STRENGTH CHARACTERISTICS OF LATERITIC SOIL- ROCK FLOUR MIXTURES STABILISED WITH LIME FOR ROAD PAVEMENT APPLICATION

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MEng/SIPET/2018/7873

DEPARTMENT OF CIVIL ENGINEERING

FEDERAL UNIVERSITY OF TECHNOLOGY, MINNA

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A THESIS SUBMITTED TO THE POSTGRADUATE SCHOOL, FEDERAL UNIVERSITY OF TECHNOLOGY, MINNA, NIGERIA IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE OF MASTER OF ENGINEERING IN CIVIL ENGINEERING (GEOTECHNICAL ENGINEERING)

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ABSTRACT

The study evaluated the strength characteristics of lateritic soil-rock flour mixtures stabilized with 0,3,6,9 and 12% soil- rock flour ratio with lime ratio of 0, 2.5, 5,7.5 and 9%, compacted at optimum moisture content (OMC) using British Standard Light (BSL), West Africa Standard (WAS) and British Standard Heavy (BSH) compactive efforts. Results showed that the lateritic soil sample used for this study is classified as A-7-6 according to American Association of State Highway and Transportation Officials (AASHTO). The formulated mixtures from the A-7-6 soil and rock flour with lime showed an improvement in the index properties of the stabilized soil. The results showed a decrease in Liquid limits, Plastic limits and plasticity index respectively with increase in rock flour and lime content. The results also showed a slight decrease in maximum dry density (MDD) and increased optimum moisture content (OMC). Addition of rock flour and lime to the natural soil shows improvement in the CBR and Durability values of the stabilized specimen. BSH gave the highest value of 38.97% at 12% RF and 10% lime for CBR, while BSH gave the highest value of 855.94 kN/m^2 at 12% RF and 10% lime for durability. Furthermore, the unconfined compressive strength (UCS) of the sample were greatly improved with the addition of rock flour and lime at various compaction energies, The results showed that the unconfined compressive strength increased with increasing lime and rock flour content from 132.57 kN/m^2 at 0 % to 497.53 kN/m^2 for 12% rock flour and 10% lime content when compacted with British Standard light compaction effort, 850.04 kN/m^2 at 0 % to 1,176.62 kN/m^2 for 12% rock flour and 10% lime content for West Africa Standard compaction effort and 740.27 kN/m^2 for 0 % to 1,129.56 kN/m^2 for 12% rock flour and 10% at British standard heavy compaction effort. This therefore indicates overall, that the addition of rock flour and lime (12%RF and 10% lime) to poor/weak lateritic soil improved the soil strength and resistance to loss in strength. This study has established the potentials of using rock flour and lime to stabilize a class of lateritic soil for pavement construction purposes.

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CHAPTER ONE

1.0 INTRODUCTION

1.1 Background of the Study

Lateritic soils are sustainable road construction materials and are described as materials that meet the needs of the present generation without compromising the ability of future generations to meet their own needs satisfactorily (Oluremi *et al.*, 2012). The high cost of construction projects led to a call for the integration of laterite in the past and recent projects. Road constructed of earth materials are the most common and affordable, since earth materials are readily available almost anywhere on the planet. Laterite is a group of highly weathered soils formed by the concentration of hydrated oxides of iron and aluminum (Ola, 1983). Other definitions have used the ratio of silica (SiO_2) and sesquioxides ($Fe_2O_3 + Al_2O_3$) where the ratios are less than 1.33 is lateritic soil. Lateritic soil has been the most widely known and used construction material in building and road construction. In tropical parts of the world, lateritic soils are used as a road making material and they form the subgrade of most tropical roads. They are used as sub-base and bases for low-cost roads and these carry low to medium traffic (Olugbenga *et al.*, 2011). Furthermore, in rural areas of Nigeria, they are used as building material for molding of blocks and plastering (Onyelowe and Okafor, 2016).

Stabilization of soil is the process of changing one or more soil properties through mechanical or chemical means, to produce soil with improved and desired engineering properties. According to the American Society for Testing and Materials (ASTM, 1992), the main purpose of soil stabilization includes increasing the strength of an existing soil to enhance its load bearing capacity, permeability improvement and enhancement of soil resistance to the process of weathering and traffic usage among others. Soil stabilization achieves a number of objectives that are important in obtaining a long-lasting structure from locally available earth materials, including better mechanical characteristics; better cohesion between particles which reduces the porosity and changes in volume due to moisture fluctuations; and improved resistance to rain, wind, and erosion. Soil stabilization techniques include mechanical, physical and chemical stabilization. Laterite soil consists of high plastic clay; the plasticity of soil may cause cracks and damage on building foundations, pavement, highway or any other construction projects. It is therefore important, to understand the behavior of laterite soil and thus figure out the method of soil stabilization.

1.1.1 Rock flour as stabilizing agent

Rock flour, also known as stone dust, is generated during processing of coarse aggregates from rock at rock crushing plants and is available as waste material; it can also be obtained during bore hole drilling. The rock flour is a granular material like sand with a larger amount of angular particle. Rock flour is a stable material under varying moisture conditions since it contains the rock minerals such as quartz, feldspar and silica (Satyanarayana, 2016). At present, rock flour is used in basement fining of buildings, mechanical stabilization of subbase and base courses and to improve roughness of bituminous surface course (Hussaini and Perry, 2021).

1.1.2 Lime stabilization

Lime stabilization is one of the oldest process of improving the engineering properties of soils and can be used for stabilizing both base and sub base materials (Garber and Hoel, 2000). The addition of lime to reactive fine-grained soils has beneficial effects on their engineering properties, including reduction in plasticity and swelling potential, improved workability, increased strength and stiffness, and enhanced durability. In addition, lime has been used to improve the strength and stiffness properties of unbound base and sub base materials. Lime can be used to treat soils to varying degrees, depending upon the objective.

The least amount of treatment is used to dry and temporarily modify soils. Such treatment produces a working platform for construction or temporary roads. A greater percentage of treatment--supported by testing, design, and proper construction techniques—produces permanent structural stabilization of soils.

Generally, the oxides and hydroxides of calcium and magnesium are considered as _lime‘, but the materials commonly used for lime stabilization are calcium hydroxide ($Ca(OH)_2$) and dolomite ($Ca(OH)_2 + MgO$) (Garber and Hoel, 2000). Calcium hydroxide (hydrated lime) is a fine, dry powder formed by _slaking‘ quicklime (calcium oxide, CaO) with water; quicklime is produced by heating natural limestone (calcium carbonate, $Ca(CO)_3$) in a kiln until carbon dioxide is driven out. Quicklime is also an effective stabilizer used but not usually used for stabilization because it is caustic hence dangerous to handle, susceptible to moisture uptake in storage, and gives off much heat during hydration.

Dolomite used as stabilizing agent contains not more than 36 % by weight of magnesium oxide (Garber and Hoel, 2000), (MgO). The percentage of lime used for any project depends on the type of soil being stabilized.

The determination of the quantity of lime is usually based on an analysis of the effect that different lime percentages have on the reduction of plasticity and the increase in strength of the soil. Lime is used extensively to change the engineering properties of fine-grained soils and the fine-grained fractions of more granular soils. It is most effective in treating plastic clays capable of holding large amounts of water. The particles of such clays have highly negative-charged surfaces that attract free cations (i.e. positively charged ions) and water dipoles. The addition of lime to a fine-grained soil in the presence of water initiates several reactions. The two primary reactions, cation exchange and flocculation agglomeration, take place rapidly and produce immediate improvements in soil plasticity, workability, uncured strength, and load-deformation properties.

The effects of lime treatment or stabilization on pertinent soil properties can be classified as immediate and long-term. Immediate modification effects are achieved without curing and are of interest primarily during the construction stage. They are attributed to the cation exchange and flocculation–agglomeration reactions that take place when lime is mixed with the soil. Long-term stabilization effects take place during and after curing, and are important from a strength and durability standpoint. While these effects are generated to an extent by cation exchange and flocculation–agglomeration, they are primarily the result of pozzolanic strength gain (Moses, 2006).

1.2 Statements of Research Problem

Previous studies revealed that unsuitable materials with both poor physical and geotechnical properties are frequently encountered in road construction sites around the world. Hence, the need to improve their properties to make them acceptable for construction purpose (Jho *et al*, 2020). The lack of consistent data on most Nigerian lateritic soil reduces their efficient application especially as pavement material (Adeyeri, 1996).

No definite research has been carried out on the durability of lateritic soils- rock flour mixtures stabilized with lime. Previous researches showed that the mixtures of soil and rock flour attain a degree of strength when combined and compacted, but failed in durability due to lack of cementitious reaction, therefore addition of lime to soil-rock mixtures will provide cementitious reaction that will enable durability (Amadi, 2015).

1.3 Aim and Objectives

1.3.1 Aim

The aim of this study was to evaluate the strength characteristics of lateritic soil-rock flour mixtures stabilized with lime for road pavement application.

1.3.2 Objectives

The objectives of the study were to;

i. Determine the physical properties of natural and lime stabilized lateritic soil-rock flour mixtures
ii. Determine the engineering properties of natural and lime stabilized lateritic soil-rock flour mixtures
iii. Determine the durability characteristics of lime stabilized lateritic soil-rock flour mixtures.

1.4 Justification for the Study

The assessment of strength characteristics of lateritic soil- rock flour mixtures stabilized with lime serve as alternative materials for road construction project and effective use of locally available soil as well as the waste (rock flour). It is therefore expected that the data generated will be of significant importance to highway engineers in road construction and for further research work.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Formation of Laterite

Laterite is a soil and rock type rich in iron and aluminum commonly formed in hot and wet tropical areas. Almost all laterites are of rusty-red coloration because of the high iron oxide content. They are referred to as a soil type as well as being a rock type. Laterites are formed from the leaching of parent sedimentary rocks, metamorphic rocks and igneous rocks which leaves the more insoluble ions of mainly iron and aluminum (Mitchel and Hooper, 1961). Ola (1983) prefers to define a laterite as a rock or part of a soil, not a true soil. The mineralogical and chemical compositions of laterites are dependent on their parent rocks. The mechanism of leaching involves acid dissolving the host mineral lattice, followed by hydrolysis and precipitation of insoluble oxides and sulfates of iron, aluminum and silica under high-temperature conditions (Portelinha *et al,* 2012). The above processes usually produce yellow, brown, red or purple materials, with red being the predominant color. While tropical weathering in oxidizing conditions generally leads to reddening, this does not necessarily produce a lateritic material–hence the widespread confusion concerning laterite and its behavior. Geology of Nigeria by Kogbe (1975) described laterites to consist of three layers, a basal lateritic clay, a middle laterite gravel and a surface crust. Hence, types of laterites are as follows:

(i) Laterite crust: This has a cellular texture and is usually hard to break with a geologists‘ hammer. Light explosives may be required to excavate this type of laterite. It is commonly found on top of flat-topped hills or as boulders on slope surfaces and often is encountered while digging building foundations.

(ii) Laterite gravel: Laterite gravel may be found below a layer of laterite crust. At some locations, the gravel deposit is only covered by a thin layer of soil. Laterite gravel is usually pisolitic.

(iii) Laterite Clay: Laterite clay is often located below the gravel or the crust, and usually above the weathered basement. It has a very rich reddish-brown colour, with patches of pinkish white material (probably Kaolinite). Flakes of micas are visible in hand specimens. It is often used in the construction of earth dams.

Construction of roadways over soft subgrade is one of the most frequent problems for highway construction in many parts of the world (Antonia, 2016). Stabilization of soft subgrades with costly stronger materials like crushed rock is widely used, hence the need for cheaper alternative construction methods on soft subgrades (Cetin *et al*., 2010; Consoli *et al*., 2016; Quadri *et al*., 2019a; Quadri et al., 2019b). Clay stabilization using low-cost materials such as cement, lime, rice husk ash, cement kiln dust, calcined clay, steel slag or fly ash are better compared to crushed rock (Antonia, 2016). Steel slag, calcined clay, fly ash, rice husk ash amongst others are useful in many construction applications because they are pozzolanas (Quadri *et al*., 2019b).

2.1.1 Definition of lateritic soil

Tuncer and Lohnes (1977) classified laterite on the basis of silica- sequioxide ratio (SiO_2/ Al_2O_3 +Fe_2O_3) adopting the same limiting values as those proposed by Ola, (1974) using silica-alumina ratio (SiO_2 / Al_2O_3). Ratio less than 1.33 was considered indicative of true laterite, those between 1.33 and 2.00 of lateritic soil and those greater than 2.00 of non-lateritic tropically weathered soils. Bell (1993) also used a similar classification.

These controversies surrounding the definition of laterites further proves the complexity of its genesis, occurrence, texture and nature of the laterite which are primarily due to the properties of the parent rock and the weathering process. Alexander and Cady (1962) settled on a broader and concise definition of laterite soils. They defined laterite as a highly weathered material rich in secondary oxides of iron, aluminium or both, nearly void of bases and primary silicates but may contain large amounts of quartz and kaolinites; hard or capable of hardening on exposure to wetting and drying.

Ola (1975) used local terminology in defining laterite as all products of tropical weathering with reddish, brown colour with or without nodules or concretion and but not exclusively found below hardened ferruginous crust of hardpan. On the other hand, Osuola (1984) defined laterite, as a highly weathered tropical soil rich in secondary oxides of any or a combination of iron, aluminium and manganese.

Gidigasu (1976) reviewed all the available definitions of laterite and finally summarized the term laterite soils as ―all the residual and non-residual tropically weathered soils which genetically form a chain of materials ranging from decomposed rocks through clays to the sesquioxide rich concretionary rocks. This definition formed the basis most of where all laterite soils today are defined, described and identified.

Agbede and Joel, (2011) reported that the geotechnical characteristics and engineering behavior of red soils depend mainly on the genesis and degree of weathering (i.e. decomposition, laterisation, desiccation and hardening). Morphological characteristics as well as the type and content of secondary minerals are other genetic characteristic. The behaviour of laterite in pavement structure has been found to depend mainly on their particle size characteristics, the nature and strength of the gravel particles, the degree to which the soils have been compacted as well as the traffic and environmental conditions (Gidigasu, 1976).

The geotechnical characteristics and field performance of laterite are considerably influenced by the mode of formation (genesis), morphological characteristics, degree of weathering and the chemical and mineralogical composition, all of which can in turn be related to the weathering system determined by the joint effects of the pedogenic factors (parent materials, climate, vegetation, etc) (Dumbleton and Newill., 1962; Gidigasu, 1976). These factors also influence and are influenced by topography and drainage conditions so that soils having similar mineralogical and geotechnical characteristics can often be associated with particular topographical areas (Dumbleton and Newill, 1962).

2.2 Geotechnical Properties of Laterite Soils

Laterite soil is a product of tropical weathering with red, brown or dark brown colour, with or without nodules or concretions and generally (but not exclusively) found below hardened ferruginous crusts or hardpan (Ola, 1983). Laterite soils are known to be expansive depending on the quantity of halloysite and montmorillonite they contain. This property is also a factor of the composition of the parent rocks and on the laterization process (Satyanarayana, 2016).

The engineering properties of laterite soils are those properties relevant to the engineer and are only determined in the laboratory from the soil samples obtained on the field either by methods of disturbed or undisturbed sampling. They are used for the classification of the soils and subsequently for use (or otherwise) in engineering applications e.g. in foundation and highway construction. Gidigasu (1976) identified that the geotechnical properties of laterite soil are influenced by their genesis, degree of weathering, morphology, chemical and mineralogical composition and also on the environmental factors. These factors are responsible for their variability and inhomogeneity. The engineering properties of laterite soils are only determined in the laboratory (Gidigasu, 1976; Ola, 1983) and they are; particle size analysis and grading carried out via sieve analysis, Atterberg limits tests to determine the plasticity of clay soils, moisture-density relationship done via compaction, permeability tests to determine the ease or otherwise of water movements, the strength tests in service done via unconfined compression and triaxial for plastic soils, direct shear tests for cohesionless soils, consolidation properties to determine its response to modelled foundation load, the California bearing ration (CBR) tests.

The geotechnical properties of lateritic soils were determined from different soil samples by many researchers. Their results further confirmed the variations in properties of the soils being dependent on many factors including the climatic and environmental conditions. Ola 1983; Osinubi, 1998b; Ijimdiya *et al*, 2007; Amadi, 2010a,b; Eberemu *et al.,* 2013, have all done in-depth research using laterite soils collected from borrow pits in Shika, Zaria. The liquid limits of these soils were all around 40 %; plastic limits were a little in excess of 20 % having MDD above 1.7 Mg/m^3 corresponding to OMC of about 17.5 %, specific gravity between 2.55 and 2.8 and are generally predominantly clays with above 50 % passing through sieve No.200.

Okunlola *et al.* (2014) worked on laterite soils collected from borrow pits along Ogbomosho-Ibadan road in south-western Nigeria. Their results showed that the soils had average specific gravity of 2.72. Atterberg limits tests conducted showed the liquid limit and plastic limits were averagely 49 and 27 %, respectively. The soil had a Maximum Dry Density (MDD) of between 1.57 and 1.87 g/cm^3 corresponding to moisture content of 11 - 16.5 % with CBR values of 17-60 % soaked and 42-74 % unsoaked. Habeeb *et al.* (2012) also collected laterite from Oyo state and investigated the geotechnical properties of the soil for use as subgrade and base materials. 35 % of the soils passed BS No. 200 sieve. The liquid limit was found to be 48% and plastic limit 25 %. MDD was 1.9 Mg/m^3 and OMC 14.3 %. Specific gravity of the laterite was 2.65, CBR unsoaked was 78 %.

In the Niger Delta region of Nigeria, the geotechnical properties of laterite soil were investigated by Ugbe (2011). The soil was predominantly A-2 with fines ranging from 14 to 50 % having very low gravel contents (0 – 6 %) and thus not suitable as base course material. CBR value was reported to be between 3 and 43 % (soaked condition), MDD 1.7 to 2.14 Mg/m^3, OMC 7.7 % to 18 % and mean specific gravity of 2.62

2.3 Soil Stabilization

The increasing population and development of construction industry requires that geotechnical engineer possess sufficient knowledge and information about the methods of improving soils for use in various construction projects. Soil stabilization is the process of improving the physical and engineering properties of a soil to obtain some predetermined targets (Eisazadeh, 2010). This technique is done in order to render the material suitable and satisfactory for use as foundation or subgrade, subbase or base course material. The chief aim of stabilizing a soil is to improve the soil strength, bearing capacity and durability under adverse stress conditions i.e. stabilization is aimed at the enhancement of the engineering properties of deficient soils to enable them perform and sustain their intended engineering use (Yoder and Witczak, 1975; Gillott, 1987; Osinubi, 1995; Nicholas and Lester, 1999; Amu *et al*., 2011; Portelinha *et al.*, 2012). Soft, compressible soils also termed problem soils because of their ability to reduce in volume with applied pressure cannot carry loads satisfactorily and thus, need to be stabilized so that they can satisfy the purpose intended. Brook *et al.,* (2011) reported that the improvements in engineering properties caused by stabilization can include the following: increases in soil strength (shearing resistance), stiffness (resistance to deformation) and durability (wear resistance), reductions in swelling potential or dispersivity (tendency to deflocculate) of wet clay soils and other desirable characteristics, such as dust proofing and water proofing unsealed roads.

Soil modification essentially involves the improvement of the soil frictional characteristics and the reduction of its plasticity characteristics. This is distinct from soil stabilization, which is the improvement of the strength of the soil (Ovuarume, 2011). As with modification, many kinds of agents are used in stabilization. Soil modification/stabilization can also be defined as the improvement of the original soil properties to meet specific engineering requirements.

The industrially manufactured additives for modifying soils are lime, cement and bitumen. In view of increasing demand for safe and cost-effective engineering in modern technology, construction materials in their natural forms may not satisfy all technology engineering requirements, hence the necessity for modification of construction materials to enhance their purposes. This explains why effort is being directed to material conversion of industrial wastes and ―bio-wastes to engineering products and materials (Demers and Haile, 2003). One of the ways of achieving such optimum engineering is to use lime, cement, bitumen, and agricultural and industrial waste such as iron ore tailings, rice husk ash to stabilize soils such as lateritic soils which otherwise will be unworkable and unstable for engineering purposes in their natural form.

