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ESTABLISHED AD 2001

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To establish and maintain standards and codes for the practice of Civil Engineering.

To be constantly in touch with the Government at all levels, for necessary inputs and contributions to policies and matter affecting members and the development of the profession.

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Thank You.

Engr. Dr. Jang C. Tanko, FNSE,FINCE
National Chairman

TECHNICAL COMMITTEE CHAIRMAN'S COMMENT

I wish to present a new edition of the Journal of Civil Engineering published by the Nigerian Institution of Civil Engineers. This Journal is conceived to disseminate original contributions from the Academia as well as the Public and Private Sector Practitioners of the profession of Civil Engineering. This year's edition contain scholarly interventions that permeate several branches of the Civil Engineering family profession such as Structural, Geotechnical, Water Resources and Environment, Highway and Transportation Engineering. It is with a great sense of honour and responsibility that I invite you to read and enjoy this new edition.

Engr. Dr. Ezekiel B. Coker, FNSE, FNICE
Chairman Sub-Technical Committee

2022 NATIONAL EXECUTIVE COMMITTE

COMPARATIVE ANALYSIS OF THE COMPRESSIVE STRENGTHS OF COMPRESSED STABILIZED EARTH BRICKS PRODUCED WITH LATERITIC SOIL AND CLAY

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Abstract

The aim of this study was to compare the mechanical properties of stabilized bricks made from lateritic soil and clay obtained in Benin City with a view to determining their suitability for bricks production. The objectives included: the determination of the compressive strength, water absorption rate, durability of lateritic and clay- stabilized and unstabilized bricks. Samples of reddish- brown laterite and clay were provided for the production of bricks of size 100 x 140 x 290mm and with different percentages of stabilization (0%, 5% and 10%). The results after 28days curing period showed that at 0%, 5% and 10% stabilization, lateritic bricks had compressive strength of 1.12N/mm^2, 1.65N/mm^2 and 2.18N/mm^2 and clay bricks had 1.04N/mm^2, 0.96N/mm^2 1.25N/mm^2, respectively. Generally, the brick specimens showed improvements when the percentages of the stabilizers increased. The study, therefore, concluded that the use of ordinary Portland cement improved significantly the strength and durability properties of compressed earth bricks.

1.0 Introduction

In Africa, the Egyptian civilization provides abundant evidence of the use of earth in building as found in the early human settlements at the Mermaid and Fayum sites in the Nile delta, which dates from the fifth millennia before Christ. The dominance of the Egyptian dynasty promoted buildings of prestigious structures made of brick from the Nile clay, desert sand and straw from the grain fields. These bricks were made by hand and dried in the sun before the development of the mould. The excavation at Saqqara and Abydos shows the use of bricks which were covered by stone. The art of brick vaulting was also developed in the lower Nubia, between Luxor and Aswan (Arnold, Dieter & Donald, 2001).

It is essential to look at historical evidence of the success of earth construction. It is currently estimated that over one third (Dethier, 1981) to over one half Smith, (Smith & Austin, 1989) of the world's population lives in some type of earthen dwelling. The history of earth building lacks documentation, because it has not been highly regarded compared to stone and wood (Houben & Guillaud,1989). There are cities built of raw earth, such as: Catal Huyuk in Turkey; Harappa and

Mohenjo-Daro in Pakistan; Akhlet-Aton in Egypt; Babylon in Iraq; (Easton, 1998). "30% of the world's population, or nearly 1,500,000,000 people, live in a home built in unbaked earth. Roughly 50% of the population of developing countries, the majority of rural populations, and at least 20% of urban and suburban populations live in earth homes" (Houben & Guillaud, 1989).

According to (Denyer, 1978) "Earth architecture should not of course be considered a miraculous solution to neither all our housing problems, nor one which can be applied successfully anywhere, everywhere." Before any building is constructed with earth, it is essential to identify the soil to be used. The identification process involves various tests, which need the use of a laboratory. Apart from the laboratory identification process, local knowledge of the soil and traditional skills are necessary. In Africa, suitable soil is found in most of the countries.

