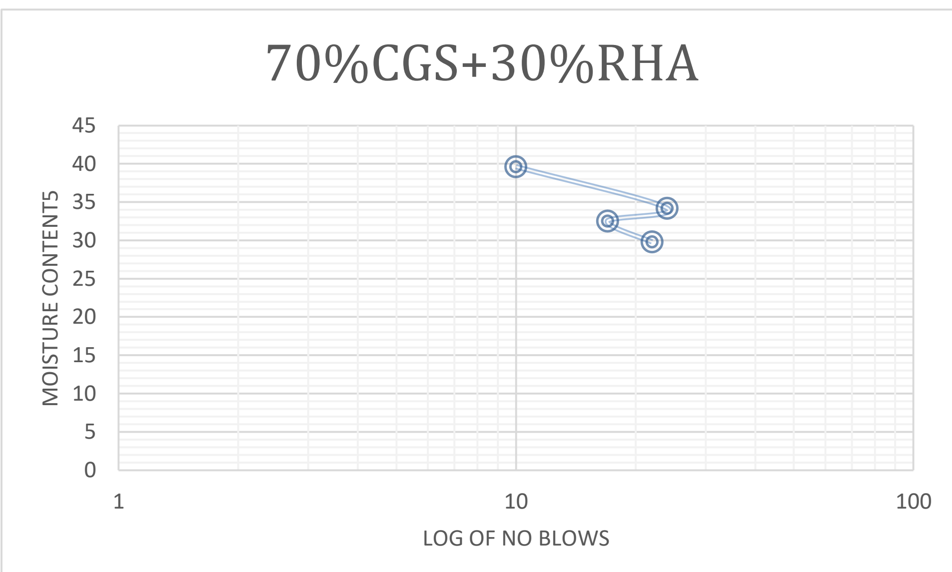


Table C6 Liquid Limit Result for coarse grained soil sample (CGS) 70%+30%RHA

BLOWS	10	24	17	22
Wt of empty tin (g)	16.10	15.17	16.00	16.50
Wt of tin + wet soil (g)	30.90	31.80	27.00	30.20
Wt of wet soil (g)		12.58	13.24	10.90
Wt of tin +dry soil (g)	26.70	27.70	24.30	26.90
Wt of dry soil (g)	10.60	12.53	9.30	10.40
Wt of water (g)	3.96	3.42	3.25	2.98
Moisture Content (g)	39.60	34.20	32.50	29.80



APPENDIX D

COMPACTION TEST

Table D1 Dry Unit Weight Determination for COARSE GRAINED SOIL (CGS) 100%.

Percentages of Water	**Vol of Mould**	**Wt of Mould**	**Wt of Mould + Wet Soil**	**Bulk Density**	**Moisture Content**	**Dry Unit Weight**
(%)	(m^3)	(kg)	(kg)	(N/m^3)	(%)	(kN/m^3)
4	0.001	4.20	5.95	1750	4.06	16.49
8	0.001	4.20	6.10	1900	10.10	17.00
12	0.001	4.20	6.30	2100	11.20	18.53
16	0.001	4.20	6.45	2250	15.60	19.09
20	0.001	4.20	6.35	2150	16.10	18.20

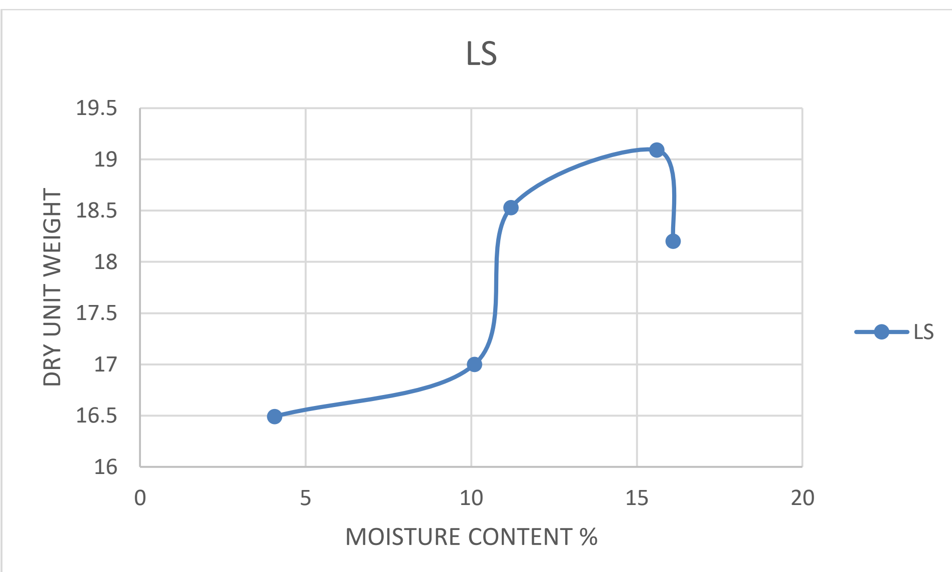


Table D2 Dry Unit Weight Determination for COARSE GRAINED SOIL (CGS) 90% +10 %(QD).

