IMPROVEMENT OF BEARING CAPACITY OF LATERITIC SOIL FOR PAVEMENT SUBGRADE BY ADDITION OF ROCK FLOUR

BY

GBADAMOSI, Olusegun Samuel
MEng/SEET/2017/6775

DEPARTMENT OF CIVIL ENGINEERING
FEDERAL UNIVERSITY OF TECHNOLOGY, MINNA

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ABSTRACT

This study evaluated the improvement of bearing capacity of lateritic soil for pavement subgrade by addition of rock flour stabilized with 0 - 12% rock flour by dry weight of soil at incremental rate of 3% and compacted using British Standard Light (BSL), West Africa Standard (WAS) and British Standard Heavy (BSH) compactive efforts. Results show 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 showed an improvement in the index properties of the mixtures with increasing rock flour. The Liquid limit and Plasticity index reduced from 42 - 32% and 29.30 - 13.48% respectively as rock flour increased from 0 – 12%, while plasticity index of A-7-6 soil with rock flour showed considerable reduction in plasticity indices of mixtures with increasing content rock flour content. 1.802, 1.820 and 1.870g/cm³ were obtained as MDD values and 12.40, 11.90 and 11.60% as OMC values for the natural soil using BSL, WAS and BSH compaction efforts respectively. Highest set of values were obtained at 9% addition of rock flour. BSH gave the highest UCS value of 250.89kN/m², while BSL and WAS gave 180.8 and 218.12kN/m² respectively. Generally, a progressive improvement in soaked and unsoaked CBR values were observed for the stabilized specimen with increasing rock flour content. The least CBR value for soaked and unsoaked conditions were observed at 3 % addition of rock flour. For soaked condition at 3% addition of rock flour, the soaked CBR values obtained for BSL, WAS and BSH were 21.15, 25.93 and 28.21% respectively, while 37.88, 41.74 and 42.73% were obtained for unsoaked CBR in the same other of energy level adopted. In terms of consistency limits, the mixtures did not achieve the required threshold values for subgrade specified as LL < 35 and PI < 12% in local codes suggesting the use of higher rock flour contents to enhance these parameters. As regards to soaked and unsoaked CBR, results from the three energy levels adopted that is; BSL, WAS and BSH, meets the minimum requirements of 10% for flexible pavement subgrade according to NGS, (1997). To optimize their structural strength for subgrade application, the mixtures should be compacted to 100% of the relative densities.

TABLE OF CONTENTS

CHAPTER TWO

CHAPTER THREE

LIST OF TABLES

LIST OF FIGURES

LIST OF PLATES

ABBREVIATIONS, GLOSSARIES AND SYMBOLS

AASHTO	American Association for State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
BS	British Standard
BSH	British Standard Heavy
BSL	British Standard Light
CBR	California Bearing Ratio
CH	Clay of High Plasticity
Gs	Specific gravity
IS	Indian Standard
LL	Liquid Limit
M	mass
MCHW	Manual of Contract documents for Highway Works
MDD	Maximum Dry Density
MIT	Massachusetts Institute of Technology
M_s	Mass of compacted soil
NBS	Nigeria Bureau of Statistics
NGS	Nigeria General Specifications for Roads and Bridges
OMC	Optimum Moisture Content
OPC	Ordinary Portland Cement
PI	Plasticity Index
PL	Plastic Limit
RHA	Rice Husk Ash
SC	Sandy Clay

UCS	Unconfined Compressive Strength
USCS	Unified Soil Classification System
V_s	Volume of mould
WAS	West Africa Standard
ρ_b	Bulk density
ρ_d	Dry density
W	Natural moisture content
γ	Weight of the compacted moist soil
γ_d	Dry density

CHAPTER ONE

1.0 INTRODUCTION

1.1 Background to the Study

Natural soils vary in its properties and are mostly heterogeneous. These properties changes at different depth in the ground because of several reasons which include depositional environment, physical environment and extent of weathering (Elkateb *et al.*, 2003; Lumb, 1974; Jones *et al.*, 2002). Soil has wide range of variation in its structure, texture and composition at different depositions, which influences its index, engineering and geotechnical properties. Upon initial deposition. Soil undergo continuous modification due to external stresses, chemical reactions, weathering (Uzielli *et al.*, 2006). The inherent properties of some soil types limits their application in some engineering projects. Therefore, the need to modify these properties to make it fit for specific engineering purpose becomes inevitable. Nigeria, being a country that depends mainly on land transportation of goods and person requires functional, serviceable and durable roads. Several deformations and failures seen on Nigeria roads are due to structural failure of the road component, overloading beyond the design load, poor construction, among others (Ndefo, 2012).

AASHTO (1986) classified soil with respect to their behaviour as subgrade material into seven groups, which are A-1, through A-7. Groups A-1, A-2 and A-7 are further classified into A-1-a, A-1-b, A-2-4, A-2-5, A-2-6, A-2-7 and A-7-5, A-7-6 respectively. A-7 soils are Silt-clay materials having more than 35 % of total sample passing No. 200 sieve with Liquid Limit of 41 minimum and Plasticity index of 11 minimum. This class of material is largely available in Nigeria but are challenging to geotechnical engineer in its application for road construction. A-7-6 soil are generally clayey according to AASHTO

(1986) classification, this limits its application for road construction because of its poor strength and susceptibility to volumetric changes on exposure to moisture. This class of soil are readily available across the country. The challenge of instability of clay is as a result of swelling nature of expansive clay material thereby making the soil unsuitable material for construction in foundation of buildings, highway, railway or any other engineering structures (Ogunribido and Abiola, 2015). Several methods have been adopted in the past to stabilize A-7-6 soil so that it meets certain criteria. Several research works have been done using different methods of soil stabilization. Some of the methods have shown to be efficient and effective and at varying cost, while sourcing the stabilizing agent have rendered some of the findings impracticable.

Modern constructions involve high speed road and rail networks that require highly stable retaining and foundation systems (Reddy *et al.,* 2017). Silt and clay are predominant in natural soils having high content of plastic fines which are responsible for large scale of deformation under traffic loads when such soil is in saturated state and their subsequent settlement leads to road failures (Satyanarayana and Pradeep, 2013). The deplorable nature of Nigerian roads is mostly caused by failure of the subgrade, subbase or base course, other possible cause is overloading of the pavement beyond design considerations (Afolayan and Abidoye, 2017: Ndefo, 2012). In addition, potholes, pavement surface wash, depressions of roadway, block and longitudinal cracks, drainage collapse are also responsible for road failure in Nigeria. Structurally stable road with adequate structural components is critical to durability and functionality of road.

Developed and developing countries produces rock flour as an industrial waste while processing coarse aggregate from rock from crusher plants. Rock flour can also be obtained as an effluent while drilling through rock. Rock flour as the name implies is in powder form, having angular constituent particles. Rock flour is a stable material at different degree of moisture content, they contain mineral such as quartz, silica and feldspar (Reddy and Moorthy, 2002). It has varying applications in its use for infrastructure developments such as a fill material in Highway construction, retaining material without reinforcement (Satyanarayana and Pradeep, 2013). Rock flour was found to be a good stabilizing material for lateritic soil when used with Ordinary Portland Cement to stabilize lateritic soil (Ogunribido and Abiola, 2015).

1.2 Statement of the Research Problem

A-7-6 soil though a poor material for road construction it is abundantly available across Nigeria (Itafe, 2020). There is need to effectively utilize this grade of lateritic soil by improving its, geotechnical and engineering properties. Huge cost, time lag and high rate of wear on construction equipment is associated with hauling material through long distances, which also influences construction duration. Considering high availability and poor nature of this grade of lateritic soil, the need to find ways of stabilizing this poor grade of soil with local additives to meet requirements for road subgrade becomes inevitable (Amadi, 2010; Okunade, 2010; Mohammed and Alhaji, 2015). It is therefore necessary that the properties of this soil grade is improved with rock flour. Rock flour being an industrial waste in quarry cannot be rendered an absolute waste therefore, it is intended to check its suitability as replacement for cement, lime and other additive in soil treatment. It is expected that Rock flour used as soil additive should have high tendency of reducing construction cost hence its cheaper when compared with cement or lime.

1.3 Aim and Objectives of the Study

The aim of this research work is to evaluate the improvement of bearing capacity of lateritic soil for pavement subgrade by addition of rock flour. To achieve the aim, the objectives of this work are:

i. To determine the Atterberg limits i.e., Liquid Limit, Plastic limit and plasticity index of the natural soil samples and samples stabilized with varying percentages of rock flour.
ii. To determine the California Bering Ratio, (CBR) value, Unconfined Compressive strength (UCS) value of the natural soil sample and samples stabilized with varying percentages of rock flour.
iii. To determine the durability of samples stabilized with varying percentages of rock flour.

1.4 Scope of the Study

Rock flour and lateritic soil were sourced in Chancahaga and Agaie local government areas respectively in Niger state, Nigeria. The laboratory tests were carried out at the Civil Engineering laboratory of Federal University of Technology, Minna. All tests were carried out in accordance with procedure outline in BS 1377 (1990) for natural samples and BS 1924 1990, for the stabilized specimens using 3 - 12% of rock flour by dry weight of natural soil, at incremental rate of 3%. All tests on the natural soil and stabilized samples using three energy levels that is; BSL, BSH and WAS, following procedures outlined in BS 1924-2: (1990) and NGS (1997) respectively.

1.5 Justification of the Study

Land transportation is mostly used for commuting persons and goods in Nigeria through over 200,000 km stretch of road (Ndefo, 2012). The need to construct functional and durable roads founded on strong and stable subgrade through 923,768 km^2 land mass become necessary irrespective of soil class which construction Engineers come across. This research work sought to provide alternative to the already known additive such as cement, lime and iron tailings among others for the improvement of poor soil bearing capacity. In addition, this study hope to provide an efficient utilization of quarry dust which is regarded as industrial waste in quarries. To provide an alternative to stabilization of A-7 soils and other fair to poor soils such as A-4, A-5 and A-6. In general, the results of this study would provide an easier and cheaper technique for improving bearing capacity of A-7-6 soil.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Nature of Clay

Instability of clay material renders it unfit in its natural state for road construction due to its volumetric changes in and off wet seasons. Swelling and shrinking nature of clayey material has been a major challenge to geotechnical engineers. In effect, the stability of building and highway foundations have undergone several studies due to its continuous consolidation under load. Most roads in Nigeria are barely functional and serviceable because of their current state of pronounced deformation. While it is a general knowledge that Nigerian roads are overstressed due to overloading, issues of construction material quality significantly affect our roads. Though, Nigeria has abundant deposit of laterite and lateritic materials but some grades of lateritic soil requires modification to make them fit for some specific engineering purposes. Utilization of local materials is critical to having cost effective roads especially in developing country like Nigeria with a GDP of 2.01% for the first quarter of 2019 according to Nigeria Bureau of Statistics. Another cause of poor state of Nigeria road is operations of quacks in the field of road construction who have been using laterite and lateritic materials arbitrarily without referring to Nigerian General Specification for Roads and Bridges in (1997) for material specifications. Clays are formed from weathering of primary rock, the constituent mineral of clay is called secondary silicate. Clay are fine graded and flaked shape, having small mineral particle size of (<0.002 mm). The negative electrical load on the crystal edges and positive electrical load on the face separates it from gravel, sand and silt (Nazile, 2018).

Engineering structures such as bridges, highway, dam, tunnel and other civil engineering structures are founded on soil. The suitability of soil is a major requirement before embarking on foundation works, basically to check for its properties and behaviour under loading (Surendra and Sanjeev, 2017). Laterite are mostly used as imported fill material from borrow pit for the purpose of subbase and base course in many road projects (Amadi, 2010; Okunnade, 2010). Lateritic soils are highly weathered soils which are formed from material having high concentrate of hydrated iron oxide and aluminium (Amu *et al.*, 2011). Index, geotechnical and engineering properties of lateritic soil are required for the classification of soils. Soils are either classified using AASHTO or Unified soil classification system (USCS). The geotechnical properties of lateritic soil such as specific gravity, Atterberg limits, swelling potential and petrographic potentials are influenced by the mineral composition of the soil (Amadi *et al.,* 2012). Saliu (2018), investigated the correlation between unified and AASHTO soil classification using samples from seven locations within South West Nigeria. He observed that there was no major difference between results obtained from the two soil classification systems, therefore, they can be used interchangeably. Other soil classification systems are U.S. Department of Agriculture (USDA), Burmister Soil Identification System and Massachusetts Institute of Technology (MIT). AASHTO classifies soil into seven major groups which are A-1 through A-7 and A-8 otherwise known as peat or organics with groups A-1, A-2 and A-7 having subgroups. These groupings are done using their sieve analysis, Liquid Limit and Plasticity index. Unified Soil Classification System are broadly categorized into two groups which are, Coarse Grained Soils; Gravels (G) and Sands (S) less than 50% passing through No 200 sieve and Fine-Grained Soils; Silts (M) and Clays (C) with 50% or more passing through No 200 sieve.

2.2 Nature and properties of A-7-6 soil

According to AASHTO soil classification system, A-7 soils are subdivided into, A-7-5 and A-7-6. A-7-6 soils are those having minimum 36% passing through sieve No 200, minimum liquid limit of 41 and minimum plasticity index of 11. Their mineral composition are quarts, silicate, aluminium, oxides, hydroxides, feldspar and organic materials (Zaid *et al.,* 2017). This class of soil is typically called clayey soil. Clayey soils are poor in strength, they are quite challenging to work with especially in highway construction. Stability of clay is dependent on the mineral constituent and degree of moisture content. Clay with kaolinite are less active and more stable whereas clay containing montmorillonite are very active and subject to volumetric changes depending of the degree of moisture content (Nazile, 2018). A-7-6 soils are prone to volumetric changes upon application of moisture which renders it susceptible to swelling and shrinkage in and out of wet seasons. This makes the superstructure experience continuous differential movement and excessive settlement causing damage to structural elements, aesthetic features and the foundation system (Monica and Sanjeev, 2013; Onuoha *et al.*, 2014). Deformation and poor strength characteristics of clay soils makes it challenging to erect structure on it. Although A-7-6 soils are regarded as poor in its engineering applications, it can be stabilized for the purpose of using it as a subgrade material.

Continuous increase in axle load and vehicular volume necessitate the need to have roads that are very stable, durable without escalating the construction cost (Athanasopoulou and Kollaros, 2011). A-7-6 being a weak soil have California Bearing Ratio (CBR) value ranging from 6.38 – 8.24% for soaked and unsoaked soil samples (Ewa *et al.,* 2018; Charles *et al.,* 2018). This does not meet the 30-80% requirement for subbase material as stated in the Nigerian General Specification for Roads and Bridges in (1997).

2.2.1 Effect of clay minerals on geotechnical properties

Certain features of the clay affect the structure of the soil, which determines its properties such as shear strength, hydraulic conduction, settlement and swelling. These features include surface anion and cat-ion exchange capacity and isomorphic substitution, isomorphic is when different atom replaced either octahedral or tetrahedral sites of the clay structure. The specific surface area is the property of solids, which is defined as the total surface area of a material per unit of mass. With the separation of hydroxyl ions from the clay surface, which results in crystal head, anions subsequently attach to the surface and organic molecule content causes an electrical load imbalance. This imbalance results in clay's extreme affinity to water and cat-ions in environment.

Water is a dipolar molecule, namely, it has one positive and one negative charge. The surface of the clay crystal is electro-statically held to the water molecule. In addition, water is held to the clay crystal by hydrogen bonding. Also, negatively charged clay surface attract cat-ions in the water. The cat-ion/anion changes in the clay are different between clay minerals. Therefore, it is expected that the clay that attracts more water molecules to the surface will have more plasticity, more swelling/shrinkage and more volume change, depending on the load on it. Thus, water influences clay minerals. For example, the water content changes consistency limits and this affects the ground plasticity. Ultimately the change in clay plasticity directly affects the mechanical behavior of the soil. Therefore, the behavior of clays is affected by the individual clay particle arrangements and pore water content. The surface of clays are negatively charged, and so they tend to absorb the positively charged cat-ions implore water. In this way, the cat-ions on the surface of a clay particle that are entering the water spread into the liquid. This spreading is called the double layer. Briefly, the cat-ions are distributed around the negatively charged surface of the clay particles. Briefly, the cat-ions are distributed around the negatively charged surface of the clay particles, with the greatest density near the surface and decreased density with increasing distance from the surface. The cat-ions form a positively charged layer and the double layer arrangements of the clay particles, and hence, the physical and mechanical properties of the soil are also affected. The interaction of these forces controls the engineering behavior of soils to a great extent. At the same time, this interaction leads to the formation of different compositions and settlements in the soil planes, which are defined as structures in clay soils (Uzielli, 2006). Environmental temperature, precipitation, ground water level and pH and salinity all play roles in clay properties, as well as in the conversion of rock into clay. Different clay may be derived from the same rock under different environmental conditions.

In geotechnical engineering, it is important to identify a clay type, as the type directly affects the important properties of clay, such as Atterberg's limit, hydraulic conductivity, swelling shrinkage, settlement (compression) and shear resistance. Atterberg's limits, known as consistency limits, define the relationship between ground particles and water and the state of the soil relative to varying water contents. With increasing moisture content, clay changes from solid state, to semisolid state, to plastic state and to liquid state, is equivalent to the volume of water lost around the liquid and plastic limits, as the clay transitions from liquid to dry, and if the decrease in water content continues, no reduction in volume is observed. This limit value is called the shrinkage limit. Therefore, the shrinkage limit is the moisture content at which the soil volume will not reduce further if the moisture content is reduced. The plastic limit is the moisture content at which the soil changes from a semisolid to a plastic (flexible) state. The liquid limit is the moisture content at which the soil changes from a plastic to a viscous fluid state (Lumb, 1974). In geotechnical engineering, the liquid and plastic limits are commonly used. These limits are used to classify a fine-grained soil, according to the Unified Soil Classification system and AASHTO system of classification.

2.3 Rock Flour

It is estimated that 6.0m³ of rock flour is produced for every 30m³ of rock crushed representing 20% of the crushed rock (Reddy and Moorthy, 2002). Reddy *et al.* (2017), established that the physical properties of rock flour varies depending on the parent rock from which the rock flour is obtained when they studied rock flour from Basalt, Charnockite and Granite. Reddy *et al.* (2017), observed that Rock flour of Charnockite origin has higher angle of internal friction over rock flour from granite and Basalt. Typically, rock flour are stable material in saturated state and permeable to moisture through it particles. They predominately consist of mineral such as quartz, silica and feldspar (Reddy and Moorthy, 2002). Rohini *et al.* (2018), found rock flour to be effective in reducing Liquid limits, Plastic limit and Plasticity index of the test soil, while increasing the Shrinkage limit. Rohini *et al.* (2018), further concluded that addition of quarry dust to expansive soil decreases the cohesion and increases the angle of internal friction in the soil-rock flour mix. In an investigation carried out by (Ogunribido and Abiola, 2015) they found rock flour to be a good stabilizer, though not as effective as cement, this outcome was corroborated by (Rohini *et al.*, 2018; Ademila, 2019).

2.4 Soil - Rock Flour Stabilization

Several research works have been done in a bid to modify A-7-6 soil in order to make it suitable for engineering purposes, due to its predominance in Nigeria. Soil stabilization process is a basic requirement when road is required to be functionally durable and serviceable throughout its life span especially when using a weak soil as subgrade. Techniques for soil stabilization have been introduced over the years with the sole purpose of improving the engineering properties of weak material such as clay, to meet specific engineering requirements (Amadi, 2010; Ogunribido, 2012; Amu *et al.,* 2011; Okunade, 2010; Mohammed and Alhaji, 2015). Reduction in clay ability to swell, better soil gradation and increased durability and California Bearing Ratio (CBR) value are mostly improved when clay is stabilized. Sometimes, stabilization could be done to improve working area for construction operations where clay deposit is predominant (Monica and Sanjeev, 2013). Cementing material were found to perform effectively as pozzolana in its reaction with cement, pozzolans also improves mechanical properties of stabilized samples when cement is used as a stabilizer in clay (Cong *et al.,* 2014). Stabilization of clay can be achieved mechanically or chemically. In most cases, the two methods are combined to achieve a satisfactory result.

