UTILIZATION OF GLASS CULLET AS AN ADMIXTURE TO CEMENT STABILIZED BLACK COTTON SOIL

BY

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A THESIS SUBMITTED TO THE DEPARTMENT OF CIVIL ENGINEERING, FACULTY OF ENGINEERING, BAYERO UNIVERSITY, KANO IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF MASTER DEGREE IN CIVIL ENGINEERING

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DECLARATION

I **RAHAMA BABALE SHU'AIBU** hereby declare that this thesis is a product of my own research efforts undertaken under the supervision of Dr. A. Y. Abdulfatah. All sources of information utilized which are not mine where specifically acknowledged by means of reference.

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CERTIFICATION

This is to certify that the research work for this thesis was conducted solely by the student; RAHAMA BABALE SHU'AIBU (SPS/15/MCE/00012), under my supervision, and has been presented in accordance with the rules and regulations governing the presentation and preparation of Master's thesis in Bayero University, Kano.

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APPROVAL PAGE

This is to certify that this research work titled; “Utilization of Glass Cullet as an admixture to Cement Stabilized Black Cotton Soil” has been thoroughly read and approved as having satisfied the requirements for the award of Master’s Degree (M. Eng Geotechnical), in the Department of Civil Engineering, Faculty of Engineering, Bayero University, Kano.

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DEDICATION

This thesis is dedicated to my husband Umar Haruna Rawayau.

TABLE OF CONTENT

CHAPTER TWO

LITERATURE REVIEW

CHAPTER THREE

METHODOLOGY

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

ABBREVIATIONS, DEFINITIONS, SYMBOLS

AASHTO – American Association of State Highway and Transportation Officials

ASTM – American Society for Testing and Materials

BCS – Black Cotton Soil

BGA – Bagasse Ash

BS – British Standard

CBR – California Bearing Ratio

Cm – Cement Content

C – S - H – Calcium Silicate Hydrates

DICON – Defense Industries Corporation of Nigeria

G_S – Specific Gravity

LL – Liquid Limit

PL – Plastic Limit

PI – Plasticity Index

LS – Linear Shrinkage

MDD – Maximum Dry Density

OMC – Optimum Moisture Content

BSL – British Standard Light

SP – Standard Proctor

BSH - British Standard Heavy

MP – Modified Proctor

NBRRI – Nigerian Building and Road Research Institute

OPC – Ordinary Portland Cement

PVC – Potential Volume Change

RSH – Rice Husk Ash

UCS – Unconfined Compressive Strength

UCSC – Unified Soil Classification System

USBR – United State Bureau of Reclamation

W – Moisture Content

GC – Glass Cullet

XRD – X-Ray Diffraction

XRF – X-Ray Fluorescence

ρ = Bulk density

ρ_d = Dry density

ε = Strain

Δl = Amount of compression at any stage

R = Load ring reading at strain

C_r = Mean calibration of load ring

l_o = Initial length of specimen

A_o = Initial cross-sectional area

σ = Compressive stress at strain

ABSTRACT

Expansive soils such as black cotton soil (BCS) pose serious challenge in engineering construction due to their heaving and shrinkage behaviour. Many techniques are used around the world to improve their engineering properties among which are soil stabilization. This research work presents the efficacy of glass cullet (GC) when used together with cement for stabilization of BCS. Laboratory tests were conducted in accordance with the British Standards on the BCS treated with 0%, 2%, 4%, 6%, and 8% ordinary Portland cement (OPC) as well as 0%, 5%, 10%, 15% and 20% GC. The soil was classified as A-7-5(14) and CH according to the American Association of State Highway and Transportation Officials and the Unified Soil Classification System respectively. California Bearing Ratio (CBR) and unconfined compressive strength (UCS) were conducted using British Standard Light (BSL) and British Standard Heavy (BSH) compactive efforts, UCS were cured for 7, 14, and 28 days. The result showed a significant improvement on the engineering properties of the soil. There was a decrease in "liquid limit, plastic limit and plasticity index," and an increase in maximum dry density with increase in GC content for all cement proportions used and GC alone. The maximum UCS values of 1152 kN/m^2 and 1568 kN/m^2 for BSL and BSH were respectively obtained after 7 days curing. These are, however, much lower than the recommended value of 1720 kN/m^2 by the Nigerian General Specification. The maximum CBR values of 53.8% and 63% were obtained for BSL and BSH compactive efforts corresponding to the blend containing 8% OPC and 5% GC. More importantly, with an 8% OPC content, all samples containing GC from 5% to 20% satisfied the sub-base requirements of 30% CBR. BCS treated partially with OPC and GC within that range is therefore recommended for use as subbase and subgrade materials in road construction.

CHAPTER ONE

INTRODUCTION

1.1 General

Expansive soils pose a significant hazard to foundations of buildings founded in them. Such soils can exert uplift pressures which cause considerable damage to lightly loaded structures. (Das and Roy, 2014). Due to this behaviour, a lot of damages occur on structures founded on this type of soil. This soil is generally found in arid and semi-arid regions of the world.

The primary problem that arises with regard to expansive soils is that plastic deformations are significantly greater than elastic deformations and they cannot be predicted by classical elastic or plastic theory. The annual cycle of wetting and drying causes the soil to swell and shrink. Thus, the arid and semi-arid regions are much susceptible to damage from expansive soils throughout the year (Zumrawi et al., 2017) This soil being expansive creates several types of damages to pavement structures, and in some cases the pavement may even become unserviceable (Tailor and Shah, 2015).

The damages normally appear as cracks in buildings, canal beds and linings, pavements, lifting of water supply pipeline and sewerage lines etc. A number of innovative techniques are available for construction on this type of soil. They include changing the physical and chemical alteration of soil using solid wastes such as fly ash, rice husk ash, marble dust, phosphogypsum, granulated blast furnace slag, red mud, waste tyre, among others (Pandian et al., 2001; Swami, 2002; Phanikumar and Sharma, 2004; Kalkan, 2006; Degirmenci et al., 2007; Cokca et al., 2009; Sabat and Nanda, 2011; Patil et al., 2011; Agrafioti et al., 2014). Utilization of solid wastes in this manner not only protects the environment from degradation but also improves the engineering properties of the expansive soil.

The materials generally used in large proportion for soil stabilisation include cement, lime, bitumen, others include, lignin, molasses, and several of the synthetic resins (Mwanga, 2015). Despite the proven performance of lime and Portland cement in the modification of the engineering and allied properties of problematic soils, the cost of blending soils with these stabilizers is usually prohibitive. In order to abate the cost of road base stabilisation, one reasonable alternative is to mix the soil-lime or soil-cement blends with requisite amount of an admixture that can serve similar purpose (Matawal et al., 2006). The intent of this research is to investigate the potentials of using Glass Cullet (GC) as an admixture to cement stabilized expansive soil.

1.2 Problem Statement

When structures are built on expansive soils, they experience either settlement or heave depending on the swelling pressure of the soil (Khan et al., 2013). The design and construction of civil engineering structures on expansive soils pose serious challenges as the frequency of road pavement failures has been of great concern to road engineers in Nigeria (Eze-Uzomaka and Agbo, 2010). Structures sitting on expansive soils (such as black cotton soil) suffer from both structural and non-structural damages due to the alternate expansion and contraction of the soil caused by moisture regime (Meshram et al., 2013; Bajaj et al., 2016; Sani et al., 2017). In road construction, the sub-grade soil is an integral element that supports pavements and must therefore give adequate support and stability under adverse loading conditions (Akanbi, 2010).

In Nigeria, black cotton soil, which is expansive clay, is typically found in low lying areas of North Eastern States of Gombe, Borno, Yobe, Adamawa, Taraba, and Bauchi. The soils, according to Akanbi (2010), occur in discontinuous stretches as superficial deposits; usually not more than 2 m thick. In view of the problematic nature of the black cotton soil, attempts have been made by researchers all over the world to stabilize the expansive soil using various materials and methods. This is evidenced by the volume of literature on the subject matter (e.g Bajaj et al., 2016; Etim et al., 2017; Miao et al., 2017).

In order to mitigate the problems posed by black cotton soil to road construction and similar applications, it becomes necessary to economically stabilize the soil in order to enhance its engineering properties to meet the desired objectives. However, the major problem associated with chemical stabilisation is the high cost of lime, cement and bitumen. Attempts have been made by researchers to utilize locally obtainable low cost materials including industrial by products as well as agricultural wastes to economically modify the engineering and allied properties of deficient soils at a moderate cost (Osinubi, 1999; Elinwa and Awari, 2001; Jade and

Shaikh, 2018) This makes it vital to determine the potential of Glass Cullet as a soil stabilizer so as to provide a cheaper alternative.

1.3 Justification for the Study

Lime, OPC or bitumen has a proven record of good performance when used properly to stabilize soils of high plasticity indices. Previous studies indicated that the use of these stabilizers could be expensive especially in large projects. The economic benefits of additives in conjunction with well know stabilizers have been highlighted in the literature (Alhassan and Alhaji, 2017; Murmu et al., 2018)

Chemical analysis has shown that many industrial wastes are rich in main oxides such as CaO, SiO_2, Al_2O_3, and Fe_2O_3 (Edeh et al., 2016). These oxides have self-cementing characteristics similar to OPC.

With regard to Glass Cullet (GC) materials, limited glass recycling opportunities exist and those related to color stored (clear, amber, brown) opportunities such as bottling are both sporadic and dwindling from the increasing use of plastic containers. Yet in some cases (liquor, soft drinks and certain foods), glass remains the container material of choice and it will thus remain in the recycling stream because its weight makes it the leading candidate for attaining community-based recycling target objectives (% by weight basis). In addition, mixed color broken glass continues to accumulate due to the lack of beneficial use markets in certain regions and also due to its relatively high density, is expensive to transport long distances. Transportation costs often outweigh the market price of GC as container batch. One application is to crush curbside-collected glass for freely geotechnical field applications. However, Glass Cullet has received extremely limited use due to unfamiliarity, negative perception, and lack of approved specifications (Wartman et al., 2004).

The use of GC stems from, the need to reduce the cost of waste disposal, use cheap and economic soil stabilizer in geotechnical and road work, as well as make the unsuitable natural soils fit for use in engineering work at economic cost, especially where avoiding or by-passing them is difficult.

1.4 Aim and Objectives of Research

1.4.1 Aim

The Aim of this research work is to establish the stabilisation potential of GC when used as an admixture in ordinary Portland cement (OPC) stabilized black-cotton soil in road construction.

1.4.2 Objectives

The specific objectives of the study include:

(i) Determination of the index properties of Black cotton soil sample and classified it using AASHTO soil classification system.

(ii) Determination of the effect of GC on the index and engineering properties of black cotton soil treated with OPC.

(iii) Determination of the optimum blend of cement and GC admixture that gives maximum improvement of the engineering properties of black cotton soil.

1.5 Significance of the Study

Various stabilizers have been used to stabilize or improve the geotechnical properties of black cotton soils and the commonest stabilizers used for this purpose in Nigeria are cement, lime and bitumen. However, the cost implication has made researchers to focus on the use of potentially cost-effective materials that are available to improve the properties of deficient soils.

It is in this light that an available mineral solid waste is sought for use as a placement. If this is realized, it will serve as a disposal technique to glass producing factories.

1.6 Scope of the Work

The research work basically involved the following phases;

The first phase of the investigation is the preliminary analysis which involves tests on natural soil to determine the engineering properties of the soil. The tests include:

a. X-Ray Diffraction (XRD)
b. X-Ray Fluorescence (XRF)
c. Moisture Content
d. Atterberg limit
e. Sieve Analysis
f. Specific Gravity
g. Free swell
h. Compaction test (BSL and BSH)
i. California Bearing Ratio (CBR)
j. Unconfined Compressive Strength (UCS)

The second phase involved the addition of varying proportions of cement and crushed glass (i.e., 0, 2, 4, 6 and 8% for cement and 5, 10, 15, and 20% for Glass Cullet) by dry weight of soil to determine the soil properties when cement alone and Glass Cullet alone are used as a single stabilizing agent.

The third phase involved stabilisation of the soil using OPC and addition of varying proportion of GC in the range of 5%, 10%, 15%, and 20 % to the various percentages of the cement used.

The tests conducted in phases two and three are Atterberg limits, compaction (BSL and BSH), CBR and UCS.

1.7 Limitation of the Study

Results of this study relied on a set of limitations and criteria that were taken into account during the experimental work. These limitations include:

a. The glass used in this study was derived from post-consumer glass sources. Other types of glass such as ceramic plates, vacuum tubing, mirrors, medical or laboratory glass, etc. are not within the concern of this research study.
b. Glass Cullet powder with an average particle size of < 300 µm was used.

CHAPTER TWO

LITERATURE REVIEW

2.1 Expansive Soils

Expansive soils, which usually contain the clay mineral montmorillonite, include sedimentary and residual soils, clay stones, and shales. In arid and semiarid climates, they exist in a moisture-deficient, unsaturated condition. The expansive nature of soil is most obvious near ground surface where the profile is subject to seasonal, environmental changes (Terzaghi et al,. 1996; Hazelton and Murphy, 2016).

There are many correlations that are useful in identifying potentially expansive soils. It may also be possible to identify them visually. Visual indications include (Fauzi et al., 2017):

1) Wide and deep shrinkage cracks occurring during dry periods

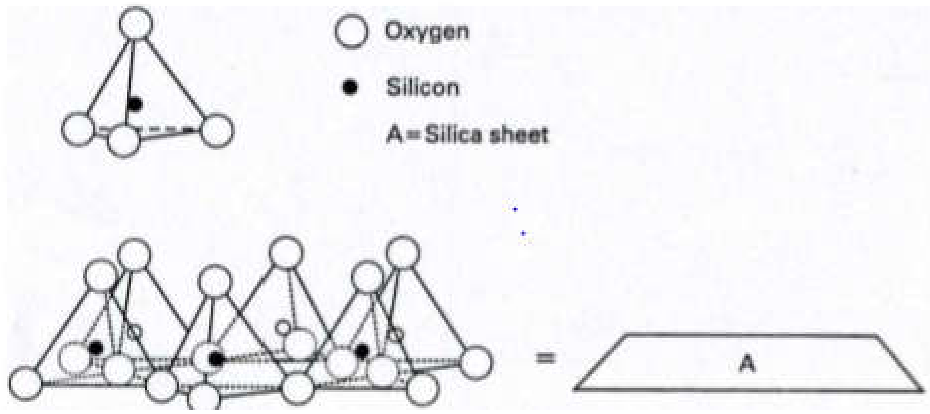
2) Soil is rock-hard when dry, but very sticky and soft when wet

3) Damages on the surrounding structures due to expansion of soil

2.1 Clay Mineralogy

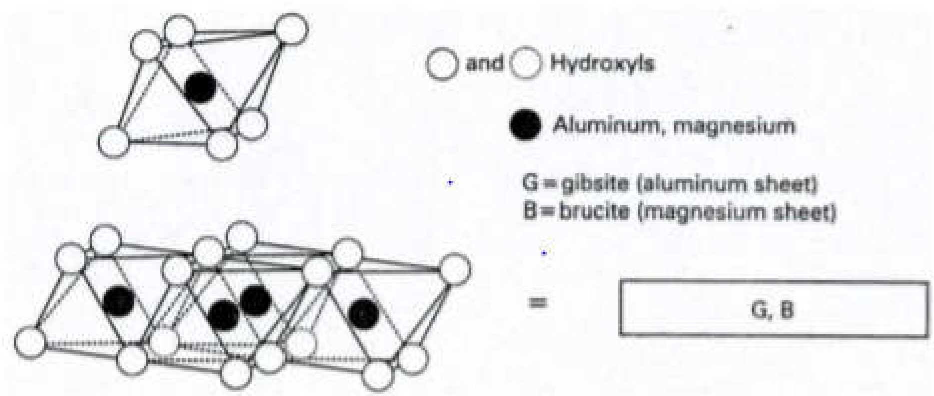
The term clay can refer both to a size and to a class of minerals. As a size term, it refers to all constituents of a soil smaller than a particular size, usually 0.002 mm in engineering classifications. As a mineral term, it refers to specific clay minerals that are distinguished by (1) small particle size, (2) a net electrical charge, (3) plasticity when mixed with water and (4) high weathering resistance (Mitchell and Soga, 2005).

The basic idealized crystalline structural unit of a clay mineral is composed of a silica tetrahedron block and an aluminum octahedron block. Aluminum octahedron block may have Aluminum (Al^{3+}) or magnesium (Mg^{2+}). If only aluminum is present, it is called gibbsite [$Al_2(OH)_6$]; if only magnesium is present, it is called brucite [$Mg_3(OH)_6$]. Various clay minerals are formed as these sheets stack on top of each other with different ions bonding them together (Dai and Zhao, 2018). A silica tetrahedron and a silica sheet, also an octahedron and an octahedron sheet are presented in Figure 2.1 and Figure 2.2, respectively. Also, these figures consist of schematic representations of silica and octahedron sheets.



(a) (b)

Figure 2.1 (a) Silica Tetrahedron and (b) Silica Sheet



(a) (b)

Figure 2.2 (a) Octaheron and (b) Octaheron Sheet

Three important structural groups of clay minerals are described for engineering purposes as follows:

1. Kaolinite group - generally nonexpansive
2. Mica-like group - includes illites and vermiculites, which can be expansive but generally do not pose significant problems.

3. Smectite group - includes montmorillonites, which are highly expansive and are the most troublesome clay minerals (Zumrawi et al., 2017).

2.1.1 Kaolinite group

Kaolinite

Kaolinite crystals consist of tetrahedron and octahedron sheets. The bonding between successive layers is by van der Waals forces and hydrogen bonds. The bonding is sufficiently strong that there is no interlayer swelling in the presence of water (Mitchell and Soga, 2005).

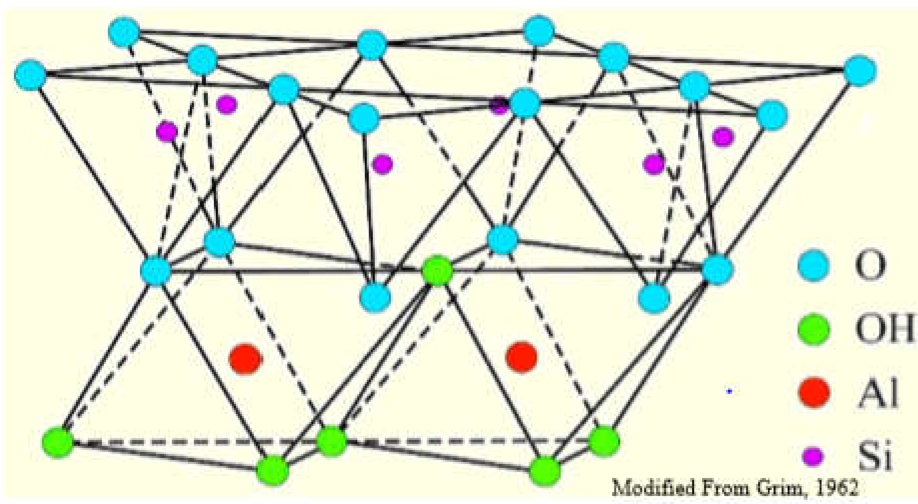


Figure 2.3 Diagrammatic Skectch of the Kaolonite (source: USGS, 2001)

2.1.2 Illite

Illite has a basic structure consisting of a sheet of alumina octahedrons between and combined with two sheets of silica tetrahedrons. In the octahedral sheet there is partial substitution of aluminum by magnesium and iron, and in the tetrahedral sheet there is partial substitution of silicon by aluminum. The combined sheets are linked together by fairly weak bonding due to (non - exchangeable) potassium ions held between them (Craig, 2004; Dai and Zhao, 2018).

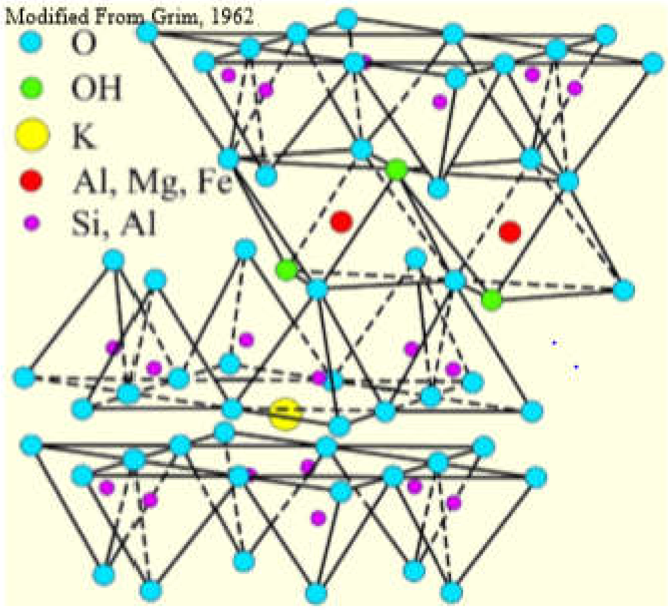


Figure 2.4 Diagrammatic Skectch of the Illite (source: USGS, 2001)

2.1.3 Montmorillonite

Montmorillonite is formed from weathering of volcanic ash under poor drainage conditions or in marine waters. The basic building sheets for smectite are the same as for illite except there is no potassium ion present. The space between the combined sheets is occupied by water molecules and exchangeable cations. There is a very weak bond between the combined sheets due to these ions. Considerable swelling of montmorillonite can occur due to additional water being absorbed between the combined sheets (Craig, 2004).

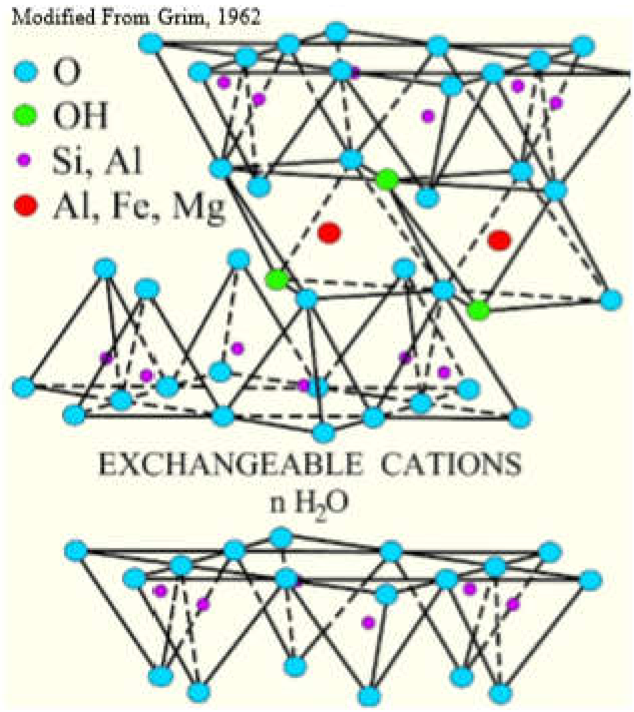


Figure 2.5 Diagrammatic Skectch of the Montmorillonite (source: USGS, 2001)

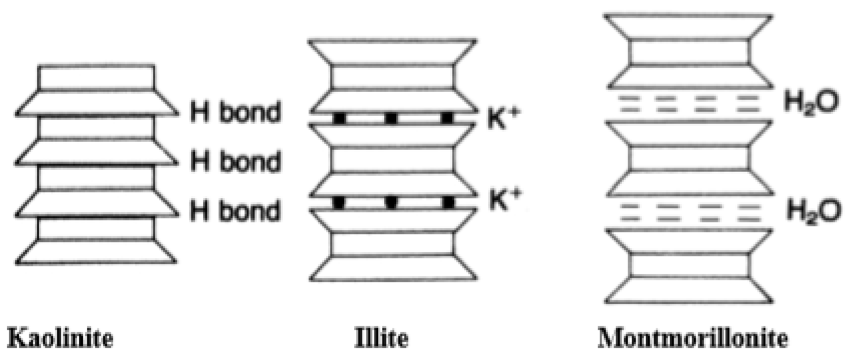


Figure 2.6 Schematic Representations of Clay Minerals (source: Craig, 2004)

2.2 Mechanism of Swelling

Swelling of clay minerals is directly related with diffused double layer and cation exchange capacity of them.

