# PERFORMANCE EVALUATION OF FULL DEPTH RECLAIMED SURFACE-DRESSED PAVEMENT TREATED WITH CEMENT AND CALCIUM CARBIDE RESIDUE AS ROAD BASE

**BY**

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**SEPTEMBER, 2021**

## ABSTRACT

Performance evaluation of Full Depth Reclaimed Surface-dressed Pavement (FDRSP), treated with cement and Calcium Carbide Residue (CCR) as a road base material was undertaken. Wearing and base course of a surface-dressed road was scarified and mixed to form the FDRSP, which was found to consist of 28.7 % Reclaimed Surface-dressed (RSP) and 71.3% soil from the base course. Laboratory tests were carried out to determine the most economic mixture of the FDRSP/cement/CCR that will give a Californian Bearing Ratio (CBR) value of 150%, required for heavy traffic roads. The results showed that the original base course material of the road classified under A-2-5, but when mixed with the Reclaimed Surface-dressed Pavement (RSP), the resulting material (FDRSP) classified under A-2-4 according to AASHTO soil classification system. 2% cement and 4% CCR, added to the FDRSP, satisfied the 150% CBR required for heavy traffic roads. From laboratory and field density results for the FDRSP, FDRSP/cement mixture and FDRSP/cement/CCR mixtures, it was observed that more than 95% density can be achieved after 14 days exposure to the traffic load. Field CBR results of the compacted FDRSP/2% cement/4% CCR agreed with the laboratory CBR after 14 days of exposure to traffic load on the road, while the field CBR result of the section with only FDRSP agreed with laboratory values after 7 days, after which the values became higher than the laboratory values.

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## ABBREVIATIONS, GLOSSARIES AND SYMBOLS

| | |
|---|---|
| AASHTO | American Association for State Highway and Transportation Officials |
| ASTM | American Society for Testing and Materials |
| BA | Bagasse Ash |
| BCS | Black Cotton Soil |
| BS | British Standard |
| BSH | British Standard Heavy |
| BSL | British Standard Light |
| CBR | California Bearing Ratio |
| CCR | Calcium Carbide Residue |
| CKD | Cement Kiln Dust |
| CL | clayey soil |
| FA | Fly ash |
| FDR | Full Depth Reclamation |
| FUT MINNA | Federal University of Technology, Minna |
| FWD | Falling Weight Deflectometer |
| Gs | specific gravity |
| HMA | Hot Mix Asphalt |
| LBR | Limerock Bearing Ratio |
| LKD | Lime Kiln Dust |
| LL | liquid Limit |
| M | mass |

| | |
|---|---|
| m | metre |
| MDD | Maximum Dry Density |
| MDUW | Maximum Dry Unit Weight |
| $M_s$ | Mass of compacted soil |
| OMC | Optimum Moisture Content |
| OPC | Ordinary Portland Cement |
| TRRL | Transport and Road Research Laboratory |
| PI | Plasticity Index |
| PL | Plastic Limit |
| SCBA | Sugar Cane Bagasse Ash |
| UCS | Unconfined Compressive Strength |
| USCS | Unified Soil Classification System |
| $V_s$ | Volume of mould |
| WAS | West African Standard |
| $\rho_b$ | Bulk density |
| $\rho_d$ | Dry density |
| $w$ | Natural moisture content |

# CHAPTER ONE

## 1.0 INTRODUCTION

### 1.1 Background to the Study

The rate at which deposits of natural resources are fast depleting has become a global concern. This has prompted the concept of '*use and reuse'* of these resources, which is an aspect of the globally known concept of '*sustainable development'*. Some of the deposits under this threat are those of lateritic soils. Good lateritic soil deposits were initially thought to be inexhaustible, but their current situation (especially in Minna, the capital city of Niger state and environs) have shown that nothing can be farther from the truth as that. Lateritic soil has been extensively used as sub-grade, sub-base and base courses for low to medium trafficked roads in Nigeria (Amu *et al*. 2010) and some other countries, where their deposit exists (Alhaji *et al*., 2019). Some of these soils performed well when used as sub-base and base course materials for road structures, while others have been observed to fall short of the specifications for them to be used as such (Aginam *et al*., 2014; Oghenero *et al*., 2014 and Mustapha*et al*., 2014). In the later situation, the engineering properties of such soils are improved (Alhassan and Mustapha, 2007; Mu'azu, 2007; Osinubi *et al*., 2007; Alhassan, 2008; Osinubi and Alhaji, 2009; Eberemu *et al*., 2012; Sultan and Guo, 2016; Horpibulsuk *et al*., 2017; Alhaji and Alhassan, 2018) to make them fit for the intended use.

In most cases, lateritic soil materials that were initially found to be good for use as road bases become deteriorated with age while in service or during routine maintenance/reconstruction work. In such instances, and considering the current global trend, such materials are now being recycled and reused, curtsey of utilization of recycling/improvement techniques, using locally available and cheap additives. An example of these recycling/improvement techniques is the use of reclaimed pavement surface materials (for example Reclaimed Asphalt Pavement - RAP). In recent past, studies have been carried out on the possibility of using RAP for road pavement structures.

Mohammad *et al.* (2003) investigated the potential use of foamed asphalt treated RAP as a base course material instead of crushed limestone base and concluded that the foam asphalt showed higher in-situ stiffness than limestone base. In an attempt to reuse aged asphalt surface, Gregory and Halsted (2007), used Full Depth Reclamation (FDR) of RAP and the existing base and sub-base materials, mixed with small amount of cement to form new road base material that was considered excellent.

Edeh *et al*. (2012a) investigated the possibility of using reclaimed asphalt pavement-lime stabilized clay as a highway pavement material, and obtained an unsoaked CBR of 36.56% and a 24 hour soaked CBR of 34.23%, concluding that the material could be used for sub-grade and sub-base courses. A study aimed at increasing strength and reducing creep of RAP, by adding high quality aggregate and/or adding chemical stabilizer was carried out by Bleakley and Cosentino (2013), using Limerock Bearing Ratio (LBR) and creep tests to evaluate the strength and creep of the mixture respectively. Ochepo (2014) stabilized A-7-6 lateritic soil using RAP and Sugarcane Bagasse Ash (SCBA) for pavement construction, and observed that the soil, stabilized with 6 and 8% SCBA gave a CBR value that was sufficient for the mixture to be used as subgrade and sub-base courses for road, while that treated with 10% SCBA gave CBR value that was sufficient for the mixture to be used as base course material.

Mustapha *et al.* (2014) worked on possible stabilization of A-6 lateritic soil using RAP without any chemical admixture, and reported minimal increase in Unconfined Compressive Strength (UCS) from 346 kN/m$^2$ for the natural soil to 384 kN/m$^2$ at 40% soil mixed with 60% RAP, while the CBR increased marginally from 45.1% for natural soil to 48.6% at 40:60 mixtures. Alhaji and Alhassan (2018) also investigated the effect of RAP stabilization on the microstructure and strength of Black Cotton Soil (BCS), and reported optimal UCS value of 947kN/m$^2$ at optimal mixture of 30% RAP-70% BCS, representing 54.5% increase, maximum modulus of elasticity (E) of 42.52MPa at same mix ratio, representing 75.5% increase, reduction in free swelling of the compacted mixtures from 16.08% at 0% RAP to 0% at 80%, with 9.99% at optimal mixture of 30% RAP content, translating to 37.9% reduction in free swelling.

Mishra (2015) studied the use of RAP material in flexible pavements in which typical values of unit weight, natural moisture content, asphalt content, compaction densities and CBR values were reported, with the author concluding that 30% replacement of natural aggregate with RAP can successfully be used in base course. The use of geopolymer materials to stabilize RAP for road base courses was carried out by Avirneni *et al.*(2016), with the authors observing that fly-ash stabilization alone could not impact sufficient strength on the RAP-VA mixtures. They therefore concluded that 7 days UCS of the compacted RAP-FA blend at OMC met the strength requirement for base course specified by national road authority.

Alhaji and Alhassan (2018) worked on the microstructure and strength of RAP stabilized clay for road structure, with the result indicating CBR increased from 11% at 0% RAP-100% clay to 35% at 30% RAP-70% clay, after which the values reduced to 5% at 100% RAP- 0% clay.Suebsuk *et al*. (2014)studied effect of RAP on compaction characteristics and UCS of cement-treated soil–RAP mixtures, adopting porosity as a state parameter for assessing strength of the mixtures, with the results showing that as RAP content increases, OMC tended to decrease to an optimum soil-RAP ratio of 50/50. The asphalt fixation point was recorded to be at an asphalt content of 3.5% (50/50 soil-RAP ratio).

Kamel *et al*. (2016) evaluated the suitability of soil-RAP mixture for use as sub-bases, and from an extraction test observed the bitumen content of RAP to be 5.09% and maximum CBR to be 61.2% in a 50% soil-50% RAP mixture. Abukhettala (2016) also investigated the possibility of using RAP for road pavement structure. Rupnow *et al*. (2015) conducted a case study on the stabilization of a RAP-soil mixture with class C fly ash for use as a sub-grade., using Dynamic Cone Penetration (DCP) test to evaluate the strength gain in the field.

From the above, it is evident that a lot have been done on the possibility of using RAP, either alone or mixed with additives, as road pavement structures. Study on the possibility of using Reclaimed Surface-dressed Pavement (RSP) material or Full Depth Reclaimed Surface-dressed Pavement (FDRSP) material, either alone or with additives has not received much attention in the literature. FDRSPmaterial is obtained when surface-dressed layer together with the base course of a surface-dressed road are removed for reuse. This study is therefore intended to investigate the possibility of using this FDRSP material together with cement and Calcium Carbide Residue (CCR) as road pavement structure.

**1.2 Aim and Objectives of the Study**

The aim of the study was to evaluate the performance of full depth reclaimed surface-dressed pavement treated with cement and calcium carbide residue as road base. To achieve the aim, the following objectives were setup to;

i. determine the physical properties of the full depth reclaimed surface-dressed pavement (FDRSP) material.
ii. determine the compaction characteristics of the FDRSP, mixed with various percentages of cement and Calcium Carbide Residue (CCR).

iii. determine the California Bearing Capacity (CBR) FDRSP, mixed with various percentages of cement and CCR.
iv. determine the composition that gave the require 150% CBR value for heavy trafficked road.
v. determine the field performance of the optimum composition, using in-situ density and Dynamic Cone Penetration (DCP) tests.

### 1.3 Scope of the Study

The study focused on performance evaluation FDRSP, mixed with cement and CCR. Compaction and CBR test were carried out on the material mixed with various percentages of the additives. Index properties tests were also carried out on the materials to determine their suitability as base materials for road. Using in-situ density and Dynamic Cone Penetration tests, field investigation was carried out on a section of Morris Fertilizer Company road in Minna, to determine the field performance of the treated FDRSP.

### 1.4 Justification of the Study

Result of the study has enriched the literature on the use of wastes for improvement of the properties of road base materials. This also reduce the problems associated with sourcing of good borrow materials for road construction, and in another way, addressing the problem associated with disposal of aged surface dressed materials.

# CHAPTER TWO

# 2.0 LITERATURE REVIEW

## 2.1 Lateritic Soils

In tropical region of the world, base or sub-base materials for either paved or surface dressed roads are mostly lateritic soils. The recognition of laterite as an earth material, with unique properties, dates back to 1807 when Buchanan first encountered a material in India which he called laterite and defined it as a material,soft enough to be readily cut into blocks but upon exposure to air, quickly becomes hard as brick, and is reasonably resistant to the action of air and water(Persons, 2012).Laterite is rich in iron oxide and derived from a wide variety of rocks weathering under strongly oxidizing and leaching conditions and is most common in tropical and subtropical regions where the climate is humid (Encyclopedia Britannica, 2010). Alexander and Cady (1962) also defined lateritic soils as the products of intensive weathering of rocks that occurs under tropical and subtropical climatic condition, resulting in the accumulation of hydrated iron and aluminium oxides. These soils are products of weathering of rocks under conditions of high temperatures and humidity with well-defined alternating wet and dry season.

Studies have shown that appraisal of geotechnical characteristics and engineering behavior of these soilsappear to depend on the simultaneous consideration of all the major factors that affect the behaviour of rocks and their derived soils i.e, rock type, weathering condition, degree of weathering, type of derived materials as well as their chemical and mineralogical composition (Gidigasu, 1974). Roads constructed in areas with unsuitable lateriticsoils experience problems. In attempting to solve this problem, the usual method is to remove the unsuitable or poor soil and replace it with a competent material or stabilizing it to achieve the required engineering properties (Gidigasu, 1972). Various methods of stabilizing soils have been used.

## 2.2 Soil Stabilization

Soil stabilization refers to the process of changing properties of soils to improve their strength and durability(Afrin, 2017). In earlier times, soil improvement (stabilization) has been in the qualitative sense only, but more recently, it has also been associated with quantitative values of strength and durability, which are related to performance (Amu *et al*., 2005).

Soil stabilization involves utilization of physical, physico-chemical and chemical methods to make deficient soils serve intended purpose as pavement component material (Koteswara, 2011) or any other civil engineering structure. Osinubi and Katte (1997)referred to soil stabilization as the alteration or control of any soil property. It covers not only the increase or decrease of the properties, but also their variation with changes in environmental condition (mostly moisture or pressure). Primarily, the objectives of soil stabilization are to improve strength, decrease permeability and water absorption and improve bearing capacity and durability under cyclic conditions of varying moisture content (Eren and Filiz, 2009). Soil stabilization has been used in the building of roads, aircraft runways, earth dams/embankment and in erosion control (Diamond, 1975; Kawamura and Diamond, 1975). Soil stabilization falls into two broad categories namely: mechanical stabilization and chemical stabilization.

### 2.2.1 Mechanical stabilization

Mechanical stabilization involves soil densification through application of mechanical energy, which brings about reduction of air void in the soil, with little or no reduction in water content, and also lowers permeability with increase in dry density (Markwick, 1945). Mechanical stabilization improves soil properties by mixing other soil materials with the target soil to change the gradation and therefore change the engineering properties(Afrin, 2017). A soil structure is said to be mechanically stable when it can resist lateral displacement under load(Guthrie *et al.,*2017).A stable soil can be obtained through controlled grading (of the coarse aggregate, fine aggregate, silt and clay, correctly proportioned) and compaction(Bahar *et al.*, 2004).

Over the years, several methods have been used to achieve mechanical stabilization which includes: compaction employing heavy weight to increase soil density by applying pressure from above, with care taken to avoid over compaction which could lead to crushing of the aggregates, thereby losing its engineering eminence.Soil Reinforcement, which is an engineered solution to soil problems, involves the use of geo-textiles and plastic mesh, designed to trap soils and help control erosion, moisture conditions and permeability(Patel, 2019). Larger aggregates such as gravel, stones and boulders are often employed where additional mass and rigidity can prevent soil migration or improve load-bearing properties(Rogers and Glendinning, 1997).

Addition of graded aggregate materials is a common method of improving engineeringproperties of a soil through introduction of certain aggregates that lend to the soil desirable attributes such as increased strength or decreased plasticity(Road Packer Solutions, 2017). This method provides material economy, improves support capabilities of subgrades and provides good working platform for othercivil engineering structures (Onyelowe and Aguwamba, 2012).Mechanical Remediation has been an accepted practice in dealing with soil contamination(Abioye, 2011). This technique involves physical removal and relocation of contaminated soil to a designated hazardous waste facility far from centers of human population. In recent times however, chemical and bioremediation have proven to be a better solution both economically and environmentally (Road Packer Solutions, 2017).

#### 2.2.1.1 Physical stabilization

Physical stabilization consists of modifying the properties of soil by intervening with its texture (granulometry treatment, heat (dehydration or freezing) or electric (electrosmosis) treatments that lead to drainage of the soil and therefore conferring new structural properties (Rakotonimaro *et al.,* 2017). Physical stabilization may also involve introduction of synthetic fibers or fibers originating from plants, animals and minerals into soil. This method is used when there need not to alter the particle size distribution of the soil or if the material is sensitive to movements, induced by factors such as water action and thermal expansion(Lemougna *et al.,* 2011). These movements can then be countered by the frame made of fibers (Rakotonimaro *et al.,* 2017).When mechanical methods of soil stabilization are inadequate and replacing an undesirable soil with a desirable one is not possible or is too costly, chemical stabilization is employed.

### 2.2.2 Chemical stabilization

According to Gregory and Tuncer (2009), soil stabilization using chemical admixtures is the oldest and most widespread method of ground improvement. Stabilization of soil can be achieved with a variety of chemical additives such as lime, fly ash, by-products such as Lime-Kiln Dust (LKD) and Cement Kiln Dust (CKD), addition of cementitious or pozzolanic materials to improve the soil properties (Winterkorn and Pamukçu, 1991).Over the years, chemical stabilization has traditionally relied on Portland cement, lime and bitumen. However, there are a variety of non-traditional additives available from the commercial sector such as polymer emulsions, acids, lignin derivatives, enzymes, tree resin emulsions, and silicates (Rakotonimaro *et al.,* 2017)that have been considered for soil improvement. Inorganic salts such as sodium chloride and calcium chloride have also long been used in stabilization(Sani *et al.*, 2019). Their main function is to reduce plasticity and facilitate densification (Sani *et al.*, 2019), which are determined from testing.

Proper design and testing is an important component of any stabilization project. This allows for establishment of design criteria as well as determination of the proper chemical additive and admixture to be used to achieve desired engineering properties (Gregory and Tuncer, 2009).Chemical admixtures are often used to stabilize soils when mechanical methods of stabilization are inadequate and replacing an undesirable soil with a desirable one is not possible or is too costly. When selecting a stabilizer additive, the factors that must be considered are:type of soil to be stabilized, purpose for which the stabilized soil layer will be used, type of soil quality improvement desired, required strength and durability of the stabilized soil layer, cost and environmental conditions(Gregory and Tuncer, 2009). Various studies have been carried out on stabilization of deficient soils using chemical additives.