In many countries, the need for locally manufactured building materials can hardly be over emphasized because there is an imbalance between the demands for housing and expensive conventional building materials coupled with the depletion of traditional building materials. To address the situation, attention has been focused on low-cost alternative building materials (Madedor, 1992).

1.1 Advantages and Disadvantages of Building with Earth

The perceived hegemony of the industrialized world has for decades been directly responsible for causing an inferiority complex among earth-building cultures. Today, the most common building material on the planet is classified as ―alternative or worse, primitive‖ (Rael, 2008). At the dawn of every country's transition to an industrialized society, the phenomenon of abandoning its earth-building traditions creates a significant risk of depleting precious natural resources such as forest wood used for brick firing and an unsustainable and unaffordable investment in construction projects using, industrially-produced materials such as concrete, which often performs poorly in developing nations‖ (Adam, 2001), and in doing so, also causes the regrettable loss of traditional cultural knowledge and heritage.

While it is true that the makeup of soil, which differs from one place to another, makes it difficult to create material standards for earth and building codes for earth buildings (Rael, 2008), its potential cannot be overlooked with the considerable benefits of earth construction, namely ecological and economic sustainability.

1.1.1 Advantages

Given the long-term vitality of soil-based construction materials in human history, it is not surprising that these materials offer several advantages, some of which have been alluded to in earlier sections. The principal advantages of CEBs include their ability to insulate against thermal extremes, their very low cost of construction and transportation, greatly reduced environmental pollution, their recyclability; their relatively low technological complexity (which allows for do-it-yourself construction); and their marked preservation of timber and other materials, reducing ecological degradation through mining and deforestation (Minke, 2006).

Soil is available in large quantities in most regions of the world and is easily accessible to low-income groups. In some locations, earth is the only material available and it requires simple, low-cost equipment to produce building materials. Even with the addition of stabilizers does not drastically weaken these key advantages, and CEBs are suitable as a construction material for the majority of many common architectural designs. Lastly, CEBs are highly competitive against their more conventional alternatives in their fire resistance and are virtually non-combustible (Adam, 2001).

1.1.2Disadvantages

The perceived lack of durability of earth building techniques has created a barrier to its use and adaptation in modern construction industries. Physical disadvantages of building materials constructed from soil commonly include their reduced durability if not regularly maintained and properly protected, particularly in areas affected by medium to high rainfall, their low tensile strength (being particularly poor in their resistance to bending moments and seismic activity, and their limited applicability only in compression (e.g. bearing walls, domes and vaults), their low resistance to abrasion and impact if not sufficiently reinforced or protected, and perhaps most notably their low acceptability amongst most social groups (being considered by many to be a second class and generally inferior building material). On account of these problems, earth as a building material lacks institutional acceptability in most countries and as a result building codes and performance standards have not been fully developed, despite highly redeeming qualities and potential for technological development through stabilization techniques and augmented construction (Adam, 2001).

In many countries, the need for locally manufactured building materials can hardly be overemphasized because there is an imbalance between the demands for housing and expensive conventional building materials coupled with the depletion of traditional building materials. To address this situation, attention has been focused on low-cost alternative building materials (Balkema, 1983). Like many urbanizing societies, Nigeria is experiencing acute difficulties with the provision of adequate housing for her citizens, especially in the urban centre. The building industry in Nigeria as in other African countries is characterized by a high component of imported raw materials. Dependency on imported building materials is a factor that has contributed to the high cost of houses and it is a factor that has made it increasingly impossible to build low cost houses (Ogu & Ogbuozobe, 2001). Successive research proposals have emphasized the need to encourage the use of alternative building technologies as a way of minimizing dependence on imported substitutes, the enormous potentials of alternative building systems have been discovered and effectively utilized in contemporary ways in countries such as India and Mexico. But in Nigeria, these sustainable solutions are not promoted; the unworkable and unsustainable solutions are still been imported and used for construction (Oruwari, Jev, & Owei, 2000).