2.2.1 Double Layer of Clay Minerals

The negatively charged clay particle surface and the concentration of positive ions in solution adjacent to the particle form what is referred to as a diffuse double layer or DDL (Bohn et al. 1985). Overlapping DDL between clay particles generate interparticle repulsive forces or microscale “swelling pressures”. Interaction of the DDL and, hence, swelling potential, increases as the thickness of the DDL increases (Mitchell, 1976). The thickness of DDL is associated with valence of cations, concentration of cations, temperature and pH.

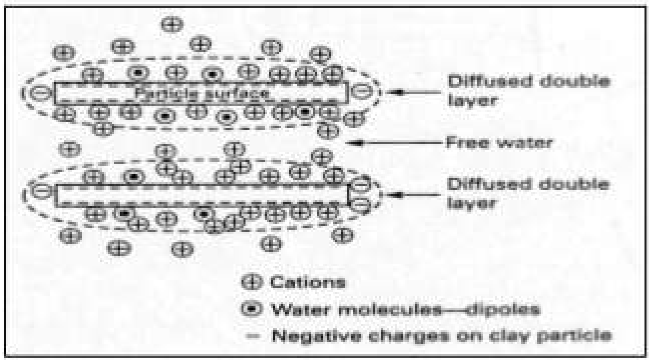


Figure 2.7 Double Layer of Clay Minerals

a) Effect of valence of cations: The lower valences of cations results in increase in DDL thickness. Thus, for the same soil mineralogy, more swelling would occur in a sample having exchangeable sodium (Na^{+}) cations than in a sample with calcium (Ca^{2+} or magnesium Mg^{2+}) cations (Zumrawi et al., 2017).
b) Effect of concentration of cations: The high concentration of cations near the surface of clay particle creates a repulsive force between the diffuse double layer system (Chen, 1975). In general, a thicker DDL and greater swelling are associated with lower cation concentrations (Mitchell, 1976).
c) c) Effect of temperature: An increase in temperature cause an increase in DDL thickness, thus temperature change has effect on strength, compressibility and swelling of soils (Mitchell and Soga, 2005).
d) d) Effect of pH: Hydroxyls (OH)- are exposed on the surfaces and edges of clay particles. The tendency for hydroxyls to dissociate in water, “SiOH Æ SiO + H” is strongly influenced by pH. The higher pH, the greater is the tendency for H to go into solution, and the greater the effective negative charge of the particle. Alumina, exposed at the edges of clay particles, is amphoteric (capable of functioning either as an acid or a base), and it ionizes positively at low pH and negatively at high pH. As a result, positive diffuse layers can develop at the edges of some clay particles in an acid environment which

promotes a positive edge to negative surface interaction, often leading to flocculation from suspension (Mitchell and Soga, 2005).

e) **2.2.2 Cation Exchange Capacity (CEC)**

Cations that neutralize the net negative charge on the surface of soil particles in water are readily exchangeable with other cations. The exchange reaction depends mainly on the relative concentrations of cations in the water and also on the electrovalence of cations (Terzaghi, Peck and Mesri, 1996). The cation exchange capacity is the quantity of exchangeable cations required to balance the negative charge on the surface of the clay particles. CEC is expressed in milliequivalents per 100 grams of dry clay (Zumrawi et al., 2017).

Table 2.1 CEC of Principle Clay Minerals (modified from Terzaghi, Peck and Mesri, 1996)

MINERAL	CEC (meq/100g)
Kaolinite	3 – 10
Illite	20 – 30
Montmorillonite	80 – 120

2.2.2 Stages of swelling

The swelling phenomenon has two basic mechanisms: (Popescu, 1986).

1) Interparticle or intercrystalline swelling, effective for all kinds of clay minerals (Fig. 2.8). In a nearly dry clay deposit relict water holds the particles together under tension from capillary forces. On wetting, the capillary tensions are relaxed and the clay expands.

2) Intracrystalline swelling is chiefly a characteristic of the montmorillonite group of minerals. The layers that make up the individual single crystals of montmorillonite are weakly bonded, mainly by water in combination with exchangeable cations. On wetting, water enters not only between the single crystals, but also between the individual layers that make up the crystals (Fig. 2.8).

2.3 Factors Influencing Swelling

The swell potential of a clayey soil may be affected by either the soil properties influencing the nature of the internal force field, the environmental factors those may change the internal force system or the state of stress present on the soil.

2.4 Oedometer Methods to Determine Swell Properties

The most satisfactory and convenient method of determining the swelling properties of an expansive clay is by direct measurement. Direct measurement of expansive soils can be achieved by the use of the conventional one-dimensional consolidometer (Chen, 1975).

According to ASTM D4546 - 03 (Standard Test Methods for OneDimensional Swell or Settlement Potential of Cohesive Soils), test methods for swell properties can be grouped into three; Method A, Method B and Method C. Initially, the terminology of experiments is presented.

Swell, L = Increase in elevation or dilation of soil column following absorption of water. Free swell, % = Percent heave, h h 100 xΔ following absorption of water at the seating pressure. Primary swell, L = An arbitrary short-term swell usually characterized as being completed at the intersection of the tangent of reverse curvature to the curve of a dimensional change-logarithm of time plot with the tangent to the straight line portion representing long-term or secondary swell (Fig. 2.9). Secondary swell, L = An arbitrary long-term swell usually characterized as the linear portion of a one dimensional change-logarithm of time plot following completion of short-term or primary swell (Fig. 2.9). Swell Pressure, kPa: A pressure preventing the specimen from swelling.

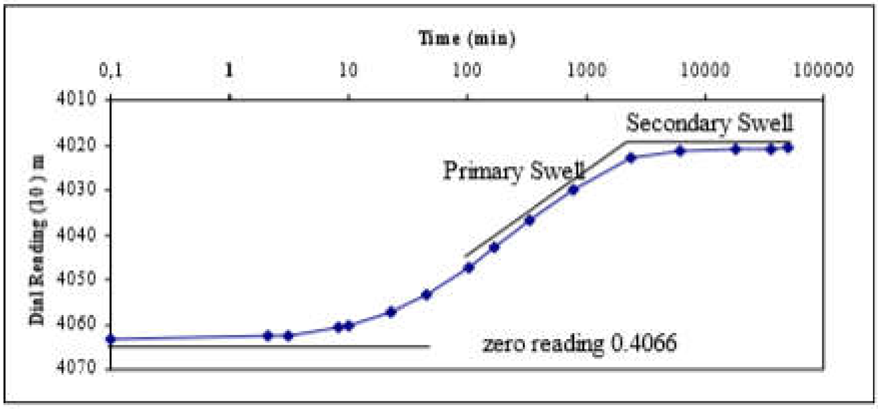


Figure 2.8 Time - Swell Curve (modified from ASTM, 1999)

2.4.1 Method A

After taking initial deformation readings, the seating pressure is applied on specimen and the specimen inundated to swell vertically. While swelling of specimen, deformations are recorded at 0.1, 0.2, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, and 30.0 min and 1, 2, 4, 8, 24, 48, and 72 hours (Fig. 2.9). After primary swell is complete (Fig.2.10), a vertical pressure of approximately 5, 10, 20, 40, 80, etc., kPa is applied till the specimen is recompressed to its initial void ratio and original height complete (Fig.2.10).

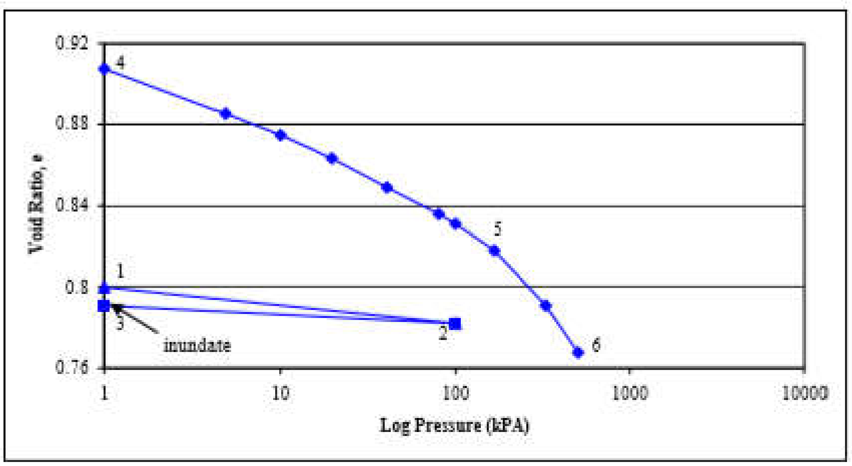


Figure 2.9 Void Ratio - Log Pressure Curve for Method A (modified from ASTM, 1999)

Method A may be modified to place an initial vertical stress, σ1, on the specimen equivalent to the estimated vertical pressure on the in situ soil within 5 min of placing the seating pressure and securing the zero deformation reading. Then, the deformation is read within 5 min and the vertical stress is removed, except for the seating pressure. The deformation is recorded within 5 min after removal of σ1, the specimen is inundated, and the test continues as in stated above (Fig.2.10). Method A measures (a) the free swell, (b) percent heave for vertical confining pressures up to the swell pressure, and (c) the swell pressure.

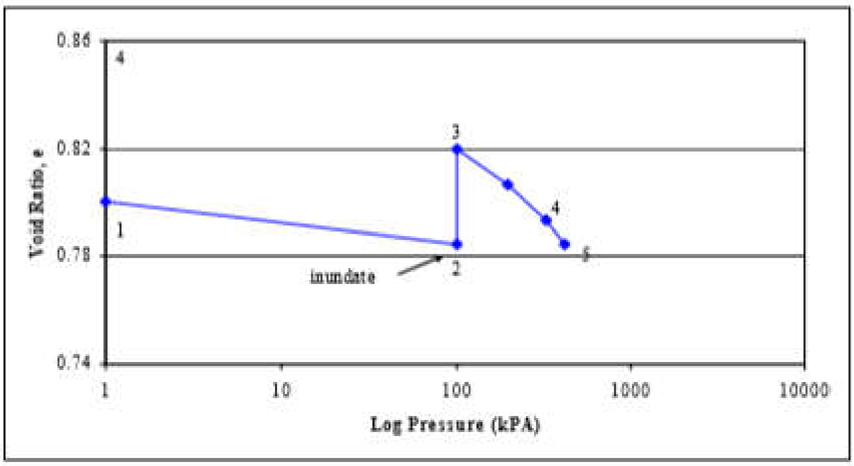


Figure 2.10 Void Ratio – Log Pressure Curve for Method B (modified from ASTM, 1999)

2.4.2 Method B

After applying a vertical pressure exceeding the seating pressure within 5 min of placing the seating pressure, the deformation is read within 5 min of placing the vertical pressure (Fig. 2.11, step 1-2). The specimen is inundated immediately after the deformation is read (Fig. 2.11, step 2) and deformation is recorded after elapsed times similar to Method A until primary swell is complete (Fig. 2.11, step 2-3). After primary swell is complete, vertical pressures of are applied as stated in Method A. Method B measures (a) the percent heave or settlement for vertical pressure usually equivalent to the estimated in situ vertical overburden and other vertical pressure up to the swell pressure, and (b) the swell pressure.

2.4.3 Method C

An initial stress, σ1, is applied, equivalent to the estimated vertical in situ pressure or swell pressure within 5 min after placement of the seating pressure. The deformation within 5 min is read after placing σ1 (Fig. 2.12, step 1), and immediately the specimen is inundated with water (Fig. 2.12, step 2). Increments of vertical stress as needed to prevent swell is applied and final load is recorded (Fig. 2.12, step 3). The specimen is loaded vertically as in Method A (Fig. 2.12, step 4-7). The rebound curve following consolidation is then, determined (Fig. 2.10, after step 7). Method C measures (a) the swell pressure, (b) preconsolidation pressure, and (c) percent heave or settlement within the range of applied vertical pressures.

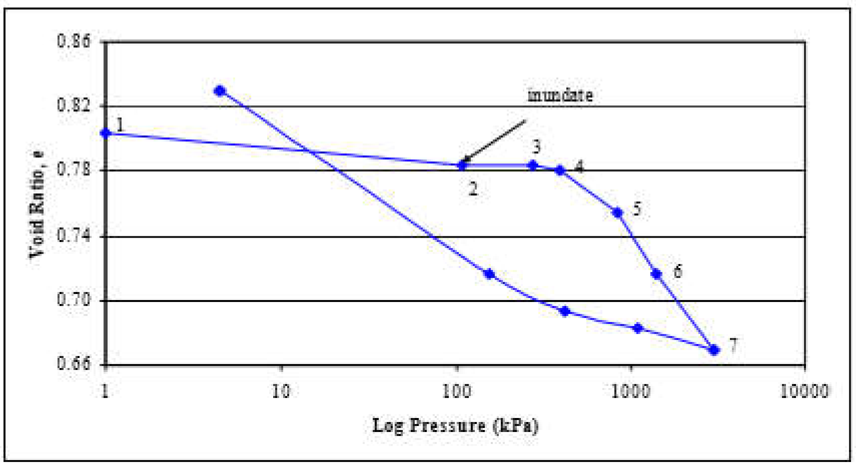


Figure 2.11 Void Ratio - Log Pressure Curve for Method C (modified from ASTM, 1999)

2.5 Expansive soil treatment options

To take care of the various effects of expansive soils, there is need for treatment. The various treatment options, which range from a simple primitive solution to a costly sophisticated one. Treatment of expansive soils before and after construction of structures and highways are listed below (based on researches conducted by Zumrawi et al., 2017; Bullen, 2002; Day, 2006) These include:

- Chemical additives Pre-wetting
- Soil replacement with compaction control
- Moisture control Surcharge loading

- Lateral confinement Thermal methods
- Deep foundation systems

Generally, the selection of treatment option (s) is not a straight forward procedure. The choice of an appropriate method depends on the preliminary site investigation and evaluation of the soil properties (Zumrawi et al., 2017). The options available to engineers in remote arid areas are most likely to be limited to minimization of subgrade moisture changes or to stabilisation of the subgrade. Replacing expansive foundation soils with non-expansive soils such as sand is a simple and easy solution to eradicate expansive soil problems. However, these tasks are expensive and time consuming (Puppala and Musenda, 2000). Hence, the material modification with various chemical additives like lime, cement and bituminous material are commonly performed on expansive soils because the admixers control the potential of soils for a change in volume.

2.5.1 Lime stabilisation

Lime stabilisation is one of the oldest chemical stabilisation methods which is well recognized and widely practiced for structural improvement of many types of soils and aggregates. The addition of lime neutralizes the electrical imbalance in the soil particles with appropriate ions. This in turn reduces the plasticity of soil and increases its workability. In addition, the compressive strength and load bearing properties are also improved (Bhattacharja et al., 2003). Currently, lime stabilisation has been used in highways, railroads, and airport construction projects to improve roadbeds and pavement supporting layers. It is also used to improve the properties of the soft soils and dredged soils (Bergado et al., 1994). Examples of the specific applications of lime stabilisation are construction embankments, soil improvements under foundation slabs and lime columns (Brandl, 1981).

Amongst the different types of lime that can be used for soil stabilisation, the slaked or hydrated lime and quick lime, are the most commonly used varieties. Slaked lime (Calcium hydroxide) and quick lime (Calcium oxide) react with soils to form cementitions compounds like calcium silicates, which increase soil strength and durability (Biswal et al., 2018).

However, lime stabilisation methods have some limitations. For example, lime stabilisation methods are affected by leaching. It was observed from field studies that swelling and plasticity index reverted almost to those of natural untreated soils (Bhattacharja et al., 2003). They are not appropriate in project sites where significant strength improvements are essential or where granular deposits are encountered. This is due to the fact that lime itself has neither appreciable friction nor cohesion. Thus, after certain percentage of lime addition, a strength gain is not found in the soils. Similarly, granular soils have no cations for the replacement by lime ions (Biswal, 2018). Lime also induces distress problems in lime treated sulphate-rich soils because of ettringite formations and creating a "roller coaster" effect in roads and structures (Kota et al., 1996).

2.5.2 Cement Stabilisation

Cement is generally the best type of admixture for effectively stabilizing a wide variety of soils, including granular materials, silts and clays. Ordinary Portland cement is manufactured primarily from calcareous materials such as lime stone and argillaceous materials like clay and shale. The raw material used in the manufacture of OPC consists mainly of lime, alumina, silica, and iron oxide.

Stabilisation with cement for soft soils or expansive soils involves mixing of such soils with cement and water, and subsequent compaction of this mix to a high density makes the material more resistant to variations in physical, thermal and chemical stresses (Winterkorn and Pamukcu, 1991). The addition of cement reduces the plasticity and increases the shear strength of the soil (Craig, 2004), which may result in the reduction of potential volume change of expansive soils (Chen, 1988). However, cement stabilisation has a number of limitations such as high cost, possible corrosion of the soil environment, brittle failure and low temperature cracking as result of the hydration and moisture loss, and proneness to sulphates attack (Punthutaecha, 2002).

2.5.3 Stabilisation with bituminous materials

In suitable conditions, soil may be stabilized by the addition of bituminous materials, such as asphaltic bitumen, 'cut-back' bitumen and bitumen emulsions. The bitumen seals the pores of the soil, reducing the permeability, and may also increase the shear strength considerably by binding the particles together. The extend of the latter effect depends mainly on the quality of the stabilizing agent used, and the nature of the soil. The principal advantage of this method of stabilisation is that the immediate effect in saturated soils is to reduce the shear strength because of the increase in fluid content. For this reason, the method has not been extensively used

2.6 Other innovative methods

One method of controlling swell-shrink potential in expansive soils is to stabilize them with admixtures that prevent volume changes, if possible, or adequately modify the volume change characteristics of expansive soils (Kehew, 1995).

2.7 Soil stabilisation

Soil stabilisation may be defined as any process by which a soil material is improved and made more stable resulting in improved bearing capacity, increase in soil strength, and durability under adverse moisture and stress conditions (Joel and Agbede, 2011).Chemical analyses have shown that many modern industrial wastes are rich in main oxides such as CaO, Al_2O_3, SiO_2, Fe_2O_3, (Kamon et-al, 2000). These oxides are the major constituents that induce improvement in the engineering properties of lime or cement stabilized soils. Some of these waste include fly ash, cement kiln dust, copper slag, red mud, granulated blast furnace slag, glass cullet and quarry dust e.t.c.

2.7.1 Rice Husk Ash

Rice Husk Ash (RHA), a by-product of rice processing, is produced in large quantities globally every year and due to the difficulty involved in disposal, can lead to RHA becoming an environmental hazard in rice producing countries, potentially adding to air and water pollution. RHA is a natural pozzolana, which is a material that when used in conjunction with cement, has cementitions properties. Muntohar (2002), carried out a series of laboratory experiments individually and in combination of RHA and lime in stabilizing expansive soils in Indonesia. He found out that the geotechnical properties of expansive soils improved with addition of RHA and lime. RHA and lime altered the texture of clay soil by reducing the fine particles.

The admixtures also reduce the liquid limit, swelling potential of the expansive soils and also the compressibility characteristics.

Ali et al (2004), carried out an investigation to study the influence of RHA and lime on Atterberg limits, strength, compaction, swell and consolidation properties of bentonite. His results indicated that the plasticity properties of bentonite were significantly modified upon the addition of RHA and lime. The RHA and lime have noticeable influence on compaction, swell and consolidation properties of bentonite soil particularly at 15% RHA and 8% lime contents individually and combined at 15% RHA +4% Lime.

Ramkrishna and Pradeepkumar (2006) had studied combined effects of RHA and cement on engineering properties of black cotton soil, from strength characteristics point of view. The study recommended 8% cement and 10% RHA as optimum dose for stabilisation. Sabat (2012) studied the effects of polypropylene fiber on engineering properties of RHA-lime stabilized expansive soil. He found the optimum proportion of soil: RHA: lime: fiber to be 84.5:10:4:1.5 respectively. Ashango and Patra (2014) studied the static and cyclic properties of clay subgrade stabilized with RHA and Portland slag cement. They found the optimum percent of RHA to be 10% and 7.5% Portland slag cement for stabilisation of expansive soil. They concluded that the stabilized expansive soil was found suitable for subgrade of flexible pavement as, there was significant increase in strength and the stabilized soil was durable.

2.7.2 Bagasse Ash

The Bagasse is the fibrous waste produced after the extraction of the sugar juice from cane mills. Bagasse Ash (BGA) is the residue obtained from the incineration of bagasse in sugar producing factories.

Moses and Osinubi (2013), studied the 'Influence of Compactive Efforts on Cement-Bagasse Ash Treatment on Expansive Black Cotton Soil, they observed an optimum blend of 8% OPC and 4% bagasse ash for treatment of expansive black cotton soil for use as a sub-base material. Sabat (2012), investigated the effects of bagasse ash and lime sludge on OMC, MDD, UCS, soaked CBR and swelling pressure of an expansive soil in order to study its cost effectiveness in strengthening the sub-grade of a flexible pavement in expansive soil areas. He deduced the stabilisation effect to be optimum at 8% bagasse ash and 16% lime sludge.

2.7.3 Fly Ash

Pulverized fuel ash commonly known as fly ash is a useful by-product from thermal power stations using pulverized coal as fuel. The high temperature of burning coal turns the clay minerals present in the coal powder into fused fine particles mainly comprising aluminium silicate. Fly ash produced thus possesses both ceramic and pozzolanic properties. Fly ash by itself has little cementations value, but in the presence of moisture it reacts chemically and forms cementations compounds and attributes to the improvement of strength and compressibility characteristics of soils. It has a long history of use as an engineering material and has been successfully employed in geotechnical applications. There are two major classes of fly ash, Class C and F. The former is produced from anthracite and latter is produced from burning lignite. Both the classes are pozzolans which are defined as siliceous and aluminous materials. Thus expansive soil can be potentially stabilized by cation exchange using fly ash.