Rice husk, fly ash, iron tailings among others have been used for the stabilization of A-7-6 soil (Alhassan, 2008; Alhassan and Alhaji, 2017). Some of the research outcomes are quite satisfactory while others seem to be inadequate for the purpose of subbase course in road pavement design. Ewa *et al.* (2018), investigated the influence of rice husk ash from different mills on road subgrade properties. It was found out that rice husk ash improves the California Bearing Ratio (CBR) value of the subgrade. The extent of stabilization was dependent on chemical composition of the Rice husk Ash. Ogunribido and Abiola (2015), made a comparison between cement and rock flour as stabilizing agents in improving engineering properties of lateritic soil. It was observed that both are good stabilizing agents, but cement gave higher strength parameters. Unlike cement stabilized samples, the Shear Strength and unconfined compressive strength for rock flour stabilized samples reduced with increasing percentage of rock flour. Reddy *et al.* (2017), studied the properties of rock flour from different parent rock and their suitability as material for fill in reinforced structures. Rock flour samples were taken from the following parent rocks; Basalt, Charnockite and granite. Results of the engineering properties on the samples showed that, rock flour of Charnockite origin has more gravel sized particles, while Basalt has higher fine particles. Charnockite and Granite have satisfactory angle of internal resistance. It was therefore concluded that Granite and Charnockite rock flour were considered satisfactory as frictional fill in reinforced soil structures.

Garata *et al.* (2014) and Narayana *et al.* (2016), evaluated the potential of rock flour for use as fill material in reinforced soil structures. They established the prospect for the use of rock flour in highway pavement design and construction. The samples of rock flour used for the research were classified as well graded sand according to IS 1498 (1987). Rock flour was found to possess coarse grained material with more sand size particles having good frictional properties. It was observed that the rock flour has a coefficient of permeability K, as ($k = 2.4\times10^{-3}$ cm/s - 4.31×10^{-4} cm/s), thereby making it permeable to water (Reddy and Moorthy, 2002; Garata *et al.*, 2014; Narayana *et al.*, 2016). The angle of internal friction of coarse sand studied was obtained as ($\emptyset = 35^o$), while rock flour gave a higher value angle of ($\emptyset = 47^o$), this results was found to meet the requirements as frictional fill in construction of reinforced structure (Narayana *et al.*, 2016). Rock flour mobilizes 88 to 93 % of angle of internal friction as interfacial friction angle with geotextiles (Reddy and Moorthy, 2002). Rock flour of granitic origin possess higher frictional characteristics when compared with rock flour from Leptynite. It was concluded that Rock flour can be effectively used for filling in construction of Reinforced Earth Structures such as Reinforced Soil Bed in road pavement.

Ogunribido (2012), researched on the effects of rock flour on some engineering properties of lateritic soil at incremental rate of 2%. The samples were taken from two borrow pits along Igbatoro road in Southwestern Nigeria and were stabilized using rock flour using 2

- 10% by weight of dry soil. It was observed that the unconfined compressive strength and CBR values of rock flour stabilized samples gave improved values, but are lower when compared with cement stabilized samples (Ogunribido and Abiola, 2015). It was therefore concluded that rock flour is a good stabilizer for laterite at optimum percentage of 4%. Satyanarayana and Pradeep (2013), studied the performance of crusher dust as fill and subgrade material as a replacement for red soil. They observed that the composition of crushed dust and sand are similar, though it offers more shear strength at wider range of moisture content. Also, crusher dust has higher CBR value and angle of shearing resistance. Sakshi *et al.* (2018), stabilized subbase soil using crusher dust for flexible road pavement. They observed that the plasticity characteristics of the sample reduced with increase in crusher dust, while the California Bearing Ratio (CBR) value increased. Crusher dust was used at 5% incremental rate up to 50% by weight of dry mass of soil. Compaction characteristic test was done in accordance with (IS: 2720 - Part 8; 1983). Results showed that the maximum dry density (MDD) got to a peak value of 2.13g/cm^3 at 9% Optimum Moisture Content (OMC) with 35% replacement. A peak value of 38% California Bearing Ratio (CBR) was obtained at 35% replacement of gravel.

Ademila (2019), evaluated the structural stability of selected lateritic soil samples stabilized with rock flour along Ibadan-Iwo-Osogbo Highway, Southwestern Nigeria. He used rock flour as stabilizing agent to improve the geotechnical properties of the test soils at 2, 4, 6, 8 and 10% by dry weight of soil. He was able to establish the ability of rock flour in reducing plasticity of soil. In addition, the maximum dry density (MDD) of all stabilized samples experienced general improvement, while a corresponding reduction in Optimum Moisture Content (OMC) was observed. The strength properties of the stabilized samples improved as CBR values for all test specimen increased with increasing rock flour content. This outcome corroborates result obtained by (Ogunribido and Abiola, 2015). UCS and shear strength parameters of the stabilized specimen improved optimally at 8% of rock flour addition, but further addition of rock flour beyond 8% resulted in reduction of UCS and shear strength parameters. He therefore concluded that rock flour can be used as a sole lateritic soil stabilizer, and soil-rock flour mix is a cheaper alternative to river or mining sand for lateritic soil stabilization.

Malaya *et al.* (2018), evaluated the effect of quarry dust on compaction characteristics of clay using quarry dust of different graduations. Which revealed that quarry dust is a good stabilizer for clay with optimum performance at 30% replacement. The research work showed that mixes prepared using intermediate range of particles indicated higher MDD, with higher quarry content, OMC values reduced. Mixes were prepared using quarry dust and sand of similar gradation with high compressible clay for all proportion, the MDD value for both were somewhat similar though OMC was observed to increase at higher content of quarry dust. Generally, rock flour or quarry dust are good stabilizer, but their performance when used to for sub-base material is dependent on its percentage replacement and its gradation. Murty *et al.* (2016), researched on the utilization of soil with low CBR value for flexible pavements for low volume roads with robo sand stabilization. Robo sand are also categorized as fine aggregate, produced by crushing gravel, stone or slag, having aggregate seize passing through 4.75mm sieve. Robo sand was used at 5 - 20% replacement of dry weight of soil, with the Unconfined Compressive Strength value ranging from 140.233 to 149.06 kPa. It was discovered that the Optimum Moisture Content and Maximum Dry Density varied between 12.9 - 12.50% and 1.779 - 1.7142g/cm^3 respectively for mixture of robo sand between 10 - 20%. It was observed that the CBR value increased with increasing percentage of robo sand, in which the researchers adopted 25% of robo sand stabilization for local soil due to economic considerations.

CHAPTER THREE

3.0 MATERIALS AND METHODS

3.1 Materials

Materials used for this research work are; lateritic soil, rock flour and distilled water. Materials were carefully transported to the Civil Engineering Laboratory of Federal University of Technology, Minna for analysis. This research was carried out in stages to achieve the outlined objectives of the study.

3.1.1 Study Soil

Lateritic soil sample for this study is a dark brownish soil, which was collected as disturbed samples from identified borrow pit at depths varying between 1 – 2m in Agaie local government, Niger state (Latitude 9.06658^0 N and longitude 6.36958^0 E) Nigeria. Samples obtained for this research work were transported to the Civil Engineering Laboratory of Federal University of Technology, Minna for analysis.

3.1.2 Rock Flour

Rock flour for this research work was obtained as an effluent of drilling process of a crystalline basement rock at a borehole drilling site in Tunga, Chancahaga local government, Minna, Niger state, Nigeria. It is a powdery whitish non-plastic material. Only fractions passing through BS sieve No. 200 (75μm) were used for the purpose of the.

3.1.3 Water

Water for the test was obtained from borehole at the Civil Engineering laboratory Federal of University of Technology, Minna. The water used was colourless, odourless and free from visible impurities in accordance with BS EN 1008:2002.

3.2 Soil-Rock Flour Mixtures

Rock flour was added to the natural soil at 3 - 12% by dry weight of the soil sample, at incremental rate of 3%. The Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI) were determined for the natural soil sample and the rock flour stabilized specimens. Compaction characteristics of the natural soil and rock flour stabilized samples were also obtained using three energy levels that is; British Standard Light, British Standard Heavy and West Africa Standard, following procedures outlined in BS 1924-2: (1990) and NGS (1997). Also, Unconfined Compressive Strength (UCS) and shear strength of the natural soil and rock flour stabilized samples were obtained according of procedure outlined in BS 1377 (1990) Part 7, using three energy levels that is; BSL, BSH and WAS. California Bearing Ratio (CBR) was also determined for soaked and unsoaked samples following procedures outlined in ASTM 1883 and AASHTO T193-81.

In line with the aim and objectives of this research work, The following tests were carried out; Sieve Analysis Test in accordance with BS 1377-2 (1990); Compaction Tests in accordance with BS 1377-2 (1992), BS1924-2: 1990 for stabilized sample and NGS, (1997) for WAS compaction; California Bearing Ratio Test (CBR) in accordance with BS 1377-2 (1992); at different mix percentages of 3 - 12%, Atterberg Limits Test; was conducted using cone penetration test, to get the index properties of the stabilized soil as outlined in B.S. 1377 (1990). Below is a list of test carried out;

i. Specific Gravity
ii. Particle Size Distribution
iii. Consistency Limits (Liquid Limit, Plastic Limit and Plasticity Index)
iv. British Standard Light (BSL), West Africa Standard (WAS) and British Standard Heavy (BSH) compaction

v. Unconfined Compressive Strength (UCS) for BSL, WAS and BSH.

vi. California Bearing Ratio (Soaked and Unsoaked)

3.2.1 Particle size distribution

This test was carried out to determine the particle size distribution of the soil sample in accordance with BS 1377-2:1990. A representative air-dried sample of the natural soil weighing 300g was washed and sieved using 4.75mm BS sieve thereafter, the portion of the collected sample was placed in the oven to dry. Set of test sieves were prepared and arranged in order, with size 5mm the top, and sieve 0.075mm at the bottom. A receiver pan was placed under all of the sieves to collect samples, the weight of all the sieves and the pan were measured separately. The prepared sample was poured into top of the set of sieves. The stack in the mechanical shaker were properly fixed, the timer was set between 10 and 15 minutes before switching on the shaker. As the shaker stopped, masses of each sieve and retained soil/material was taken.

Mass of soil retained = (Weight of sieve + sample) – Weight of sieve (3.1)

% retained = Mass of soil retained ÷ Total mass of soil x 100 (3.2)

% fine = 100 – Cumulative % retained (3.3)

Finally, the percentage passing was plotted against B.S. sieve sizes using logarithmic graph. The result of the particle size distribution is presented in Appendix ‘A’.

3.2.2 Determination of specific gravity

Specific gravity (G_s) of soil is the ratio of the weight of soil solids to the mass of an equal volume of distilled water. Specific gravity links the index property of soil with mineral or chemical composition. This test was carried out according to procedures outlined in

BS 1377 (1990) test (B) for fine–grained soils. Weigh of empty and dry volumetric flask / Pycnometer to the nearest 0.01 gram was recorded as W_1. 100 grams of oven dried soil was placed into the Pycnometer. The weight Pycnometer and dry soil to the nearest 0.01 gram was recorded as W_2. Water was added to Pycnometer until about it is two thirds full. The mixture was gently shacken and, additional water was added into the Pycnometer until the bottom of the meniscus is exactly at the volume mark. Weigh of the Pycnometer was recorded as W_3. Thereafter, the pycnometer was emptied and washed, it was then filled with water up to the mark and weighed as W_4. This procedure was repeated three times. Specific Gravity (G_s) of Soil was then computed by dividing the weight of soil by the weight of an equal volume of water as in equation (3.4).

$$Gs = \frac{w2-W1}{(w4-w1)-(w3-w2)} \tag{3.4}$$

Where;

W_1 = Weight of Empty Pycnometer (gm)

W_2 = Weight of Pycnometer + Soil (gm)

W_3 = Weight of Pycnometer + Soil + Water (gm)

W_4 = Weight of Pycnometer + Water (gm)

3.2.3 Consistency limits

Consistency limits or Atterberg limits is a measure of water contents at which sample soil changes from one state to the other. Depending on its water content fine-grained soil can exist as liquid, plastic, semi-solid, or solid state. The consistency limits obtained for the natural and stabilized samples are; Liquid Limit (LL), Plastic Limit (PL) and Plasticity

Index (PI). These tests were conducted according to the procedures outlined in B.S. 1377 (1990).

3.2.3.1 Liquid limit

Liquid limit (LL) defines the state whereby fine grain soil no longer flows like liquid. It is determined as the moisture content, expressed as a percentage of the weight of oven dried soil at the boundary between liquid and plastic states of consistency. Take a sample of the soil sufficient size to test for specimen for liquid limit and plastic, 200gm of sample passing through sieve 425µm. Transferred the soil sample to a glass plate, water was added and mix thoroughly with two spatula until it form a homogeneous paste, so as the first penetrometer reading will be 15mm or above. Put a portion of the mix soil into cup using the spatula and taking care of trap air, gently tapping the air against a firm surface if necessary. Strike off excess soil with a straight edge to give a smooth surface. With the penetration cone in position lower the dial gauge to contact the cone shaft and record the reading of the dial gauge to the nearest 0.1mm. Lift the cone and clean carefully. Pour back the sample on the glass plate, add some distilled water and repeat the process until the sample exceed 200mm. Take moisture content of 20gm in moisture cans for each trial and then transferred to Oven for 24hrs. The trials is carried out for 3 or more times, and calculated using equation 3.5.

$$W = \frac{M2-M3}{M3-M1} \text{ X } 100(\%) \tag{3.5}$$

Where

W = Moisture content

M_1 = mass of the moisture can in gm

M_2 = mass of the Moisture can + wet sample in gm

M_3 = mass of the Moisture can + dry sample in gm

The relationship between moisture content and penetration was plotted to obtain the liquid limit.

3.2.3.2 Plastic limit

Plastic limit (PL) of a soil is the moisture content at which a soil begins to behave as a plastic material. 20gm of air-dried soil, which passed thorough 425 mm sieve was mixed with distilled water thoroughly in an evaporating dish to form uniform paste. Several ellipsoidal shaped soil masses was formed by squeezing the soil between your fingers. One of the soil masses was rolled on the glass plate with my hands. The pressure of rolling was just enough to make thread of uniform diameter throughout its length. Rolling continued until a thread diameter of 3 mm was obtained. The process continued until the thread crumbled when the diameter is 3 mm. Crumbled thread samples were collected for moisture content determination. The test was repeated for at least 3 times. The average value of the moisture content was taken as the Plastic Limit (PL) of the soil. The value is expressed in the nearest whole number in percentage.

3.2.3.3 Plasticity index

Plasticity index (PI) measures the plasticity of soil. The plasticity index indicates the range of water contents at which soil exhibits plastic properties. Plasticity index (PI) is obtained as numerical difference between liquid limit and plastic limit of soil as shown in equation 3.6.

$$PI = LL - PL \tag{3.6}$$

3.2.4 Compaction

Compaction is the densification of soil by direct application of mechanical load with the sole aim of reducing the air voids between the soil particles. Upon compaction, compacted soil sample experiences reduction in volume. To achieve the maximum dry density (MDD), water must be applied at optimum quantity that is; Optimum Moisture Content (OMC). The soil sample was air dried and thoroughly pulverized so that it passes through BS sieve No. 4 (4.75mm). Test specimens were obtained by mixing reasonable quantity of dry soil with 3 - 12% rock flour by dry weight of soil. Compaction characteristics of the natural soil and rock flour stabilized samples were also obtained using the three energy levels that is; British Standard Light (BSL), West Africa Standard (WAS) and British Standard Heavy (BSH), following procedures outlined in BS 1377-4: 1990, BS 1924-2: 1990 and NGS (1997).

3.2.4.1 British standard light (BSL)

3000gm of soil sample that passes through sieve No. 4 was used, the weight of the mould is denoted as W_1**.** Sample soil was gradually mixed with water to achieve the desired moisture content (w). The thoroughly mixed soil was placed in the mould in three (3) layers. 25 blows of 2.5kg rammer was applied on each layer with a free fall of 300mm. thereafter, mould collar was carefully removed and trimmed so that the soil leveled with the mould, then the weight of mould with the soil sample was taken as (W_2). Soil was extruded from the mould using a metallic extruder to determine the moisture content at the top and bottom of the sample. The soil was placed again in the mixer, water was added to achieve higher moisture content. This process was repeated for 6 times. Therefore, the dry density γ_d was obtained using equation 3.7.

$$\gamma_d = \frac{100\,\gamma}{100 + w} \tag{3.7}$$

Where;

$\boldsymbol{\gamma}$ = weight of the compacted moist soil / volume of mold = $\frac{W2 - W1}{V}$ (3.8)

w = compaction moisture content

3.2.4.2 West Africa standard (WAS)

West Africa Standard (WAS) was conducted following the procedure used in British Stand Light compaction. While 25 blows were applied in BSL and BSH compactions, 10 blows was used for WAS compaction with 4.5kg rammer as the compactive effort, falling through a height of 300mm. Procedure for Calculating dry density is similar as with BSL.

3.2.4.3 British standard heavy (BSH)

In this type of compaction, the mould and amount of soil used are the similar to that of British Standard Light compaction, except that a heavier rammer of 4.5kg falling from a height of 300mm to the soil surface was used. Also, the compacted layers for British Standard Heavy increased to 5 while the number of blows per layer remains the same. Procedure for Calculating dry density is the same as that of BSL.

3.2.5 Unconfined compressive strength (UCS)

Unconfined Compressive Strength (UCS) is the maximum axial compressive stress a specimen can withstand under zero confining stress. UCS is obtained as the axial load per unit area at which the cylindrical sample of a cohesive soil fails under compressive load. Roy, (2014) stated that UCS is commonly used and adaptable method of evaluating stabilized soils strength. The unconfined compressive strength (UCS) tests were performed on the stabilized soil sample according to the procedures outline in BS 1377; 1990 Part 7 and NGS, (1997).

800gm weighed of soil sample that passed through sieve 5mm was prepared for the UCS. The optimum moisture content (OMC) was obtained for the compactive efforts adopted. A properly oiled mould of 80mm height and 38mm diameter was used to prepare the stabilized soil for UCS. Three energy levels namely; British Standard Light, West Africa Standard and British Standard Heavy were adopted. Stabilized specimen were placed in the mould in three layer, hammer of 3.19kg falling from a height of 300mm was used for BSL while same was used but with different number of blows for WAS and BSH. 3, 6, 10 blows were applied on each three (3) layers for BSL, WAS and BSH respectively.

The sampling tube was used to collect sample from the large mould thereafter, the soil sample in the sampling tube was saturated. Mould used was coated with grease then, mould weight was taken. Sample was extruded from the sampling tube into the split mould, the two ends of the samples were carefully trimmed, and weight of mould with specimen was taken. Specimen was then removed from the split mould by splitting the moulds into two. Length and diameter of the specimen were measured using Vernier caliper thereafter, the measured specimen was placed on the bottom plate of the compression machine. Adjustment was made to ensure that the upper plate made contact with the specimen. The dial and proving ring gauges was adjusted to zero before applying compression load to cause an axial strain on the specimen. Record of the dial gauge and proving ring gauge were taken after every 25 seconds as strain on the specimen increased. This process continued until the failure surface clearly developed. Angle of failure between the surface and the horizontal was taken. Sample from the failure zone of the specimen was taken to determine its moisture content. Unconfined Compressive Strength was thus calculated using equation 3.9.

$$\text{Unconfined Compressive Strength} = \frac{Failure\ load}{surface\ area\ of\ specimen} \tag{3.9}$$

3.2.6 California bearing ratio

The California Bearing Ratio (CBR) is strength test used to compare the bearing capacity of a given material with that of well graded crushed stone. CBR measures the resistance of a material to penetration of standard plunger under moisture and density conditions. CBR is primarily used for, but not limited to evaluating the strength of cohesive materials possessing 19 mm particle sizes or less, such as in subgrade, sub-base and base course materials for flexible pavement. The CBR test involves application of load to a small penetration piston at a rate of 1.3mm/minutes and recording the load at 0.64mm – 7.62mm penetration. This test was carried out in accordance with procedures outlined in AASHTO T193-81.