The cost of building materials and components is known to constitute about 60-70% of the cost of the buildings. This inevitably implies that high cost of building materials will make construction cost equally high. On the other hand, availability on a sustainable basis of relatively cheap, locally produced and tested building materials and technologies is the obvious anti-dote to high cost of housing construction in Nigeria (Mansur, Abdul-Hamid, & Yusof, 2016). However, the scenario is different; unworkable and unsustainable solutions are still been imported and used for construction. The continued dependence on imported building materials has not only imposed additional strains on an already acute balance of payments situation in Nigeria; it has fuelled inflation in the construction sector causing cost over- runs in public projects. It also inhibits private initiatives in shelter production and makes it impossible to provide shelter for the low-income households in the country. Ironically, Nigeria (although it suffers from scarcity and import dependence) is endowed with abundant building materials that have the lowest gross energy requirements. Crucial to these facts would be the strengthening of domestic technological capability to produce sustainable alternative building materials from the available local resources (Oruwari, Jev, & Owei, 2000).

Earth, undoubtedly is the oldest building material known. Even though building with earth once fell out of popularity when modern building materials and methods were discovered. However, there is growing concern in environmental and ecological issue globally, thus, an increased used of earth as a building material. In comparison with another building material, Compressed Stabilized Earth Bricks (CSEB) offered numbers of advantages including an increase in the utilization of local material and reduction in the transportation cost as the production is in-situ. It also makes quality housing available to more people and generates local economy rather than spending for imported materials, faster and easier construction method resulted in less skilled labor required, good strength, insulation and thermal properties, less carbon emission and embodied energy in the production phase, create extremely low level of waste and easily dispose off, cause no direct environmental pollution during the whole life cycle. Earth brick also have the ability to absorb atmospheric moisture which resulted create healthy environment inside a building for its occupant (Morton, 2008). The earth used is generally subsoil, thus the topsoil can be used for agriculture. Building with local materials can employ local people, and is more sustainable in crisis (Morel, 2001). One of the drawbacks using earth alone as a material for construction is its durability which is strongly related to its compressive strength (Guettala, Abibsi, & Houari, 2006). Because most soil in their natural condition lack the strength, dimensional stability and durability required for building construction, the technique to enhance natural durability and strength of soil defined as soil stabilization. There are several types of stabilization: first, mechanical stabilization; second; physical stabilization; and third chemical stabilization (Walker, 1995; Billong, 2008). Presently, the gap between the rising demand and the stagnating and in many cases declining production levels is widening at an alarming rate leading to the spiraling of prices of building materials in Nigeria, seriously affecting the affordability of housing for the vast majority of the population. An average urban dweller spends between 40-60% of his income on housing. Though there is dare scarcity of data on the quantum of housing shortages, according to Federal Ministry of Works and Housing, some 5 million housing units were estimated as required at the national level over the period from 1994-2000 to maintain a reasonably dynamic balance in the market. But recently the figure is put at 15-17 million. The question is what has building materials development and technology done to ameliorate the situation?

1.2 Statement of the Research Problem

On sustainability stand point, in recent time researches are now been focused on investigating locally available materials that could be used effectively to substitute conventional materials. Compressed Stabilized Earth Brick (CSEB) is a green technology in building construction, which has been explored using locally available materials. There is housing deficit in Nigeria; authors established that the housing demand in Nigeria which will address the deficit is 500,000 units of houses per year (Mabogunje, 2007). However, achieving adequate housing cannot be only by conventional materials usage because of their huge cost hence, the need for local materials utilization in low cost housing cannot be overemphasized. CSEB being readily available in most part of the Country is less expensive compared to sandcrete bricks (Adam & Agib in Sudan, 2001).