#### 2.2.2.1 *Cement*

Cement can be described as a material with adhesive and cohesive properties which make it capable of bonding mineral fragments into a solid mass of adequate strength and durability (Encyclopedia Britannica, 1999). In general, however for soil to be stabilised with cement, it should have a PI of less than 30% (Anil and Ahsan, 2017). Portland cement can be used either to modify and improve the properties of soil or to transform it into a cemented mass, which significantly increases its strength and durability (Lea, 1965). The amount of cement to be added depends on whether the soil is to be modified or stabilized. Anil and Ahsan (2017) found that with higher quantity of cement added to soil, dry density decreased and optimum moisture content increased. Addition of cement to the soil, increased unconfined compressive strength which was also found to increase at higher curing period.

Joel and Agbede (2011) attempted to improve the physical and strength properties of a lateritic soil using cement and sand. 15–60% sand was used as a modifier, together with 3–12% of cement. Classification, compaction, California Bearing Ratio (CBR), and Unconfined Compressive Strength (UCS) tests were carried out on the stabilized soil. The results indicated decreased in plasticity index from 17% for the untreated soil to 2.5% when treated with a combination of 60 and 6% sand and cement respectively. For the two energy levels of compaction considered, Optimum Moisture Content (OMC) was found to increase with increase in cement content but decreased as the sand content increased. The OMC at the West African Standard (WAS) energy level was consistently lower than the values obtained at the Standard Proctor (SP) energy level, while the corresponding values of Maximum Dry Density (MDD) were higher at WAS energy level than those at SP energy level. The CBR requirements for base course material were met when the soil was admixed with 45% sand +6% cement and 30% sand +6% cement, and 15% sand +6% cement, 30% sand +6% cement, and 45% sand +3% cement at the SP and WAS energy levels, respectively. It was therefore concluded that sand enhanced the effective stabilization of laterite with cement within the maximum cement content specified by the Nigerian code. Lime also has cementitious material that makes it suitable for soil stabilization.

#### 2.2.2.2 *Lime stabilization*

Lime stabilization technology is widely used in geotechnical and environmental applications. Lime has been used in the past in one form or the other to improve the engineering behavior of clayey soils. Lime stabilization may refer to pozzolanic reaction in which pozzolana materials reacts with lime in presence of water to produce cementitious compounds (Sherwood, 1993, EuroSoilStab, 2002). The effect can be brought by either quicklime ($CaO$) or hydrated lime [$Ca(OH)_2$]. Slurry lime can also be used in dry soils conditions where water may be required to achieve effective compaction (Afrin, 2017). There are two phases of stabilization in a lime-soil system(Makusa, 2012).The first is an immediate reaction of cation exchange; the second is flocculation-agglomeration(Sherwood, 1993). The second one occurs to some extent with all fine-grained soils.Due to textural changes caused by these reactions, the soils are improved. These improvements are reflected in improved workability, immediate strength improvement and reduce swell susceptibility (Little, 1998). When lime is mixed with clayey material in the presence of water, several chemical reactions take place. They include cation exchange, flocculation-agglomeration, pozzolanic reaction, and carbonation (Mallela *et al*., 2004). Cation exchange and flocculation-agglomeration are the primary reactions that take place immediately after mixing soil with lime (Makusa, 2012 and Sherwood, 1993). During these reactions, the monovalent cations that are generally associated with clay minerals are replaced by the divalent calcium ions. These reactions contribute to immediate changes in plasticity index, workability, and strength gain in soil (Sherwood, 1993). Because 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 (Clare and Cruchley, 1957; Ormsby and Kinter, 1973; Locat *et al*., 1990).Lime stabilization will result in the plasticity of the soil and an increase in the soil strength. Lime is generally restricted to warm to moderate climates since lime-stabilized soils are susceptible to breaking under freezing and thawing (Ismeik and Shaqour, 2018).

Eades and Grim (1960) found that there exists a chemical reaction between lime and pure clay minerals (kaolinite, illite and montmorillonite) with accompanying increase in bearing capacity. The quantity of lime needed to effectively treat a soil to develop increased strength varies with the type of clay mineral present in the soil. Akawwi and Al-Kharabsheh (2002) reported that swelling and shrinkage potential of soils are affected by mineralogical constituents and surrounding environment. The optimum amount oflime for maximum strength gain in stabilizing soil with lime, according to Eades and Grim (1960) is 4 - 6 % for kaolinite dominated soils and about 8 % for illite and montmorillonite dominated soils.

In recent times, industrial wastes have also been studied for use as soil stabilizers.

## 2.3 Industrial Waste Stabilization

Industrial wastes are usually by products of various industrial processes and are formed during or as result of the process involved. They are usually dumped in the open or stored in ponds for disposal in the vicinity of the industrial plants (Ferronato and Torreta, 2019). There are several industrial wastes that are produced all over the world that can be used in soil stabilization. Some of these wastes that have been used in soil stabilization include fly ash, iron ore tailing, phosphogypsum, cement kiln dust, steel slag, silica fume, lime kiln dust, waste water sludge ash, calcium carbide residue, glass waste, limestone waste ash, cement bypass dust, copper slag and granulated blast furnace slag.

### 2.3.1 Bone ash as a soil stabilizer

Bone is a dynamic tissue that performs mechanical, biological, and chemical functions. The main component of bone is hydroxyapatite as well as amorphous forms of calcium phosphate, possibly including carbonate. Bone chemical and physical properties are affected by age, nutrition, hormonal status, and diseases (Loveridge, 1999). Cattle bones are the source of production of bone ash. Bone ash is grey-white powdery ash obtained from the burning (calcination) of bones. It is primarily composed of calcium phosphate. Calcination is known as a process of high-temperature heating in the presence of atmospheric oxygen. The end product being pure bone mineral, a compound related to hydroxyapatite. All organic materials are combusted to $CO_2$. Bone ash is significant because some of its important properties are due to the unique cellular structure of bones that is preserved through calcination (Ayininuola *et al.,* 2009). Bone ash has excellent nonwetting properties; it is chemically inert, free of organic matters and has very high heat transfer resistance. According to Ayininuola and Sogunro (2014), calcined bone ash contains the following: CaO (45.53%), $P_2O_5$ (38.66%), MgO (1.18%), $SiO_2$ (0.09%), $Fe_2O_3$ (0.1%), $Al_2O_3$ (0.06%) and Moisture (0.11)

### 2.3.2 Fly Ash

Fly ash is one of the most plentiful and industrial by-products. It is generated in vast quantities as a by-product of burning coal at electric power plants (Senol *et al*., 2006). Fly ash generated by coal combustion based power plants typically fall within the ASTM fly ash classes C and F (Reyes and Pando, 2007). Yudhbir and Honjo (1991) stated that pozzolanic fly ashes can be advantageously used to improve geotechnical properties of black cotton soils.

Bhuvaneshwari *et al.* (2005) studied stabilization of expansive clay using fly ash. In the study, highly expansive soil was mixed with different percentages of fly ash (15, 20, and 30%), and the test results showed unconfined compression stress of the clay increasing from 114 to 123 $kN/m^2$ on addition of 20% fly ash. The liquid and plastic limits were reduced from 74.4 to 72.5% and 38.4 to 32.93% respectively, with increases in fly ash up to 30%. Also, the result depicted that maximum dry density, at 14% optimum moisture content, increased from 1.68 to 1.71 $g/cm^3$, on increasing fly ash to 30%.

Takhelmayum*et al*. (2013) studied effect of fly ash on expansive soil by treating the soil with fly ash, at 5% increment. At 20 % the Liquid limit (LL) was observed to reduce by 55%. Plasticity index reduced by 86% at 20% fly ash. At 15% fly ash, 75% decrease was noticed in swelling index of the soil.Beeghly (2003) used a mixture of lime and fly ash to stabilize a pavement subgrade soil and reported that the strength gain was sufficient for a stable subgrade.

### 2.3.3 Calcium carbide residue

Calcium Carbide Residue (CCR) is a by-product from acetylene gas production. This gas is used around the world for welding, lighting, metal cutting and to ripen fruit (Kumrawat*et al.*, 2014). CCRis obtained from a reaction between calcium carbide and water to form acetylene gas and calcium hydroxide in a slurry form which mainly consists of calcium hydroxide $Ca(OH)_2$ along with silicon dioxide $SiO_2$, $CaCO_3$ and other metal oxides(Zhang *et al.*, 2019 ). For high content of natural pozzolanic materials in clayey soil, calcium hydroxide [$Ca(OH)_2$] is a rich material that can be used as soil stabilizer(Horpibulsuk *et al.*,2012). For environmental and economic impact, such waste rich materials can be utilized collectively with natural pozzolanic material in clay to produce a cementitious material. Calcium carbide residue production is described in the following equation:

$$\underset{\text{(Calcium Carbide)}}{CaC_2} + 2H_2O \rightarrow \underset{\text{(Acetylene)}}{C_2H_2} + \underset{\text{(Calcium Hydroxide or CCR)}}{Ca\,(OH)_2} \qquad (2.1)$$

From the Eqn,Kumrawat and Ahirwar (2014) stated that 64g of calcium carbide ($CaC_2$) will produce 26g of acetylene gas ($C_2H_2$) and 74g of CCR in terms of $Ca(OH)_2$. Table 2.2 shows the chemical composition of CCRas reported by Sun*et al.,* (2015).

**Table 2.2: Chemical composition of dry CCR**

| Ingredient | Content (%) |
|---|---|
| $Ca(OH)_2$ | 92 |
| $CaCO_3$ | 2.9 |
| $SiO_2$ | 1.32 |
| $Fe_2O_3$ | 0.94 |
| $Al_2O_3$ | 0.06 |
| LOI (loss on ignition) | 1.02 |

Source:Sun*et al*. (2015)

#### 2.3.3.1 *Use of calcium carbide residue for stabilization of soils*

Studies have been carried out on the use of calcium carbide residue to stabilize soils. Investigation of calcium carbide residue as a stabilizer for tropical sand for use as pavement material was carried out by Akinwunmi *et al.* (2019).The soil used was classified as well-graded sand with silt (SW-SM), and application of CCR to the sand resulted in reduction of plasticity index, specific gravity and Maximum Dry Unit Weight (MDUW), while the OMC, CBR and UCS of the soil increased. They attributed this to reaction of the CCR with some of the constituent of the soil. Based on strength, an optimal CCR content of 4% was recommended for stabilization of soils with similar engineering properties. Comparing the results of particle size distribution, Atterberg limits and CBR of the soil with the Nigerian General Specification (1997),the results indicated that the unstabilized and CCR-stabilized soils only satisfied the requirements for use as subgrade materials. Though the stabilized soil satisfied the particle size and CBR requirements for use as sub-base material, it did not meet the plasticity requirement. The authors recommended that further research work should be carried out to investigate addition of low percentage of cement with the CCR-soil mixture in order to further strengthen the soil and also reduce the plasticity index. Base and sub-base soil have also been treated with reclaimed road surfacing materials (e.g. Reclaimed Asphalt Pavement), for improvement of strength and durability.

### 2.4 *Soil Stabilization using Reclaimed Asphalt Pavement (RAP)*

Reclaimed Asphalt Pavement (RAP) is defined as pavement materials containing asphalt and aggregates which have been removed and reprocessed. When properly crushed and screened, RAP consists of high-quality, well-graded aggregates coated by asphalt cement (Jirayut *et al.,* 2014). RAP is gotten from Hot Mix Asphalt (HMA) layer of an existing roadway; a mixture of aggregate coated by bitumen and collected from failed asphalt pavement surfaces (Cooper, 2011).

Full Depth Reclamation (FDR) refers to the removal and reuse of HMA and the entire base course layer and part of the underlying subgrade, implying a mixture of pavement layer materials (Gregory and Tuncer, 2009). RAP has been in use in most developed countries for more than 30 years. It has been used as aggregate in the cold recycling of asphalt paving mixtures either by method of cold mix plant recycling or cold in place recycling processes. Some of the engineering properties of RAP that are of importance when used include its gradation, asphalt content, and the penetration and viscosity of the asphalt binder. Aggregate gradation of RAP is somewhat finer than that of the virgin aggregate. This is due to mechanical degradation during asphalt pavement removal and processing. RAP aggregates usually can satisfy the requirements of ASTM D692 (2000) for coarse aggregate and ASTM D1073 (2016) for fine aggregates.

Osinubi and Bajeh (1994) carried out stabilization of lateritic soil using bitumen and discovered that the MDD of the mixture decreased with increase in the bitumen content. However, there was moderate increase in the CBR of the bitumen-lateritic mixture. The researchers conducted comparative studies on the stabilization of soil with anionic bitumen and cement and also observed reduction in both the MDD and OMC of the lateritic-bitumen mixtures with resultant increase in the CBR. Physically, RAP also contains crushed gravel which can be used in mechanical stabilization. Due to high specific gravity of gravel compared to Laterite, the combination of the two will result to increase in MDD of the laterite gravel mixtures. Therefore, addition of RAP to a deficient lateritic soil can improve engineering properties of the deficient soil.

Gregory and Halsted (2007) worked on full depth reclamation (FDR) of RAP to form a new base. In their study, attempt was made to mix the aged asphalt surface with the in-situ existing base and possibly, sub-base material with small amount of cement which was compacted to form an excellent base course. It was observed that a cement content that will provide 7days unconfined compressive strength of between 2.1 to 2.8MPa is satisfactory for FDR applications. According to the authors, other stabilizing additives used for FDR are asphalt emulsion, cement, foamed asphalt and lime/fly ash.

Edeh *et al.* (2012) worked on stabilization of lateritic soil using RAP and 2% cement for application as flexible pavement materials, and noticed that CBR of the material increased from 17.9 to 55.0%, implying that the stabilized material can be used for subgrade and sub-base courses, based on Nigerian General Specification for Roads and Bridge Works (1972).

Attempt was also made byEdeh *et al*(2012) to stabilize clay with lime and RAP for use as pavement materials. Results of the study indicated that 90% Rap + 4% clay + 6% lime has CBR of 36.6%, while 90% RAP + 2% clay + 8% lime resulted in a CBR of 34.23%, both of which can be used as subgrade and sub-base courses for road structure based on Nigerian Standard.

A study aimed at increasing strength and reducing creep of RAP by adding high quality aggregate and or adding chemical stabilizer was carried out by Bleakley and Cosentino (2013). Limerock Bearing Ration (LBR) test was used by the authors to evaluate the bearing capacity of the mixture, while creep test was used to evaluate the creep properties. There was increase in LBR from 142 at 50% RAP/50% laterite to 284 at 25% RAP/75% laterite. Blends of 75% RAP did not reach unsoak LBR of 100%.

Ochepo (2014) stabilized lateritic soil using RAP and Sugar Cane Bagasse Ash (SCBA) for pavement construction. The lateritic soil stabilized, classified as A-7-6 and CL based on American Association of State Highway and Transportation Officers (AASHTO) and Unified Soil Classifications (USCS) respectively. MDD of the mixture increased from 1.77 to 1.79 $Mg/m^3$ for

60% soil/40% RAP, which further increased to 1.82 $Mg/m^3$ with addition of 4% SCBA. Increase in SCBA beyond this point (4% SCBA) reduced the MDD. Both the UCS and OMC increased with increase in SCBA. The author observed that the lateritic soil stabilized with 6 and 8% SCBA generated a CBR sufficient for the mixture to be used as subgrade and sub-base courses, while the mixture treated with 10% SCBA gave CBR that is sufficient for the mixture to be used as a base course material.

Mustapha *et al.* (2014) worked on possible stabilization of A-6 lateritic soil using RAP only. The A-6 soil was replaced with RAP at 0:100, 10:90 to 100:0, and observed that optimum mixture of 60:40 gave the highest MDD and was used as basis on which other tests were carried out. The MDD was observed to increase from 1.895$Mg/m^3$ at 100:0 to its maximum of 2.170 $Mg/m^3$ at 40:60 after which the values reduced to 2.017 $Mg/m^3$ at 0:100. The unconfined compressive strength was observed to minimally increase from 346 $kN/m^2$ for the natural soil to 384 $kN/m^2$ at 40:60 mixtures, while the CBR increased marginally from 45.1% for natural soil to 48.6% at 40:60 mixtures.

Mishra (2015) studied the use of RAP in flexible pavements in which typical values of unit weight, moisture content, bitumen content, compaction densities and CBR values were reported. The typical values were 1900-2250 $kg/m^3$ for density, 3-5% for natural moisture content, 5-6% for bitumen content, 1500-1950$kg/m^3$ for compacted unit weight and 20-25% for CBR at 100% RAP. The author used 50% RAP replacement for granular sub-base and concluded that 30% replacement of natural aggregate with RAP can successfully be used in base course strength improvement.

Suitability of RAP as sub-base using factorial experiments was studied by Kamel *et al.* (2016). The RAP ratios used were 0, 10, 50, 90 and 100%, mixed with subgrade soil, while the test criteria used to evaluate the strength were UCS and CBR. Extraction test on the RAP gave 5.09% bitumen. MDD was observed to increase from 2.155 g/cm$^3$ at 0% RAP-100% soil to 2.212 g/cm$^3$at 100% RAP-0% soil. OMC on the other hand, reduced from 5.8% at 0% RAP-100% soil to 4.6% at 100% RAP-0% soil. Increase in CBR values of 43% at 0% RAP-100% soil to 59% at 50% RAP-50% soil after which the value reduced to 22% at 100% RAP-0% soil. UCS values were also observed to decrease from 321 kN/m$^2$ at 0% RAP-100% soil to 55 kN/m$^2$ at 100% RAP-0% soil.

