CEMENT STABILIZATION OF BLACK COTTON SOIL USING LOCUST BEAN WASTE ASH AS ADMIXTURE

BY

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DECLARATION

I hereby declare that this thesis is a product of my effort under the supervision of Professor K. J. Osinubi. It has not been in any previous application for a higher degree. All quotations and their sources are specifically acknowledged by means of references

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2011

CERTIFICATION

The thesis titled ***"Cement stabilization of black cotton soil using locust bean waste ash as admixture"*** by **Oyelakin**, **Mutiu Adekunle** meets the regulations governing the award of the degree of Masters of Science in Civil Engineering Department of Ahmadu Bello University and is approved for its contribution to knowledge and literary presentation.

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DEDICATION

To Almighty Allah (SWT) and to the entire Ummah of Prophet Muhammad (SAW), who adhere strictly to his teachings.

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All praise is to Allah, the Lord of the Worlds, who through His infinite mercies made me a Muslim and Whose grace never left me, even for a second, and to Him is my final return. May His Peace and Blessings be upon the noble soul of Prophet Muhammad (SAW), his households, companions and those who tread his path till the end of time.

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my siblings, cousins, in-laws and all my family members, who stood by me especially during the odd days; may Allah reward you all with paradise.

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by myself. May He continue to be my guide and admit me (and my households) into paradise with ease. May His Peace and Blessings be upon the noble soul of our Prophet Muhammed (SAW).

ABSTRACT

The preliminary investigation of the black cotton soil collected from New Marte, Borno State, shows that it belongs to A-7-6 (13) in AASHTO and CH in the Unified Soil Classification System (USCS), respectively. Soil under these groups are usually very poor for engineering use which was why it was treated with an ordinary Portland cement - locust beans waste ash (LBWA) blend in stepped concentration of 0, 2, 4, 6 and 8% by dry weight of soil. Compaction was carried out using three energy levels namely: the British Standard Light, West African Standard and the British Standard Heavy. The liquid limit increased from 63% for the natural soil to 77% at 6% OPC/6% LBWA while the plastic limit decreased from 26.6% for the natural soil to 21.4% at 6% OPC/4% LBWA treatment. Unconfined compressive strength (UCS) values of 151, 324 and 637kN/m^2 recorded for the natural soil were increased to 837, 1220 and 1420kN/m^2 at 6% OPC/6% LBWA for specimens compaction using British Standard Light (BSL), West African Standard (WAS) and British Standard Heavy (BSH) energies respectively. The unconfined compressive strength (UCS) values for natural soil compacted with BSL, WAS and BSH energies at 7 days curing period are 179, 381 and 750kN/m^2 respectively and increased to 986, 1436 and 1650kN/m^2 at 6% OPC/6% LBWA treatment. The treated specimens did not attain the 7 days UCS value of 1710kN/m^2 recommended for an adequate cement stabilized base, but the value is, however, acceptable for a sub-base material. The CBR values of the

natural soil are 5, 6 and 9% for BSL, WAS and BSH energies. For the recommended value of 80%, the unsoaked CBR values of 73% at 6% OPC/6% LBWA for WAS compaction and 83% obtained for BSH compactive efforts also at 6% OPC/6% LBWA could be acceptable. The soaked CBR values obtained was 66% for both WAS and BSH compactive energies both at 6% OPC/6% LBWA; a value which meets the recommendation of 30% sub-base when compacted at optimum moisture and at 100% West African Standard compactive effort. The 6% OPC/6% LBWA blend at BSH compaction attained 71% resistance to loss in strength (when subjected to 7 days curing then 7 days soaking) as against 80% recommended for 7 days curing and 4 days soaking.

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CHAPTER ONE

INTRODUCTION

1.1 Preamble

To the Civil Engineer, soil is any uncemented accumulation of mineral particles formed by weathering of rocks. Soils are generally used as foundation or as construction materials. A class of soil, known as expansive soils, when used as foundation material, is usually affected by environmental conditions and they undergo detrimental volumetric and hydraulic conductivity changes because of the variation in moisture contents. Expansive soils are, therefore, soils with potential for shrinking or swelling under changing moisture condition (Fredlund and Rehardjo, 1993). These soils cause more damage to structures, particularly light building and pavements, than any other natural hazards, including earthquakes and floods (Jones and Holtz, 1973)

Black cotton soil (BCS) is a type of expansive soil that principally occurs in semi arid and arid regions of the tropical/temperate zones marked with dry and wet seasons; and with low rainfall, poor drainage and exceedingly great heat. The climate condition is such that the annual evapotranspiration exceeds precipitation (Chen, 1988; Nelson and Miller, 1992; Warren and Kirby, 2004). Black cotton soils are black clays that are produced from the breakdown of basic igneous rocks where seasonal variation of weather is extreme. The Nigerian black cotton soils are formed from the weathering of shaly and clayey sediments and basaltic rocks.

They contain more of montmorillonite with subsequent manifestation of swell properties and expansive tendencies (Ola, 1983).

Although poor and undesirable for engineering purposes, its properties could be improved to meet standard specification by modification/stabilization processes. Stabilization of the soil with chemical admixtures is a common method of reducing the swell – shrink tendencies of the soil and also makes the soil less plastic (Ola, 1983; Balogun, 1991; Osinubi, 1995; 1999). Cement stabilization is one of the most widely used means to improve the shrink – swell characteristics of the soil (Matawal and Tomarin, 1996).

More recent research works (Mohammedbhai and Bagant, 1990; Osinubi, 1995; 1998a, b; 1999; 2000a, b; Cokca, 2001; Medjo and Riskowski, 2004; Stephen, 2006, George, 2006; Akinmade, 2008; Ochepo, 2008) in the field of geotechnical engineering focus on the search for cheaper and locally available materials for use in stabilization; and a large percentage of such materials are agricultural wastes that produce cementituous compounds on exposure to moisture. These studies try to match the need for safe and environmental disposal of waste, for the society, and the engineer's need for better and cost effective construction materials (Collins and Ciesiellski, 1993).

1.2 Statement of the problem

In Nigeria, black cotton soil occupies an area of about $10.4x10^4$ Km^2 in the north eastern part (Ola, 1983); including major roads linking the country with

neighbouring West African countries like Niger, Chad Republic and Cameroun. These roads are so poor that it takes vehicles plying them about three times as much time as they are supposed to spend; and with high degree of damages to their vehicles. This poses a danger to the economy of the country because the roads carry an estimated 250,000 tones of assorted goods from Nigeria to these countries and vice versa (Diden, 2001). This calls for more concern especially at a time members of the African Union (AU), are clamouring for Free Trade Zones; to promote international trades within the continent. Furthermore, the country is aspiring to be among the most successful economies of the world. These therefore pose a big challenge to the geotechnical engineers in their efforts to forstall our bad image from the neighbouring countries using these roads and in their own way, help the country to achieve her set goals.

In line with the above challenges, the geotechnical engineers are working hard to provide the most cost effective approach to shallow and deep ground improvement techniques so as to meet the engineering requirements necessary for the design and construction of stable and durable infrastructure facilities. The choice of conventional method of stabilization has been employed successfully but with a relatively high cost. Consequently, recent researches have focused on the use of locally available (agricultural) materials with pozzolanic properties, as possible substitutes or admixtures to achieve more economic stabilization of the soil.

Most recent research (Akinmade, 2008) in the Department of Civil Engineering, Ahmadu Bello University, Zaria, focused on the use of locust bean waste ash (LBWA), the product of combustion of the waste husk of locust bean pod, to stabilize black cotton soil. The result showed that LBWA has a good potential in improving some of the geotechnical properties of the soil. This research is intended to use LBWA as an admixture in the cement stabilization of black cotton soil.

1.3 Justification for the Study

For highly cohesive soils like black cotton soil (BCS), a number of guidelines for determining the suitability of the soil for any type of modification/stabilization have been proposed by researchers on the basis of major soil components' responses to modification/stabilization (Osinubi, 1995). The efficacy of lime and Portland cement stabilization of problem soils have been highlighted by researchers (Osinubi and Umar, 2003; Umar and Elinwa, 2005; Matawal and Tomarin, 1996), and hydrated and quicklime have been found to be the most effective agents in reducing the swelling properties of expansive soils (Osinubi, 1997). But in regions where lime is scarce, cement has been found to be a suitable alternative. Cement has therefore been confirmed as one of the most successful stabilizers for the improvement of the geotechnical properties of expansive soils (Ola, 1974; Osinubi, 1999).

The cost of blending these soils with cement is usually exhorbitant. To minimize the cost of construction of stabilized roads and airfield, a practical alternative is to mix the cement–soil blend with requisite quantity of a cheap admixture (like locust bean waste ash). Since the potential of LBWA as a stabilizer has been established, there is need, therefore, to assess its effect as an admixture in cement stabilization of soil. This is with a view to reduce the amount of cement to be used for stabilization and consequently to reduce the cost of stabilization.

1.4 Aim and Objectives

The aim of this research is to establish the effect of using locust bean waste ash, as an admixture, on the geotechnical properties of cement stabilized black cotton soil.

The objectives are highlighted below.

i. Determination of the natural properties of the black cotton soil
ii. Determination of the properties/oxide composition of LBWA
iii. Evaluation of the effect of LBWA (0, 2, 4, 6 and 8% by dry weight of soil) on the index and strength properties of the black cotton soil stabilized with cement (0, 2, 4, 6, and 8% by dry weight of the soil)
iv. Determination of the optimum amount of cement and LBWA admixtures needed for the stabilization of the soil.

1.5 Scope of Research

The research was limited to finding the engineering properties of stabilized black cotton soil when used as highway pavement material under changing environmental conditions (e.g. access to excessive moisture content).The oxide compositions of locust bean waste ash and ordinary Portland cement were determined. The optimum percentages that produced an economic mix proportion for the stabilization of the black cotton soil if used as fill material, in line with the Nigerian General Specification (1997), requirements for road contraction materials were determined.

CHAPTER TWO

LITERATURE REVIEW

2.1 Expansive Soils

Expansive soil refers to soil material that has the potential for swelling and shrinking due to changing moisture conditions. The major problem is that its deformations are significantly greater than elastic deformation, thereby preventing its prediction by simple classical elastic or plastic theories. The movement is usually in an uneven pattern and of such as magnitude as to cause extensive damage to the structures and pavements resting on them (Nelson and Miller, 1987). Expansive soils cause more damage to structures particularly light buildings and pavements than any other natural hazard, including earthquakes and floods. The annual cost of these damages to civil engineering structures is estimated at one billion dollars in the United States and many more billions of dollars worldwide (Gourley et al., 1993).

Expansive soils can be found anywhere in the world but they are basically confined to semi–arid and arid regions of the tropical/temperate zones. These areas are naturally characterized by marked dry and wet seasons with low rainfall, poor drainage and exceedingly great heat. The climate condition is such that the annual evapotranspiration exceeds the precipitations (Chen, 1988). They are found in the north-eastern part of Nigeria, Cameroon, Lake Chad, Sudan, Ethiopia, Kenya, South Zimbabwe, South Africa and other Eastern African countries. They are also found in India, Australia, South- Western USA and Israel (Ola, 1978).

Two groups of parent materials have been associated with the formation of expansive soils. The first group comprises of sedimentary rocks of volcanic origin which can be found in North America, South Africa and Israel (Ola, 1978) while the second group of parent materials are basic igneous rocks found in India, Nigeria and South–Western USA (Plait, 1953). The most well–known example of expansive soils is the black cotton soil which is dark–grey to black in colour and the name originated from India where locations of these soils are favourable for growing cottons. It is also the Nigerian type of expansive soils (Osinubi, 1999).

2.1.1 Origin of black cotton soil

Black cotton soils (BCS) are black clays that are produced from the breakdown of basic igneous rocks, where seasonal variation of weather is extreme. Nigerian black cotton soils are formed from the weathering of shaly and clayey sediments and basaltic rocks. The Nigerian BCS contains more of montmorillonite with subsequent manifestation of swell properties and expansive tendencies (Ola, 1983).

The parent igneous rocks are made up of calcium-rich feldspar and dark minerals which are high in the weathering order and all the constituents are weathered to form amorphous hydrous oxide and under suitable conditions clay minerals develop. The absence of quartz leads to the formation of fine grained plastic soil highly impermeable and easily waterlogged. Other conditions favouring the formation of BCS are evaporation exceeding precipitation, poor

leaching, alkaline conditions and retention of magnesium and calcium in the soil (Ola, 1983).

2.1.2 Mineralogy and chemical composition of black cotton soil

Montmorillonite and kaolinite are the two predominant clay minerals found in BCS. Ola (1983) showed that the Nigerian BCS contains about 70% montmorillonite and 30% kaolinite. The swell and shrink characteristics of the soil are largely due to montmorillonite minerals while kaolinite is likely responsible for high strength value because of its inability to swell with adsorbed water.

The montmorillonite clay structure consists of layer sheet formed and stacked one above the other and the bonding between successive layers is by van der Waals forces and by cation that may be present to balance charge deficiencies in the structure. These bonds are, however, weak and easily separated by cleavage or adsorption of water and other liquids. There is an extensive isomorphous substitution for aluminium and silicon within its lattice which gives the clay a net negative charge resulting in the water absorbing tendencies and an attraction for hydroxyl ions and water molecules to the clay surface (Nelson and Miller, 1992).

2.1.3 The effect of black cotton soil on highway pavements

The most vulnerable engineering structure susceptible to damage due to shrink-swell potential (volume change) of black cotton soil are the highway pavements because of their light weights and very shallow foundations. The

multi–layered flexible road pavement is a complex structure made up of materials whose engineering properties are very much affected by the loading to which they are subjected and also by the environmental conditions (Aitken, 1971)

A road pavement is expected to fulfill two basic functions: to remain structurally sound to satisfy the engineer's design; and to provide a satisfactory ride to the road user during its design life. Unfortunately and very frequently in practice, road pavements do not fulfil their basic functions especially when they are constructed on expansive soils. There is, therefore, a need to incorporate an economical soil stabilization process before these roads are constructed to forstall these unfortunate results, because experience has clearly shown that the cost of repairs (of bad roads) is very much higher than the cost of a proper initial design, and the results are much less satisfactory (Krazynski, 1980).

2.2 Soil Stabilization

Soil stabilization is the improvement of the original soil properties to meet specific engineering requirements. It is aimed at the enhancement of the engineering properties of deficient soils to enable them perform and sustain their intended engineering use (Yoder and Witczak, 1975; Gillot, 1987; Osinubi, 1995; Nicholas and Lester, 1999). Its objectives are: improvement of the strength of the soil and bearing capacity, decreasing permeability and water absorption, and to increase the durability under varying moisture content.

2.2.1 Mechanical stabilization

The stabilization of soils through mechanical means includes compaction, vibration (of various techniques) and blasting. Mechanical stabilization by compaction is the densification of the soil by the application of mechanical energy. It involves the modification of the water content as well as the gradation of the soil.

Cohesionless soils are compacted by some means of confining the soil coupled with vibration energy. In the field, hand–operated vibrating plates and motorized vibratory rollers of various sizes are very efficient in compacting sand and gravely soils. Large falling weights have been used to dynamically compact loose granular fills (Markwick, 1944)

Fine grained cohesive soils are compacted in the field by using common compaction equipments like hand–operated tampers, sheeps foot rollers, rubber–tyred rollers and other types of specialty equipment. Considerable compaction can also be obtained by proper routing of the hauling equipment over the loose soil. The objective of mechanical compaction is the improvement of the engineering properties of the soil mass, and the advantages which are obtained through compaction are as follows (O' Flaherty, 1988):

i. Reduction in settlement due to reduced void ratio

ii. Increase in soil strength

iii. Reduction in shrinkage.

2.2.2 Chemical stabilization

Generally, the addition of chemical compounds, to expansive soils increase the strength, bearing capacity and durability of the soil. These organic or inorganic chemical compounds perform as cementatious and bonding agents or water proofers/repellants (Slate and Johnson, 1953).

Organic compounds including resinous and bituminuous materials act as water proofers and sometimes behave similar to glue. These water proofing agents reduce the capacity for water intake and help the soil to retain its dry strength, even under wet conditions (Bowles, 1979).

Inorganic agents employed for soil stabilization include Portland cement, lime, slag, sodium silicate, etc. Their functions are to reduce plasticity and facilitate densification (Balogun, 1991).