A Loading compression machine operated at constant rate of 1.25mm per minute was used. Cylindrical moulds of 150mm diameter and 175mm height provided with a collar of about 50mm length and detachable perforated base. Compaction rammer, surcharge weight-annular weights each of 2.5 kg and 147mm diameter. IS sieve 20mm, coarse filter paper and weighing balance.

The samples were sieved through 20mm IS sieve. 6 kg of the sample of soil specimen was taken. Water was added to the soil in the quantity such that optimum moisture content or field moisture content was reached.

Then soil and water were mixed thoroughly. Spacer disc was placed over the base plate at the bottom of mould and a coarse filter paper was placed over the spacer disc.

The prepared soil water mix was divided into five. The mould was cleaned and oil was applied. Then one fifth of the mould was filled with the prepared soil. That layer was compacted by giving 62 evenly distributed blows using a hammer of weight 4.5 kg.

The top layer of the compacted soil was scratched. Again second layer was filled and process was repeated. After 3rd layer, collar was also attached to the mould and process was continued.

After fifth layer collar was removed and excess soil was struck off. The base plate was removed and the mould was inverted. Then it was clamped to baseplate.

Surcharge weight of 2.5kg were placed on top surface of soil. Mould containing specimen was placed in position on the testing machine.

The penetration plunger was brought in contact with the soil and a load of 4kg (seating load) was applied so that the contact between soil and plunger was established. Then dial readings were adjusted to zero.

Load was applied such that penetration rate was 1.25mm per minute. Loads at penetration of 0.5, 1, 1.5, 2, 2.5, 3, 4, 5, 7.5, 10 and 12.5mm were noted.

Observations during CBR Test

Weight of soil taken = W

Weight of surcharge =Ws

Area of plunger = A

Proving Ring Calibration Factor = X

Result of California Bearing Ratio Test

1. California Bearing Ratio at 2.5mm penetration was obtained = $CBR_{2.5}$
2. California Bearing Ratio at 5.0mm penetration was obtained as = $CBR_{5.0}$
3. California Bearing Ratio of subgrade soil was obtained as = CBR_s

3.2.7 Durability test

Notman, (2011), defines design durability as the effects on a material due to workmanship or design elements such as inadequate compaction, frost, poor choice of binder. Durability test was carried out on the stabilized specimen prepared by mixing the natural soil with 3, 6, 9 and 12% of rock flour. For each % rock flour content, three samples were taken for durability test. A reasonable amount of water was added to bring the moisture content of the soil-rock flour mixture to the desired optimum moisture content as British Standard Heavy compaction. The mixtures were remoulded in a cylindrical mould measuring 38mm diameter and 76mm length by static compression in compaction frame to the desired maximum dry density as British Standard Heavy compaction. Thereafter, the compacted specimens were cured for 7 and 28 days at 90 – 100% relative humidity in a desiccators partially filled with water at room temperature.

3.2.7.1 Procedure for wetting and drying test

After storing the compacted specimens in the desiccators, the specimens were submerged in potable water at room temperature for a period of 5hr and removed. All the specimen collapsed and the durability test was discontinued.

3.2.7.2 Loss of strength on immersion test

The loss of strength on immersion test is defined in Series 800 (MCHW 1, 2007) using the procedure given in Clause 880.4. Cylinders with a ratio of 1:1 (Height: Diameter) were prepared and cured for 14 days in air. They were then cured for a further 7 days immersed in water. The compressive strength of these immersed samples ($R_{C\ imm}$) was determined together with that of the control specimens ($R_{C\ Control}$). The control specimens were cured for 7 days in a sealed condition. All curing was undertaken at 20 °C for the soil-rock flour mix as assessed in this study.

The mixture was considered to be durable if equation 3.10 applies:

$$R_{c\ vs} = \left(\frac{R_{c\ imm}}{R_{c\ control}}\right) * 100 \geq 80\% \qquad (3.10)$$

Rc vs is the relative volumetric stability (assumed to be durable if ≥ 80 %)

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

Oxide composition of the natural soil sample is shown in Table 4.1. The Silicate – Alumina ratio indicated that the soil is lateritic in nature having a ratio of 1.42.

Table 4.1; Oxide composition of Natural Lateritic Soil

Oxide	Quantities
SiO_2	48.23
CaO	0.38
Al_2O_3	20.68
Fe_2O_3	13.32
SO_3	0.85
MgO	2.18
K_2O	0.69
Mn_2O_3	0.94
TiO_2	0.19
Na_2O	1.5

Table 4.2; Index and Geotechnical Properties of Natural Lateritic Soil

Property	Value
Liquid Limit	42%
Plastic Limit	12.70%
Plasticity Index	29.30%
% Passing BS sieve No. 200	36.10%
Specific Gravity	2.62
Maximum Dry Density (MDD)	
British Standard Light (BSL)	1.802 g/cm³
West African Standard (WAS)	1.820 g/cm³
British Standard Heavy (BSH)	1.870 g/cm³
Optimum Moisture Content (OMC)	
British Standard Light (BSL)	12.40%
West African Standard (WAS)	11.90%
British Standard Heavy (BSH)	11.60%
AASHTO Classification	A-7-6
USCS classification	SC
Colour	Brownish

4.1 Index and Geotechnical Properties of Test Soil

Results of test conducted to determine the index and geotechnical properties of natural lateritic soil sample for this research is presented in Table 4.2. The natural soil has 36.1% of silt – clay material passing through sieve No. 200, Liquid Limit (LL) of 42%, Plastic Limit (PL) of 12.70% and Plasticity index of 29.30% .Therefore, the soil is classified under A-7-6 according to the AASHTO soil classification system and as Sandy Clay (SC) according to Unified Soil Classification System (USCS). Based on the AASHTO classification system, the natural soil is rated as unsuitable material for most Civil and Geotechnical Engineering works. The plasticity index and liquid limit are above the maximum values of 12 and 30% respectively recommended for subgrade according to NGS (1997). The consistency limits results indicated that the soil has high tendency of retaining water, due to the high compressibility and low shear strength (Arora, 2011). Results of the specific gravity test showed that the soil has a specific gravity of 2.65. The Maximum Dry Density for BSL, WAS and BSH compactive effort yielded 1.802, 1.820 and 1.870g/cm^3, with the corresponding OMC values of 12.40, 11.90 and 11.60% respectively in the same order. According to Flaherty (1988), the maximum dry density anticipated for silty clay soil using proctor test ranges between 1.60 and 1.845g/cm^3 and the optimum moisture content is expected to range between 15 – 25%. He estimated the maximum dry density for sandy clay soils to vary between 1.75 and 2.165g/cm^3 and OMC values between 5 and 18%. In terms of compliance to specification for subgrade material, 36.1% passing BS sieve No. 200, being slightly higher than 35% as recommended in NGS (1997). The tested soil sample is classified under A-7-6. Therefore implies that the natural soil is not suitable for use as pavement subgrade without stabilization. Result of particle size distribution of the natural soil is presented in Appendix A while the value of specific gravity test result is presented in Appendix B. Table 4.3 shows the geotechnical properties of rock flour.

Table 4.3: Geotechnical Properties of Rock flour

Property	Value
Specific Gravity	2.67
Liquid Limit (%)	NP
Plastic Limit (%)	NP
Absorption	1.20 – 1.50
Water Content	Nill
Appearance	Fine grained

The proprieties of rock flour as shown is Table 4.3 gave a clear indication of rock flour capacity to improve the consistency limits of the soil – rock flour mixture because rock flour is a non-plastic material. Also, higher value of rock flour specific gravity compared with that of natural soil indicated its capacity to improve the mix maximum dry density.

4.2. Consistency Limits

Liquid limit of the natural soil decreased from to 32% upon addition of 0 - 12% rock flour to the unstabilized specimen. Also, the Plastic Limit progressively increased from 12.70% for the natural soil to 18.52% when stabilized with 12% rock flour. Rock flour being a non-plastic material according to (Satyanarayana *et al.*, 2013: Garata *et al*., 2014: Satyanarayana *et al.,* 2016), improved the consistency limits of A-7-6 – rock flour mixture. The progressive decrease observed in this study is consistent with result obtain by (Ogunribido, 2012; Ademila, 2019). The improved consistency limits however, failed to meet requirements for subgrade materials which is specified as; LL < 35 and PI < 12% according to NGS (1997). The failure of mixtures to meet the required threshold values for consistency parameters suggests that higher rock flour contents may be needed to achieve the specification requirements. Figure 4.1 shows the relationship between consistency limits of the natural soil and stabilized soil specimens using rock flour at 3, 6, 9 and 12%.

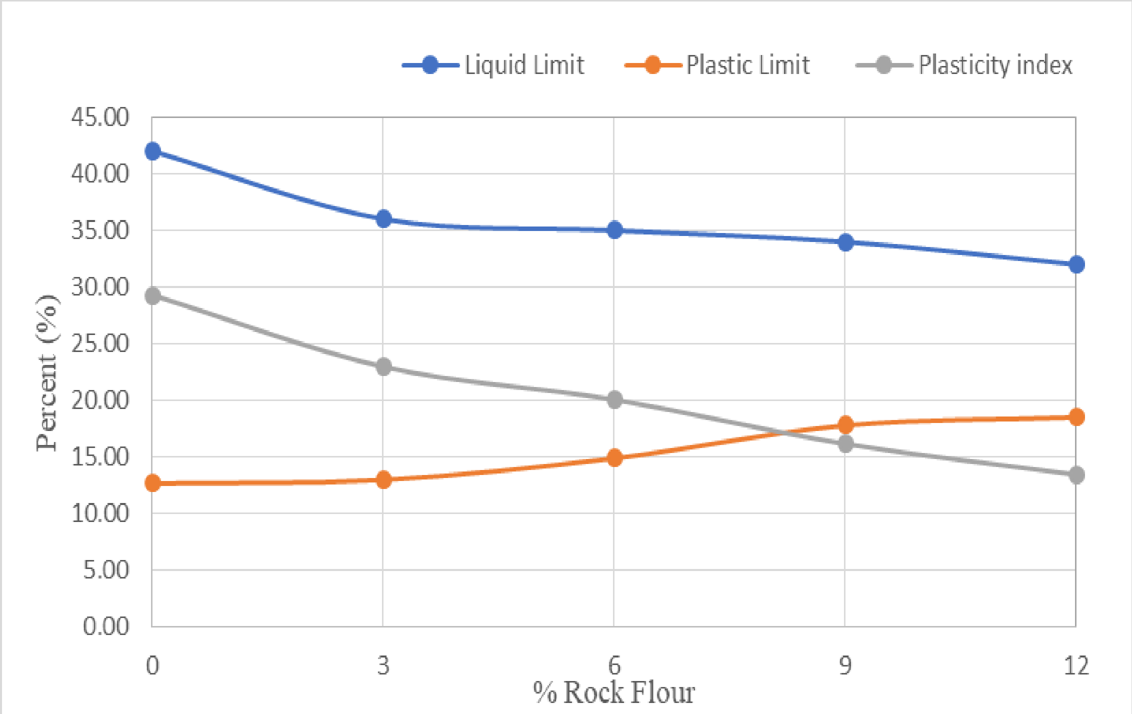

Figure 4.1: Variation of Atterberg limits of soil mixtures with rock flour content

4.3. Effect of rock flour on compaction

4.3.1 Maximum dry density (MDD)

Generally, MDD increased from 1.870 to 2.010g/cm³ for BSH, from 1.820 to 1.87g/cm³ for WAS, while BSL increased from 1.802 to 1.850g/cm³ as the rock flour content increased from 0 – 12%. Figure 4.2 shows how the maximum dry density changes with the addition of rock flour for BSL, BSH and WAS compaction.

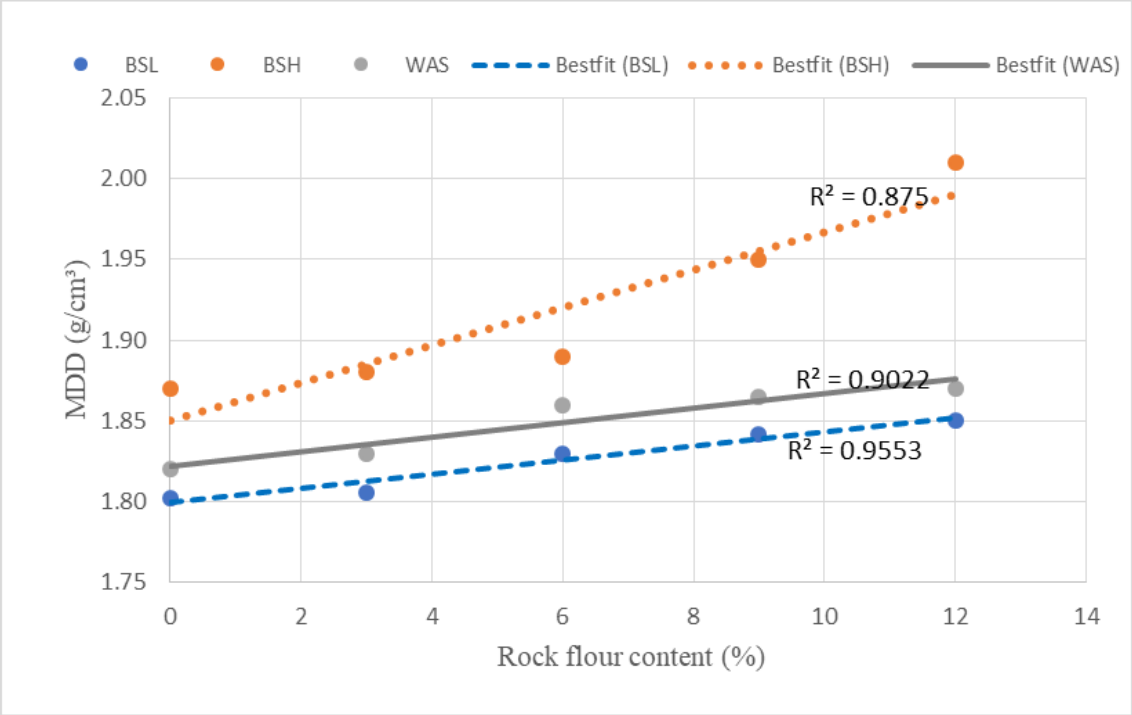

Figure 4.2: Variation of MDD with Rock flour content for BSL, WAS and BSH compaction efforts

British Standard Heavy gave highest MDD values ranging from 1.87 – 2.010g/cm³ as rock flour content increased from 0 – 12%. The increase in MDD values suggested that the increase in rock flour content has positive influence on the strength and density characteristics of tested mixtures. Rock flour is said to mobilized high interfacial angle with test soil as the pores between mixture particles reduced due to compaction (Satyanarayana *et al.*, 2013: Ademila, 2019).

4.3.2 Optimum moisture content (OMC)

The OMC increased from 11.60 to 13.90% for BSH, from 11.90 to 14.40% for WAS and 12.40%to 14.80% for BSL as the rock flour content increased from 0 – 12% as shown in Figure 4.3.

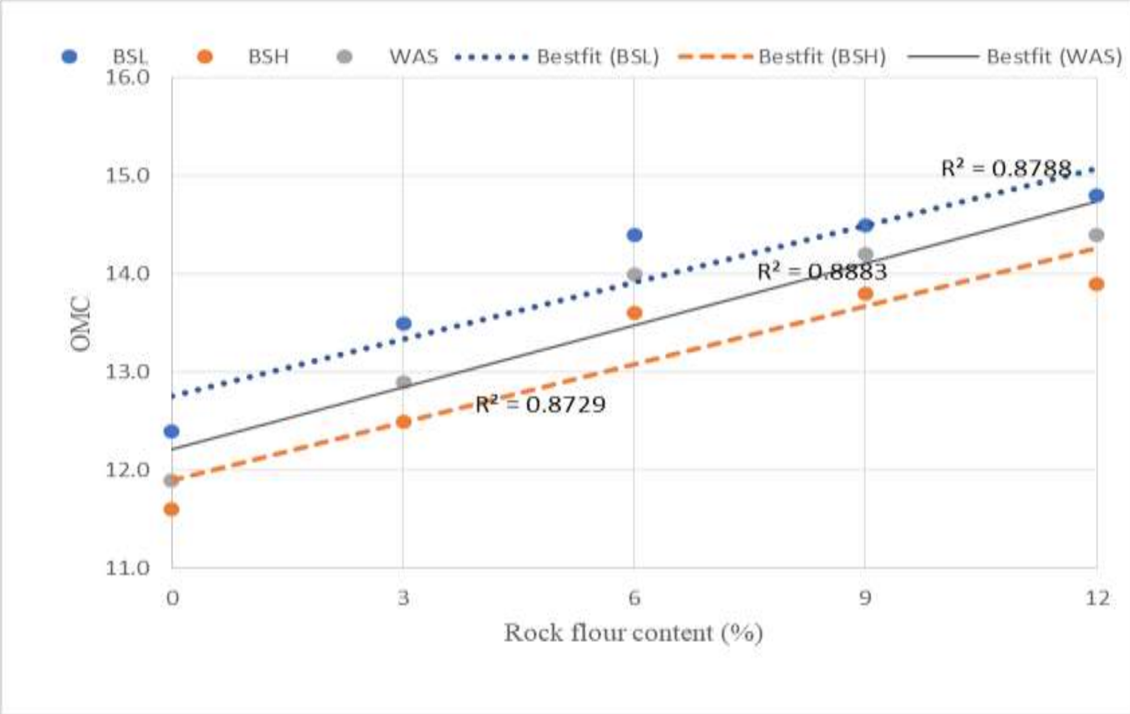

Figure 4.3: Variation of OMC with Rock flour content for BSL, WAS and BSH compaction efforts.

Decrease in OMC values for BSH compaction on tested mixtures occurred due to the replacement of silt and clay constituent with rock flour particles in the specimens, which therefore, reduced moisture intake (Satyanarayana *et. al.*, 2013). Ademila (2019), observed a reduction in OMC from an initial value of 11.11% at 10% addition of rock flour to the natural soil whose initial OMC was 19.64% when he WAS compaction was applied to the samples. In general, reduction in OMC for stabilized samples indicated improvement in the soil as well as its workability. It is therefore, suggested that reduction in OMC value is as a result of absorption capacity of the rock flour because of its porous properties.

4.4 Unconfined compressive strength (UCS)

UCS is part of parameters used to estimate bearing capacity of subgrade and subbase soils in highway pavement construction. Unconfined compressive strength of the natural soil ranges between 100 – 200 kN/m² and are regarded as stiff soil according to (Bowles, 1992). UCS for all compaction energies used initially dropped before attaining a peak at 9% rock flour. BSH gave the highest UCS value, but values obtained for BSH compaction initially reduced from 180.50kN/mm² for natural soil to 130.59 kN/m² at 3% addition of rock flour before attaining a peak value of 250.89 kN/m² at 9% of rock flour. Addition of rock flour at 9% by dry weight of soil improves the soil stiffness. Generally, values of UCS obtained for the three energy levels used indicated that, higher compaction effort gave higher UCS value, as shown in Figure 4.4.