According to the UNESCO Chair Earthen Architecture (Rigassi & Terre, 1985), clay and lateritic soils are suitable to be used in the production of CSEBs, and they are readily available in Nigeria but the more suitable of them has not been well established. Currently in Nigeria, there is no standard in producing CSEB made from lateritic soil and clay. Therefore, this study aims at comparing the compressive strength, water absorption rate and strength deterioration in sulphuric acid medium of CSEBs made from lateritic soil and clay.

1.3 Aim and Objectives

The aim of this study was to compare the mechanical properties of stabilized bricks made from lateritic soil and clay obtained in Benin City with a view to determining their suitability for bricks production. The specific objectives were:

a. To determine the compressive strength of lateritic and clay stabilized bricks
b. To determine the water absorption rate of stabilized brick.
c. To determine the durability of lateritic and clay stabilized bricks.

2.0 Research Methodology

2.1 Research Design

The approach involved laboratory experimental work. The experimental work carried out included laboratory tests on CSEB. It involved determining the compressive strength, water absorption rate, and the durability of CSEB made with laterite and clay at some percentages of cement. In addition to the description of materials, basic laboratory analysis, experimental procedures for the research and the method of data collection including statistical analysis were discussed.

2.2 Materials and Methods

2.2.1 Earth Characterization and Preparation

Preliminary tests on the soils that were used were evaluated first for the purpose of classifying and identifying the types of soils. The tests performed were as follows: Soil Particle Size Test, Jar test (sedimentation) Moisture Content Test, Specific Gravity Tests, the Atterberg Limits Tests, linear shrinkage and Compaction Factor Test in accordance with BS1377: Part 2:1990. In order to obtain initial uniform moisture content, the soils were stored under a shed at a room temperature of 22°C for some weeks before being broken down. In order that the stored soil could dry out more evenly, the samples were thinly spread out and regularly turned over several times. Attempts were also made to provide some information about the equipment that was most fundamental for the success of this work. The laterite samples were air–dried for seven days in a cool, dry place. Air drying was necessary to enhance grinding and sieving of the laterite. After drying, grinding was carried out using a punner and hammer to break the lumps present in the soil. Sieving was then done to remove over size materials from the laterite samples using a wire mesh screen with aperture of about 6mm in diameter as recommended by Oshodi (2004). Fine materials passing through the sieve were collected for use while those retained were discarded.

Plate 1: Breaking the clay lumps

Plate 2: Laterite soil sample

2.2.2 Preparation of Brick Samples

The bricks were produced using a locally fabricated wooden mould of size 100x140x290mm. The production process comprises batching, mixing, casting and compaction of the bricks. The materials used for the production of bricks were measured by weight in accordance with the predetermined percentages of stabilization (0%, 5% and 10%) and the optimum moisture contents determined from the field. According to NBRRI, one bag of cement (50kg) with 9 wheel barrows of laterite at a maximum of 5% cement stabilization will give 120 bricks. Therefore, the batching information was gotten from this based on the number of bricks needed.

Table 1: Batching Information for Laterite Samples Used

% of stabilization	Laterite (kg)	Cement (kg)	Water (kg)	Water/cement ratio
0	87.6	-	10	∞
5	87.6	5	12.2	2.44
10	87.6	10	14.1	1.41

Table 2: Batching Information for Clay Samples Used

% of stabilization	Laterite (kg)	Cement (kg)	Water (kg)	Water/cement ratio
0	78.4	-	10.5	∞
5	78.4	5	13.4	2.68
10	78.4	10	15	1.5

The mixing was performed on an impermeable surface made free (by sweeping and brushing or scraping) from all harmful materials that could alter the properties of the mix. The measured laterite sample was spread using a shovel to a reasonably large surface area. Cement was then spread evenly on the laterite and mixed thoroughly with the shovel. The dry mixture was spread again to receive water, which was added gradually while mixing until the optimum moisture content of the mixture was attained. According to Bolyn (2018), a standard 9 liter capacity bucket holds about 15kg of dried soil. The optimum moisture content (OMC) of the mixture was determined by progressively wetting the soil, collecting handfuls of the soil, compressing it firmly in the fist, then allowing it to drop on a hard and flat surface from a height of approximately 1.0 m. When the soil breaks into four or five parts, the water content is considered correct (National Building Code, 2006).