Ademila (2017) investigated the effect of rock flour on the geotechnical properties of lateritic soils. The results showed significant reduction in plasticity and linear shrinkage of the soil with increasing amount of rock flour. The strength characteristics (maximum dry density, optimum water content, CBR and shear strength) all increased with increasing rock flour content. This improvement in the geotechnical properties of the soils with rock flour shows that rock flour is a good stabilizing agent for weak soil.

Quadri *et al*., (2019a) stabilized expansive soil by calcium carbide waste (CCW) -fly ash columns and reported that a significant reduction in the swell potential and swell pressure was observed at 62% (CCW: FA=20:80) and 68% (CCW: FA=20:80) respectively. Akinwumi *et al*. (2019) investigated CCW as a stabilizer for tropical sand used as pavement material. It was observed that increasing application of CCW generally reduced the soil‘s specific gravity, plasticity index and maximum dry unit weight. The authors concluded that the soil became more workable and its strength properties were improved by stabilization with an optimal application of 4% CCW. The Subgrade characteristics of soil for use as earthwork materials for road construction were improved.

Horpibulsuk *et al*. (2013) worked on strength development in silty clay stabilized with CCW and fly ash (FA) and observed that the soaked and unsoaked strengths depended mainly on the CCW and FA contents. The authors added that most of the ratios of soaked strength to unsoaked strength varied between 0.45 and 0.65 and proved that a mixture of CCW and FA could be used for soil stabilization instead of ordinary Portland cement.

Research into new and innovative use of waste material is continually being advanced, particularly concerning the feasibility, environmental suitability and performance of the beneficial reuse of most waste materials. In order to make soil useful and meet foundation engineering design requirements, since the cost of procuring materials that meet specification requirement is increasingly becoming uneconomical, researches are being intensified with the aim of using admixtures/additives to reduce the cost of procuring cement and other modifying agents (Moses, 2006).

2.3.1 Methods of soil stabilization

The process of soil stabilization refers to changing the physical properties of soil in order to improve its strength, durability, or other qualities. Soil that has been stabilized will have a vastly improved load bearing capacity, and will also be significantly more resistant to being damaged by water, frost, or inclement conditions. Different types of soil stabilization have been used for thousands of years. They include mechanical, chemical, physical and polymer soil stabilization (Lemougna *et al*., 2011).

2.3.1.1 Chemical stabilization

Chemical solutions are one of the major types of soil stabilization. One method to improve expansive soils is chemical stabilization. Chemical stabilization includes the use of chemicals and emulsions as compaction aids to soils, as binders and water repellents, and as a means of modifying the behaviour of soil (Das, 2003). It involves deep mixing and grouting. Chemical stabilization can aid in dust control on roads and highways, particularly unpaved roads, in water erosion control, and in fixation and leaching control of waste and recycled materials. Portland cement, lime, asphalt, calcium chloride, sodium chloride, and paper mill wastes are common chemical stabilization agents. The effectiveness of these additives depends on the soil conditions, stabilizer properties, and type of construction (houses, roads). The selection of a particular additive depends on costs, benefits, availability, and practicality of its application. The behaviour of each of these admixtures differs vastly from the others; each has its particular use and conversely, each has its own limitations (Gidigasu, 1976). Chemical stabilization can be achieved via various combinations which include the following:

(a) Lime as a soil stabilizer

Lime has been used in the past in one form or the other to improve the engineering behavior of clayey soils. As a result of the proven success of lime stabilization in the field of highways and airfield pavements, it is being extended for deep in-situ treatment of laterite/clayey soils to improve their strength and reduce compressibility. The improvements in the properties of soil are attributed to the soil-lime reactions (Jung and Bobet, 2008; Ormsby and Kinter, 1973; Locat *et al*., 1996).

Hydrated lime is a fine powder, whereas quicklime is a more granular substance. Quicklime is more caustic than hydrated lime, so additional safety procedures are required with this material.

The type of lime used as a stabilizing agent varies from country to country. The most commonly used products are hydrated lime [Ca (OH)2], MgO, calcitic quicklime [CaO], and dolomitic quicklime CaO. MgO. Lime will primarily react with medium, moderately fine, and fine-grained soils to produce decreased elasticity, increased workability, reduced swell, and increased strength. The addition of lime increases the soil pH, which also increases the cation exchange capacity.

Consequently, even calcium-rich soils may respond to lime treatment with a reduction in the soil‘s plasticity. A reduction in plasticity is usually accompanied by reduced potential for shrinking or swelling. Stabilization occurs when the proper amount of lime is added to reactive soil. When introducing lime into soil for stabilization, Ca^{2+} is partly adsorbed on the surfaces of clay particles in replacement of monovalent cations such as Na^{+} and K^{+}. The amount of Ca^{2+} adsorbed depends on the cation exchange capacity of the treated soil. In fact, all the adsorbed cations are no longer available for pozzolanic reactions. The amount of lime required to satisfy the affinity of soil for lime is called the Lime Fixation Point (LFP). The lime in excess of the LFP is involved in the process of cementing. The reactions between the lime, silica and alumina-free, contributing to the formation of new minerals such as CSH (calcium silicate hydrates), CAH (calcium aluminate hydrates) and CASH (alumino-calcium silicate hydrates), are primarily responsible for the consolidation (Lemougna *et al.*, 2011). Lime is generally restricted to the warm to moderate climates since lime-stabilized soils are susceptible to breaking under freezing and thawing. Lime stabilization will result in the plasticity of the soil and an increase in the soil strength.

(b) Cement as a soil stabilizer

The mineralogy and granulometry of cement treated soils have little influence on the reaction since the cement powder contains in itself everything it needs to react and form cementitious products (Lemougna *et al*, 2011). The main reaction in a soil/cement mixture results from the hydration of the two anhydrous calcium silicates [3CaO. SiO_2 (C_3S)] and 2CaO. SiO_2 (C_2S), the major constituents of cement, which form two new compounds: calcium hydroxide (hydrated lime called portlandite) and CSH, the main binder of concrete. Cement will create physical links between particles, increasing the soil strength; meanwhile lime needs silica and alumina from clay particles to develop pozzolanic reactions (Amadi, 2010b). Cement stabilization mechanism is mainly controlled by hydrolysis and hydration. Cement stabilization usually results in decreased density, increased compressive strength, decreased plasticity, decreased volume, and change in characteristics of expansive clays when compared to the natural soil (PCA, 1992).

(c) Soil stabilization with waste

Calcium Carbide Waste (CCW) is another form of industrial waste being used by researchers to improve properties of expansive soils (Krammart and Tergtermisirikul, 2004; Du *et al.*, 2011; Quadri *et al.*, 2019a). It is a by-product obtained from the acetylene gas (C_2H_2) production process, as shown CaC_2 +$2H_2O$ C_2H_2 + Ca $(OH)_2$ (Quadri *et al*., 2019a, Quadri *et al*., 2019b). When CCW is mixed with certain pozzolans, which have high silicon dioxide (SiO_2) or aluminum oxide (Al_2O_3) content, it could yield pozzolanic reactions, resulting in final products that are similar to those obtained from the cement hydration process (Ormsby and Kinter, 1973).

(c) Mechanical stabilization

The most basic form of mechanical stabilization is compaction, which increases the performance of a natural material. Mechanical method of soil improvement by compaction is the densification of the soil by the application of mechanical energy (Gazdama and Osinubi, 2009). Mechanical stabilisation of a material is also achieved by adding a different material in order to improve the grading or decrease the plasticity of the original material. The physical properties of the original material will be changed, but no chemical reaction is involved. For example, a material rich in fines could be added to a material deficient in fines in order to produce a material nearer to an ideal particle size distribution curve. This will allow the level of density achieved by compaction to be increased and hence improve the stability of the material under traffic. The proportion of material added is usually from 10 to 50 per cent.

Provided suitable materials are found in the vicinity, mechanical stabilisation is usually the most cost-effective process for improving poorly-graded materials. This process is usually used to increase the strength of a poorly-graded granular material up to that of a well-graded granular material. The stiffness and strength will generally be lower than that achieved by chemical stabilisation and would often be insufficient for heavily trafficked pavements. It may also be necessary to add a stabilising agent to improve the final properties of the mixed material.

In the field, hand–operated vibrating plates and motorized vibratory rollers of various sizes are very efficient in compacting sand and gravely soils. Large falling weights have been used to dynamically compact loose granular fills. Fine-grained cohesive soils are compacted in the field by using common compaction equipments like; sheepsfoot rollers, rubber-tyred rollers. The objective of mechanical compaction is the improvement of the engineering properties of the soil mass which include a reduction in settlement due to reduced void ratio, Increase in soil strength and a Reduction in shrinkage (O_Flaherty, 1988).

(c) Admixture stabilization

Results reported by researcher (Ola, 1983; Balogun, 1991; Matawal and Tomarin, 1996) shows that the conventional stabilization of expansive soils with lime or cement or both are effective; however the cost of these stabilizers are high thereby making the process uneconomical. In various attempts to achieve an economically effective stabilization of deficient soil, many chemical/ agricultural or industrial additives have been mixed with lime or cement or both (Osinubi *et al*., 2009; Osinubi and Alhassan, 2008; Muazu and Osinubi , 2010; Osinubi *et al.,* 2009)

(d) Rock flour as an admixture for stabilization

Rock flour, also called stone dust, is generated during processing of coarse aggregates from rock at rock crushing plants and is available as waste material. The rock flour is a granular material like sand with a larger amount of angular particle. Rock flour is a stable material under varying moisture conditions since it contains the rock minerals such as quartz, feldspar and silica (Hussaini and Perry, 2021). The grain size distribution of rock flour and its angle of internal friction in wet condition indicate that it satisfies the requirernents of frictional fill for use in reinforced soil constructions (Sridharan and Singh, 1988). It can be also noticed from result that it has high values of internal frictional angles even under light compaction conditions. This is very much beneficial because the frictional characteristics are ensured even if the dry unit weight values reduce due to dilation effects. The minimum value of coefficient of permeability of rock flour in densest possible state ($k = 5.3x\ 10^{-5} m/s$) infers the free draining nature of the material.

The interfacial friction angle of rock flour with synthetic geotextiles presented also reflects the interaction of the material with the fabrics. The mobilized interfacial friction angles are about 85 to 90 percent of angle of internal friction of the rock flour, which is higher and reliable in comparison to sand. The strength envelopes of rock flour indicate uniform interfacial friction angle at all normal stress values. It may be attributed to the roughness and better interlocking of the rock flour particles. The CBR values of rock flour obtained for the rock flour specimens in the laboratory are high. Being an unbound material, it should not be directly used in design and construction of flexible pavements (Sridharan and Singh, 1988).

Ogunribido and Abiola, (2015), carried out a comparative study of cement and rock flour stabilization on the engineering properties of lateritic soil in Supare-Akoko, Southwestern Nigeria. This was carried out in order to determine the effects of additives as stabilizer on lateritic soil in road construction. The addtion of 2, 4, 6, 8 and 10% by weight of cement and rock flour for soil sample obtained showed continuous increase in the California Bearing Ratio. Cement additives shows higher percentage increase when compared with rock flour. Unconfined Compressive Strength decreased for the three soil samples with increase in the percentage of rock flour and the reverse is the case when cement was added. Also, the addition of cement showed increase in the Shear Strength of all the soil samples and decreased with the addition of equal percentage of rock flour. Unconfined Compressive Strength decreases for the three soil samples with increase in the percentage of rock flour and the reverse is the case when cement was added. Also, the addition of cement shows increase in the Shear Strength for all the soil samples and decreases with the addition of equal percentage of rock flour.

Ogunribido, (2012) also investigated the properties of samples of lateric soil from two locations along Igbatoro road in Southwestern Nigeria, the soil sample obtained were stabilized differently with varying quantities of rock flour from 2 to 10%. The objective of the study was to determine the effect of rock flour on some engineering properties of lateritic soil. This investigation includes evaluation of properties such as shrinkage limits, Atterberg limits, natural moisture contents, compaction, California bearing ratio and unconfined compressive strength of the soil with rock flour contents of 2, 4, 6, 8, and 10% by weight of the dry soil. The results obtained shows that the addition of rock flour improved the engineering properties of the soil. This investigation also confirmed that rock flour is an appropriate stabilizer with optimum amount of stabilizer needed as 4%.

CHAPTER THREE

3.0 MATERIALS AND METHODS

3.1 Materials Used

3.1.1 Materials

The study focused on collection of disturbed soil sample from borrow pit along Talba road in Minna metropolis, purchasing of lime from commercial dealer in Minna, while rock flour sample was collected from bore hole drilled beside National Examination Council (NECO) office in Minna, Nigeria.

3.2 Laboratory Tests

The following laboratory tests were conducted on the natural soil and soil mixtures constituted with rock flour and lime.

i. Moisture content
ii. Sieve analysis
iii. Atterberg limits
iv. Specific gravity
v. Compaction
vi. Unconfined Compressive Strength (UCS)
vii. California Bearing Ratio (CBR)
viii. Durability

All the tests were carried out in accordance with British standard code of practice (BS1377 (1990), BS1924 (1990), Methods of test for soils for civil engineering purposes. All the tests were carried out in the soil laboratory at the department of Civil Engineering, Federal University of Technology, Minna, Niger state, Ngeria

3.2.1 Natural moisture content determination

The oven drying method is the definitive procedure used in standard laboratory practice. The moisture content of the soil samples were determined according to (BS1377: 1990 Part 2:3)

The procedure is as follow:

The natural moisture content of the soil as obtained from site was determined in accordance with BS 1377(1990). Three weighing containers were cleaned and weighed to the nearest 0.001g as M_1 using an electronic weighing balance of 0.001g accuracy. The sample as freshly collected was crumbled and placed loosely in the containers and the container with the samples were weighed together to the nearest 0.001g as M_2. The containers were placed in the oven and dried at 105-110^{oc} for 24 hours. The containers and the sample were removed and weighed dry to the nearest 0.001g as M_3. The results for natural moisture content were as presented in Appendix A1.

The natural moisture content as collected from the site is calculated as the average of the three oven dried samples given

$$moisture\ content\ (\%) = \frac{weight\ of\ moisture}{weight\ of\ dry\ soil}\ x\ 100\%$$

(3.1a)

$$= \frac{(M_2 - M_3)}{(M_3 - M_1)} \text{ X } 100\ \%$$

(3.1b)

Where: M_1 = weight of container

M_2= weight of container + sample

M_3= weight of dry sample +container

3.2.2 Sieve analysis (particle size distribution) of natural soil

This is usually conducted to know the particle size distribution of the soil sample and also for the classification of soil. The size distribution is often of critical importance to the way the material performs in used and it will determine in accordance to (BS 1377: 1990 part 2:9). 200g of natural soil sample was weighed, wet sieved to remove clay and silt particles using BS No. 200 (0.075mm) sieve under tap water. Washing was done carefully to avoid damage to the sieves. After washing, the sample was dried in an oven set to 105^0C for 24hours. After drying the BS sieves was arranged in descending order of sieve size. The oven dried sample was transferred individually into the sieves and then shaken for at least 10 minutes manually. After sieving the mass retained on each sieve was weighed.

The results of the particle size distribution were as presented in Figure 4.1 and Appendix A4.

The percentages passing each sieve was calculated and plotted on a semi- log graph of percentage passing against sieve sizes using equation 3.2

$$percentage\ retained\ on\ any\ sieve = \frac{mass\ of\ soil\ retained}{total\ soil\ mass}\ x\ 100\%$$

(3.2)

3.2.3 Atterberg limits test

The Atterberg limit is a measure of the nature of a fine grained soil, depending on the moisture content of the soil. It was determined according to (BS 1377: 1990 part 2:4 & 2:5).

3.2.3.1 Liquid limit test (cone penetrometer method)

The liquid limit test was done with the cone penetrometer equipment and carried out in accordance to BS 1377 (1990) at the Federal University of Technology, Minna Civil Engineering Laboratory. The equipment and apparatus were drop- cone penetrometer, flat glass plate, metal cup, washing bottle containing distilled water, moisture content containers, spatulas, palette knives and electronic weighed balance. The test was done on the natural soil sample and soil–rock flour mixtures with lime at varying percentage proportions specified.

An air dried sample of 200g of the sample obtained after passing through the sieve of 0.425mm aperture was placed on the glass plate and mixed thoroughly with water with aid of the palette knives of spatulas to obtain a good paste consistency. A small portion of the mixture was set aside in a sealed nylon bag for plastic limit determination. The remaining soil paste was compacted into the penetrometer metal cup with a spatula to avoid void spaces and to level the soil to the top edge of the metal cup. The cone penetrometer was adjusted to 0.0mm reading.

The penetrometer metal cup filled with the compacted sample paste was placed beneath the cone. The knob was then adjusted until the tip of the cone is slightly in contact with the top of the leveled sample paste in the metal cup. Then, the knob was pressed for a few seconds, which aid in releasing the cone in order to penetrate the soil in the metal cup; the reading of the penetration was recorded. Some portion of the soil at the point of penetration was taken for moisture content determination. The remaining sample in the metal cup was then remixed with the addition of water on the glass plate until a uniform and softer consistency is achieved.

3.2.3.2 Plastic limits test

About 150g of the soil sample passing through sieve 425μm sieve aperture, prepared in the manner as in liquid limit test is used. The ball of the soil was then rolled between the hand and glass plate. The rolling continued until a thread of about 3mm in diameter was obtained, the thread is crumbled at this stage. The portion of the crumbled soil was then gathered and placed in a moisture can for moisture content determination.

Thus, moisture content was determined using Equation 3.3

$$PL = \frac{(M_2 - M_3)}{(M_3 - M_1)} \qquad (3.3)$$

3.2.3.3 Plasticity index

The plasticity index (PI) of the natural soil sample is the difference between the liquid limit (LL) of the soil sample and the corresponding plastic limit (PL). The plasticity index was calculated as:

PI= LL- PL

(3.4)

3.2.4 Particle density (specific gravity)

This is a property of the mineral material forming soil grains; it will be determined in accordance to (BS1377: 1990 part 2:8). The term particle density is used instead of the term specific gravity, which was used in previous editions of the British standard, to comply with current usage in other standards. In this standard particle density is quoted in Mg/m^3, which is numerically equal to the specific gravity.

Laboratory determination of particle density

The determination of particle density was carried out according to BS 1377 (1990) test (B) for fine- grained soils. The density bottle and the stopper were weighed to the nearest 0.001g as M_1. The air dried soil was transferred into the density bottle, and the bottle content and cover was weighed as M_2. Water was then added just enough to cover the sample and the solution was gently stirred to remove any air bubble. The bottle was then completely filled with water and covered. The covered bottle was wiped dry and the whole weighed to the nearest 0.001g as M_3. The bottle was subsequently emptied and filled completely with water, wiped dry and weighed to the nearest 0.001g as M_4. The specific gravity was calculated using Equation 3.5a and 3.5b

$$specific\ gravity\ (Gs) = \frac{mass\ of\ dry\ soil\ (M_s)}{mass\ of\ an\ equivalent\ volume\ of\ water\ \ (M_w)}$$
(3.5a)

$$G_s = \frac{M_s}{M_w} = \frac{(M_2 - M_1)}{(M_4 - M_1) - (M_3 - M_2)}$$
(3.5b)

where: G_s = specific gravity

M_1= mass of empty density bottle (g)

M_2= mass of density bottle + dry soil

M_3 = mass of density bottle +soil + water

M_4 = mass of density bottle filled with water

3.2.5 Compaction characteristics

Properly mixed laterite soil- rock flour (0, 3, 6, 9 and 12%) mixtures with lime (0, 2.5, 5, 7.5 and 10%) by weight were compacted at British standard light, West Africa standard and British Standard heavy (BSL, WAS, and BSH) compaction efforts, respectively. The sample was mixed with small amount of water and compacted into the mould. This test was used to determine the optimum moisture content and maximum dry density of the soil sample.

3.2.5.1 Maximum dry density

The compaction tests was carried out for the natural soil sample and the stabilized soil samples using different percentages of rock flour and lime in accordance to BS 1377 (1990) part 4. 3000g of air dried soil sample was weighed and mixed thoroughly with the varying percentage of additives (rock flour and lime) and water contents. For samples compacted using the British standard light (BSL) compaction effort, the admixed samples were compacted in three layers, with each layer rammed 27 blows using 2.5kg rammer dropping from a height of 304.8mm above the sample. The blows were distributed uniformly over the surface of each layer. The collar was then removed and the sample leveled to the brim of the mould with straight edge. The mould and the compacted admixed soil sample was weighed. Two representative samples (one from the top and other from the bottom) were taken from the compacted sample for the determination of moisture content.