2.2.2.1 Lime stabilization

The use of lime for stabilization dates back to early times. Its principal use is in highway construction and maintenance, airfields construction, building foundation, rail road beds and under hydraulic conditions where the soil is partly or wholly submerged (e.g. irrigation canals, reservoirs, levees and dams). Lime stabilization has been used successfully on many projects to minimize swelling

and improve soil plasticity and workability. Generally, from 3 to 8% by weight of hydrated lime is added to the top several inches of the soil. It is also used as a follow-up treatment over ponded area to add strength to the surface, provide a working table for equipment and prevent evaporation by capillary rise from the treated layer below (McDowell, 1965; Teng et al, 1973)

Lime has been considered to be the most appropriate for the stabilization of highly clayed soils having fine contents in excess of 25% because it makes the soil more friable, less plastic and hence easier to work. The reactions of lime with soils result in strength gain mainly from chemical reactions between the lime, clay-grade minerals and amorphous constituents in the soil.

Improvement in the soil properties for soil stabilization has been attributed to the soil–lime reaction (Clare and Cruchley, 1957; Locat et al., 1990). The cation exchange, the pozzolanic reactions and carbonation are the mechanisms used in explaining the chemical changes occurring in the soil properties for soil–lime mixes (O'Flaherty, 2002). Cation exchange is believed to be mainly responsible for the change in plasticity of the soil, and magnitude of this change is affected by the soil clay mineralogy. Montmorillonite soils show the greater change, the effect in illite-chlorite is intermediate and effect in kaolinite soil is less (Hitt and Davidson, 1960).

2.2.2.2 Cement stabilization

Stabilization of soils with cement usually involves the mixing of predetermined quantities of the additive with pulverized soil particles; and the mixture is termed ‘soil-cement’. Soil-cement mixtures have been used for road sub base or base course for many years. Stabilization of soil with ordinary Portland cement (OPC) producers hardened materials which are capable of bearing loads for engineering purposes. TRRL (1977) and CEBTP (1980) recommended a minimum of 15% for soil fraction passing 0.425mm sieve and plasticity index greater or equal to 10. Generally, gravels require about 10% by dry weight of cement, sand requires about 7–10%, silt about 12 to 15% and clays, 12 to 20% by dry weight of cement. (Gillot, 1987)

Hydration is the first phase of the cement stabilization reaction. Cement hydration produces a cementitious compound; a product of cement and water. It generates bonding between the reaction products (Calcium–silicate hydrates and aluminium hydrates) and the soil particles. It results in agglomeration and flocculation of the clay particles due to exchange of ions at the surface of the particles which manifest in an early strength development, immediate swell, shrinkage and plasticity reduction

In highway construction, Portland cement is normally added to soils at optimum moisture content (OMC) and allowed to cure while the soil cement mixtures hardens by the process of hydration. Yoder and Witzak (1975), Nelson and Miller (1992), Indratna et al. (1995) reported decrease in liquid limits, plastic limits and swelling potentials of clays when treated with cement. However,

researchers such as Maclean (1953), Ingles and Metcalf (1972) and Ibrahim (1983) reported that cement stabilization may not be effective for soils of high plastic limit (of about 20%) or liquid limit in excess of about 45 to 50% and high organic matter as well as montmorillonite–rich soil.

Kedzi (1979) identified two types of reaction for soil-cement mixtures. The first being the hydration of cement while the second is the reaction between the free lime product of hydration of cement and alumina of the clay fraction of the soil. The primary reaction results in the formation of calcium silicate hydrates (CSH), which are cementing substances and are responsible for the initial strength development; while the secondary reaction is pozzolanic in nature, and it is responsible for the time dependent gain in strength. The amount of reduction of maximum dry density (MDD) is dependent on the rate of hydration which is expected to decrease with increase in both time and cement content. O'Glensby and Hicks (1982) attributed the effects of OPC in the strength properties of soil as (1) the surface action of cement quickly produces flocculation and reduces the moisture affinity of clay soils; and (ii) the reaction which is time dependent, promotes the cementation of the soil cement mix. When OPC hydrates, it liberates calcium silicates and aluminum ions into the water and these subsequently combine to form hydrated calcium silicate, which constitute the matrix of the hardened cements (Neville and Brooks, 1994)

2.2.3 Admixture stabilization

As it has been proven that the conventional stabilization of expansive soils with lime or cement or both is effective; however the cost of these stabilizers is high thereby making the process uneconomical. In various attempts to achieve an economically effective stabilization of black cotton soil, many chemical/agricultural or industrial additives have been mixed with lime or cement or both.

2.2.3.1 Chemical admixture stabilization

Balogun (1991) indicated that a significant increase in the geotechnical properties of black cotton soil is achieved with addition of 2% sodium chloride with lime–clay mixture. An increase in dry unit weight as well as the thioxotropic strength of black cotton soil is also reported by Sambhandharasksa and Moh (1971) on the admixture of high sodium chloride with lime. Maclean (1953) used water soluble calcium chloride simultaneously with cement to stabilize clay soils with appreciable amount of organic matter; noting that the absorption capacity of the organic matter for calcium ions permits the calcium from OPC to complete reaction with the other compounds in the normal way.

2.2.3.2 Industrial / agricultural wastes admixtures stabilization

It is worthy to note that most industrial or agricultural wastes possess pozzolanic properties, that is, having cementituous tendencies on exposure to moisture (O'Flaherty, 1988). Pozzolanas are, therefore, siliceous and aluminuous

materials which themselves possess little or no cementituous value but, will, in the presence of moisture, chemically react with calcium hydroxide at ordinary temperature, to form compounds possessing cementitous properties (Robert, 1993).

In line with the above, recent researchers in the Department of Civil Engineering, Ahmadu Bello University, Zaria, have focused on the use of these wastes as possible stabilizers/admixtures. Studies have shown that solid minerals and agricultural wastes could be used for the stabilization of expansive soils (Fakiyesi and Osinubi, 1995; 1996; Osinubi, 1997). Their admixtures with lime or cement or both have also been considered in improving the engineering properties of expansive soils (Osinubi and Toro, 1996; 1997; Osinubi and Medubi, 1997a, b). Recent studies have also focused on the use of bagasse ash as possible stabilizers/admixtures for the stabilization of expansive soils. (Osinubi and Stephen, 2005; 2006a, b; 2007; 2009; Osinubi and Mustapha, 2005; 2008; Osinubi and Eberemu, 2006; Osinubi et al., 2007a, b, c; 2008a, b, c). Stephen (2006) studied the potential of bagasse ash as stabilizer on black cotton soil and George (2006) and Osinubi et al (2008a, b) both studied the potential of admixed baggase ash with lime and cement in stabilizing black cotton soil.

Another agricultural waste (Locust Bean Waste Ash) was studied by Akinmade (2008) who checked its effect on the geotechnical properties of black cotton soil. The results obtained showed some favourable improvements on the geotechnical properties, especially with the reduction of the swelling pressure

from 42.6 to 32.2kN/m^2; but it was, discovered that the ash could not be used as a stand–alone stabilizer because of its low calcium oxide content.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Materials

3.1.1 Black cotton soil

The disturbed soil samples used for this study were collected at the Chad Basin Development Authority (CBDA) reserved site at New Marte (Latitude 13^0 27'N and longitude 13^0 50'E) along the Maiduguri–Gamboru Road in Borno State. The top soil was removed to a depth of 0.5m before the soil samples were taken, sealed in plastic bags and put in sack to avoid loss of moisture during transportation. The soil samples were then allowed to dry before pulverizing to obtain particles passing sieve BS No. 4.

Although it was reported by Nelson and Miller (1992) that there is no standard classifications for expansive soils, different methods are practically employed in different parts of the world. NBRRI (1983) classified the variability of black cotton soil in terms of particle size distribution, clay and silt content, liquid and plastic limits as well as swelling potentials; as summarized in Table 3.1.

Table 3.1: Categorization of Black Cotton Soil of North–Eastern Nigeria

S/No	Plasticity Index (%)	Free Swell (%)	% Lesser than 1µm	Swelling Potential
1	< 20	<50	<20	Low
2	15 – 30	50 – 80	15 – 30	Medium
3	> 30	> 80	> 30	High

(Source NBRRI, 1983))

3.1.2 Locust bean waste ash

The locust bean waste ash (LBWA) used for this study were obtained locally from the burning of locust bean husks sourced from Doko village around Bida in Niger State and Malali village in Kaduna State. The husks were completely burnt under atmospheric condition, sealed up in plastic bags and transported to the laboratory. The ash was then passed through British Standard No 200 sieve and kept to be mixed with the 'soil-cement' in the appropriate percentages.

The oxide composition of the ash was determined at the Centre for Energy Research and Training (CERT), A.B.U Zaria by the method of Energy Dispersive X-Ray Flourescence. LBWA is a pozzolana and the classification of pozzolanas as given by ASTM C618-78 is summarized in Table 3.2

Table 3.2: Properties of Pozzolanas

Property	**Class N**	**Class F**	**Class C**
Chemical Properties			
$SiO_2 + Al_2O_3 + Fe_2O_3$ (%)	70	70	50
SO_3 (Max %)	4	5	5
MgO (Max %)	5	5	5
Loss on ignition	10	2	6
Physical Properties			
Moisture content (%)	3	3	3
Fineness (%) on sieve No. 200 (mm)	85	85	85
Pozzolanic Activity			
Index with OPC at 28 days (%)	75	75	75
Pozzolanic Activity Index with lime at 7 days	5.5	5.5	5.5

Source: (**ASTM, 1978)**

3.1.3 Cement

The ordinary Portland cement (OPC) used for the study was purchased in the open market at Samaru market, opposite A.B.U, Zaria. A typical oxide composition of OPC is given in Table 4.3.

3.2 Methods

3.2.1 Natural Moisture content

The natural moisture content of the soil as obtained from the site was determined in accordance with BS 1377 (1990) Part 2. Three weighing containers were cleaned and weighed to the nearest 0.01g (M_1). The sample as freshly collected was crumbled and placed loosely in the containers and the containers with the samples were weighed together to the nearest 0.01g as M_2. The containers were then placed in the oven and dried at 105^{O} C -110^{o}C for 24 hours. The containers and the samples were then removed and weighed dry to the nearest 0.01g as M_3. The natural moisture content (as collected from the site) is calculated as the average as given by eq. (3.1)

$$w = \frac{M_2 - M_1}{M_3 - M_1} \times 100 \quad \text{----------} \quad (3.1)$$

where w is the moisture content in percentage.

3.2.2 Specific gravity

The determination of specific gravity was carried out according to BS 1377 (1990) test (B) for fine–grained soils. The density bottle and the stopper were weighed to the nearest 0.001g (m_1). The air dried soil was transferred into the density bottle, and the bottle, content and the cover were weighed as m_2. Water was then added just enough to cover the soil, the solution is gently stirred to remove any air bubble. The bottle was then completely filled up and covered. The covered bottle was then wiped dry and the whole weighed to the nearest 0.001g (as m_3). The bottle was subsequently emptied and filled completely with water, wiped dry and weighed to the nearest 0.001g (m_4). The specific gravity is calculated using eqn. (3.2)

$$G_S = \frac{m_2 - m_1}{(m_4 - m_1) - (m_3 - m_1)} \quad \text{---------- (3.2)}$$

The same procedure was repeated for all the percentages of the admixtures, and the results are shown in Table A3.1

3.2.3 Linear shrinkage

The test was conducted in accordance with Test 5 BS 1377 (1990). It involves the mixing of about 125g of soil, passing the 425 BS sieves, with water in order to obtain a homogenous paste (the water added to the natural soil corresponded to the moisture content at liquid limit). The paste was then placed in the shrinkage mould and vibrated gently in order to expel air pockets from the mixture. The soil was then leveled by spatula and air dried at 60^{o}C. Initially, the soil shrank clear of the mould. Subsequent drying was made at 105^{O}C–100^{0}C in an oven in order to complete the shrinkage. On cooling, the length of the sample was measured with a ruler and the linear shrinkage was calculated using eqn. (3.3)

$$\text{Linear Shrinkage} = \frac{\text{(Initial lenght - Dried lenght)}}{\text{Initial Length}} \times 100\,\% \quad \text{----------} \quad (3.3)$$

3.2.4 Free swell

The test was conducted in accordance with the United States Bureau of Reclamation (USBR) method (Holtz and Gibbs, 1956). About 10 g of soil passing BS No 4 sieve (425μm aperture) was oven dried and allowed to cool in a dessicator. 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 homegous mixture of soil and water after which it was allowed to settle for 2 hours or more before the swell volume was recorded.

$$\text{Free Swell} = \frac{\text{Final Volume - Initial Volume}}{\text{Inital Volume}} \times 100 \text{ ---------- (3.4)}$$

3.2.5 Particle size distribution

The particle size analysis test carried out in accordance with BS 1377; 1990 Part 2. Wet sieving was conducted by measuring 200g of the soil sample and soaking it for 24hours. The sample was then washed through BS sieve No 200. The particles retained were then dried in the oven for 24 hours and a dry sieving was carried out on the dried sample to obtain the particle size distribution. The test summary is in Table A3.2

3.2.6 Atterberg limits

The test includes the determination of the liquid limits, plastic limits and the plasticity index for the natural soil and the stabilized soils. They were also conducted in accordance with Test 1(A) BS 1377 (1990) Part 2 for the natural soil and BS 1924 (1990) for the stabilized soils.

3.2.6.1 Liquid limits

The soil sample for liquid limit was air dried and 200g of the material passing through BS No 4 sieve (425μm aperture) was obtained and thoroughly mixed with water to form a homogeneous paste on a flat glass plate A portion of the soil water mixture is then placed in the cup of the Casangrande apparatus, leveled off parallel to the base and divided by drawing the grooving tool along the

diameter through the centre of the hinge. The cup was then lifted up and drops by turning the crank until the two parts of the soil come into contact at the bottom of the groove. The number of blow at which that occurs in recorded and a little quantity of the soil was taken and its moisture content was determined as section 3.2.1. The test was performed for well–spaced out moisture content from the drier to the wetter states. The values of the moisture content (determined) and the corresponding number of blows was the plotted on a semi–logarithmic graph and the liquid limit was determined as the moisture content corresponding to 25 blows.

This same procedure was then repeated for each of the sample with cement and LBWA admixture for the percentages listed earlier; and Table A3.3 gives the liquid limit results for the natural and the stabilized soils with for the various percentages of cement and LBWA mixes.

3.2.6.2 Plastic Limit

A portion of the soil/soil–cement–LBWA mixes used for the liquid limit test was retained for the determination of plastic limit. The ball of the soil/soil admixtures was moulded between the fingers and rolled between the palms of the hand until it dried sufficiently (even though the soil was already relatively drier than the ones used for liquid limit). The sample was then divided into approximately four equal parts. Each of the parts was rolled into a thread between the first finger and the thumb. The thread was then rolled between the tip of the fingers of one hand and the glass. This continued until the diameter of the thread is

reduced to about 3mm in five to ten forward and backward movements of the hand. The movement continued until the thread shears both longitudinally and transversely. The crumbled soil was then put in the moisture container and the moisture content determined. Table A3.4 gives the result of plastic limit for the various soil-cement-LBWA mixes.

3.2.6.3 Plasticity Index

The plasticity index of the soil/soil–cement–LBWA mix is the difference between the liquid limits of the natural/various mixes of the soil and their corresponding plastic limits. Table A3.5 shows the plasticity index of the samples which were calculated as:

$$PI = LL - PL \quad \text{---------- (3.5)}$$

3.2.7 Compaction characteristics

3.2.7.1 Maximum dry density

The compaction tests were carried out for the natural soil and the stabilized soils (in different percentages); all according to BS 1377 (1990) Part 4, using the British Standard Light, West African Standard and the British Standard Heavy, in accordance with the Nigerian General Specification (1997)

3kg of the soil/soil-admixtures sample were mixed thoroughly with 8% of water (and the water is added at 8% for each of the compactions). The sample was then compacted into the 1000cm^3 (of mass m1); in three layers of approximately

equal mass with each layer receiving 27 blows of 2.5kg rammer falling through a height of 300mm, for the British Standard Light compaction; 10 blows of 4.5kg rammer in five layers for West African Standard compaction and 27 blows of 4.5kg rammer in fire layers for the British Standard Heavy. The blows were uniformly distributed over the surface of each layer. The collar was then removed and the compacted sample leveled off at the top of the mould with a straightedge. The mould containing the leveled sample was then weighed to the nearest 1g, $m_{2.}$

Two small samples were then taken from the compacted soil for the determination of moisture content. The sample was then removed from the mould, crushed and water added (8%) and the same procedure was repeated until minimum of five sets of samples were taken for moisture content determination. The bulk density in Mglm^3 was later calculated for each compacted layers using:

$$\rho = \frac{m_2 - m_1}{1000} ------------------------------(3.6)$$

The dry density was also calculated using the equation:

$$\rho_d = 100\rho/(100+W) ---------------------------(3.7)$$

where w is the moisture content of each compacted layer.