The use of geopolymer materials to stabilize RAP for base course was carried out by Avirneni *et al.* (2016). The technology involves alkaline treatment of pozzolanic materials to form highly alkaline medium (pH>12). The author observed that fly-ash stabilization alone did not impact sufficient strength on the RAP-VA mixtures. However, activation of fly-ash with 2 and 4% sodium hydroxide was observed to enhance strength gain of the mixture to UCS greater than design strength of 4.5 MPa. Durability was also observed to perform satisfactorily. Horpibulsuk *et al.* (2017) also used the same approach for sustainable stabilization of RAP for use as sub-base and concluded that 7 days UCS of the compacted RAP-FA blend, at OMC met the strength requirement for base course.

Alhaji and Alhassan (2018) worked on microstructure and strength of RAP stabilized clay for road structure. The clay studied classified as clay of high plasticity (CH) based on Unified Soil Classification system, while the bitumen content of the RAP was 5.99%. The study showed MDD increasing from 1.890 Mg/m$^3$ at 0% RAP-100% clay to maximum of 2.036 Mg/m$^3$ at 30% RAP-70% clay, after which the values reduced to 1.925 Mg/m$^3$ at 100% RAP-0% clay. The OMC reduced from 13.7% at 0% RAP-100% clay to 8.0% at 100% RAP-0% clay. The CBR was also observed to increase from 11% at 0% RAP-100% clay to 35% at 30% RAP-70% clay, after which the values reduced to 5% at 100% RAP- 0% clay.From the foregoing, it can be observed that RAP has been extensively used in stabilization of base and sub-base materials, but little has been researched on the use of surface dressed material (pavement).

Surface dressed Pavement comprises of a thin film of binder, generally bitumen or tar, which is sprayed onto the road surface and then covered with a layer of stone chippings(Road Note 39, 1992). The thin film of binder acts as a waterproofing seal preventing entry of surface water into the road base. The stone chippings protect this film of binder from damage by vehicle tyres, and form a durable, skid-resistant and dust-free wearing surface. In some circumstances the process may be repeated to provide double or triple layers of chippings (Road Note 39, 1992). Not much has been carried out on soil stabilization using Reclaimed Surface-dressed Pavement (RSP). This research seeks to investigate the performance of lateritic soil base mixed with reclaimed surface dressed material, cement and calcium carbide residue. The lateritic soil base-reclaimed surface-dressed pavement composite material was obtained through full depth reclamation method, giving rise to Full Depth Reclaimed Surface-dressed Pavement (FDRSP), while in-situ strength evaluation was achieved using the Dynamic Cone Penetration (DCP) Test.

### 2.5 Dynamic Cone Penetration (DCP) Test for Pavement Strength Evaluation

The use of DCP test to evaluate strength of pavement structure through penetration index has been studied for few decades. This method of evaluating strength of in-situ pavement structure through penetration index, has been observed to give accurate and reproducible results, when compared to laboratory results (Gabr *et al.,* 2000; Alhaji *et al.,* 2019)

Siekmeier *et al.* (1999) compared DCP with other tests during subgrade and granular base characterization. The author observed that the existing empirical method of quality assurance testing of subgrade and base materials which are based on soil classification, grading, moisture control, lift thickness limits and compaction testing, does not work well for mechanistic empirical methods of design. The quality assurance testing for mechanistic empirical design would include in-situ shear strength test using DCP test. The DCP index for each drop was used to calculate the CBR using the expression:

$$CBR(\%) = \frac{292}{DPI^{1.12}} \tag{2.1}$$

For CBR > 10% and

$$CBR(\%) = \frac{1}{(0.017019 * DPI)^2} \tag{2.2}$$

For CBR < 10%

Where:

DPI = Depth of Penetration Index

The resulting CBR were then used to evaluate the modulus of elasticity from the expression

$$E(MPa) = 17.6 * CBR^{0.64} \tag{2.3}$$

Patel and Patel (2012) correlated test results of Plate Bearing Test (PBT), UCS and CBR with DCP on various soils under soaked condition. The authors created in-situ conditions in the laboratory by using bigger testing mould and carried out relevant tests. Empirical correlations were established among test results using linear regression procedure. The formulations were validated using other set of data. Nguyen and Mohajerani (2012) worked on the effect of vertical confinement from CBR mould on the DCP index. It was observed that effect of vertical confinement is very significant, especially with hammer mass greater than 4.6kg, but is not significant if the hammer mass is less than 2kg. The author therefore, developed a new light weight DCP with a hammer mass of 2.25kg which could be used both in the laboratory and on the field.

Comparative study of subgrade soil strength estimation models developed based on California CBR, DCP and Falling Weight Deflectometer (FWD) test results was carried out by Kumar*et al.* (2015). Regression models were developed to establish relationship between CBR and DCP as well as between modulus of elasticity and DCP. Similarly, Singh *et al.* (2016) evaluated subgrade soil using in-situ tests. DCP and FDW were used to determine the strength of compacted surfaces at different locations. The results obtained were used to correlate DCP with other parameters.

Study on modelling of light dynamic cone penetration test – Panda 3(R) in granular materials using 3D discrete element method was carried out by Tran *et al.* (2017). Light dynamic penetration test – Panda 3(R) provides dynamic load-penetration curves for each blow. This curve is influenced by mechanical and physical properties of the granular medium. It was possible to use force and acceleration measured in the top part of the rod, separate the incident and reflected waves and calculates the tips load-penetration curve.

# CHAPTER THREE

## 3.0 MATERIALS AND METHODS

### 3.1 Preamble

This study was to determine the practicability of using Full Depth Reclaimed Surface-dressed Pavement (FDRSP), treated with cement and calcium carbide residue for road base. The full depth reclaimed surfaced dressed pavement material was gotten from a section of Morris Fertilizer Road in Minna Metropolis (Plate I). The samples were collected and laboratory tests were conducted on the FDRSP material mixed with cement at 2 and 4% and up to 6% calcium carbide residue at 2% variations. The percentage combination of the FDRSP material treated with cement and calcium carbide residue, tested at the laboratory, which gave a value of CBR for heavy traffic road was then replicated on the field.

### 3.2 Materials

The materials used for this project were Full Depth Reclaimed Surface-dressed Pavement (FDRSP) material, cement and Calcium Carbide Residue (CCR).

#### 3.2.1 Calcium carbide residue

The Calcium Carbide Residue (CCR) was obtained from welding shops in Minna Metropolis. It was air dried, ground sieved through sieve No. 200 (75µm sieve) before usage.

#### 3.2.2 Reclaimed surface dressed pavement/laterite base material

The Full Depth Reclaimed Surfaced-dressed Pavement (FDRSP) material was gotten from a section of Morris Fertilizer Road in Minna Metropolis (Plate I)

**Plate I:** Section of road where FDRSP material was obtained and field work carried out

### 3.2.3 Cement

The ordinary Portland cement used in the study was gotten from building materials market in Minna Metropolis and conformed to specification and standard before usage.

## 3.3 Methods

### 3.3.1 Sample preparation

The Full Depth Reclaimed Surfaced-dressed Pavement (FDRSP) material was air dried and samples were taken and mixed with 2 and 4% cement and calcium carbide residue in varying proportions of 0, 2, 4 and 6% before tests. The percentage combination of calcium carbide residue and cement added to FDRSP material tested in the laboratory that gave a value of CBR for heavy traffic roads was then replicated on the field.

### 3.3.2 Laboratory tests

The following standard laboratory and field tests were conducted on the FDRSP material, cement and calcium carbide residue in accordance with BS: 1377 (1990) and BS: 1924 (1990).

Natural Moisture Content

Sieve analysis

Specific Gravity Test

Atterberg limits test

Compaction test

California bearing ratio (CBR)

Compaction and CBR tests were carried out on both the natural and stabilized materials.

#### 3.3.2.1 *Determination of natural moisture content*

The procedure adopted in determining the moisture content of FDRSP material was as highlighted in BS 1377 (1990). Two empty cans used were cleaned, labeled and weighed. Portion of the FDRSP was placed in the empty cans and weighed before they were placed in the oven at temperature of between 105 to 110$^{\circ}$C for 24 hours. The dried samples were then weighed to determine their moisture content. The water content is the ratio of weight of water loss to that of dry samples, expressed in percentage.

#### 3.3.2.2 *Sieve analysis*

300g of the lateritic soil base mixed with reclaimed surface dress material was washed and oven dried. The sample was sieved through BS sieves sizes of 5.00mm, 3.35mm, 2.00mm, 1.18mm, 850µm, 600µm, 425µm, 300µm, 150µm, 75µm and base pan. Another sample consisting of 300g of the existing lateritic soil base only was washed and passed through the same BS sieve sizes and yet another sample consisting of 1000g of the reclaimed surface dress pavement material only unwashed was passed through the same BS sieve sizes. The percentage mass retained was then determined for each sieve size and used in the computations for plotting the particle size distribution graph.

$$\text{Percentage Retained} = \frac{\text{(mass of the soil sample on each sieve)}}{\text{Total mass of soil sample}} \times 100 \qquad (3.1)$$

### 3.3.2.3 *Specific gravity*

This test was carried out on the calcium carbide residue adopting the procedure highlighted in BS 1377 (1990). 20g of air dry calcium carbide residue sample was put into the pyconometer, weighed and then filled with water to the mark and weighed again. The pyconometer was emptied of its content and then filled with water to the mark and weighed. The weight of displaced water was determined by comparing the weight of the sample and water in the pyconometer with the weight of pyconometer containing water only. The specific gravity was then calculated by dividing the weight of the dry sample by the weight of the displaced water.

$$G_s = \frac{\text{Weight of soil}}{\text{Equal Volume of water}} = \frac{W_2 - W_1}{(W_4 - W_1) - (W_3 - W_2)} \qquad (3.2)$$

### 3.3.2.4 *Atterberg limits*

**Liquid limit**

This test was carried out adopting the procedure highlighted in BS 1377 (1990). 200g of air dried sample that passes sieve 425µm was weighed and placed on a glass plate; the sample was then mixed with distilled water into a paste. A metal cup of approximately 55mm in diameter and 40mm deep was filled with the paste and the surface struck off and leveled, the cone was placed at the center of the paste, then released so that it penetrated into the soil and the amount of penetration over a time period of 5 seconds was taken and recorded from dial gauge.

The procedure was repeated for about five times with increasing water content. Liquid limit was then obtained by plotting graph of penetration (at ordinate) against moisture content (at abscissa), using the best straight line drawn through the experimental points, the liquid limit was taken to be the moisture content corresponding to a cone penetration of 20mm (expressed as a whole number)

**Plastic limit**

20g of soil sample passing sieve 425μm was used for the plastic limit. The sample was thoroughly mixed with water and rolled between the fingers and palm on a glass plate to form a thread of 3mm in diameter and crumble. The moisture content of the crumbled soil thread was determined as the plastic limit of the soil sample. The plasticity index was determined based on the difference between the liquid and plastic limit obtained. This can be represented mathematically as:

$$PI = LL - PL \tag{3.3}$$

Where

PI = Plasticity Index (%)

LL = Liquid Limit (%)

PL = Plastic Limit (%)

**3.3.2.5 *Compaction***

The compaction test was carried out in accordance with BS 1377 (1990), using BSH (British Standard Heavy) compaction effort. In the British Standard Heavy (BSH), soil sample was compacted in five layers with each receiving twenty seven (27) blows of 4.5kg hammer dropping from a height of 450mm. The test was carried out on full depth reclaimed surface dress pavement material mixed with cement and calcium carbide residue at 2 and 4% cement and 0, 2, 4 and 6% calcium carbide residue. The values obtained from the compaction tests were used to plot the graph of dry density against moisture content to establish the compaction curve from which Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) was obtained. This test was also carried out on the lateritic base material and mixture of the reclaimed surface dressed material and the lateritic base on the Field. This mixture is called Full Depth Reclaimed Surface–dress Pavement (FDRSP).

#### 3.3.2.6 *California bearing ratio*

This is the basic test used to measure the strength (bearing capacity) of soil for pavement construction. It involves penetration of a molded soil sample with a cylindrical plunger at a constant rate 1mm/min. The force corresponding to penetration of 2.5 and 5.0mm are then computed and compared to the standard force attained by the California materials (Reported as percentage). In accordance with BS 1377 (1990), about 6000g of air dried soil sample was weighed using electronic weighing balance. The soil sample was placed on a metal tray and the lumps crushed to finer particles using a mallet. Optimum Moisture Content obtained from the compaction test was used in adding water to the soil. The soil was thoroughly mixed with both hands and then divided into five portions. The first portion of the mixed soil was poured into the CBR mold and uniformly rammed 62 times using 4.5kg rammer then the second portion was poured and rammed 62 times too, this process continued up to the fifth layer. The collar of the CBR mold was removed and the soil levelled to the brim of the mold. A dead load used as an assumed thickness of a flexible pavement was placed on the side of the soil. The plunger was adjusted to rest on the soil and the bolts at the bottom and top were all tightened. The lower dial gauge that measures depth of penetration of the plunger was set in contact with the single load placed on the mold, and the load dial gauge was set to zero.

The CBR machine was switched on (electrically operated). The penetration was conducted between the range of 0.00 to 7mm at an interval of 0.25mm and the corresponding readings were taken from the load dial gauge. The procedure was repeated on the bottom side of the sample. The values obtained from the top and bottom dial gauges at 2.5 and 5.0mm penetration were converted to CBR values and the highest value was taken as the CBR value for the soil sample. The test was carried out on the lateritic soil base, FDRSP, and FDRSP mixed with 2and 4% cement and with varying percentages of CCR at it variation.

Due to the nature of the traffic (Plate II) on the chosen road, CBR value of the FDRSP-cement-CCR that met the required CBR value for Heavy Traffic roads (average of 150) was selected for the field investigation.

### 3.4 Field Tests

The field study was carried out on a section of Morris Fertilizer road in Minna Metropolis (Plate II). The road has a carriage width of 15.0m and a length of 900m. Half of the carriage width corresponding to 7.5m and a section of 15m was used for the field study. The 15.0m length was divided into three sections (A, B and C) of 5.0m each. The first section was prepared with FDRSP material only, while the second section was prepared with FDRSP mixed with cement, and the third section of the road consisted of FDRSP mixed with cement and calcium carbide residue.

The test sections of the road was mechanically scarified to depth of 30cm using ripper (Plate III), with the resulting lumps of the FDRSP properly pulverized (Plate IV), in preparation for addition of cement and CCR (Plate V). The 15.0m length of the test section was divided into three sections of 5.0m each (Figure 3.1). To effectively study effect of these additives on the reclaimed pavement, the first test section consisted of the reclaimed pavement (FDRSP) only, the second section consisted of the reclaimed pavement (FDRSP) + 2% cement, while the third section consisted of reclaimed pavement (FDRSP) + 2% cement + 4% CCR. Figure 3.1 shows a sketch of test sections of the road and the test points.

**Plate II**: Nature of traffic on the road

The test sections of the road were cleared of organic and other impurities, before the mechanical scarification and crumbling of the lumps were carried out. The test sections were then demarcated using pegs, into three sections of 5m length each. On the second and third sections, the additives were added and properly mixed (Plate III), making sure that each of the sections consisted of only the additives intended. After adding and properly mixing the FDRSP-cement and FDRSP-cement-CCR mixtures at the respective sections, compaction was carried out using sheep-foot (Plate VI) and smooth drum vibrating rollers (Plate VII). During the compaction, in-situ density determination using sand replacement method (Plate VIII) was carried out intermittently to determine the maximum in-situ density, which eventually became constant with further compaction. The pegs and lines used in demarcating the sections were left standing throughout the test. Average of three in-situ densities was performed after 1, 7, 28, 60 and 90 days of compaction. Dynamic Cone Penetration (DCP) test (Plate IX) was also conducted on the three sections, after 1, 7, 14, 28, 60 and 90 days of compaction, as shown on Figure 3.1. Data from the

DCP test were used to compute CBR of the road base. The field CBR was evaluated using the DCP test results on the compacted surfaces with the aid of an empirical relationship developed by TRL (2014).

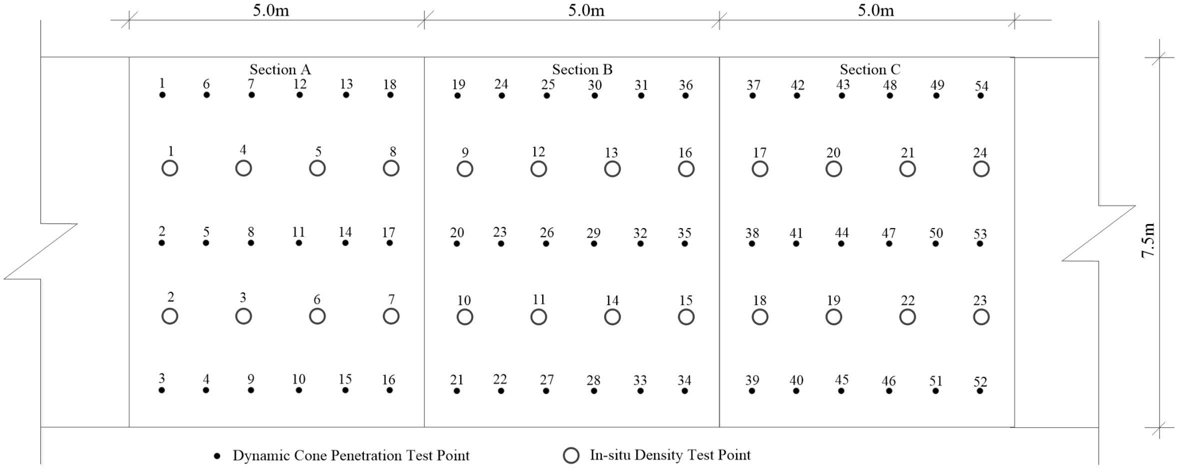


**Figure 3.1:** Schematic diagram of the test section of the road showing the test points

**Plate IIIa:** Scarification of the test sections using ripper

**Plate IIIb:** Scarification of the test sections using ripper

**Plate IV:** Crumbling/pulverizing the lumps using sheep-foot roller

**Plate V:** Manual addition of cement and calcium carbide residue with FDRSP

**Plate VI:** Compaction of the test sections using sheep-foot roller


**Plate VII:**Field compaction using a smooth wheel roller

**Plate VIIIa:** In-situ Density test

**Plate IX:** Dynamic Cone Penetration Test (DCPT) being carried out on all sections of the road at Day 1

**Plate VIIIb:** In-situ Density test

**Plate IX:** Dynamic Cone Penetration Test (DCPT) being carried out on all sections of the road at Day 1

**Plate X**: View of sections of the road under the compaction

# CHAPTER FOUR

# 4.0 DISCUSSION OF RESULTS

## 4.1 Preamble

This chapter presents and discusses results of the study. The results include laboratory and Field results. The laboratory results include Index properties, compaction and CBR tests while the field results include density and CBR. CBR of 75 % was recorded for the existing base material and 111% for the resulting FDRSP.