After the wooden mould was rid of all impurities, it was coupled together and oiled to enhance the demoulding of the blocks. The wet mixture was filled into the mould in 2 layers, with each layer being compacted on a level and rigid platform. The excess mixture was scraped off, and the mould was levelled using a straight edge. The mould and its contents were left for two hours before the removal of the mould. The bricks were first allowed to air dry for 24 hours under a shade constructed from a polythene sheet. Thereafter, water was sprinkled on the bricks in the morning and evening, and the bricks were covered with a polythene sheet for one week to continue the curing process and prevent rapid drying of the blocks, which could lead to shrinkage cracking. The blocks were later stacked in rows and columns until they were ready for strength and durability tests.

Plate 3: Batching of both soil samples

Plate 4: mixing of both soil samples

Plate 5: Mearsurement of water

Plate 6: Moulding already mixed samples

Plate 7: brick samples after productiion

Plate 8: stsacking of brick samples

2.2.3 Testing of Bricks

Previous study by the Nigerian Building and Road Research Institute (NBRRI) involved the production of laterite bricks that were used for the construction of a bungalow. In that study, the NBRRI proposed the following minimum specifications as requirements for laterite bricks: a bulk density of 1810 kg/m^3, a water absorption of 12.5%, a compressive strength of 1.65 N/mm^2 and a durability of 6.9% with a maximum cement content fixed at 5%. Compressive strength decreases because of increase of laterite content.

Durability, water absorption and compressive strength tests were performed on the bricks. The durability of the bricks was determined through abrasion testing. After the bricks attained the specified ages, two bricks were selected at random and weighed in the laboratory; their weight was recorded. In these tests the bricks were subjected to mechanical erosion applied by brushing, with a metal brush at a constant pressure over a number of cycles in a back-and-forth motion 50 times, where one back and forth motion was considered a single stroke. After being brushed, the bricks were weighed again to determine the amount of material or particles abraded. This procedure was repeated for all the bricks produced with various cement contents and for bricks of various ages. The aim of this test was to ascertain the effect that weathering would have on the bricks. Wind, rain storm and other factors generally have wearing effect on building walls. It is useful at this stage to know how the bricks would stand the test of the weather conditions so as to improve the stabilization of the bricks.

The percentage of abrasive resistance is stated as follows:

$$D = \frac{(W_1 - W_2)}{W_1} \times 100$$

Where:

D= Percentage Durability

W_1 = weight of brick before abrasion

W_2 = weight of brick after abrasion

For the water absorption tests, two bricks were randomly selected from each group of the specified age and were weighed on a balance. These bricks were then immersed completely in water for 24 hours, after which they were removed and weighed again. The percentages of water absorbed by the bricks were estimated as follows:

$$Wa = \frac{Ws - Wd}{Wd} \times 100$$

Where:

Wa = percentage moisture absorption

Ws = weight of soaked brick

Wd = weight of dry brick

Plate 9: Curing

Plate 10: Samples before weighing

Plate 11: Weighing of samples

Compressive strength tests were performed to determine the load-bearing capacities of the bricks. The dry compressive strengths were determined. For the dry compressive strength tests, the bricks aged 7, 14, 21 and 28 days were transported from the curing or stacking area to the laboratory two hours prior to the test to normalize the temperature and to ensure that the brick was relatively dry. The weight of each brick was measured before the brick was placed onto the compression testing machine such that the top and bottom, as moulded, lied horizontally on a flat metal plate. The brick was then crushed, and the corresponding failure load was recorded. The crushing force was divided by the sectional area of the brick to arrive at the compressive strength. This formula was used in the determination of the compressive strength:

$$Cs = \frac{Cf}{A}$$

where: Cs = compressive strength

Cf = crushing force

A = sectional area

Plate 12: Compressive strength test

3.0 Data Presentation and Analysis

3.1 Water Absorption Tests

Table 3: Analysis of results for water absorption rate of laterite and clay bricks