Mir (2015) studied the effect of Fly ash on expansive soil and concluded that both high and low calcium fly ash can be recommended as effective stabilizing agents for improvement of the soil. Similarly, Pandian et.al. (2002), studied the effect of two types of fly ashes Raichur fly ash (Class F) and Neyveli fly ash (Class C) on the CBR characteristics of the black cotton soil. He found out that the addition of fly ash to BC soil increases the CBR of the mix up to the first optimum level. Further addition of fly ash beyond the optimum level causes the decrease up to 60% and then up to the second optimum level there is an increase. Thus the variation of CBR of fly ash – black cotton soil mixes can be attributed to the relative contribution of frictional or cohesive resistance from fly ash or black cotton soil respectively.

2.8 Glass Cullet (GC)

With the rapid economic growth and continuously increased consumption, a large amount of waste materials is generated on a daily basis. Waste glass material is an important part of this accumulation of waste. Glass is a non-metallic and inorganic material made by sintering selected raw materials, so it can neither be incinerated nor decomposed (Wu et al*.,* 2003).

Glass is widely used in our lives through manufactured products such as sheet glass, bottles, glassware, and vacuum tubing. Glass is a transparent material produced by melting a mixture of materials such as silica, soda ash, and $CaCO_3$ at high temperature followed by cooling where solidification occurs without crystallization (Gautam et al., 2012).The reuse of Waste Glass in concrete has recently captured attention not only as secondary aggregate, but also as a substitute for Portland cement in concrete (Bignozzi, et al. 2009).

Extensive research funded by Waste and Resources Action Programme (WRAP) has been carried out on waste glass inclusion in Portland cement concrete by Byars, et al. (2004 a and b). The research findings indicated that waste glass could be used as aggregate or as partial Portland cement substitute in concrete. The application of waste glass as finely ground additive in concrete represents a potential option for waste glass recycling.

Perkins (2008) found that glass powder possesses analogous technical characteristics to Portland cement, such as silica and if finely grounded exhibits pozzolanic properties. The authors work confirmed earlier works by Shayan and Xu (2004 and 2006), which deduced that 30 – 70 % of cement in concrete mixtures could be replaced by glass powder without compromising the technical properties of the concrete. Other findings by Pereira de Oliveira et al (2008) indicate that reducing glass particle size enhances pozzolanic reactivity and attacks such as Alkali silica reaction (ASR) which reduces until risk of ASR is totally eliminated.

Eberemu et al. (2012), investigated the stabilizing effects of glass cullet on engineering properties of expansive soil. They observed that with an increase in percentage addition of glass cullet, there was continuous decrease in liquid limit, plastic limit, cohesion, optimum moisture content and swelling pressure. This study also indicated continuous increase in plasticity index, specific gravity, maximum dry density, Unconfined Compressive Strength, angle of internal friction, hydraulic conductivity and California Bearing Ratio.

Nebojša et al (2012), studied waste glass as additive to clayey material in subgrade and embankment of road pavement, and concluded that the addition of waste glass in clay material does not have a negative impact on the geo-mechanical properties of such mixture.

In view of the above, this study proposes to stabilize expansive soil using cement Glass Cullet admixture.

CHAPTER THREE

METHODOLOGY

3.1 Preamble

This chapter describes the methods used in data collection, conduct of experiments, location of study area, sampling, preparation and testing of specimen.

3.1.1 Black cotton soil

The soil used for this study is black cotton soil (dark grey in colour) obtained from Baure village near Dadin-kowa in Yamaltu-Deba Local Government Area of Gombe state. The location lies approximately on latitude 10^0 16'N and longitude11^0 21'E. In terms of extent of deposit, black cotton clays are not restricted to the area of study, but are widely spread throughout the North–eastern part of Nigeria.

A disturbed sample of the clay which was dug after removing the top soil was collected for laboratory analysis. Some amount of the sample was sealed in a polythene bag for determination of natural moisture content. In the laboratory, the soil was air dried, pulverized and sieved with British Standard Sieve No. 4 which is of 4.75 mm aperture as required for the tests.

3.1.2 Cement

Ordinary Portland cement used throughout the research was obtained from the open market and its, properties conform to specifications of BS 12: Part 2: Clause 5 (1971).

3.1.3 Waste glass

Broken waste glass bottles were obtained from post-consumer waste. The glass was cleaned and crushed into smaller sizes using compressive machine. The Glass Cullet was then finely ground with a grinding machine to achieve a finer particle size, and an average particle size of < 300 μm was used. This size range was chosen based on previous findings from researchers (e.g. Dyer and Dhir, 2001; Shayan and Xu, 2004; Nwaubani, 2013).

3.2 Preparation of specimens

Twenty four batches of the soil with stabilizer and admixture were prepared for each test conducted. Specifically, these mixes consist of 2%, 4%, 6%, and 8% OPC and 5%, 10%, 15% and 20% GC admixture, at each percentage increment of OPC, 5%, 10%, 15% and 20% of GC was added to the blend.

Tests were first performed on compacted soil specimens without OPC or GC admixture in order to evaluate the engineering properties of the soil. For the stabilized material, tests were conducted on samples containing varying amounts of OPC and GC admixture in order to evaluate the changes imparted to the soil at each concentration of OPC/GC blend. The summary of experimental mix proportion is presented in Table 3.1

3.3 Laboratory tests

3.3.1 *Mineralogical tests*

3.3.1.1 X-ray diffraction analysis

X-ray diffraction analysis was carried out to investigate the mineralogical properties of the expansive soil. The prepared sample was placed into the sample holder and pressed down with a powder press block. The pressed powder inside the sample holder was loaded into the XRD spectrometer to determine the clay minerals in the sample. The test was conducted using an Empyrean X-ray diffractometer at the National Geosciences Laboratories, Kaduna.

3.3.1.2 X-ray fluorescence analysis

X-ray florescence (XRF) is widely used to measure the elemental composition of materials; XRF is the emission of characteristic. "Secondary" (or florescence) x-rays from a material that has been excited by bombarding with high-energy x-rays or gamma rays. The phenomenon is widely used for elemental analysis and chemical analysis, particularly in the investigation of metals, glass, ceramics and building materials etc. The x-ray florescence of the soil was determined using Philips PW 1606 X-ray florescence spectrometer at the National Geosciences Laboratories, Kaduna.

Table 3.1: Summary of Mix proportion by dry weight of soil

	Mix proportion by dry weight of soil		
Sample Number	Soil	OPC	GC
1	100	0	0
2	100	2	0
3	100	4	0
4	100	6	0
5	100	8	0
6	100	0	5
7	100	0	10
8	100	0	15
9	100	0	20
10	100	2	5
11	100	2	10
12	100	2	15
13	100	2	20
14	100	4	5
15	100	4	10
16	100	4	15
17	100	4	20
18	100	6	5
19	100	6	10
20	100	6	15
21	100	6	20
22	100	8	5
23	100	8	10
24	100	8	15
25	100	8	20

3.3.2 Index Properties

3.3.2.1 Natural moisture content

The natural moisture content of the soil was determined in accordance with BS 1377: Part 2: (1990). An appropriate weight of the wet soil was placed in a container of known mass, M_1. A sample of the natural soil was added to the container, weighed and recorded as M_2. They were then in an oven and allowed to dry at a temperature of 105°C to 110°C for a period of 24 hours. After drying, the container plus the dry soil were weighed again and recorded as M_3. All measurements were made to the nearest 0.01g. The moisture content was then calculated using the relationship in equation 3.1

$$Moisture\ content\ (w) = \frac{M_2 - M_3}{M_3 - M_1} \times 100\ \% \qquad (3.1)$$

This procedure was repeated three times, from which average natural moisture content was determined.

3.3.2.2 Particle size distribution

The sieve analyses performed for the soil sample was in accordance with BS 1377 (1990) and Head (1992), in order to determine the particle distribution of the soil particles. The soil sample weighing 200 g was soaked overnight for the wet sieving. The sample was washed thoroughly through the No. 200 sieve (75 µm aperture). Particles retained on the sieve for the soil was placed in the oven and dried at 105°C before dry sieving was done to obtain the particle size distribution. The filtrate passing through the afore-mentioned sieve size was subjected to hydrometer analysis with sodium hexametaphosphate as the dispersant. It was transferred into a 1 litre measuring cylinder, mixed with a little solution of Sodium hexa-metaposphate and allowed to settle after shaking thoroughly. Hydrometer satisfying requirements of BS 718 was then dipped into the liquid periodically to observe the rate of settlement, at the same time recording the temperature. The result was used to compute the particle sizes and percentages by mass as outlined in the code. The two results, namely that of sieving and that of sedimentation were combined to plot the particle size distribution curve.

3.3.2.3 Specific gravity

The specific gravity test was conducted for the natural soil in accordance with BS 1377: Part 2: (1990). About 5-10g of the soil sample passing 2 mm BS sieve size was oven dried at 105- 110^{o}C. The sample was allowed to cool in a dessicator and then placed in a 50ml density bottle with the stopper attached. The weight of the soil, bottle and stopper were measured as W_2. Distilled water was then added to the bottle to cover the sample; care was taken not to trap air in the bottle. The bottle, stopper plus the soil and water were also weighed as W_3. The bottle was emptied and filled with distilled water alone, again the weight of the bottle, stopper and water were recorded as W_4. The entire procedure was repeated three times from which an average value of the specific gravity was calculated. The procedure was repeated for each concentration of OPC/WG blend. The specific gravity of the soil particle is calculated using the expression in equation 3.2

$$Specific\ gravity\ (G_s) = \frac{(W_2 - W_1)}{(W_4 - W_1) - (W_3 - W_2)} \tag{3.2}$$

3.3.2.4 Free swell

The test was conducted in accordance with the United States Bureau of Reclamation (USBR 1902) method. About 10 g of soil passing BS No. 4 sieve (425 µm aperture) was oven-dried and allowed to cool down in a desiccator. The sample was slowly poured into a 100 cm^3 measuring cylinder to which water was added in order to fill the cylinder. The cylinder was then agitated in order to obtain a homogeneous mixture of soil and water after which it was allowed to settle for at least 2 hours before the swell volume was recorded.

The procedure was repeated for each concentration of OPC/GC blend. The free swell is calculated using equation 3.3.

$$Free\ swell = \frac{Final\ volume - Initial\ volume}{Initial\ volume} \times 100 \tag{3.3}$$

3.3.2.5 Atterberg limits

The Atterberg limits, comprising Liquid Limit, Plastic Limit, and Plasticity Index are a basic measure of the nature of a fine-grained soil. Depending on the water content, a soil may appear in four states: solid, semi-solid, plastic and liquid. In each state, the consistency and behaviour of a soil is different and so are its engineering properties. Thus, the boundary between each state can be defined based on a change in the soil's behaviour.

The procedure for determination of the liquid limit test of a soil is outlined in BS 1377 (1990). An air dried soil amount of 200g passing 425–μm sieve size is taken and mixed with water and kneaded for 7 minutes to achieve uniformity. The soil paste is placed in the liquid limit cup, and leveled off with the help of a spatula. A clean and sharp groove is cut in the middle by means of a grooving tool. The crank is rotated at about 2 revolutions per second and the number of blows required to make the halves of the soil pat separated by the groove meet for a length of about 12 mm is counted. The number of blows at which the groove closed was counted and recorded after which part of the soil was taken for moisture content determination. The soil moisture was varied by adding soil or water each time repeating the procedure above to obtain well-spaced number of blows. The values of the moisture content determined and the corresponding numbers of blows were then plotted on a semi-log paper and the Liquid Limit was determined as the moisture content corresponding to 25 blows.

The Proportion of the material passing sieve with aperture 425 μm which used for the determination of the liquid limit (LL) is also used for the determination of the plastic limit. A sample of the wet soil is taken and moulded between the palms of the two hands. The sample is rolled and sub-divided into two sub samples which are further subdivided into parts. The rate of rolling is between 80 and 90 strokes per minute, counting a stroke as one complete motion of the hand forward and back to the starting position again. The rolling is done until the threads are of 3 mm diameter as specified by BS 1377 (1990). The soil is kneaded together to a uniform mass and rolled again. This process of alternate rolling and kneading is continued until the thread crumbles under the pressure required for rolling and the soil can no longer be rolled into a thread. The pieces of crumbled soil thread are collected and the moisture content was determined and recorded as the plastic limit. Plasticity index is computed as the difference between the liquid limit and plastic limit.

3.3.3 Engineering properties

3.3.3.1 Compaction tests

3.3.3.1.1 Standard proctor (SP) compaction

The tests were conducted in accordance with BS 1377: Part 4: (1990) and BS 1924: Part 2: (1990) for the natural and stabilized soil samples respectively.

Approximately 3 kg of the air- dried sample that passes the BS No. 4 (425 µm aperture) was thoroughly mixed with 6-8% of water. The soil was then compacted into 3 equal layers. Each layer was given 27 blows with the 2.5 kg rammer falling freely through a height of 300 mm. At the end of compaction, the extension collar was removed and the top of the soil trimmed off by means of a palette knife. The weight of the mould and the compacted soil was determined.

The compacted soil was quickly extruded from the mould and a representative sample was taken for moisture content determination. The soil was then broken up, re-mixed with more water in order to obtain higher moisture content. The entire process was repeated several times until the total mass of soil and mould began to drop. The optimum moisture content (OMC) and maximum dry density (MDD) were determined by plotting a graph of dry density versus moisture content. The procedure was repeated by using requisite levels of OPC/GC blend.

3.3.3.1.2 Modified Proctor (MP) compaction

The compactive effort in case of the Modified Proctor (MP) compaction test consist of the energy derived from a 4.5 kg rammer falling freely through a height of 450 mm onto five equal layers of the soil. Each layer was subjected to 27 blows of the rammer. Apart from the size of the rammer, height of fall and number of blows, the entire operation is similar to that of Standard Proctor (SP) compaction test. equations 3.4 and 3.5 are used to compute the bulk and dry densities respectively.

$$\rho = \frac{M_2 - M_1}{V} \tag{3.4}$$

$$\rho_d = \frac{100\rho}{100 + w} \tag{3.5}$$

Where:

ρ = Bulk density (Mg/m^3)

M_1 = mass of empty mould (g)

M_2 = mass of mould + wet soil (g)

W = moisture content (%)

ρ_d = dry density (Mg/m^3)

The equations (6) and (7) also apply to the SP compaction tests.

3.3.3.2 Unconfined Compressive Strength (UCS)

The unconfined compressive strength (UCS) test entails the determination of the compressive strength of both the natural and stabilized soil samples at the energy levels of the SP and the MP, respectively.

The UCS was conducted in accordance with BS 1377: part 7: (1990). The soil samples were prepared using the optimum moisture contents (OMCs) derived from moisture density relation determined for the natural soil and for each blend of soil OPC/WG. After preparation of the test specimens, compaction was done using the energy levels of the SP and subsequently the MP. After the compaction stage, twelve specimens of diameter 38 mm and length 76 mm were produced at the end of compaction. From that, 3 specimens each were cured for 7, 14, 21 and 28 days while the last batch were cured for 7 days and thereafter they were soaked for 7 days so as to simulate worst case scenario. Before the curing, the specimens were labelled accordingly and then wrapped in cellophane bags.

After the specified days of curing, the prepared sample was placed in a load frame machine driven with strain rate of 0.10 %/min until failure occurred. This test generally involves subjecting a cylindrical specimen of soil to a steadily increasing axial load, which is the only force or stress until failure occurred. The procedure was repeated for each blend of OPC/GC. The procedure is the same for the MP compactive effort. However, the 4.5 kg rammer was employed to compact 5 layers of the sample; specifically 27 blows with rammer were applied to each layer in this case. The unconfined compressive strength (UCS) and strain were calculated using Equations (3.6) and (3.7). For each sample the highest value of the stress was taken to represent the stress of that sample.

$$\sigma = \frac{R \times C_r \times (100 - \varepsilon\%) \times 1000}{100 \times A_o} \tag{3.6}$$

$$\varepsilon = \frac{\Delta l}{l_o} \tag{3.7}$$

Where:

ε = Strain (%)

Δl = Amount of compression at any stage (mm)

R = Load ring reading at strain ε

C_r = Mean calibration of load ring

l_o = Initial length of specimen (mm)

A_o = Initial cross-sectional area (mm^2)

σ = Compressive stress at strain, ε

3.3.3.3 California Bearing Ratio (CBR)

The CBR test was conducted in accordance with BS1377: Part 4: (1990) for natural soil and BS1924 (1990) for the stabilized soil. The aim of the CBR test is to determine the relationship between the force being applied and the resultant penetration. For the SP compaction, about 5 kg of the pulverized soil sample was mixed with the required OPC/GC blend at OMC. The mixture was then compacted in 3 layers in the CBR mould, 62 blows of the 2.5 kg rammer was applied to each layer. The extension collar was removed and the top of the compacted sample trimmed carefully and waxed.

In testing the specimens, the mould containing the compacted soil with the base plate in place were positioned on the lower plate of the machine. The plunger was then made to penetrate the specimen until it failed. The mould was then inverted, base plate removed and the procedure repeated for the base of the specimen. The value of the force at each 0.25 mm interval was recorded until failure of the specimen.

From the values of penetration and force recorded, a curve of force against penetration was thus obtained. Correction was made where the curve concave upward by drawing a tangent in all cases, the point where the tangent cuts the penetration axis serve as the new origin. The CBR value was calculated at 2.5 mm and 5.0mm penetration, the greater of the two values was taken as the CBR of the soil for the blend (OPC/GC) in question. However, where the values were within 10% of each other, the mean value of the two readings was considered, otherwise the higher value was recorded as the CBR of the specimen. The standard load for 2.5mm and 5.0mm are 13.24KN and 19.96KN respectively.

The California Bearing Ratio (CBR) was calculated using Equation (3.8).

$$CBR = \frac{Measured\ load(kN)}{Standard\ load\ (kN)} \times 100\% \qquad (3.8)$$

The same procedure was carried out for the MP compaction tests, but, 4.5 kg rammer was employed and the sample was compacted in 5 equal layers. Each layer was subjected to 62 blows of the rammer.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Preamble

This chapter presents the results of various tests carried out during the experimental investigation. Table 4.1 gives the Mineral composition of the natural soil and Glass Cullet, Table 4.2 shows the summary of the index properties of black cotton soil and Table 4.3 shows the summary of the engineering properties of black cotton soil.

Table 4.1: Mineral Compositions of BCS and GC

	Mineral Compositions	
Compounds	**BCS (%)**	**GC (%)**
SiO_2	49.2	69.2
Al_2O_3	15.0	2.29
CaO	9.61	15.1
MnO	0.17	0.02
Fe_2O_3	16.7	1.57
TiO_2	2.17	-
K_2O	1.52	1.10
CuO	-	0.13
Na_2O	-	8.75
MgO	-	0.49
SO_3	0.21	-
Others	5.42	1.35

Table 4.2: Index Properties of Black Cotton soil

Property	Value/Description
Natural moisture content (%)	9.76
Liquid limit (%)	51.4
Plastic limit (%)	32.6
Plasticity index (%)	18.8
Linear shrinkage (%)	14.3
Free swell (%)	68.3
Specific gravity (%)	2.66
Percentage passing No. 200 sieve	79.1
Percentage sand fraction (%)	20.9
Percentage silt fraction (%)	15.1
Percentage clay fraction (%)	64
AASHTO classification	A-7-5(14)
Group Index	14
Colour	Dark grey
USCS classification	CH

Preliminary tests were conducted for the identification and determination of the soil properties of the natural soil (i.e., the addition of GC/OPC). The test results for the natural soil are summarised in Table 4.2. Physical inspection showed that the soil is dark grey in colour. From the particle size distribution curve shown in Figure 4.1 the soil is well graded, containing 20.9% sand fraction, 15.1% silt fraction and 64 % clay fraction. The soil has activity value of 3.06 which indicate that the soil is active. The high activity value suggests montmorillonate clay mineral to be dominant clay mineral in the soil sample.

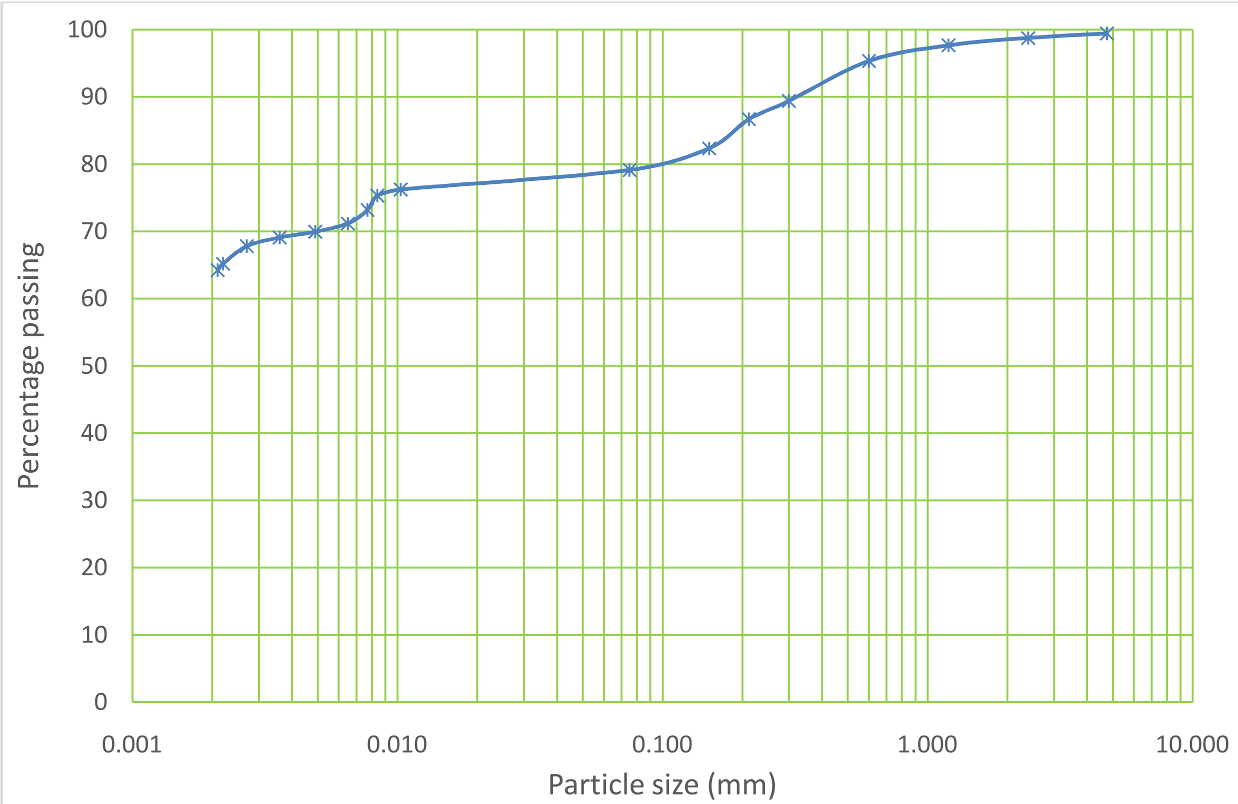


Figure: 4.1 Particle Size Distribution Curve

The result obtain from atterberg limit and sieve analysis test classified the soil as A-7-5(14) and CH using the American Association of State Highway and Transportation Officials (AASHTO,1986) soil classification system and Unified soil Classification System (ASTM,1992), respectively. The soil classifications showed that the soil is clayey with high plasticity.