## 4.2 Laboratory Results

### 4.2.1 Index properties of the original lateritic soil base and the resulting FDRSP

Preliminary tests were conducted in the laboratory in order to determine the index and strength properties of the FDRSP. The results of the index properties of the original base material and the resulting FDRSP material are shown in Table 4.1. From the results, the existing base soil is classified as SC and A-2-5, upon mixture with the surface dress material the resulting FDRSP classified as GC and A-2-4 according to USCS and AASHTO respectively.

**Table 4.1: Geotechnical properties of the original base material and the resulting FDRSP**

| Property | Existing base | FDRSP |
|---|---|---|
| Fraction passing BS No 200 sieve (%) | 26.6 | 14.7 |
| Liquid limit (%) | 57.84 | 40.1 |
| Plastic limit (%) | 48.58 | 33.5 |
| Plasticity index (%) | 9.26 | 6.5 |
| USCS | SC | GC |
| AASHTO classification | A-2-5 | A-2-4 |
| MDD (g/cm$^3$) | 1.92 | 2.20 |
| OMC (%) | 17.0 | 10.0 |
| CBR (%) | 75 | 111 |

From these results, it was observed that the original base material was modified from SC and A-2-5 to GC and A-2-4 according to USCS and AASHTO respectively. This indicates that full depth reclamation of the surface dressed material resulted to mechanically modifying the base soil from clayey sand (SC) to clayey gravel (GC), which will certainly perform better as base material.

**4.2.2Geotechnical properties of the original base soil and the resulting FDRSP material**

**4.2.3 Variation of compaction characteristics of the FDRSP material with dosage of the additives**

FromTable 4.1, the original base course material of the road classified under A-2-5 and SC according to (AASHTO) and (USCS) respectively, while the resulting FDRSP classified under A-2-4 and GC, according to AASHTO and USCS respectively. This indicates that the surface-dressed material improved both grading and consistency of the original base course material, by changing it from clayey sand (SC) to clayey gravel (GC) and reducing the PI from 9.26 to 6.49%. This improvement is evident in the MDD and OMC of the resulting FDRSP as seen on Table 4.2. The existing base soil has (MDD of 1.92 $Mg/m^3$, while the resulting FDRSP has Maximum Dry Density (MDD) of 2.20 $Mg/m^3$,Optimum Moisture Content (OMC) for the existing base material was 17.0%, while for the resulting FDRSP, it was 10%.

**Table 4.2: Variation of compaction characteristics with changes in dosage of the additives**

| Cement (%) | **Compaction Characteristics** | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 0 % CCR | | 2 % CCR | | 4 % CCR | | 6 % CCR | |
| | MDD | OMC | MDD | OMC | MDD | OMC | MDD | OMC |
| 2 | 2.213 | 8.40 | 2.132 | 9.68 | 2.12 | 10.2 | 2.110 | 10.70 |
| 4 | 2.224 | 9.45 | 2.141 | 9.60 | 2.13 | 10.4 | 2.130 | 10.81 |
| 6 | 2.228 | 9.90 | 2.183 | 9.81 | 2.14 | 10.60 | 2.139 | 11.68 |

Variation of compaction characteristics of the FDRSP with varied dosage of the additives is presented on Table 4.2. The result indicates gradual increase in MDD of the FDRSP material with increase in cement content. This is expected, as cement with higher specific gravity and fineness fills the voids in the FDRSP, this results to a more compact and dense material. OMC of the FDRSP initially decreased on first dosage of cement, but with subsequent increase, the OMC gradually increased. The initial decrease in OMC with first dosage of cement is as a result of the consistency of the fines in the FDRSP which is lowered with introduction of cement. The subsequent increase in OMC with increase in cement content is attributed to hydration reaction of the cement, which requires water to proceed. At constant cement content, the MDD of the mixtures is observed to decrease, while the OMC increased. The decrease in MDD with increase in CCR is a result of the lower specific gravity being contributed to the mixture by the CCR. This observed trend in variation of MDD and OMC with increase in CCR is similar to those reported by Latifi *et al*. (2018) and Du *et al*. (2016).

**Table 4.3: Variation of laboratory CBR with changes in dosage of the additives**

| | **CBR (%)** | | | |
|---|---|---|---|---|
| Cement (%) | 0% CCR | 2% CCR | 4% CCR | 6% CCR |
| 2 | 123 | 136 | 159 | 107 |
| 4 | 165 | 192 | 270 | 175 |
| 6 | 231 | 296 | 320 | 281 |

### 4.2.4 Effect of additive dosages on CBR of the FDRSP

Variation of laboratory CBR of the FDRSP with changes in dosage of the additives is presented on Table 4.3. From the table, it is observed that CBR value of the FDRSP increased with increase in cement content. This is expected, as more cement means more binding material in the mixture. On the other hand, at constant percentage of cement, CBR of the mixture initially increased to their maximum values at 4% CCR, after which the value decreased. Based on the value of CBR for heavy traffic roads (150%), the optimal percentage combination for stabilization of the FDRSP with cement and CCR for use as base material for heavy traffic roads will be 2% cement and 4%CCR. Therefore, the performance of this mixture was studied on the field.

## 4.3 Field Results

### 4.3.1 Field densities

Sand replacement method of in-situ density determination was used in accordance with BS 1377 (1990).During compaction, the test was routinely conducted on the three sections of the road until three consecutive trials gave very close results. This was repeated after 1, 7, 14, 28, 60, and 90 days. Summary of the results are presented on (Table 4.4). From the table, it is observed that the dry densities of the three sections changes throughout the 90 days of the study. The rate of increase in the densities was more pronounced in section B, section A has recorded the least rate of increase in the densities. In all the sections, more than 95% of the laboratory densities were achieved after 14 days, while more than 100% was achieved after 28 days. Alhaji *et al*. (2019) recorded 99.8 and 98.8% for lateritic soil/RAP/cement and lateritic soil/RAP mixtures respectively, after 60 days. The relatively early attainment of laboratory density is attributed to the nature of traffic the road is exposed to.

**Table 4.4**: Summary of the field densities for the three sections of the road

| Test | **Density (Mg/m$^3$)** | | | | | |
|---|---|---|---|---|---|---|
| Section | 1 day | 7 days | 14 days | 28 days | 60 days | 90 days |
| A | 2.180 | 2.194 | 2.198 | 2.210 | 2.215 | 2.223 |
| B | 2.191 | 2.211 | 2.234 | 2.258 | 2.261 | 2.262 |
| C | 2.080 | 2.108 | 2.118 | 2.120 | 2.123 | 2.130 |

### 4.3.2 Field CBR

The field CBR of the compacted surfaces was determined using Dynamic Cone Penetration (DCP) test data with the help of the empirical relation, developed by Transport Research Laboratory-TRL (2014).

$$Log\ (CBR) = 2.48 - 1.057(PI) \tag{4.1}$$

Where PI, is the penetration index

Variation of CBR with number of days is presented in Figure 4.1.

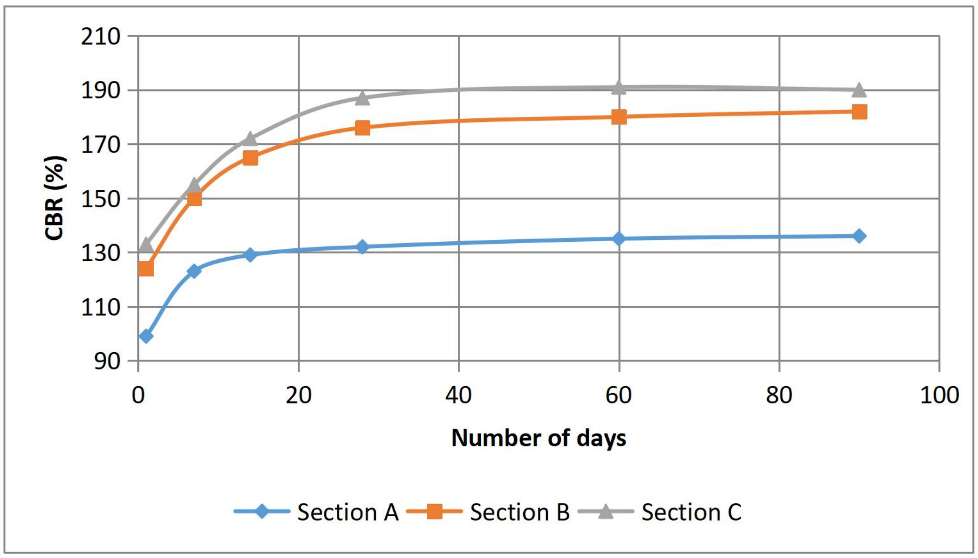


**Figure 4.1:** Variation of In-Situ CBR values with number of days

From figure 4.1, it is observed that the CBR of section A is generally less than those of sections B and C. The relatively higher strength (CBR), recorded from sections B and C in comparison to A, is as a result of the cementation, resulting from reactions of the additives (cement and CCR). At 7 days after compaction, CBR values of sections A and B were generally more than 100% of the laboratory CBR, while that of section C was 98%. This tremendous increase in CBR values of the test sections is as a result of the nature of traffic the road is exposed to. This road, being an access road to Morris Fertilizer Company, Minna, is plied by heavy and articulate vehicles, transporting raw materials and products, in and out of the company. After 14 days, only marginal increase in CBR was noticed in section A, which could be attributed to the marginal increase in density of the section. Sections B and C recorded relatively noticeable increase in CBR up to 28 days, after which the increase became marginal to 90 days. Similar trend in CBR was reported by Alhaji *etal.* (2019) for lateritic soil treated with Reclaimed Asphalt Pavement (RSP).

# CHAPTER FIVE

## 5.0 CONCLUSION AND RECOMMENDATIONS

### 5.1 Conclusion

From the study carried out to the following conclusions were drawn:

The initial lateritic soil that constituted the base course of the road classified under SC and A-2-5. When the soil was mixed with RSP, the resulting material (FDRSP) classified under GC and A-2-4 according to USCS and AASHTO soil classification systems respectively.

The laboratory test proved 2% cement and 4% CCR, added to FDRSP satisfied the 150% CBR required for heavy traffic roads.

Field CBR results of compacted FDRSP/2% cement/4% CCR used in sections C attained the laboratory CBR after 14 days of exposure to traffic.

The field CBR results of the FDRSP used in section 'A' attained laboratory value after 7 days, after which the value became higher than the laboratory value.

From laboratory and field density results for the FDRSP, FDRSP/cement mixture and FDRSP/cement/CCR mixture, it was observed that more than 95% density can be achieved after 14 days exposure to traffic load.

### 5.2 Recommendations

From the study, 2% cement /4% CCR can be used to improve the strength of SC (A-2-5) lateritic soil base mixed with surface dressed pavement for heavy traffic roads.

## 5.3 Contribution to knowledge

The study established the treatment of full depth reclaimed surface-dressed pavement (FDRSP) with 2% ordinary Portland cement (OPC)/ 4% calcium carbide residue (CCR) increased the California bearing ratio (CBR) value to 150% required for adequate stabilization of base material in heavy trafficked roads. It also established FDRSP and CCR as waste materials that can be beneficially re-used in road construction.

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# APPENDICES

## APPENDIX A

**APPENDIX A: Natural Moisture Content of Existing Base and FDRSP**

**Table A1: Natural Moisture Content of Existing Base**

| **Description** | **Values** | |
|---|---|---|
| Trial | 1 | 2 |
| Weight of Can, (g) | 20.3 | 23.1 |
| Weight of Can+Wet Soil, (g) | 94.4 | 101 |
| Weight of Can+Dry Soil, (g) | 88.1 | 94.4 |
| Weight of Dry Soil, (g) | 67.8 | 71.3 |
| Weighy of Water, (g) | 6.3 | 6.6 |
| Moisture Content, (%) | 9.29 | 9.26 |
| Average Moisture Content, (%) | 9.27 | |

**Table A2: Natural Moisture Content of FDRSP**

| **Description** | | **Values** | |
|---|---|---|---|
| Trial | 1 | 2 | 3 |
| Weight of Can, (g) | 23.1 | 25.2 | 26.2 |
| Weight of Can+Wet Soil, (g) | 74.2 | 78.7 | 83.8 |
| Weight of Can+Dry Soil, (g) | 69.7 | 74.3 | 78.3 |
| Weight of Dry Soil, (g) | 46.6 | 49.1 | 52.1 |
| Weighy of Water, (g) | 4.5 | 4.4 | 5.5 |
| Moisture Content, (%) | 9.66 | 8.96 | 10.56 |
| Average Moisture Content, (%) | | 9.72 | |

**APPENDIX B**

**APPENDIX B: Grain Size Analysis of Existing Base and FDRSP**

**Table B1: Grain Size Analysis of Existing Base**

| Sieve size (mm) | Mass of sieve (g) | Mass of sieve+Soil (g) | Mass retained (g) | Percentage Retained | Percentage Passing |
|---|---|---|---|---|---|
| 5.000 | 476.10 | 542.90 | 66.80 | 22.27 | 77.73 |
| 3.350 | 467.90 | 499.80 | 31.90 | 10.63 | 67.10 |
| 2.360 | 433.70 | 452.90 | 19.20 | 6.40 | 60.70 |
| 2.000 | 416.80 | 423.80 | 7.00 | 2.33 | 58.37 |
| 1.180 | 384.80 | 399.00 | 14.20 | 4.73 | 53.63 |
| 0.850 | 352.00 | 360.70 | 8.70 | 2.90 | 50.73 |
| 0.600 | 467.80 | 480.70 | 12.90 | 4.30 | 46.43 |
| 0.425 | 435.00 | 448.60 | 13.60 | 4.53 | 41.90 |
| 0.300 | 382.50 | 390.80 | 8.30 | 2.77 | 39.13 |
| 0.150 | 420.50 | 445.50 | 25.00 | 8.33 | 30.80 |
| 0.075 | 394.60 | 407.20 | 12.60 | 4.20 | 26.60 |
| Pan | 297.50 | 298.90 | 1.40 | 0.47 | 26.13 |

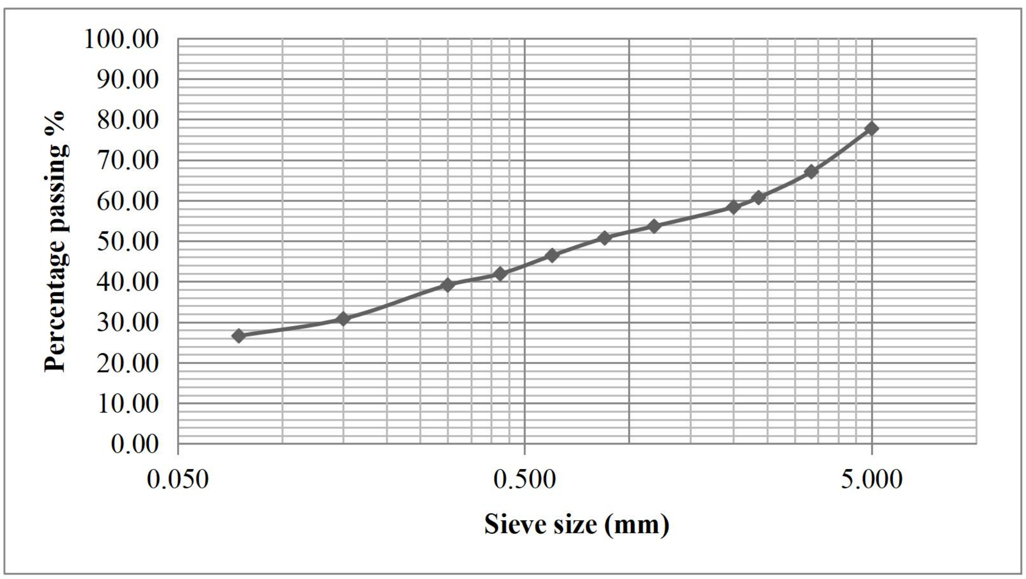


**Figure B1: Particle size distribution curve for existing base**

**Table B2: Grain Size Analysis of FDRSP**

| Sieve size | Mass of sieve | Mass of sieve+Soil | Mass retained | Percentage | Percentage |
|---|---|---|---|---|---|

| (mm) | (g) | (g) | (g) | Retained | Passing |
|---|---|---|---|---|---|
| 5.000 | 476.10 | 578.40 | 102.30 | 34.10 | 65.90 |
| 3.350 | 467.90 | 488.00 | 20.10 | 6.70 | 59.20 |
| 2.360 | 433.70 | 441.90 | 8.20 | 2.73 | 56.47 |
| 2.000 | 416.80 | 419.20 | 2.40 | 0.80 | 55.67 |
| 1.180 | 384.80 | 398.92 | 14.12 | 4.71 | 50.96 |
| 0.850 | 352.00 | 354.91 | 2.91 | 0.97 | 49.99 |
| 0.600 | 467.80 | 486.58 | 18.78 | 6.26 | 43.73 |
| 0.425 | 435.00 | 454.79 | 19.79 | 6.60 | 37.13 |
| 0.300 | 382.50 | 388.31 | 5.81 | 1.94 | 35.20 |
| 0.150 | 420.50 | 473.70 | 53.20 | 17.73 | 17.46 |
| 0.075 | 394.60 | 402.90 | 8.30 | 2.77 | 14.70 |
| Pan | 297.50 | 299.91 | 2.41 | 0.80 | 13.89 |