The values of the dry densities as obtained from eqn. (3.7) were plotted against their respective moisture contents and the Maximum Dry Density, MDD was

deduced as the maximum point on the resultant curves. Tables A3.6a, b, and c, give the maximum densities for each of the compactions.

3.2.7.2 Optimum moisture content

The corresponding values of moisture contents at maximum dry densities (MDD), deduced from the graph of dry density against moisture contents, gives the optimum moisture contents (OMC). The results are given in Tables A3.7a, b, c for each compactive effort.

3.2.8 Strength characteristics

3.2.8.1 Unconfined compressive strength

The unconfined compressive strength (UCS) tests were performed on the soil samples according to BS 1377; 1990 Part 7 using the British Standard Light (BSL), West African Standard (WAS) and British Standard Heavy (BSH) energy levels. The natural soil samples and the treated soil samples were compacted in 1000cm^3 moulds at their respective OMC. The samples were extruded from the moulds and trimmed into a cylindrical specimen of 38.1mm diameter and 76.2mm length. The three cylindrical specimens from the mould were cured for 7 days. At the elapsed day of curing, the specimens were then placed centrally on the lower platen of a compression testing machine and a compressive force in applied to the

specimen with a strain control at 0.10% mm. Record was taken simultaneously of the axial deformation and the axial force at regular interval until failure of the sample occurs. The UCS of the sample was determined at the point on the stress–strain curve at which failure occurred. The same was done for 14 and 28 days curing and the UCS was calculated from the following equation:

$$Compressive\ \text{strength} = \frac{\text{Failure load}}{\text{Surface Area of Specimen}} \text{ – – – – – – – – – – – – – – –} (3.8)$$

Test results are summarized in Tables A3.8 (a)-(c) for 7 days curing period, Tables A3.9 (a)-(c) for 14 days curing period and Tables A3.10 (a)-(c) for 28 days curing, for the three compactive efforts.

3.2.8.2 California bearing ratio

The California bearing ratio (CBR) test was conducted in accordance with BS 1377(1990) for the natural and treated soils. The CBR is expressed by the force exerted by the plunger and the depth of its penetration into the specimen; it is aimed at determining the relationship between force and penetration.

3.5kg of the soil sample/soil-admixture sample were mixed at their respective optimum moisture contents in 2360cm^3 mould, at the three compactive energies. For BSL, the compaction was in three layers each receiving 62 blows from the 2.5kg rammer; for the WAS, the samples were compacted in five layers

with 25 blows from the 4.5kg rammer, and the samples received 62 blows in 5 layers from the 4.5kg rammer for the BSH.

The base plates were removed (after compaction) and the compacted specimens placed in sealed plastic bags for curing (for 6 days) and after the 6^{th} day the specimens were immersed in water for 24 hours before testing according to the specification by the Nigerian General Specification (1997). The base plates were later replaced and the specimens transferred to the CBR testing machine and positioned on the lower plate of the machine. The plunger was then made to penetrate the specimen at a rate of 1.3mm/min until the specimen failed. The mould was then inverted, base plate removed and the procedure repeated for the base of the specimens.

From the values of the penetration and force recorded, a curve of force against penetration was obtained. The measured load for the CBR was obtained using penetration of 2.5mm or 5.0mm (the greater of the two values) and as their average where the value are within 10% of each other. The CBR was calculated as:

$$\text{CBR} = \frac{\text{Measured load}}{\text{Standard load}} \times 100\% \text{ ---------- (3.9)}$$

where standard load = 13.24kN of 2.5mm penetration

= 19.96kN of 5.0mm penetration

Test results are summarized in Tables A3.12 (a)-(c) for CBR values of the unsoaked samples and Tables A3.13 (a)-(c) for CBR values for the soaked samples.

3.2.9 Durability Assessment (Resistance to Loss in strength)

The durability assessment (under adverse field conditions) of the soil sample was determined by resistance to loss in strength when immersed in water. It was expressed as the ratio of UCS of the specimen cured for 7 days and soaked for another 7 days to the UCS of the specimen cured for 14 days. Ola (1974) proposed 80% resistance to loss in strength for the ratio of the UCS of a specimen cured for 7 days and soaked for 4 days to the UCS of the specimen cured for 14 days.

$$\text{Resistance to loss in strength}(\%) = \frac{\text{UCS}(7 \text{ day cured} + 7 \text{ day soaked})}{\text{UCS}(14 \text{ day cured})} \times 100\% \quad \text{----------(3.10)}$$

Test results for 7 days soaking then 7 days curing UCS values are summarized in Tables A3.11 (a)-(c) while the test results for the resistance to loss in strength are shown in Tables A3.14 (a)-(c).

CHAPTER FOUR

DISCUSSION OF RESULTS

4.1 Properties of Materials Used in the Study

4.1.1 Natural soil

Preliminary tests conducted to determine the natural properties of the soil revealed that the soil has very high moisture content of 35%, due the period of its collection (in September during the rainy season). The index properties are summarized in Table 4.1, while its oxide compositions are summarized in Table 4.2. The soil belongs to the CH group in the Unified Soil Classification System (ASTM, 1992) or A-7-6(13) soil group of the AASHTO soil classification system (AASHTO, 1986). The soil is grayish black in colour (from wet to dry states) with a liquid limit of 63%, plastic limit of 27% and plasticity index of 36%.

The soil has a free swell of about 75%, soaked CBR values of 5%, 6% and 9% for the three energy levels of British Standard Light, West African Standard and the British Standard Heavy, respectively and UCS values of 151, 324 and 637kN/m^2 for the energy levels, respectively. The soil was found to be highly plastic and falls below the standard recommendation for most geotechnical construction works especially for sub-base or base courses in highway construction (Butcher and Sailie, 1984; Osinubi and Medubi, 1997).

Table 4.1: Properties of the Natural Soil

Property	Quantity
Percentage passing BS No 200 sieve, %	71.0
Natural Moisture Content, %	35.0
Liquid Limit, %	63.0
Plastic Limit, %	27.0
Plasticity Index, %	36.0
Linear Shrinkage, %	17.0
Free Swell, %	75.0
Specific Gravity	1.94
AASHTO Classification	A-7-6(13)
USCS	CH
NBRRI Classification	High swell potential
Maximum Dry Density, Mg/m^3	
British Standard light	1.34
West African Standard	1.45
British Standard heavy	1.53
Optimum Moisture Content, %	
British Standard light	24.0
West African Standard	21.0
British Standard heavy	19.0
Unconfined Compressive Strength, kN/m^2	
British Standard light	220
West African Standard	419
British Standard heavy	824
California Bearing Ratio, %	
British Standard light	5
West African Standard	6
British Standard heavy	9

Colour	Greyish black
Dominant clay minerals	Montmorillonite

Table 4.2 Oxide Composition of Black cotton soil

Oxide	**Concentration (%)**
CaO	-
SiO_2	31.01
Fe_2O_3	4.74
Al_2O_3	16.19
MnO	0.13
TiO_2	1.34
LOI,Others	≤ 52

4.1.2 Additives

The little amount of calcium in LBWA (1.08%) to be used for the stabilization process was supplemented by the calcium present in a typical of OPC (of 63%). The higher silicon and aluminium oxides in LBWA also aided those in OPC to provide the required oxides needed for stabilization. Table 4.3 shows the summary of the oxide compositions of LBWA and OPC.

Table 4.3 Oxide composition of LBWA and typical OPC

Oxide	**LBWA (%)**	***OPC (%)**
CaO	1.08	63
SiO_2	55.38	20
Al_2O_3	14.93	6
Fe_2O_3	0.278	3
MnO	0.09	-
Na_2O	0.18	1
K_2O	2.00	-
SO_3		2
P_2O_5	0.23	-
Loss on Ignition	10.63	2

*After Czernin, 1962

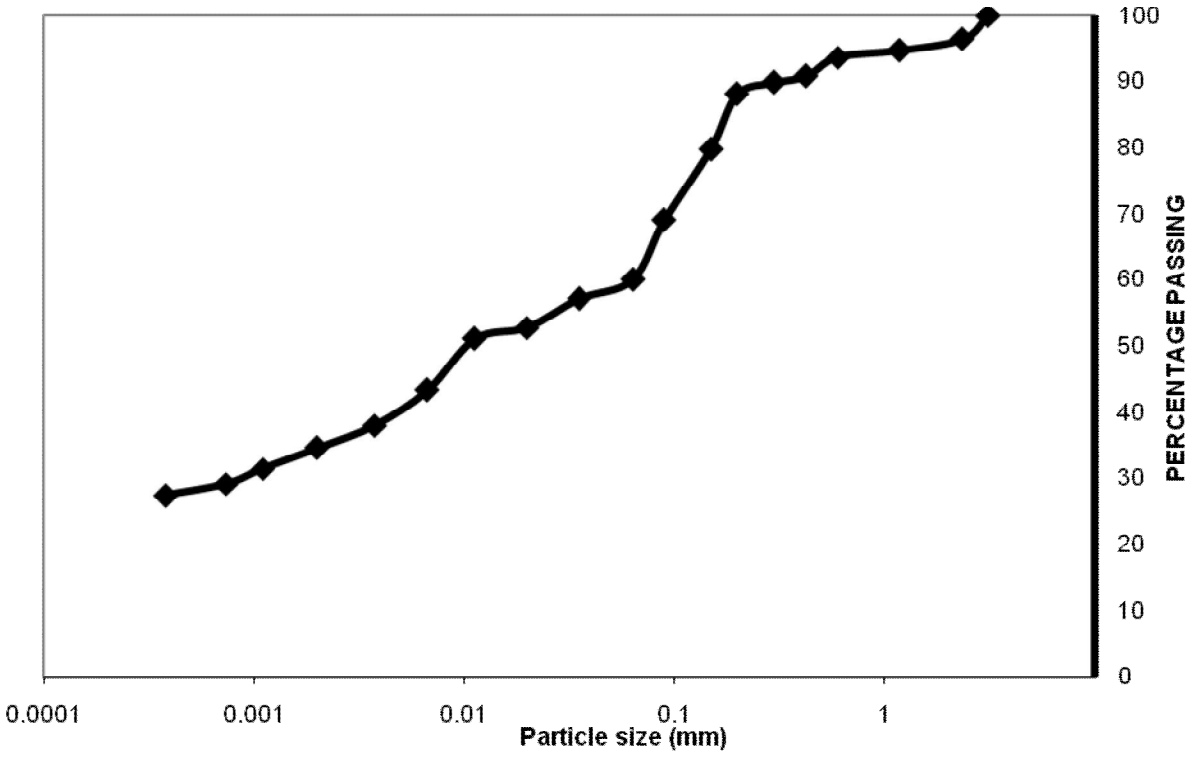


Fig. 4.1: Particle size distribution curve of the natural soil

4.2 **Atterberg Limits**

4.2.1 **Liquid limit**

The variation of liquid limit of the soil/soil-cement mixtures with LBWA is shown in Fig. 4.2. The reaction is in agreement with the concept of liquid limit, which is the water content at which the soil exhibits dynamic shear strength. The addition of LBWA and cement, especially, introduces calcium for its strength which caused a decrease in the repulsive force of the soil mixture; thereby needing more water to take the soil to its dynamic shear strength (Osinubi, 1995).

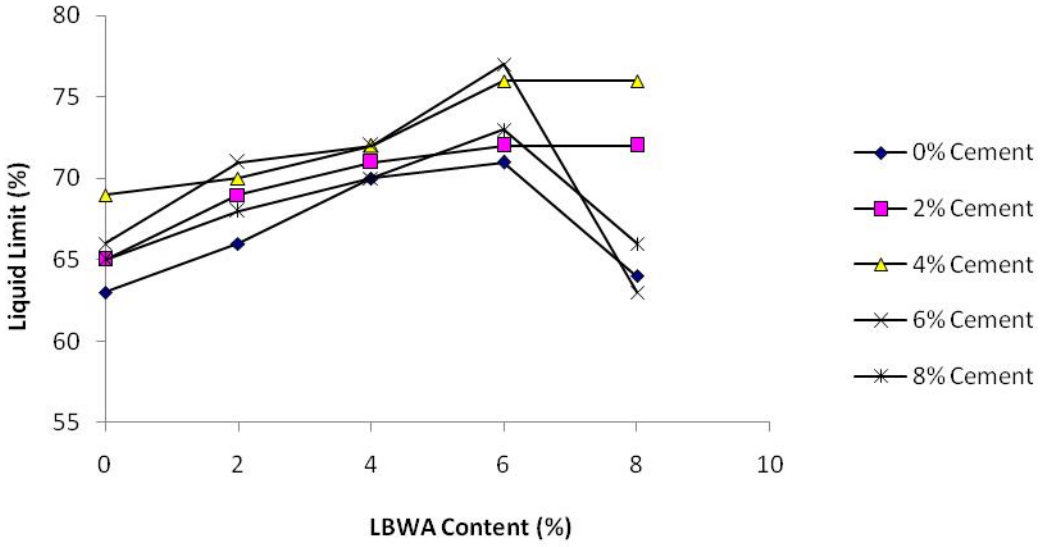


Fig.4.2 Variation of liquid limit of soil-cement mixtures with LBWA contents

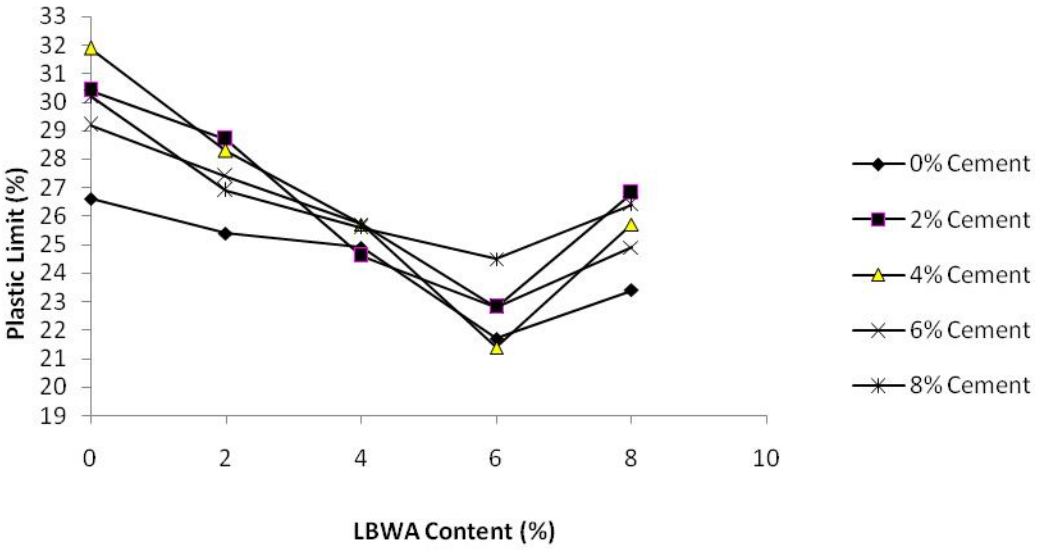


Fig. 4.3 Variation of plastic limit of soil-cement mixtures with LBWA contents

The value of the liquid limit of the soil increased from 63% of the natural soil to a peak value of 77% at 6% OPC/6% LBWA. It was observed that the liquid limit increased steadily from its natural state to 6% OPC/6% LBWA before the values reduced. This reduction may not be unconnected with the agglomeration and flocculation of the clay particles and a result of ion exchange at the surface of the clay particles; as the excess Ca^{2+} in the admixtures reacts with the lower valence metallic ions in the clay structure.