The whole sample was removed from the mould and placed on a large mixing tray; 8-10% water equivalent added to the sample and mixed properly. The compaction was repeated using the same number of blows and layers as described above. Successive increments of water were added until when reduction in weight of the mould and the soil sample was noticed. At least two more compactions were carried out after the peak weight of the mould and the compacted sample was obtained. In the West Africa standard (WAS) compaction test, the same procedure was adopted but a 4.5kg rammer was used on 5 layers with each layer receiving 10 blows from a rammer dropping from a height of 450mm. On the other hand, British Standard heavy (BSH) compaction involved the same procedure adopted for WAS compaction except that each of the 5 layers received 27 blows. After determining the moisture content for every water increment, the results were plotted. Smooth curves were drawn through the resulting points and the positions of optimum moisture content and maximum dry density was determined on the graphs.

3.2.5.2 Optimum moisture content

The corresponding values of moisture contents at maximum dry densities (MDD) deduced from the graph of dry density against moisture contents gave the optimum moisture content (OMC).

3.2.6 Unconfined compression test

Unconfined compression test was determined according to (BS 1377:1990 part 7:7). The primary purpose of this test was to determine the unconfined compressive strength, which was used to calculate the unconsolidated undrained shear strength of the soil under unconfined conditions. The unconfined compressive strength (q_u) is defined as the compressive stress at which unconfined cylindrical specimen of soil will fail in a simple compression test.

3.2.7 Durability test

The durability assessment (under adverse field conditions) of the soil sample was determined by resistance to loss in strength when immersed in water. It was expressed as the ratio of UCS of the specimen wax-cured for 7 days and de-waxed top and bottom before being soaked for another 7 days to the UCS of the specimen cured for 14 days:

3.2.8 California bearing ratio (CBR) test

The California Bearing Ratio (CBR) test enables a measure of the strength of the natural lateritic soil and the stabilized sample. The CBR is expressed by the force exerted by the plunger against the depth of penetration into soil specimen. The tests were carried out in conformity with British Standard Institute, Methods of testing soils for civil engineering purposes, BS 1377, (1924) (1990), and the Nigerian General Specification (1997). The unsoaked California bearing ratio test was conducted, Soil samples were prepared by dynamic compaction method and placed on the bottom plate of the loading device. Predetermined weights of the soil sample were placed into the 2360 cm^3 mould and compacted at the optimum moisture content and at the three compactive efforts of BSL, WAS, and BSH. Since the weights of the soil samples were predetermined using the density – volume relationship, the soils was completely compacted into the CBR moulds. After the compaction, the base plates were removed and then the compacted specimens were placed into sealed plastic bags for curing. Plastic bags were used to avoid loss of moisture due to evaporation. The specimens were cured for seven (7) days before testing in accordance with the specification by the Nigerian General Specification (1997). The specimens were removed from the plastic bags and the base plates replaced, they were then transferred to the CBR testing machine. The plunger was then made to penetrate the prepared specimen at a uniform rate. The procedure was repeated for every successive increment in the concentration of the rock flour and lime. The CBR curves were plotted (i.e. force versus penetration of plunger) using the values obtained from the tests. The greatest value calculated for penetrations at 2.5mm and 5.0mm was recorded as the CBR**.** The summary of the CBR values are shown in Appendix C1.

$$\text{CBR} = \left(\frac{Load}{Standard\ force}\right) \times 100 \tag{3.7}$$

California bearing ratio CBR calculation

Proving ring constant 41.6N/division

Standard force at 2.5mm =13.24 kN

Standard force at 5.0mm =19.96 kN

Load = proving ring reading × proving ring constant

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

4.1 Physical Properties of Sampled Lateritic Soil

4.1.1 Natural Moisture Content

The result of the natural moisture content test shows that the soil has average natural moisture content of 10.28%. The role of natural moisture content in pavement materials (soil) is crucial in the increasing or reducing density indices (DI) of the soil. The natural moisture content of soil for use in construction of road is important in determining compaction and proportion of additional materials, (Mohammed, 2021)

4.1.2 Particle size distribution analysis

This was carried out to determine the fineness of the sample. To have an idea of the quality of fine particle contained in the sample. The result is as presented in Figure 4.1.

The particle size analysis of the lateritic soil from this location indicates silty-clay sand with 0.46% gravel fraction, 24.54% sand fraction, with 75% silty- clay fractions (passing through sieve BS No 200 sieve). The summary of the physical properties of the studied soil sample is presented in Table 4.1 and Figure 4.1

Table 4.1: Summary of physical properties of the studied soil sample

Property	Quantity
Percentage Passing	
BS No. 200 Sieve	75.00
Natural Moisture Content, %	10.28
Liquid Limit, %	50.55
Plastic Limit, %	27.33
Plasticity Index, %	23.22
Specific gravity	2.63
USCS Soil Classification	CH
AASHTO Classification	A-7-6

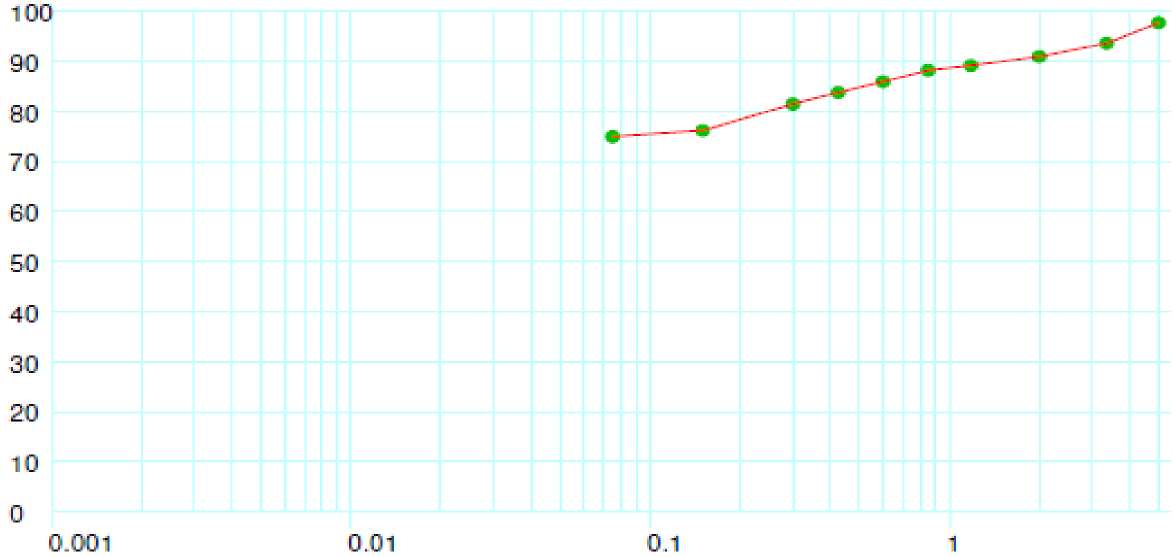

Figure 4.1: Particle size distribution curve of the studied soil sample

4.1.3. Specific gravity analysis of studied soil sample

The true specific gravity is actually the weighted average of the specific gravities of all the mineral particles present in the sample. The average value of 2.63 was obtained which shows that the specific gravity (G_S) of the sample is a poor material in road pavement construction (Oyediran and Durojaiye, 2011). Specific gravity is an important index property of soils that is closely linked with mineralogy or chemical composition and also reflects the history of weathering, Tuncer and Lohnes, (1977). It is relatively important as far as the qualitative behavior of the soil is concerned and useful in soil mineral classification, for example iron minerals have a larger value of specific gravity than silica, (Gush, 2021). It gives an idea about suitability of the soil as a construction material; higher value of specific gravity gives more strength for roads and foundations. It is also used in calculation of void ratio, porosity, degree of saturation and other soil parameters (Hadyra, 2021).

4.1.4 Mineralogy

In order to determine the mineralogy of the soil, x-ray diffraction analysis was used to identify the various minerals present in the natural soil sample. Figure 4.2 shows x-ray diffraction analysis on the fraction of natural soil passing the BS No. 200 sieve and the minerals present in the soil. The quartz (SiO_2) with red indicator has much higher presence in this sample with 2500 cps (count per second).

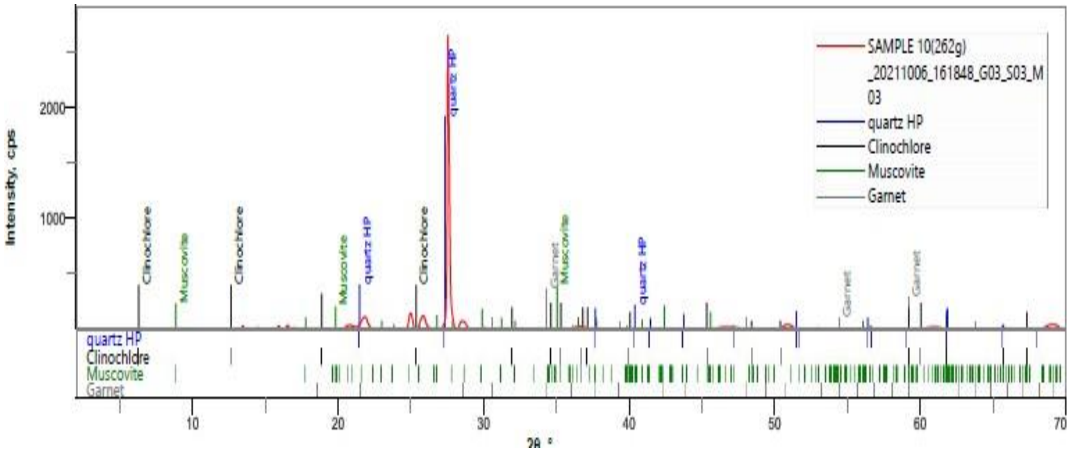

Figure 4.2: X-ray diffraction pattern for the ADPO sample

4.1.5 Consistency limits test analysis

Having Liquid Limit of 50.55%, Plastic Limit of 27.33% and Plasticity Index of 23.22%, the values of these parameters were decreased consistently with the addition of up to 12% RF and 10% lime to the studied soil. The Plasticity index decreased linearly from 23.22% (unmixed lateritic soil) to 8.42% (when mixed with 12% RF and 10% lime). The low value for the Liquid limit, Plastic limit and Plasticity Index is consistent with the values obtained by Oluyemi-Ayibiowu (2015) when he carried out laboratory research on the Tropical Clay obtained from Numan – Yola road in Adamawa state. Akinola (2021) shows that high plasticity often leads to high swelling potentials of the soil which has adverse effect on pavement service life. Reduction in plasticity index improves the workability of the soil, the greater the plasticity index, the more it is difficult to work on the soil, (Ojuri, 2021). In general, the plasticity index depends only on the amount of clay present. It indicates the fineness of the soil and its capacity to change shape without altering its volume. A high plasticity indicates excess clay or colloids in the soil. (Ojuri, 2021).

Atterberg limits for various rock flour and lime contents is presented in the Figure 4.3

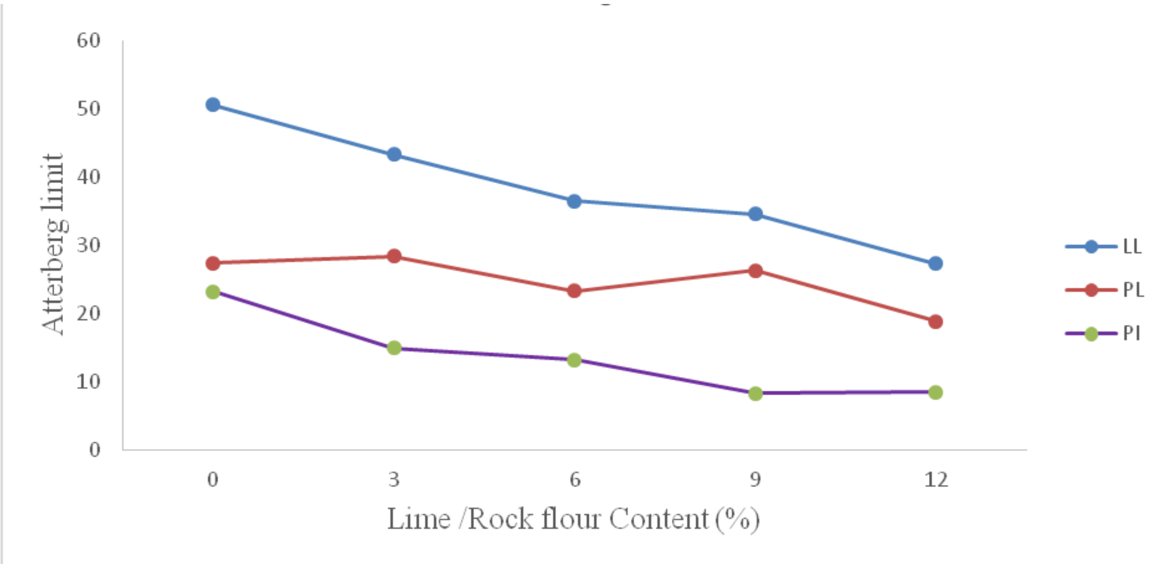

Figure 4.3: Variation in Atterberg limits with rock flour + lime content

4.1.6 Compaction characteristics

The MDD for the BSH compactive effort reduced from 2.02kN/m^3 (for natural lateritic soil) to 1.82kN/m^3 (when mixed with 6% rock flour and 5% lime), the BSL effort MDD also decreases from 1.61kN/m^3 (natural lateritic soil) to 1.51kN/m^3 (when mixed with 6% rock flour and 5% lime) and that of WAS effort MDD reduces from 1.96kN/m^3 (natural lateritic soil) to 1.92 kN/m^3 (when mixed with 6% rock flour and 5% lime). The OMC for the BSH effort increased from 13.35% (natural lateritic soil) 18.33% (when mixed with 6% rock flour and 5% lime), the BSL effort OMC increases from 19.45%

(natural lateritic soil) to 20.63% (when mixed with 12% rock flour and 10% lime) and that of WAS effort MDD increases from 12.27% (unmixed lateritic soil) to 16.55% (when mixed with 6% rock flour and 5% lime) as shown in Figure 4.4- 4.5

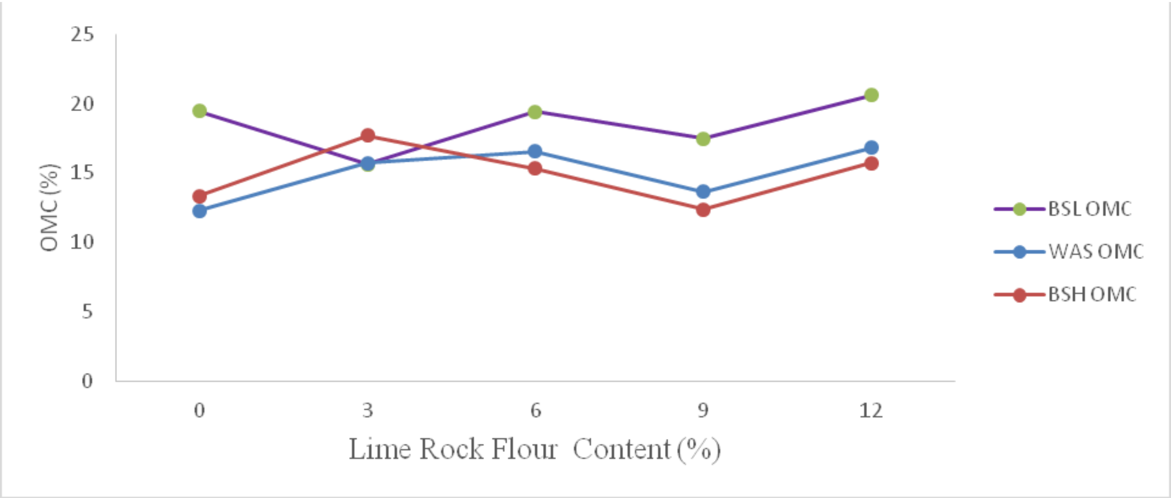

Figure 4.4: Variation of OMC with varying rock flour +lime content when compacted using WAS, BSL and BSH compaction efforts

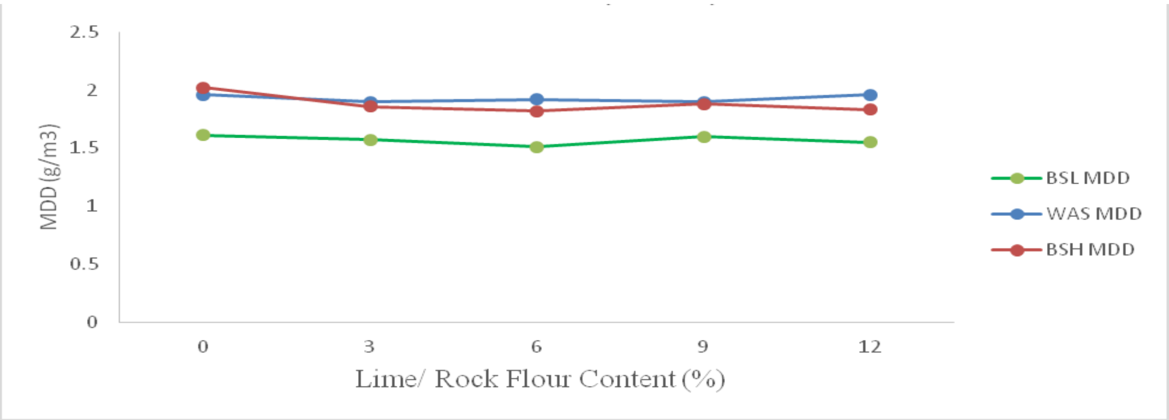

Figure 4.5: Variation of MDD with rock flour +lime content when compacted using WAS, BSL and BSH compaction efforts

4.1.7 California bearing ratio (CBR)

The strength of sub-grade and sub-base is the main factor in determining the required thickness of flexible pavements for roads and airfields. The strength of a sub-grade, sub-base and base course materials is expressed in terms of their California Bearing Ratio (CBR) value. The CBR of the soil sample under saturated condition was tested. The results of the test at various proportions and energy level are as presented in figure 4.6

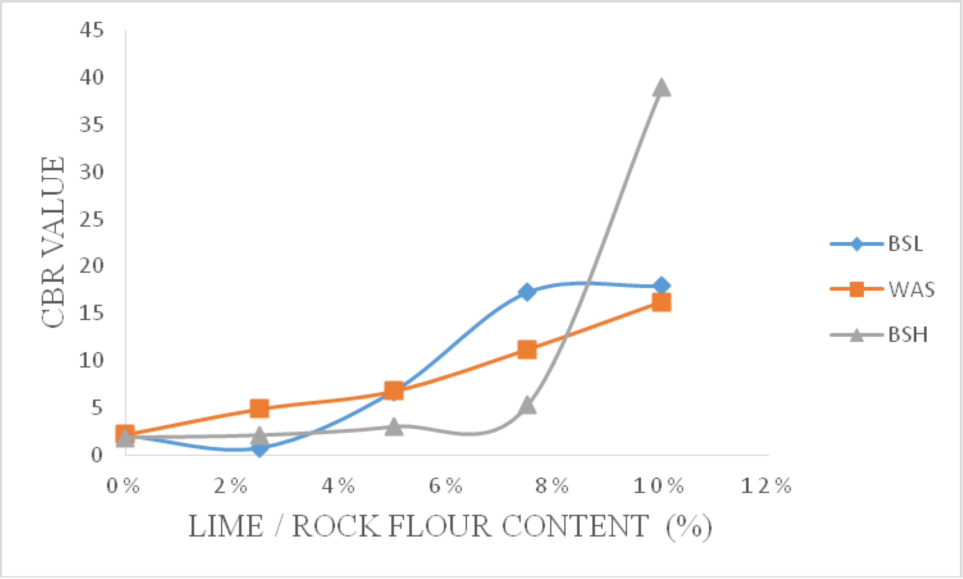

Figure 4.6: CBR plot at various energy level

The reason for the increases in strengths could be due to adequate amounts of calcium from lime required for the formation of calcium hydrate (CSH) which is the major element for strength gain. According to Gidigasu and Dogbe (1976) a minimum CBR value of 60 to 80% is required for bases and 20 to 30% for sub-bases both when compacted at optimum moisture and 100% intermediate/West African Standard. The optimum CBR values obtained for the stabilized soil conformed to the specified by the Nigerian General Specification (1997).