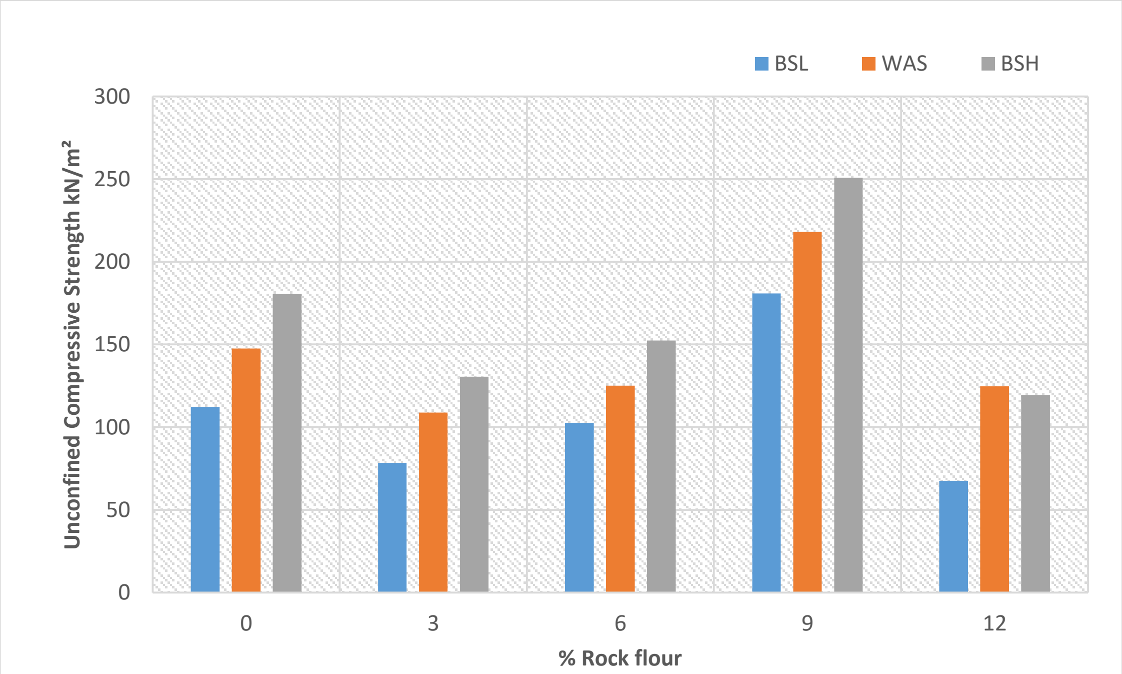

Figure 4.4: Variation of UCS with Rock flour content

Results of UCS for the three energy levels validate the potential of using rock flour to improve the strength parameters of subgrade. Addition of rock flour to poor subgrade soil improves the soil stiffness and resistance to deformation as also reported by (Ogunribido and Abiola, 2015; Ademila, 2019). The quality of the mixture is presented in Table 4.4.

Table 4.4; Quality of mixture according to (Bowles, 1992)

	% addition of rock flour				
	0	3	6	9	12
British Standard Light	112.3	78.33	102.57	180.81	67.59
Quality of Mixture	Stiff	Firm	Stiff	Stiff	Firm
British Standard Heavy	180.500	130.590	152.360	250.890	119.510
Quality of Mixture	Stiff	Stiff	Stiff	Very stiff	Stiff
West Africa Standard	147.480	108.870	125.140	218.120	124.620
Quality of Mixture	Stiff	Stiff	Stiff	Very stiff	Stiff

However, the quality of mixtures with 0 – 12% rock flour for BSH is stiff except for 9% where it is very stiff. Similar scenario is observed for WAS at 12%, both BSH and WAS exhibited stiff qualities. Whereas, the quality of mixtures with 0 – 12% rock flour for BSL is stiff except for 3 and 12% of rock flour, where the mixture quality is rated firm.

Seyed *et al.* (2012), observed that the plasticity index has significant impact on UCS value, higher Plasticity index results in lower UCS value. Therefore, results of UCS obtained indicated that high value of plasticity index of the mixture is responsible for the initial drop in UCS value from 112.3, 180.50 and 147.48 kN/m² to 78.33, 130.59 and 108.87kN/m² for BSL, BSH and WAS compaction respectively at 3% of rock flour. Changes in the stress strain behaviour of the mixture above 3% addition of rock flour is due to increasing compaction energy which made the mixture to be more brittle thereby increasing the UCS values. This agrees with conclusion made by (Mohamed *et al.,* 2016). Reduction in UCS value observed for higher percent of rock flour above 9% can be attributed to the significant reduction of cohesion in the mixture.

4.5 California bearing ratio.

CBR is used as a semi-empirical test to evaluate the strength property of subgrade soil. Both soaked and unsoaked conditions were tested for the three compaction energy levels adopted in this research as rock flour increased from 0 - 12%. The soaked CBR test helps to evaluate the soil strength in an in-situ condition, where the subgrade soil is exposed to moisture. Results for soaked and unsoaked CBR showed progressive increment in CBR value of the stabilized soil. Generally, unsoaked CBR gave higher values when compared with soaked CBR. Figures 4.5 and 4.6 show the CBR values for unsoaked and soaked conditions, using BSL, WAS and BSH compaction efforts.

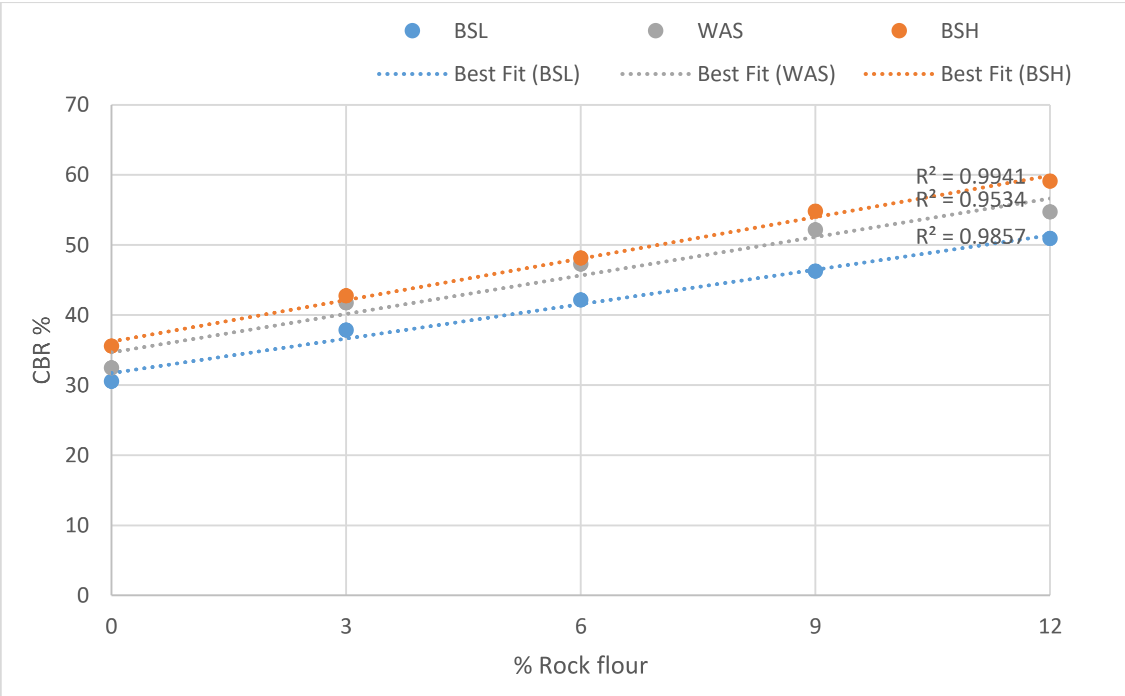

Figure 4.5: Variation of unsoaked CBR with Rock flour content

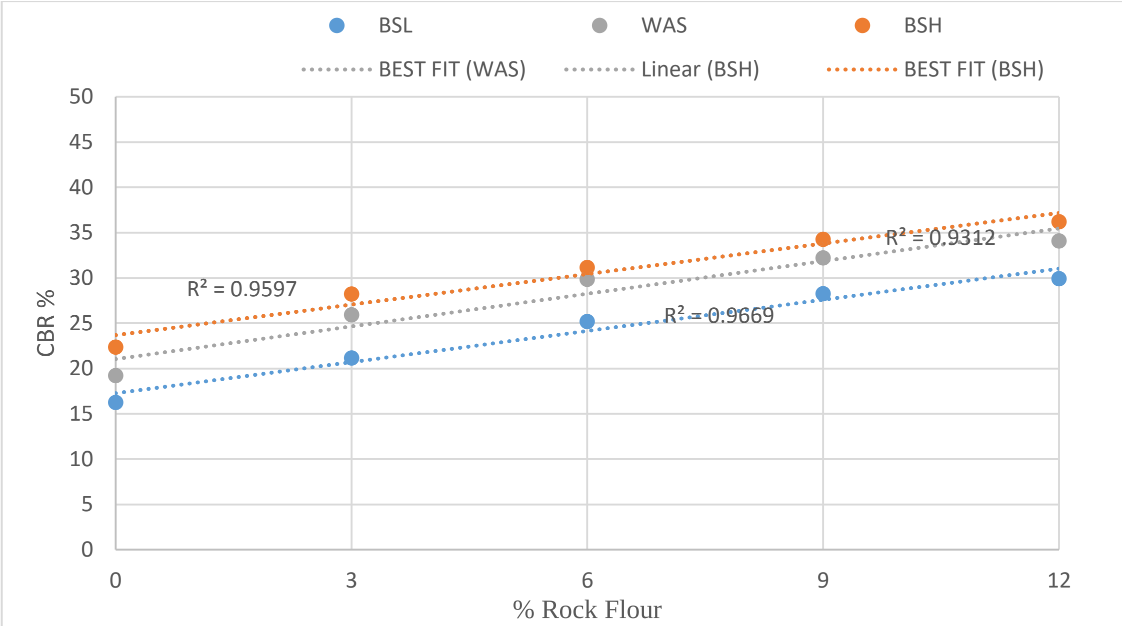

Figure 4.6: Variation of soaked CBR with Rock flour content

The result however, has minimum and maximum values of 30.55 and 59.12% respectively. Also, addition of rock flour improved the soaked CBR value for all energy levels adopted which varied between 16.25 – 36.2%. Therefore, this results is said to have satisfied the minimum requirement of 10% for soaked CBR when used as subgrade in flexible pavement construction according to NGS (1997). Also, CBR value of 31.15% at 6% addition of rock flour for BSH and 32.20% at 9% of rock flour for WAS met the minimum requirement of 30% in soaked condition when used as subbase in flexible pavement construction. The CBR results showed that rock flour has positively influenced the strength properties of the natural soil for both soaked and unsoaked conditions. The progressive increase in the California Bearing Ratio (CBR) for soaked and unsoaked condition with increasing rock flour content is an indication of the strength and stiffness of the lateritic soil-rock flour mixture. Also, the weight of hammer used, number of blows and layers are factors responsible for higher CBR values of 36.2 and 59.12% obtained for soaked and unsoaked conditions using BSH compaction at 12% of rock flour content. Conversely, BSL and WAS with lighter hammer and fewer number of layers gave CBR values of 50.94 and 54.70% for unsoaked and 29.90 and 34.08% for soaked conditions respectively at 12% rock flour content. This is in agreement with (Ademila, 2019).

The grading of mixture for soaked CBR is presented in Table 4.5.

Table 4.5; CBR Grading of Mixture According to NGS (1997).

Soaked CBR (%)	General Rating	Uses
0 - 10	Poor - Fair	Subgrade
30 - 80	Fair - Good	Subgrade, Subase
> 80	Excellent	Base

4.6 Durability

4.6.1 Wetting and drying test

No data was obtained owing to the total collapse of the samples when immerse in water for wetting and drying procedures. This occurred because rock flour does not possess binding properties that could hold the specimen particles together without collapse during curing. Collapse samples are presented in Plate I.

Plate I: Collapsed samples using wetting and drying procedure

4.6.2 Loss of strength on immersion test

No data was obtained for the immersed condition due to the collapse of the sample after curing for 7 days as shown in Plate II.

Plate II: Collapsed samples using loss of strength on immersion test

The collapse could be attributed to non pozzolanic nature of rock flour. The 7 days UCS values obtained for the control, $R_{control}$ ranges between 353.14 - 667.78kN/m², with a peak value of 667.781kN/m² obtained at 6% rock flour content as shown in Figure 4.7.

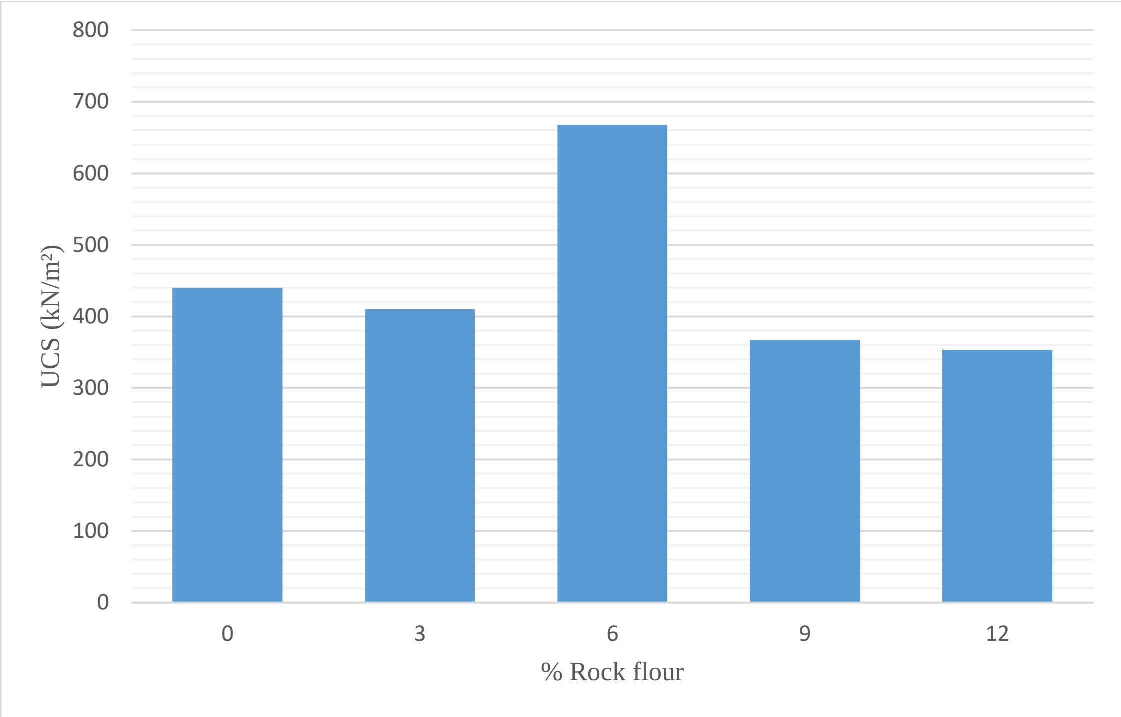

Figure 4.7: Variation of UCS value with Rock flour content for durability

Considering that no results obtained from the immersed samples, no relationship could be established as the relative volumetric stability, *Rc vs* for the soil-rock flour mix.

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

This study evaluated the impact of adding 0 to 12% rock flour on the consistency limits, compaction characteristics, unconfined compressive strength and CBR of lateritic soil sample. Soil mixtures for compaction tests were compacted using British Standard Light (BSL), West Africa Standard (WAS) and British Standard Heavy (BSH) compactive efforts. The following conclusions were drawn from the study;

The natural soil has been classified as A-7-6 according to AASHTO classification system and SC under Unified Classification System. The Liquid Limit and Plasticity Index were observed to have reduced with increasing percentage of rock flour, while the Plastic Limit increased with increasing rock flour content. Results of the consistency limits failed to meet the requirements for subgrade material, which is specified as; LL < 35 and PI < 12% according to (NGS, 1997). Addition of rock flour to the natural soil showed improvement in the UCS values of the stabilized specimen, with the highest value of 250.89kN/m^2 for BSH compactive effort at 9% rock flour content.

Although, addition of rock flour to the natural soil seemed to improve the CBR values for both soaked and unsoaked conditions using the three compaction energy levels. However, the soaked CBR values met the minimum requirements of 10% for flexible pavement subgrade according to NGS, (1997). Therefore, addition of rock flour to the natural soil significantly improved the soil load bearing capacity. The soil - rock flour mixture could not be cured for durability test due to the non-pozzolanic nature of rock flour. Therefore, rock flour cannot be used as a stand-alone addictive for the purpose of stabilization.

5.2 Recommendations

1. A-7-6 soil stabilized with rock flour content above 12% can be investigated to determine if the mixture meets the minimum requirements for subgrade material +as specified in local codes.
2. Combination of lateritic soil, rock flour and cementitious stabilizer can investigated to examine if such combinations will improve the mixture durability.

5.3 Contribution to knowledge

This research established the usage of rock flour as an additive when added to the natural soil at 3, 6, 9 and 12% respectively. BSH gave the highest values of MDD which ranges between of 1.870 – 2.010 g/cm^3 and corresponding OMC values between 11.60 – 13.90%. BSH gave the highest value of 250.89kN/m^2 for UCS at 9% rock flour content. Soaked CBR values for BSL, WAS and BSH compaction meets the minimum requirements of 10% for flexible pavement subgrade according to NGS, (1997). Rock flour cannot be used as stand-alone additive.

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APPENDICES

Appendix A: Particle Size Analysis of Test Soil

Table A1: Particle Size Distribution Results

Sieve	Percent by Weight		
Designation	**Mass. Retained**	**% Retained**	**% PASSING**
5.00	8.70	2.90	97.10
3.35	7.70	2.57	94.53
2.36	7.70	2.57	91.97
2.00	6.40	2.13	89.83
1.180	26.00	8.67	81.17
0.850	50.60	16.87	64.30
0.600	15.10	5.03	59.27
0.425	16.60	5.53	53.73
0.300	17.50	5.83	47.90
0.150	23.30	7.77	40.13
0.075	12.10	4.03	36.10

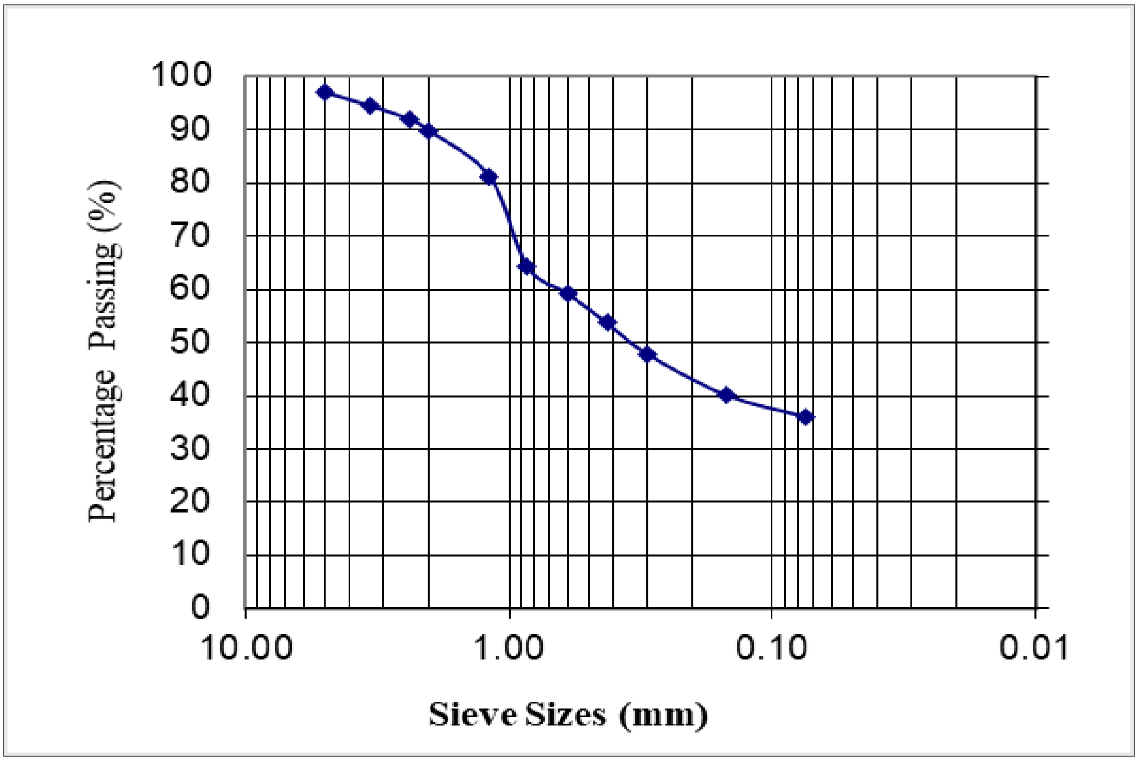

Figure A1: Particle size distribution curve of natural lateritic soil

Appendix B: Specific Gravity of Test Soil

Table B1: Specific Gravity test results of natural lateritic soil

	B1	B2	B3
Mass of bottle (g)	126.5	69	97.4
Mass of bottle + wet soil (g)	419.3	187.3	388.7
Mass of bottle + dry soil (g)	197.77	101.05	167.5
Mass of bottle + water (g)	374.3	168	345.3
Specific Gravity, Gs	2.65	2.64	2.66
Average Specific Gravity, Gs		2.62	