	Stabilization	Laterite	Clay	P
Dry mass	0%	6.65	6.05	0.143
	5%	6.75	6.15	0.333
	10%	7.35	6.65	0.081
Wet mass	0%	-	-	-
	5%	7.55	7.15	0.576

	10%	8.	7.30	0.323
Water absorption	0%	-	-	-
	5%	10.60*	15.25*	0.014
	10%	9.60	9.66	0.992

* Significant difference
Source: field work (2018)

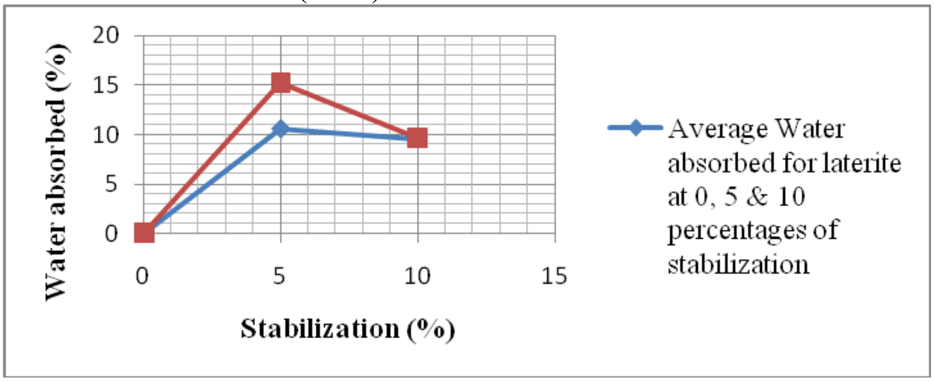

Fig 1 showing the level of water absorbed by the samples at different stabilization

Table 2: Analysis of results for durability of bricks

	Stabilization	Laterite	Clay	P
Mass before abrasion	0%	7.65	7.55	0.293
	5%	6.85	6.75	0.764
	10%	6.80	7.30	0.349
Mass after abrasion	0%	7.42*	5.35*	0.044
	5%	6.68	5.85	0.167
	10%	6.66	6.95	0.589
Abraded away	0%	2.5	29.1	0.062
	5%	2.55	13.4	0.075
	10%	2.13	4.84	0.012

* Significant difference Source: field work (2018)

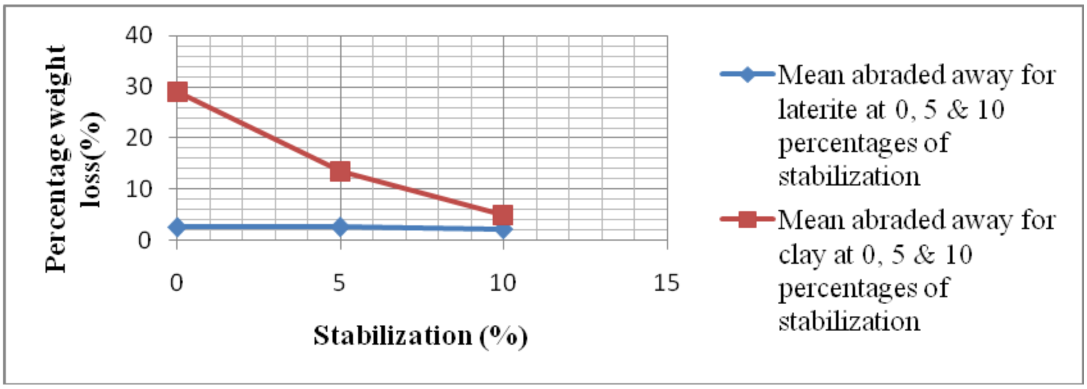

Fig. 2: Percentage weight loss by the samples at different stabilization after abrasion