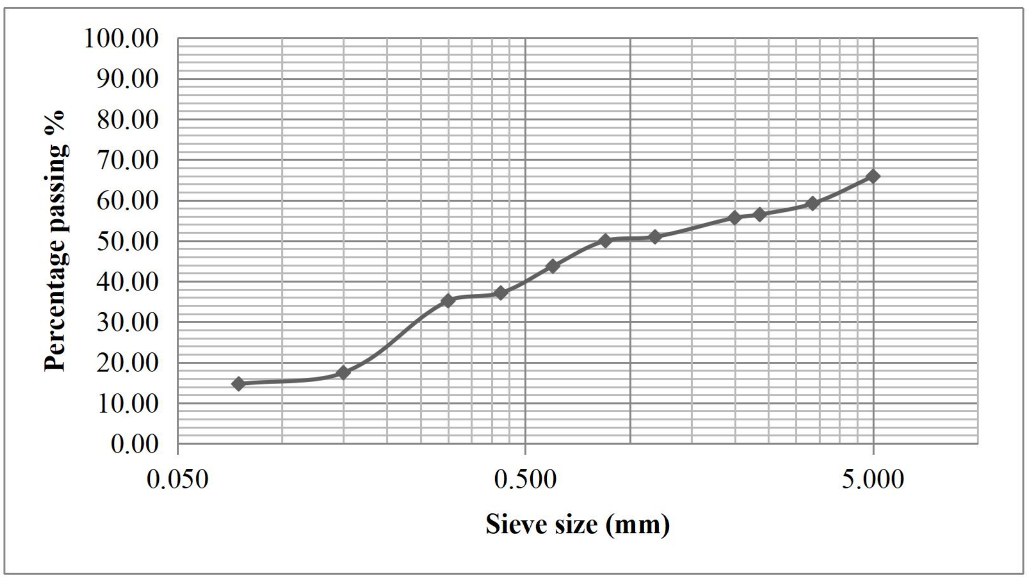


**Figure B2: Particle size distribution curve for FDRSP**

**Table B3: Summary of the field densities for the three sections of the road**

| Test Section | Density (Mg/m$^3$) After | | | | | |
|---|---|---|---|---|---|---|
| | 1 day | 7 days | 14 days | 28 days | 60 days | 90 days |
| A | 2,180 | 2,194 | 2.198 | 2,210 | 2,215 | 2,223 |
| B | 2,191 | 2,211 | 2.234 | 2,258 | 2,261 | 2,262 |
| C | 2,080 | 2.108 | 2.118 | 2,120 | 2,123 | 2,130 |

# APPENDIX C

**APPENDIX C: Atterberg limits result of test soil (Cone Penetrometer Method)**

**Table C1: Liquid and Plastic Limit Determination for FDRSP**

| | LIQUID LIMIT | | | | | | PLASTIC LIMIT | |
|---|---|---|---|---|---|---|---|---|
| Can Number | 1 | 2 | 3 | 4 | 5 | 6 | 1 | 2 |
| Penetration | 5.00 | 9.00 | 10.20 | 14.20 | 18.40 | 21.70 | | |
| Can Weight | 27.90 | 26.00 | 22.80 | 20.40 | 24.40 | 23.40 | 26.1 | 24.1 |
| Weight of can +wet soil | 34.30 | 32.80 | 30.00 | 27.10 | 31.20 | 35.20 | 30.7 | 30.2 |
| Weight of can + Dry soil | 32.50 | 30.70 | 27.60 | 24.80 | 28.80 | 30.80 | 29.2 | 28.2 |
| Weight of Moisture | 1.80 | 2.10 | 2.40 | 2.30 | 2.40 | 4.40 | 1.5 | 2 |
| Weight of Dry Soil | 4.60 | 4.70 | 4.80 | 4.40 | 4.40 | 7.40 | 3.1 | 4.1 |
| Moisture Content | 39.13 | 44.68 | 50.00 | 52.27 | 54.55 | 59.46 | 48.39 | 48.78 |
| **Liquid Limit:** | **57.84%** | | **Average Plastic Limit:** | | | | **48.58%** | |

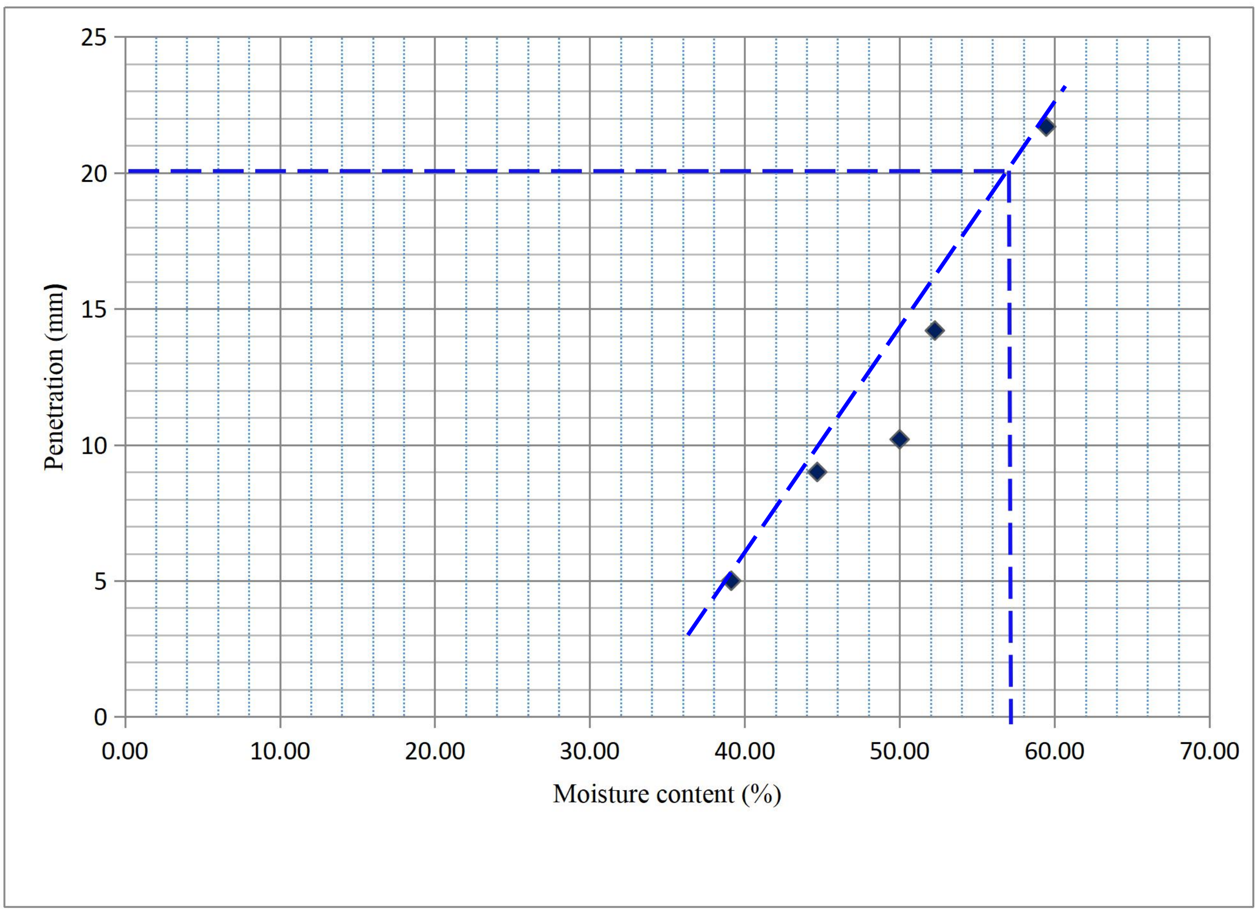


**Figure C1**: Liquid limit determination for FDRSP

**APPENDIX D**

## APPENDIX D: Compaction Test Results

**Table D1: Compaction Test for 0% Cement with 0% CCR**

| Trial | 1 | | 2 | | 3 | | 4 | | 5 | |
|---|---|---|---|---|---|---|---|---|---|---|
| Weight of Mould (g) | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | |
| Weight of Mould + Wet Sample (g) | 6500 | | 6677 | | 6753 | | 6628 | | 6580 | |
| Weight of wet Soil (g) | 2170 | | 2347 | | 2423 | | 2298 | | 2250 | |
| Wet Density (g/cm3) | 2.30 | | 2.49 | | 2.57 | | 2.43 | | 2.38 | |
| Can Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| Weight of Can (g) | 18.40 | 19.90 | 22.40 | 22.20 | 22.00 | 21.90 | 19.10 | 22.00 | 24.30 | 19.70 |
| Weight of Can + Wet Soil (g) | 33.80 | 36.30 | 35.90 | 35.10 | 35.30 | 37.80 | 33.70 | 37.50 | 44.80 | 41.10 |
| Weight of Can + Dry Soil (g) | 33.00 | 35.30 | 34.90 | 34.10 | 33.90 | 36.40 | 32.20 | 35.70 | 42.90 | 38.60 |
| Weight of Water (g) | 0.8 | 1 | 1 | 1 | 1.4 | 1.4 | 1.5 | 1.8 | 1.9 | 2.5 |
| Weight of Dry Soil (g) | 14.60 | 15.40 | 12.50 | 11.90 | 11.90 | 14.50 | 13.10 | 13.70 | 18.60 | 18.90 |
| Moisture Content (g) | 5.48 | 6.49 | 8.00 | 8.40 | 11.76 | 9.66 | 11.45 | 13.14 | 10.22 | 13.23 |
| Average Moisture Content (g) | 5.99 | | 8.20 | | 10.71 | | 12.29 | | 11.72 | |
| Dry Density (g/cm3) | 2.1689 | | 2.2978 | | 2.3184 | | 2.1678 | | 2.1334 | |

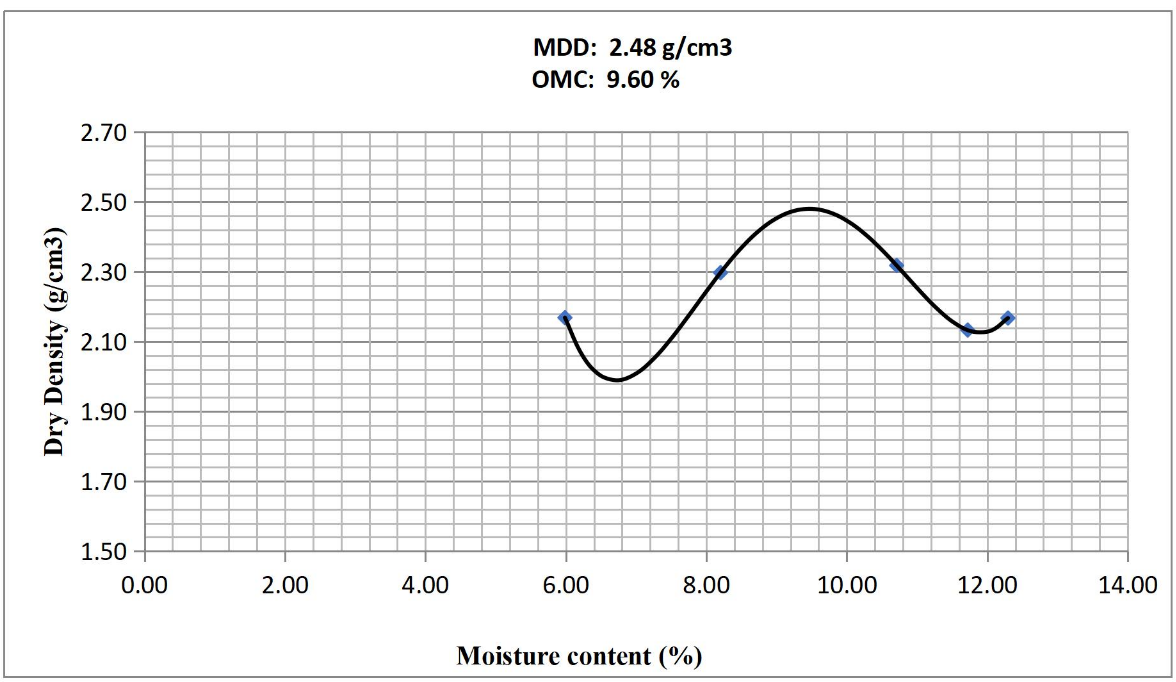


**Figure D1:** Compaction Test for 0% Cement with 0% CCR

**Table D2: Compaction Test for 2% Cement with 0% CCR**

| Trial | 1 | | 2 | | 3 | | 4 | | 5 | |
|---|---|---|---|---|---|---|---|---|---|---|
| Weight of Mould (g) | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | |
| Weight of Mould + Wet Sample (g) | 6438 | | 6656 | | 6734 | | 6685 | | 6602 | |
| Weight of wet Soil (g) | 2108 | | 2326 | | 2404 | | 2355 | | 2272 | |
| Wet Density (g/cm3) | 2.23 | | 2.46 | | 2.55 | | 2.49 | | 2.41 | |
| Can Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| Weight of Can (g) | 38.90 | 37.90 | 38.00 | 38.30 | 38.20 | 25.50 | 24.50 | 15.90 | 19.80 | 38.30 |
| Weight of Can + Wet Soil (g) | 63.60 | 63.80 | 67.20 | 65.40 | 68.40 | 46.70 | 42.00 | 34.30 | 41.10 | 68.20 |
| Weight of Can + Dry Soil (g) | 62.30 | 62.90 | 65.20 | 63.70 | 65.40 | 44.70 | 40.10 | 32.30 | 38.50 | 64.40 |
| Weight of Water (g) | 1.3 | 0.9 | 2 | 1.7 | 3 | 2 | 1.9 | 2 | 2.6 | 3.8 |
| Weight of Dry Soil (g) | 23.40 | 25.00 | 27.20 | 25.40 | 27.20 | 19.20 | 15.60 | 16.40 | 18.70 | 26.10 |
| Moisture Content (g) | 5.56 | 3.60 | 7.35 | 6.69 | 11.03 | 10.42 | 12.18 | 12.20 | 13.90 | 14.56 |
| Average Moisture Content (g) | 4.58 | | 7.02 | | 10.72 | | 12.19 | | 14.23 | |
| Dry Density (g/cm3) | 2.1353 | | 2.3023 | | 2.3000 | | 2.2237 | | 2.1069 | |

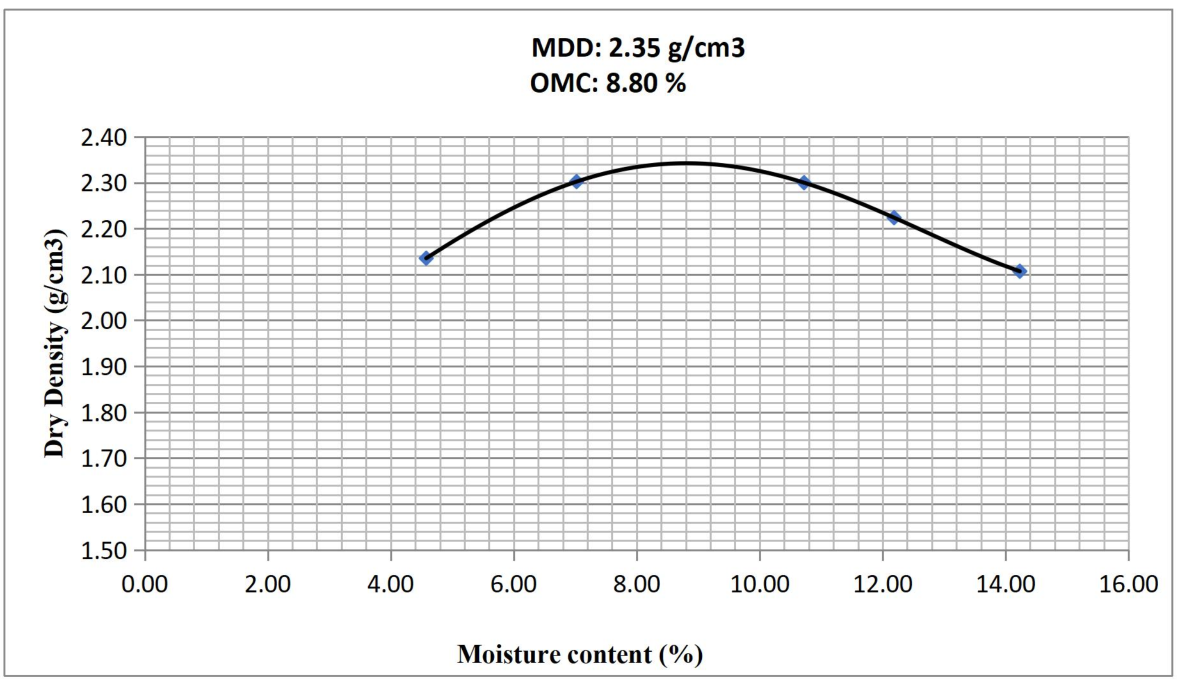


**Figure D2:** Compaction Test for 2% Cement with 0% CCR

**Table D3: Compaction Test for 2% Cement with 2% CCR**

| Trial | 1 | | 2 | | 3 | | 4 | | 5 | |
|---|---|---|---|---|---|---|---|---|---|---|
| Weight of Mould (g) | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | |
| Weight of Mould + Wet Sample (g) | 6455 | | 6646 | | 6684 | | 6635 | | 6522 | |
| Weight of wet Soil (g) | 2125 | | 2316 | | 2354 | | 2305 | | 2192 | |
| Wet Density (g/cm3) | 2.25 | | 2.45 | | 2.49 | | 2.44 | | 2.32 | |
| Can Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| Weight of Can (g) | 19.70 | 22.00 | 19.80 | 19.90 | 19.10 | 15.90 | 22.40 | 21.90 | 22.20 | 18.50 |
| Weight of Can + Wet Soil (g) | 39.90 | 39.90 | 35.10 | 37.10 | 36.70 | 31.60 | 42.10 | 40.80 | 57.20 | 45.40 |
| Weight of Can + Dry Soil (g) | 38.80 | 39.00 | 33.80 | 35.50 | 34.90 | 30.00 | 40.10 | 38.70 | 52.40 | 41.00 |
| Weight of Water (g) | 1.1 | 0.9 | 1.3 | 1.6 | 1.8 | 1.6 | 2 | 2.1 | 4.8 | 4.4 |
| Weight of Dry Soil (g) | 19.10 | 17.00 | 14.00 | 15.60 | 15.80 | 14.10 | 17.70 | 16.80 | 30.20 | 22.50 |
| Moisture Content (g) | 5.76 | 5.29 | 9.29 | 10.26 | 11.39 | 11.35 | 11.30 | 12.50 | 15.89 | 19.56 |
| Average Moisture Content (g) | 5.53 | | 9.77 | | 11.37 | | 11.90 | | 17.72 | |
| Dry Density (g/cm3) | 2.1332 | | 2.2350 | | 2.2391 | | 2.1821 | | 1.9724 | |