4.1.8 Unconfined Compressive Strength (UCS) Analysis of the tested soil sample

Unconfined Compressive Strength is a special case of the unconsolidated undrained triaxial test. In this case, no confining pressure to the soil sample is applied (i.e., $\sigma_3 = 0$) for such conditions. Axial stress on the soil sample is gradually increased until the sample fails. Results obtained from the laboratory for untreated lateritic soil (0%), and rock flour with lime from 2.5 to 10% rock flour and 3 to 12% lime is as presented.

The results show that the unconfined compressive strength increased with increasing lime and rock flour content from 132.57 kN/m^2 for 0 % to 497.53 kN/m^2 for 12% rock flour and 10% lime content for British Standard light compaction effort, 850.04 kN/m^2 0 % to 1,176.62 kN/m^2 for 12% rock flour and 10% lime content for West Africa Standard compaction effort and 740.27 kN/m^2 for 0 % to 1,129.56 kN/m^2 for 12% rock flour and 10% for British standard heavy compaction effort.

The mixture strength steadily increases with increase in rock flour- lime content. According to Das (2003), unconfined compressive strength ranging from 136.91-309.55 kN/m^2 indicates poor to fair soil and UCS greater than 938.85 kN/m^2 as good pavement construction material.

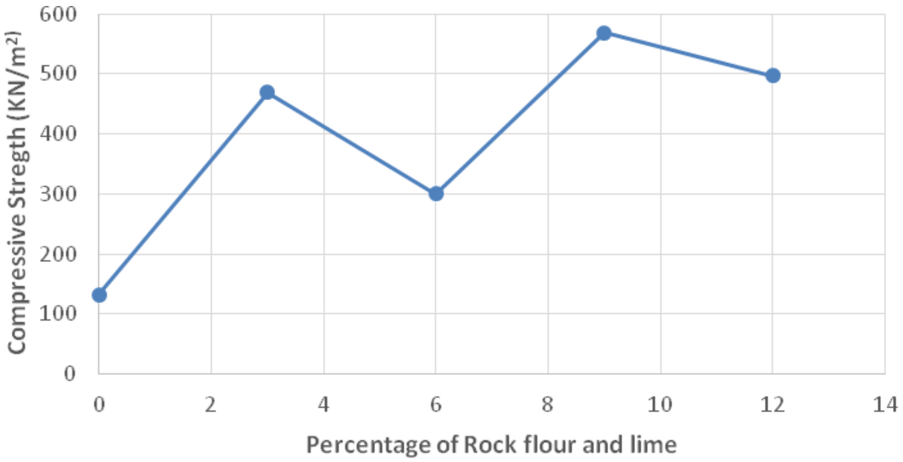

Figure 4.7: UCS plot at British standard light energy level

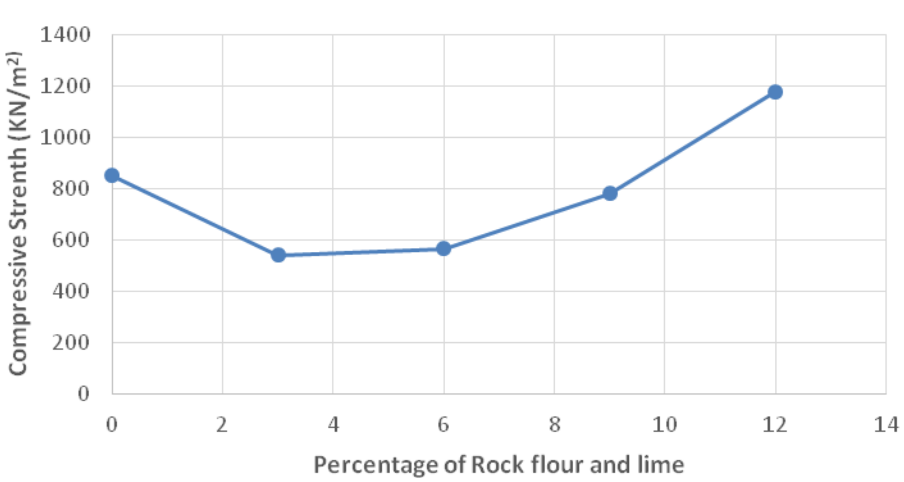

Figure 4.8: UCS plot at West Africa standard energy level

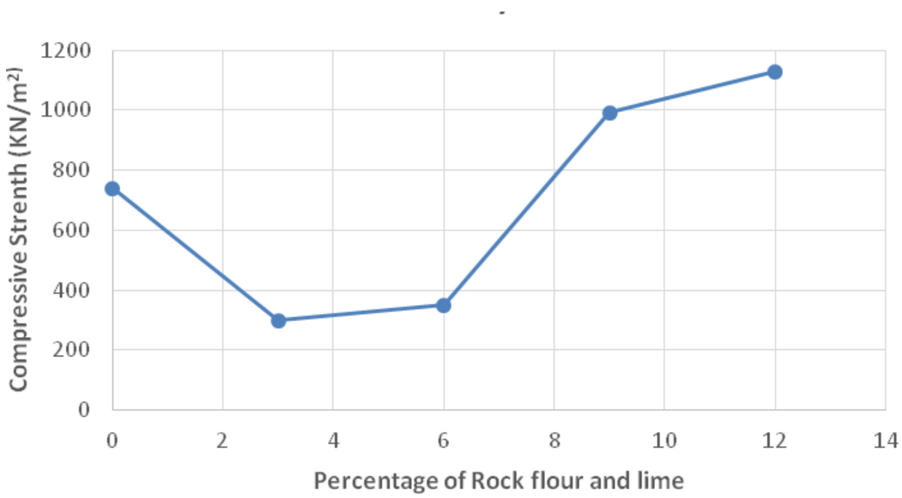

Figure 4.9: UCS plot at British standard heavy energy level

4.1.9 Durability test

The resistance to loss in strength for the sampled soil shows the variation of the durability of soil with rock flour and lime mixtures as shown in figure 4.10. The resistance to loss in strength increased as rock flour (RF) and lime content increased to a value 3.816% (12% rock flour and 10% lime for BSL), 7.108% (12%RF and 10% lime for WAS) and 8.331% (12% RF and 10% lime for BSH). The recorded loss in strength was less than the maximum 20% allowable loss in strength (Osinubi et al., 2009). The result shows that the soil after treatment does meet the durability requirement for use in pavenment construction.

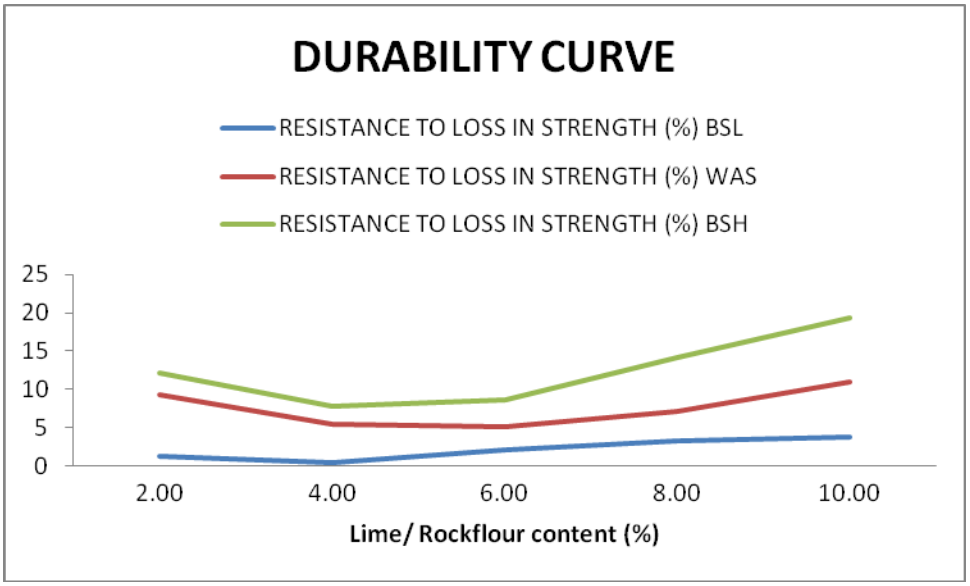

Figure 4.10: Durability plot for various energy levels

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

The aim of this investigation was to evaluate the strength characteristics of soil- rock flour mixtures stabilized with lime for road pavement application. In an attempt to achieve the above aim and improve the performance of rock flour as road construction material, the air-dried soil-rock flour mixtures (0,3,6,9 and 12% rock flour) was treated with lime (0,2.5,5,7.5 and 10%) respectively.

Consequent upon the tests carried out on the mixtures obtained, the following conclusion can be made.

The soil sample has average natural moisture content of 10.28%, specific gravity of 2.63. The untreated sample has liquid limit of 50.55%, plastic limit of 27.33% and plasticity index of 23.22%, which according to Amadi (2015) can be classified as high plasticity. The addition of rock flour and lime caused the liquid limit to decrease to 27.25% the plastic limit to decrease significantly to 18.83%, and plasticity index to 8.42% when treated with 12% and 10% rock flour and lime respectively.

The particle size analysis of the lateritic soil from this location indicates silty clay sand with 0.46 % gravel fraction, 24.54% sand fraction, with 75% of silt-clay fractions (passing through sieve BS No.200 sieve).

The maximum dry density (MDD) shows an increase with higher compaction effort

Addition of rock flour and lime to the natural soil showed an improvement in the CBR and UCS values of the stabilized specimen. BSH gave the highest value of 38.97% at 12% RF and 10% lime for CBR, while WAS gave the highest value of 1,176 kN/m^2 at 12% RF and 10% lime for UCS

The result from durability test shows less than 20% percentage loss in strength, thus met the requirement for pavement construction.

5.2 Recommendations

Based on the results of this study, the following recommendations are proffered;

1. The use of rock flour and lime as improvement technique for lateritic soil (A-7-6 soil) should be encouraged in construction industries with 12% rock flour and 10% lime recommended.
2. It is recommended that field test should be carried out on any soil considered to be used for road construction so as to confirm their suitability for the intending purposes which would or could reduce cost of maintaining such roads in the long run if proper materials are selected or used, that could make the road stand a test of time.

3. Cost benefit analysis should also be undertaken, that is the economic advantage of using rock flour and Lime to stabilize lateritic soil in construction works. Whether it is cheaper, as compared with other alternatives

5.3 Contribution to Knowledge

This work revealed that mixtures of soil-rock flour stabilised with lime improved in both physical and engineering properties at various mix proportions and compaction energies. The results showed a slight decrease in maximum dry density (MDD) and increase in optimum moisture content (OMC). Also results showed that the CBR value improves with increase in rock flour and lime from 1.81% to 38.97% at British standard heavy (BSH) energy. Furthermore, the unconfined compressive strength (UCS) of the sample were greatly improved by addition of rock flour and lime at various compaction energy, The results show that the unconfined compressive strength increased with increasing lime and rock flour content from 132.57 kN/m^2 at 0 % to 497.53 kN/m^2 for 12% rock flour and 10% lime content when compacted with British Standard light compaction effort, 850.04 kN/m^2 at 0 % to 1,176.62 kN/m^2 for 12% rock flour and 10% lime content for West Africa Standard compaction effort and 740.27kN/m^2 for 0 % to 1,129.56 kN/m^2 for 12% rock flour and 10% at British standard heavy compaction effort. This therefore indicates overall, that the addition of rock flour and lime (12%RF and 10% lime) to poor/weak lateritic soil improved the soil strength and resistance to loss in strength.

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APPENDICES

APPENDIX A

Table A1: natural moisture content test of the studied soil sample

Can Number	1	2	3
Can Weight	31.9	24.0	23.0
Weight of Can + Wet Soil	76.9	60.8	72.6
Weight of Can + Dry Soil	72.6	57.4	68.1
Weight of Moisture	4.30	3.40	4.50
Weight of Dry Soil	40.70	33.40	45.10
Moisture Content	10.57	10.18	9.98

Appendix A2: specific gravity of the studied soil sample

Trial	1	2	3
Weight of cylinder (g) m_1	23.1	24.7	24.6
Weight of cylinder + sample (g) m_2	30.9	31.7	33.6
Weight of cylinder + sample + water (g) m_3	65.1	65.9	67.3
Weight of cylinder + water(g) m_4	60.8	61.4	61.9
Specific gravity	2.60	2.80	2.50
Average specific gravity	2.63		

Appendix A3: particles size distribution (sieve analysis test)

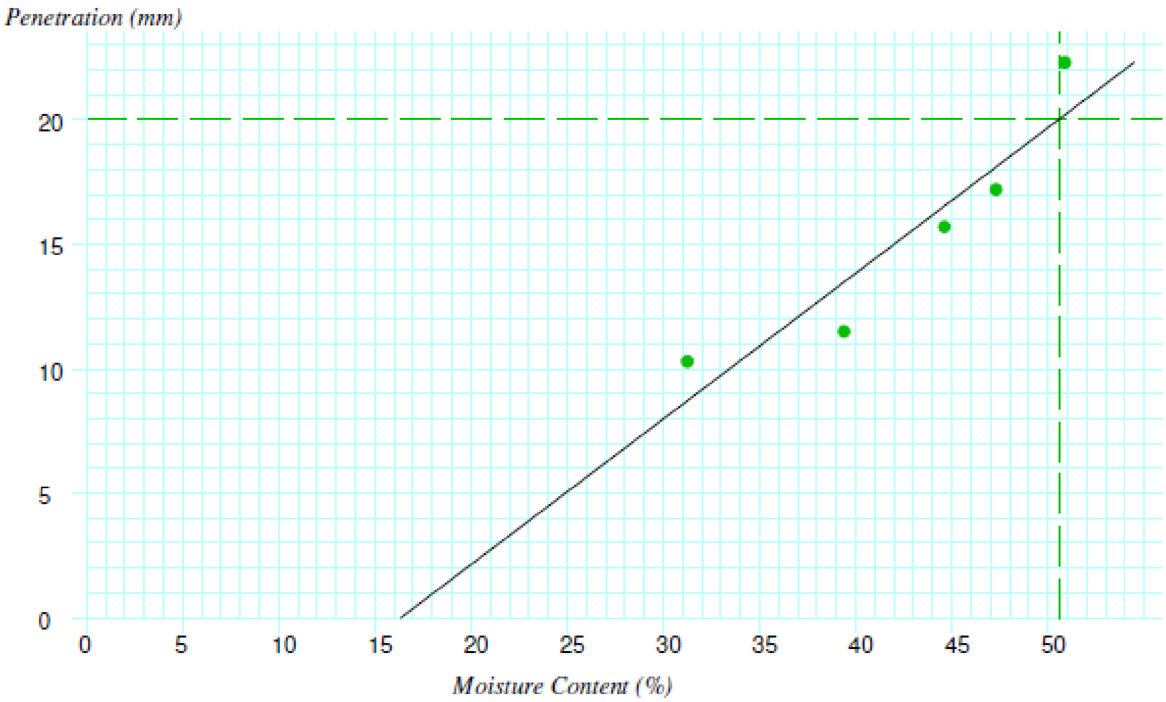
		Percent by Weight	
Sieve Designation	CUM. RET	% RET.CUM	%PASSING
5.0	6.8	2.27	97.7
3.4	19.1	6.37	93.6
2.4	27.1	9.03	91.0
2.0	29.7	9.90	90.1
1.2	32.3	10.77	89.2
0.850	35.2	11.73	88.3
0.600	42.0	14.00	86.0
0.425	48.4	16.13	83.9
0.300	55.4	18.47	81.5
0.150	71.2	23.73	76.3
0.075	74.9	24.97	75.0

APPENDIX B: PLASTICITY CHARACTERISTICS AT VARIOUS ROCK FLOUR AND LIME DOSES

Table B1: Liquid limit of tested soil sample at 0% rock flour and 0% lime (Cone Penetrometer Method)

Project: Master's Thesis		
Test Location:	**0%Rock Flower 0%Lime**	
Sample No:	**Depth of sample:**	**Date: 17/7/2021**
Sample Description:		

	LIQUID LIMIT					**PLASTIC LIMIT**	
Can Number	1	2	3	4	5	1	2
Penetration	10.3	11.5	15.7	17.2	22.3		
Can Weight	24.4	24.1	24.4	24.5	38.6	23.9	20.1
Weight of Can + Wet Soil	32.8	33.3	33.8	35.4	56.1	29.7	24.6
Weight of Can + Dry Soil	30.8	30.7	30.9	31.90	50.2	28.5	23.6
Weight of Moisture	2.00	2.60	2.90	3.50	5.90	1.20	1.00
Weight of Dry Soil	6.40	6.60	6.50	7.40	11.60	4.60	3.50
Moisture Content	31.25	39.39	44.62	47.30	50.86	26.09	28.57
Liquid Limit	**50.55%**		Average Plastic Limit			**27.33%**	



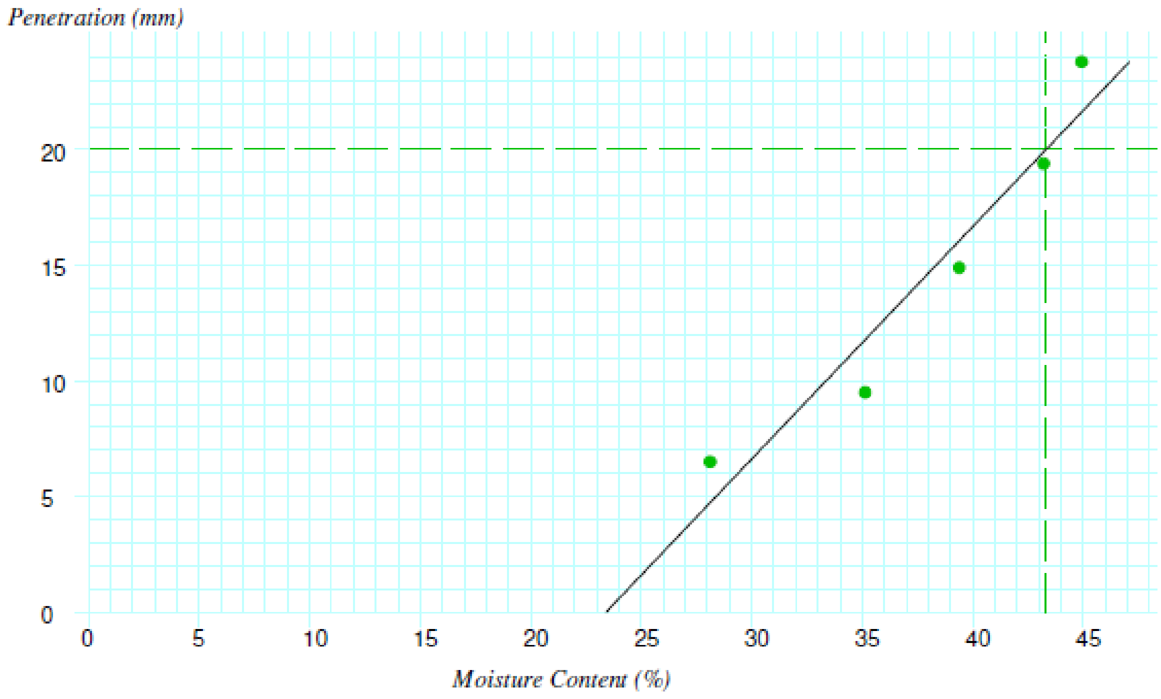
Test executed by:

Figure B1: Liquid limit of tested soil sample at 0% Rock flour and 0% Llime

Table B2: Liquid limit of tested soil sample at 3% Rock flour and 2.5% Lime(Cone Penetrometer Method)

Project: Master's Thesis			
Test Location:	**3%Rock Flour 2.5%Lime**		
Sample No:	**Depth of sample:**	**Date: 17/7/2021**	
Sample Description:			