The liquid limit and plasticity index values of 51.4% and 18.8% confirmed that the soil is highly plastic. Atterberg limits results have been reportedly shown to be very useful indicators of soil behaviour (Jefferson and Rogers, 1998).

Using these parameters the soil falls below the standard recommended for most engineering works. The findings reported by Thakur et al. (2016) as well as Osinubi and Katte (1997) support this view.

The specific gravity of the soil was observed to be 2.66 which indicated the prevalence of montmorillonite, in conformity with Das (2005) which gave a specific gravity range of 2.65 – 2.80.

Table 4.3: Engineering Properties of Black Cotton soil

Property	Value/Description
Maximum Dry Density (MDD) BSL (Mg/m^3)	1.46
Maximum Dry Density (MDD) BSH (Mg/m^3)	1.53
Optimum moisture content (OMC) BSL (%)	15
Optimum moisture content (OMC) BSH (%)	14
Unconfined compressive strength using BSL (kN/m^2)	80
Unconfined compressive strength using BSH (kN/m^2)	184
Soaked California bearing ratio BSL (%)	8.7
Soaked California bearing ratio BSH (%)	10.4
Un-soaked California bearing ratio BSL (%)	9.5
Un-soaked California bearing ratio BSH (%)	11.2

Compaction test results for the stabilized soil at various percentages of OPC/GC are presented in Appendix B1-B5. Appendices B6 and B7 show the maximum dry density and optimum moisture content results. These results are obtained from plots of compaction test data which gave a moisture density relationship in terms of compactive effort. The Unconfined compressive strength and California bearing ratio test results for the stabilized soil using both compactive efforts (BSL and BSH) are presented in Appendices C1, C2 and C3.

The MDD values of 1.46 Mg/m^3, 1.53 Mg/m^3 and OMC values of 15% and 14% for the British Standard Light (BSL) and British Standard Heavy (BSH) compactive energy respectively obtained were in agreement with their findings. Free swell value of 68.29% indicated that the soil is of medium plasticity, physical inspection showed that the soil is dark grey in color. From the particle size distribution curve shown in Figure 4.1, the soil is well graded, containing 20.9% sand fraction, 15.1% silt fraction and 64% clay fraction. The UCS values of 80kN/m^2 and 184kN/m^2 recorded for the natural soil at OMC for BSL and BSH energy levels are relatively low. Similarly, low Un-soaked CBR values of 9.5% and 11.2% obtained at the energy levels of BSL and BSH compactive efforts, respectively further revealed that the material is not suitable for use as a construction material. This is in agreement with Nigerian General Specification (1997) and Osinubi (1999).

4.2 Mineralogical tests

4.2.1 X-ray diffraction of soil

The results obtained from the x-ray diffraction analysis of the clay mineralogy revealed that montmorillonite, kaolinite, Illite, Albite with some mixtures of Quartz were the primary clay minerals with montmorillonite been the predominant mineral as shown in Figure 4.3.

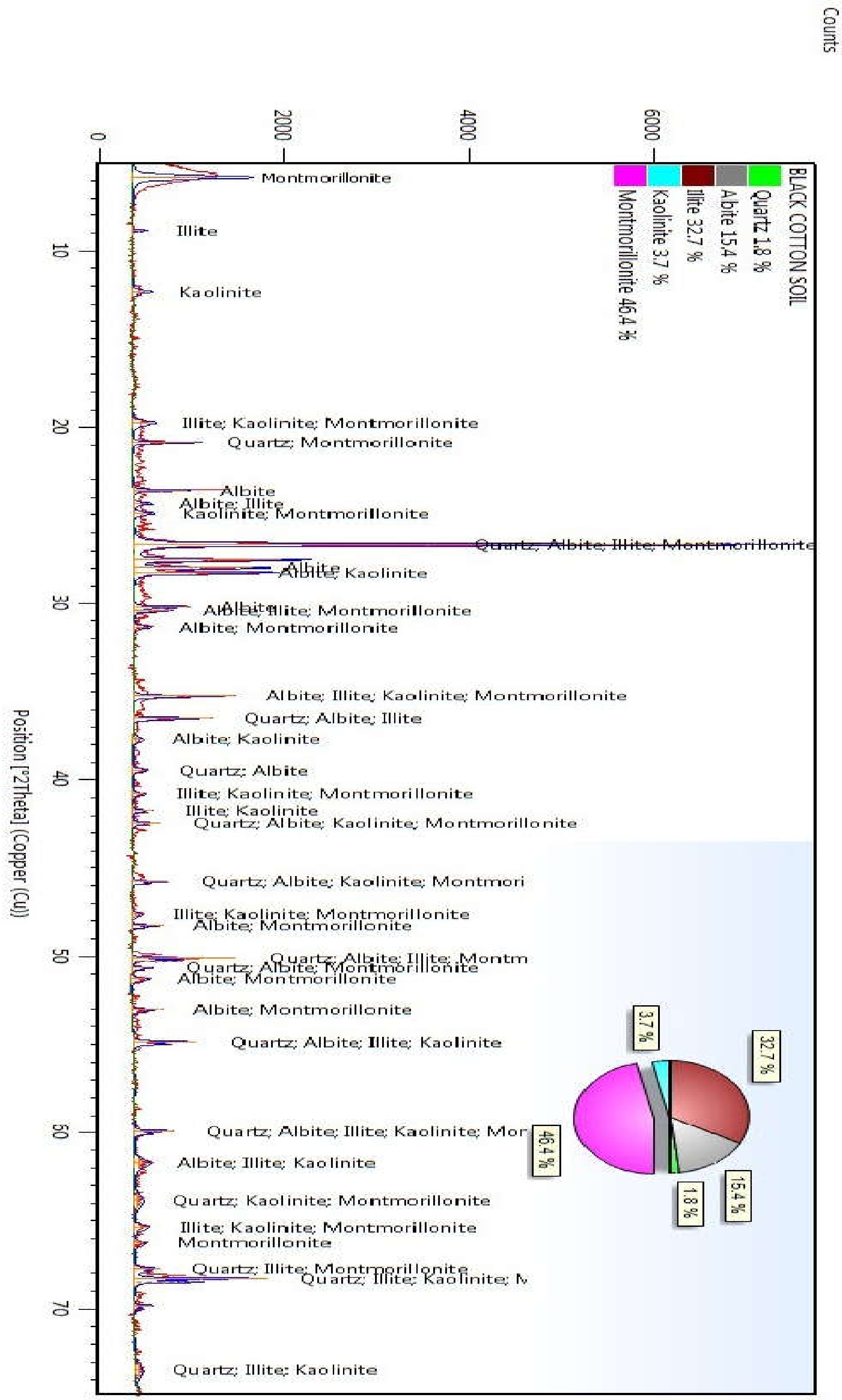


Figure 4.2 X-ray diffraction of Natural Black Cotton Soil

4.2.2 X-ray fluorescence of soil and GC

4.2.2.1 Soil

The oxide composition of the natural soil determined using the XRF spectrometer revealed that the major components of the soil are Silicon dioxide (SiO_2), Aluminium Oxide (Al_2O_3), Iron Oxide (Fe_2O_3) and Sodium Oxide (Na_2O) contributing 31.01%, 16.19% and 4.74%.

4.2.2.2 Waste glass

The major chemical compositions of Glass Cullet (GC) are Silicon Oxide (SiO_2), Aluminium Oxide (Al_2O_3), Iron Oxide (Fe_2O_3), Calcium Oxide (CaO) and Sodium Oxide (Na_2O) contributing 69.2%, 2.29%, 1.57%, 15.1%, and 8.75% of the total, as shown in Table 4.3. According to ASTM standard specification (C 618 - 2012) the sum of S_iO_2, Al_2O_3 and Fe_2O_3 should exceed 70% for any material to be used as Class C pozzolana.

4.3 The Effect of Glass Cullet on Engineering Properties of Black Cotton Soil

4.3.1 Atterberg limits

The results of the Atterberg Limit shows that the Liquid Limit decreased from 51.4% (0%GC) to 42.0% (20%GC), Plastic Limit increased from 32.6% (0%GC) to 35.2% (20%GC), Plasticity Index decrease from 18.8% (0%GC) to 6.8% (20%GC) and Linear Shrinkage decrease from 14.3% (0%GC) to 10.7% (20%GC). Generally the liquid limit decreased with increase in GC content; and this could be attributed to GC being a pozzolana aided flocculation and aggregation of the clay particles. Therefore it increases the effective grain size due to agglomeration of the clay particles. This improvement in the plastic limit and linear shrinkage of the stabilized soil for each of the blend is due to the GC reactivity with the clay fraction of the soil, which resulted in an increase in the total surface area of the soil.

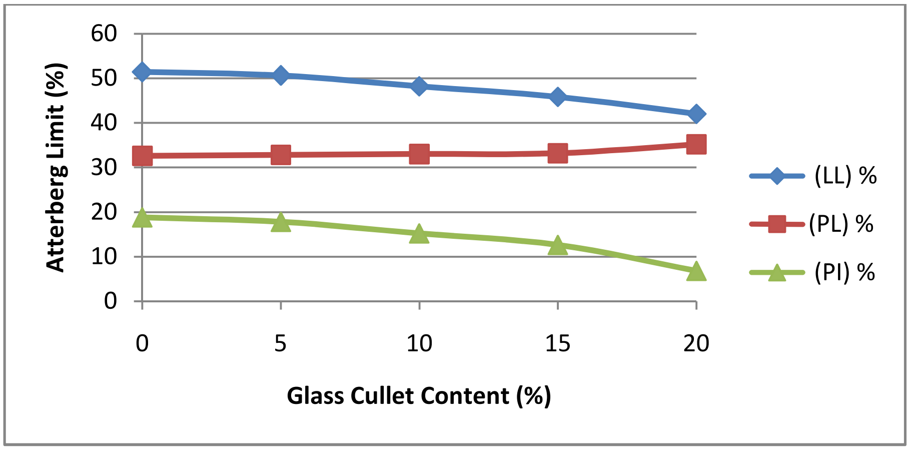


Figure 4.3 Variation of Atterberg Limit of Black Cotton Soil with Glass Cullet content

4.3.2 Compaction Characteristics

The results of the Compaction Characteristics for the British Standard Light (BSL) and British Standard Heavy (BSH) Compactive Efforts shows that the Maximum Dry Density (MDD) increase from 1.46Mg/m^3 and 1.53Mg/m^3 at 0%GC to 1.51Mg/m^3 and 1.61Mg/m^3 at 20%GC respectively, while the Optimum Moisture Content (OMC) decrease from 15% and 14% at 0%GC to 13.02% and 11.22% at 20%GC respectively. The increase in MDD recorded for all the two compactive efforts are attributable to flocculation and agglomeration of the clay particles due to ion exchange, decreases in OMC was as a result of self-desiccation where by all the available water molecules where use up in the hydration reaction with subsequent lower hydration and an incomplete hydration that affected the OMC's.

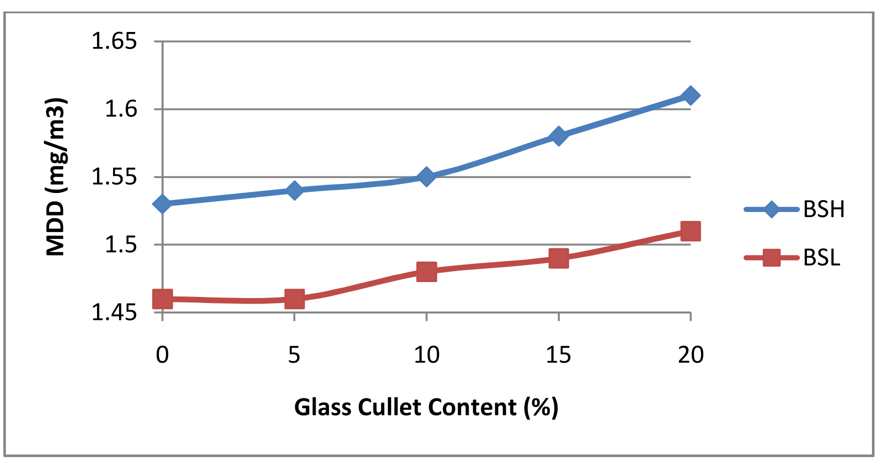


Figure 4.4 Variation of Maximum Dry Density of Black Cotton Soil with Glass Cullet content

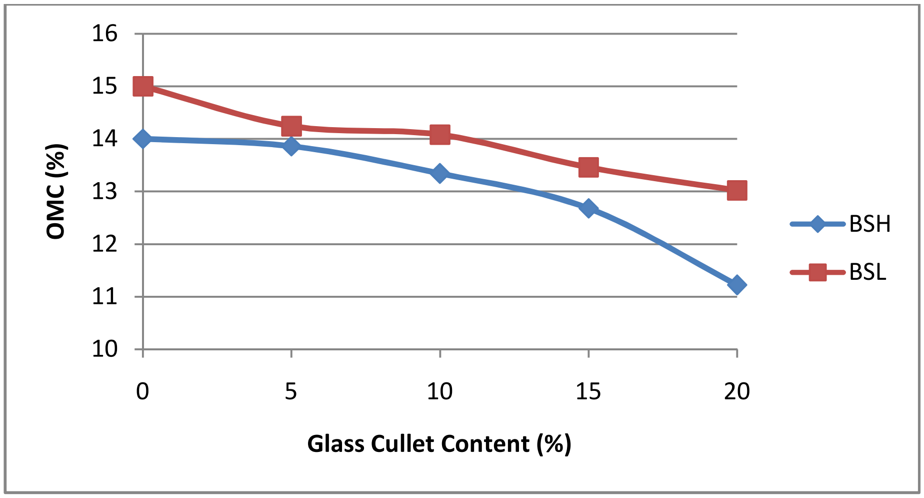


Figure 4.5 Variation of Optimum Moisture Content of Black Cotton Soil with Glass Cullet content

4.3.3 Free swell

The free swell was observed to decrease significantly for each of the test series conducted when compared with the result obtained for the un-stabilized soil it decrease from 64.3N/mm^2 (0%GC) to 56.52N/mm^2 (20%GC). It could be inferred that such a decrease in free swell may be due to chemical reaction between the soil and the GC blend which inhibits an immediate absorption of water, attributable to montmorillonite clay mineral.

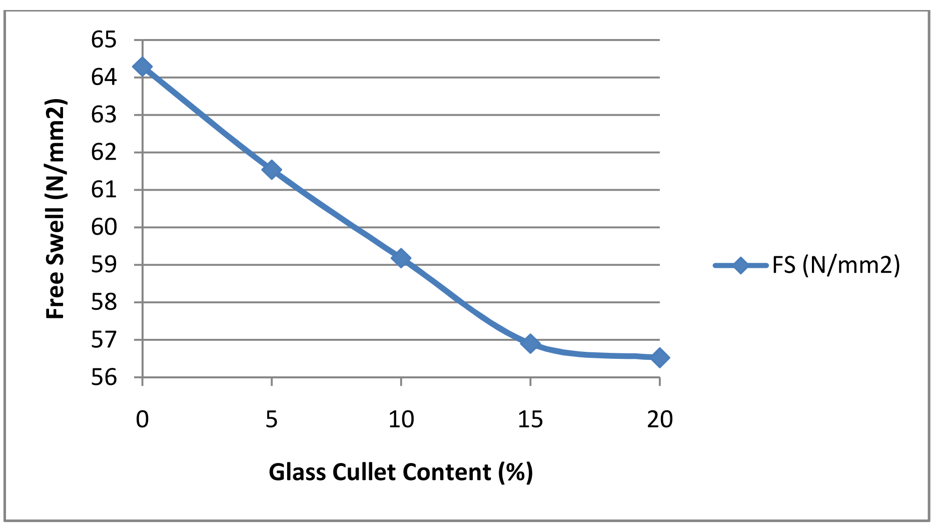


Figure 4.6 Variation of Free Swell of Black Cotton Soil with Glass Cullet content

4.3.4 Unconfined Compressive Strength

The relationship between UCS and GC contents for the two compactive efforts shows a general increase for both BSL and BSH compactive effort, at 7days curing period shows that the UCS increase from 122kN/m^2 and 386kN/m^2 at 0%GC to 138kN/m^2 and 496kN/m^2 at 15%GC and 20%GC respectively, the variation of UCS with GC content for the 14days cured specimens shows an increase in UCS from 186kN/m^2 and 484kN/m^2 at 0%GC to 246kN/m^2 and 534kN/m^2 at 20%GC respectively, also for the 28days curing period the UCS increase from 214kN/m^2 and 544kN/m^2 at 0%GC to 242kN/m^2 and 628kN/m^2 at 20%GC for both BSL and BSH compactive effort respectively. The increase in UCS observed can be attributed to ion exchange at the surface of clay particles as the Ca^{2+} in the glass reacts with the lower valence metallic ions in the clay microstructure which results in agglomeration and flocculation of the clay particles. The reduction in UCS, GC content was due to excess of lower valence cations that could not be neutralized with the available higher valence cations. With the increase in GC content the amount of cations are increased and able to balance the net negative disequilibrium or in other words caused a reduction in the zeta potential of the clay minerals and GC mixture; that resulted in a reduction in the affinity for H^+. This consequently led to the increase in UCS.

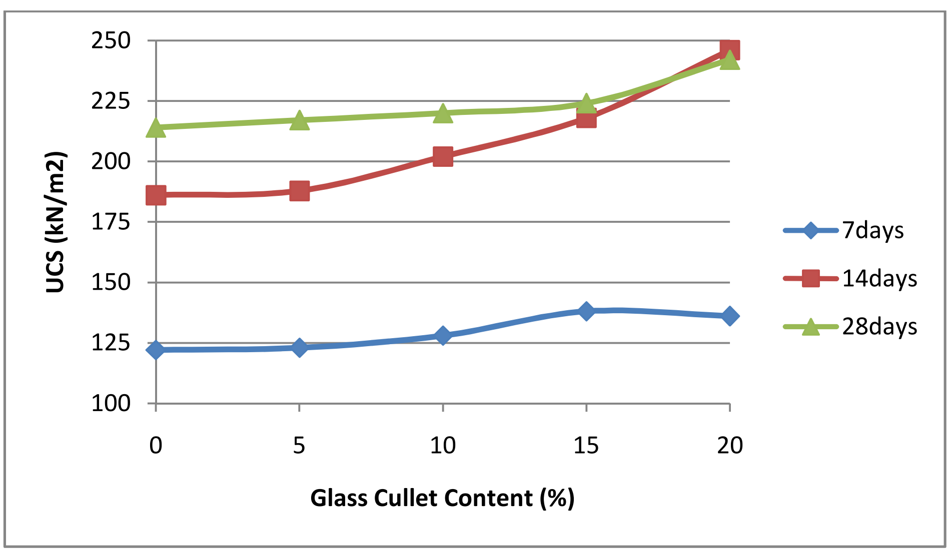


Figure 4.7 Variation of Unconfined Compressive Strength of Black Cotton Soil with Glass Cullet content (BSL)

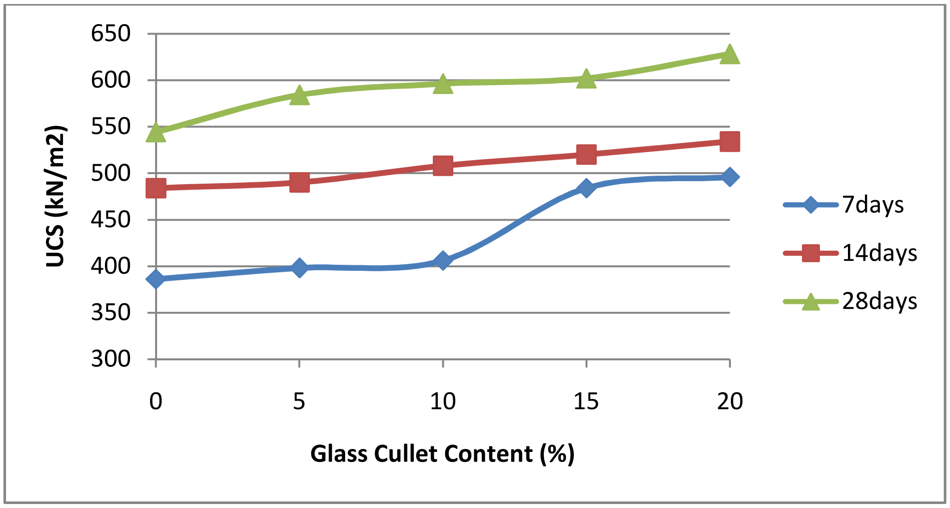


Figure 4.8 Variation of Unconfined Compressive Strength of Black Cotton Soil with Glass Cullet content (BSH)

4.3.4 California Bearing Ratio

The result of CBR shows the variation of soaked and un-soaked CBR with various percentages of GC blends using the BSL and BSH compactive effort. The variation of un-soaked CBR with various percentage of GC blend using the BSH compactive effort showed slightly higher values than for the BSL compactive effort.

The CBR values obtained from the results of GC stabilisation of the black cotton soil for the BSL and BSH compactive efforts increase slightly from 9%, 10% soaked CBR at 0% GC and 10%, 11% un-soaked CBR at 0% to optimum values of 14%, 15% soaked CBR at 20%GC and 13%, 14% un-soaked CBR at 20%GC for both BSL and BSH compactive effort respectively. The Nigerian General Specification (1997) 30% CBR criterion for sub base was not achieved with GC content. Based on these observations, BCS treated with GC alone can be recommended as a sub grade material in road construction with 20%GC content being the optimum.

On a general note, there is a noticeable contribution of the glass cullet in all the tests. The patterns of increase/decrease in results are the same. This shows that both the GC and OPC enhance the soil's properties positively when treated separately or in contribution.