Table B2: Specific Gravity Test Results of Rock flour

	B1	B2	B3
Mass of bottle (g)	46.6	43.7	52.5
Mass of bottle + wet soil (g)	66.1	64.5	68.3
Mass of bottle + dry soil (g)	165.7	162.1	169.5
Mass of bottle + water (g)	153.5	148.9	159.8
Specific Gravity, Gs	2.67	2.74	2.59
Average Specific Gravity, Gs		2.67	

Appendix C: Consistency Limits of Natural soil

Table C1: Consistency Limit Results

LIQUID LIMIT DETERMINATION							
	LIQUID LIMIT					PLASTIC LIMIT	
Trial Number	1	2	3	4	5	1	2
Penetration (mm)	6.00	13.00	14.50	17.80	21.50		
Wt. of wet soil + can	24.500	23.200	26.700	25.100	25.100	25.700	26.500
Wt. of dry soil + can	23.824	22.402	25.788	24.567	24.440	25.593	26.283
Wt. of can	19.900	19.000	22.200	22.900	22.900	24.600	24.800
Wt. of dry soil	3.92	3.40	3.59	1.67	1.54	0.99	1.48
Wt. of water	0.68	0.80	0.91	0.53	0.66	0.11	0.22
Water content %	17.23	23.46	25.42	31.97	42.86	10.78	14.63
Liquid limit %	42.00		Average Plastic Limit			12.70	

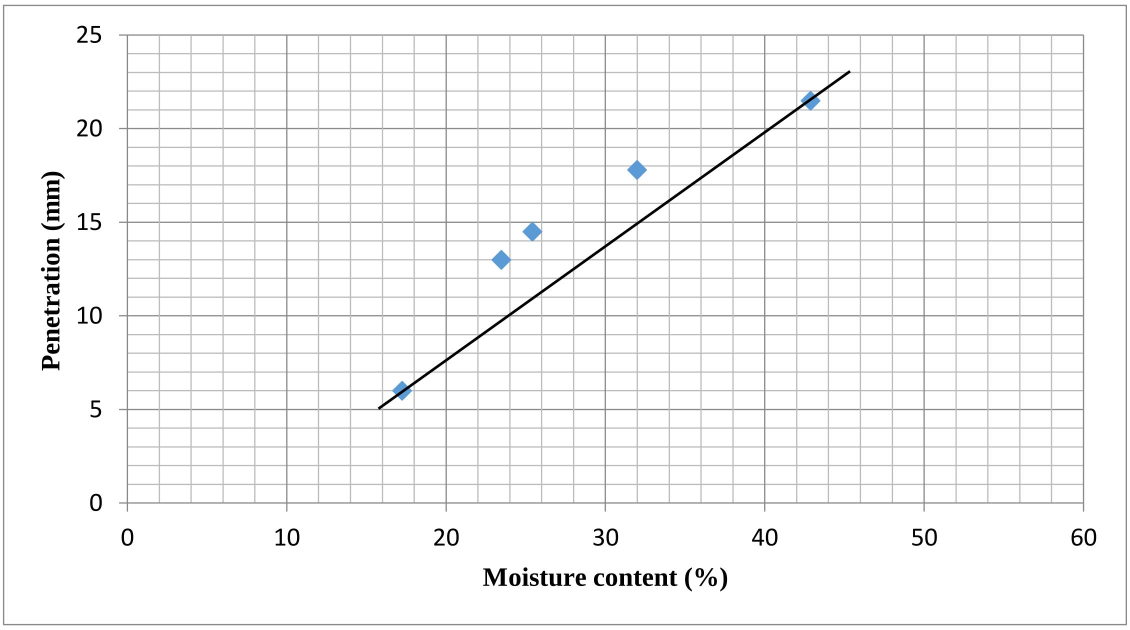

Figure C1: Liquid Limit Determination Curve

Appendix C2: Consistency Limits of stabilized soil at 3% rock flour

Table C2: Consistency Limit Results at 3% rock flour

LIQUID LIMIT DETERMINATION								
	LIQUID LIMIT						PLASTIC LIMIT	
Trial Number	1	2	3	4	5	6	1	2
Penetration (mm)	3.50	9.40	12.10	14.00	18.50	20.10		
Wt. of wet soil + can	26.440	31.474	34.931	31.285	31.563	31.503	26.961	23.425
Wt. of dry soil + can	26.049	30.817	33.968	30.196	30.134	29.700	26.761	23.345
Wt. of can	24.108	27.878	30.100	26.033	25.292	24.029	25.687	22.260
Wt. of dry soil	1.94	2.94	3.87	4.16	4.84	5.67	1.07	1.09
Wt. of water	0.39	0.66	0.96	1.09	1.43	1.80	0.20	0.08
Water content %	20.14	22.35	24.90	26.16	29.51	31.79	18.62	7.37
Liquid limit %	35.00		Average Plastic Limit				13.00	

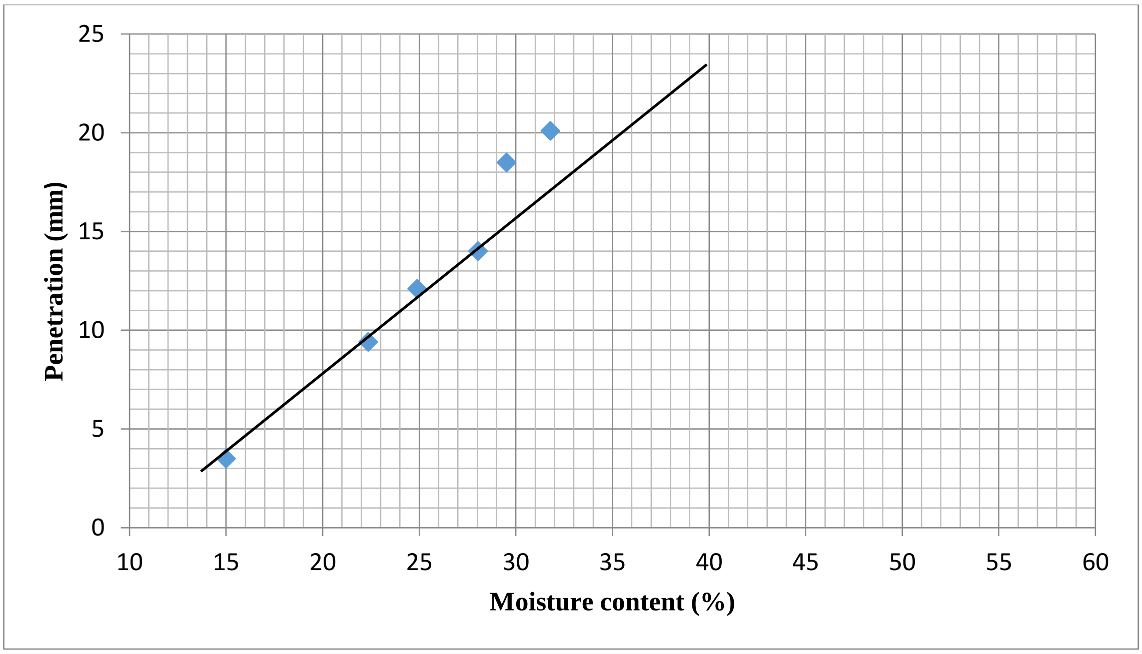

Figure C2: Liquid Limit Determination Curve

Appendix C3: Consistency Limits of stabilized soil at 6% rock flour

Table C3: Consistency Limit Results at 6% rock flour

LIQUID LIMIT DETERMINATION								
	LIQUID LIMIT						PLASTIC LIMIT	
Trial Number	1	2	3	4	5	6	1	2
Penetration (mm)	3.50	9.40	12.10	14.00	18.50	20.10		
Wt. of wet soil + can	26.340	31.474	34.931	31.363	31.563	31.503	26.961	23.425
Wt. of dry soil + can	26.049	30.817	33.968	30.196	30.134	29.700	26.790	23.279
Wt. of can	24.108	27.878	30.100	26.033	25.292	24.029	25.687	22.260
Wt. of dry soil	1.94	2.94	3.87	4.16	4.84	5.67	1.10	1.02
Wt. of water	0.29	0.66	0.96	1.17	1.43	1.80	0.17	0.15
Water content %	14.99	22.35	24.90	28.03	29.51	31.79	15.50	14.33
Liquid limit %	35.00		Average Plastic Limit				14.92	

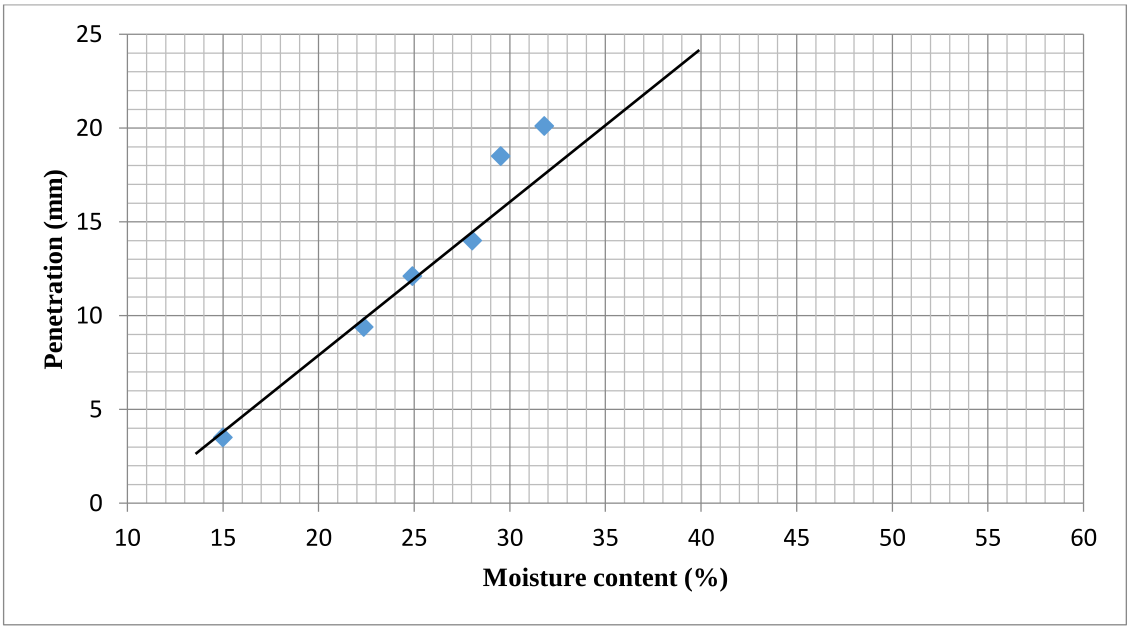

Figure C3: Liquid Limit Determination Curve

Appendix C4: Consistency Limits of stabilized soil at 9% rock flour

Table C4: Consistency Limit Results at 9% rock flour

LIQUID LIMIT DETERMINATION								
	LIQUID LIMIT						PLASTIC LIMIT	
Trial Number	1	2	3	4	5	6	1	2
Penetration (mm)	2.00	6.00	8.50	13.50	17.50	20.50		
Wt. of wet soil + can	21.600	21.100	22.100	28.200	27.900	30.100	20.600	23.100
Wt. of dry soil + can	21.320	20.530	21.356	27.311	26.545	28.223	20.445	22.958
Wt. of can	19.800	18.200	18.500	24.200	22.000	22.500	19.700	22.000
Wt. of dry soil	1.52	2.33	2.86	3.11	4.55	5.72	0.75	0.96
Wt. of water	0.28	0.57	0.74	0.89	1.36	1.88	0.16	0.14
Water content %	18.42	24.46	26.05	28.58	29.81	32.80	20.81	14.82
Liquid limit %	34.00		Average Plastic Limit				17.81	

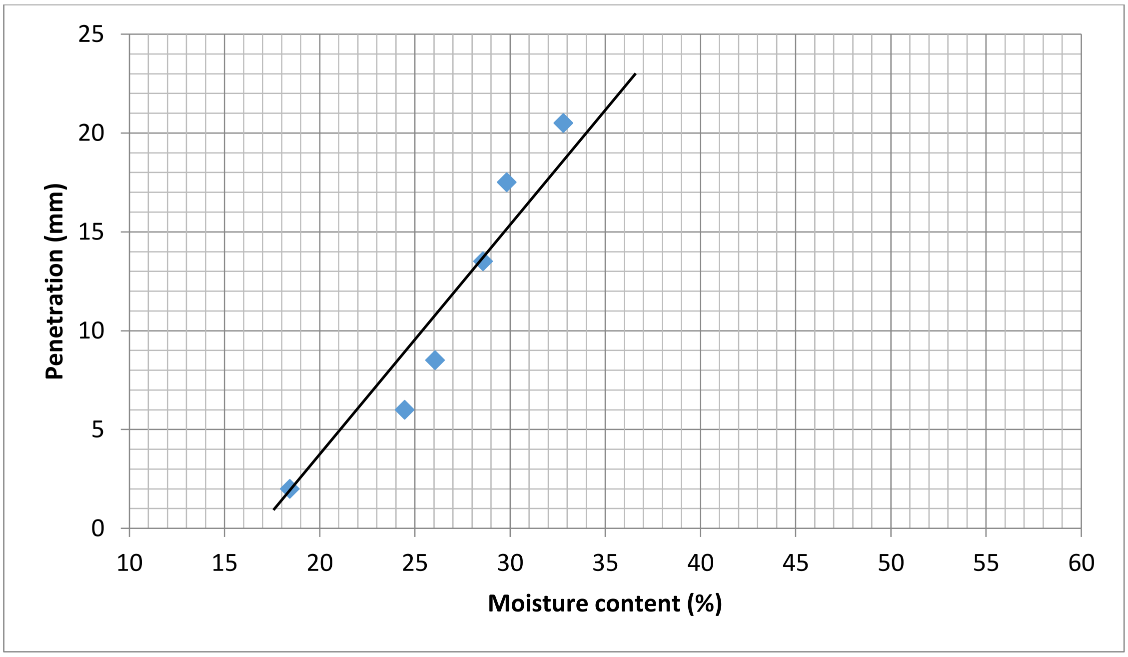

Figure C4: Liquid Limit Determination Curve

Appendix C5: Consistency Limits of stabilized soil at 12% rock flour

Table C5: Consistency Limit Results at 12% rock flour

LIQUID LIMIT DETERMINATION								
	LIQUID LIMIT						PLASTIC LIMIT	
Trial Number	1	2	3	4	5	6	1	2
Penetration (mm)	2.00	6.00	8.50	13.50	17.50	20.50		
Wt. of wet soil + can	21.600	21.100	22.100	28.200	27.900	30.100	20.645	23.105
Wt. of dry soil + can	21.320	20.530	21.356	27.362	26.645	28.223	20.560	22.944
Wt. of can	19.800	18.200	18.500	24.200	22.000	22.500	19.700	22.000
Wt. of dry soil	1.52	2.33	2.86	3.16	4.65	5.72	0.86	0.94
Wt. of water	0.28	0.57	0.74	0.84	1.26	1.88	0.09	0.16
Water content %	18.42	24.46	26.05	26.50	27.02	32.80	9.88	17.06
Liquid limit %	32.00		Average Plastic Limit				13.47	