	LIQUID LIMIT					**PLASTIC LIMIT**	
Can Number	1	2	3	4	5	1	2
Penetration	6.5	9.5	14.9	19.4	23.8		
Can Weight	38.6	38.2	38.6	37.5	38.5	38.6	38.1
Weight of Can + Wet Soil	42.7	43.2	47.8	49.1	48.5	47.1	44.5
Weight of Can + Dry Soil	41.8	41.9	45.2	45.6	45.4	45.2	43.1
Weight of Moisture	0.90	1.30	2.60	3.50	3.10	1.90	1.40
Weight of Dry Soil	3.20	3.70	6.60	8.10	6.90	6.60	5.00
Moisture Content	28.13	35.14	39.39	43.21	44.93	28.79	28.00
Liquid Limit	**43.31%**		Average Plastic Limit			**28.39%**	



Test executed by: ____________________

Figure B2: Liquid limit of tested soil sample at 3% Rock flour and 2.5% Llime

Table B3: Liquid limit of tested soil sample at 6% Rock flour and 5% Lime (Cone Penetrometer Method)

Project: Master's Thesis		
Test Location:	**6%Rock Flour 5%Lime**	
Sample No:	**Depth of sample:**	**Date: 17/7/2021**
Sample Description:		

	LIQUID LIMIT					**PLASTIC LIMIT**	
Can Number	1	2	3	4	5	1	2
Penetration	10.9	11.9	17.7	17.2	22.3		
Can Weight	21.7	25.4	24.3	24.5	38.6	23.9	24.8
Weight of Can + Wet Soil	27.5	32.6	30.6	35.4	56.1	29.7	29.3
Weight of Can + Dry Soil	26.2	30.9	29.0	31.9	50.2	28.5	28.5
Weight of Moisture	1.30	1.70	1.60	3.50	5.90	1.20	0.80
Weight of Dry Soil	4.50	5.50	4.70	7.40	11.60	4.60	3.70
Moisture Content	28.89	30.91	34.04	47.30	50.86	25.00	21.62
Liquid Limit	**36.46%**		Average Plastic Limit			**23.31%**	

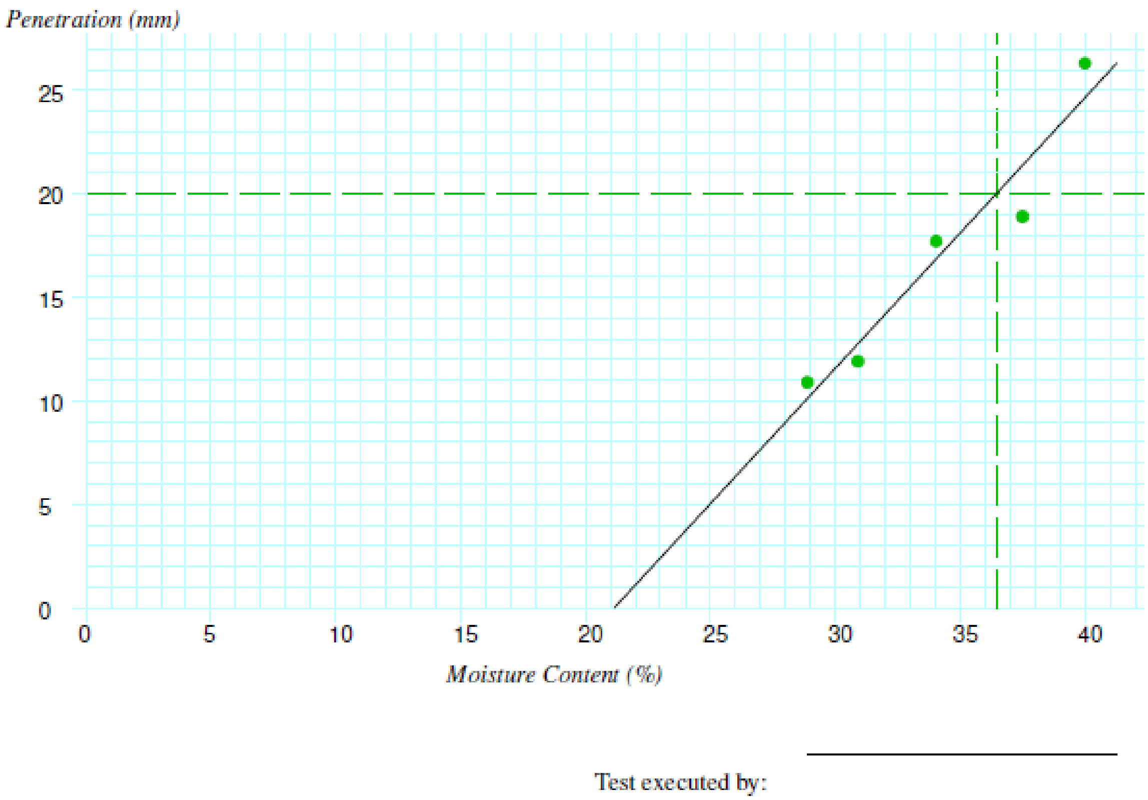


Figure B3: Liquid limit of tested soil sample at 6% Rock flour and 5% Llime

Table B4: Liquid limit of tested soil sample at 9% Rock flour and 7.5% Lime (Cone Penetrometer Method)

Project: Master's Thesis			
Test Location:	**9%Rock Flour 7.5%Lime**		
Sample No:	**Depth of sample:**		**Date: 17/7/2021**
Sample Description:			

	LIQUID LIMIT					**PLASTIC LIMIT**	
Can Number	1	2	3	4	5	1	2
Penetration	10.4	15.1	18.3	23.8	26.5		
Can Weight	25.5	20.3	24.2	32.3	26.0	27.6	29.7
Weight of Can + Wet Soil	34.1	26.9	34.4	31.3	36.2	31.7	34.8
Weight of Can + Dry Soil	32.1	25.3	31.8	29.2	33.4	30.8	33.8
Weight of Moisture	2.00	1.60	2.60	2.10	2.80	0.90	1.00
Weight of Dry Soil	6.60	5.00	7.60	5.90	7.40	3.20	4.10
Moisture Content	3030	32.00	34.21	37.84	40.00	28.13	24.39
Liquid Limit	**34.53%**		Average Plastic Limit			**26.26%**	

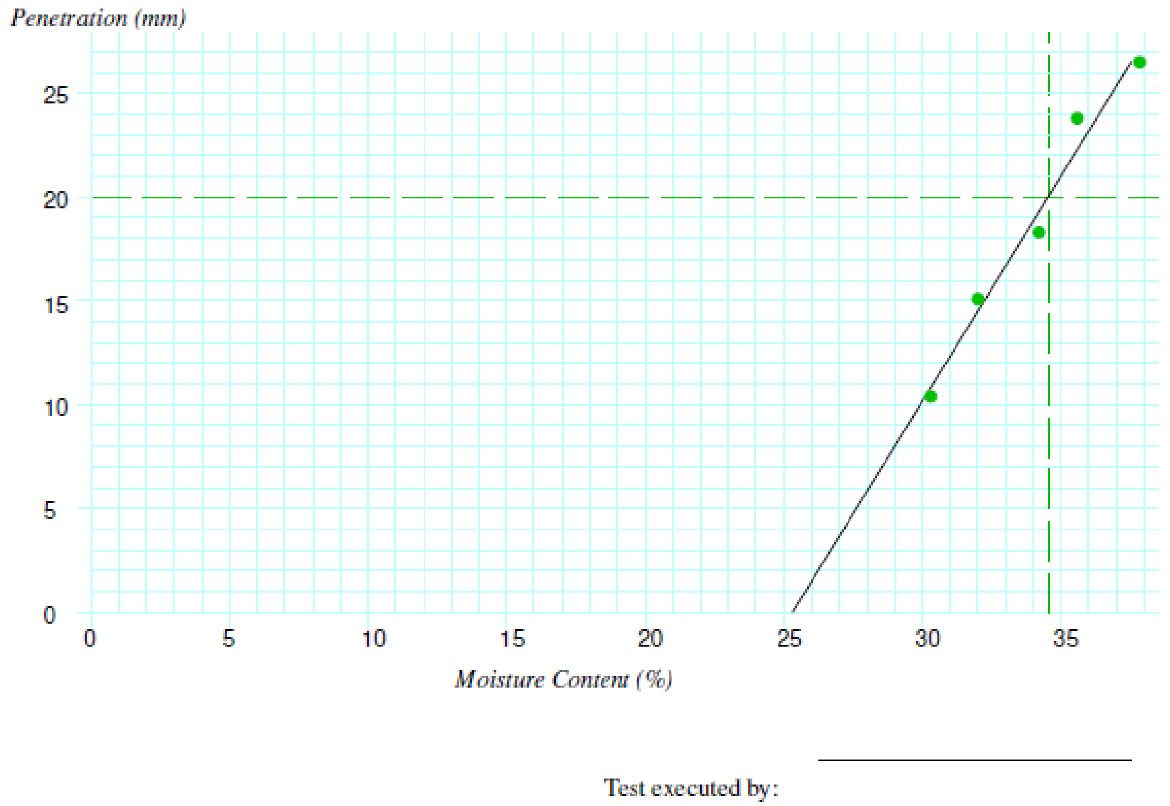


Figure B4: Liquid limit of tested soil sample at 9% Rock flour and 7.5% Llime

Table B5: Liquid limit of tested soil sample at 12% Rock flour and 10% Lime (Cone Penetrometer Method)

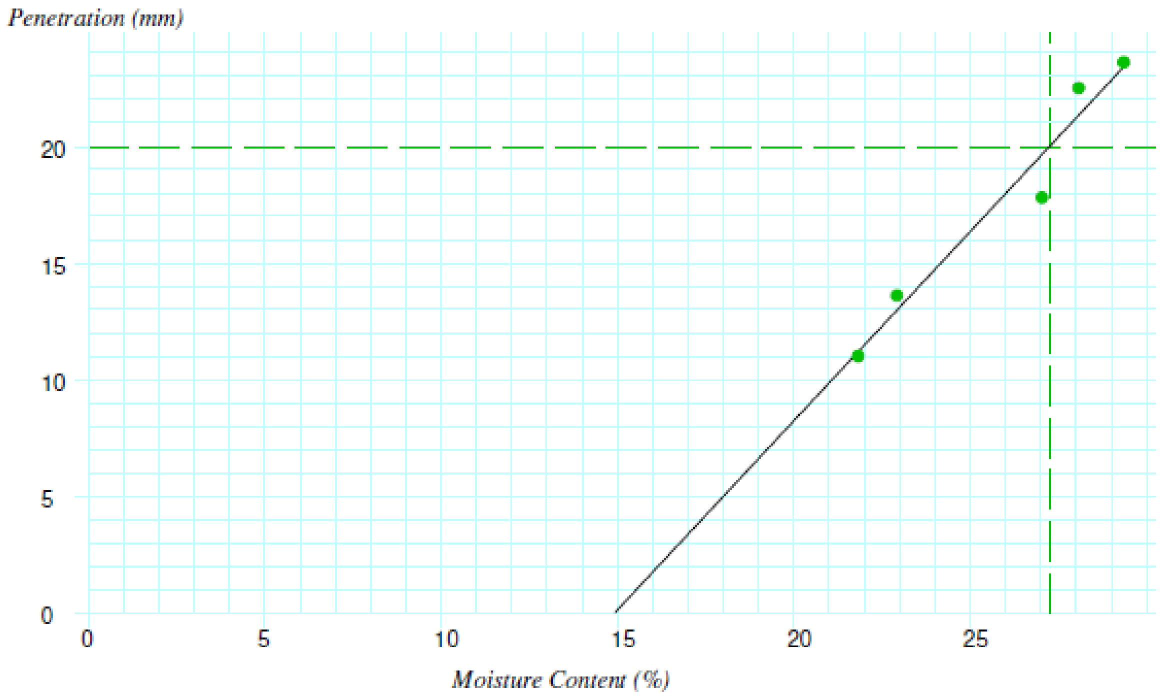
Project: Master's Thesis

Test Location: **12%Rock Flower 10%Lime**

Sample No: **Depth of sample:** **Date: 17/7/2021**

Sample Description:

	LIQUID LIMIT					**PLASTIC LIMIT**	
Can Number	1	2	3	4	5	1	2
Penetration	11.0	13.6	17.8	22.5	23.6		
Can Weight	19.9	15.6	21.6	19.7	21.9	21.9	19.0
Weight of Can + Wet Soil	26.6	27.7	31.0	27.0	33.8	25.3	24.7
Weight of Can + Dry Soil	25.4	25.5	29.0	25.4	31.1	24.9	23.9
Weight of Moisture	1.20	2.20	2.00	1.60	2.70	0.40	0.80
Weight of Dry Soil	5.50	9.60	7.40	5.70	9.20	3.00	4.90
Moisture Content	21.82	22.92	27.03	28.07	29.35	13.33	16.33
Liquid Limit	**27.25%**		Average Plastic Limit			**14.83%**	



Test executed by: ______________________

Figure B5: Liquid limit of tested soil sample at 12% Raock flour and 10% Llime

APPENDIX C: COMPACTION CHARACTERISTICS AT VARIOUS ROCK FLOUR AND LIME DOSES

Table C1: Compaction test result of soil with 0% rock flour and 0% lime (British Standard heavy)

Project: Master's Thesis		
Test Location:	**0%Rock Flower 0%Lime**	**British Standard Heavy**
Sample No:	**Volume of Mold: 944cm^3**	**Date: 17/7/2021**
Sample Description:		

Weight of Mold (g)	3388		3388		3388		3388		3388	
Weight of Mold + Wet S	5128		5538		5556		5458		5372	
Weight of wet Soil (g)	1,740.00		2,150.00		2,168.00		2,070.00		1,984.00	
Wet Density (g/cm^3)	1.84		2.28		2.30		2.19		2.10	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	19.9	19.7	21.9	22.4	21.6	19.0	15.9	18.2	22.2	22.0
Weight of Can + Wet Soil (g)	43.0	45.4	37.5	35.6	37.8	38.4	38.8	39.6	53.8	56.4
Weight of Can + Dry Soil (g)	41.3	43.2	35.6	34.2	35.6	35.8	34.7	36.1	47.9	50.0
Weight of water (g)	1.70	2.20	1.90	1.40	2.20	2.60	4.10	3.50	5.90	6.40
Weight of Dry Soil (g)	21.40	23.50	13.70	11.80	12.00	16.80	18.80	1.90	25.70	28.00
Moisture Content (g)	7.94	9.36	13.87	11.86	15.71	15.48	21.81	19.55	22.96	22.86
Ave. moisture Content (g)	8.65		12.87		15.60		20.68		22.91	
Dry Density (g/cm^3)	1.6964		2.0179		1.9868		1.8170		1.7100	

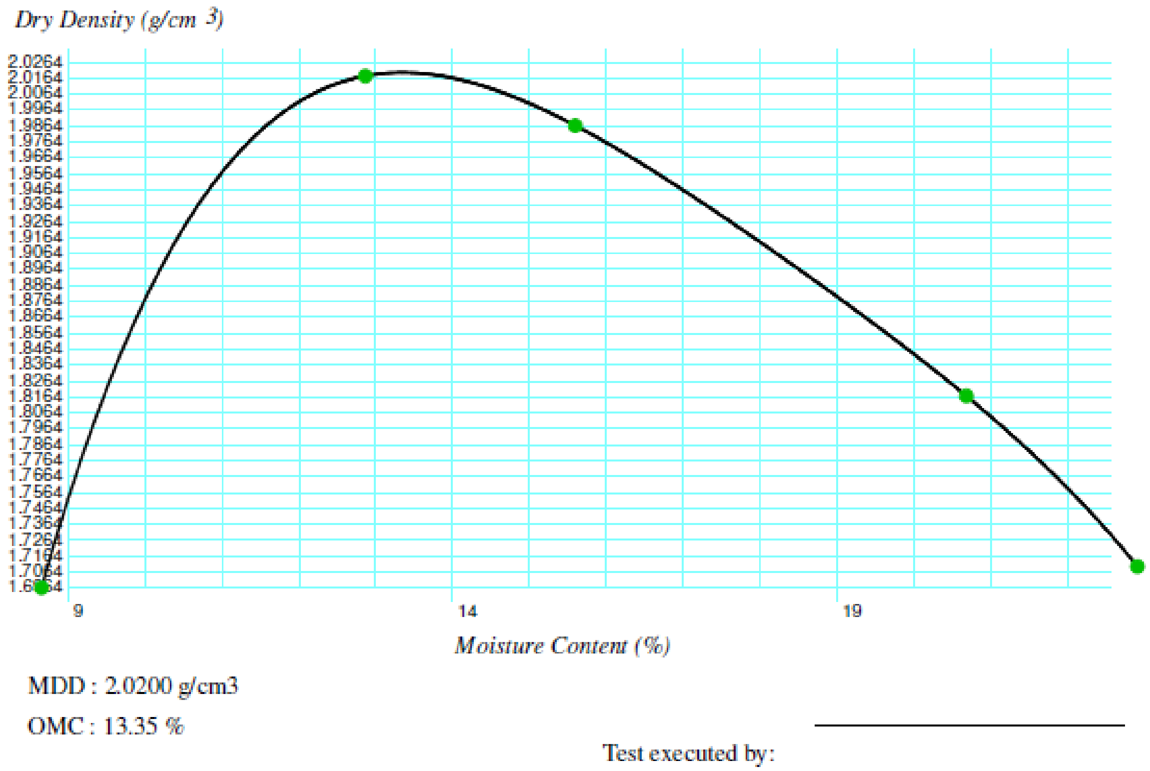


Figure C1: MDD and corresponding OMC of studied soil sample treated with 0% rock flour and 0% lime (BSH)

Table C2: Compaction test result of soil with 3% rock flour and 2.5% lime (British Standard heavy)

Project: Master's Thesis

Test Location: **3%Rock Flower 2.5%Lime British Standard Heavy**

Sample No: **Volume of Mold: 944cm^3** **Date: 17/7/2021**

Sample Description:

Weight of Mold (g)	3388		3388		3388		3388		3388
Weight of Mold + Wet S	5156		5268		5378		5460		5410
Weight of wet Soil (g)	1,768.00		1,880.00		1,990.00		2,072.00		2,022.00
Wet Density (g/cm^3)	1.87		1.99		2.11		2.19		2.14
Can Number	1	2	3	4	5	6	7	3	
Weight of Can (g)	24.3	23.9	24.6	23.3	23.1	22.7	23.8	3.4	
Weight of Can + Wet Soil (g)	40.0	50.5	47.1	45.7	39.4	39.3	48.8	5.6	
Weight of Can + Dry Soil (g)	38.6	48.1	44.5	43.3	37.2	37.2	45.0	2.2	
Weight of water (g)	1.40	2.40	2.60	2.40	2.20	2.10	3.80	.40	
Weight of Dry Soil (g)	14.30	24.20	19.90	20.00	14.10	14.50	21.20	8.80	
Moisture Content (g)	9.79	9.92	13.07	12.00	15.60	14.48	17.92	8.07	
Ave. moisture Content (g)	9.85		12.53		15.04		18.00		23.09
Dry Density (g/cm^3)	1.7049		1.7697		1.8324		1.8600		1.7402