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(N/m^3)	(%)	(kN/m^3)
4	0.001	4.20	5.90	1750	5.64	16.60
8	0.001	4.20	6.00	1800	10.50	17.10
12	0.001	4.20	6.20	2000	11.10	18.60
16	0.001	4.20	6.30	2100	15.10	19.90
20	0.001	4.20	6.15	1950	15.50	17.10

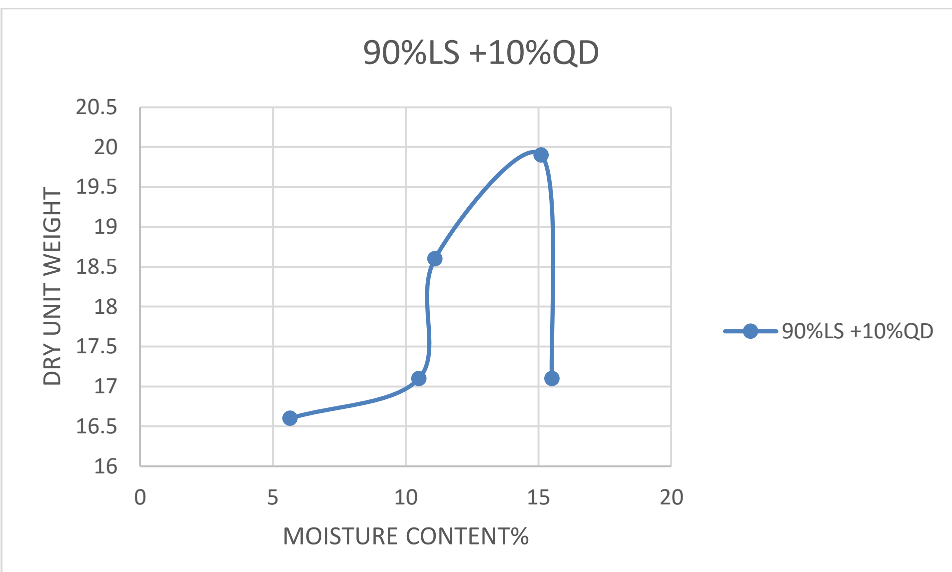


Table D3 Dry Unit Weight Determination for COARSE GRAINED SOIL (CGS) 80% +20 %(QD).

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(N/m^3)	(%)	(kN/m^3)
4	0.001	4.20	5.95	1750	6.40	16.80
8	0.001	4.20	6.00	1900	10.70	17.20
12	0.001	4.20	6.20	2000	11.50	18.70
16	0.001	4.20	6.30	2200	14.70	20.78
20	0.001	4.20	6.15	2000	15.50	18.50