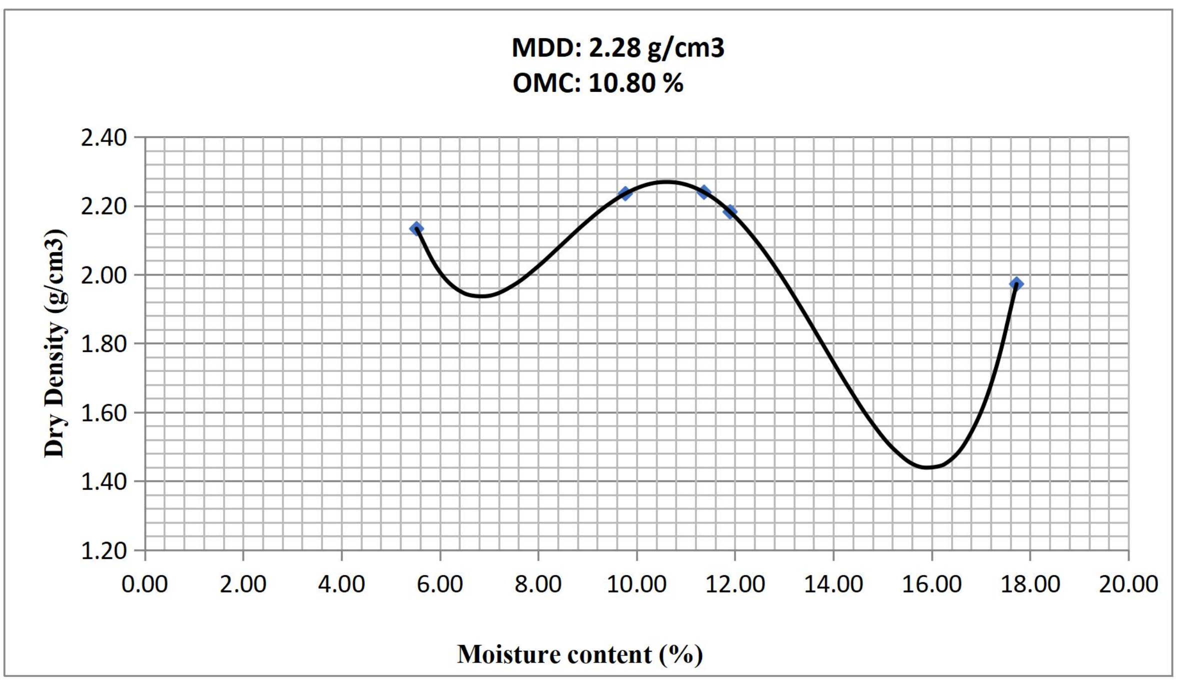


**Figure D3:** Compaction Test for 2% Cement with 2% CCR

**Table D4: Compaction Test for 2% Cement with 4% CCR**

| Trial | 1 | | 2 | | 3 | | 4 | | 5 | |
|---|---|---|---|---|---|---|---|---|---|---|
| Weight of Mould (g) | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | |
| Weight of Mould + Wet Sample (g) | 6436 | | 6565 | | 6677 | | 6668 | | 6575 | |
| Weight of wet Soil (g) | 2106 | | 2235 | | 2347 | | 2338 | | 2245 | |
| Wet Density (g/cm3) | 2.23 | | 2.37 | | 2.49 | | 2.48 | | 2.38 | |
| Can Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| Weight of Can (g) | 22.00 | 24.30 | 24.40 | 25.40 | 38.20 | 38.90 | 38.20 | 37.90 | 38.30 | 38.00 |
| Weight of Can + Wet Soil (g) | 38.60 | 40.70 | 41.70 | 43.70 | 62.50 | 66.00 | 59.50 | 58.70 | 80.80 | 70.60 |
| Weight of Can + Dry Soil (g) | 37.60 | 39.80 | 40.30 | 42.50 | 60.10 | 63.90 | 57.20 | 56.40 | 76.20 | 66.70 |
| Weight of Water (g) | 1 | 0.9 | 1.4 | 1.2 | 2.4 | 2.1 | 2.3 | 2.3 | 4.6 | 3.9 |
| Weight of Dry Soil (g) | 15.60 | 15.50 | 15.90 | 17.10 | 21.90 | 25.00 | 19.00 | 18.50 | 37.90 | 28.70 |
| Moisture Content (g) | 6.41 | 5.81 | 8.81 | 7.02 | 10.96 | 8.40 | 12.11 | 12.43 | 12.14 | 13.59 |
| Average Moisture Content (g) | 6.11 | | 7.91 | | 9.68 | | 12.27 | | 12.86 | |
| Dry Density (g/cm3) | 2.1025 | | 2.1940 | | 2.2668 | | 2.2060 | | 2.1071 | |

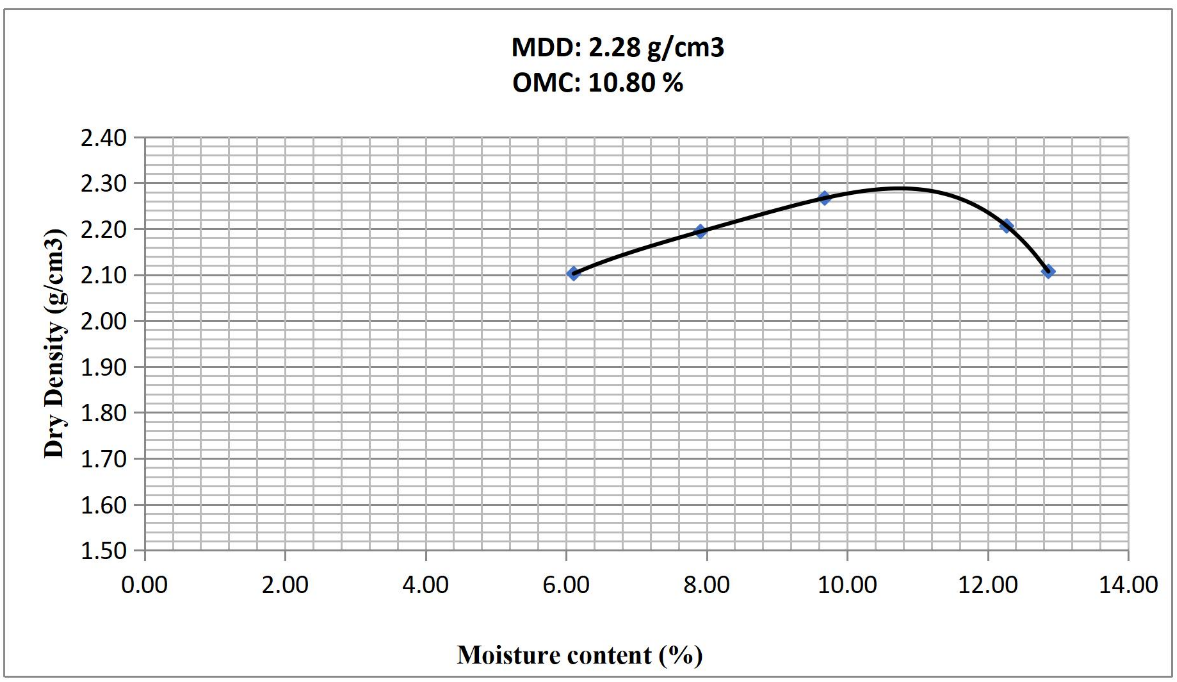


**Figure D4:** Compaction Test for 2% Cement with 4% CCR

**Table D5: Compaction Test for 2% Cement with 6% CCR**

| Trial | 1 | | 2 | | 3 | | 4 | | 5 | |
|---|---|---|---|---|---|---|---|---|---|---|
| Weight of Mould (g) | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | |
| Weight of Mould + Wet Sample (g) | 6394 | | 6555 | | 6695 | | 6642 | | 6520 | |
| Weight of wet Soil (g) | 2064 | | 2225 | | 2365 | | 2312 | | 2190 | |
| Wet Density (g/cm3) | 2.19 | | 2.36 | | 2.51 | | 2.45 | | 2.32 | |
| Can Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| Weight of Can (g) | 24.40 | 24.50 | 24.70 | 24.90 | 24.80 | 24.60 | 38.40 | 38.80 | 39.10 | 19.80 |
| Weight of Can + Wet Soil (g) | 51.70 | 47.90 | 56.40 | 47.70 | 55.60 | 48.30 | 68.40 | 68.60 | 67.60 | 53.40 |
| Weight of Can + Dry Soil (g) | 50.10 | 46.70 | 53.90 | 45.90 | 52.90 | 45.80 | 65.00 | 64.90 | 63.60 | 48.40 |
| Weight of Water (g) | 1.6 | 1.2 | 2.5 | 1.8 | 2.7 | 2.5 | 3.4 | 3.7 | 4 | 5 |
| Weight of Dry Soil (g) | 25.70 | 22.20 | 29.20 | 21.00 | 28.10 | 21.20 | 26.60 | 26.10 | 24.50 | 28.60 |
| Moisture Content (g) | 6.23 | 5.41 | 8.56 | 8.57 | 9.61 | 11.79 | 12.78 | 14.18 | 16.33 | 17.48 |
| Average Moisture Content (g) | 5.82 | | 8.57 | | 10.70 | | 13.48 | | 16.90 | |
| Dry Density (g/cm3) | 2.0663 | | 2.1710 | | 2.2631 | | 2.1582 | | 1.9845 | |

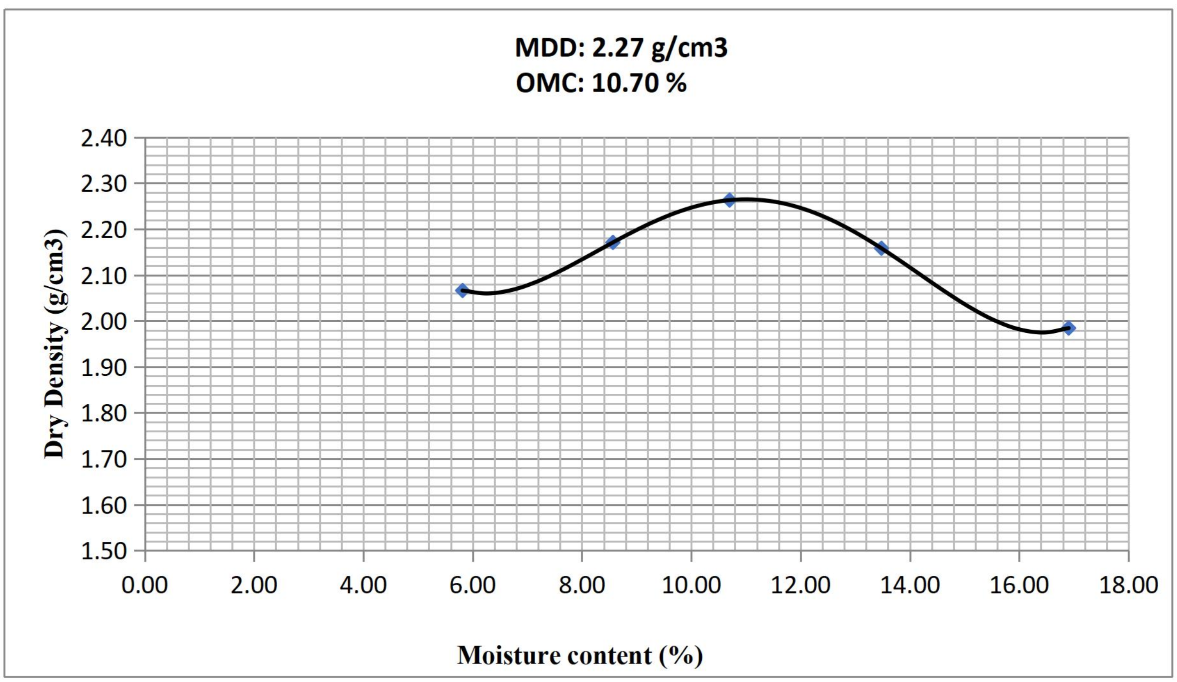


**Figure D5:** Compaction Test for 2% Cement with 6% CCR

**Table D6: Compaction Test for 4% Cement with 0% CCR**

| Trial | 1 | | 2 | | 3 | | 4 | | 5 | |
|---|---|---|---|---|---|---|---|---|---|---|
| Weight of Mould (g) | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | |
| Weight of Mould + Wet Sample (g) | 6412 | | 6568 | | 6718 | | 6656 | | 6590 | |
| Weight of wet Soil (g) | 2082 | | 2238 | | 2388 | | 2326 | | 2260 | |
| Wet Density (g/cm3) | 2.21 | | 2.37 | | 2.53 | | 2.46 | | 2.39 | |
| Can Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| Weight of Can (g) | 39.40 | 9.90 | 38.60 | 24.40 | 19.90 | 24.90 | 24.60 | 24.90 | 24.50 | 24.80 |
| Weight of Can + Wet Soil (g) | 76.90 | 40.60 | 79.90 | 50.20 | 41.20 | 47.00 | 62.30 | 56.00 | 53.60 | 59.10 |
| Weight of Can + Dry Soil (g) | 75.30 | 39.10 | 76.90 | 48.40 | 39.40 | 44.90 | 58.80 | 52.70 | 49.70 | 54.10 |
| Weight of Water (g) | 1.6 | 1.5 | 3 | 1.8 | 1.8 | 2.1 | 3.5 | 3.3 | 3.9 | 5 |
| Weight of Dry Soil (g) | 35.90 | 29.20 | 38.30 | 24.00 | 19.50 | 20.00 | 34.20 | 27.80 | 25.20 | 29.30 |
| Moisture Content (g) | 4.46 | 5.14 | 7.83 | 7.50 | 9.23 | 10.50 | 10.23 | 11.87 | 15.48 | 17.06 |
| Average Moisture Content (g) | 4.80 | | 7.67 | | 9.87 | | 11.05 | | 16.27 | |
| Dry Density (g/cm3) | 2.1046 | | 2.2020 | | 2.3025 | | 2.2188 | | 2.0590 | |