4.2.2 **Plastic Limit**

The variation of the plastic limit of the soil/soil-cement mixtures with LBWA contents is shown in Fig. 4.3. Plastic limit generally decreased with higher admixture contents, from a natural soil value of 26.6% to minimum value of 21.4% at 6% OPC/4% LBWA. The reduction is in agreement with the findings of

Peter (1993) and Osinubi (1995; 1999). The reduction was due to the cation exchange reaction whereby the more active and higher valent cations (i.e. Ca^{2+}) in the admixtures replaced the weakly bonded ions in the clay structure; thereby leading to the flocculation and liberation of water bonded at the outer layers.

4.2.3 **Plasticity Index**

The variation of the plasticity index of soil-cement mixtures with LBWA contents are shown in Fig. 4.4. The reduction in plastic limit and increase in liquid limit were accompanied by a general increase in the plasticity index value from a value of 36.4% to a peak value of 54.6% at 4% OPC/6% LBWA treatment. Akinmade (2008) reported the same trend of increasing LL, PI and decreasing PL with increasing admixture contents on stabilizing black cotton soil with LBWA. The same trend was also reported by Stephen (2005) when he researched into the effect of baggase ash on the stabilization of the soil.

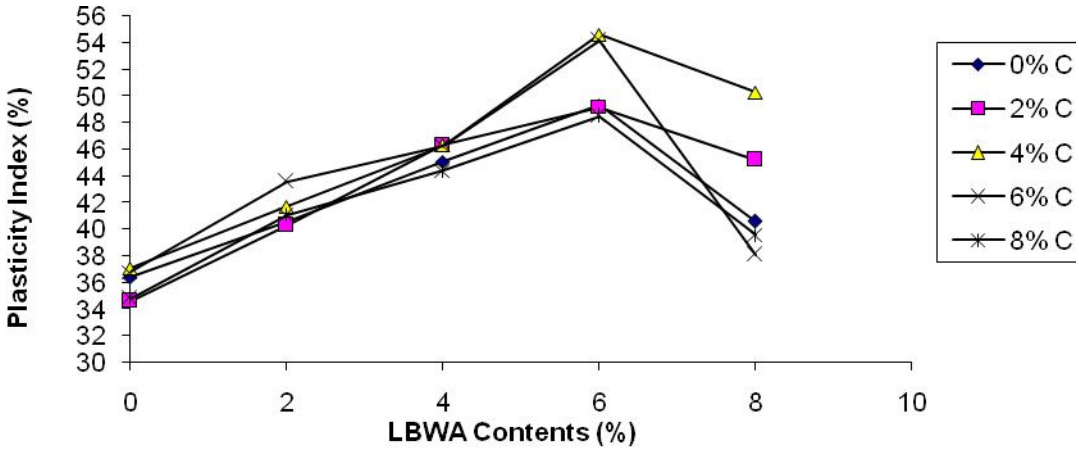


Fig.4.4 Variation of plasticity index of soil-cement mixtures with LBWA contents

4.3 Compaction Characteristics

4.3.1 Maximum dry density

The variation of maximum dry densities (MDD) of soil-cement mixtures with LBWA contents for the three compactive efforts, respectively, are presented in Figs. 4.5(a)-(c). Generally MDD values increased with higher admixture contents which are due to the presence of cement/ash particles filling the voids within the soil.

For British Standard Light compaction (BSL), the MDD increased from 1.3 to 1.5Mg/m^3 at 6% OPC/6% LBWA contents and later reduced to 1.4Mg/m^3 at 0% OPC/8% LBWA treatment. The reduction may be due to the minimal effect of LBWA (with little calcium) on the workability of the stabilized soil.

An increase in MDD was also noticed for the West African Standard (WAS) compaction. The MDD increased from a value of 1.4Mg/m^3 for the natural state to 1.5Mg/m^3 at 6% OPC/6% LBWA due to cement occupying the void within the soil matrix and, in addition, the flocculation and agglomeration of the clay particles due to ion exchange (O' Flaherty, 1988).

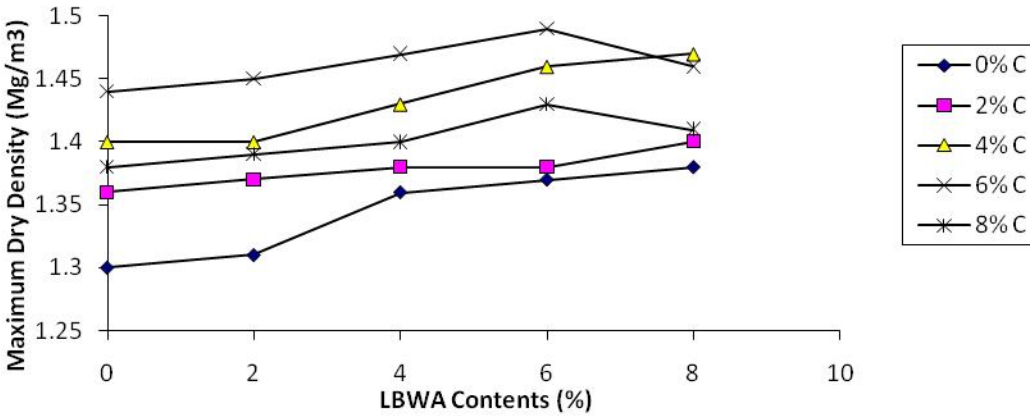


Fig.4.5(a) Variation of maximum dry density of soil-cement mixtures with LBWA contents (BSL Compaction)

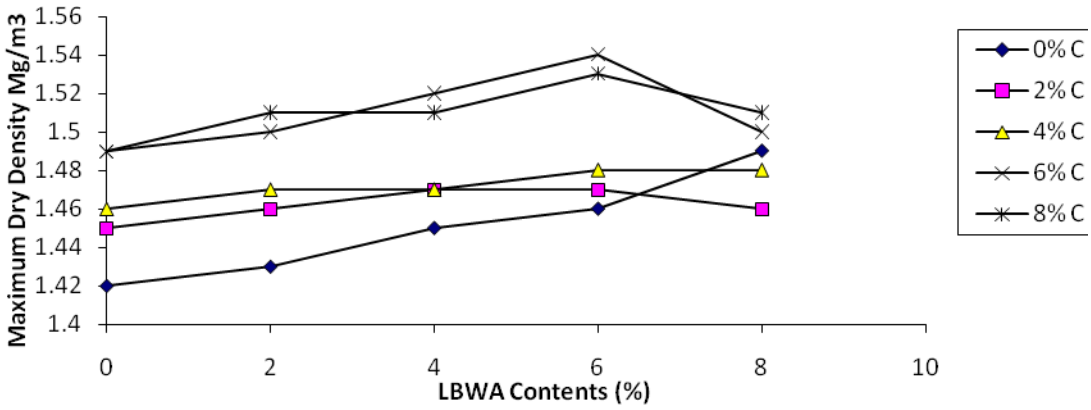


Fig.4.5(b) Variation of maximum dry density of soil-cement mixtures with LBWA contents (WAS Compaction)

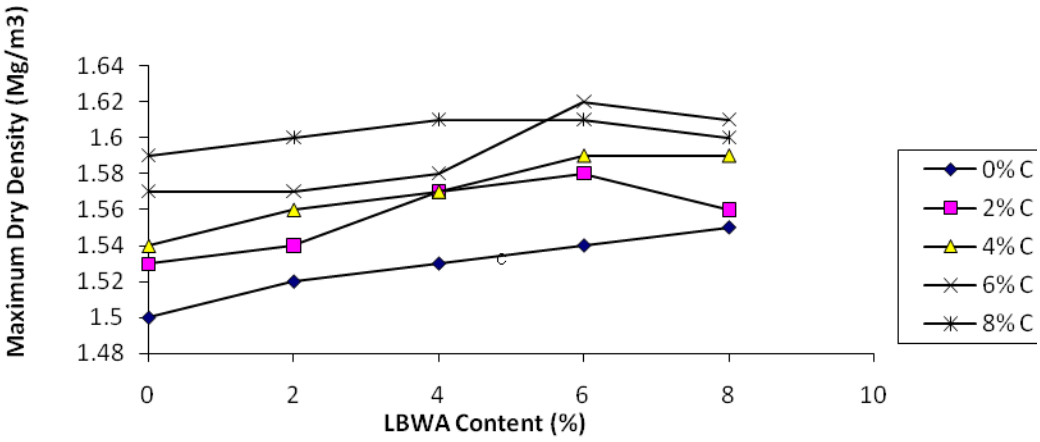


Fig.4.5(c) Variation of maximum dry density of soil-cement mixtures with LBWA contents (BSH Compaction)

The WAS compaction MDD later decreased to a value of 1.5Mg/m^3 at 2% OPC/8% LBWA, and that may not be unconnected with the presence of large low

density aggregate of LBWA particles replacing the soil with a higher specific gravity.

The variation of MDD for the British Standard Heavy (BSH) compaction is also in conformity with the other two energy efforts. Its MDD increased from a minimum 1.5Mg/m^3 in the natural state to a maximum 1.6Mg/m^3 at 6% OPC/6% LBWA. The trend of increasing MDD with admixture contents was reported by Osinubi (1999) and to the same peak value of 6% for OPC before it experienced reduction in the values at higher admixture contents. Stephen (2005) and Akinmade (2008) also reported the same trend of increasing MDD to peak values before falling at higher admixture contents where further additives had no significant impact on the MDD values. The same trend was also reported by Osinubi et al. (2007b).

4.3.2 **Optimum moisture content**

The variation of optimum moisture content of soil-cement mixtures with LBWA contents for BSL, WAS and BSH compactive efforts are shown in Figs. 4.6(a)-(c) respectively. It is observed that the OMC values when compaction was at the three energy levels increased to peak values at 6% LBWA for all cement contents for all the compactive efforts.

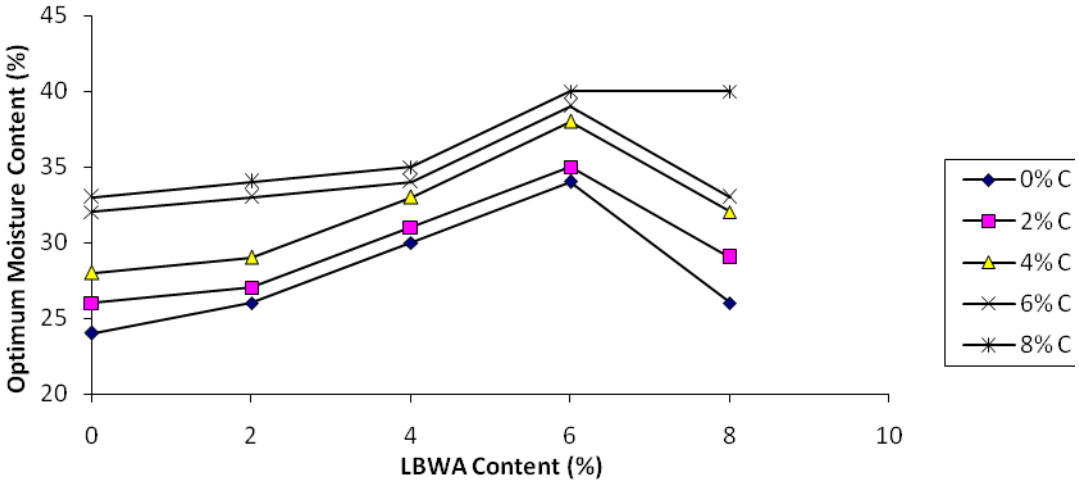


Fig.4.6(a) Variation of optimum moisture contents of soil-cement mixtures with LBWA Contents (BSL Compaction)

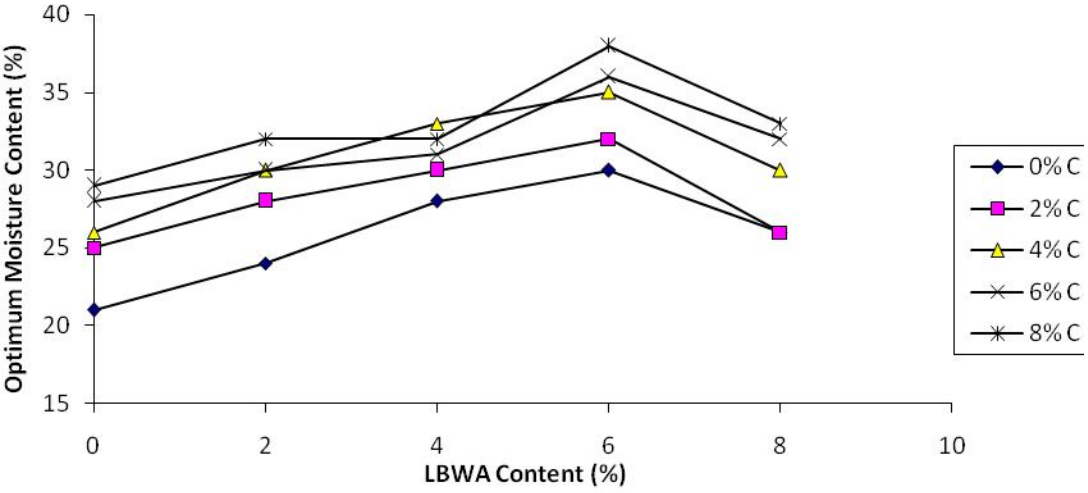


Fig.4.6(b) Variation of optimim moisture contents of soil-cement mixtures with LBWA contents (WAS Compaction)

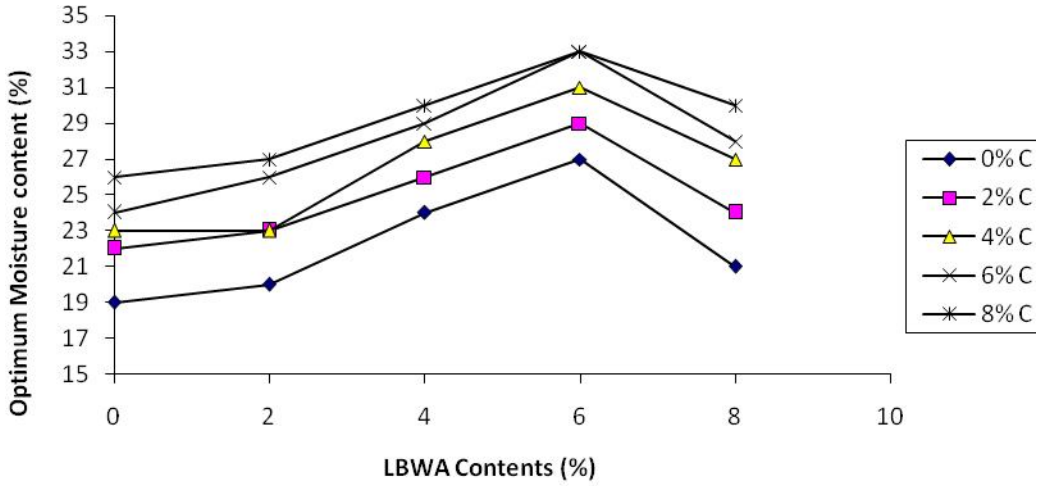


Fig.4.6(c) Variation of optimum moisture contents of soil-cement mixtures with LBWA contents (BSH Compaction)

The OMC for BSL compaction increased from 24% for the natural state to 40% at 8% OPC/6% LBWA. Further addition of LBWA (to 8%) led to a decrease in the OMC to a value of 26% at 0% OPC/8% LBWA. In the case of the WAS compaction, the OMC increased from 21% for the natural state to a maximum value of 38% at 8% OPC/6% LBWA which later decreased to 26% at 2% OPC/8% LBWA. For the BSH compaction, a similar trend was also noticed whereby the OMC increased from its natural state of 19% to 33% at 6% OPC/6% LBWA.

Research works of Osinubi (1999), Stephen (2005), George (2006), osinubi et al. (2007b), Akinmade (2008) and Ochepo (2009) agree with the trend of increasing OMC with admixture contents. An explanation that was offered for these trends is that there was increasing desire for water which commensurates with the higher amount of the additives because more water was required for the

dissociation of admixtures with Ca^{2+} and OH^{-} ions to supply more Ca^{2+} for the cation exchange reaction. Another reason could be due to the increasing surface area caused by the higher amount of the additives, which required more water for the lubrication of the entire matrix.

It was also observed that OMC decreased with higher compactive efforts because it was easier to breakdown flocculated aggregates, destroy shear planes and eliminate large pores, at higher compactive efforts. (Osinubi, 1999).