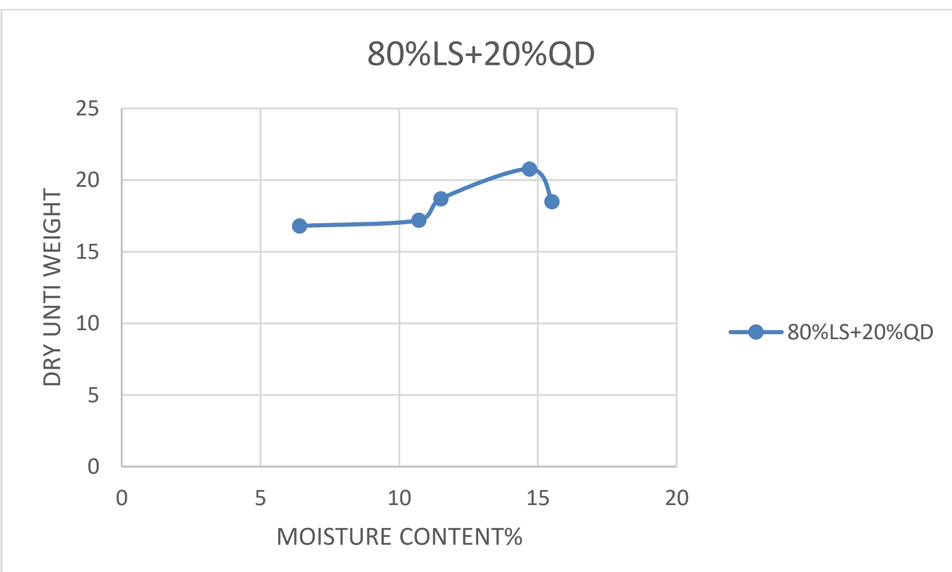


Table D4 Dry Unit Weight Determination for COARSE GRAINED SOIL (CGS) 70% +30 %(QD).

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(N/m^3)	(%)	(kN/m^3)
4	0.001	4.20	5.95	1800	5.64	16.50
8	0.001	4.20	6.00	1850	6.16	17.10
12	0.001	4.20	6.20	1900	8.38	19.10
16	0.001	4.20	6.30	2250	14.10	21.70
20	0.001	4.20	6.15	2200	15.40	20.01

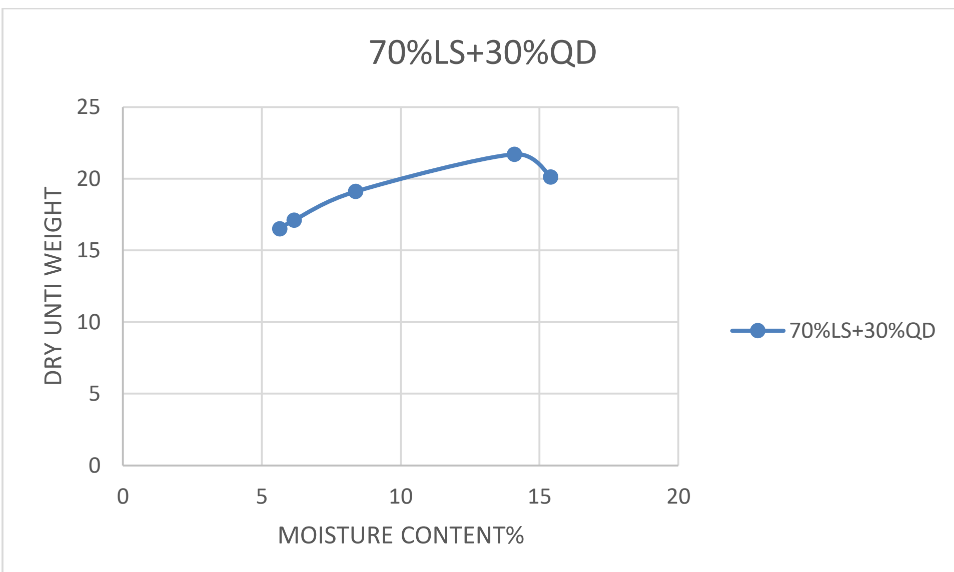


Table D5 Dry Unit Weight Determination for COARSE GRAINED SOIL (CGS) 90% +10 %(RHA).

Percentages of Water	**Vol of Mould**	**Wt of Mould**	**Wt of Mould + Wet Soil**	**Bulk Density**	**Moisture Content**	**Dry Unit Weight**
(%)	(m^3)	(kg)	(kg)	(N/m^3)	(%)	(kN/m^3)
4	0.001	4.20	5.60	1400	3.0	13.33
8	0.001	4.20	5.65	1450	6.0	13.42
12	0.001	4.20	5.75	1550	7.1	14.20
16	0.001	4.20	5.90	1700	9.25	15.26
20	0.001	4.20	6.00	1800	11.50	15.84
24	0.001	4.20	5.85	1650	14.20	14.17