3.2 Compressive Strength

The results of the compressive strengths are presented in tables and Figures below for laterite and clay CSEB. It can be observed from Figure 3 that the compressive strength of cement stabilized bricks increases as the percentage of stabilization increases. The compressive strength of unstabilized bricks (The control) varies from 0.39 N/mm^2 to 1.10 N/mm^2 as the curing age increases from 7 to 28 days. For cement stabilized laterite bricks, it varies from 0.76 N/mm^2 to 1.65 N/mm^2, 0.92 N/mm^2 to 2.18 N/mm^2 for 5% and 10% stabilization, respectively, during the same period. The minimum 7 days dry compressive strength for 5% cement stabilized bricks of not less than 1.60N/mm^2 as recommended by National Building Code (2006) could not be satisfied. The 28 days dry compressive strength of manually produced bricks with 5% cement stabilization, of not less than 2.0N/mm^2 as recommended by NBRRI (2006) was also not satisfied.

Beyond 5% cement stabilization however, all the bricks satisfied the minimum 28 days dry compressive strength. Since lateritic bricks with 5% cement stabilization do not satisfy the minimum requirements as specified by the operating codes, 10% stabilization is recommended for use. The extra 5% cement content over what was used by Madedor (1992) is compensated for by the non usage of mortar in laying the bricks. The compressive strength of unstabilized and stabilized clay bricks do not follow a regular pattern as discussed above. The 7 days strengths could not be determined as the samples were very weak to be crushed. The compressive strength varies from 0 N/mm^2 to 1.04 N/mm^2, for 0% stabilization and 0.34 N/mm^2 to 1.06 N/mm^2, 0.61 N/mm^2 to 1.47N/mm^2 for 5% and 10% stabilization respectively as the curing age increases from 7 to 28 days. None of the bricks met the minimum requirements at 7 and 28 days as specified by the available codes. This is due to the fact that clay has very high mud content; and therefore, could not bear the compressive stress imposed.

Table 5: Analysis of results for compressive strength brick samples at 0% stabilization

	Days	Laterite	Clay
Dry mass	7	3.45	
	14	7.55	6.75
	21	6.75	7.30
	28	6.60	6.60
Dry density	7	1797.50	
	14	1859.00	1662.00
	21	1662.00	1797.50

	28	1625.00	1625.00
Dry crushing force	7	16.00	
	14	35.10	10.70
	21	36.25	25.65
	28	45.35	42.35
Dry compressive strength	7	0.39	
	14	0.86	0.26
	21	0.89	0.63
	28	1.12	1.04

Source: field work (2018)

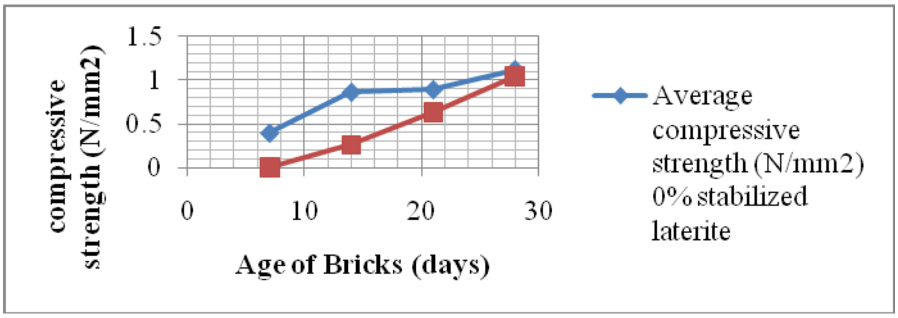

Fig 3 Graph showing comparison of compressive strength development for 0% stabilization for both clay & laterite samples

Table 6: Analysis of results for compressive strength brick samples at 5% stabilization