4.4 Strength Characteristics

4.4.1 Unconfined ompressive strength

The general test recommended for use in the determination of the additive to be used in the stabilization of soil is the unconfirmed compressive strength (UCS) test (Singh, 1991). It is an important factor in the evaluation of the design criteria for the use of soil as a pavement material (Ola, 1983).

4.4.1.1 7 days curing period

The variation of UCS of soil-cement mixtures cured for 7 days with LBWA contents for BSL, WAS and BSH compactive efforts are shown in Fig. 4.7(a)-(c) respectively. The UCS values for the BSL compaction showed a sharp increase from 17kN/m^2 for the natural soil to 986kN/m^2 at 6% OPC/6% LBWA. Subsequent addition of LBWA content did not increase the UCS, rather, it

recorded a slight decrease to a value of 823kN/m^2 at 6% OPC/8% LBWA because of insufficient water to take the pozzolanic reaction to completion.

For specimen compacted with WAS energy, the UCS value increased from an initial value of 381kN/m^2 to 1436kN/m^2 at 6% OPC/6% LBWA. Further treatment of the soil did not produce a positive effect, because the UCS value reduced to a minimum 1062kN/m^2 at 2% OPC/8% LBWA.

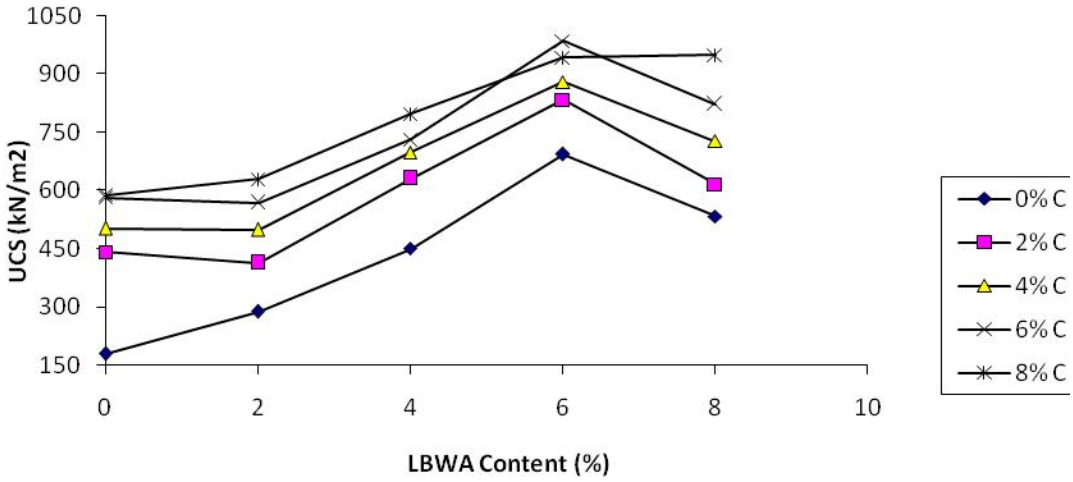


Fig.4.7(a) Variation of UCS (7 days curing) of soil-cement mixtures with LBWA contents (BSL Compaction)

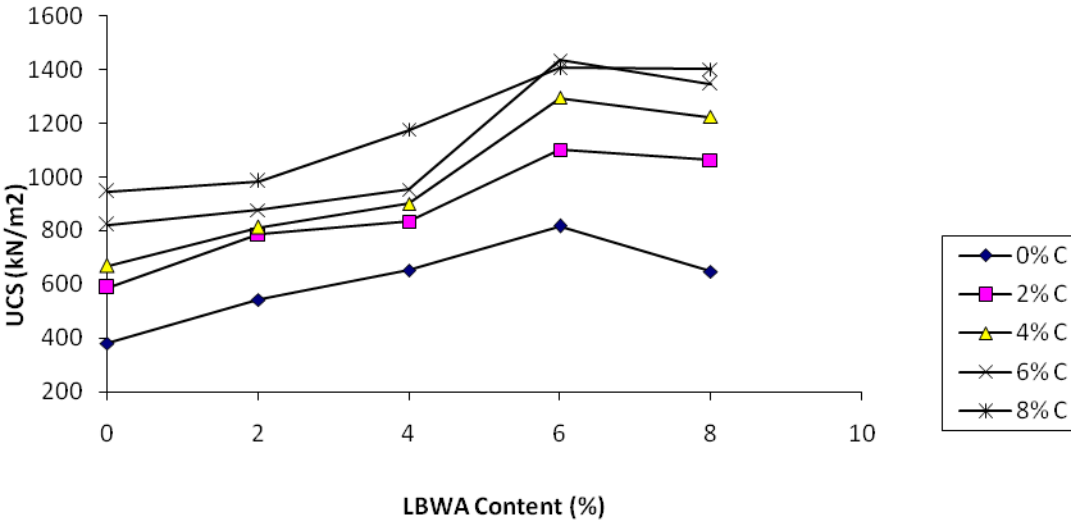


Fig4.7(b) Variation of UCS (7 days curing) of soil-cement mixtures with LBWA contents (WAS Compaction)

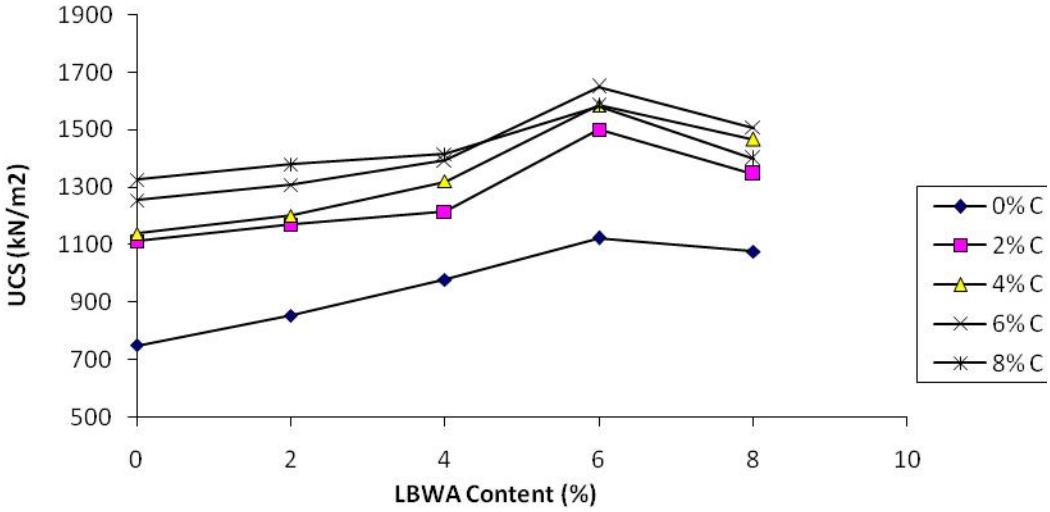


Fig.4.7(c) Variation of UCS (7 days curing) of soil-cement mixtures with LBWA contents (BSH Compaction)

The UCS for BSH compaction increased from 750kN/m^2 for the natural soil to 1650kN/m^2 at 6% OPC/6% LBWA. Additional treatment with LBWA reduced the UCS value to 1508kN/m^2 at 6% LBWA/8% OPC.

The increase in the UCS values (or the gain in strength) was primarily due to the formation of various compounds such as calcium silicate hydrates (CSH) and calcium aluminate hydrates (CAH) and micro fabric changes, which are responsible for strength development. Though, there were increments in the UCS values, they fell short of 1710kN/m^2 specified by TRRL (1977) as criterion for adequate stabilization using OPC. The decrease in the values of the UCS for all the compactive energies beyond 6% LBWA was as a result of insufficient water to bring the pozzolanic reaction to completion (Osinubi, 1999). It was stated that larger percentages of cement (8% and beyond) are considered in view of the recommendation of Ingles (1974), that larger addition of cement is normally considered bad due to the generation of high strength and hence widely spaced cracking by shrinkage or by service stresses or both (Osinubi, 1999).

4.4.1.2 **14 days curing period**

The variation of UCS values (14 days curing) for soil-cement mixtures with LBWA contents are shown in Figs. 4.8 (a)-(c)), for the BSL, WAS and BSH compaction respectively. All the compactive efforts recorded increases in the UCS values for the natural soils from with 190, 616 and 767kN/m^2 to peak values of 1326, 2048 and 2519kN/m^2 at 6% OPC/6% LBWA, 4% OPC/6% LBWA and 6% OPC/6% LBWA respectively.

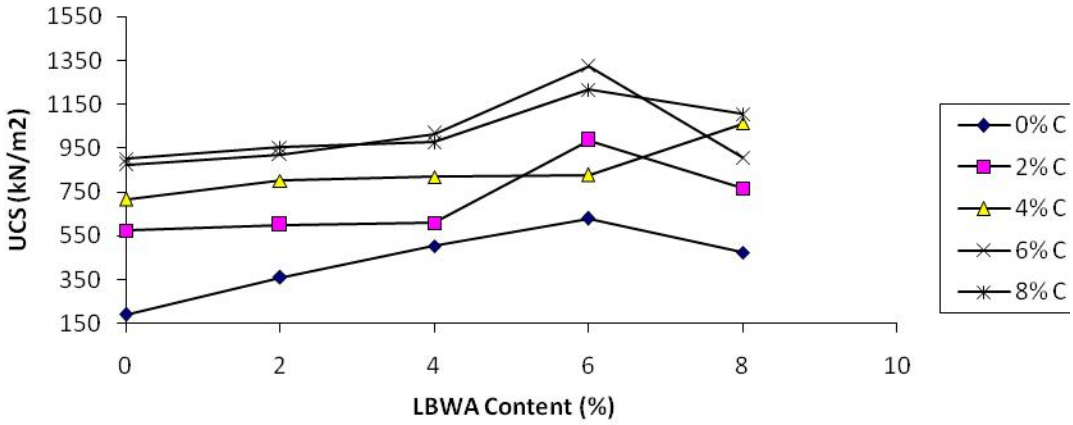


Fig.4.8(a) Variation of UCS (14 days curing) of soil-cement mixtures with LBWA contents (BSL Compaction)

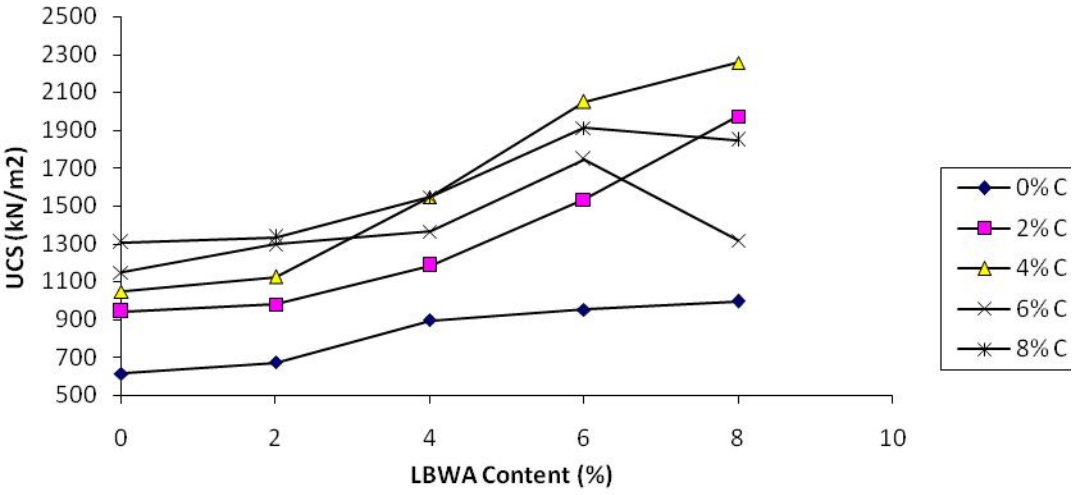


Fig.4.8(b) Variation of UCS (14 days curing) of soil-cement mixtures with LBWA contents (WAS Compaction)

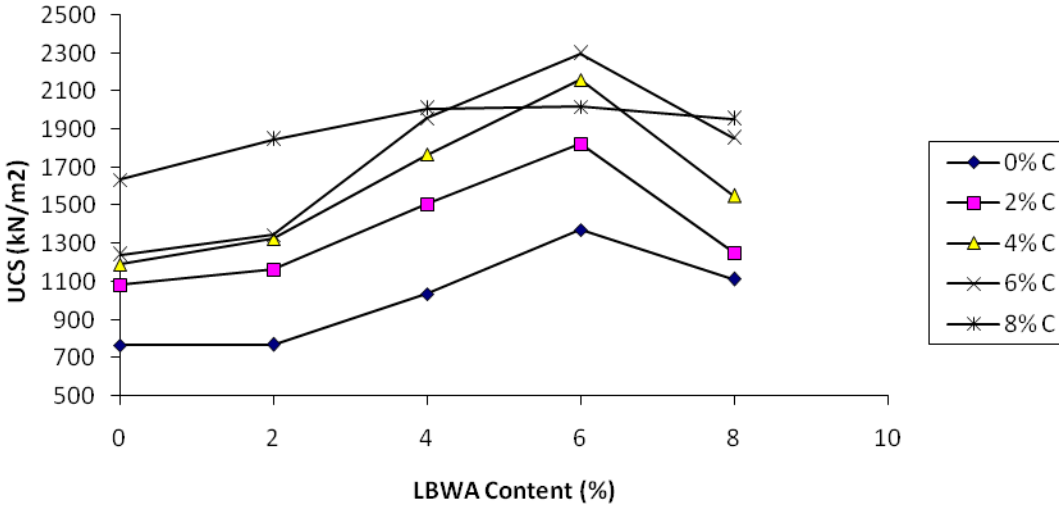


Fig.4.8(c) Variation of UCS (14 days curing) of soil-cement mixtures with LBWA contents (BSH Compaction)

The trend of increased compressive strength with curing period can be attributed to time dependent strength gain of the pozzolanas. The increase is due to (Osinubi and Medubi, 1997) availability of sufficient water which enhanced hydration reaction that has contributed to the reaction between lime liberated from the hydration reaction of cement and the LBWA to form secondary cementation compounds.

4.4.1.3 **28 days curing period**

The variation of UCS (28 days curing period) of soil-cement mixtures with LBWA contents for BSL, WAS and BSH compactive efforts, respectively are shown in Figs. 4.9 (a)-(c). The same trend of increase in UCS values recorded for specimens cured for 7 and 14 days was also observed for 28 days curing period except that the UCS values increased with the curing periods and higher

compactive efforts. The UCS for 28 days curing period increased from 220, 419 and 824kN/m^2 for the natural soil to 1348, 2325 and 2616kN/m^2 at 6% OPC/6% LBWA, 8% OPC/6% LBWA and 6% OPC/6% LBWA for BSL, WAS and BSH compactive efforts, respectively.