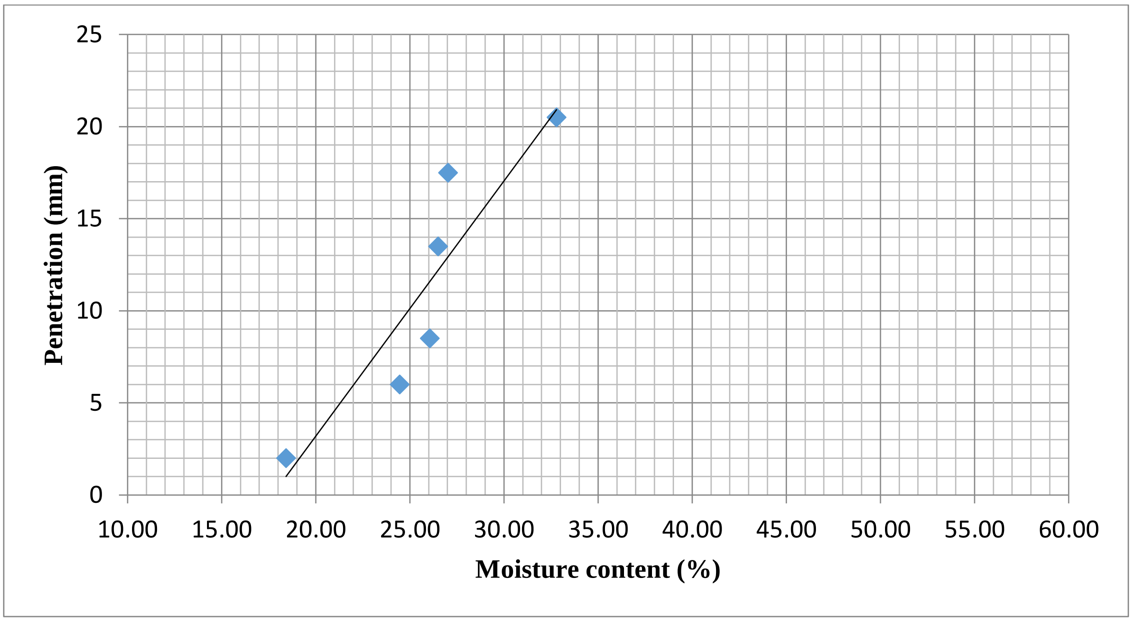

Figure C5: Liquid Limit Determination Curve

Appendix D: Compaction Test of Soil

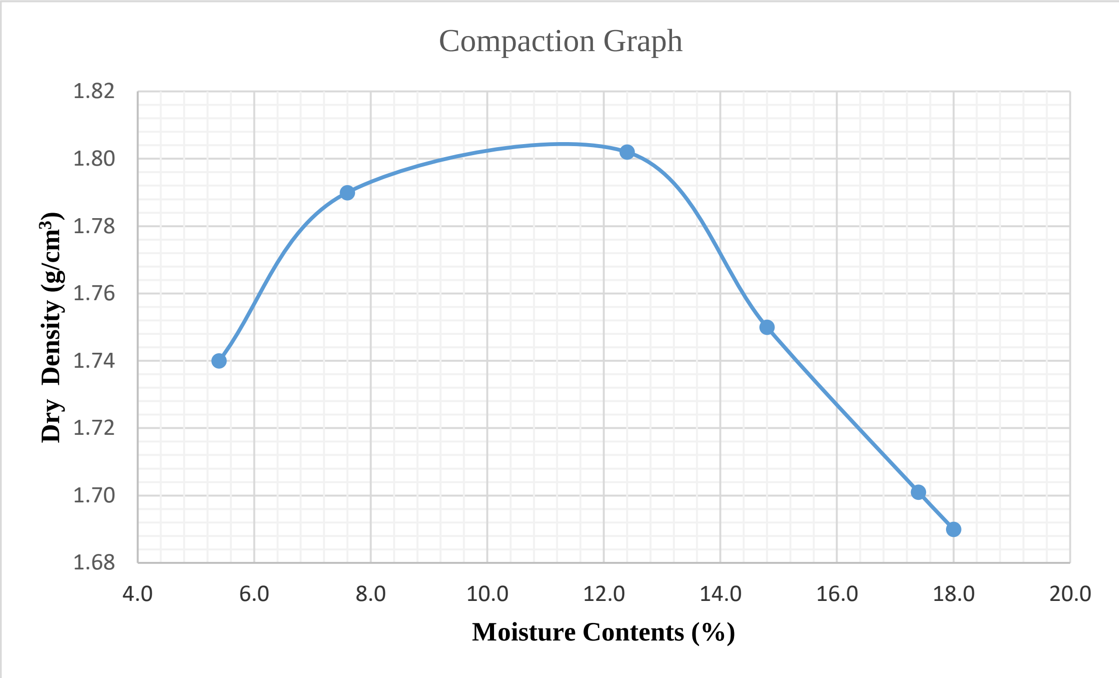

Figure D2: British Standard Light (BSL) at 0% addition of rock flour

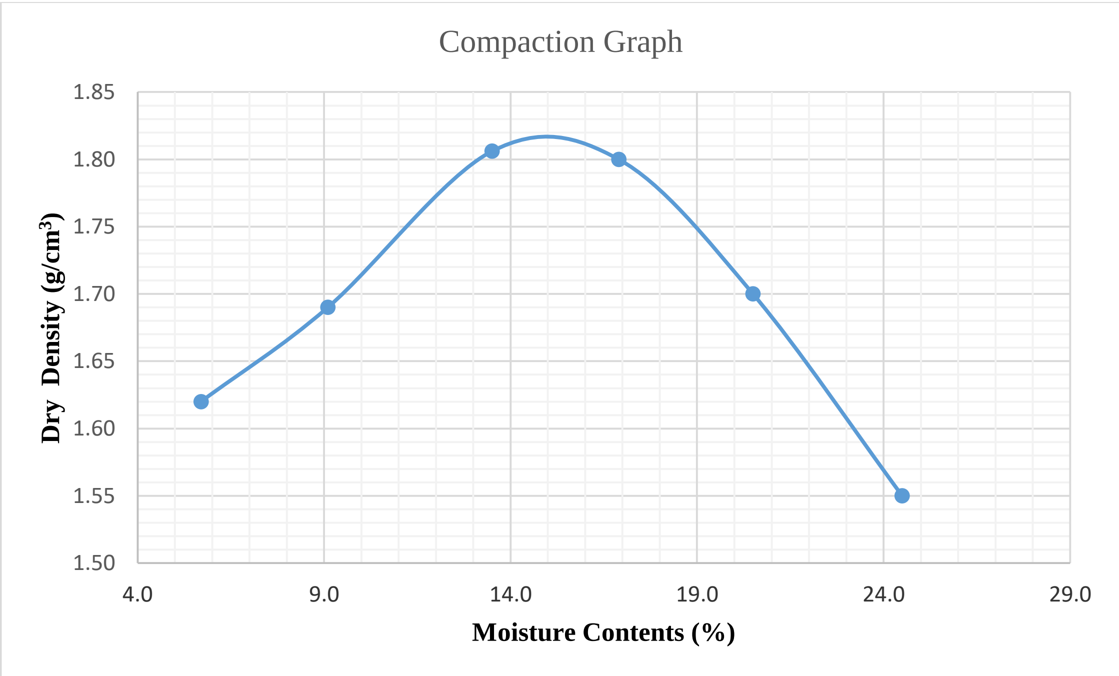

Figure D3: British Standard Light (BSL) at 3% addition of rock flour

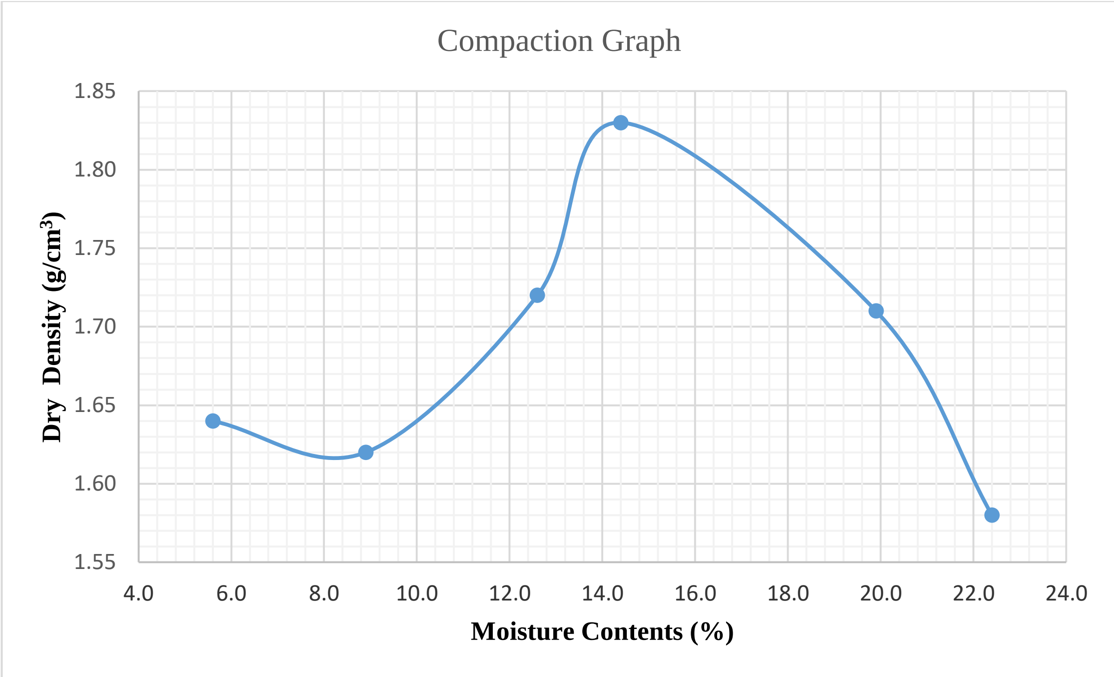

Figure D4: British Standard Light (BSL) at 6% addition of rock flour

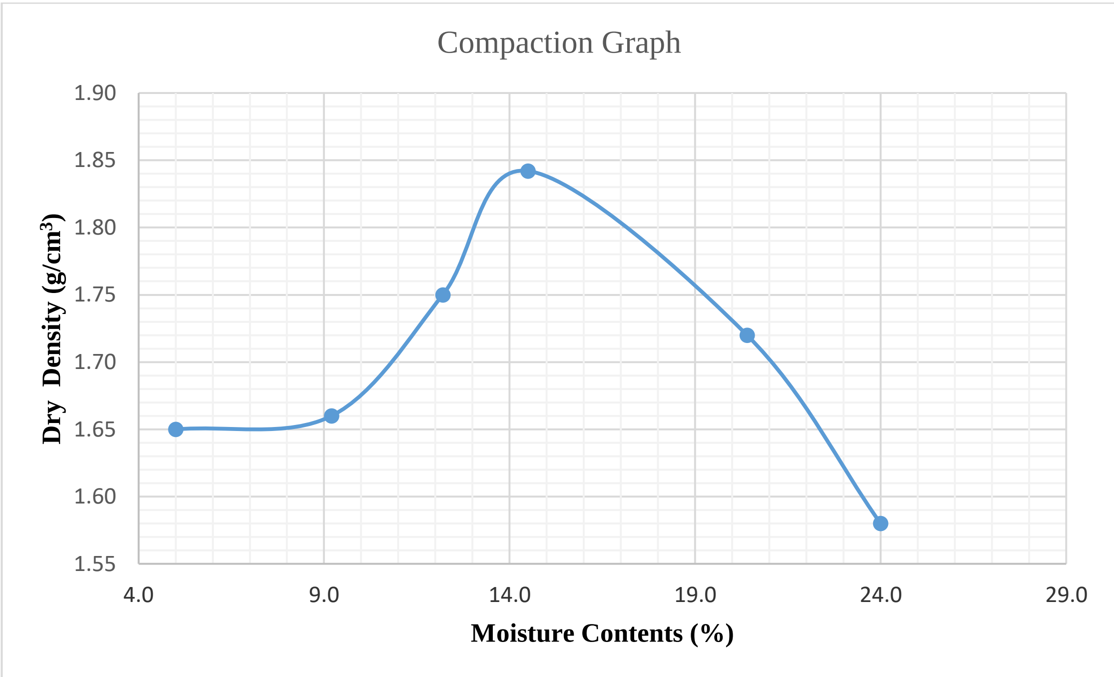

Figure D5: British Standard Light (BSL) at 9% addition of rock flour

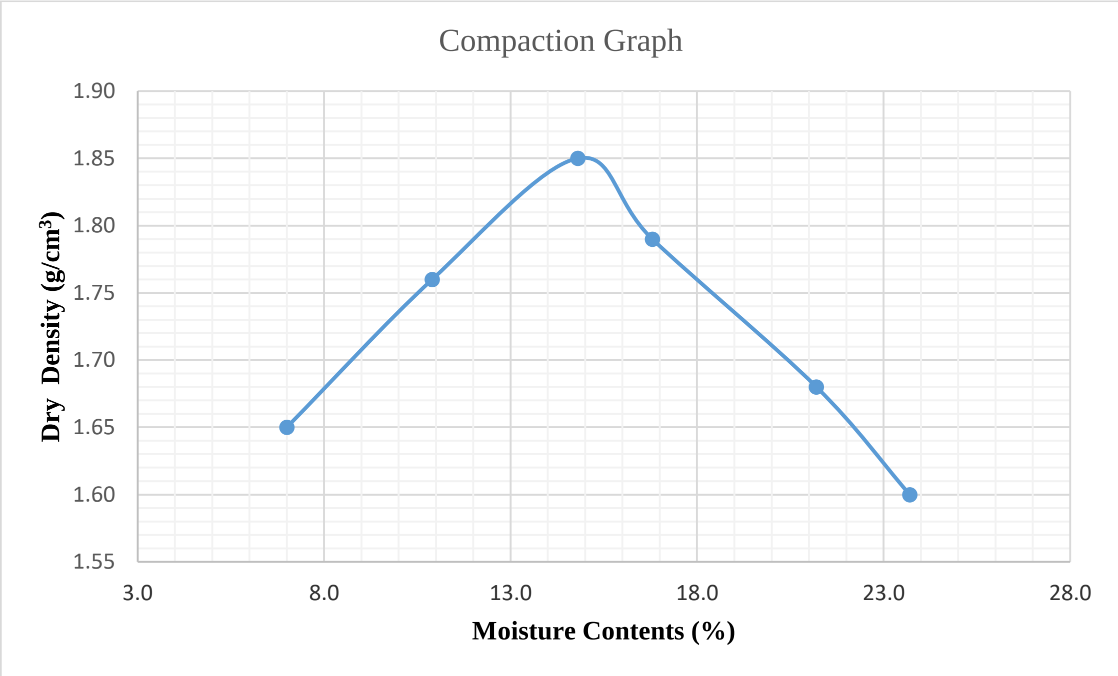

Figure D5: British Standard Light (BSL) at 12% addition of rock flour

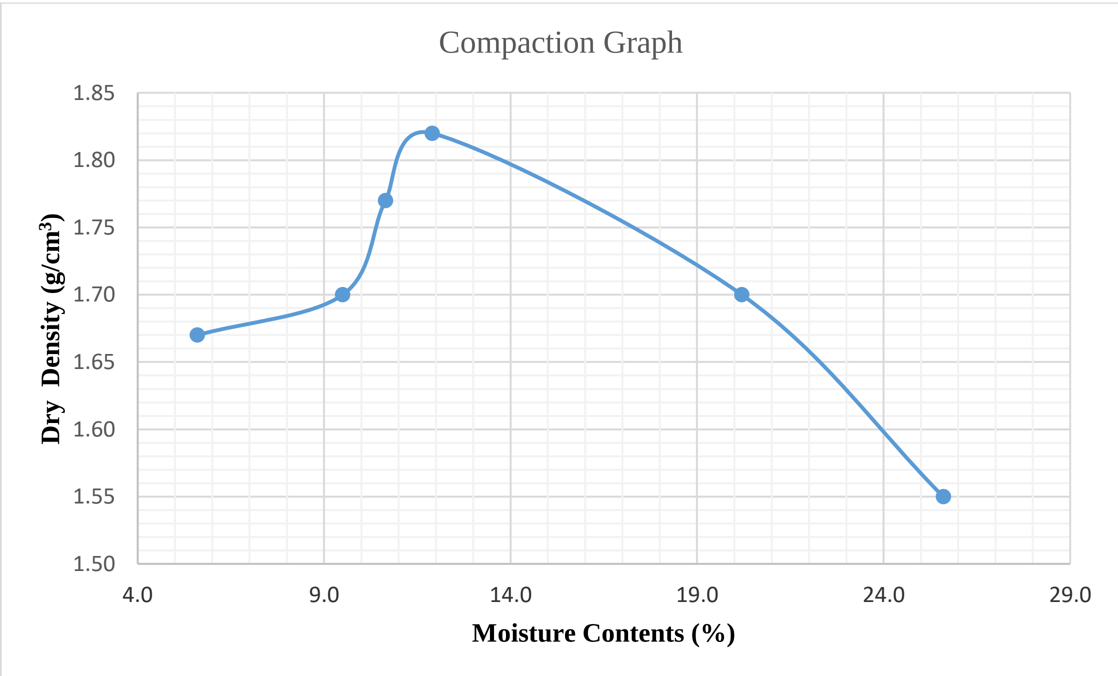

Figure D6: West African Standard (WAS) for natural soil

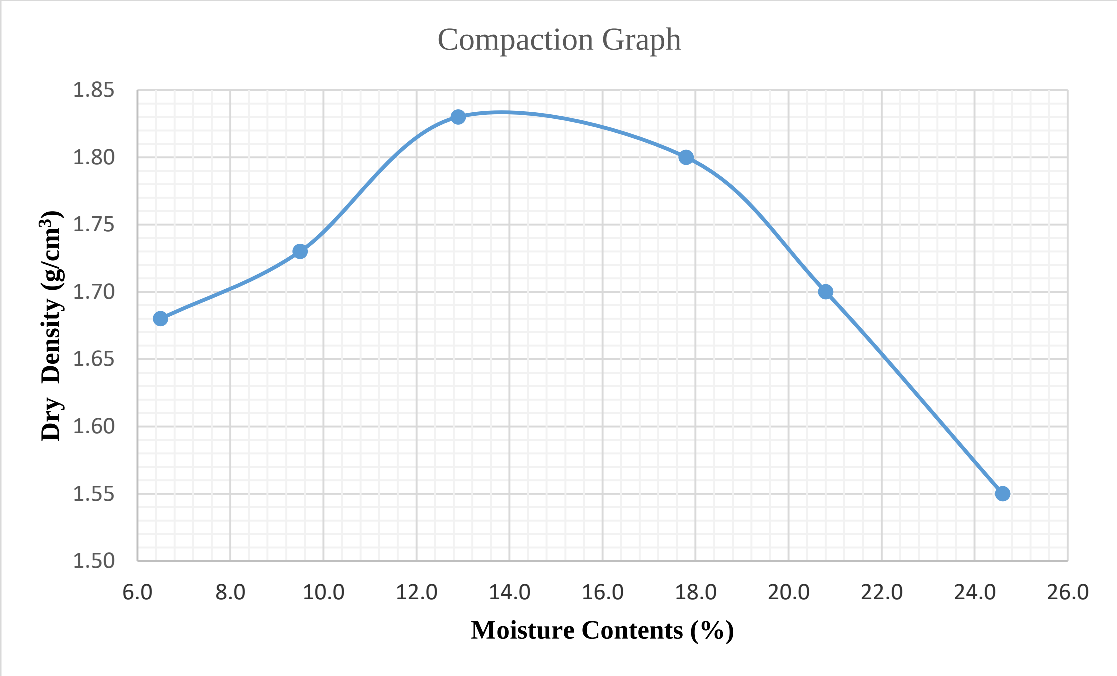

Figure D7: West African Standard (WAS) at 3% addition of rock flour

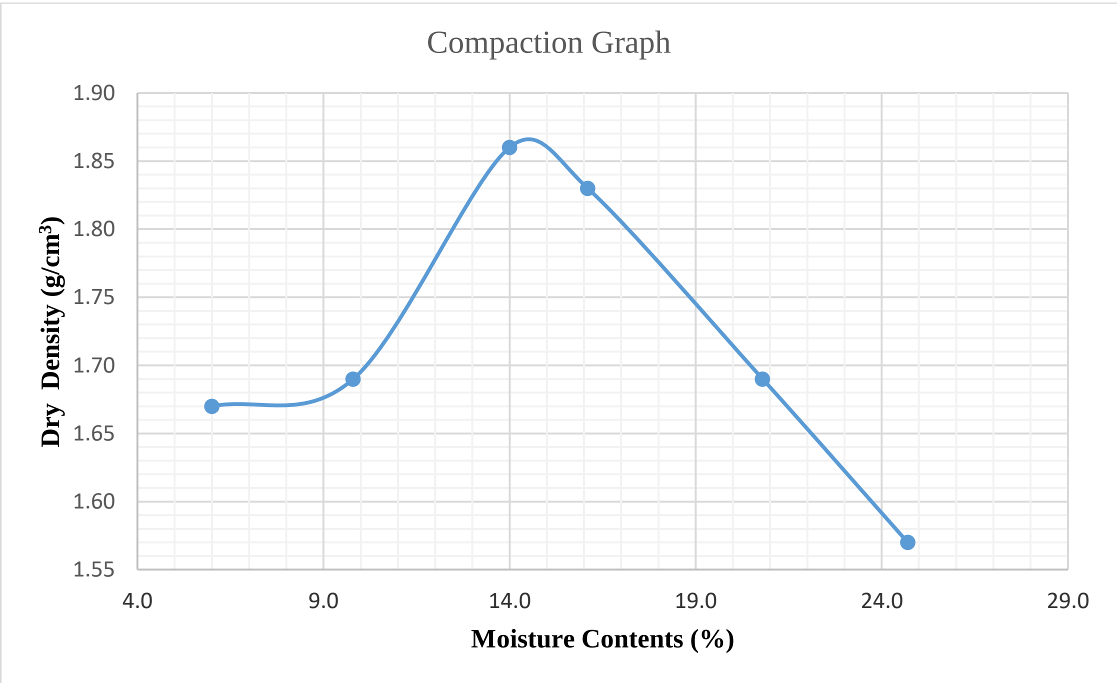

Figure D8: West African Standard (WAS) at 6% addition of rock flour

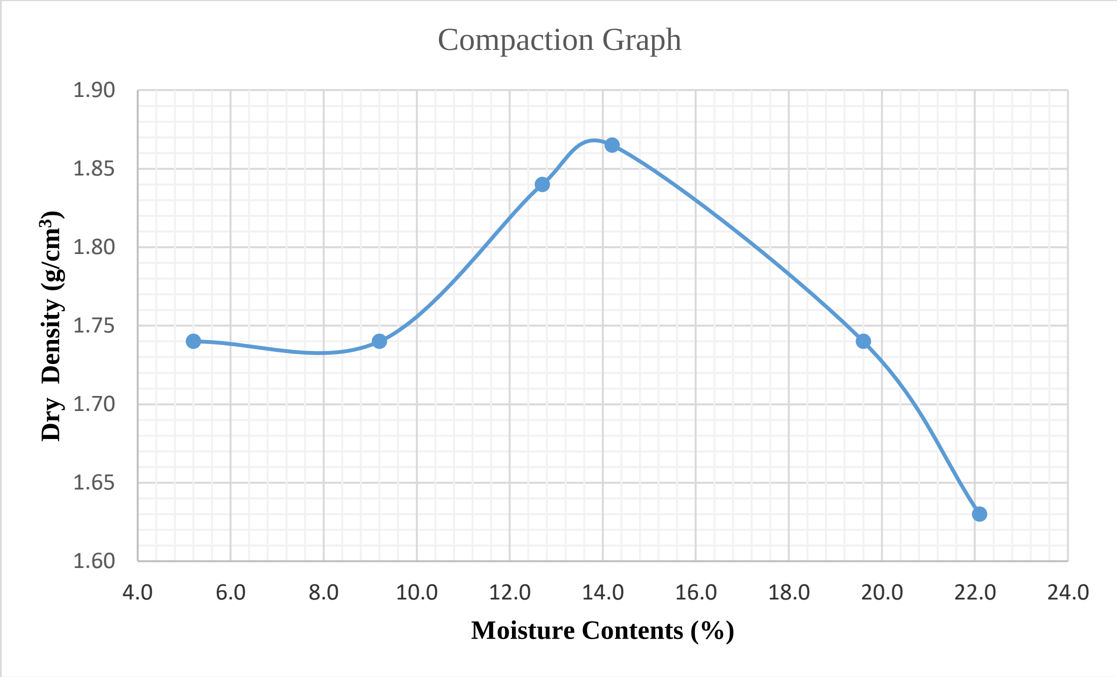

Figure D9: West African Standard (WAS) at 9% addition of rock flour

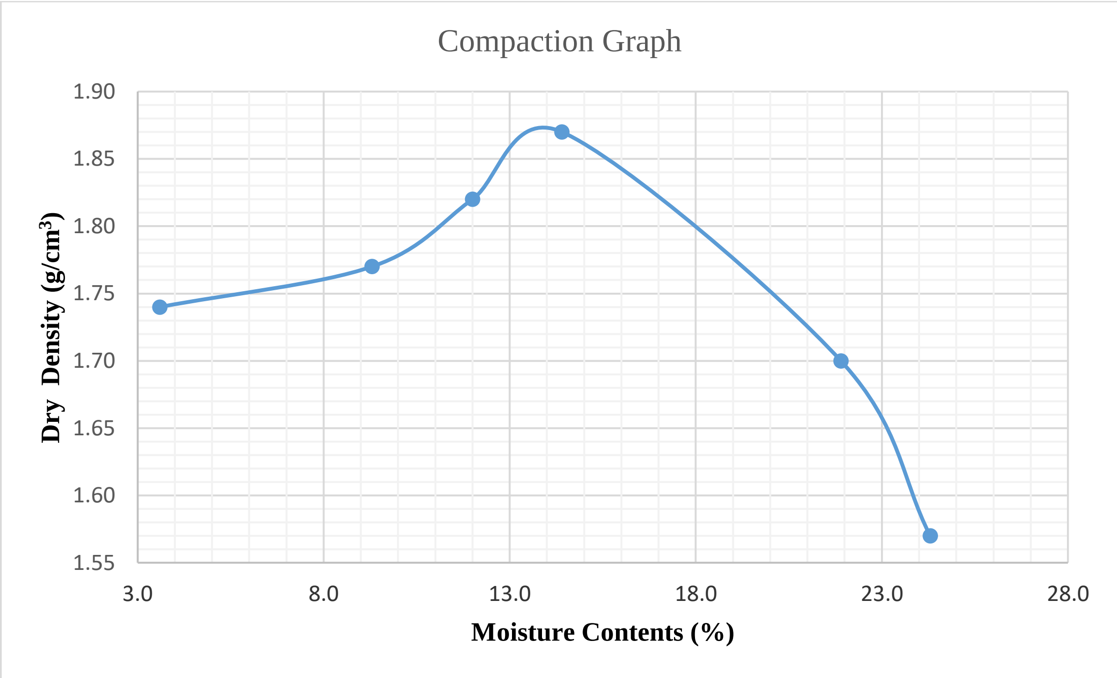