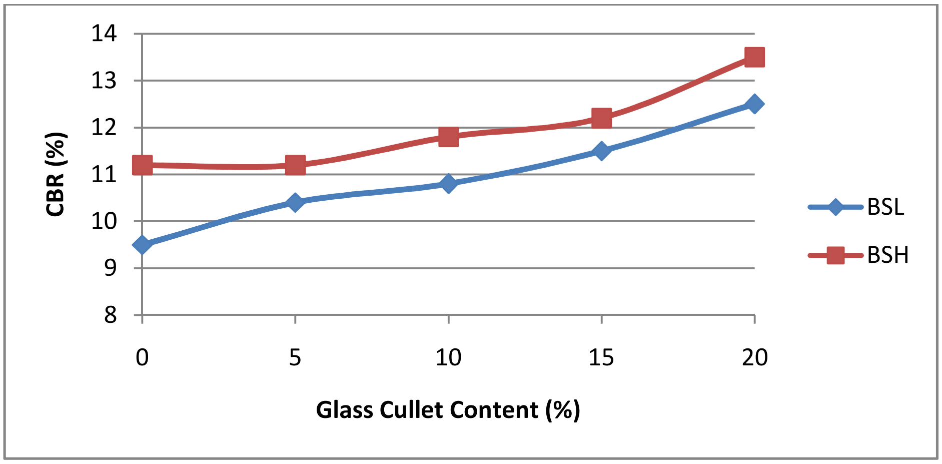


Figure 4.9 Variation of California Bearing Ratio of Black Cotton Soil with Glass Cullet content (BSL)

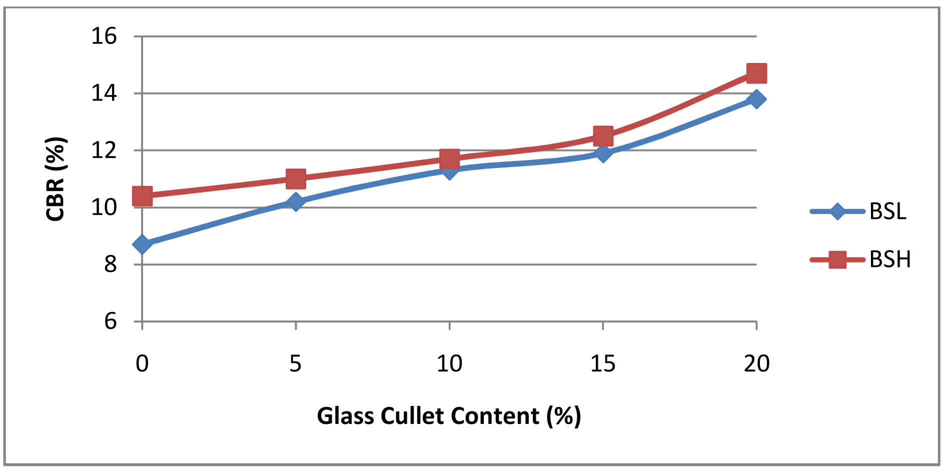


Figure 4.10 Variation of California Bearing Ratio of Black Cotton Soil with Glass Cullet content (BSH)

4.4 The Effect of OPC/GC on Engineering Properties of Black Cotton Soil

4.4.1 Atterberg limits

4.4.1.1 Liquid limit

There was a decrease in the liquid limit from 51.4% for the natural soil to a minimum of 47.5% for soil treated with 8%OPC / 20% GC blend, with the overall decrease in the liquid limit of the soil being attributed to the effect of OPC/GC blend. This is attributing to the fact that GC has negligible water absorption capacity and increased desiccation, thus resulting to lowering of the minimum water content at which soil will flow.

Similar trends for glass cullet dredged material blend were reported by Dennis et al. (2006) and Grubb et al. (2006). Figure 4.3 shows the variation of liquid limit for OPC/GC blend stabilized soil.

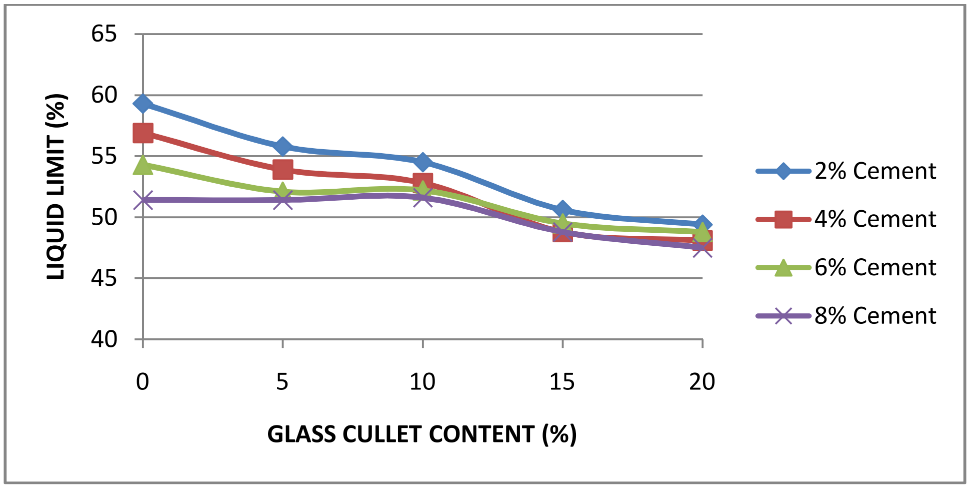


Figure 4.11 Variation of Liquid Limit of Black Cotton Soil – Ordinary Portland Cement mixture with Glass Cullet content

4.4.1.2 Plastic limit

The variation of plastic limit of the treated soil in Figure 4.12 shows that the plastic limit increased with OPC/GC content and the increase is linear with higher dosage of GC admixture. The peak value of 36.2% was observed at 8% OPC/20% GC blend. This presents an increase of 4.0% when compared with the value obtained for the natural soil.

This improvement in the plastic limit of the stabilized soil for each of the blend is due to the OPC/GC reactivity with the clay fraction of the soil, which resulted in an increase in the total surface area of the soil. However, the increase in the plastic limit with growing concentration of OPC/GC, may, in part be due to the excess CaO at level. It expected that plastic limit of the soil will continue to increase with further addition of GC to the soil/OPC mixture until it becomes non plastic, based on the fact that GC has negligible water absorption capacity and increased desiccation. Similar trends were reported by Denis et al. (2006).

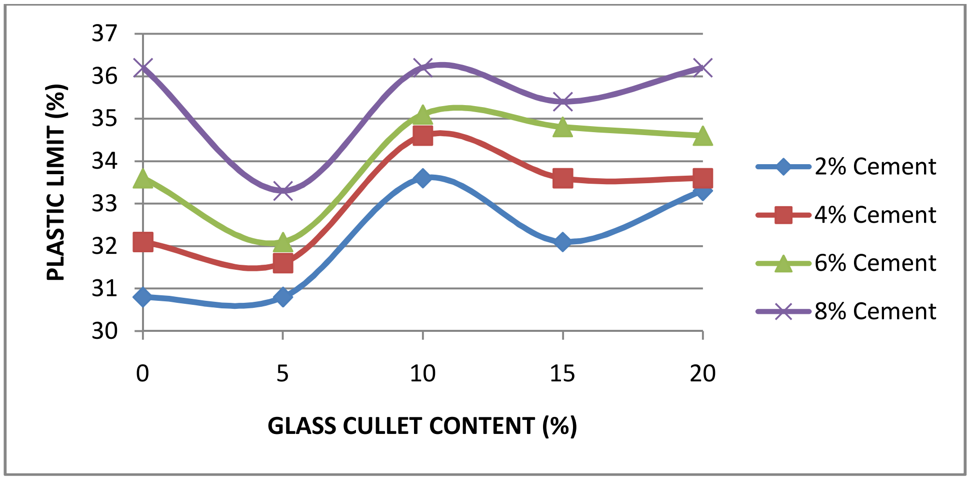


Figure 4.12 Variation of Plastic Limit of Black Cotton Soil – Ordinary Portland cement mixture with Glass Cullet content

4.4.1.3 Plasticity index

It has been shown by several researchers such as Osinubi et al., (2011) and Sarkar et al., (2012) that OPC changes the water film on the soil particles, modify clay minerals to some extent and decrease the soils PI. Generally, the PI of the soil reduced significantly on addition of GC to the soil/OPC blend. Figure 4.14 shows the variation of OPC/GC blend on the PI of the soil. It was observed that the PI decreased for a combination of OPC/GC blend up –to 20% GC. The highest decrease from 18.8% to 11.3% was recorded at 8% OPC/20%GC blend.

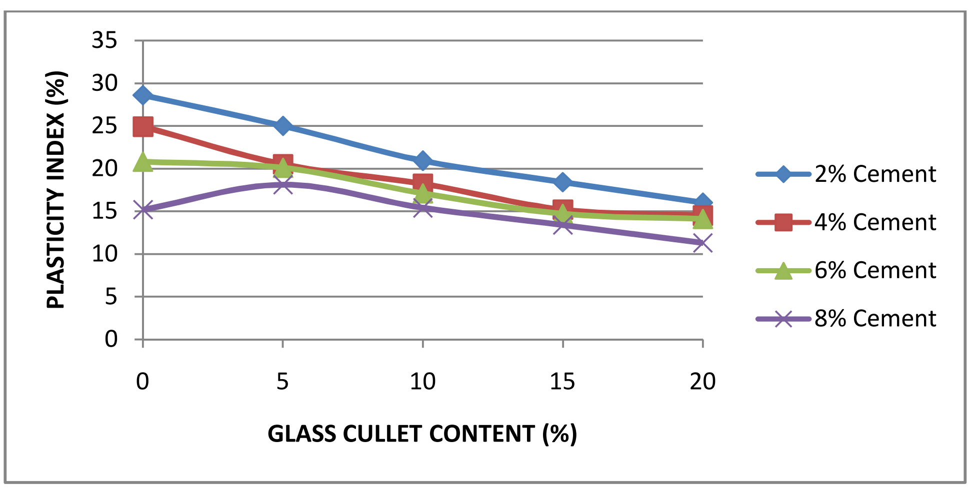


Figure 4.13 Variation of Plasticity Index of Black Cotton Soil – Ordinary Portland Cement mixture with Glass Cullet content

4.4.2 Linear Shrinkage

The results obtained suggested that the use of GC in conjunction with OPC in a stabilized soil of lesser shrinkage when compared with OPC treatment alone.

Figure 4.14 shows the variation of OPC stabilized black cotton soil with GC content. It was observed that 8% "OPC alone" treated specimens yielded shrinkage values slightly higher than values for 8% OPC/20% GC treated soil. Similar results were obtained for 6, 4 and 2% OPC treated specimens. Although the linear shrinkage was found to decrease slightly with OPC content, the overall reduction could be attributable to the slow reduction of montmorillonite lattice by the OPC/GC blend. This view is in agreement with Rogers and Enright (2016), who observed that at higher dosage of OPC, soil might exhibit shrinkage of fatigue cracks.

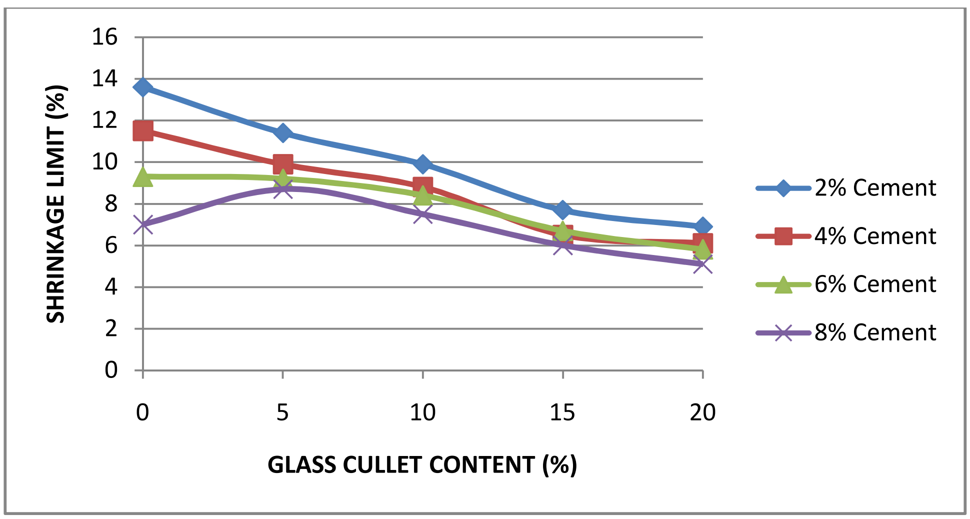


Figure 4.14 Variation of Linear Shrinkage of Black Cotton Soil – Ordinary Portland cement mixture with Glass Cullet content

4.4.3 Free swell

The free swell criterion provides reference point for the behaviour of the stabilized soil after rainfall. The effect of OPC/GC blend on the swelling potential of the stabilized material is shown in Figure 4.15. The free swell was observed to decrease significantly for each of the test series conducted when compared with the result obtained for the un-stabilized soil. At 0, 5, 10 and 20% GC content the swell reduced almost linearly, a maximum decrease, which is an improvement in the property of the soil, was observed at 5% GC content. Higher dosage of GC produces slight improvement in the free swell. It is interesting to not that at 8% OPC/5%GC content the soil yielded the lowest value of 30.7% for the free swell as compared with the value of 64.3% obtained from the natural soil.

It could be inferred that such a decrease in free swell may be due to chemo-physical reaction between the soil and OPC/GC blend which inhibits an immediate absorption of water, attributable to montmorillonite clay mineral.

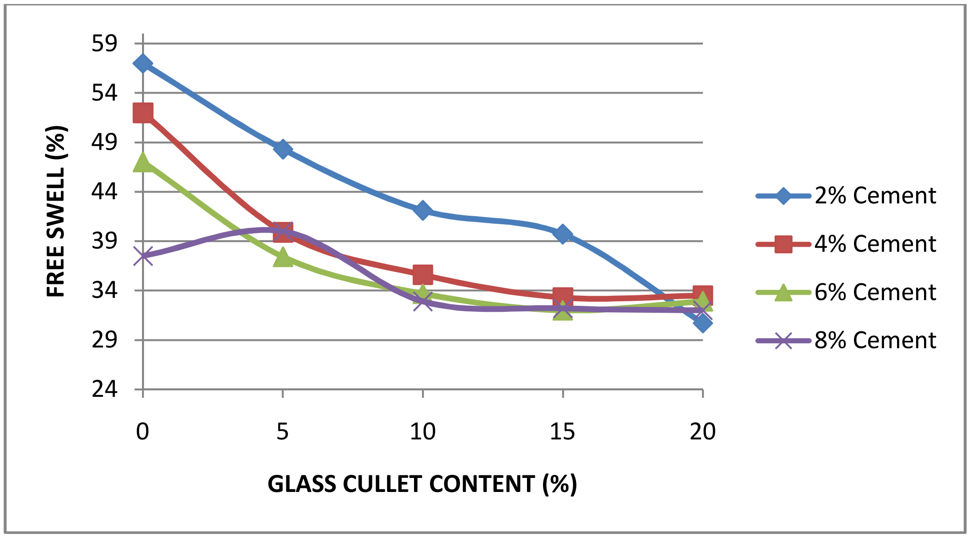


Figure 4.15 Variation of Free Swell of Black Cotton Soil – Ordinary Portland Cement mixture with Glass Cullet content

4.4.4 Compaction Characteristics

The natural soil when compacted at energy level of the BSH yielded the highest MDD value of 1.53Mg/m^3 which corresponded to the OMC value of 14%.on the other hand, soil compacted at the energy level of the BSL gave an MDD value of 1.46 Mg/m^3 with a corresponding OMC value of 15%.

The relationship between the moulding water content and the compactive effort for the soil is as shown in Figure 4.16 and 4.17. The trend is one of increasing MDD with an increase in compactive effort and corresponding decreasing in OMC with higher compactive effort. Otalvaro et al. (2016) indicated that an increase in the dry density of soils is a function of compaction energy. Similarly, Abichou et al., (2000), showed an increase in MDD with attendant decrease in

OMC for soils compacted at the energy levels of british standard heavy and british standard light reduced proctor. The trend of the compaction curves is in agreement with the findings of a number of researchers such as (Mitchell et al., 1965; Osinubi and Nwaiwu, 2005).

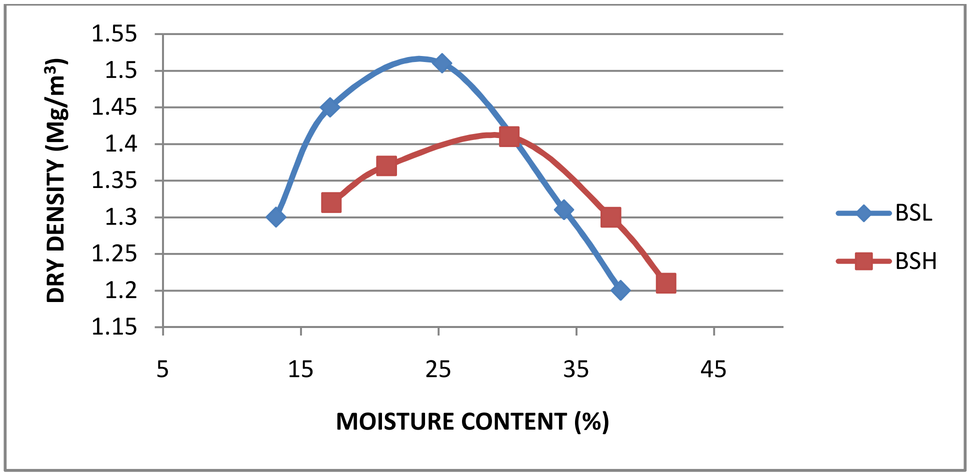


Figure 4.16: Compaction curves for Black cotton soil (BCS) BSL and BSH

4.4.4.1 Compaction curves for OPC/GC treated soil

The results obtained on the dry densities for the soil and treated with OPC and OPC/GC, showed that as the replacement levels of OPC and GC increase, the dry densities increase with the maximum value achieved at 20% replacement level for GC and at 8% replacement level for OPC. These behaviours show that maximum dry densities for soil treated with 2 – 8% OPC for both BSL and BSH energy level are 1.60Mg/m^3 and 1.70Mg/m^3, while that of 2% OPC (5-20% GC), 4% OPC (5-20% GC), 6% OPC (5-20% GC) and 8% OPC (5-20% GC) and compacted at the energy levels of BSL and BSH yielding 1.62, 1.62, 1.62, 1.65 and 1.68, 1.70, 1.74, 1.77 Mg/m^3 respectively. It is evident from the plot of the MDD and OMC; the best results were achieved using the BSH compactive effort. The effects of the various replacement levels on the moisture contents, showed a divergent behaviour. As the replacement levels are increasing, the moisture content decrease which is an indication of better performance achieved at 20% replacement level.

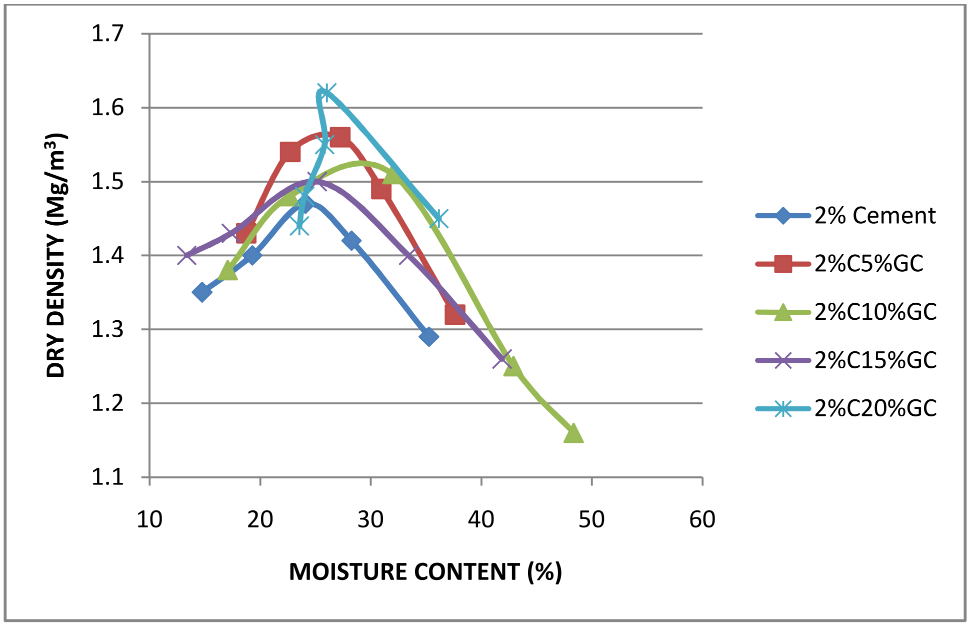


Figure 4.17: Variation of Dry Density with Moisture content for 2%OPC/ (5-20%) GC stabilized soil using BSL compactive effort.

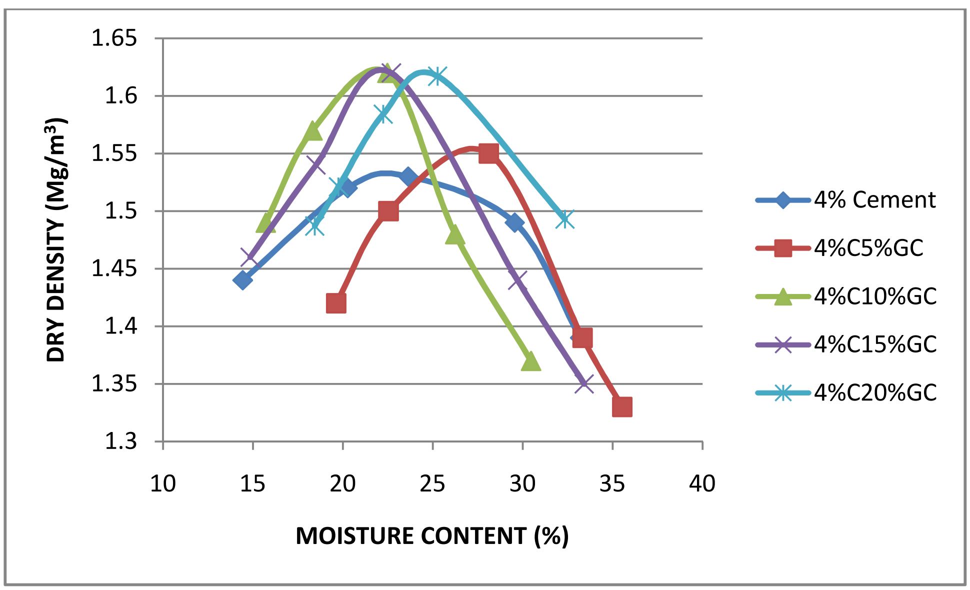


Figure 4.18: Variation of Dry Density with Moisture content for 4%OPC/ (5-20%) GC stabilized soil using BSL compactive effort.