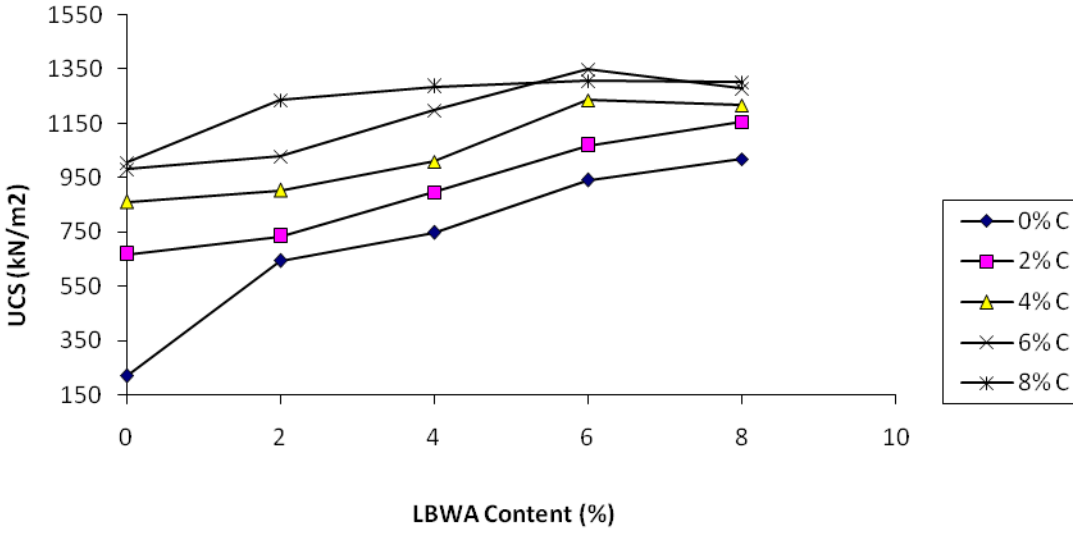


Fig.4.9(a) Variation of UCS (28 days curing) of soil-cement mixtures with LBWA contents (BSL Compaction)

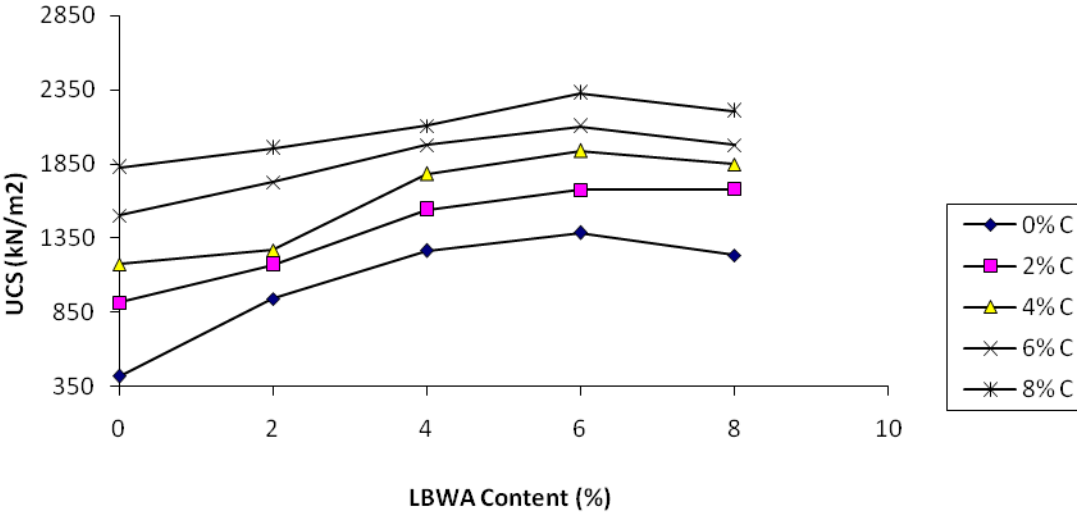


Fig.4.9(b) Variation of UCS (28 days curing) of soil-cement mixtures with LBWA contents (WAS Compaction)

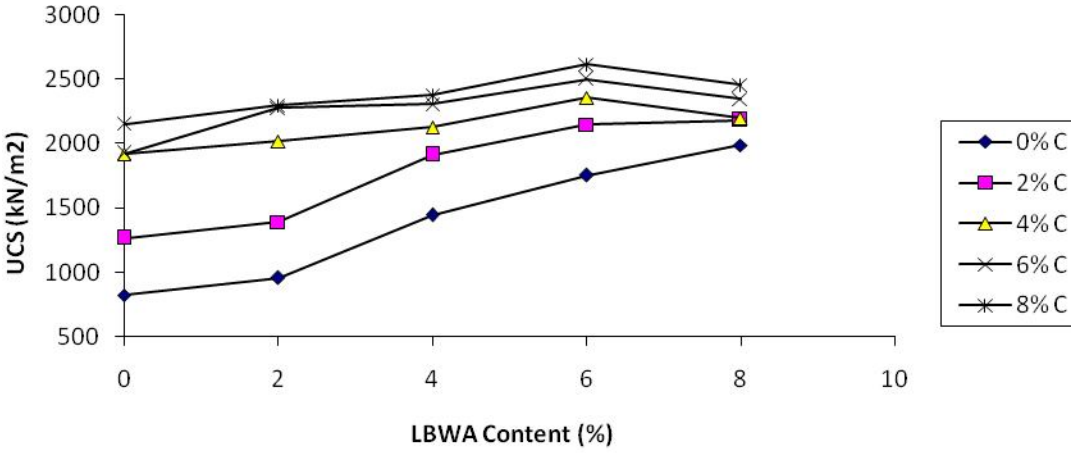


Fig.4.9(c) Variation of UCS (28 days curing) of soil-cement mixtures with LBWA contents (BSH Compaction)

Although, there were slight drops in the UCS values of beyond 6% LBWA content, which was due to insufficient water needed to bring the pozzolanic reaction to completion, the results indicates that LBWA admixture has long time strength improving capability (Akinmade, 2008).

4.4.1.4 **7 days curing and 7 days soaking**

The variations of UCS values (7 days curing, 7 days soaking) of soil-cement mixtures with LBWA contents for the three energy compactive efforts are shown in Figs. 4.10 (a)-(c), respectively. Generally, the UCS values increased with higher compactive efforts and with higher additive contents.

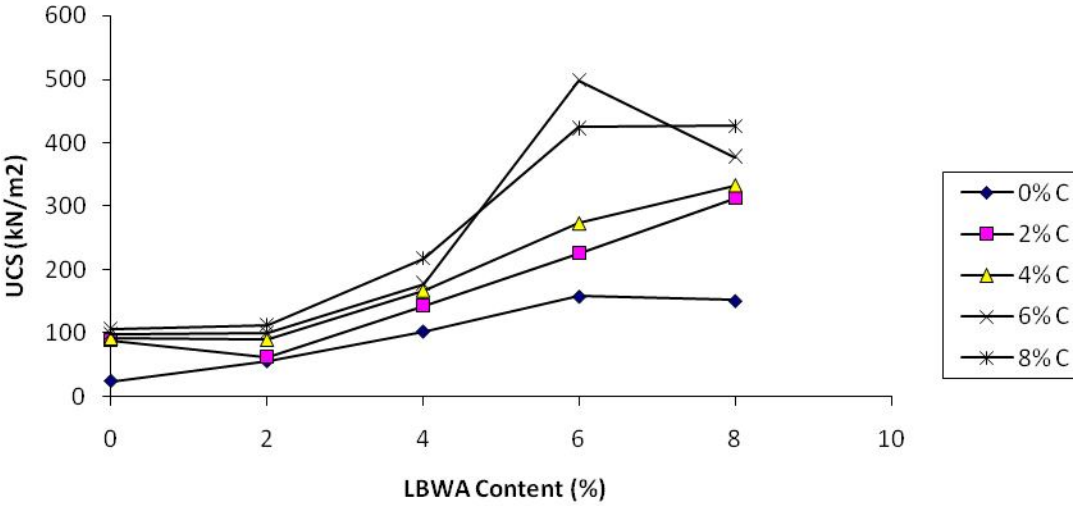


Fig.4.10(a) Variation of UCS (7 days curing, 7days soaking) of soil-cement mixtures with LBWA contents (BSL Compaction)

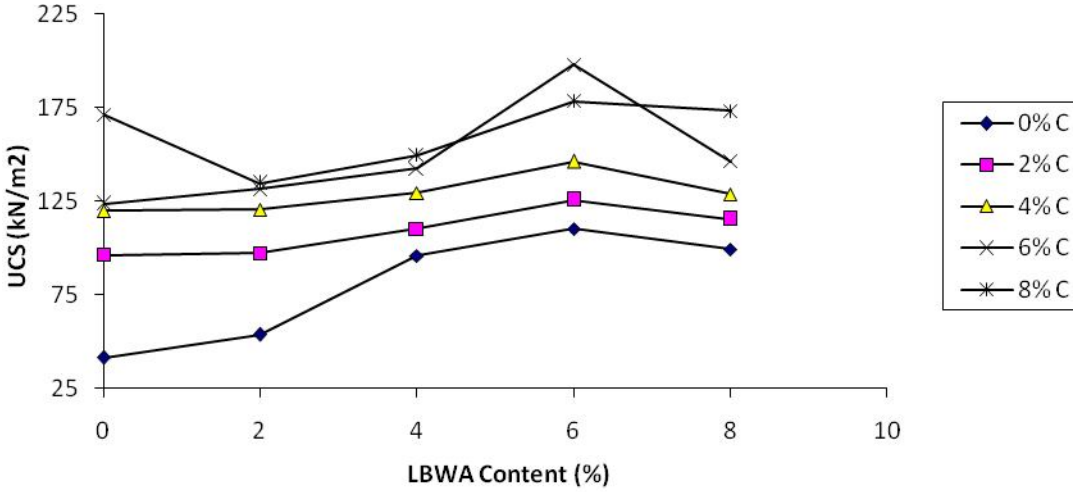


Fig.4.10(b) Variation of UCS (7 days curing, 7 days soaking) of soil-cement mixtures with LBWA contents (WAS Compaction)

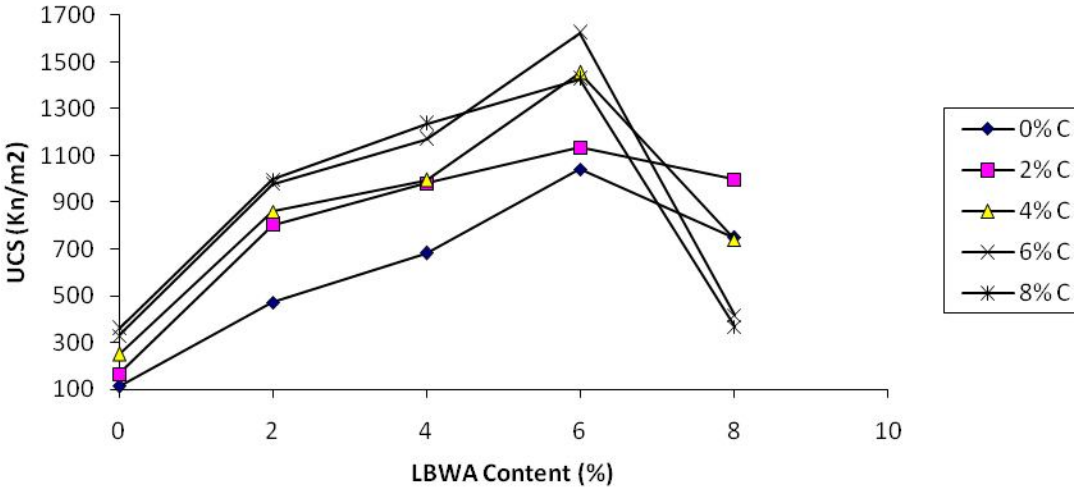


Fig.4.10(c) Variation of UCS (7 days curing, 7 days soaking) of soil-cement mixtures with LBWA contents (BSH Compaction)

It should be noted that the effect of the BSL compactive effort is the highest on the values of the UCS because the rate of increment. The UCS of the natural soil increased from a value of 24kN/m^2 to a peak value 497kN/m^2 at 6% OPC/6% LBWA before reducing to 377.48kN/m^2 at 6% OPC/8% LBWA. The UCS values increased from 42 and 112kN/m^2 to 197 and 1530kN/m^2 at both 6% OPC/6% LBWA for those compacted with WAS and BSH energies, respectively. Addition of the admixtures beyond 6% LBWA did not have impacts on the UCS values; and due to the same reasons given earlier. It is pertinent to note here that the UCS values for the soaked specimens are far too low compared to those obtained for 7 and 14 days curing periods; which is due to the ingress of water that resulted in loss of strengths.

4.4.2 **California bearing ratio**

The California bearing ratio (CBR) value of a soil/stabilized soil is an important parameter in gauging the suitability of the soil/stabilized soil for engineering use. It gives the indication of the strength and bearing ability of the soil. The Nigerian General Specification (1997) recommends a CBR of 180% to be attained in the laboratory for cement stabilized materials to be constructed by the mix-in-place method.

4.4.2.1 **Unsoaked CBR**

The variation of the CBR (unsoaked) of soil-cement mixtures with LBWA contents for the three energy efforts used are given in Figs. 4.11(a)-(c). The unsoaked CBR for BSL compaction gave a peak value of 46% at 6% OPC/6% LBWA from a value of 5% for the natural soil. For WAS and BSH compactions, the CBR values increased from 7 and 11% to 77 and 83%, respectively at 6% OPC/6% LBWA.

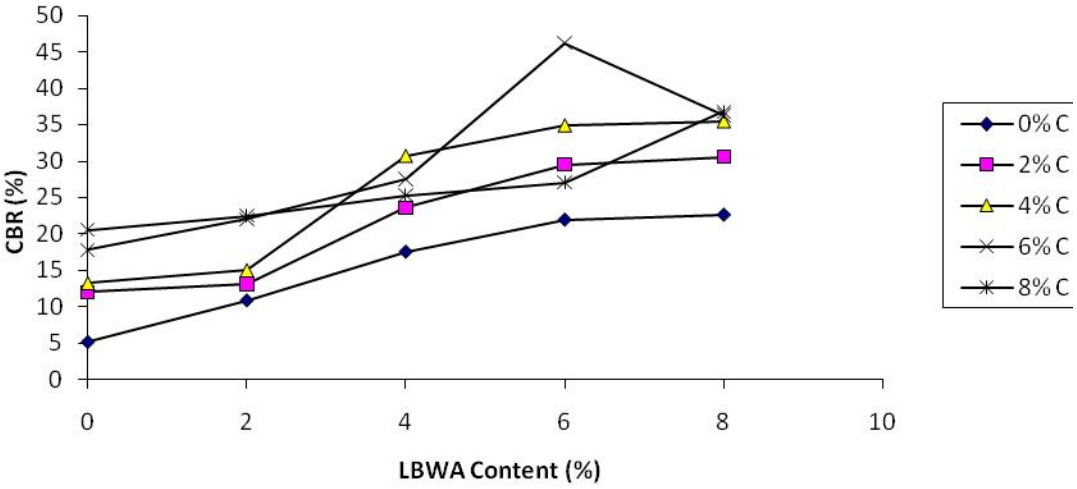


Fig.4.11(a) Variation of CBR (Unsoaked) of soil-cement mixtures with LBWA contents (BSL Compaction)

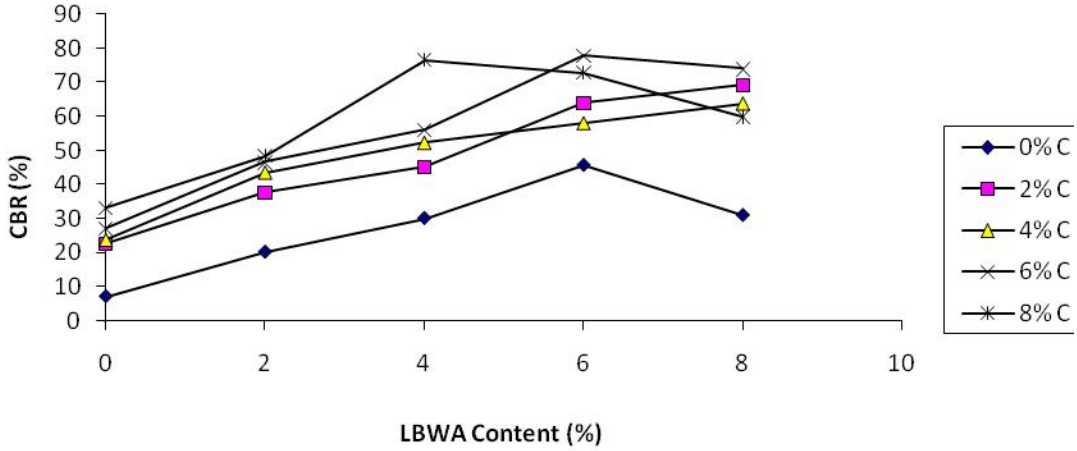


Fig.4.11(b) Variation of CBR (Unsoaked) of soil-cement mixtures with LBWA contents (WAS Compaction)

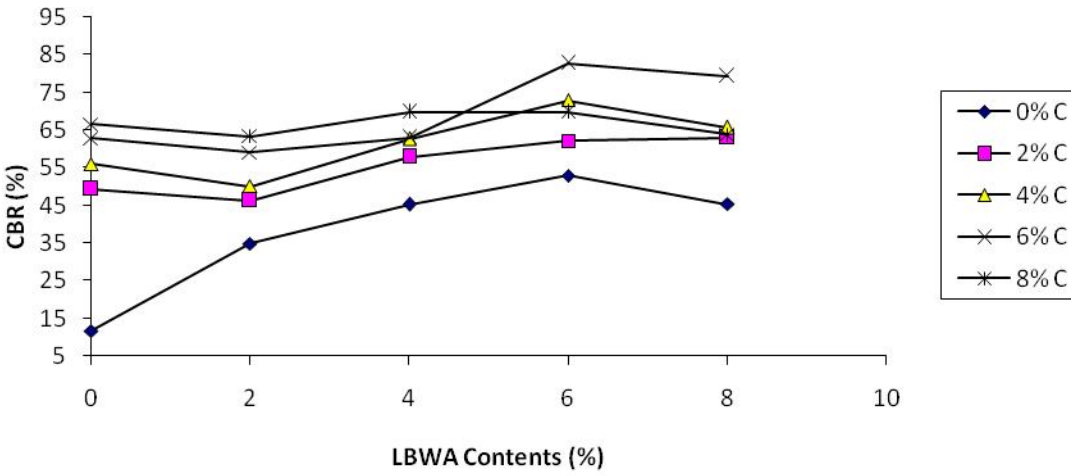


Fig.4.11(c) Variation of CBR (Unsoaked) of soil-cement mixtures with LBWA contents (BSH Compaction)

It was observed that, though, the value of the CBR increased with higher additive contents at higher compactive efforts, the compactive efforts do not have appreciable effect on the values of the CBR. The effect is much less in BSL compaction as all its values did not show appreciable improvement. The WAS and BSH compactive efforts showed little improvements with values of 77% and 83%, respectively, at 6% OPC/6% LBWA because of their little differences with their natural values. However, they meet the requirement to be used as sub-base courses (Gidigasu, 1982).