Figure D10: West African Standard (WAS) at 12% addition of rock flour

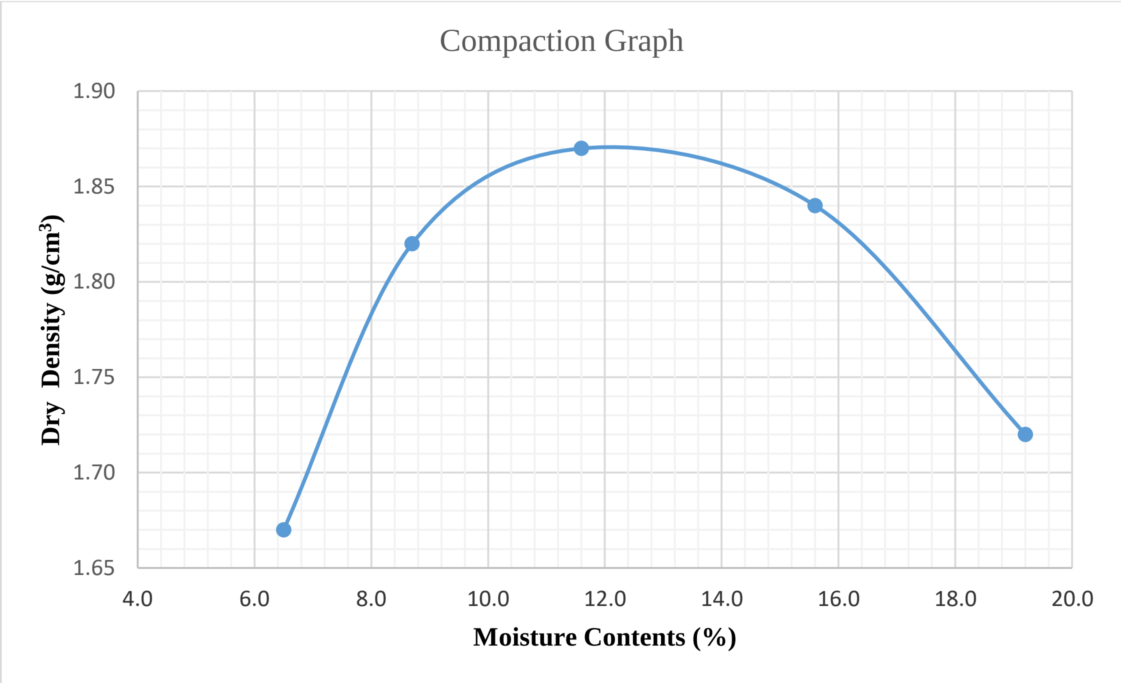

Figure D11: British Standard Heavy (BSH) for natural soil

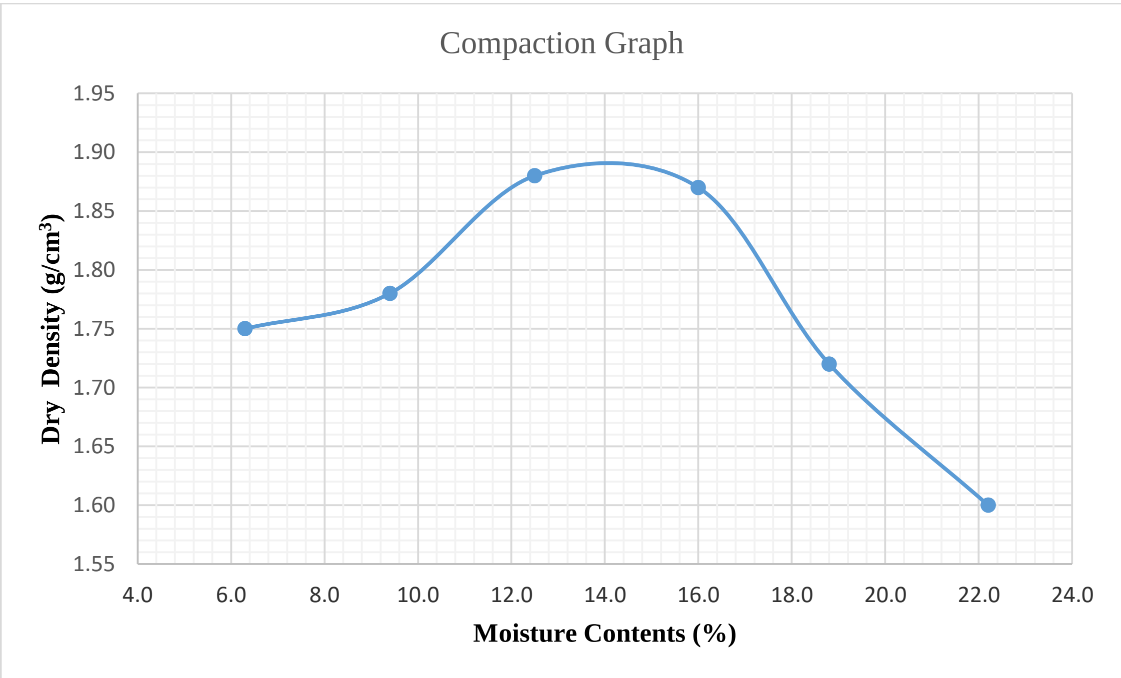

Figure D12: British Standard Heavy (BSH) at 3% rock flour addition

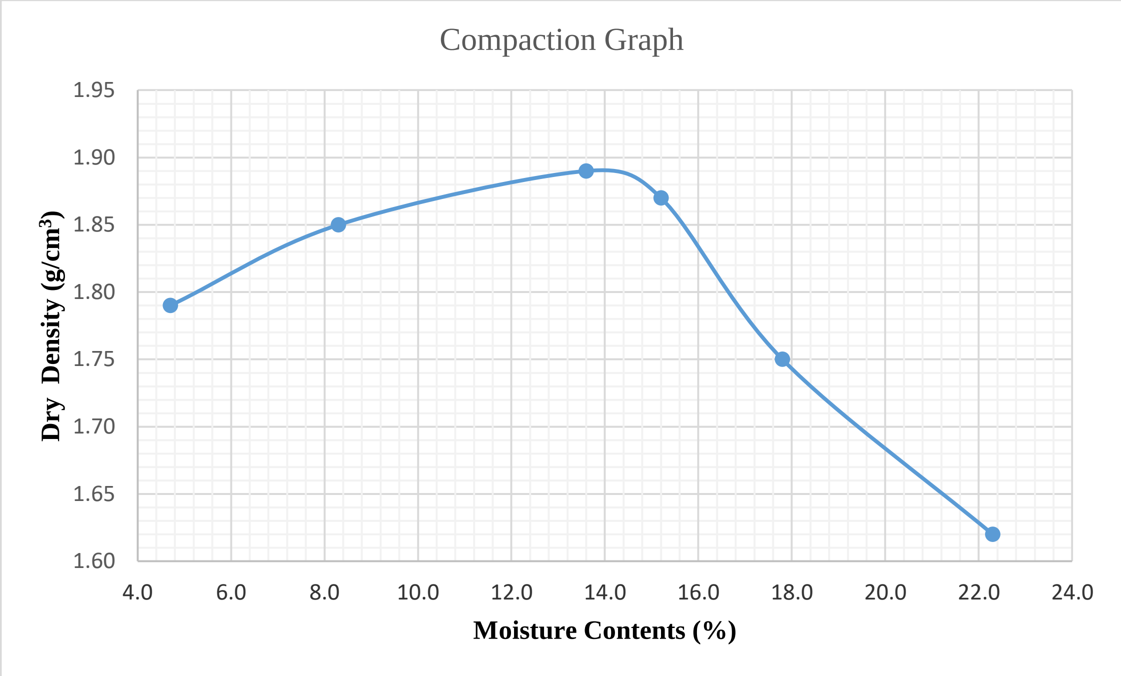

Figure D13: British Standard Heavy (BSH) at 6% rock flour addition

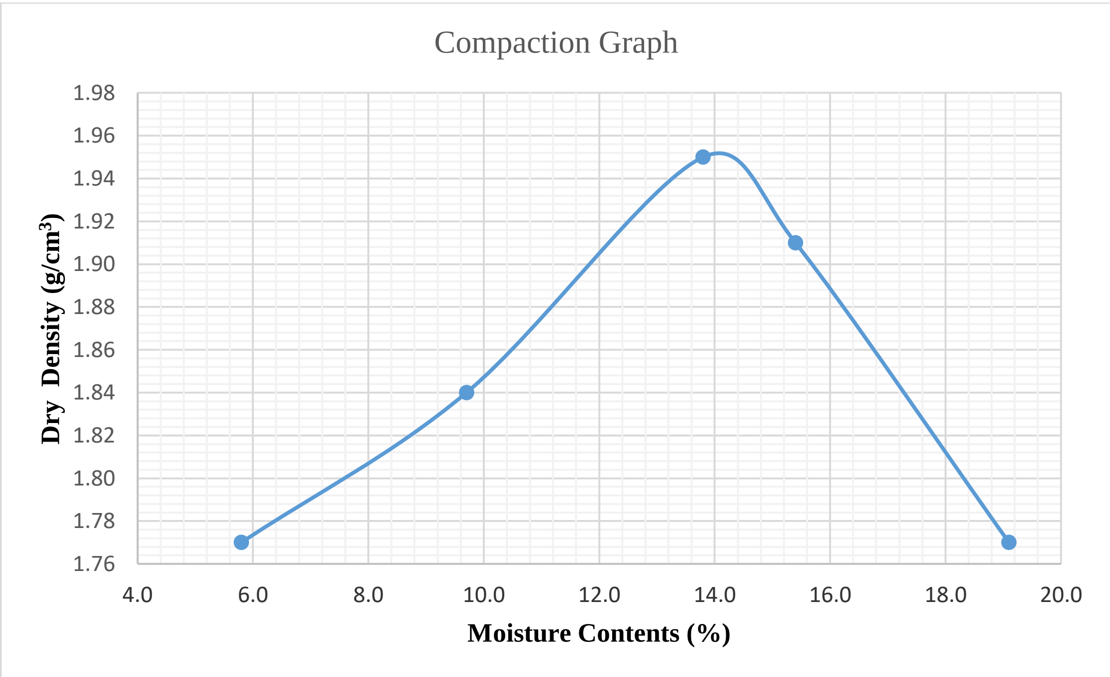

Figure D14: British Standard Heavy (BSH) at 9% rock flour addition

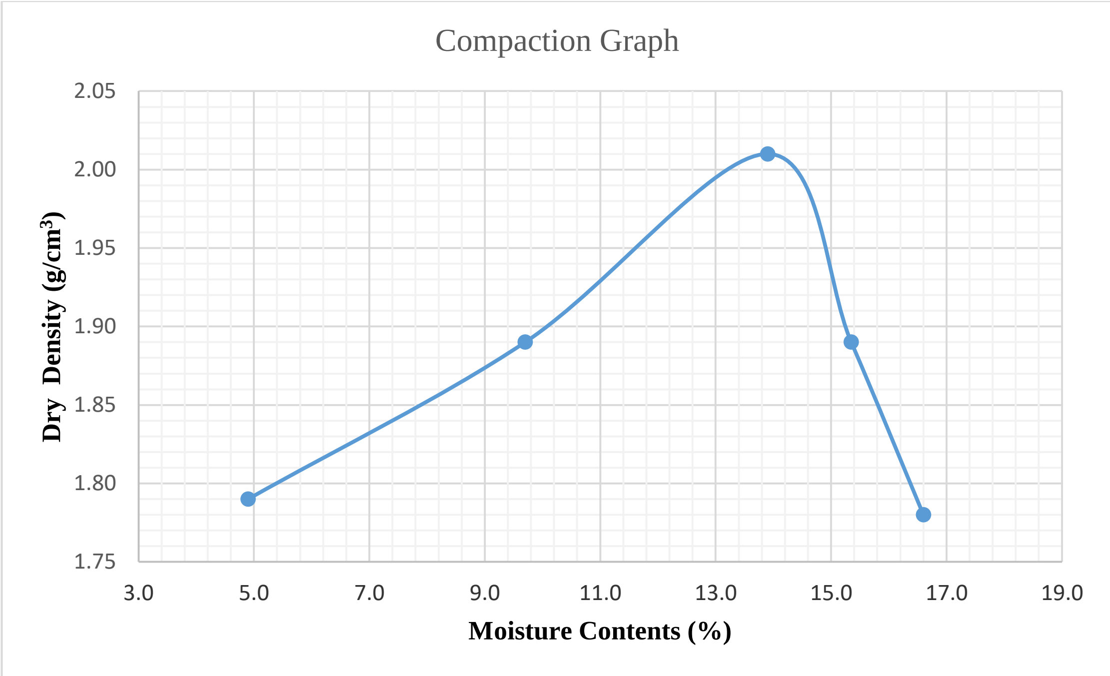

Figure D15: British Standard Heavy (BSH) at 12% rock flour addition

Appendix E: Unconfined Compressive Strength on Soil Specimen

Table E1: Unconfined Compressive Strength for BSL, WAS and BSH

British Standard Light	0	3	6	9	12
C	112.3	78.33	102.57	180.81	67.59

West Africa Standard	0	3	6	9	12
C	147.480	108.870	125.140	218.120	124.620

British Standard Heavy	0	3	6	9	12
C	180.500	130.590	152.360	250.890	119.510

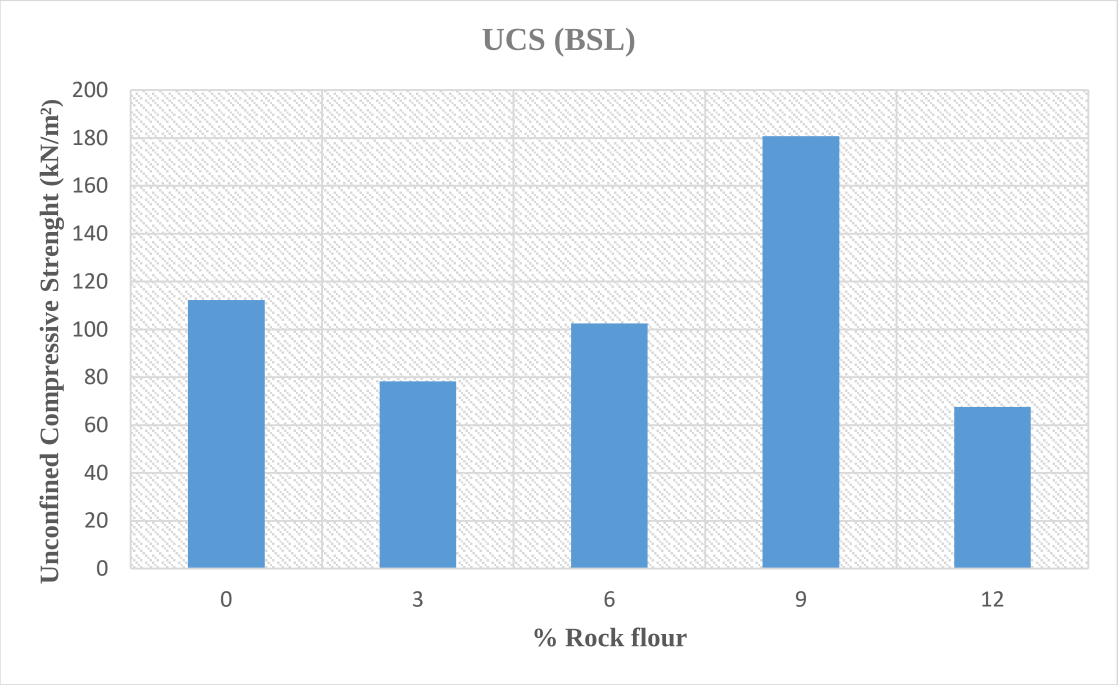

Figure E1: Unconfined Compressive Strength for BSL as rock flour increases form 0 – 12%

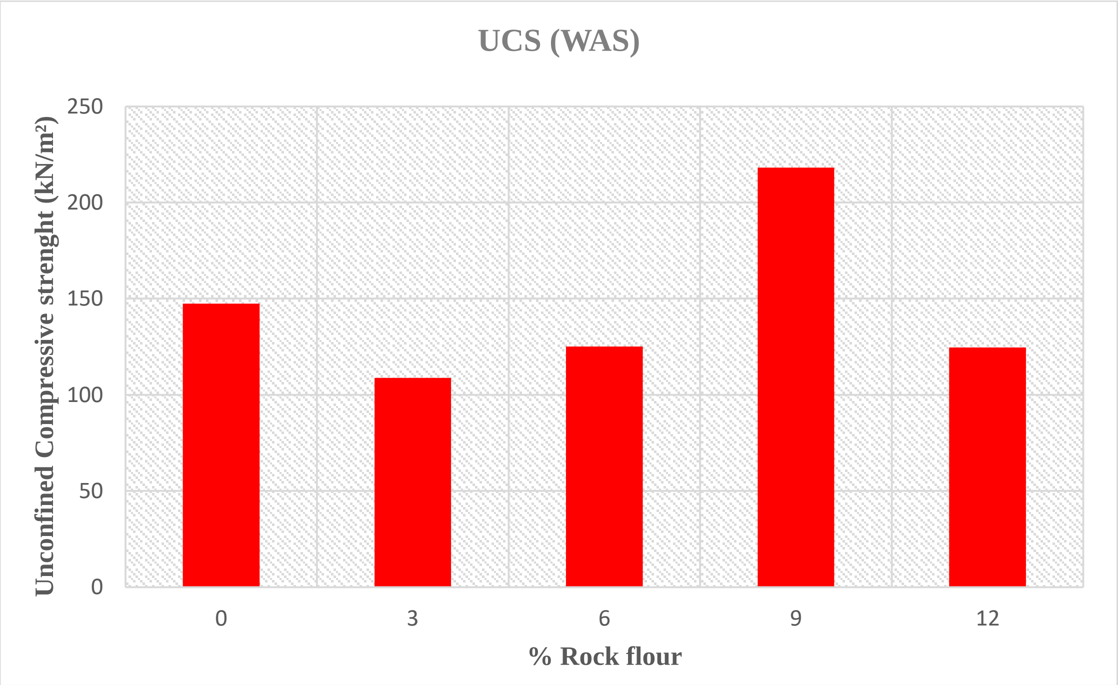

Figure E2: Unconfined Compressive Strength for WAS as rock flour increases form 0 – 12%