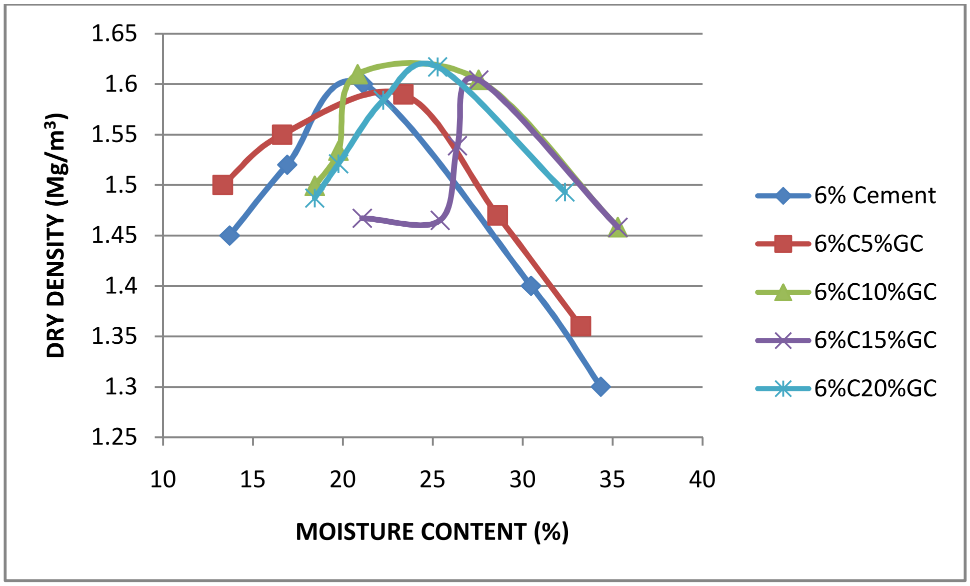


Figure 4.19: Variation of Dry Density with Moisture content for 6%OPC/ (5-20%) GC stabilized soil using BSL compactive effort.

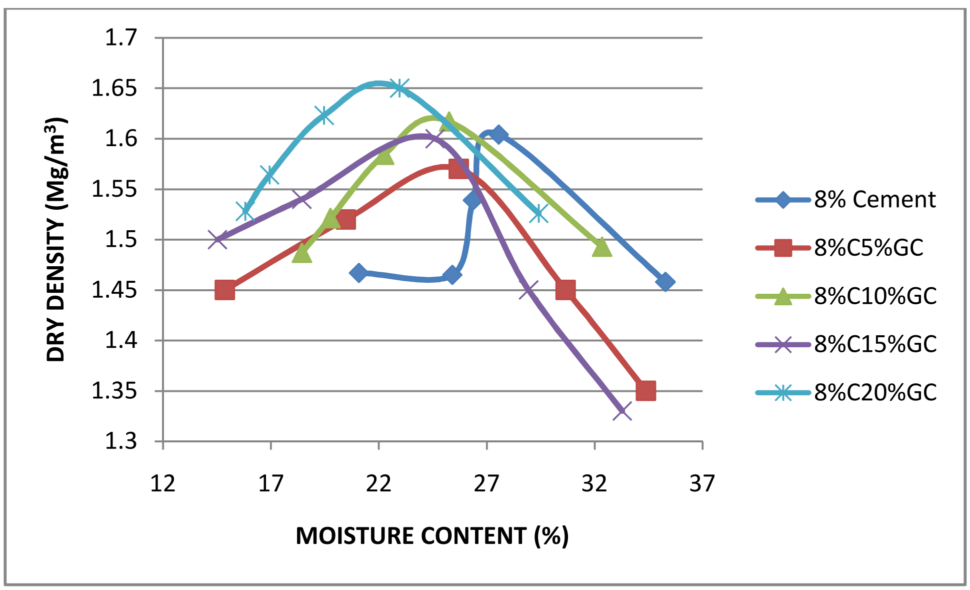


Figure 4.20: Variation of Dry Density with Moisture content for 8%OPC/ (5-20%) GC stabilized soil using BSL compactive effort.

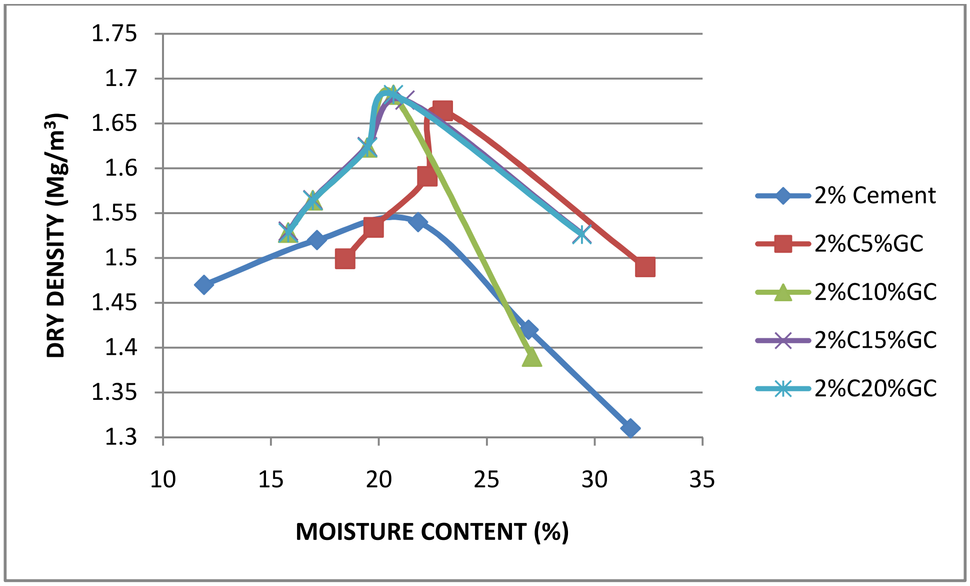


Figure 4.21: Variation of Dry Density with Moisture content for 2%OPC/ (5-20%) GC stabilized soil using BSH compactive effort.

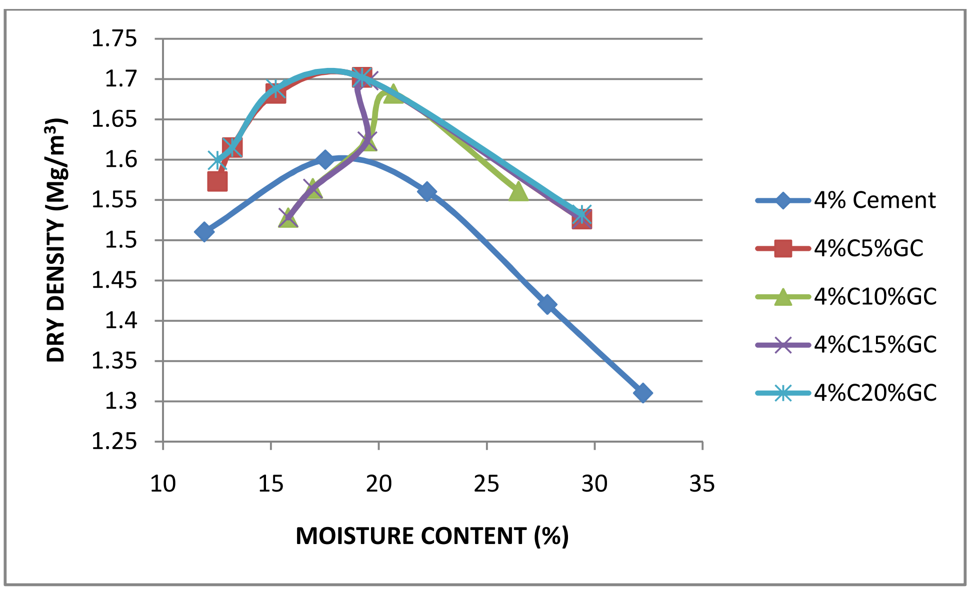


Figure 4.22: Variation of Dry Density with Moisture content for 4%OPC/ (5-20%) GC stabilized soil using BSH compactive effort.

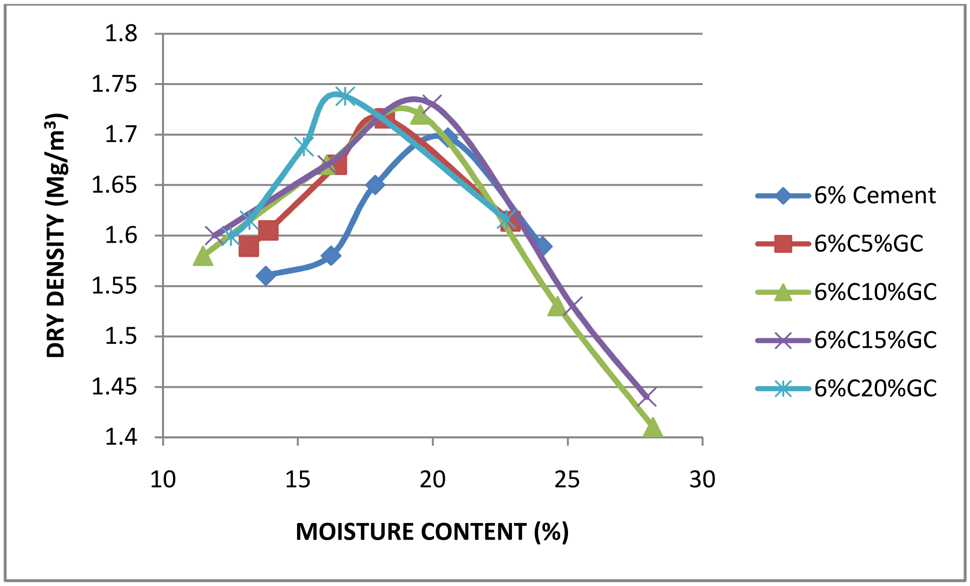


Figure 4.23: Variation of Dry Density with Moisture content for 6%OPC/ (5-20%) GC stabilized soil using BSH compactive effort.

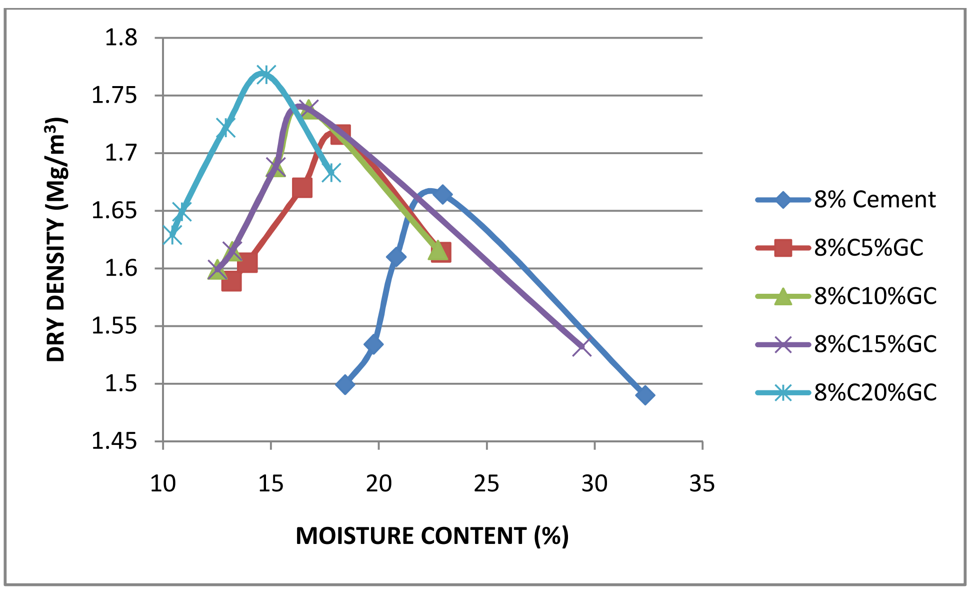


Figure 4.24: Variation of Dry Density with Moisture content for 8%OPC/ (5-20%) GC stabilized soil using BSH compactive effort.

4.4.4.2 Effect of OPC/GC on maximum dry density

On addition of GC to the soil/OPC mixture, increase in MDD was observed with GC content. The increase in MDD was attributed to the formation of new compounds, increase in surface area of particles at higher dosage of OPC/GC blend, as well as improved workability of the soil due to the increase in the alkaline activity in the mixture and the desiccating property of GC. Figure 4.25 and 4.26 shows the effects of OPC/GC blend, on the MDD of the soil. Also, the addition of water causes the bulking phenomenon in the stabilized soil. The fine cement particles influenced the reactivity of GC with the soil-cement blend, as such the soil-cement interaction resulted in the cementitious products and it gained strength. This trend is in order and agrees with Arabani et al. (2012) who reported that increase in MDD is due to the basic fact that soil-cement mix might have difference in specific gravity than the original soil.

The variation of MDD with GC content for the BSH compaction energy is depicted in Figure 4.26. An increase in compactive effort is a function of MDD, which agreed with Otalvaro et al. (2016) and Abichou et al., (2000) that indicated an increase in MDD with attendant decrease in OMC for soils compacted at the energy levels of British Standard Heavy and British Standard Light. Similarly, findings of Osinubi (1998) showed that increase in compactive effort for lateritic soil resulted in an increase in the MDD and a decrease in the OMC. This may be attributable to the poor ability of GC to absorb moisture, and/or due to high bulk densities of the additive present in the cementitious compounds (C-S-H and C-A-H) as well as the variation in mechanical energy applied in compacting the soil.

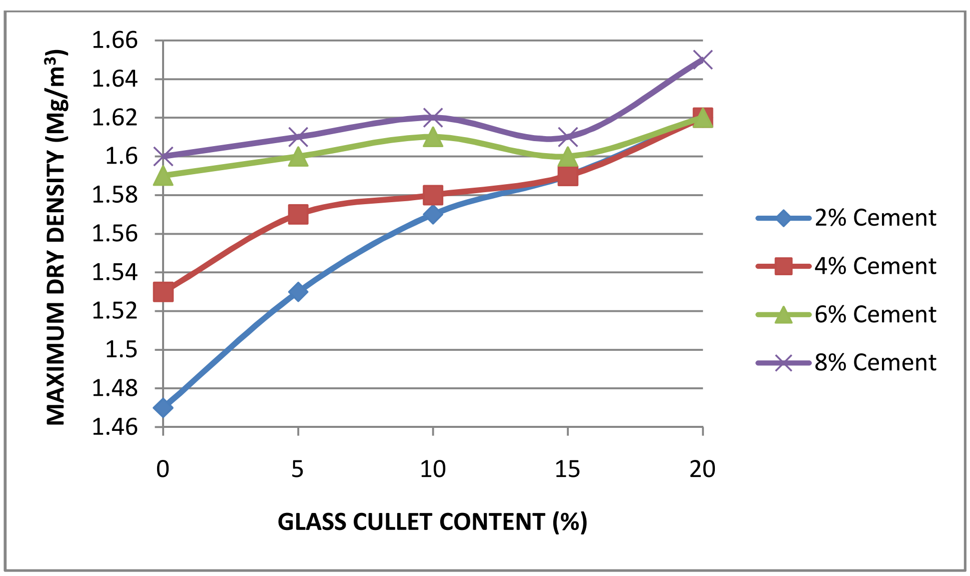


Figure 4.25: Variation of Maximum Dry Density of Black Cotton Soil – Ordinary Portland Cement mixture with Glass Cullet content (BSL compaction)

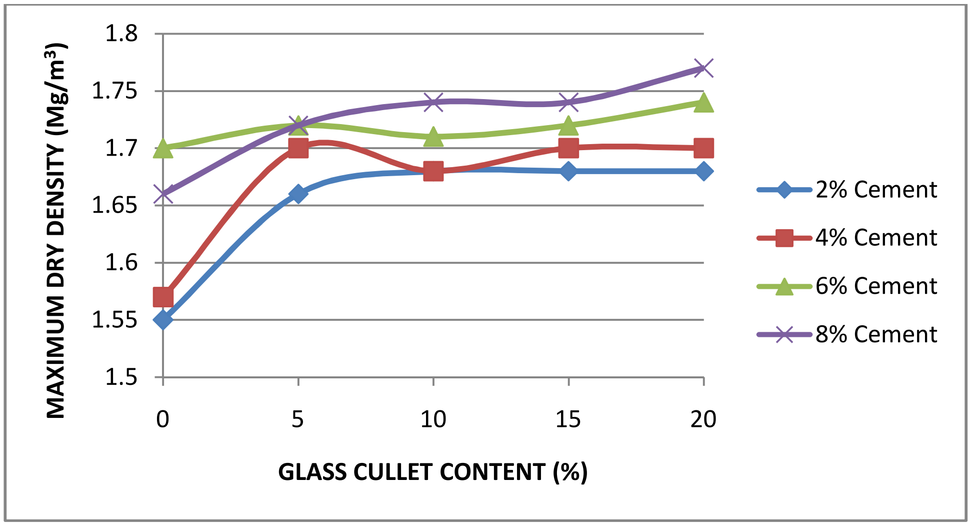


Figure 4.26: Variation of Maximum Dry Density of Black Cotton Soil – Ordinary Portland Cemant mixture with Glass Cullet content (BSH compaction)

4.4.4.3 Effect of OPC/GC on optimum moisture content

The variations of OMC for the two compactive efforts are shown in Figure 4.27 and 4.28. The OMC decreased with increase in OPC/GC blend for both compactive efforts. It could be that the OPC/GC dosage increased the surface area of soil particles due to the alkali (Na_2O) content of GC in the formation of calcium silicate hydrates (C-S-H) for strength development of the treated soil, Khmiri et al. (2013).

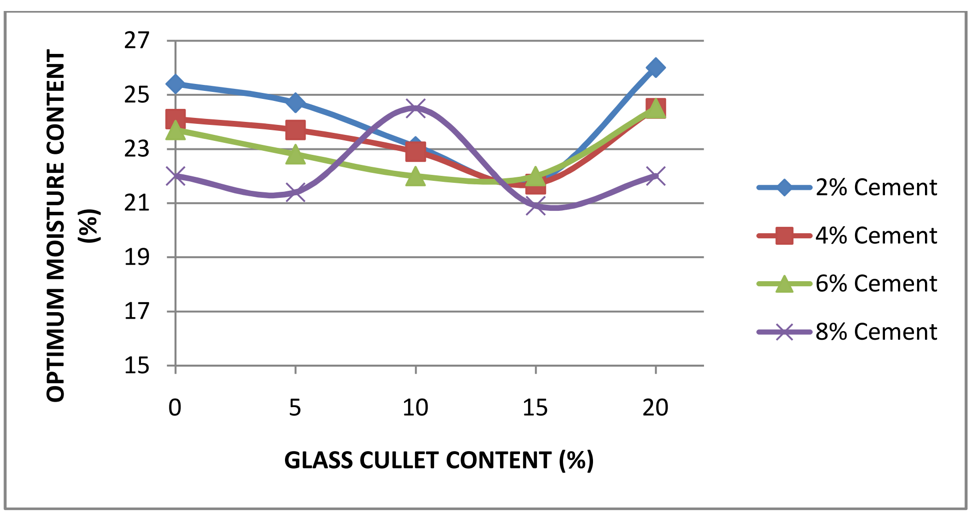


Figure 4.27: Variation of Optimum Moisture Content of Black Cotton Soil – Ordinary Portland Cement mixture with Glass Cullet content (BSL compaction)

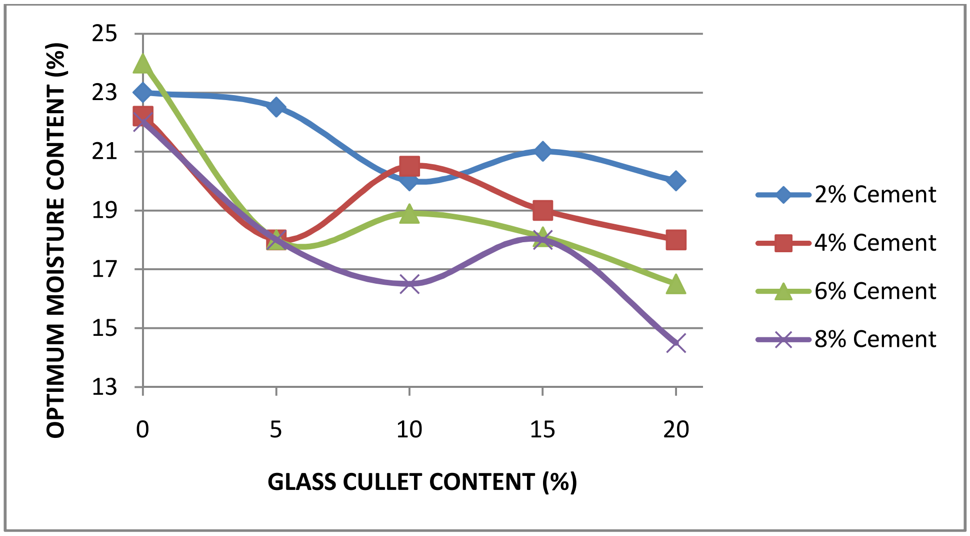


Figure 4.28: Variation of Optimum Moisture Content of Black Cotton Soil – Ordinary Portland Cement mixture with Glass Cullet content (BSH compaction)

4.4.5 Unconfined Compressive Strength

The variation of UCS with various percentages of OPC/GC blend after compacting the specimens at OMC for 7 days using BSL and BSH compaction energies showed that compressive strength improved significantly after 7 days of curing. However, the peak 7-day UCS value of 1152 kN/m^2 was recorded at 8% OPC / 20% GC blend using the BSL, while the UCS value of 1568 kN/m^2 was obtained at 8%OPC/20%GC using the BSH compactive effort, as shown in Figures 4.29 and 4.32. The TRRL (1997) specified 1720 kN/m^2 as criterion for adequate stabilisation using OPC. The influence of compactive effort on the compressive strength of the 7 days cured specimens is well pronounced for each of the blend tested.

The variations of UCS for the samples cured for a period of 14 days using the two compaction energy levels are shown in Figure 4.30 and 4.33. The influence of GC admixture on the compressive strength has a long-term effect which may be attributed the slow pozzolanic chemical reaction with GC and calcium hydroxide (CH) of cement when compared with OPC hydration. Dyer and Dhir (2001) reported that CH continuously decreased with increase GC replacement in concrete as the CH is consumed in the pozzolanic reaction of GC. This agrees with the assertion.

The UCS value of 1176 kN/m^2 obtained for a combination of 8%OPC/5%GC using the BSL compaction effort was noticeably higher than the UCS values obtained for "OPC alone" treated specimens. It could therefore be inferred that the compressive strength increased linearly by fixing the OPC content and varying GC content from 5-20%. The increase in the compressive strength may be attributable to the concentration of OPC in this range, which reduces plasticity, thereby improving cementatious properties of the soil. It is evident that the OPC/GC content, curing age, as well as the rise in the pH level of the soil gives rise the strength development of the specimens.

The BSH compactive effort resulted to higher UCS values than the specimens compacted using the BSL compactive effort. This has been comfirmed by Mateous (1964) who reported that increased compactive effort (from British Standard light to British Standard heavy) influences the strength of stabilized soil by 200% for both 7 and 28 days curing period.