4.4.2.2 **Soaked CBR**

The variation of CBR (soaked for 24 hours) of soil-cement mixtures with LBWA contents for BSL, WAS and BSH compactive efforts are shown in Figs. 4.12(a)-(c), respectively.

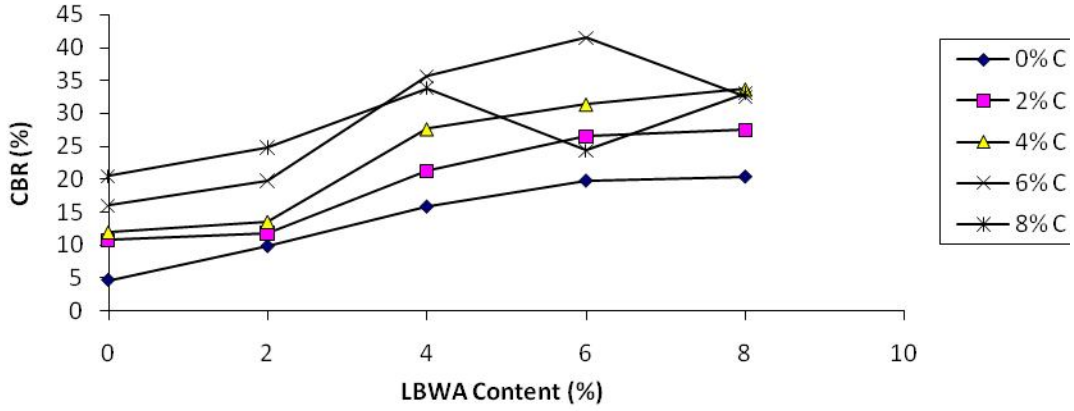


Fig.4.12(a) Variation of CBR (Soaked) of soil-cement mixtures with LBWA contents (BSL Compaction)

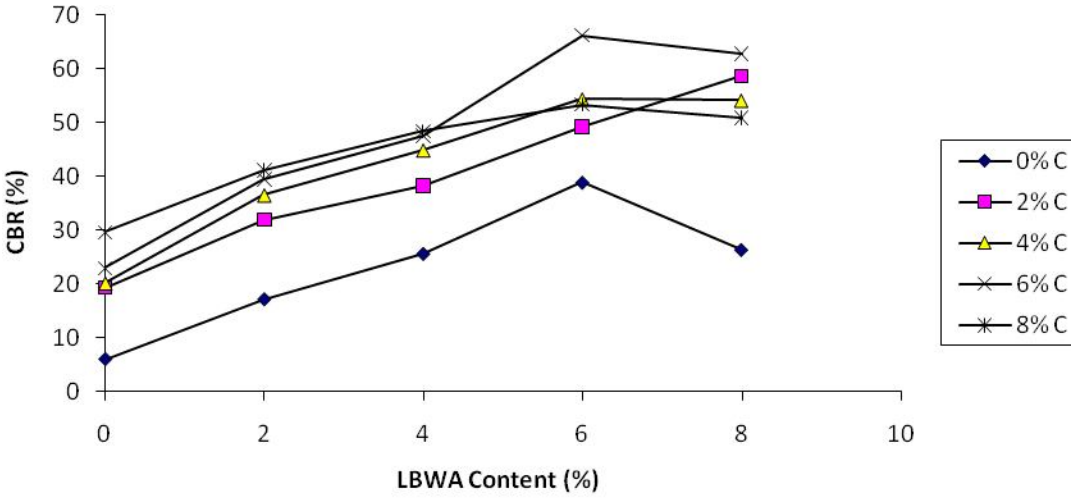


Fig.4.12(b) Variation of CBR (Soaked) of soil-cement nixtures with LBWA contents (WAS Compaction)

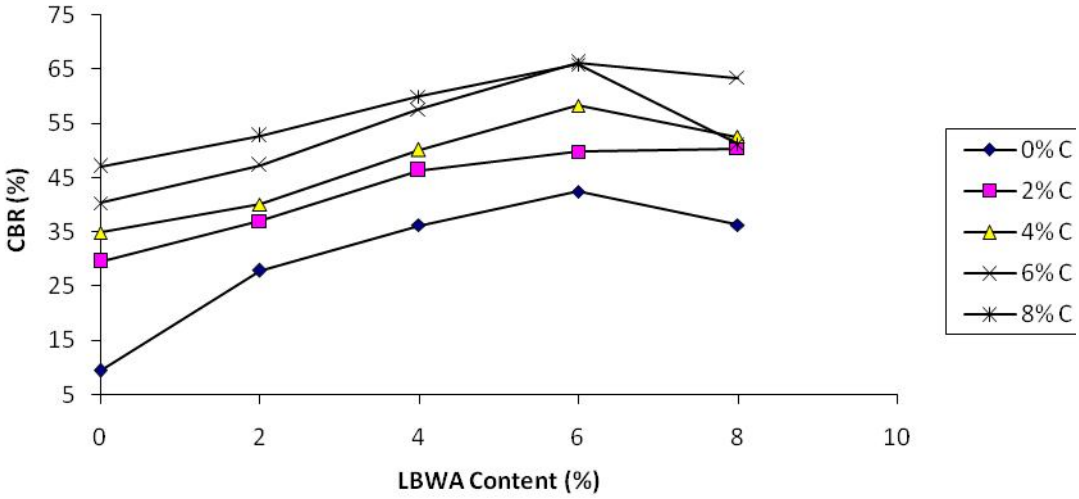


Fig.4.12(c) Variation of CBR (Soaked) of soil-cement mixtures with LBWA contents (BSH Compaction)

There were increases in the values of CBR with higher additive contents at higher compactive efforts with a peak value of 66% recorded for both WAS and BSH compactive efforts at 6% OPC/6% LBWA. The reduction with respect to the unsoaked CBR values was due to the ingress of water into the specimen when it was soaked for 24 hours, which weakened it and reduced their strength. All the values recorded fell short of the specification required for base courses. However soils compacted with WAS and BSH energies and treated with 6% OPC/6% LBWA with the values of 66% recorded, meet the Nigerian General Specification (1997) requirement of 30% for sub base materials.

4.5 **Durability Assessment**

Durability assessment of soil samples involves the simulation of some of the worst conditions that could be obtained in the field. It was evaluated by the immersion of the specimens in water to determine resistance to loss in strength which is more acceptable for tropical regions like Nigeria (Ola, 1974). The resistance to loss in strength determined as the ratio of the unconfirmed compressive strength of specimen wax-cured for 7 days, de-waxed top and bottom to allow absorption of water and later immersed in water for 7 days, to those cured for 14 days. Conventionally, an allowable 20% loss in strength is tolerated for a specimen cured for 7 day and immersed in water for 4 days (Ola, 1974; Osinubi, 1998a; 1999).

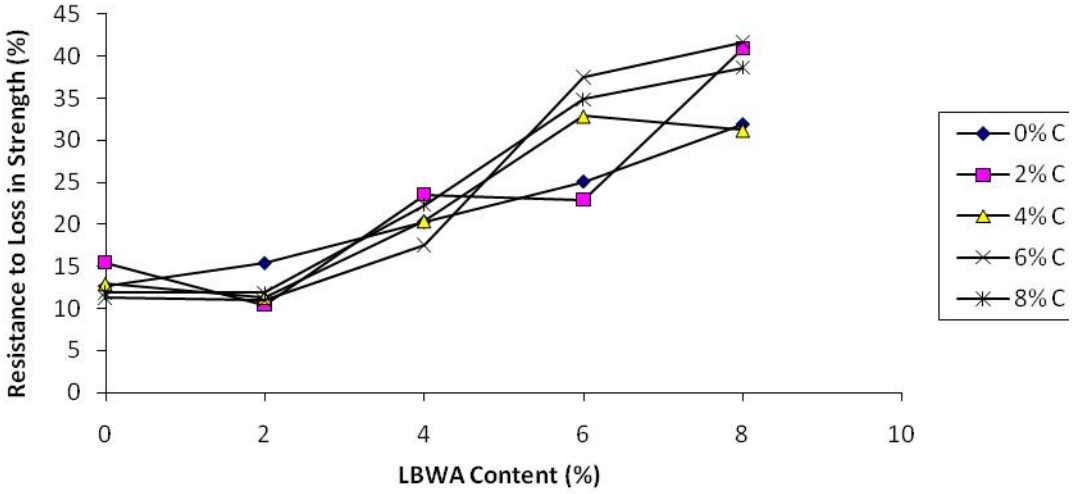


Fig.4.13(a) Variation of Resistance to Loss in Strength of soil-cement mixtures with LBWA contents (BSL Compaction)

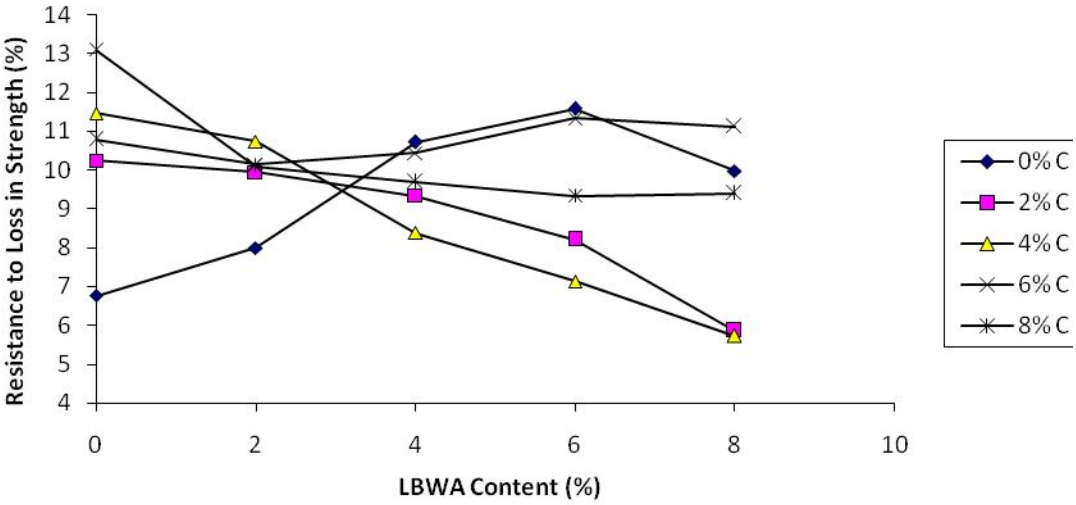


Fig.4.13(b) Variation of Resistance to Loss in Strength of soil-cement mixtures with LBWA contents (WAS Compaction)

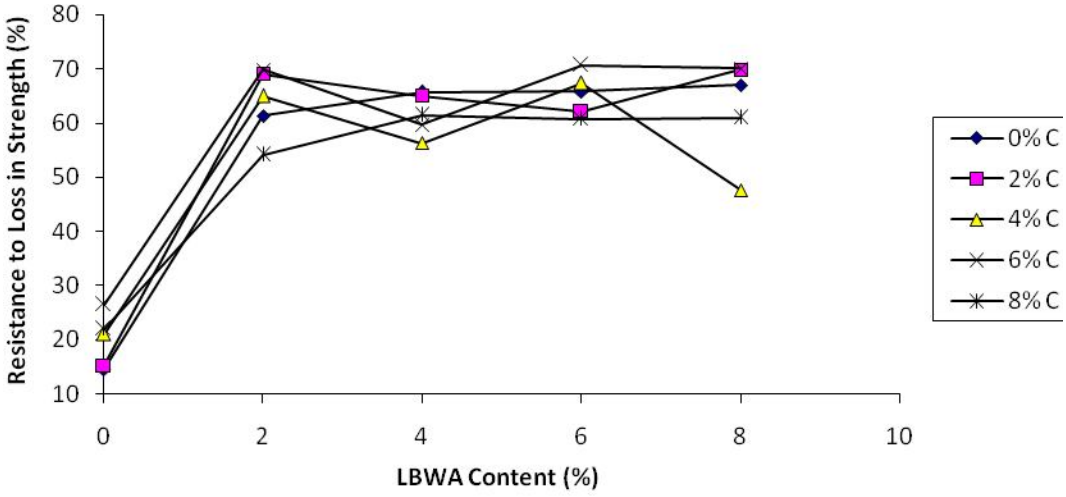


Fig.4.13(c) Variation of Resistance to Loss in Strength of soil-cement mixtures with LBWA contents (BSH Compaction)

In the variation of resistance to loss in strength of soil-cement mixtures with LBWA contents for BSL, WAS and BSH respectively shown in Figs. 4.13(a)-(c), the resistance to loss in strength increased from 13, 7 and 15% for the natural soils compacted with BSL, WAS and BSH energies to peak values of 42,

13 and 71% at 6% OPC/8% LBWA, 8% OPC/0% LBWA and 6% OPC/6% LBWA, respectively. All the peak resistance to loss in strength values fell short of the acceptable 80%. However, 71% resistance to loss in strength for BSH compaction at 6% OPC/6% LBWA may be acceptable since the limiting value proposed by Ola (1994) is based on 4 day soaking and not the 7 days soaking which the specimens for this research were subjected to.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The preliminary investigation conducted on the natural black cotton soil collected at Chad Basin Development Authority (CBDA), New Marte, Borno State shows that it falls under A-7-6(13) classification for AASHTO (1986) and CH according to Unified Soil Classification System (ASTM, 1992). The natural soil has high moisture content of 35% because it was collected during the rainy period. It has liquid limit of 63%, plastic limit of 27%, plasticity index of 36%, linear shrinkage of 17%, free swell of 75%, specific gravity of 1.94 and NBRRI Classification of high swell potential. All these values indicate that the soil is highly plastic with about 71% of the soil particles passing the B. S. No 200 sieve. The strength characteristics are also very low, thereby rendering the soil unfit for sub-base or road base courses.

In an effort to raise the soil's suitability for engineering use, the air dried samples were treated with OPC/LBWA in stepped concentration of 0, 2, 4, 6, and 8% by dry weights of the soil. The tests conducted showed that the liquid limit of the natural soil increased from 63% to 77% at 6% OPC/6% LBWA. The plastic limit, however, decreased from 26.6% for the natural soil to 21.4% at 6% OPC/4% LBWA. The plasticity index values for all the concentration of the additive

exceeded the 30% value prescribed for sub-grade materials by the Nigeria General Specification (1997).

The MDD increased with higher additive blends and compactive efforts; which is in conformity with the same trend reported by Osinubi (1999a), Osinubi et al. (2007b), Staphen (2005) and Akinmade (2008). The peak MDD values recorded for BSL, WAS and BSH compactive efforts are respectively 1.4, 1.5 and 1.6Mg/m^3 at 6% OPC/6% LBWA treatment. The OMC, on the other hand, decreased with higher compactive efforts but increased with higher LBWA contents. The decrease may be due to the effect of breakdown of the flocculated aggregates and elimination of large pores, when higher compactive energies were used. The optimum moisture content values at the natural states increased from 24, 21 and 19% to 40, 38 and 33% when compacted using BSL, WAS and BSH energies at 8% OPC/6% LBWA, 6% OPC/8% LBWA and 6% OPC/6% LBWA, respectively.