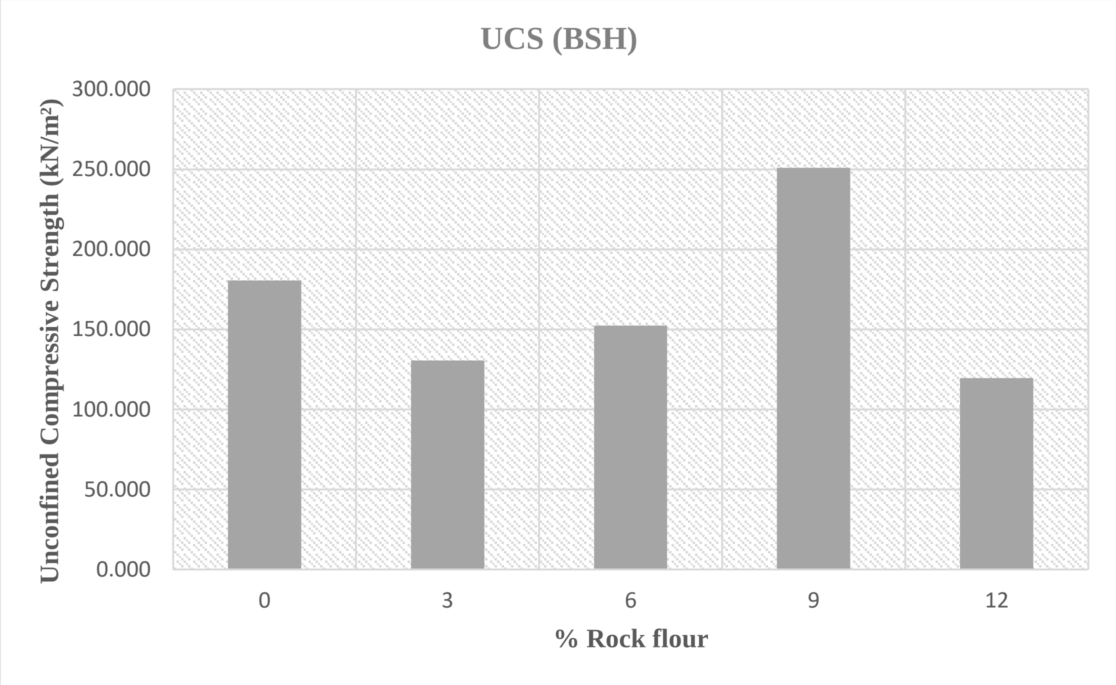

Figure E3: Unconfined Compressive Strength for BSH as rock flour increases from 0 – 12%

Appendix F: Unsoaked and soaked CBR on test Specimens

Table F1: CBR values for BSL, WAS and BSH compaction

British Light	0	3	6	9	12
Unsoaked	30.55	37.88	42.15	46.28	50.94
Soaked	16.25	21.15	25.2	28.25	29.9

British Heavy	0	3	6	9	12
Unsoaked	35.550	42.730	48.120	54.800	59.120
Soaked	22.350	28.210	31.150	34.250	36.200

West Africa	0	3	6	9	12
Unsoaked	32.45	41.74	47.25	52.15	54.7
Soaked	19.200	25.930	29.840	32.220	34.080

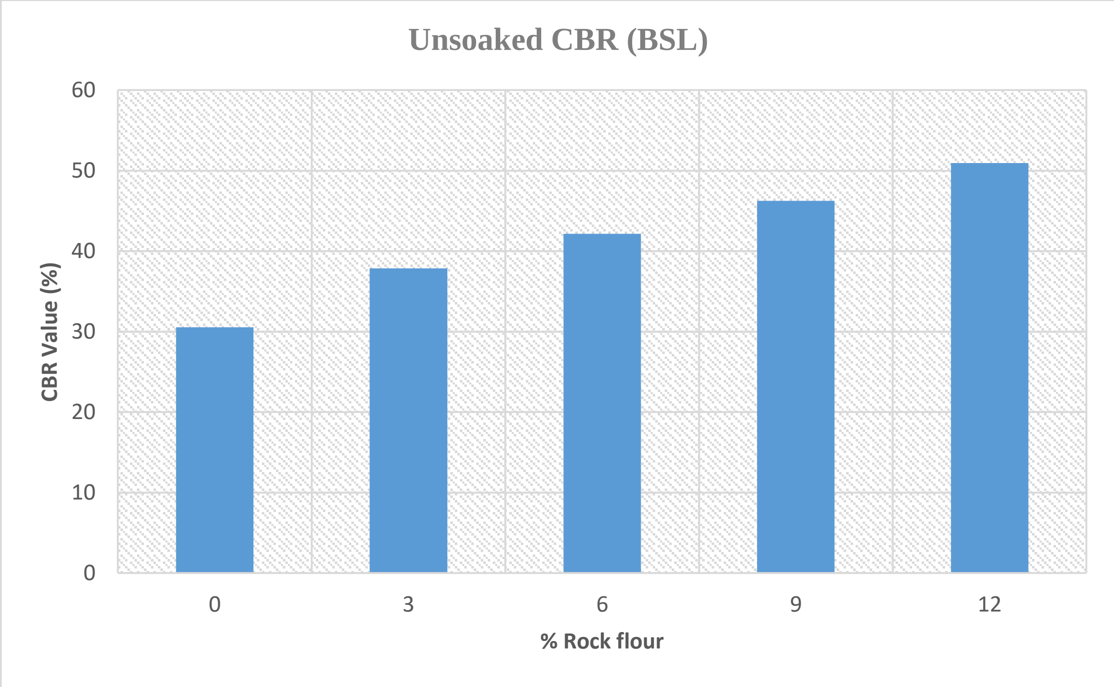

Figure F1: Unsoaked CBR values for BSL as rock flour increases form 0 – 12%

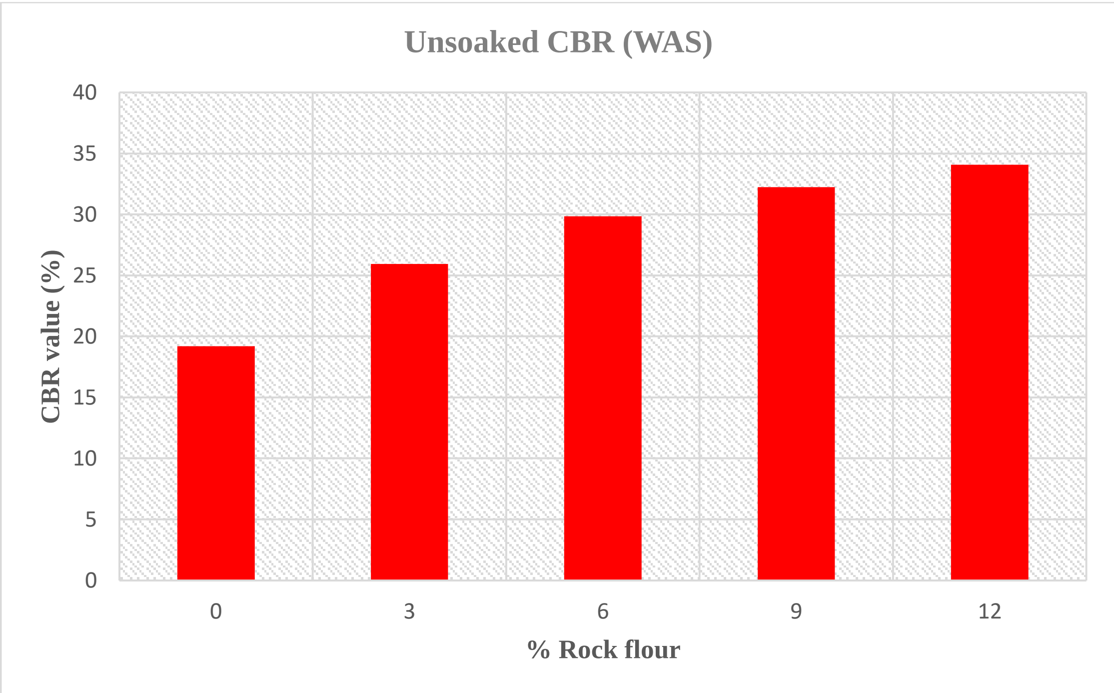

Figure F2: Unsoaked CBR values for WAS as rock flour increases form 0 – 12%

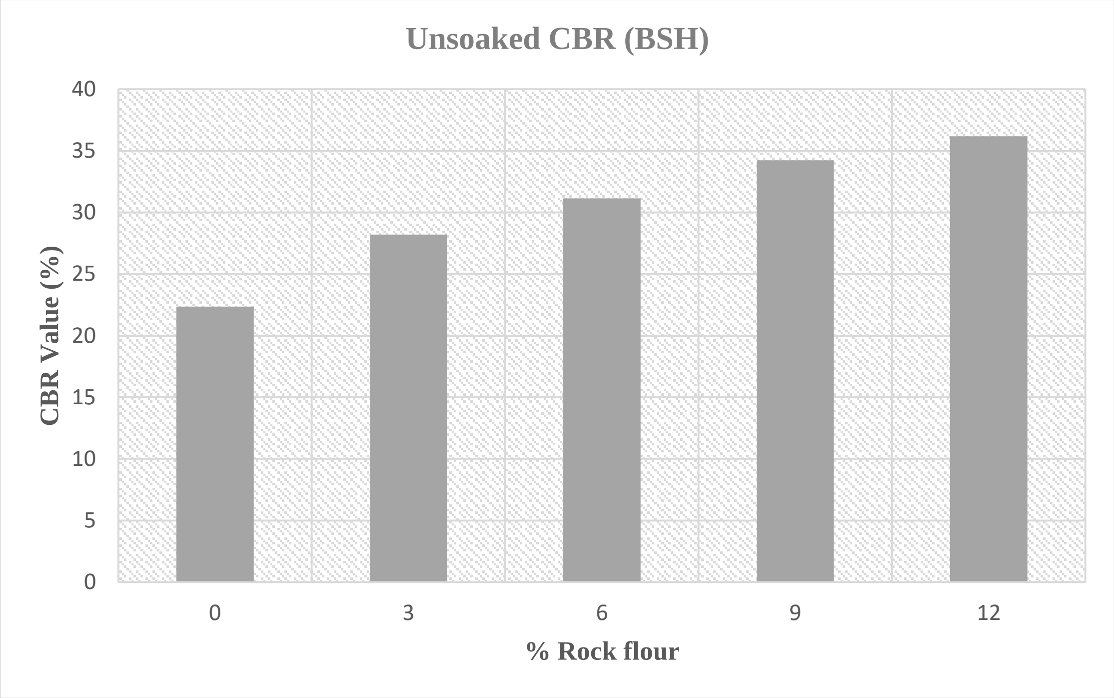

Figure F3: Unsoaked CBR values for BSH as rock flour increases form 0 – 12%

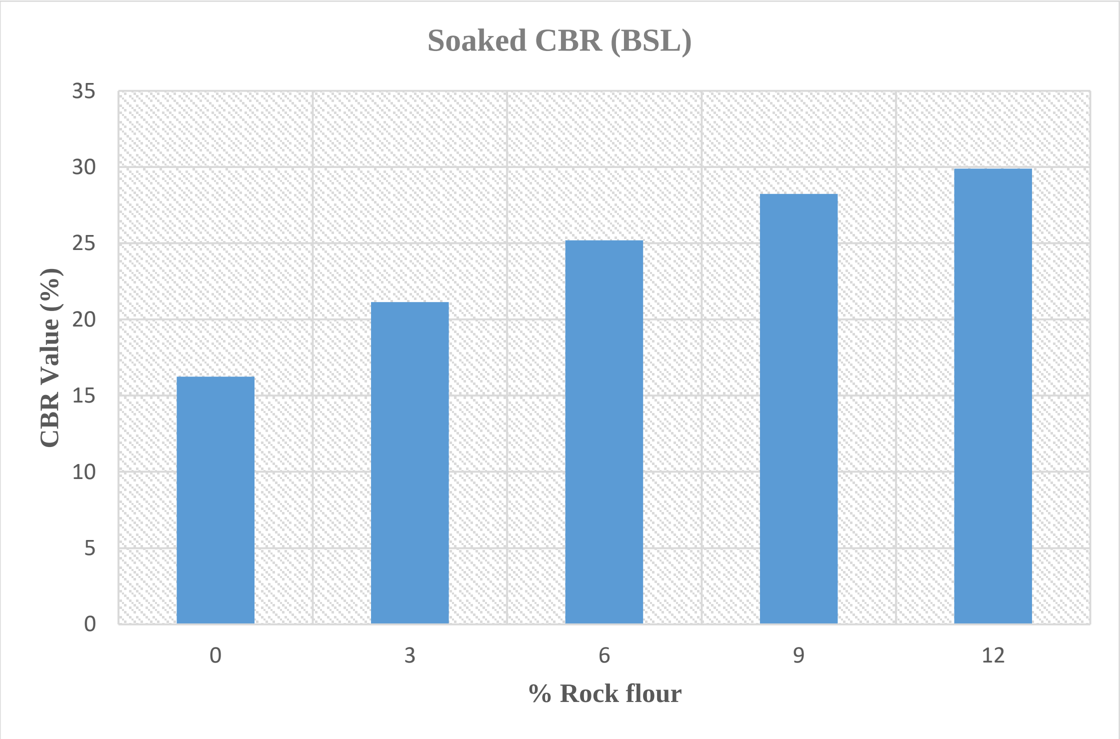

Figure F4: Soaked CBR values for BSL as rock flour increases form 0 – 12%

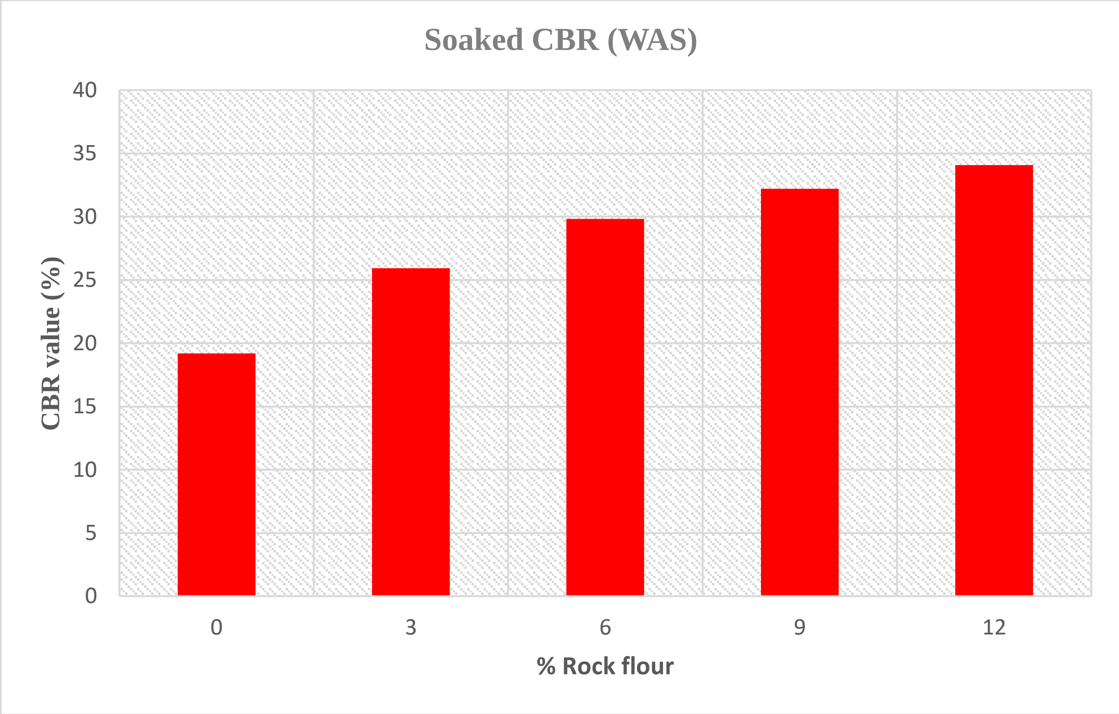

Figure F5: Soaked CBR values for WAS as rock flour increases form 0 – 12%

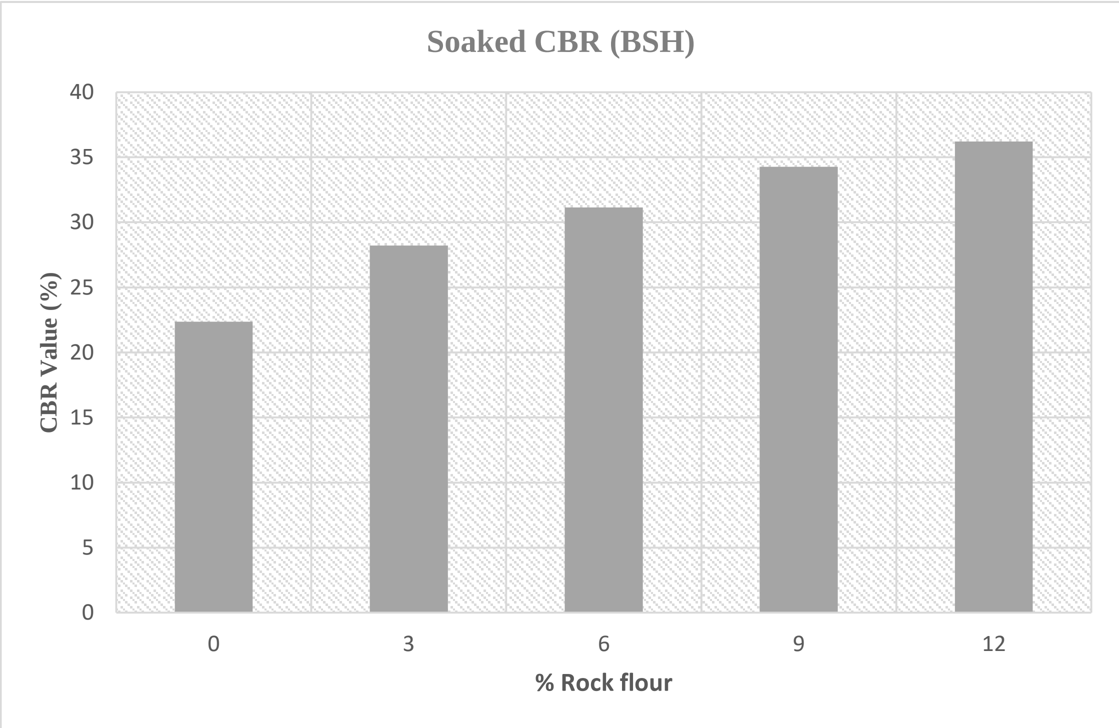

Figure F6: Soaked CBR values for BSH as rock flour increases form 0 – 12%

Appendix G: Durability Test Specimens

Plate I: Prepared samples for durability test

Plate II: Prepared samples for durability test

Plate III: Collapsed durability test specimens