The specimens compacted using BSL and BSH compactive effort cured for 28 days shown in Figures 4.31 and 4.34 respectively. The results illustrated that at 4, 6 and 8% OPC the compressive strength decreased with higher dosage of GC admixture. However, at 2% OPC content the strength increased with higher dosage of GC in agreement with Eberemu et al. (2012).

The higher strength archived with the BSH compactive effort may be due to the closer orientation of adjacent particles. The BSL and BSH compactive efforts yielded the highest compressive strength at 8% OPC / 20% GC and at 8% OPC / 5% GC blend in each case. This may be due to pozzolanic reaction, which progressively enhances the strength of the treated soil. The developments of high UCS values at the 28 days curing period is attributed to the effect of OPC which promote the production of alkaline compounds that increases the pH value of the soil and promote the self-hardening Characteristics of the GC admixture.

By comparing the 28 days strength with those obtained at 7 and 14 days, it is obvious that the lower values obtained at 7 and 14 days were as a result of premature failure of the specimens (splitting of ends and spalling of the surface). Also, Wartman et al. (2004) suggested that the impact of GC on strength of fine grained soils may be delayed until GC particles cease floating in the fine grained matrix and develop particle interactions which subsequently dominate the strength behaviour, with the presence of OPC, glass powder (finer than < 0.300mm and below) can act like a pozzolanic material adjacent cement Shayan and Xu (2004), as such, an increase in strength is expected.

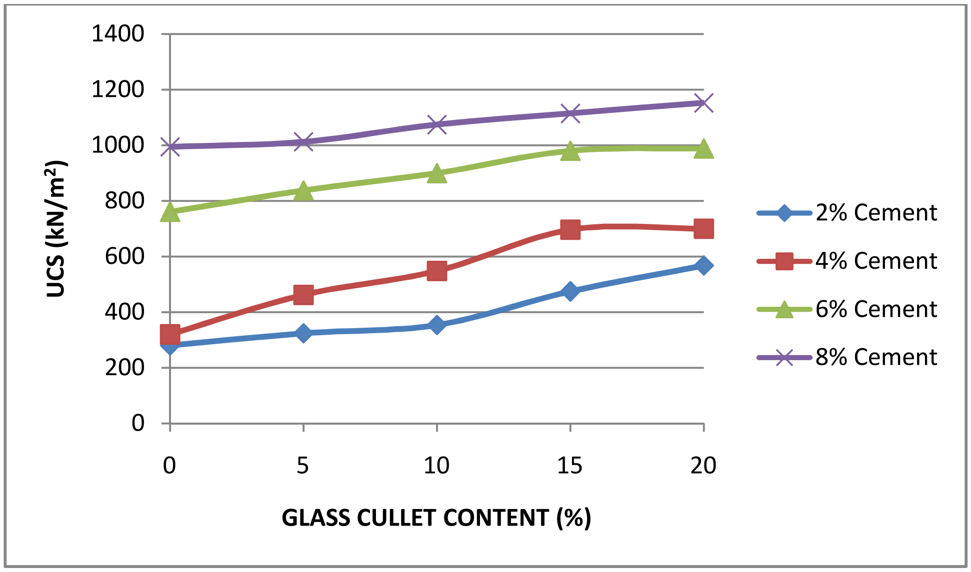


Figure 4.29: Variation of UCS (7 days curing period) of Black Cotton Soil – Ordinary Portland Cement mixture with Glass Cullet content (BSL compaction)

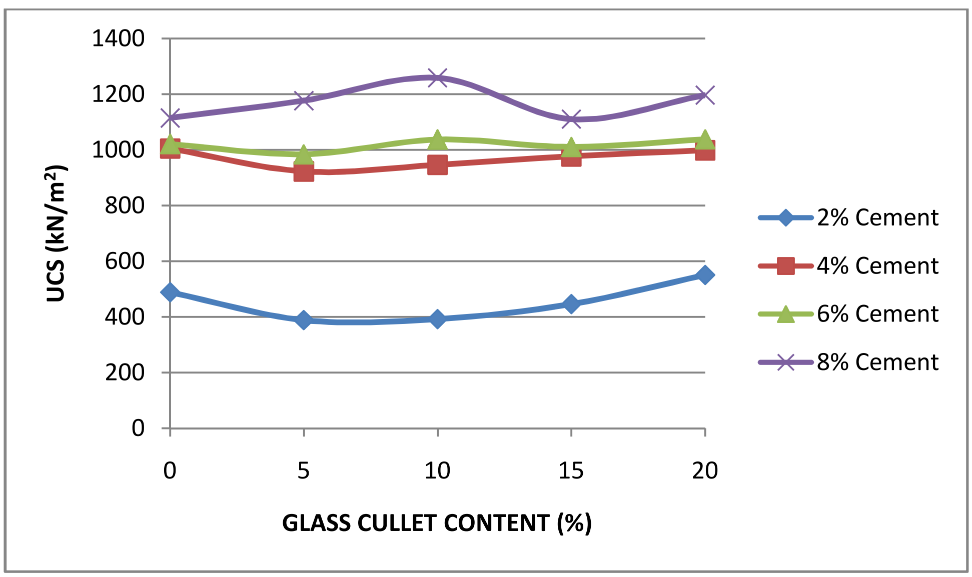


Figure 4.30: Variation of UCS (14 days curing period) of Black Cotton Soil – Ordinary Portland Cement mixture with Glass Cullet content (BSL compaction)

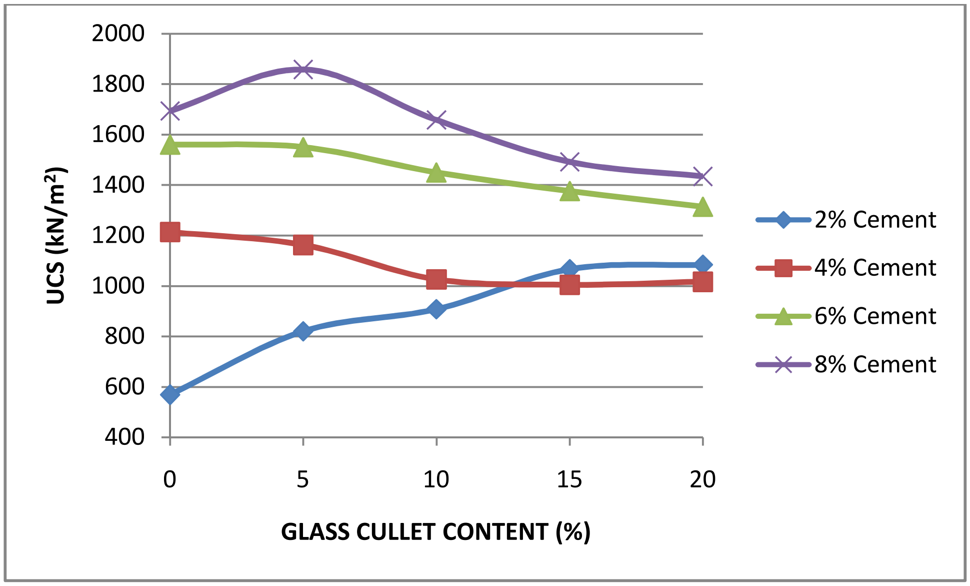


Figure 4.31: Variation of UCS (28 days curing period) of Black Cotton Soil – Ordinary Portland Cement mixture with Glass Cullet content (BSL compaction)

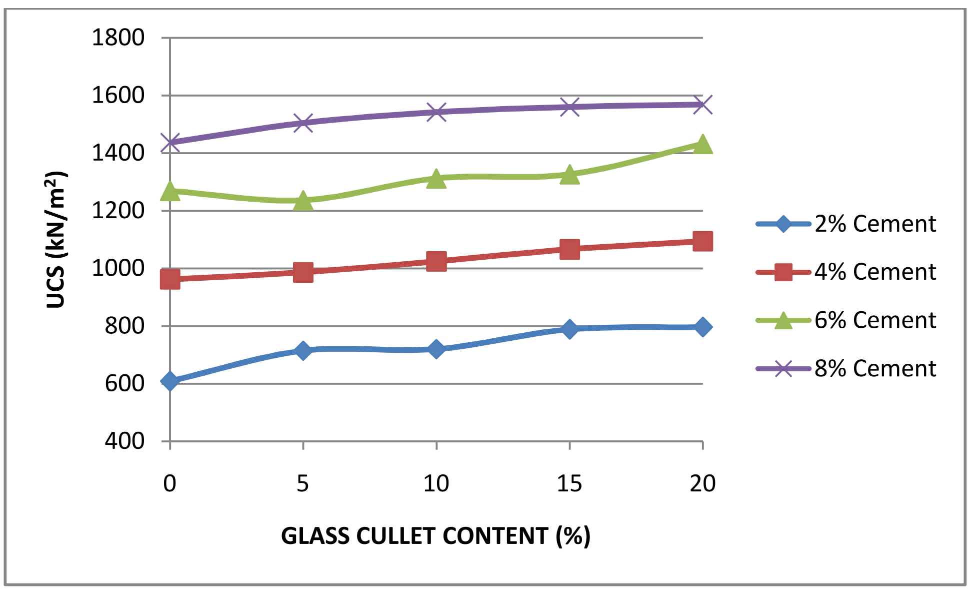


Figure 4.32: Variation of UCS (7 days curing period) of Black Cotton Soil – Ordinary Portland Cement mixture with Glass Cullet content (BSH compaction)

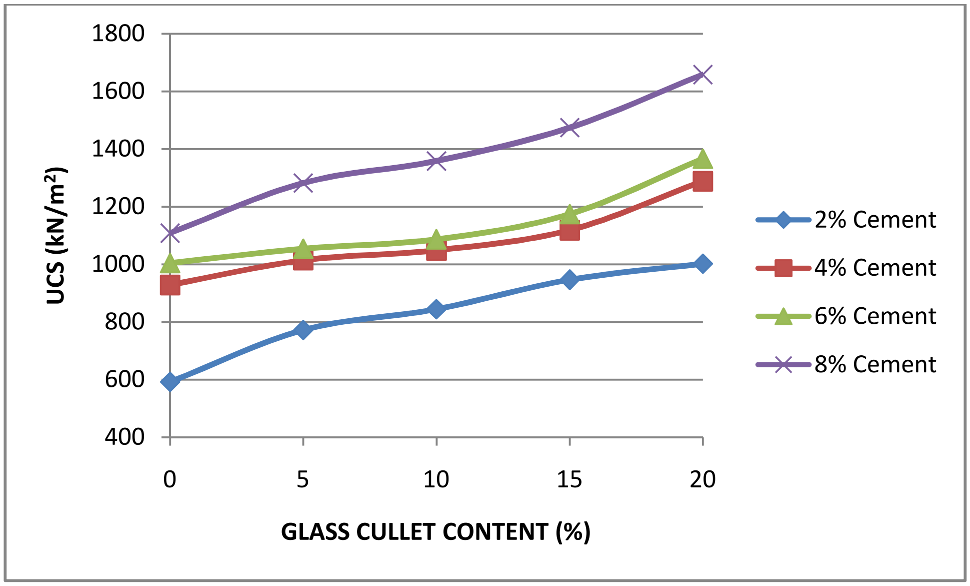


Figure 4.33: Variation of UCS (14 days curing period) of Black Cotton Soil – Ordinary Portland Cement mixture with Glass Cullet content (BSH compaction)

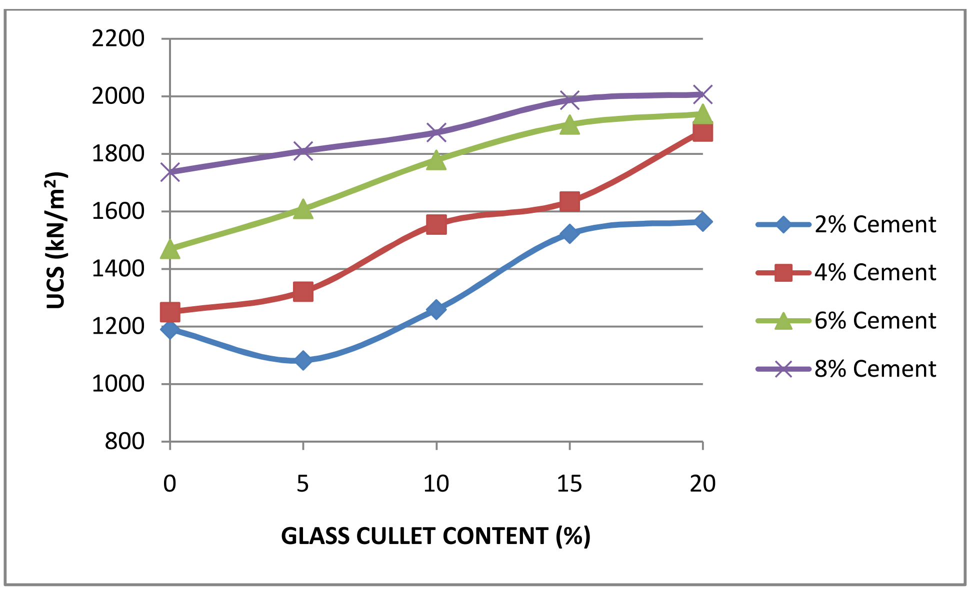


Figure 4.34: Variation of UCS (28 days curing period) of Black Cotton Soil – Ordinary Portland Cement mixture with Glass Cullet content (BSH compaction)

4.4.6 California Bearing Ratio

Figure 4.35 and 4.36 show the variation of un-soaked CBR with various percentages of OPC/GC blend using the BSL and BSH compactive effort. The variation of un-soaked CBR with various percentages of OPC/GC blend using the BSH compactive effort showed slightly higher values than for the BSL compactive effort.

The CBR value of the specimen treated with OPC only increase in OPC content for both the compactive efforts. With the addition of GC, the CBR value increased at higher percentage of GC. This may be attributed to the increase in the contact area and adhesion between OPC and soil by GC which will create a dense network of interconnected particles. A peak value of 63% was observed at 8% OPC/5%GC blend, for BSH compactive effort. Similarly, an increase of about 407.5% in the CBR value was noticed when the natural soil was compacted using BSL effort and the increase occurred at 8% OPC/20%GC blend.

It was reported by Arabani et al. (2012), in their study of cement stabilized with crushed class sand blends that the cementitious reaction between cement and admixture took place as aprimary process. The hydration of the cement was regarded as primary reaction and formed the normal hydration products that bound particles together. The increase in the CBR value may be due to shear transfer mechanism between the soil and GC, and the improvement in the strength might be due to the pozzolanic action of GC/OPC mix. Furthermore, the peak CBR value of 39% and 37.8% achieved at BSH and BSL compactive efforts, respectively, with 0% GC content did not meet the 180% CBR value criterion recommended by the Nigerian General Specification (1997) for OPC stabilized soil. This is attributed to the high content of montmorillonate in the soil, which seems to negate the effectiveness of OPC.

Specimen treated with 8% OPC (5-20%) GC and compacted with both the two energy levels meet the requirement of 30% CBR of the Nigerian General Specification (1997) for use of the soil material for sub-base in roads, other mixes also met this requirement , they include 6% OPC/(5-20%) GC, 4%OPC/20% GC compacted at BSH energy level and 6%OPC/(5-10%)GC compacted at BSL energy level.

However, at 2, 4 and 6% OPC with varying proportions of GC compacted with both compactive effort did not meet the requirement for good quality base or sub-base for road pavement, it however, suffices for use as a sub-grade material.

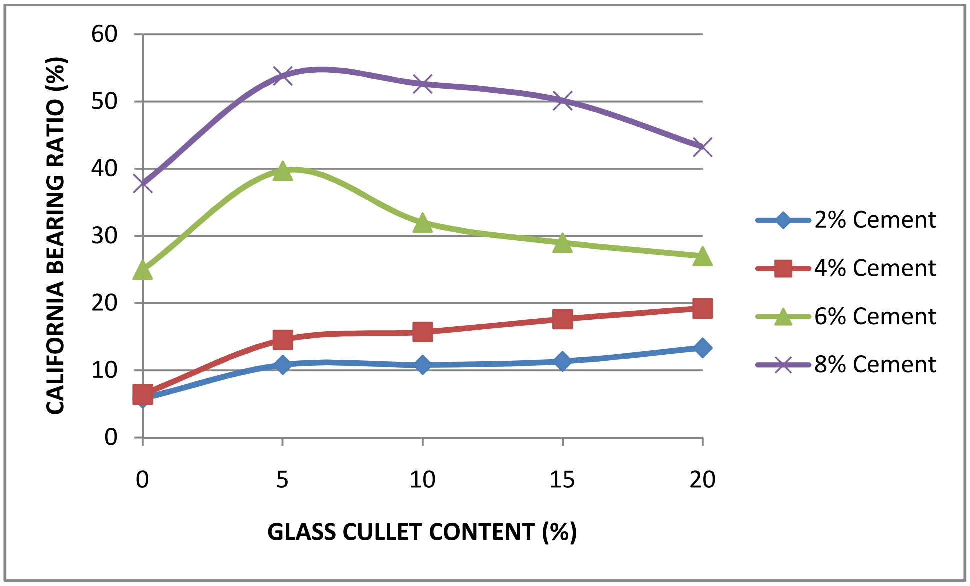


Figure 4.35: Variation of Un-soaked California Bearing Ratio of Ordinary Portland Cement stabilized Black Cotton Soil with Glass Cullet content for BSL compactive effort.

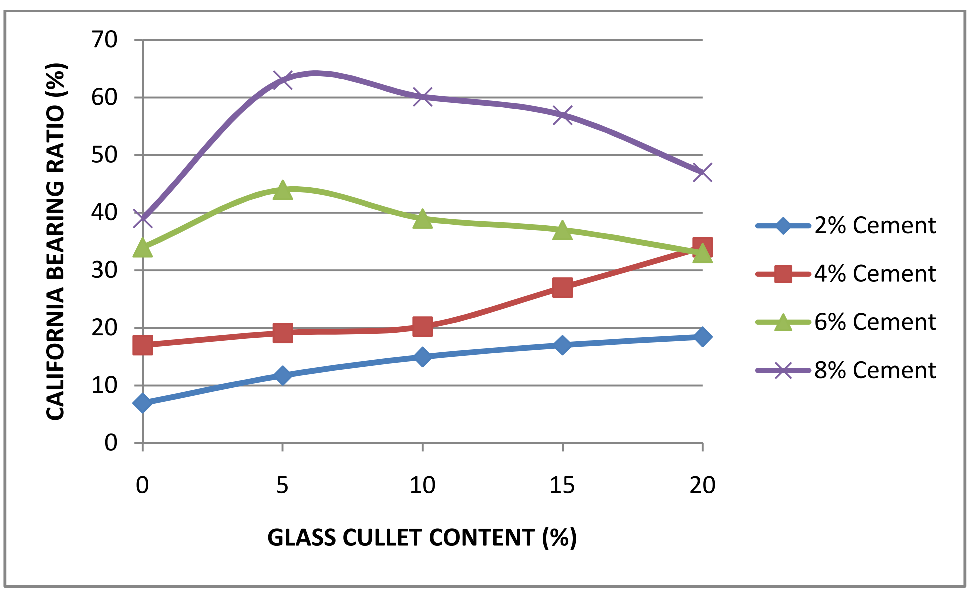


Figure 4.36: Variation of Un-soaked California Bearing Ratio of Ordinary Portland Cement stabilized Black Cotton Soil with Glass Cullet content for BSH compactive effort.

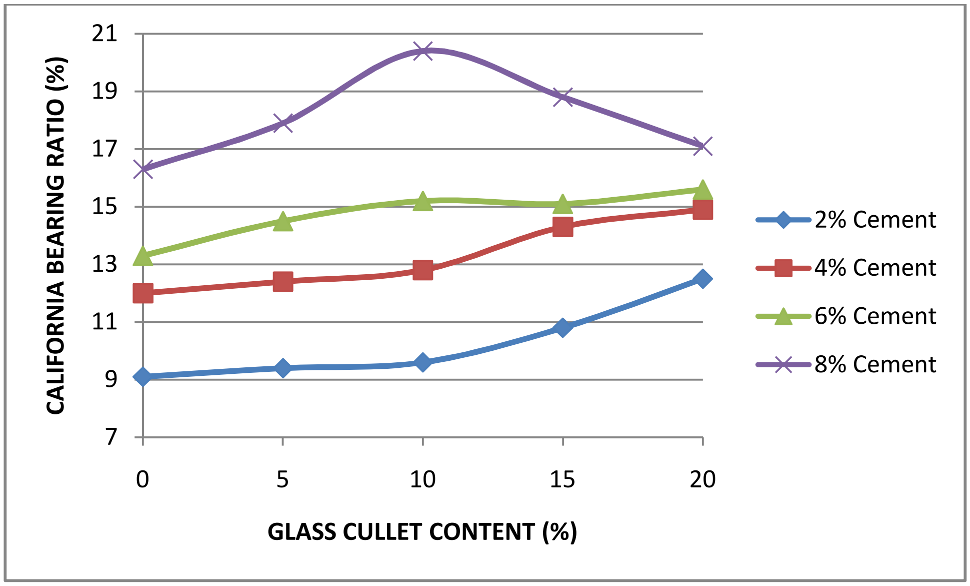


Figure 4.37: Variation of Soaked California Bearing Ratio of Ordinary Portland Cement stabilized Black Cotton Soil with Glass Cullet content for BSL compactive effort.

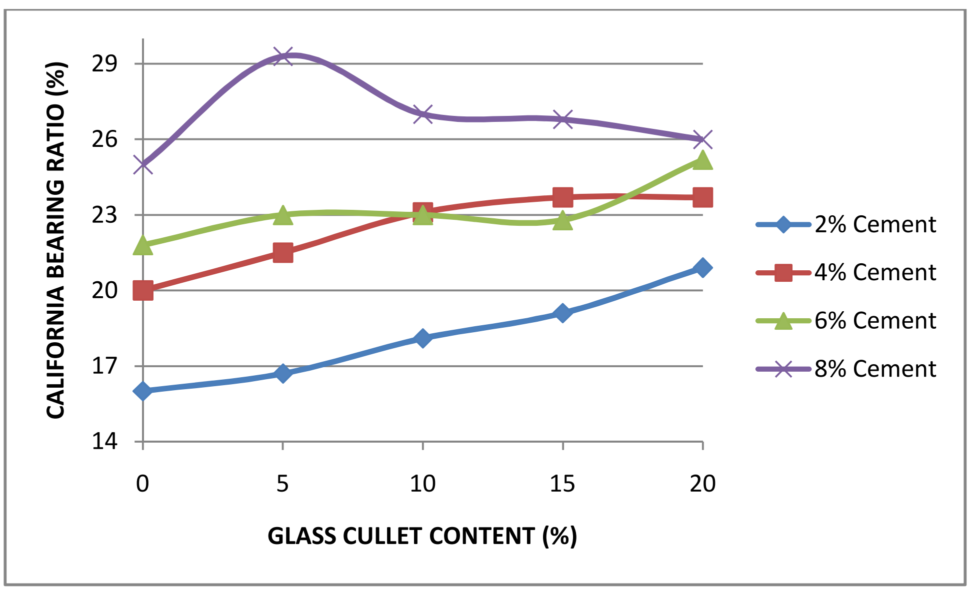


Figure 4.38: Variation of Soaked California Bearing Ratio of Ordinary Portland Cement stabilized Black Cotton Soil with Glass Cullet content for BSH compactive effort.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

This chapter carries the conclusions on the experimental results on the potentials for the use of GC in improving the engineering properties of OPC stabilized black cotton soil which was obtained from Baure village near Dadin-kowa in Yalmatu-Deba Local Government Area of Gombe state.