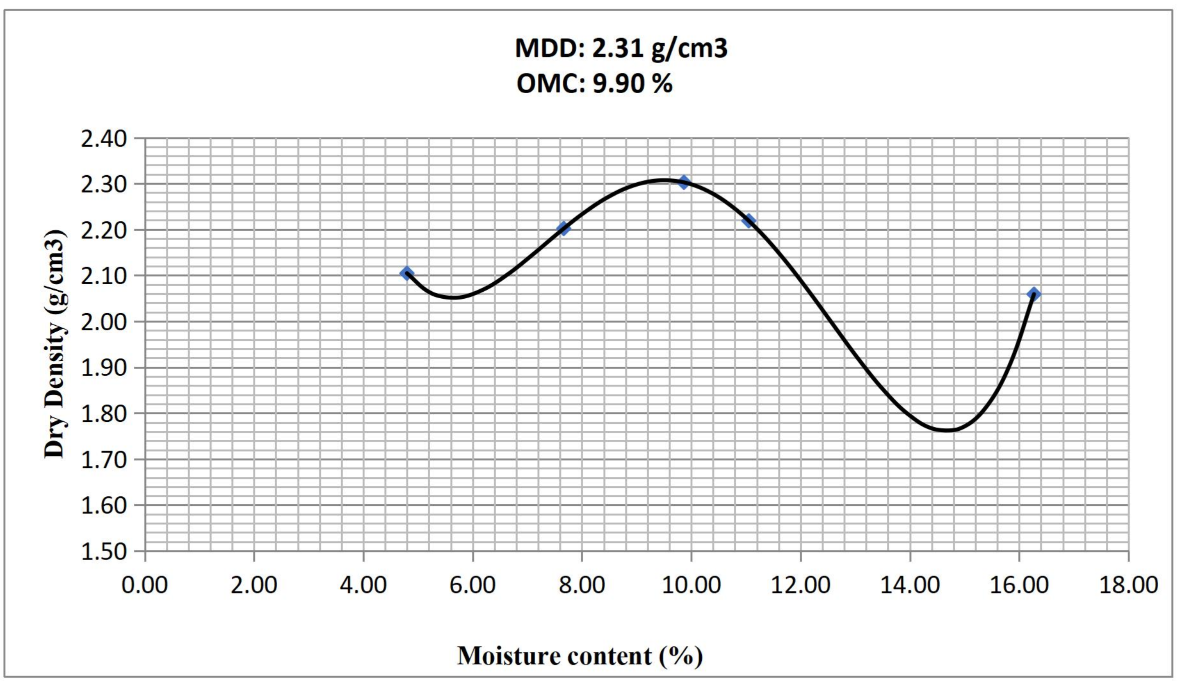


**Figure D6:** Compaction Test for 4% Cement with 0% CCR

**Table D7: Compaction Test for 4% Cement with 2% CCR**

| Trial | 1 | | 2 | | 3 | | 4 | | 5 | | 6 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Weight of Mould (g) | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | |
| Weight of Mould + Wet Sample (g) | 6430 | | 6596 | | 6635 | | 6626 | | 6520 | | 6470 | |
| Weight of wet Soil (g) | 2100 | | 2266 | | 2305 | | 2296 | | 2190 | | 2140 | |
| Wet Density (g/cm3) | 2.10 | | 2.27 | | 2.31 | | 2.30 | | 2.19 | | 2.14 | |
| Can Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| Weight of Can (g) | 22.20 | 19.80 | 22.00 | 21.90 | 22.00 | 22.40 | 19.70 | 19.90 | 15.90 | 19.10 | 38.20 | 38.40 |
| Weight of Can + Wet Soil (g) | 43.60 | 37.60 | 39.20 | 38.70 | 43.00 | 39.30 | 37.50 | 38.20 | 35.50 | 35.30 | 72.00 | 65.60 |
| Weight of Can + Dry Soil (g) | 42.50 | 36.80 | 38.00 | 37.40 | 41.20 | 38.00 | 35.60 | 36.30 | 33.00 | 33.20 | 67.50 | 61.80 |
| Weight of Water (g) | 1.1 | 0.8 | 1.2 | 1.3 | 1.8 | 1.3 | 1.9 | 1.9 | 2.5 | 2.1 | 4.5 | 3.8 |
| Weight of Dry Soil (g) | 20.30 | 17.00 | 16.00 | 15.50 | 19.20 | 15.60 | 15.90 | 16.40 | 17.10 | 14.10 | 29.30 | 23.40 |
| Moisture Content (g) | 5.42 | 4.71 | 7.50 | 8.39 | 9.37 | 8.33 | 11.95 | 11.59 | 14.62 | 14.89 | 15.36 | 16.24 |
| Average Moisture Content (g) | 5.06 | | 7.94 | | 8.85 | | 11.77 | | 14.76 | | 15.80 | |
| Dry Density (g/cm3) | 1.9988 | | 2.0992 | | 2.1175 | | 2.0543 | | 1.9084 | | 1.8480 | |

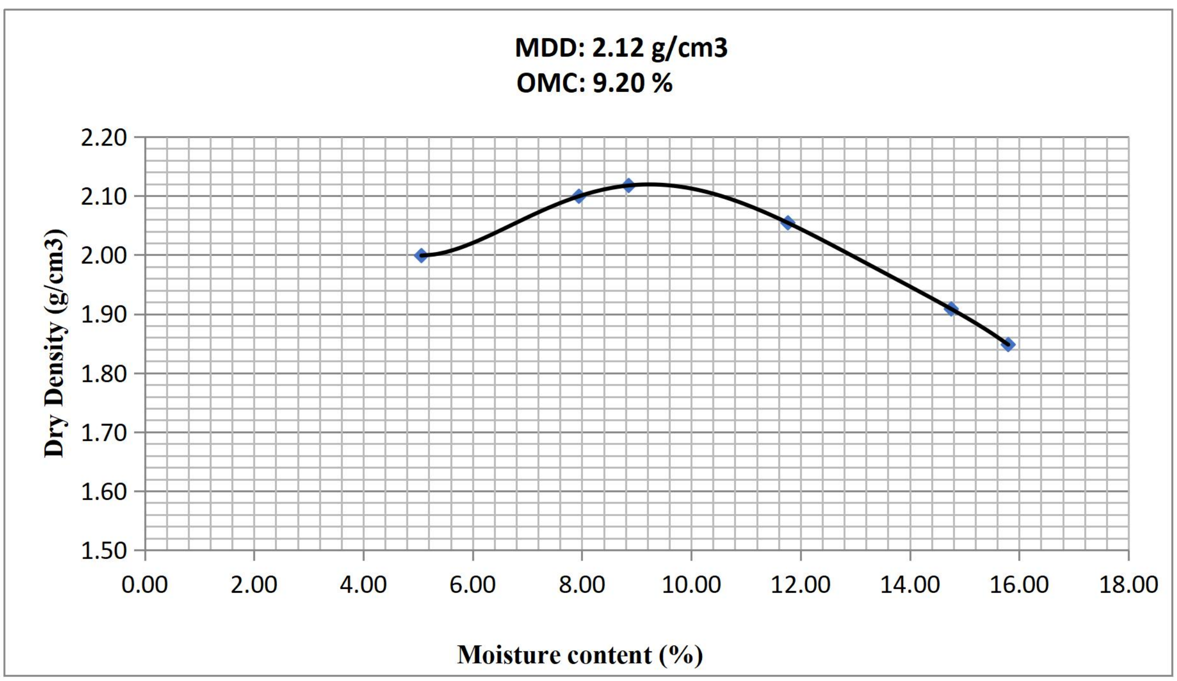


**Figure D7:** Compaction Test for 4% Cement with 2% CCR

**Table D8: Compaction Test for 4% Cement with 4% CCR**

| Trial | 1 | | 2 | | 3 | | 4 | | 5 | | 6 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Weight of Mould (g) | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | |
| Weight of Mould + Wet Sample (g) | 6448 | | 6608 | | 6685 | | 6685 | | 6618 | | 6504 | |
| Weight of wet Soil (g) | 2118 | | 2278 | | 2355 | | 2355 | | 2288 | | 2174 | |
| Wet Density (g/cm3) | 2.24 | | 2.41 | | 2.49 | | 2.49 | | 2.42 | | 2.30 | |
| Can Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| Weight of Can (g) | 18.50 | 24.30 | 24.30 | 25.30 | 24.60 | 24.90 | 24.40 | 24.90 | 24.70 | 24.80 | 38.60 | 39.40 |
| Weight of Can + Wet Soil (g) | 31.10 | 43.40 | 41.40 | 51.20 | 44.30 | 42.60 | 48.20 | 50.80 | 46.60 | 44.20 | 66.20 | 67.50 |
| Weight of Can + Dry Soil (g) | 30.60 | 42.40 | 40.30 | 49.80 | 42.60 | 40.90 | 45.60 | 48.20 | 44.10 | 41.90 | 62.90 | 63.90 |
| Weight of Water (g) | 0.5 | 1 | 1.1 | 1.4 | 1.7 | 1.7 | 2.6 | 2.6 | 2.5 | 2.3 | 3.3 | 3.6 |
| Weight of Dry Soil (g) | 12.10 | 18.10 | 16.00 | 24.50 | 18.00 | 16.00 | 21.20 | 23.30 | 19.40 | 17.10 | 24.30 | 24.50 |
| Moisture Content (g) | 4.13 | 5.52 | 6.88 | 5.71 | 9.44 | 10.63 | 12.26 | 11.16 | 12.89 | 13.45 | 13.58 | 14.69 |
| Average Moisture Content (g) | 4.83 | | 6.29 | | 10.03 | | 11.71 | | 13.17 | | 14.14 | |
| Dry Density (g/cm3) | 2.1403 | | 2.2702 | | 2.2672 | | 2.2332 | | 2.1417 | | 2.0177 | |

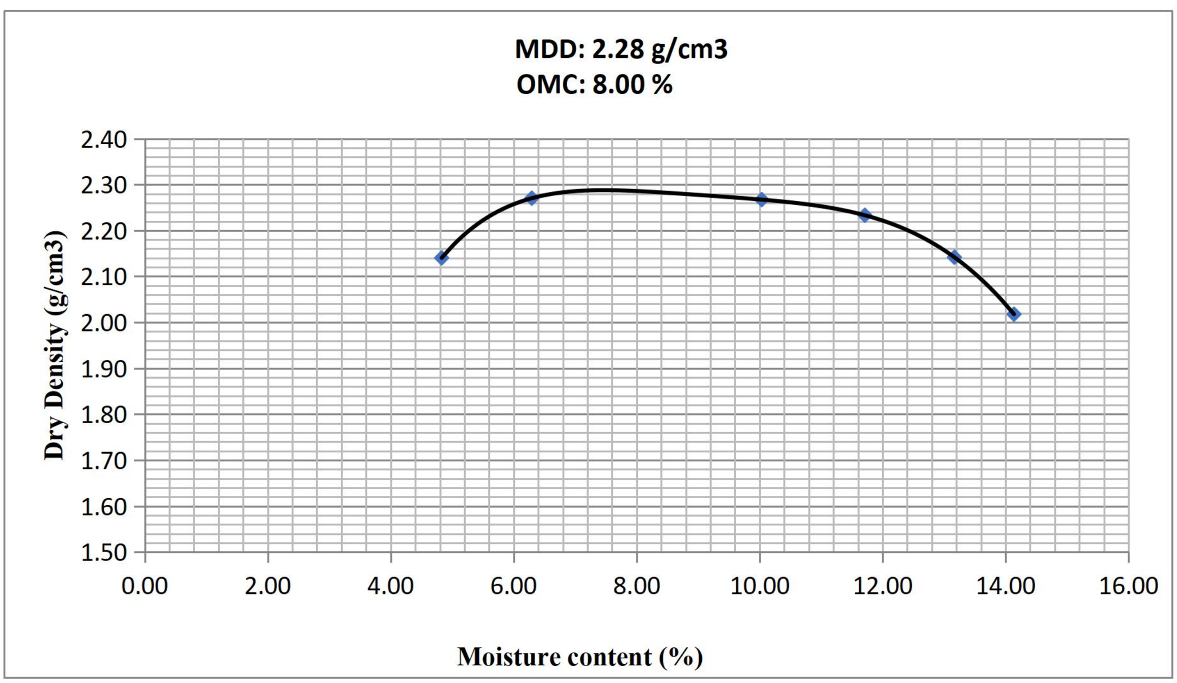


**Figure D8:** Compaction Test for 4% Cement with 4% CCR

**Table D9: Compaction Test for 4% Cement with 6% CCR**

| Trial | 1 | | 2 | | 3 | | 4 | | 5 | | 6 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Weight of Mould (g) | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | |
| Weight of Mould + Wet Sample (g) | 6410 | | 6535 | | 6630 | | 6685 | | 6602 | | 6545 | |
| Weight of wet Soil (g) | 2080 | | 2205 | | 2300 | | 2355 | | 2272 | | 2215 | |
| Wet Density (g/cm3) | 2.20 | | 2.34 | | 2.44 | | 2.49 | | 2.41 | | 2.35 | |
| Can Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| Weight of Can (g) | 24.50 | 24.60 | 24.40 | 24.50 | 24.90 | 24.80 | 19.80 | 19.80 | 38.40 | 38.00 | 38.80 | 39.10 |
| Weight of Can + Wet Soil (g) | 55.30 | 52.80 | 45.00 | 46.00 | 48.10 | 44.10 | 44.20 | 43.30 | 69.10 | 64.20 | 76.10 | 73.30 |
| Weight of Can + Dry Soil (g) | 53.70 | 51.50 | 43.30 | 44.70 | 46.10 | 42.40 | 41.60 | 41.00 | 65.30 | 61.20 | 71.90 | 68.70 |
| Weight of Water (g) | 1.6 | 1.3 | 1.7 | 1.3 | 2 | 1.7 | 2.6 | 2.3 | 3.8 | 3 | 4.2 | 4.6 |
| Weight of Dry Soil (g) | 29.20 | 26.90 | 18.90 | 20.20 | 21.20 | 17.60 | 21.80 | 21.20 | 26.90 | 23.20 | 33.10 | 29.60 |
| Moisture Content (g) | 5.48 | 4.83 | 8.99 | 6.44 | 9.43 | 9.66 | 11.93 | 10.85 | 14.13 | 12.93 | 12.69 | 15.54 |
| Average Moisture Content (g) | 5.16 | | 7.72 | | 9.55 | | 11.39 | | 13.53 | | 14.11 | |
| Dry Density (g/cm3) | 2.0954 | | 2.1685 | | 2.2241 | | 2.2397 | | 2.1200 | | 2.0562 | |

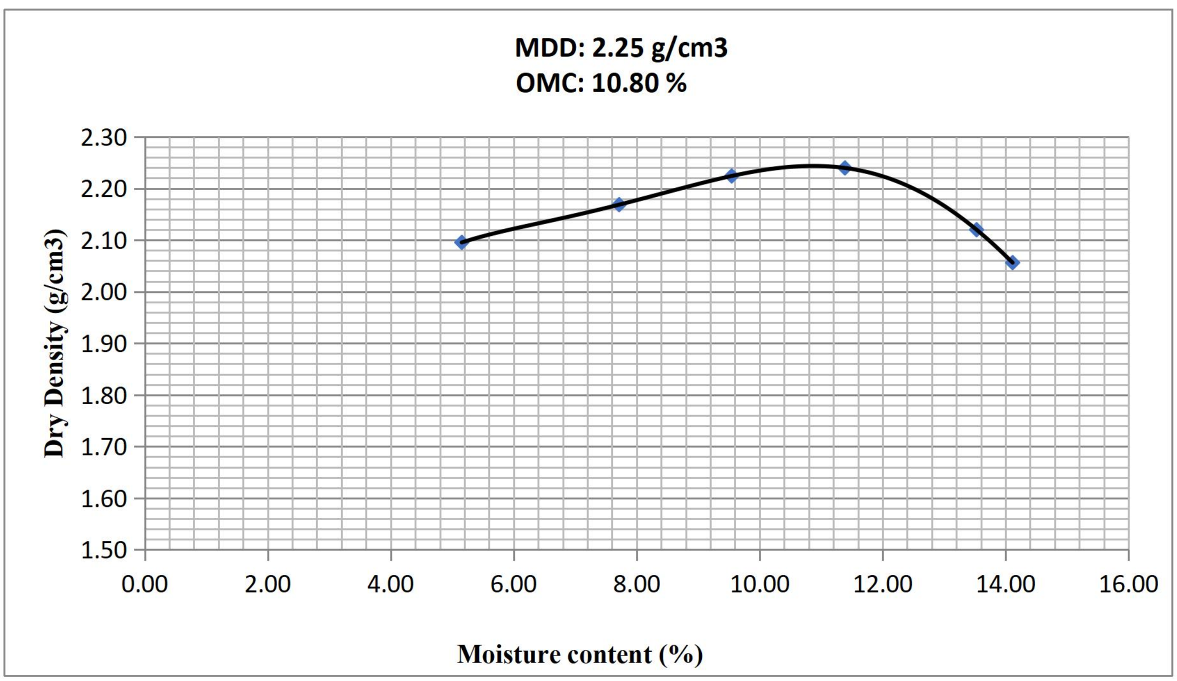


**Figure D9:** Compaction Test for 4% Cement with 6% CCR

**Table D10: Compaction Test for 6% Cement with 0% CCR**

| Trial | 1 | | 2 | | 3 | | 4 | | 5 | | 6 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Weight of Mould (g) | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | |
| Weight of Mould + Wet Sample (g) | 6500 | | 6610 | | 6750 | | 6695 | | 6636 | | 6585 | |
| Weight of wet Soil (g) | 2170 | | 2280 | | 2420 | | 2365 | | 2306 | | 2255 | |
| Wet Density (g/cm3) | 2.30 | | 2.42 | | 2.56 | | 2.51 | | 2.44 | | 2.39 | |
| Can Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| Weight of Can (g) | 24.50 | 24.50 | 24.70 | 25.00 | 24.40 | 24.50 | 24.80 | 24.80 | 25.30 | 24.80 | 38.90 | 38.20 |
| Weight of Can + Wet Soil (g) | 50.40 | 51.90 | 46.10 | 46.90 | 53.00 | 61.70 | 45.40 | 50.70 | 52.60 | 47.90 | 82.20 | 89.90 |
| Weight of Can + Dry Soil (g) | 49.10 | 50.50 | 44.70 | 45.40 | 50.60 | 58.40 | 42.80 | 48.30 | 49.40 | 45.40 | 76.00 | 82.70 |
| Weight of Water (g) | 1.3 | 1.4 | 1.4 | 1.5 | 2.4 | 3.3 | 2.6 | 2.4 | 3.2 | 2.5 | 6.2 | 7.2 |
| Weight of Dry Soil (g) | 24.60 | 26.00 | 20.00 | 20.40 | 26.20 | 33.90 | 18.00 | 23.50 | 24.10 | 20.60 | 37.10 | 44.50 |
| Moisture Content (g) | 5.28 | 5.38 | 7.00 | 7.35 | 9.16 | 9.73 | 14.44 | 10.21 | 13.28 | 12.14 | 16.71 | 16.18 |
| Average Moisture Content (g) | 5.33 | | 7.18 | | 9.45 | | 12.33 | | 12.71 | | 16.45 | |
| Dry Density (g/cm3) | 2.1823 | | 2.2535 | | 2.3423 | | 2.2303 | | 2.1674 | | 2.0514 | |