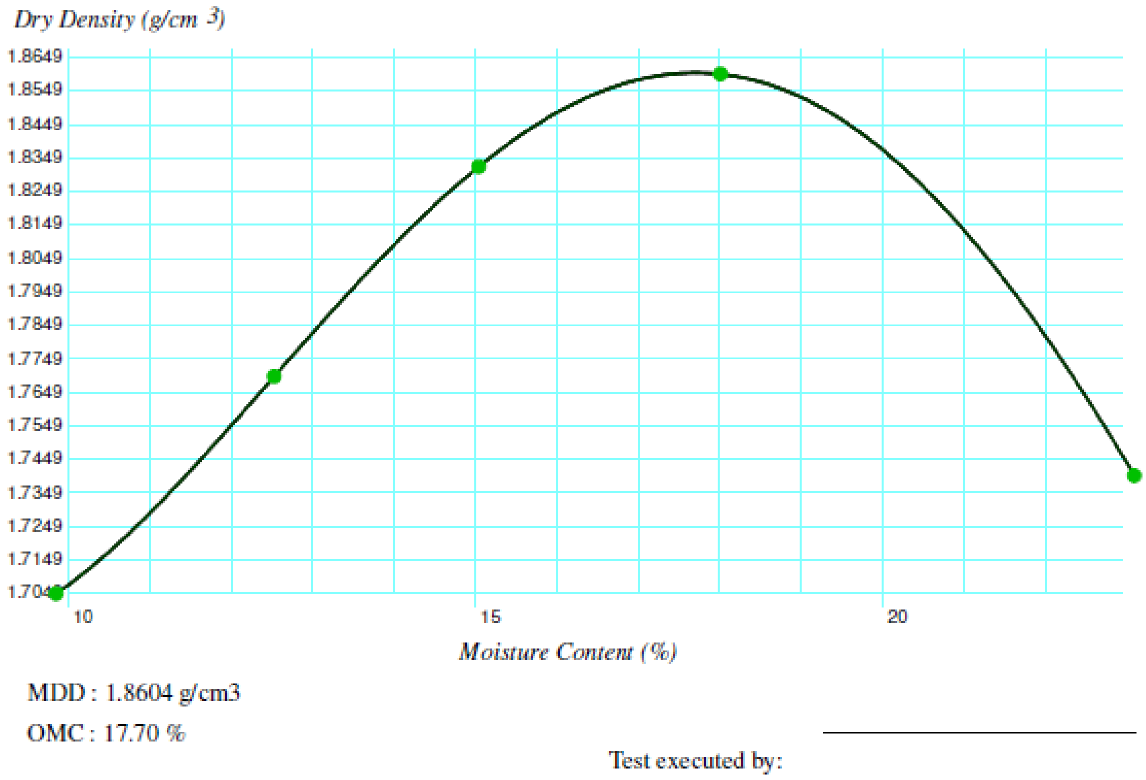
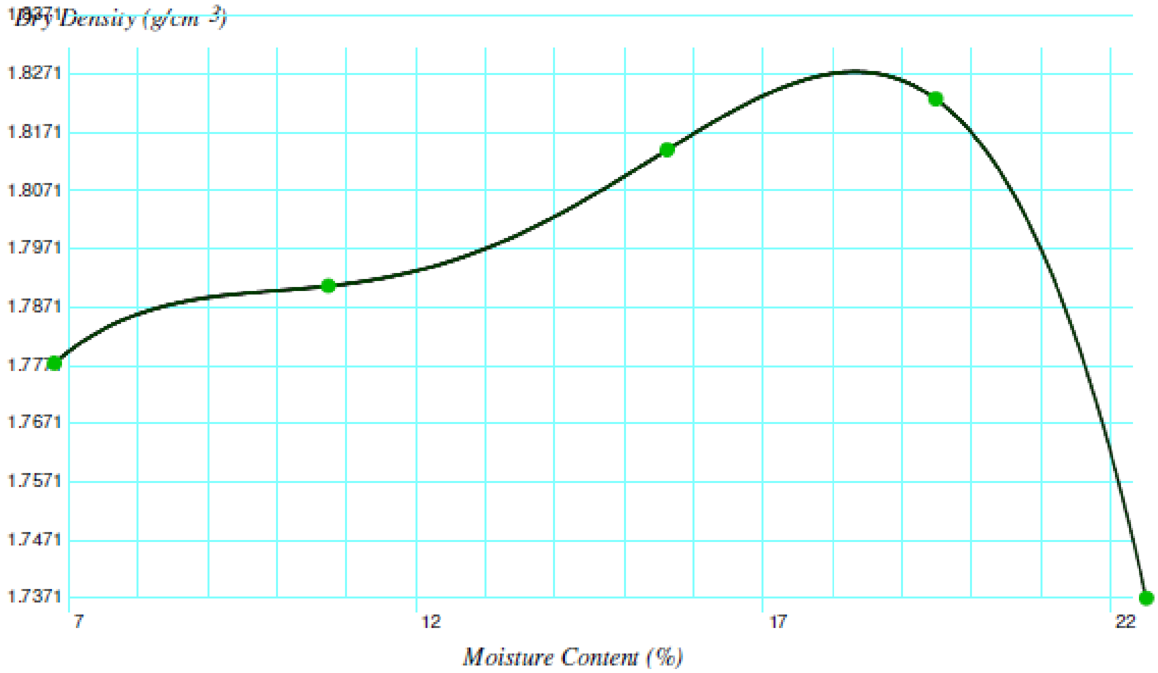


Figure C2: MDD and corresponding OMC of studied soil sample treated with 3% rock flour and 2.5% lime (BSH)

Table C3: Compaction test result of soil with 6% rock flour and 5% lime (British Standard heavy)

Project: master's thesis										
Test location:	**6% rock flour and 5% lime**				**British standard heavy**					
Sample no:	**volume of mold: 944cm^3**				**Date: 15/7/2021**					
Sample description:										
Weight of mold	3388		3388		3388		3388		3388	
Weight of mold+ wet soil (g)	5180		5260		5368		5444		5397	
Weight of wet soil (g)	1,792.00		1,872.00		1,980.00		2,056,00		2,009.00	
Wet density (g/cm3)										
Can number	1	2	3	4	5	6	7	8	9	10
Weight of can (g)	23.2	23.0	24.0	25.0	24.2	23.9	23.1	25.8	24.2	23.1
Weight of can + wet soil(g)	45.8	42.7	41.3	45.9	45.2	42.9	44.3	46.41	1,453.8	41.5
Weight of can + dry soil(g)	44.3	41.5	39.6	43.9	42.4	40.3	40.8	43.0	38.3	38.1
Weight of water (g)	1.50	1.20	1.70	2.00	2.80	2.60	3.50	3.30	3.15	3.40
Weight of dry soil (g)	21.10	18.40	15.60	18.90	18.20	16.40	17.70	17.20	14.10	15.00
Moisture content (g)	7.11	6.49	10.90	10.58	15.38	15.38	19.77	19.19	22.36	22.67
Ave. moisture (g)	6.80		10.74		15.62		19.48		22.51	
Dry density (g/cm^3)	1.7775		1,7907		1.8141		1.8229		1.7371	



MDD : 1.8275 g/cm3

OMC : 18.33 %

Test executed by:

Figure C3: MDD and corresponding OMC of studied soil sample treated with 6% rock flour and 5% lime (BSH)

Table C4: Compaction test result of soil with 9% rock flour and 7.5% lime (British Standard heavy)

Project: master's thesis

Test location: **9% rock flour and 7.5% lime** **British standard heavy**
Sample no: **volume of mold: 944cm^3** **Date: 15/7/2021**

Sample description:

Weight of mold	3388		3388		3388		3388		3388	
Weight of mold+ wet soil (g)	5172		5283		5386		5420		5408	
Weight of wet soil (g)	1,784.00		1,895.00		1,998.00		2,032,00		2,020.00	
Wet density (g/cm3)	1.89		2.01		2.12		2.15		2.14	
Can number	1	2	3	4	5	6	7	8	9	10
Weight of can (g)	22.7	23.1	23.3	24.2	24.4	23.0	24.6	23.0	22.6	24.2
Weight of can + wet soil(g)	39.1	41.3	40.5	44.0	45.0	44.1	56.2	45.2	51.7	47.5
Weight of can + dry soil(g)	38.2	40.2	39.1	42.5	42.8	41.8	50.9	41.6	46.2	43.1
Weight of water (g)	0.90	1.10	1.40	1.50	2.20	2.30	5.30	3.60	5.50	4.40
Weight of dry soil (g)	15.50	17.10	15.80	18.40	18.40	18.80	26.30	18.60	23.60	18.90
Moisture content (g)	5.81	6.43	8.86	8.20	11.96	12.23	20.15	19.35	23.31	23.28
Ave. moisture (g)	6.12		8.53		12.10		19.75		23.29	
Dry density (g/cm^3)	1.7809		1.8497		1.8881		1.7975		1.7356	

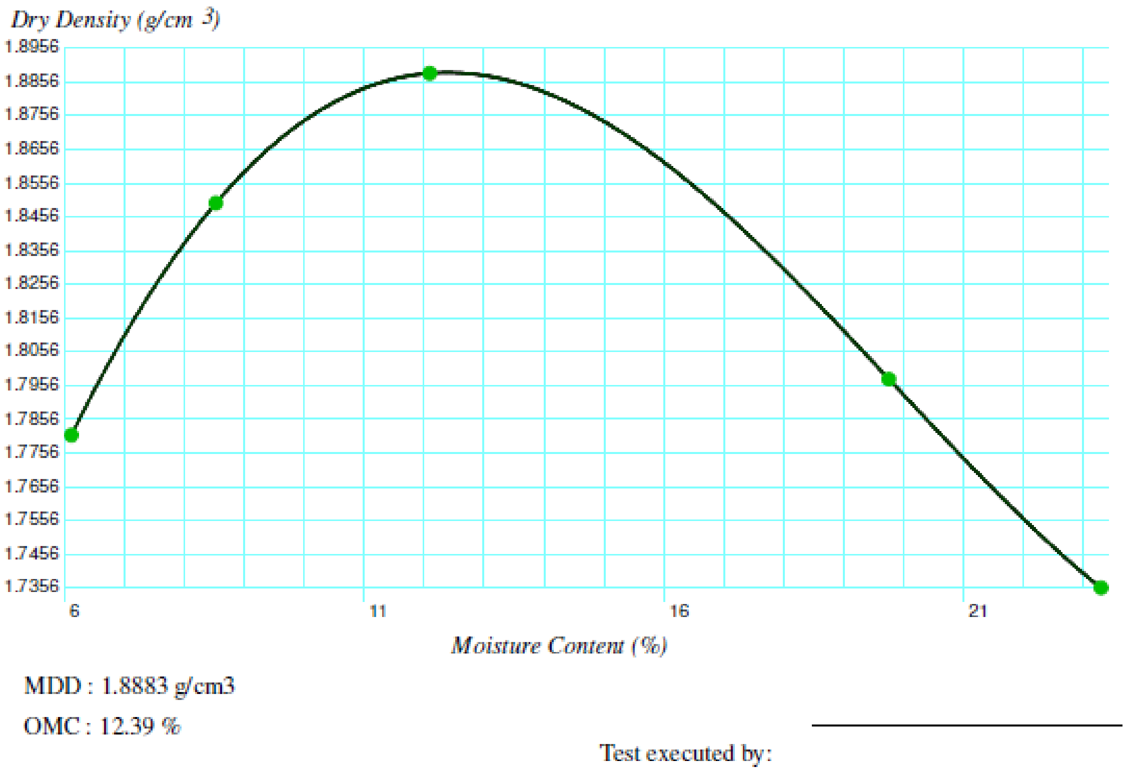


Figure C4: MDD and corresponding OMC of studied soil sample treated with 9% rock flour and 7.5% lime (BSH)

Table C5: Compaction test result of soil with 12% rock flour and 10% lime (British Standard heavy)

Project: Master's Thesis

Test Location : 12% Rock Flour and 10% Lime British Standard Heavy
Sample no : Volume of Mold: 944cm³ Date: 15/7 /2021
Sample Description:

Weight of Mold (g)	3388		3388		3388		3388		3388	
Weight of Mold+Wet Soil (g)	5116		5272		5374		5414		5354	
Weight of Wet Soil (g)	1,728.00		1,884.00		1,986.00		2,026.00		1,966.00	
Weight Density g/cm^3)	1.83		2.00		2.10		2.15		2.08	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can g)	23.4	23.1	23.1	23.2	22.9	23.1	24.3	23.0	24.1	23.8
Weight of can + wet soil (g)	46.7	45.1	37.4	43.2	47.0	40.4	48.8	42.3	51.9	46.3
Weight of Can + Dry Soil (g)	45.2	43.5	36.0	41.1	43.9	38.1	44.8	39.2	46.7	42.2
Weight of Water (g)	1.50	1.60	1.40	2.10	3.10	2.30	4.00	3.10	5.20	4.10
Weight of Dry Soil (g)	21.8	20.4	12.90	17.90	21.00	15.00	20.50	16.20	22.6	18.40
Moisture Content (g)	6.88	7.84	10.85	11.73	14.76	15.33	19.51	19.14	23.01	22.28
Ave. Moisture Content (g)	7.36		11.29		15.05		19.32		22.65	
Dry Density (g/cm^3)	1.7050		1.7933		1.8286		1.7986		1.6981	

MDD : 1.8296 g/cm3

OMC : 15.72 %

Test executed by: ____________________

Figure C5: MDD and corresponding OMC of studied soil sample treated with 12% rock flour and 10% lime (BSH)

Table C6: Compaction test result of soil with 0% rock flour and 0% lime (West Africa standard)

Project : Master's Thesis

Test Location : 0% Rock flour and 0% Lime **West Africa Standard**

Sample no : **volume of Mold: 944cm^3** **Date: 16/7 /2021**

Sample Description:

Weight of Mold (g)	3388		3388		3388		3388		3388	
Weight of Mold+Wet Soil (g)	5158		5452		5492		5432		5430	
Weight of Wet Soil (g)	1,770.00		2,064.00		2,104.00		2,044.00		2,042.00	
Weight Density (g/cm^3)	1.88		2.19		2.23		2.17		2.16	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	24.0	22.6	27.6	26.2	26.2	24.4	22.8	23.3	23.3	25.2
Weight of can + wet soil (g)	40.4	34.2	39.2	44.6	41.8	35.6	47.9	48.2	41.4	49.6
Weight of Can + Dry Soil (g)	39.3	33.5	37.9	42.8	39.7	34.3	44.0	44.4	38.0	45.2
Weight of Water (g)	1.10	0.70	1.30	1.80	2.10	1.30	3.90	3.80	3.40	4.40
Weight of Dry Soil (g)	15.30	10.90	10.30	16.60	13.50	9.90	21.20	21.10	14.70	20.00
Moisture Content (g)	7.19	6.42	12.62	10.84	15.56	13.13	18.40	18.01	23.13	22.00
Ave. Moisture Content (g)	6.81		11.73		14.34		18.20		22.56	
Dry Density (g/cm^3)	1.7555		1.9569		1.9492		1.8318		1.7649	

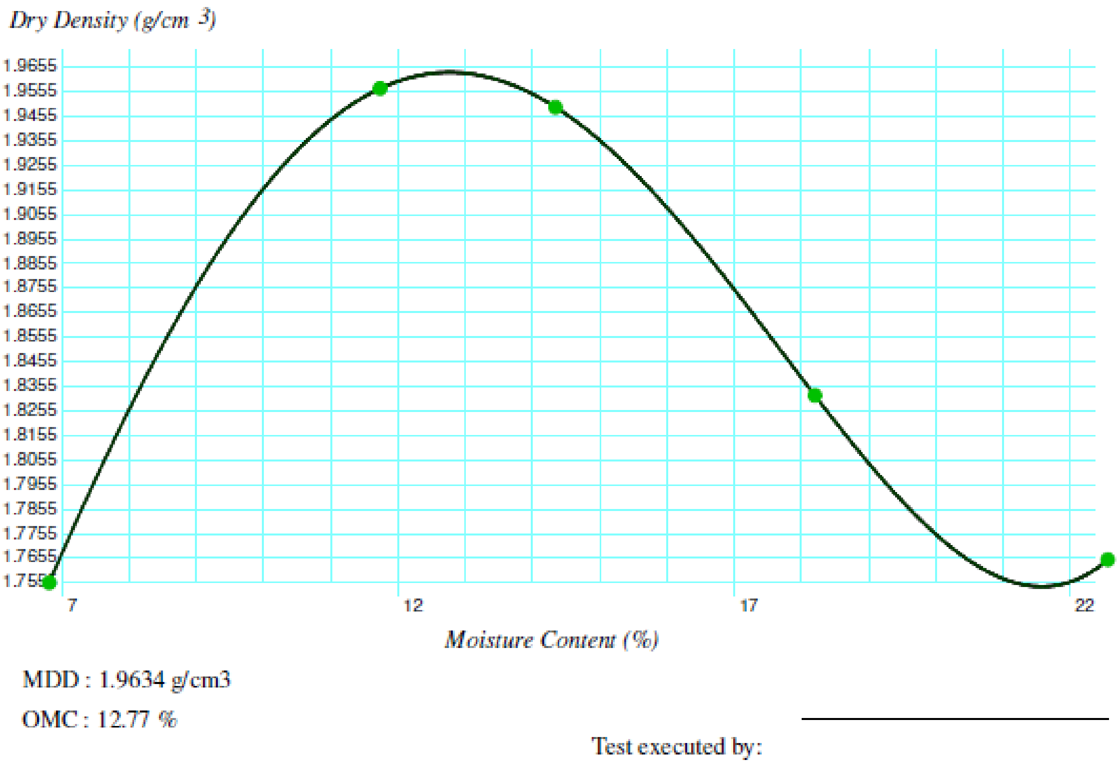


Figure C6: MDD and corresponding OMC of studied soil sample treated with 0% rock flour and 0% lime (WAS)

Table C7: Compaction test result of soil with 3% rock flour and 2.5% lime (West Africa standard)

Project : Master's Thesis

Test Location : 3% Rock Flour and 2.5% Lime West Africa Standard

Sample no : **Volume of Mold: 944cm^3** **Date: 16/7 /2021**

Sample Description:

Weight of Mold (g)	3388		3388		3388		3388		3388	
Weight of Mold+Wet Soil (g)	5178		5272		5375		5454		5405	
Weight of Wet Soil (g)	1,790.00		1,884.00		1,987.00		2.066.00		2,017.00	
Weight Density (g/cm^3)	1.90		2.00		2.10		2.19		2.14	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	25.8	25.3	24.1	23.1	30.1	26.0	22.5	20.3	22.3	21.3
Weight of can + wet soil (g)	35.3	38.1	40.5	40.5	43.3	41.8	38.2	36.5	38.73	36.4
Weight of Can + Dry Soil (g)	34.7	37.3	39.1	39.2	41.9	40.0	36.0	34.5	36.1	34.0
Weight of Water (g)	0.60	0.80	1.40	1.30	1.40	1.80	2.20	2.00	2.63	2.40
Weight of Dry Soil (g)	8.90	12.0	15.00	16.10	11.80	14.00	13.50	14.20	13.8	12.70
Moisture Content (g)	6.74	6.67	9.33	8.07	11.86	12.86	16.30	14.08	19.0	18.90
Ave. Moisture Content (g)	6.70		8.70		12.36		15.19		18.98	
Dry Density (g/cm^3)	1.7771		1.8360		1,8733		1,8999		1.7958	

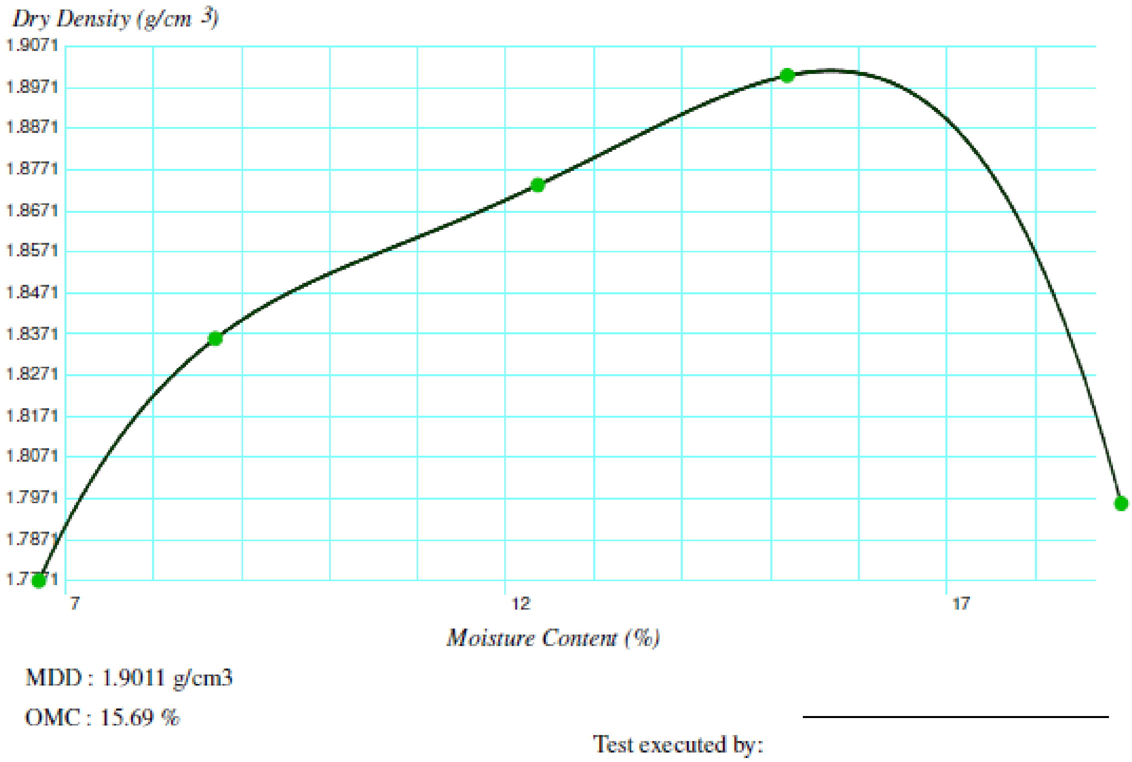


Figure C7: MDD and corresponding OMC of studied soil sample treated with 3% rock flour and 2.5% lime (WAS)