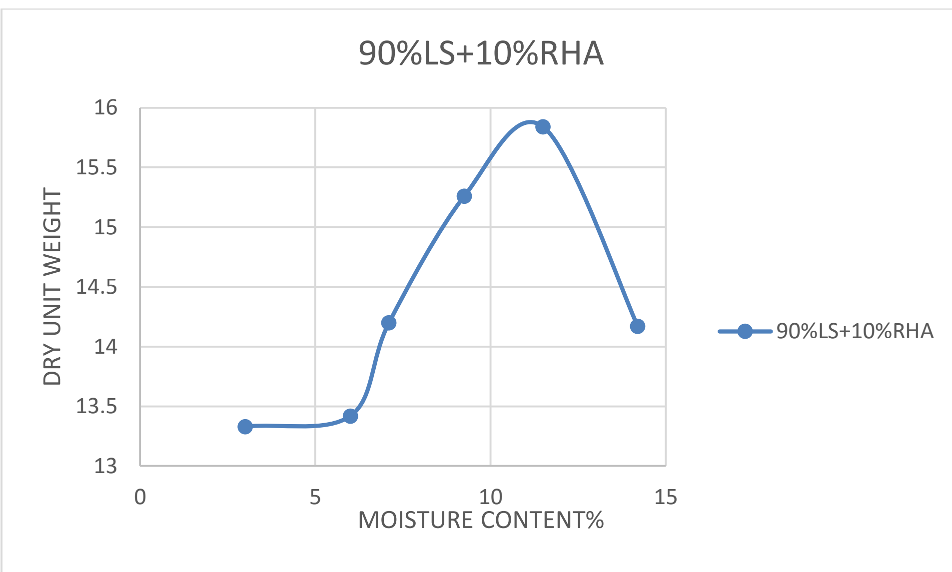


Table D6 Dry Unit Weight Determination for COARSE GRAINED SOIL (CGS) 80% +20 %(RHA).

Percentages of Water	**Vol of Mould**	**Wt of Mould**	**Wt of Mould + Wet Soil**	**Bulk Density**	**Moisture Content**	**Dry Unit Weight**
(%)	(m^3)	(kg)	(kg)	(N/m^3)	(%)	(kN/m^3)
4	0.001	4.20	5.60	1300	2.90	10.50
8	0.001	4.20	5.65	1350	5.80	11.00
12	0.001	4.20	5.75	1450	6.80	11.50
16	0.001	4.20	5.90	1500	9.0	12.70
20	0.001	4.20	5.30	1400	11.00	11.30

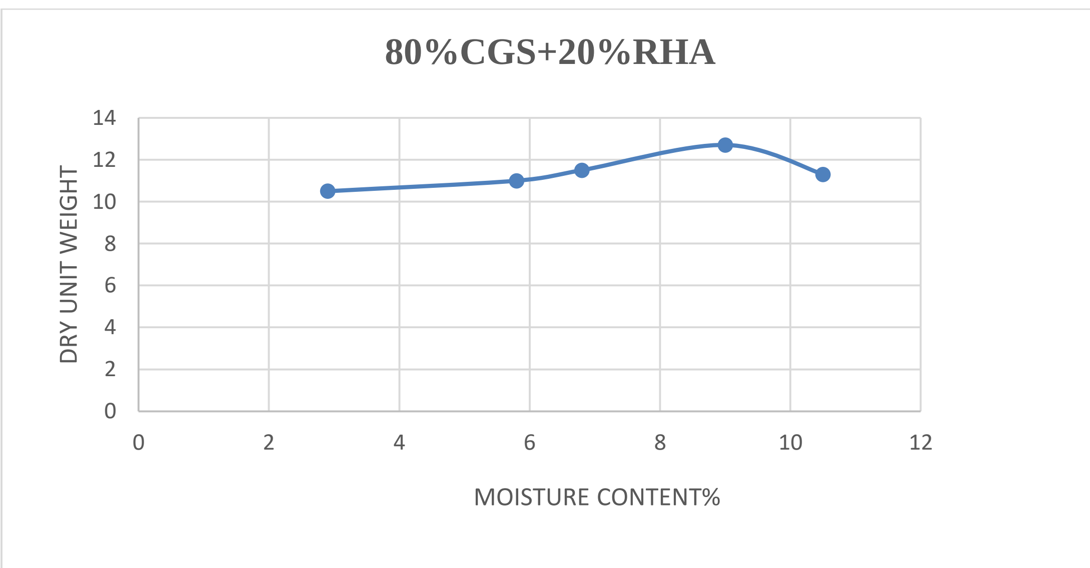


Table D7 Dry Unit Weight Determination for COARSE GRAINED SOIL (CGS) 70% +30 %(RHA).

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(N/m^3)	(%)	(kN/m^3)
4	0.001	4.20	5.60	1250	3.2	9.50
8	0.001	4.20	5.65	1300	6.50	10.50
12	0.001	4.20	5.75	1350	7.0	11.00
16	0.001	4.20	5.80	1400	7.4	11.10
20	0.001	4.20	5.35	1450	11.00	10.50