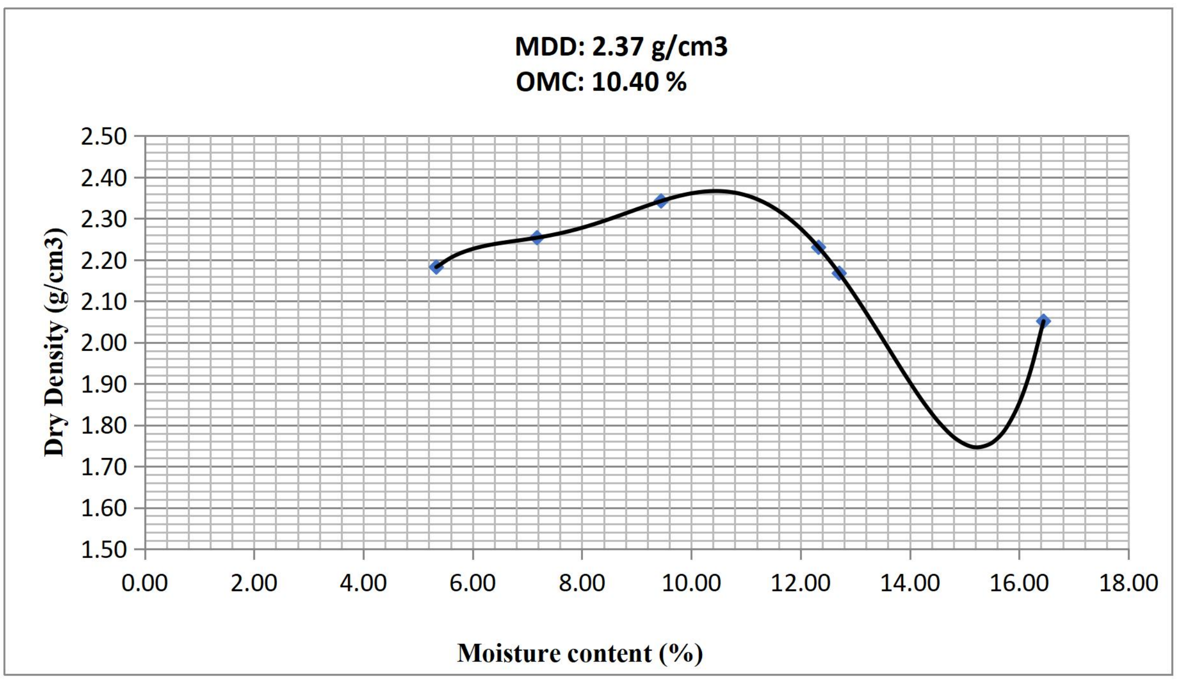


**Figure D10:** Compaction Test for 6% Cement with 0% CCR

**Table D11: Compaction Test for 6% Cement with 2% CCR**

| Trial | 1 | | 2 | | 3 | | 4 | | 5 | | 6 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Weight of Mould (g) | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | |
| Weight of Mould + Wet Sample (g) | 6490 | | 6515 | | 6759 | | 6712 | | 6620 | | 6570 | |
| Weight of wet Soil (g) | 2160 | | 2185 | | 2429 | | 2382 | | 2290 | | 2240 | |
| Wet Density (g/cm3) | 2.16 | | 2.19 | | 2.43 | | 2.38 | | 2.29 | | 2.24 | |
| Can Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| Weight of Can (g) | 20.50 | 19.90 | 19.80 | 19.80 | 25.30 | 25.00 | 25.00 | 24.30 | 24.70 | 24.80 | 24.90 | 24.00 |
| Weight of Can + Wet Soil (g) | 40.70 | 50.80 | 40.60 | 43.10 | 52.70 | 41.90 | 52.70 | 52.20 | 64.80 | 56.20 | 60.80 | 68.20 |
| Weight of Can + Dry Soil (g) | 39.70 | 49.70 | 39.40 | 41.70 | 50.70 | 40.40 | 50.00 | 49.40 | 60.90 | 52.80 | 55.90 | 64.00 |
| Weight of Water (g) | 1 | 1.1 | 1.2 | 1.4 | 2 | 1.5 | 2.7 | 2.8 | 3.9 | 3.4 | 4.9 | 4.2 |
| Weight of Dry Soil (g) | 19.20 | 29.80 | 19.60 | 21.90 | 25.40 | 15.40 | 25.00 | 25.10 | 36.20 | 28.00 | 31.00 | 40.00 |
| Moisture Content (g) | 5.21 | 3.69 | 6.12 | 6.39 | 7.87 | 9.74 | 10.80 | 11.16 | 10.77 | 12.14 | 15.81 | 10.50 |
| Average Moisture Content (g) | 4.45 | | 6.26 | | 8.81 | | 10.98 | | 11.46 | | 13.15 | |
| Dry Density (g/cm3) | 2.0680 | | 2.0563 | | 2.2324 | | 2.1464 | | 2.0546 | | 1.9796 | |

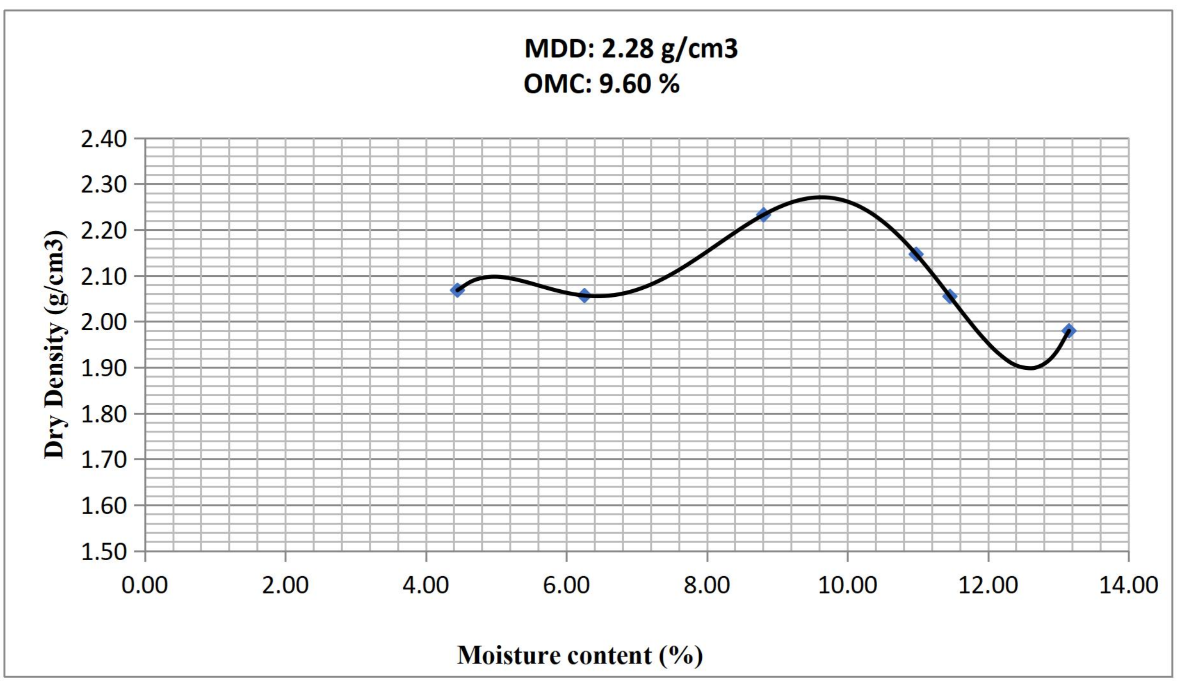


**Figure D11:** Compaction Test for 6% Cement with 2% CCR

**Table D12: Compaction Test for 6% Cement with 4% CCR**

| Trial | 1 | | 2 | | 3 | | 4 | | 5 | | 6 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Weight of Mould (g) | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | |
| Weight of Mould + Wet Sample (g) | 6422 | | 6530 | | 6750 | | 6693 | | 6650 | | 6522 | |
| Weight of wet Soil (g) | 2092 | | 2200 | | 2420 | | 2363 | | 2320 | | 2192 | |
| Wet Density (g/cm3) | 2.09 | | 2.20 | | 2.42 | | 2.36 | | 2.32 | | 2.19 | |
| Can Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| Weight of Can (g) | 22.00 | 18.60 | 15.90 | 22.50 | 19.80 | 22.10 | 19.10 | 19.90 | 24.20 | 22.00 | 24.20 | 25.00 |
| Weight of Can + Wet Soil (g) | 43.70 | 38.20 | 42.50 | 45.60 | 43.70 | 36.60 | 36.60 | 38.50 | 40.00 | 39.50 | 49.60 | 59.70 |
| Weight of Can + Dry Soil (g) | 43.00 | 37.40 | 41.00 | 44.00 | 42.40 | 34.90 | 34.90 | 36.70 | 38.50 | 37.60 | 46.20 | 55.20 |
| Weight of Water (g) | 0.7 | 0.8 | 1.5 | 1.6 | 1.3 | 1.7 | 1.7 | 1.8 | 1.5 | 1.9 | 3.4 | 4.5 |
| Weight of Dry Soil (g) | 21.00 | 18.80 | 25.10 | 21.50 | 22.60 | 12.80 | 15.80 | 16.80 | 14.30 | 15.60 | 22.00 | 30.20 |
| Moisture Content (g) | 3.33 | 4.26 | 5.98 | 7.44 | 5.75 | 13.28 | 10.76 | 10.71 | 10.49 | 12.18 | 15.45 | 14.90 |
| Average Moisture Content (g) | 3.79 | | 6.71 | | 9.52 | | 10.74 | | 11.33 | | 15.18 | |
| Dry Density (g/cm3) | 2.0155 | | 2.0617 | | 2.2097 | | 2.1339 | | 2.0838 | | 1.9031 | |

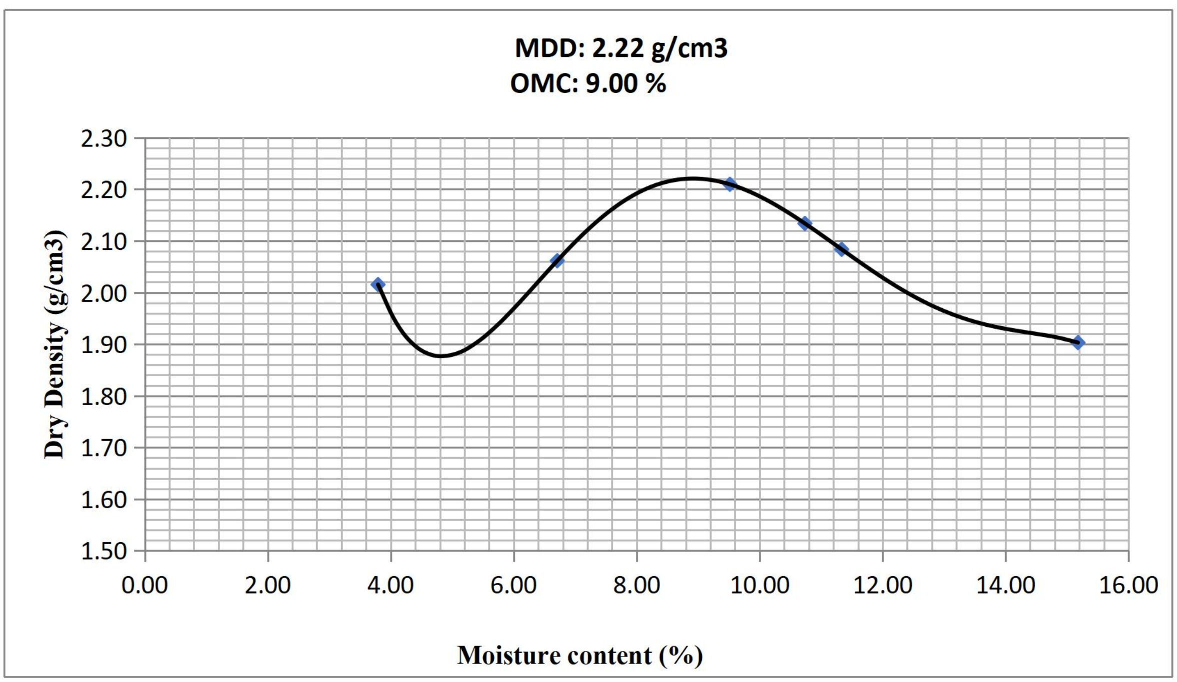


**Figure D12:** Compaction Test for 6% Cement with 4% CCR

**Table D13: Compaction Test for 6% Cement with 6% CCR**

| Trial | 1 | | 2 | | 3 | | 4 | | 5 | | 6 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Weight of Mould (g) | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | | 4330 | |
| Weight of Mould + Wet Sample (g) | 6475 | | 6527 | | 6719 | | 6690 | | 6628 | | 6510 | |
| Weight of wet Soil (g) | 2145 | | 2197 | | 2389 | | 2360 | | 2298 | | 2180 | |
| Wet Density (g/cm3) | 2.27 | | 2.33 | | 2.53 | | 2.50 | | 2.43 | | 2.31 | |
| Can Number | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| Weight of Can (g) | 25.00 | 25.40 | 24.90 | 24.80 | 24.30 | 24.70 | 24.70 | 24.80 | 24.60 | 25.20 | 24.60 | 25.30 |
| Weight of Can + Wet Soil (g) | 47.80 | 43.60 | 53.50 | 47.40 | 53.80 | 44.60 | 58.40 | 43.80 | 52.30 | 53.00 | 64.50 | 62.00 |
| Weight of Can + Dry Soil (g) | 47.00 | 42.70 | 52.20 | 45.90 | 51.40 | 42.70 | 54.90 | 41.80 | 48.60 | 50.00 | 59.20 | 57.20 |
| Weight of Water (g) | 0.8 | 0.9 | 1.3 | 1.5 | 2.4 | 1.9 | 3.5 | 2 | 3.7 | 3 | 5.3 | 4.8 |
| Weight of Dry Soil (g) | 22.00 | 17.30 | 27.30 | 21.10 | 27.10 | 18.00 | 30.20 | 17.00 | 24.00 | 24.80 | 34.60 | 31.90 |
| Moisture Content (g) | 3.64 | 5.20 | 4.76 | 7.11 | 8.86 | 10.56 | 11.59 | 11.76 | 15.42 | 12.10 | 15.32 | 15.05 |
| Average Moisture Content (g) | 4.42 | | 5.94 | | 9.71 | | 11.68 | | 13.76 | | 15.18 | |
| Dry Density (g/cm3) | 2.1761 | | 2.1969 | | 2.3068 | | 2.2386 | | 2.1399 | | 2.0049 | |

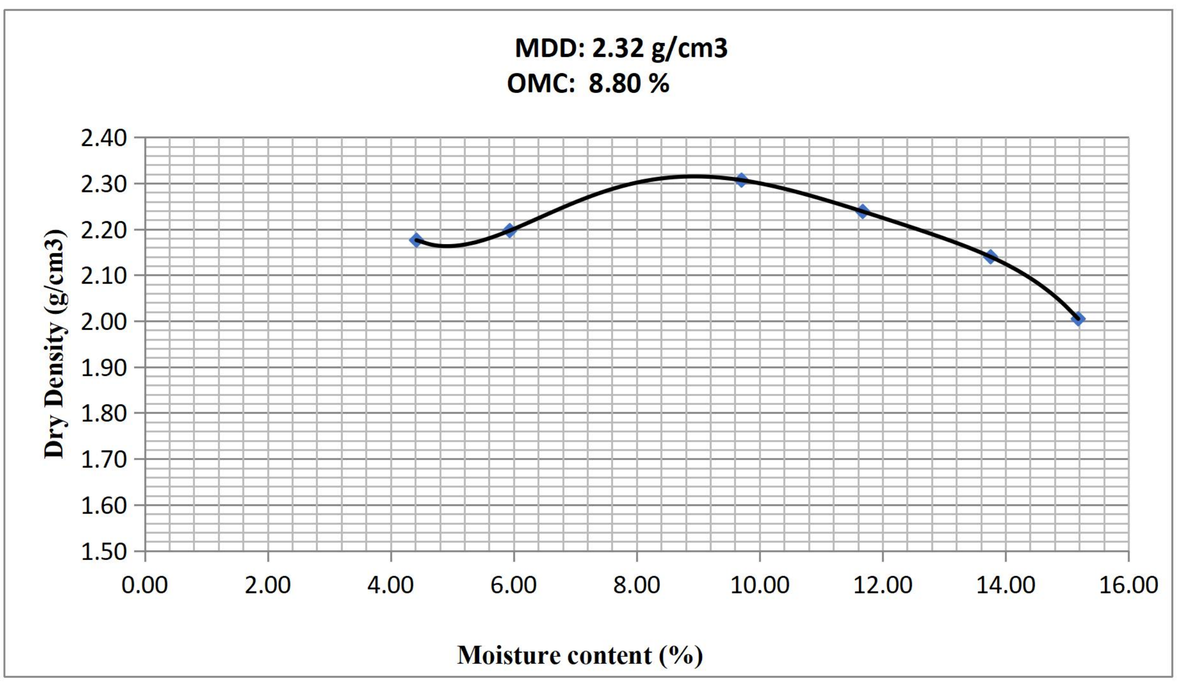


**Figure D13:** Compaction Test for 6% Cement with 6% CCR