Table C8: Compaction test result of soil with 6% rock flour and 5% lime (West Africa standard)

Project : Master's Thesis										
Test Location :	**6% Rock flour and 5% Lime**			**West Africa Standard**						
Sample no :	**Volume of Mold: 944cm^3**			**Date: 16/7 /2021**						
Sample Description:										
Weight of Mold (g)	3388		3388		3388		3388		3388	
Weight of Mold+Wet Soil (g)	5192		5277		5412		5516		5374	
Weight of Wet Soil (g)	1,804,00		1,889.00		2,024.00		2,128.00		1,986.00	
Weight Density (g/cm^3)	1.91		2.00		2.14		1.25		2.10	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g	22.2	19.1	19.8	15.9	22.0	24.3	18.3	21.8	21.6	19.9
Weight of can + wet soil (g)	33.2	32.1	36.0	31.6	41.4	38.5	35.7	39.7	37.13	35.2
Weight of Can + Dry Soil (g)	32.5	31.4	34.6	30.4	39.0	36.9	33.2	37.1	34.4	32.7
Weight of Water (g)	0.70	0.70	1.40	1.20	2.40	1.60	2.50	2.60	2.73	2.50
Weight of Dry Soil (g)	10.3	12.3	14.80	14.50	17.00	12.60	14.90	15.30	12.8	12.80
Moisture Content (g)	6.80	5.69	9.46	8.28	14.12	12.70	16.78	16.99	21.3	19.53
Ave. Moisture Content (g)	6.24		8.87		13.41		16.89		20.43	
Dry Density (g/cm^3)	1.7987		1.8381		1.8906		1.9286		1.7469	

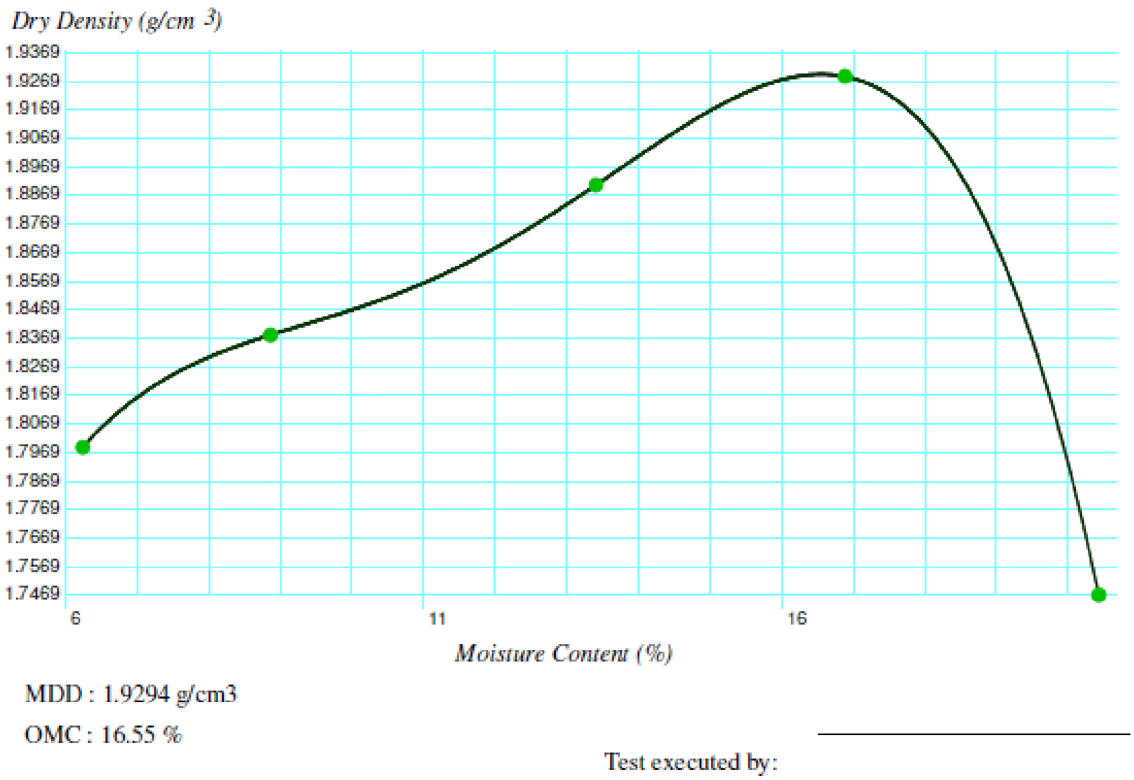


Figure C8: MDD and corresponding OMC of studied soil sample treated with 6% rock flour and 5% lime (WAS)

Table C9: Compaction test result of soil with 9% rock flour and 7.5% lime (West Africa standard)

Project : Master's Thesis
Test Location : 9% Rock Flour and 7.5% Lime West Africa Standard
Sample no : Volume of Mold: 944cm^3 Date: 16/7 /2021
Sample Description:

Weight of Mold (g)	3388		3388		3388		3388		3388	
Weight of Mold+Wet Soil (g)	5176		5277		5386		5454		5383	
Weight of Wet Soil (g)	1,788.00		1,889.00		1,998.00		2,066.0		1,995.00	
Weight Density (g/cm^3)	1.89		2.00		2.12		2.19		2.11	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	21.9	18.5	19.1	21.4	23.5	23.2	21.2	21.6	22.6	21.3
Weight of can + wet soil (g)	38.8	36.3	33.5	39.3	40.4	40.6	47.7	40.2	44.6	44.4
Weight of Can + Dry Soil (g)	38.1	35.2	32.4	37.8	38.7	38.7	43.9	37.9	40.9	40.0
Weight of Water (g)	0.70	1.10	1.10	1.50	1.70	1.90	3.80	2.30	3.70	4.40
Weight of Dry Soil (g)	16.2	16.7	13.30	16.40	15.20	15.50	22.70	16.30	18.3	18.70
Moisture Content (g)	4.32	6.59	8.27	9.15	11.18	12.26	16.74	14.11	20.2	23.53
Ave. Moisture Content (g)	5.45		8.71		11.72		15.43		21.87	
Dry Density (g/cm^3)	1,7961		1.8408		1.8945		1.8961		1.7340	

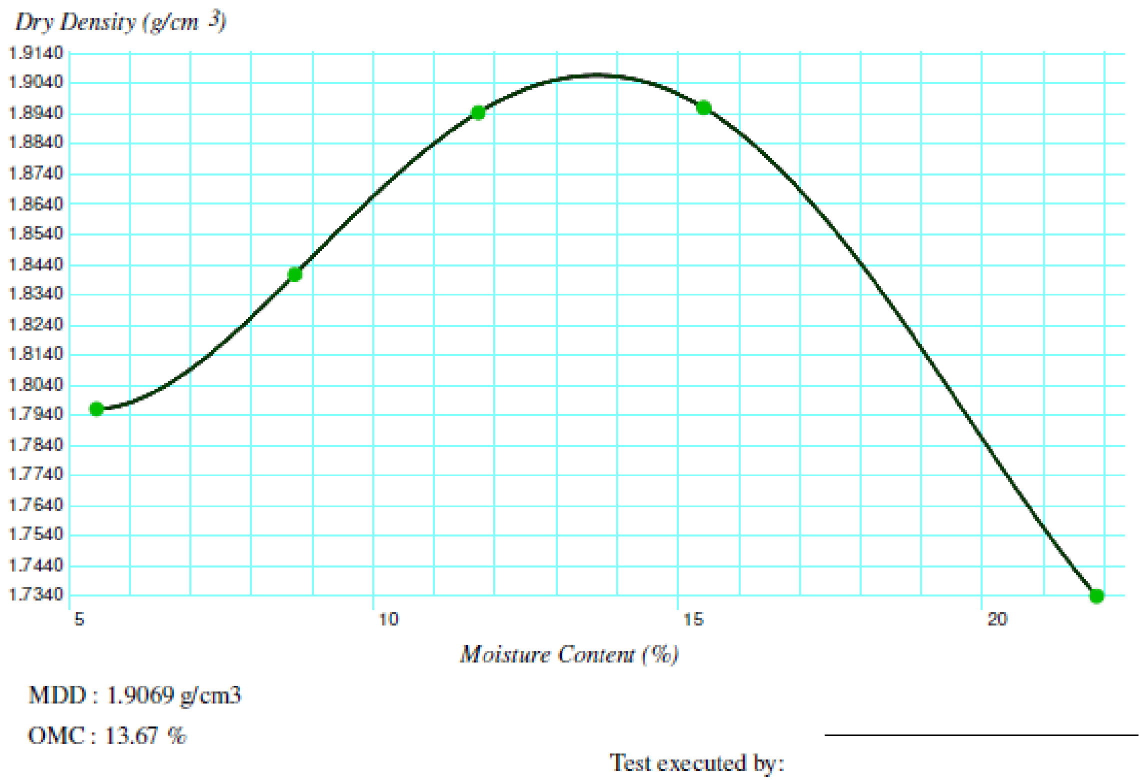


Figure C9: MDD and corresponding OMC of studied soil sample treated with 9% rock flour and 7.5% lime (WAS)

Table C10: Compaction test result of soil with 12% rock flour and 10% lime (West Africa standard)

Project : Master's Thesis

Test Location : **12% Rock flour and 10% Lime** **West Africa Standard**

Sample no : **volume of Mold: 944cm^3** **date: 16/7 /2021**

Sample Description:

Weight of Mold (g)	3388		3388		3388		3388		3388	
Weight of Mold+Wet Soil (g)	5116		5258		5354		5450		5400	
Weight of Wet Soil (g)	1,728.00		1,870,00		1,966.00		2,062.00		2,012.00	
Weight Density (g/cm^3)	1.83		1.98		2.08		2.18		2.13	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	24.8	21.6	38.3	40.1	38.3	38.7	37.6	10.0	24.9	24.1
Weight of can + wet soil (g)	39.5	37.1	54.4	61.5	58.2	52.3	55.0	24.5	46.0	46.9
Weight of Can + Dry Soil (g)	38.8	36.4	53.3	59.8	56.2	51.0	52.9	22.8	42.3	42.9
Weight of Water (g)	0.70	0.70	1.10	1.70	2.00	1.30	2.10	1.70	3.70	4.00
Weight of Dry Soil (g)	14.0	14.8	15.00	19.70	17.90	12.30	15.30	12.80	17.4	18.80
Moisture Content (g)	5.00	4.73	7.33	8.63	11.17	10.57	13.73	13.28	21.2	21.28
Ave. Moisture Content (g)	4.86		7.78		10.87		13.50		21.27	
Dry Density (g/cm^3)	1.7456		1.8345		1.8784		1.9245		1.7575	

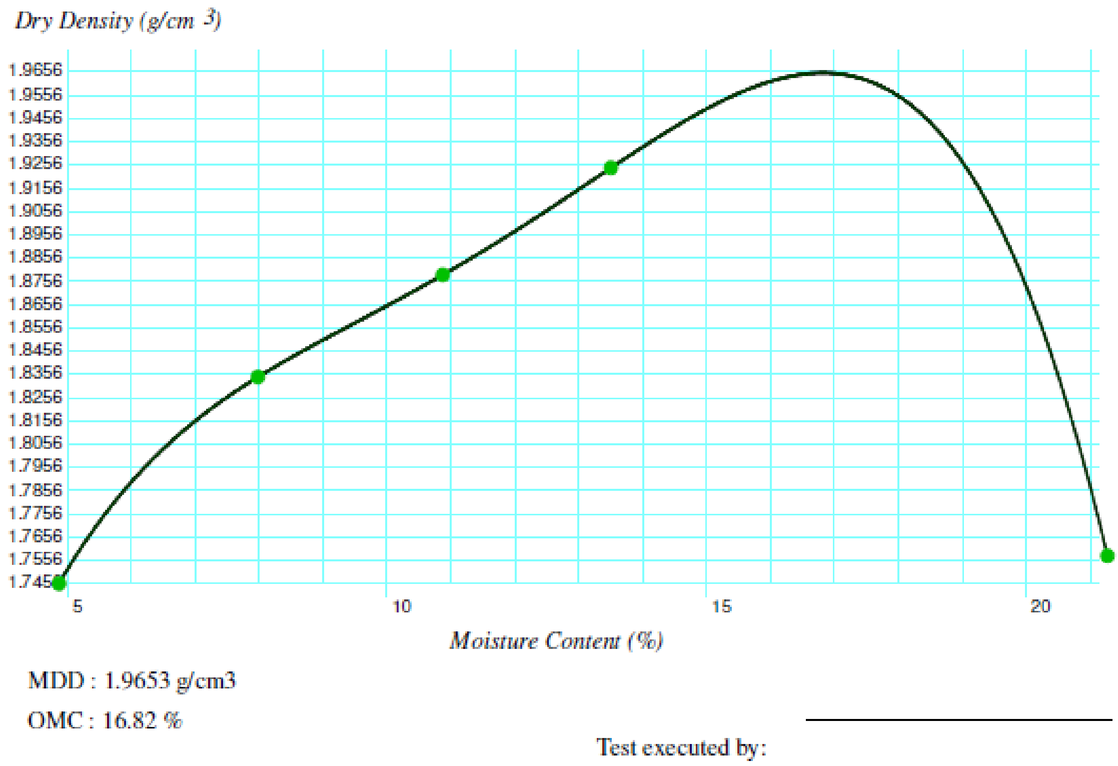


Figure C10: MDD and corresponding OMC of studied soil sample treated with 12% rock flour and 10% lime (WAS)

Table C11: Compaction test result of soil with 0% rock flour and 0% lime (British standard light)

Project : Master‘s Thesis

Test Location : **0% Rock Flower and 0% Lime** **British Standard light**
Sample no : **Volume of Mold: 944cm^3** **Date: 16/7 /2021**
Sample Description:

Weight of Mold (g)	2057		2057		2057		2057		2057	
Weight of Mold+Wet Soil (g)	3547		3635		3731		3850		3810	
Weight of Wet Soil (g)	1,490.00		1,578.00		1,674.00		1,793.00		1,753.00	
Weight Density (g/cm^3)	1.58		1.67		1.77		1 .90		1.86	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	22.9	22.9	25.1	24.5	25.9	24.5	22.9	25.0	23.2	24.0
Weight of Can + Wet Soil (g)	48.0	42.5	50.0	42.3	48.0	45.3	43.3	46.0	43.8	55.4
Weight of Can + Dry Soil (g)	46.3	40.7	47.4	40,5	45.0	42.9	40.2	42.8	39.5	49.8
Weight of Water (g)	1.70	1.80	2.70	1.80	3.00	2.40	3.10	3.20	4.30	5.60
Weight of Dry Soil (g)	23.40	17.80	22.30	16.00	19.10	18.40	17.30	17.80	16.30	25.80
Moisture Content (g)	7.26	10.11	12.11	11.25	15.71	13.04	17.92	17.98	26.38	21.71
Ave. Moisture Content (g)	8.69		11.68		14.38		17.95		24.04	
Dry Density (g/cm^3)	1.4522		1.4968		1.5504		1.6103		1.4971	

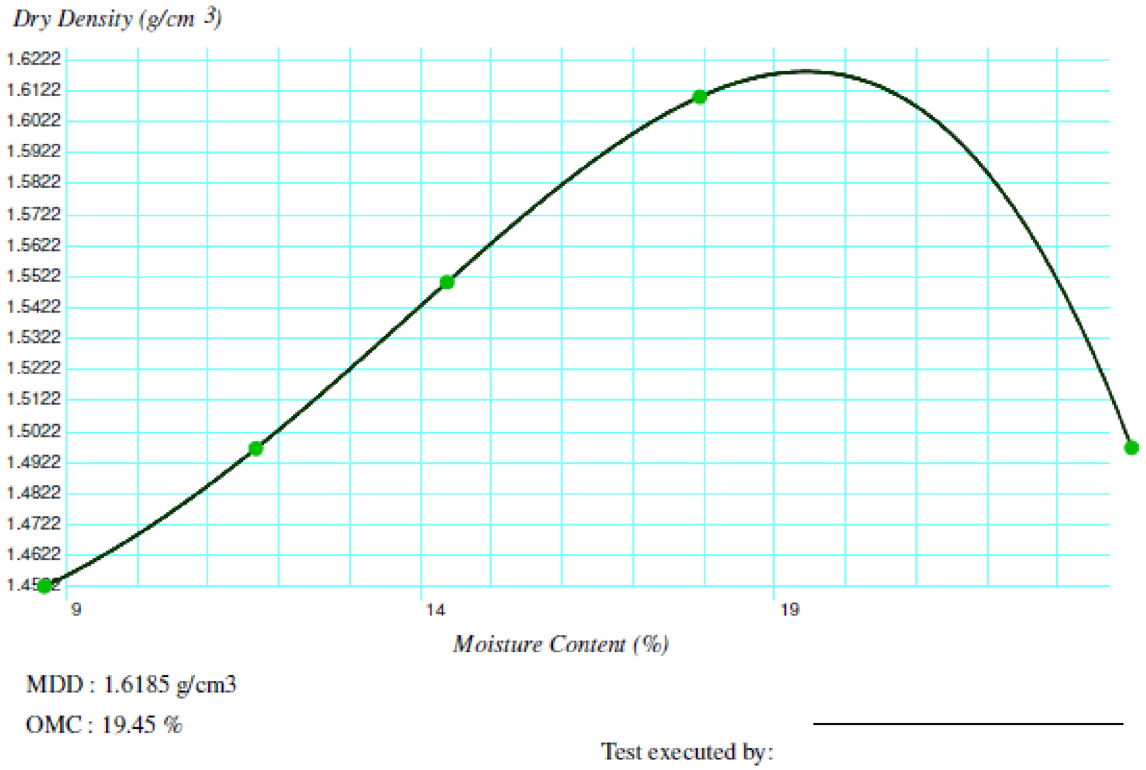


Figure C11: MDD and corresponding OMC of studied soil sample treated with 0% rock flour and 0% lime (BSL)

Table C12: Compaction test result of soil with 3% rock flour and 2.5% lime (British standard light)

Project : Master's Thesis

Test Location : 3% Rock Flour and 2.5% Lime **British Standard light**

Sample no : **Volume of Mold: 944cm^3** **Date: 16/7 /2021**

Sample Description:

Weight of Mold (g)	2057		2057		2057		2057		2057	
Weight of Mold+Wet Soil (g)	3538		3729		3795		3800		3785	
Weight of Wet Soil (g)	1,481.00		1,672.00		1,738.00		1,743.00		1,728.00	
Weight Density (g/cm^3)	1.57		1.77		1.84		1 .85		1.83	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	22.8	23.4	23.8	24.4	25.5	24.0	25.4	24.3	24.2	22.7
Weight of Can + Wet Soil (g)	47.2	55.8	44.1	41.1	46.8	48.5	59.9	47.5	45.7	53.2
Weight of Can + Dry Soil (g)	44.7	54.0	41.9	39.0	43.6	45.0	53.3	43.0	41.1	46.8
Weight of Water (g)	2.50	1.80	2.20	2.10	3.20	3.50	6.60	4.50	4.30	6.40
Weight of Dry Soil (g)	21.90	30.60	18.10	14.60	18.10	21.00	27.90	18.70	17.20	24.10
Moisture Content (g)	11.42	5.88	12.15	14.38	17.68	16.67	23.66	24.06	25.00	26.56
Ave. Moisture Content (g)	8.65		13.27		17.17		23.86		25.78	
Dry Density (g/cm^3)	1.4440		1.5637		1.5713		1.4907		1.4553	