From the results the following conclusions were drawn:

1. The black cotton soil used in this study was highly clayey with 74.95% passing the BS sieve No.200, with high liquid limit, plastic limit and plasticity index the soil falls under A-7-5(14) or CH subgroup in accordance with the AASHTO and USCS classification. The natural soil has a very high swelling pressure while UCS and CBR were very low.
2. There was a substantial reduction in the swelling characteristic of soil from 64.3% at 0% OPC/GC to 30.7% at 8%OPC/5%GC and plasticity index from 31.3% at 0% OPC/GC to 11.3% at 8%OPC/20%GC when compared to the untreated soil, this seems to give creditability to the physico- chemical reactions between the soil and OPC/GC blend. In general, the OMC decrease with increase in OPC/GC content for both compactive efforts, with BSH compactive effort yielding high MDD due to greater energy supplied. A remarkable improvement in unconfined compressive strength (UCS) was observed at 8%/OPC/20%GC treatment, yielding an average UCS value of 1152kN/m^2 and 1568kN/m^2 at 7 days curing for BSL and BSH compactive effort. None of the specimens compacted using the two energy levels for the 7 days curing period met the minimum strength requirement of 1720kN/m^2 as specified by Road Note 31 (TRRL, 1977) for an economic range of OPC stabilisation. Higher Unconfined compressive strength (UCS) values observed for both compactive efforts at 28 days curing appeared to give creditability to the pozzolanic reaction of fine GC. None of the CBR specimen met the requirements of the Nigerian General Specifications (1997) 80% CBR criterion for base material. Peak CBR values obtained for both energy level met the requirements of the

Nigerian General Specifications (1997) 30% CBR criterion for sub-base and sub-grade material in light trafficked roads.

3. To achieve a significant decrease in plasticity index an optimum blend of 8% OPC//5%GC gives the best result.

Based on the above conclusions, strength of black cotton soil depends primarily on the mineral composition, which is the best manifested in the plasticity characteristics of the soil. The addition of GC resulted in the reduction of plasticity index, thus given rise to greater workability when compared to untreated clays. The results also showed that GC could be used efficiently to reduce the swell potential of expansive soils, improve the Unconfined Compressive Strength (UCS) and CBR. Preliminary results of BCS with GC revealed that the glass cullet alone improves to a certain degree all the properties tested.

In conclusion, the range of properties obtained using cement stabilized BCS/GC blends could no doubt increase the beneficial use of BCS as fill material for roadbed and embankment in general engineering application.

5.2 Recommendations

The study has recommended the following:

i. 8%OPC/(5-20%)GC should be used as a materials for sub-base in pavement constructions.

ii. The effect of OPC/GC on other expansive soils other than black cotton soils can be ascertained in the future research work to understand the impact more broadly.

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APPENDICE XXX

Appendix A1: Natural Moisture Content Result

Natural Moisture Content Result		
Can No.	1	2
Wight of Can (g)	16	16
Wight of Can + Wet Soil (g)	84	83
Wight of Can + Dry Soil (g)	78	77
Wight of Dry Soil (g)	62	61
Wight of Water (g)	6	6
Moisture Content (%)	9.68	9.83
Average Moisture Content (%)	**9.76**	

The Natural moisture content was determined from the expression

Ms = $\frac{M2-M3}{M3-M1}\ X\ 100$

Ms = Moisture Content in %

M2 = weight of container + wet soil (g)

M3 = weight of container + dry soil (g)

M1 = weight of container (g)

Appendix A2: Specific gravity for the soil particles

BOTTLE No.	M1 (g)	M2 (g)	M3 (g)	M4 (g)	Gs (-)	AVERAGE Gs (-)
1	1072	1472	2576	2326	2.67	**2.66**
2	1072	1472	2575	2326	2.65	

<u>NOTE</u>

M1 = MASS OF DENSITY BOTTLE EMPTY (g)

M2 = MASS OF DENSITY BOTTLE + SOIL (g)

M3 = MASS OF DENSITY BOTTLE + SOIL + WATER (g)

M4 = MASS OF DENSITY BOTTLE + WATER ONLY (g)

Appendix A3: Sieve Analysis Result for the Natural Soil

SIEVE SIZE (mm)	WT. RETAINED (g)	COMM.WT. RETAINED (g)	% WT. RETAINED	TOTAL % PASSING
5.0	0	0	0.00	100.00
3.35	0	0	0.00	100.00
2.36	0	0	0.00	100.00
2.0	9.4	9.4	0.94	99.06
1.18	20.4	29.8	2.04	97.02
0.6	53.4	83.2	5.34	91.68
0.425	26.3	109.5	2.63	89.05
0.3	29.9	139.4	2.99	86.06
0.212	27.8	167.2	2.78	83.28
0.15	24.2	191.4	2.42	80.86
0.075	17.2	208.6	1.72	79.14
PAN	1.1	209.7	0.11	79.03

$$\text{Clay and Silt Content (\%)} = \frac{1000-208.6}{1000} X\ 100$$

= <u>79.14%</u>

Washed Soil Sample

Total WT. of soil washed through BS sieve No. 200 = 1000g

Weight retained on BS sieve No. 200 after washing = 208.6g

% silt and clay content = 79.14%

Appendix A4: Results of Hydrometer Analysis

S/N	Elapsed Time (min)	Observed Hydrometer Reading (R'h)	Temperature (°C)	True Hydrometer Reading (Rh)	Effective Depth HR (mm)	Modified Reading Rd	Particle Diameter D (µm)	% Finer than D K (%)
1	0.5	20	26	20.5	130.02	23	64.57	73.7
2	1	18	26	18.5	138.23	21	47.08	67.3
3	2	15	26	15.5	150.55	18	34.74	57.7
4	4	11	26	11.5	166.98	14	25.87	44.9
5	8	3	26	3.5	199.84	6	20.01	19.2
6	15	-0.5	26	0	214.21	2.5	15.13	8
7	30	-0.5	26	0	214.21	2.5	10.7	8
8	60	-1	26	-0.5	216.27	2	7.6	6.4
9	120	-2	26	-1.5	220.38	1	5.43	3.2
10	240	-2	26	-1.5	220.38	1	3.84	3.2
11	450	-2.5	26	-2	222.43	0.5	2.82	1.6
12	1440	-3	26	-2.5	224.48	2.6645E-15	1.58	0

Appendix A5: Effect of Cement/GC on the Atterberg Limits of the black cotton soil

	Replacement Proportion by dry weight of soil (%)			Linear Shrinkage	Free	Index properties (%)		
S/n.	Soil	OPC	GC	LS (%)	Swell (%)	LL	PL	PI
1	100	2	0	13.6	57	59.3	30.8	28.6
2	100	4	0	11.5	52	56.9	32.1	24.9
3	100	6	0	9.3	47	54.3	33.6	20.8
4	100	8	0	7	37.5	51.4	36.2	15.2
5	100	2	5	11.4	48.3	55.8	30.8	25.0
6	100	2	10	9.9	39.9	53.9	33.3	20.5
7	100	2	15	9.2	37.4	52.1	32.1	20.1
8	100	2	20	8.7	40	51.4	33.3	18.1
9	100	4	5	9.9	42.1	54.5	33.6	20.9
10	100	4	10	8.8	35.6	52.8	34.6	18.2
11	100	4	15	8.4	33.7	52.2	35.1	17.1
12	100	4	20	7.5	32.9	51.6	36.2	15.4
13	100	6	5	7.7	39.7	50.6	32.1	18.4
14	100	6	10	6.5	33.3	48.8	33.6	15.2
15	100	6	15	6.7	32	49.5	34.8	14.7
16	100	6	20	6.0	32.2	48.8	35.4	13.4
17	100	8	5	6.9	30.7	49.4	33.3	16.0
18	100	8	10	6.1	33.5	48.1	33.6	14.5
19	100	8	15	5.8	32.9	48.8	34.6	14.1
20	100	8	20	5.1	32	47.5	36.2	11.3

Appendix B1: Compaction test results of Soil-Cement Stabilisation

		Compactive Effort			
	Replacement	BSL		BSH	
Material	Level (%)	MC (%)	DD (Mg/m^3)	MC (%)	DD (Mg/m^3)
Cement Only	2C	14.75	1.35	11.89	1.47
		19.28	1.40	17.13	1.52
		24.10	1.47	21.81	1.54
		28.26	1.42	26.94	1.42
		35.27	1.29	31.67	1.31
Cement Only	4C	14.43	1.44	11.91	1.51
		20.26	1.52	17.52	1.60
		23.63	1.53	22.23	1.56
		29.55	1.49	27.81	1.42
		33.21	1.39	32.24	1.31
Cement Only	6C	13.70	1.45	13.80	1.560
		16.90	1.52	16.23	1.580
		21.13	1.60	17.86	1.650
		30.47	1.40	20.56	1.697
		34.34	1.30	24.07	1.589
Cement Only	8C	21.07	1.467	18.43	1.499
		25.40	1.465	19.75	1.534
		26.37	1.539	20.81	1.610
		27.55	1.604	22.96	1.664
		35.29	1.458	32.35	1.490

Appendix B2: Compaction test results of soil stabilized with 2% OPC/ 5-20% GC.

Material	Replacement Level (%)	Compactive Effort BSL MC (%)	BSL DD (Mg/m^3)	BSH MC (%)	BSH DD (Mg/m^3)
	2C-5GC	18.72	1.43	18.43	1.499
		22.71	1.54	19.75	1.534
		27.22	1.56	22.24	1.591
		30.94	1.49	22.96	1.664
		37.61	1.32	32.35	1.490
	2C-10GC	17.05	1.38	15.80	1.528
		22.65	1.48	16.93	1.564
		31.97	1.51	19.46	1.623
		42.90	1.25	20.67	1.682
		48.33	1.16	27.10	1.39
	2C-15GC	13.34	1.40	15.80	1.530
		17.37	1.43	16.93	1.565
		25.14	1.50	19.46	1.624
		33.42	1.40	21.21	1.676
		41.87	1.26	29.41	1.527
	2C-20GC	23.53	1.44	15.80	1.528
		24.01	1.48	16.93	1.564
		25.83	1.55	19.46	1.623
		26.02	1.62	20.67	1.682
		36.15	1.45	29.41	1.526

Appendix B3: Compaction test results of soil stabilized with 4% OPC/ 5-20% GC.

		Compactive Effort			
	Replacement	BSL		BSH	
Material	Level (%)	MC (%)	DD (Mg/m^3)	MC (%)	DD (Mg/m^3)
4C-5GC		19.62	1.42	12.51	1.573
		22.53	1.50	13.20	1.615
		28.11	1.55	15.22	1.682
		33.32	1.39	19.22	1.702
		35.53	1.33	29.41	1.526
4C-10GC		15.71	1.49	15.80	1.528
		18.30	1.57	16.93	1.564
		22.47	1.62	19.46	1.623
		26.24	1.48	20.67	1.682
		30.47	1.37	26.47	1.561
4C-15GC		14.83	1.46	15.80	1.528
		18.50	1.54	16.93	1.564
		22.69	1.62	19.46	1.623
		29.71	1.44	19.53	1.698
		33.42	1.35	29.41	1.526
4C-20GC		18.43	1.487	12.51	1.599
		19.75	1.521	13.20	1.615
		22.24	1.584	15.22	1.688
		25.26	1.617	19.22	1.702
		32.35	1.493	29.41	1.532

Appendix B4: Compaction test results of soil stabilized with 6% OPC/ 5-20% GC.

		Compactive Effort			
		BSL		BSH	
Material	Replacement Level (%)	MC (%)	DD (Mg/m^3)	MC (%)	DD (Mg/m^3)
6C-5GC		13.31	1.50	13.17	1.589
		16.61	1.55	13.90	1.605
		23.36	1.59	16.44	1.670
		28.60	1.47	18.22	1.716
		33.23	1.36	22.88	1.614
6C-10GC		18.43	1.499	11.48	1.58
		19.75	1.534	16.06	1.67
		20.81	1.610	19.53	1.72
		27.55	1.604	24.63	1.53
		35.29	1.458	28.16	1.41
6C-15GC		21.07	1.467	11.90	1.60
		25.40	1.465	16.08	1.67
		26.37	1.539	19.97	1.73
		27.55	1.604	25.19	1.53
		35.29	1.458	27.93	1.44
6C-20GC		18.43	1.487	12.51	1.599
		19.75	1.521	13.20	1.615
		22.24	1.584	15.22	1.688
		25.26	1.617	16.75	1.738
		32.35	1.493	22.73	1.616

Appendix B5: Compaction test results of soil stabilized with 8% OPC/ 5-20% GC.

Material	Replacement Level (%)	Compactive Effort BSL MC (%)	BSL DD (Mg/m^3)	BSH MC (%)	BSH DD (Mg/m^3)
	8C-5GC	14.85	1.45	13.17	1.589
		20.46	1.52	13.90	1.605
		25.70	1.57	16.44	1.670
		30.65	1.45	18.22	1.716
		34.38	1.35	22.88	1.614
	8C-10GC	18.43	1.487	12.51	1.599
		19.75	1.521	13.20	1.615
		22.24	1.584	15.22	1.688
		25.26	1.617	16.75	1.738
		32.35	1.493	22.73	1.616
	8C-15GC	14.51	1.50	12.51	1.599
		18.42	1.54	13.20	1.615
		24.60	1.60	15.22	1.688
		28.94	1.45	16.75	1.738
		33.28	1.33	29.41	1.532
	8C-20GC	15.80	1.528	10.42	1.629
		16.93	1.564	10.87	1.649
		19.46	1.623	12.90	1.722
		22.96	1.650	14.78	1.768
		29.41	1.526	17.79	1.683

Appendix B6: Maximum Dry Densities (MDD) of OPC/GC stabilized black cotton soil

	Maximum Proportion (%)			Maximum Dry Density MDD (Mg/m^3)	
S/No.	Soil	OPC	GC	BSL	BSH
1	100	2	0	1.47	1.55
2	100	4	0	1.53	1.57
3	100	6	0	1.59	1.70
4	100	8	0	1.60	1.66
5	100	2	5	1.53	1.66
6	100	2	10	1.57	1.68
7	100	2	15	1.59	1.68
8	100	2	20	1.62	1.68
9	100	4	5	1.57	1.70
10	100	4	10	1.58	1.68
11	100	4	15	1.59	1.70
12	100	4	20	1.62	1.70
13	100	6	5	1.60	1.72
14	100	6	10	1.61	1.71
15	100	6	15	1.60	1.72
16	100	6	20	1.62	1.74
17	100	8	5	1.61	1.72
18	100	8	10	1.62	1.74
19	100	8	15	1.61	1.74
20	100	8	20	1.65	1.77

Appendix B7: Optimum Moisture Content (OMC) of OPC/GC stabilized soil

	Maximum Proportion (%)			Optimum Moisture Content OMC (Mg/m^3)	
S/No.	Soil	OPC	GC	BSL	BSH
1	100	2	0	25.4	23.0
2	100	4	0	24.1	22.2
3	100	6	0	23.7	24.0
4	100	8	0	22.0	22.0
5	100	2	5	24.7	22.5
6	100	2	10	23.1	20.0
7	100	2	15	21.8	21.0
8	100	2	20	26.0	20.0
9	100	4	5	23.7	18.0
10	100	4	10	22.9	20.5
11	100	4	15	21.7	19.0
12	100	4	20	24.5	18.0
13	100	6	5	22.8	18.0
14	100	6	10	22.0	18.9
15	100	6	15	22.0	18.1
16	100	6	20	24.5	16.5
17	100	8	5	21.4	18.0
18	100	8	10	24.5	16.5
19	100	8	15	20.9	18.0
20	100	8	20	22.0	14.5

Appendix C1: Unconfined Compressive Strength test results for OPC/GC stabilized soil using BSL compaction

	Mix proportion (%)			Unconfined Compressive Strength (kN/m^2)		
S/No.	Soil (%)	OPC (%)	GC (%)	7days	14days	28days
1	100	2	0	280	488	568
2	100	4	0	320	1004	1214
3	100	6	0	760	1020	1560
4	100	8	0	994	1114	1694
5	100	2	5	324	388	820
6	100	2	10	354	392	908
7	100	2	15	474	446	1066
8	100	2	20	568	550	1084
9	100	4	5	462	922	1162
10	100	4	10	548	946	1026
11	100	4	15	696	976	1004
12	100	4	20	700	998	1016
13	100	6	5	838	984	1550
14	100	6	10	900	1036	1450
15	100	6	15	980	1010	1376
16	100	6	20	988	1038	1314
17	100	8	5	1012	1176	1858
18	100	8	10	1074	1258	1658
19	100	8	15	1114	1110	1492
20	100	8	20	1152	1196	1434

Appendix C2: Unconfined Compressive Strength test results for OPC/GC stabilized soil using BSH compaction

	Mix proportion (%)			Unconfined Compressive Strength (kN/m^2)		
S/No.	Soil (%)	OPC (%)	GC (%)	7days	14days	28days
1	100	2	0	608	592	1190
2	100	4	0	962	928	1250
3	100	6	0	1268	1004	1470
4	100	8	0	1436	1108	1736
5	100	2	5	714	772	1082
6	100	2	10	720	844	1258
7	100	2	15	788	946	1522
8	100	2	20	796	1002	1564
9	100	4	5	986	1014	1312
10	100	4	10	1024	1048	1554
11	100	4	15	1066	1118	1634
12	100	4	20	1094	1288	1878
13	100	6	5	1236	1054	1608
14	100	6	10	1312	1086	1778
15	100	6	15	1326	1174	1902
16	100	6	20	1432	1366	1938
17	100	8	5	1504	1282	1810
18	100	8	10	1542	1358	1874
19	100	8	15	1560	1474	1986
20	100	8	20	1568	1658	2006

Appendix C3: Soaked and Un-soaked California Bearing Ratio test results for OPC/GC stabilized soil using both BSL and BSH compaction

	Mix proportion (%)			Un-soaked CBR (%)		Soaked CBR (%)	
S/No.	Soil	OPC	GC	BSL	BSH	BSL	BSH
1	100	2	0	5.8	6.9	9.1	16
2	100	4	0	6.4	17	12.0	20.0
3	100	6	0	25	34	13.3	21.8
4	100	8	0	37.8	39	16.3	25.0
5	100	2	5	10.8	11.7	9.4	16.7
6	100	2	10	10.8	14.9	9.6	18.1
7	100	2	15	11.3	17	10.8	19.1
8	100	2	20	13.3	18.4	12.5	20.1
9	100	4	5	14.5	19.1	12.4	20.9
10	100	4	10	15.7	20.2	12.8	21.5
11	100	4	15	17.6	27	14.3	23.1
12	100	4	20	19.2	34	14.9	23.7
13	100	6	5	39.7	44	14.5	23
14	100	6	10	32	39	15.2	23
15	100	6	15	29	37	15.1	22.8
16	100	6	20	27	33	15.6	25.2
17	100	8	5	53.8	63	17.9	29.3
18	100	8	10	52.6	60.1	20.4	27
19	100	8	15	50.1	56.9	18.8	26.8
20	100	8	20	43.2	47	17.1	26

Appendix D1: Effect of GC on the Atterberg Limits of the Black Cotton Soil

GLASS CULLET %	LIQUID LIMIT (LL) %	PLASTIC LIMIT (PL) %	PLASTICITY INDEX (PI)
0% GC	51.4	32.6	18.8
5% GC	50.6	32.8	17.8
10% GC	48.2	33.0	15.2
15% GC	45.8	33.16	12.64
20% GC	42.0	35.2	6.8

Appendix D2: Effect of GC on the Linear Shrinkage of the Black Cotton Soil

GLASS CULLET %	ORIGINAL LENGTH (L)mm	CHANGE IN LENGTH (ΔL)mm	LINEAR SHRINKAGE %
0% GC	140	120.0	14.3
5% GC	140	121.0	13.6
10% GC	140	122.5	12.5
15% GC	140	124.0	11.4
20% GC	140	125.0	10.7

Appendix D3: Effect of GC on the Swelling Pressure of the Black Cotton Soil

GC (%)	0	5	10	15	20
Volume (cm^3)	28	26	24.5	23.2	23
Swelling Pressure (N/mm^2)	64.29	61.54	59.18	56.90	56.52

Appendix D4: Maximum Dry Density (MDD) of GC Stabilized Black Cotton Soil

Maximum Proportion (%)			Maximum Dry Density MDD (Mg/m^3)	
S/No.	Soil	GC	BSL	BSH
1	100	0	1.46	1.53
2	100	5	1.46	1.54
3	100	10	1.48	1.55
4	100	15	1.49	1.58
5	100	20	1.51	1.61

Appendix D5: Optimum Moisture Content (OMC) of GC Stabilized Black Cotton Soil

Maximum Proportion (%)			Optimum Moisture Content OMC (%)	
S/No.	Soil	GC	BSL	BSH
1	100	0	15.0	14.00
2	100	5	14.24	13.86
3	100	10	14.08	13.34
4	100	15	13.46	12.68
5	100	20	13.02	11.22

Appendix D6: Variation of UCS with GC (BSL) Compactive Effort

GC (%)	UCS (kN/m^2) 0days cured	UCS (kN/m^2) 7days cured	UCS (kN/m^2) 14days cured	UCS (kN/m^2) 28days cured
0	80	122	186	214
5	80.2	123	187.8	217
10	83	128	202	220
15	90	138	218	224
20	96	136	246	242

Appendix D7: Variation of UCS with GC (BSH) Compactive Effort

GC (%)	UCS (kN/m^2) 0days cured	UCS (kN/m^2) 7days cured	UCS (kN/m^2) 14days cured	UCS (kN/m^2) 28days cured
0	184	386	484	544
5	188	398	490	584
10	202	406	508	596
15	218	484	520	602
20	236	496	534	628

Appendix D8: Soaked and Un-soaked CBR test results for GC stabilized soil using both BSL and BSH compaction

	Un-soaked CBR (%)		Soaked CBR (%)	
GC (%)	BSL	BSH	BSL	BSH
0	9.5	11.2	8.7	10.4
5	10.4	11.2	10.2	11.0
10	10.8	11.8	11.3	11.7
15	11.5	12.2	11.9	12.5
20	12.5	13.5	13.8	14.7