The unconfined compressive strength (UCS) values for natural soil compacted with BSL, WAS and BSH energies at 7 days curing period are 179, 381 and 750kN/m^2 respectively and increased to 986, 1436 and 1650kN/m^2 at 6% OPC/6% LBWA treatment. The values of UCS for both BSL and WAS fell short of the requirement based on Road Note 31 (TRRI, 1977) requirement for economic range of OPC stabilization. However, the UCS value of the BSH compaction could be acceptable for base courses of pavements.

The 28 days curing period UCS values obtained showed that the OPC/LBWA blend has a long term advantage in terms of strength gain. There were tremendous increments in the values of UCS from their natural states. The 28 days curing period UCS produced a peak value of 2616kN/m^2 at 6% OPC/6% LBWA for BSH compaction, showing that the soil treated with this blend can be used (at BSH compaction) as base course of pavement material.

The unsoaked CBR values of 5, 7 and 11% (for the natural soil) compacted with BSL, WAS and BSH energy efforts, respectively, increased to 46, 77 and 83% at 6% OPC/6% LBWA. The 24 hours soaked CBR values recorded peak values of 42, 66 and 66% with BSL, WAS, BSH energies, respectively, which showed about 10-15% decrease from the unsoaked CBR values. The CBR values of 66% with BSH compaction at 6% OPC/6% LBWA blend can be used as sub base material because it meets the 29% recommended values for sub-base by the Nigerian General Specification (1997). Also the 42% recorded with BSL compaction at 6% OPC/6% LBWA treatment meets the 15% recommended for subgrade material by the Nigerian General Specification (1997).

The resistance to loss in strength of the soil increased from 13, 7, and 15% for the natural soil to peak values of 42, 13 and 71% for BSL, WAS and BSH energies at 6% OPC/8% LBWA, 8% OPC/0% LBWA and 6% OPC/6% LBWA, respectively. Only 71% resistance to loss in strength (29% loss in strength) at 6% OPC/6% LBWA with BSH compaction is close to the limiting value of 80% resistance to loss in strength (Ola, 1983) based on 4 days soaking. The 6%

OPC/6% LBWA treatment of the soil can be used, at BSH compaction, for sub-base material because the soil was subjected to a harsher condition (of 7 days soaking) and due to the time dependent gain in strength advantage of the pozzolana.

5.2 **RECOMMENDATION**

Based on the results obtained, the optimum blend of 6% OPC/6% LBWA treatment on the black cotton soil is to be used as sub-base material when compacted with the British Standard heavy energy.

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APPENDIX

Table A3.1: Specific Gravity values for Soil-Cement–LBWA mixes

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	**2**	**4**	**6**	**8**
0	1.94	1.95	1.93	1.96	1.92
2	1.93	1.90	1.91	1.94	1.92
4	1.94	1.92	1.89	1.93	1.91
6	1.91	1.90	1.89	1.91	1.92
8	1.92	1.93	1.94	1.92	1.91

Table A3.2: Sieve Analysis Results for the Natural soil

BS Sieve Size (mm)	Wt. Retained G	Wt. Retained g	Percent Retained %	Percent Passing %
3.175	2.1	2.1	8.4	91.6
2.4	0.4	0.4	1.6	98.4
1.18	0.4	0.4	1.6	94.7
600µm	0.8	0.8	3.2	96.8
425µm	1.2	1.2	4.8	95.2
300µm	3.3	3.3	13.2	86.8
212µm	7.3	7.3	29.2	70.8
150µm	8.9	8.9	35.6	64.4
75µm	0.1	0.1	0.4	99.6

Table A3.3: Liquid Limit Test Results for Soil–Cement–LBWA mixes

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	**2**	**4**	**6**	**8**
0	63	66	70	71	64
2	65	69	71	72	72
4	69	70	72	76	76
6	66	71	72	77	63
8	65	68	70	73	66

Table A3.4: Plastic Limits Test Results for Soil-Cement-LBWA mixes

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	**2**	**4**	**6**	**8**
0	26.6	25.4	24.9	21.7	23.4
2	30.4	28.7	24.6	22.8	26.8
4	31.9	28.3	25.7	21.4	25.7
6	29.2	27.4	25.7	22.8	24.9
8	30.2	26.9	25.6	24.5	26.4

Table A3.5: Plasticity Index Tests Result for Soil-Cement-LBWA mixes

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	**2**	**4**	**6**	**8**
0	36.4	34.6	37.1	36.8	34.8
2	40.6	40.3	41.7	43.6	41.1
4	45.1	46.4	46.3	46.3	44.4
6	49.3	49.2	54.6	54.2	48.5
8	40.6	45.2	50.3	38.1	39.6

Table A3.6 (a): Maximum Dry Density Test Results for Soil–Cement–LBWA mixes (BSL Compaction)

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	**2**	**4**	**6**	**8**
0	1.30	1.36	1.40	1.44	1.38
2	1.31	1.37	1.40	1.45	1.39
4	1.36	1.38	1.43	1.47	1.40
6	1.37	1.38	1.46	1.49	1.43
8	1.38	1.40	1.47	1.46	1.41

Table A3.6 (b): Maximum Dry Density Test Results for Soil-Cement–LBWA mixes (WAS Compaction).

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	**2**	**4**	**6**	**8**
0	1.42	1.45	1.46	1.49	1.49
2	1.43	1.46	1.47	1.50	1.51
4	1.45	1.47	1.47	1.52	1.51
6	1.46	1.47	1.48	1.54	1.53
8	1.49	1.46	1.48	1.50	1.51

Table A3.6 (c): Maximum Dry Density Test Results for Soil-Cement–LBWA mixes (BSH compaction)

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	**2**	**4**	**6**	**8**
0	1.50	1.53	1.54	1.57	1.59
2	1.52	1.54	1.56	1.57	1.60

4	1.53	1.57	1.57	1.58	1.61
6	1.54	1.58	1.59	1.62	1.61
8	1.55	1.56	1.59	1.61	1.60

Table A3.7 (a): Optimum Moisture Content Tests Results for Soil–Cement–LBWA mixes (BSL Compaction)

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**
0	24	26	28	32	33
2	26	27	29	33	34
4	30	31	33	34	35
6	34	35	38	39	40
8	26	29	32	33	40

Table A3.7 (b): Optimum Moisture content Tests Results for Soil–Cement–LBWA mixes (WAS Compaction)

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**
0	21	25	26	28	29
2	24	28	30	30	32
4	28	30	35	36	38
6	30	32	35	36	38
8	26	26	30	32	33

Table A3.7 (c): Optimum Moisture Content Test Result for Soil-Cement-LBWA mixes (BSH Compaction)

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	**2**	**4**	**6**	**8**
0	19	22	23	24	26
2	20	23	23	26	27
4	24	26	28	29	30
6	27	29	31	33	33
8	21	24	27	28	30

Table A3.8 (a): Unconfined Compressive Strength (7 days curing) Tests Results for Soil Cement–LBWA mixes (BSL Compaction)

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	**2**	**4**	**6**	**8**
0	178.38	441.26	502.40	582.25	588.87
2	288.39	413.01	498.30	568.14	628.67
4	450.25	631.31	698.45	731.91	798.24
6	693.04	833.65	879.58	985.46	942.23
8	533.35	615.88	727.31	822.78	948.66

Table A3.8 (b): Unconfined Compressive Strength (7 days curing) Tests Results for Soil-Cement-LBWA mixes (WAS Compaction)

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	**2**	**4**	**6**	**8**
0	381.41	588.12	668.50	821.02	945.49
2	541.89	786.23	812.65	876.31	984.25
4	652.65	834.37	899.98	953.98	1175.78
6	816.79	1100	1295	1435.68	1406.45

8	647.70	1062.36	1223.35	1345.67	1400.35

Table A3.8 (c): Unconfined Compressive Strength (7 days curing) Tests Results for Soil-Cement-LBWA mixes (BSH Compaction).

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**
0	750.28	1112.60	1140.06	1256.07	1327.19
2	854.28	1170.65	1200.67	1308	1380.29
4	979.67	1215.36	1320.65	1393.48	1415.46
6	1122.70	1501.19	1585.76	1650	1581.69
8	1076.54	1345.67	1466.44	1507.85	1400.32

Table A3.9 (a): Unconfined Compressive Strength (14 days curing) Tests Results for Soil-Cement-LBWA mixes (BSL Compaction)

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**
0	189.93	574.45	717.28	873.59	900.81
2	359.35	600.51	801.02	922.97	952.81
4	504.17	608.07	819.05	1016.25	977.39
6	629.41	984.82	828.16	1326.37	1216.41
8	473.44	765.45	1063.28	906.38	1105.31

Table A3.9 (b): Unconfined Compressive Strength (14 days curing) Tests Results for Soil-Cement-LBWA mixes (WAS Compaction)

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**
0	615.86	941.82	1045.79	1146.56	1307.11
2	673.48	980.22	1123.04	1296.80	1334.11

4	896.22	1183.59	1545.44	1364.56	1544.61
6	952.85	1531.88	2048.40	1744.64	1911.81
8	995.54	1970.55	2255.70	1317.13	1845.19

Table A3.9 (c): Unconfined Compressive Strength (14 days curing) Tests Results for Soil-Cement-LBWA mixes (BSH Compaction)

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**
0	766.54	1081.67	1189.65	1242.04	1634.69
2	770.81	1162.80	1342.64	1344.03	1845.97
4	1037.30	1508.65	1767.53	1956.76	2008.45
6	1374.03	1824.29	2158.69	2297.81	2017.90
8	1114.22	1250.96	1551.03	1853.24	1955.45

Table A3.10 (a): Unconfined Compressive Strength (28 days curing) Test Results for Soil-Cement-LBWA Mixes (BSL compaction)

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**
0	220.14	668.15	860.36	980.65	1004.27
2	643.89	733.00	903.11	1027.90	1234.56
4	747.50	895.27	1008.47	1197.57	1285.36
6	940.86	1068.13	1235.26	1347.57	1306.48
8	1018.54	1154.26	1217.63	1278.65	1300.33

Table A3.10 (b): Unconfined Compressive Strength (28 days curing) Test Results for Soil-Cement-LBWA mixes (WAS compaction)

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**

0	419.15	913.46	1175.13	1501.96	1824.91
2	938.70	1168.45	1267.98	1725.38	1955.38
4	1263.58	1540.96	1781.37	1976.39	2104.38
6	1386.20	1675.97	1938.50	2103.27	2324.91
8	1233.90	1680.36	1846.29	1975.38	2206.38

Table A3.10 (c): Unconfined Compressive Strength (28 days curing) Test Results for Soil-Cement-LWBA mixes (BSH Compaction).

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**
0	824.10	1264.37	1917.38	1922.45	2155.09
2	957.54	1392.63	2017	2275.87	2297.45
4	1445.60	1915.08	2128.39	2307.43	2378.65
6	1755.34	2146.36	2357.46	2502.88	2615.76
8	1987.68	2180.57	2199.75	2345.76	2457.80

Table A3.11 (a): Unconfined Compressive Strength (7 days curing, 7 days soaking) test results for soil-cement-LWBA mixes (BSL compaction).

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**
0	24.05	88.41	92.50	98.75	106.93
2	55.41	62.65	90	100.75	112.86
4	102.40	142.91	167.09	178.24	217.90
6	157.83	226.39	272.49	497.26	423.53
8	150.88	312.73	331.83	377.48	426.24

Table A3.11 (b): Unconfined Compressive Strength (7 days curing, 7 days soaking) Test Results for Soil-Cement-LBWA mixes (WAS compaction).

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	2	4	6	8
0	41.60	96.46	119.89	123.71	171.35
2	53.85	97.43	120.65	131.46	134.61
4	96.10	110.49	129.51	142.51	149.82
6	110.38	125.65	146.26	197.90	178.59
8	99.36	115.44	128.95	146.38	173.42

Table A3.11 (c): Unconfined Compressive Strength (7 day curing, 7 days soaking) Test result for Soil-Cement-LBWA mixes (BSH compaction).

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	2	4	6	8
0	111.50	165.30	250.43	328.95	359.91
2	472.75	802.88	860.18	979.31	998.24
4	680.99	981.15	995.09	1168.49	1234.56
6	1040.97	1134.95	1453.92	1624.40	1426.94
8	746.86	998	739.03	417.58	367.55

Table A3.12 (a): California Bearing Ratio (Unsoaked) Tests Results for Soil-Cement-LBWA mixes (BSL Compaction)

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	2	4	6	8
0	5.15	11.96	13.20	17.77	20.51
2	10.85	13.10	14.98	21.94	22.42
4	17.51	23.56	30.70	27.50	25.30
6	21.89	29.52	34.89	46.12	27.03
8	22.57	30.50	35.46	36.21	36.74

TableA3.12 (b): California Bearing Ratio (Unsoaked) Tests Results for Soil-Cement-LBWA mixes (WAS Compaction)

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	2	4	6	8
0	7.11	22.75	23.71	27.11	33.13
2	20.23	37.60	43.50	46.48	48.35
4	30.17	45.07	52.25	55.98	76.38
6	45.79	64.01	57.99	77.82	72.69
8	31.05	69.10	63.75	73.88	59.85

Table A3.12 (c): California Bearing Ratio (Unsoaked) Tests Results for Soil-Cement-LBWA mixes (BSH Compaction)

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	2	4	6	8
0	11.71	49.41	56.10	62.89	66.54
2	34.84	46.26	50.16	59.07	63.46
4	45.30	57.95	62.65	63.07	69.86
6	53.04	62.23	72.98	82.78	69.89
8	45.37	62.98	65.75	79.33	63.97

Table A3.13 (a): California Bearing Ratio (Soaked) Tests Results for Soil-Cement-LBWA mixes (BSL Compaction)

LBWA CONTENT, %	CEMENT CONTENT, %				
	0	2	4	6	8
0	4.64	10.76	11.88	16	20.45
2	9.77	11.79	13.48	19.75	24.78
4	15.76	21.20	27.63	35.74	33.77
6	19.70	26.57	31.40	41.50	24.33

8	20.30	27.45	33.71	32.59	33.07

Table A3.13 (b): California Bearing Ratio (Soaked) Tests Results for Soil-Cement-LBWA mixes (WAS Compaction)

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**
0	6.05	19.34	20.15	23.04	29.66
2	17.20	31.96	36.55	39.51	41.10
4	25.65	38.31	44.92	47.58	48.41
6	38.92	49.29	54.41	66.15	53.29
8	26.39	58.66	54.19	62.80	50.87

Table A3.13 (c): California Bearing Ratio (Soaked) Tests Results for Soil-Cement-LBWA mixes (BSH Compaction)

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**
0	9.37	29.53	34.88	40.31	47.09
2	27.87	37.01	40.13	47.26	62.77
4	36.24	46.36	50.12	57.66	59.97
6	42.44	49.78	58.38	66.22	65.91
8	36.30	50.38	52.58	63.46	51.18

Table A3.14 (a): Resistance to Loss in Strength Tests Results for Soil-Cement-LBWA mixes (BSL Compaction)

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**
0	12.66	15.39	12.90	11.30	11.87
2	15.42	10.43	11.24	10.92	11.84

4	20.31	23.50	20.40	17.54	22.29
6	25.08	22.90	32.90	37.49	34.82
8	31.87	40.86	31.21	41.65	38.56

Table A3.14 (b): Resistance to Loss in Strength Tests Results for Soil-Cement-LBWA mixes (WAS Compaction)

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**
0	6.76	10.24	11.46	10.79	13.11
2	8	9.94	10.74	10.14	10.09
4	10.72	9.34	8.38	10.44	9.7
6	11.58	8.2	7.14	11.34	9.34
8	9.98	5.86	5.72	11.11	9.4

Table A3.14 (c): Resistance to Loss in Strength Tests Results for Soil-Cement-LBWA mixes (BSH Compaction)

LBWA CONTENT, %	**CEMENT CONTENT, %**				
	0	**2**	**4**	**6**	**8**
0	14.55	15.28	21.06	26.48	22.02
2	61.33	69.05	64.94	69.86	54.08
4	65.65	65.03	56.30	59.72	61.47
6	65.76	62.21	67.35	70.65	60.71
8	67.03	69.78	47.65	22.53	19