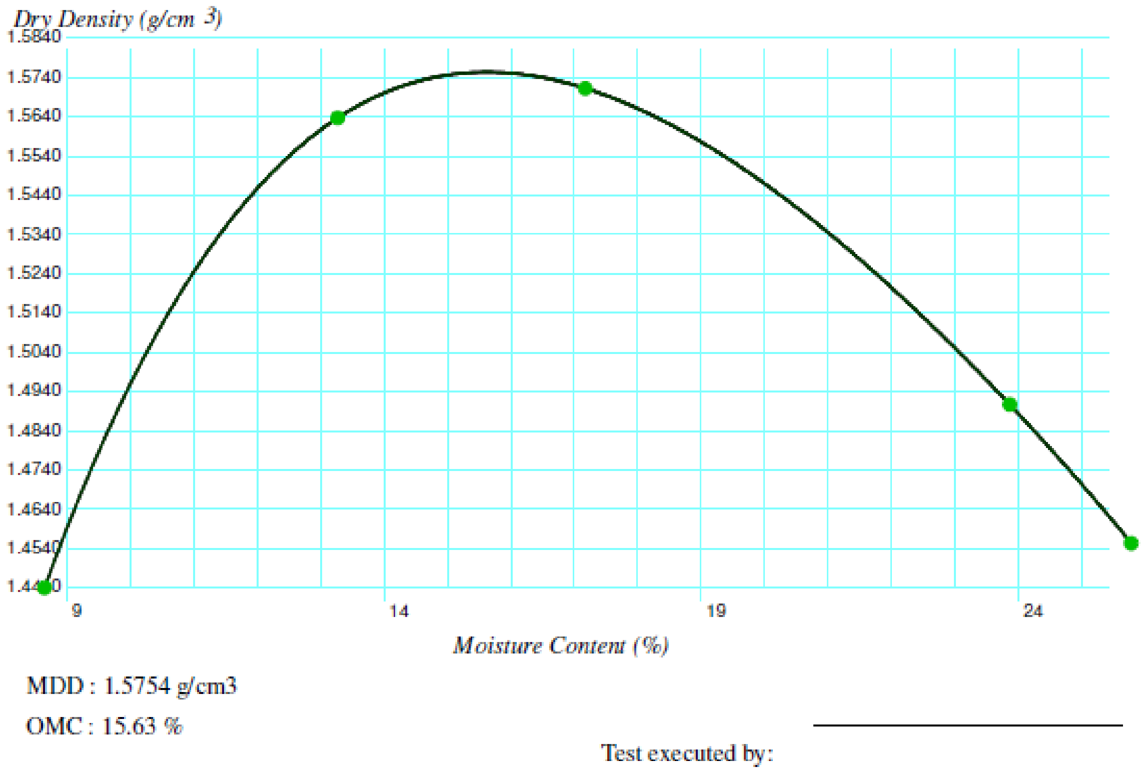


Figure C12: MDD and corresponding OMC of studied soil sample treated with 3% rock flour and 2.5% lime (BSL)

Table C13: Compaction test result of soil with 6% rock flour and 5% lime (British standard light)

Project : master's thesis

Test location : **6% rock flour and 5% lime** **British standard light**
Sample no : **volume of mold: 944cm^3** **date: 16/7 /2021**

Sample description:

Weight of mode(g)	2057		2057		2057		2057		2057	
Weight of mold +wet soil (g)	3550		3629		3760		3778		3748	
Weight of wet soil (g)	1,493.00		1,572.00		1,703.00		1,721.00		1,691.00	
Wet density (g/cm3)	1.58		1.67		1.80		1.82		1.79	
Can number	1	2	3	4	5	6	7	8	9	10
Weight of can(g)	24.0	23.0	24.0	23.0	23.1	23.1	24.6	25.6	23.7	23.0
Weight of can+ wet soil(g)	57.8	45.7	51.8	41.8	39.9	39.3	48.9	49.8	49.1	49.7
Weight of can +dry soil(g)	55.3	44.1	48.2	39.4	37.0	36.9	44.5	45.7	43.4	44.0
Weight of water(g)	2.50	1.60	3.60	2.40	2.90	2.40	4.40	4.10	5.70	5.70
Weight of dey soil(g)	31.30	20.80	24.20	16.40	13.90	13.80	19.90	20.10	19.70	21.00
Moisture content(g)	7.99	7.69	14.88	14.63	20.86	17.39	22.11	20.40	28.93	27.14
Ave. moisture content (g)	7.84		14.76		19.13		21.25		28.04	
Dry density (g/cm3)	1.4666		1.4511		1.5144		1.5035		1.3990	

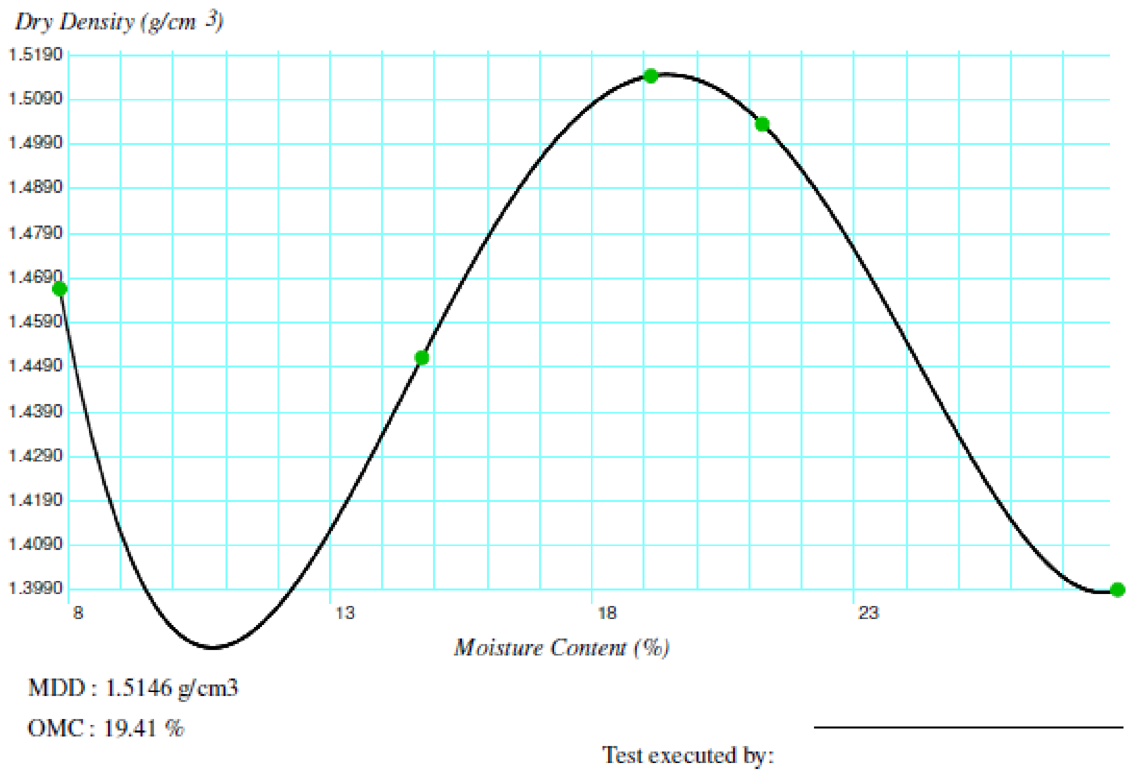


Figure C13: MDD and corresponding OMC of studied soil sample treated with 6% rock flour and 5% lime (BSL)

Table C14: Compaction test result of soil with 9% rock flour and 7.5% lime (British standard light)

Project : Master's Thesis

Test Location : **9% Rock Flour And 7.5% Lime British Standard light**

Sample No : **Volume Of Mold: 944cm^3** **Date: 16/7 /2021**

Sample Description:

Weight Of Mold (g)	2057		2057		2057		2057		2057	
Weight Of Mold +Wet Soil(g)	3560		3615		3695		3779		3806	
Weight Of Wet Soil(g)	1,503.00		1,558.00		1,638.00		1,722.00		1,749.00	
Wet Density (g/cm^3)	1.59		1.65		1.74		1.82		1.85	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight Can(g)	24.0	24.6	24.7	24.3	23.2	24.4	24.5	23.8	23.7	23.4
Weight Of Can+Wet Soil(g)	45.9	49.9	46.0	49.1	46.0	43.2	36.4	42.2	41.5	42.9
Weight Of Can + Dry Soil	44.6	48.6	44.1	47.0	43.3	41.2	34.8	39.8	38.5	39.6
Weight Of Water(g)	1.30	1.30	1.90	2.10	2.70	2.00	1.60	2.40	3.00	3.30
Weight Of Dry Soil (g)	20.60	24.00	19.40	22.70	20.10	16.80	10.30	16.00	14.80	16.20
Moisture Content (g)	6.31	5.42	9.79	9.25	13.43	11.90	15.53	15.00	20.27	20.37
Ave. Moisture Content (g)	5.86		9.52		12.67		15.27		20.32	
Dry Density (g/cm^3)	1.5040		1.5069		1.5401		1.5825		1.5399	

MDD : 1.9069 g/cm3

OMC : 13.67 %

Test executed by: ____________________

Figure C14: MDD and corresponding OMC of studied soil sample treated with 9% rock flour and 7.5% lime (BSL)

Table C15: Compaction test result of soil with 12% rock flour and 10% lime (British standard light)

Project: master's thesis		
Test location:	**12% rock flour and 10% lime**	**British standard light**
Sample no:	**volume of mold: 944cm^3**	**Date: 16/7/2021**
Sample description:		

Weight of mold	2057	2057	2057	2057	2057
Weight of mold+ wet soil (g)	3572	3635	3702	3782	3787
Weight of wet soil (g)	1,515.00	1,578.00	1,645.00	1,725.00	1,730.00
Wet density (g/cm3)	1.60	1.67	1.74	1.83	1.83

Can number	1	2	3	4	5	6	7	8	9	10
Weight of can (g)	25.2	23.9	24.9	24.1	26.3	24.3	24.0	23.0	22.6	32.9
Weight of can + wet soil(g)	47.6	45.7	47.9	45.7	40.3	39.0	43.7	37.4	48.4	63.8
Weight of can + dry soil(g)	45.5	44.1	45.3	43.5	38.4	36.9	40.6	35.1	43.7	57.8
Weight of water (g)	2.10	1.60	2.60	2.20	1.90	2.10	3.10	2.30	4.70	6.00
Weight of dry soil (g)	20.30	20.20	20.40	19.40	12.10	12.60	16.60	12.10	21.10	24.90
Moisture content (g)	10.34	7.92	12.75	11.34	15.70	16.67	18.67	19.01	22.27	24.10
Ave. moisture (g)	9.13		12.4		16.18		18.84		23.19	
Dry density (g/cm^3)	1.4706		1.4919		1.4998		1.5376		1.4877	

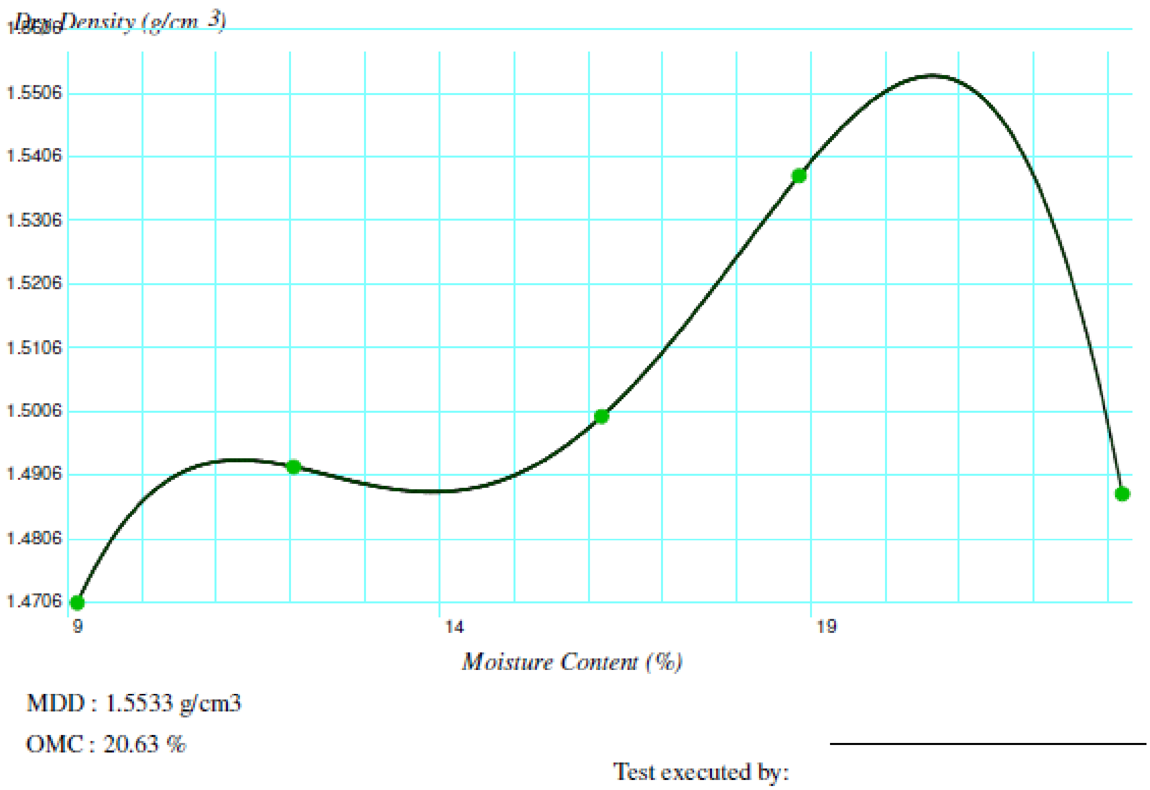


Figure C15: MDD and corresponding OMC of studied soil sample treated with 12% rock flour and 10% lime (BSL)

Appendix D: SUMARRY OF CBR RESULT

Table D1: Standard Proctor Energy Level

SAMPLE No	CBR @2.5 (mm)	CBR @5.0 (mm)	Average CBR Value
0% rock flour 0% lime	2.20	1.98	2.00
3% rock flour 2.5% lime	0.785	0.625	0.705
6% rock flour 7.5% lime	6.91	6.56	6.74
9% rock flour 7.5% lime	17.90	16.56	17.24
12% rock flour 10% lime	19.24	16.56	17.91

Table D2: West Africa energy level

SAMPLE No	CBR @2.5 (mm)	CBR @5.0 (mm)	Average CBR Value
0% rock flour 0% lime	2.04	2.34	2.19
3% rock flour 2.5% lime	4.95	4.84	4.89
6% rock flour 7.5% lime	6.52	7.03	6.78
9% rock flour 7.5% lime	11.31	11.10	11.2
12% rock flour 10% lime	12.09	20.32	16.21

Table D3: British standard heavy energy level

SAMPLE No	CBR @2.5 (mm)	CBR @5.0 (mm)	Average CBR Value
0% rock flour 0% lime	1.65	1.98	1.81
3% rock flour 2.5% lime	2.12	1.98	2.05
6% rock flour 7.5% lime	2.83	3.09	2.05
9% rock flour 7.5% lime	5.34	5.21	5.28
12% rock flour 10% lime	40.53	37.41	38.97

APPENDIX E: UNCONFINED COMPRESSIVE STRENGTH (USC) VALUE AT VARIOUS ENERGY LEVEL

Table E1: Un-confine compressive strength (UCS) Standard proctor energy level

Sample NO	Deformation	Strain E	1-E	Corrected Area (m^2)	Proving ring reading	Loads (kN)	Compressive shear Strength (kN/m^2)
0% rock Flour 0% Lime	6.75	0.0888	0.9111	0.00124	3	0.165	132.57
3% rock Flour 2.5% Lime	2.5	0.0328	0.9671	0.001172	10	0.55	469.05
6% rock Flour 5% Lime	3.5	0.046	0.9539	0.001188	6.5	0.357	300.73
9% rock Flour 7.5% Lime	1.75	0.023	0.9769	0.00116	12	0.66	568.6
12% rock Flour 10% Lime	1.75	0.023	0.9769	0.00116	10.5	0.577	497.53

Table E2: Un-confine compressive strength (UCS) West Africa standard energy level

Sample NO	Deformation	Strain E	1-E	Corrected Area (m^2)	Proving ring reading	Loads (kN)	Compressive shear Strength (kN/m^2)
0% rock Flour 0% Lime	2	0.0263	0.9736	0.001164	18	0.99	850.04
3% rock Flour 2.5% Lime	2.25	0.0296	0.703	0.001168	11.5	0.632	541.24
6% rock Flour 5% Lime	2.25	0.0296	0.9703	0.001168	12	0.66	564.78
9% rock Flour 7.5% Lime	1.75	0.023	0.9769	0.00116	16.5	0.907	781.83
12% rock Flour 10% Lime	2.25	0.0296	0.9703	0.001168	25	1.375	1176.62

Table E3: Un-confine compressive strength (UCS) British standard heavy energy

Sample NO	Deformation	Strain E	1-E	Corrected Area (m^2)	Proving ring reading	Loads (kN)	Compressive shear Strength (kN/m^2)
0% rock Flour 0% Lime	3.5	0.046	0.9539	0.001188	16	0.88	740.27
3% rock Flour 2.5% Lime	4	0.0526	0.9473	0.001197	6.5	0.357	298.66
6% rock Flour 5% Lime	2.75	0.0361	0.9638	0.001176	7.5	0.412	350.59
9% rock Flour 7.5% Lime	2	0.0263	0.9736	0.001164	21	1.155	991.71
12% rock Flour 10% Lime	2.25	0.0296	0.9703	0.001168	24	1.32	1129.56

Appendix F: Durability test result

Table F1: Standard Proctor energy level

Sample NO	Deformation	Strain E	1-E	Corrected Area (m^2)	Proving ring reading	Loads (kN)	Compressive shear Strength (kN/m^2)
0% rock Flour 0% Lime	7.75	0.1019	0.898	0.001262	3	0.165	130.66
3% rock Flour 2.5% Lime	2.751	0.036	0.9638	0.001176	1	0.055	46.74
6% rock Flour 5% Lime	1.75	0.23	0.9769	0.00116	4.5	0.247	213.22
9% rock Flour 7.5% Lime	1.25	0.0164	0.9835	0.001152	7	0.385	333.92
12% rock Flour 10% Lime	1.25	0.0164	0.9835	0.001152	8	0.44	381.62

Table F2: West Africa standard energy level

Sample NO	Deformation	Strain E	1-E	Corrected Area (m^2)	Proving ring reading	Loads (kN)	Compressive shear Strength (kN/m^2)
0% rock Flour 0% Lime	2.25	0.0296	0.9703	0.001168	17	0.935	800.1
3% rock Flour 2.5% Lime	3.5	0.046	0.9539	0.001188	11	0.605	508.94
6% rock Flour 5% Lime	1.25	0.0164	0.9835	0.001152	6.5	0.357	310.07
9% rock Flour 7.5% Lime	1	0.0131	0.9868	0.01149	8	0.44	382.9
12% rock Flour 10% Lime	1.75	0.023	0.9769	0.00116	15	0.825	710.76

Table F3: British standard heavy energy level

Sample NO	Deformation	Strain E	1-E	Corrected Area (m^2)	Proving ring reading	Loads (kN)	Co she Str (k
0% rock Flour 0% Lime	2.75	0.0361	0.9638	0.001176	15	0.825	70
3% rock Flour 2.5% Lime	3.25	0.427	0.9575	0.001184	16	0.33	27
6% rock Flour 5% Lime	2.75	0.0361	0.9638	0.001176	4.57.5	0.412	35
9% rock Flour 7.5% Lime	2	0.0263	0.9736	0.001164	76.5	0.357	30
12% rock Flour 10% Lime	3.5	0.046	0.9539	0.001188	18.5	1.017	85