	Days	Laterite	Clay
Dry mass	7	6.80	7.95
	14	6.85	6.85
	21	7.00	6.70
	28	5.65	6.55
Dry density	7	1674.50	1958.00
	14	1686.50	1687.00
	21	1723.50	1649.50
	28	1391.00	1612.50
Dry crushing force	7	32.00	13.90
	14	51.00	21.30
	21	67.00	32.25
	28	67.20	34.40
Dry compressive strength	7	0.78	0.34
	14	1.26	0.53
	21	1.65	0.80
	28	1.65	0.96

Source: field work (2018)

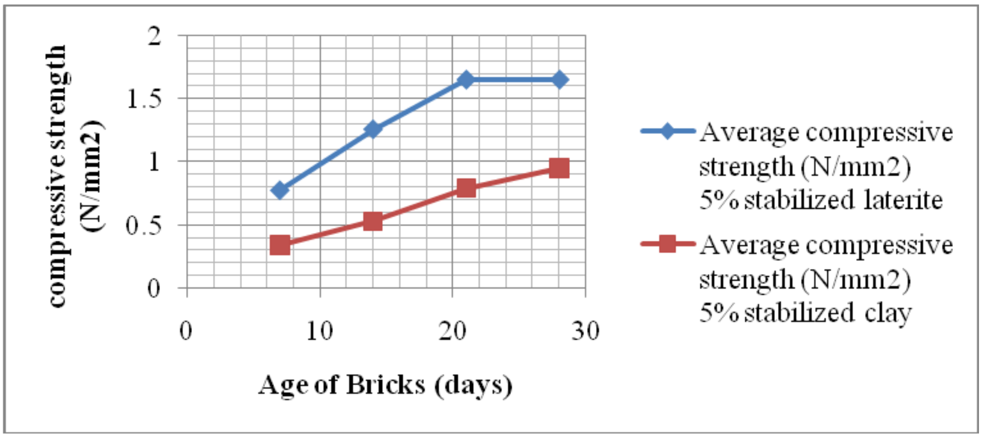

Fig 4 Graph showing comparison of compressive strength development for 5% stabilization for both clay & laterite samples

Table 7: Analysis of results for compressive strength brick samples at 10% stabilization

	Days	Laterite	Clay
Dry mass	7	7.65	6.90
	14	6.85	6.50
	21	6.80	6.50
	28	6.25	5.80
Dry density	7	1883.50	1699.00
	14	1687.00	16.50
	21	1674.50	16.50
	28	1539.00	1430.00
Dry crushing force	7	31.25	25.15
	14	36.40	33.90
	21	71.25	42.35
	28	88.75	51.10
Dry compressive strength	7	0.77	0.62
	14	0.90	0.84
	21	1.76	1.256
	28	2.18	1.04

Source: field work (2018)

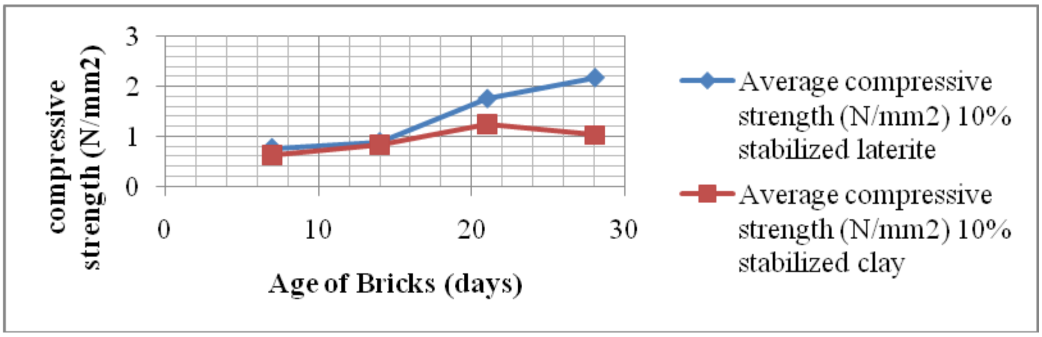

Fig 5 Graph showing comparison of compressive strength development for 10% stabilization for both clay & laterite samples

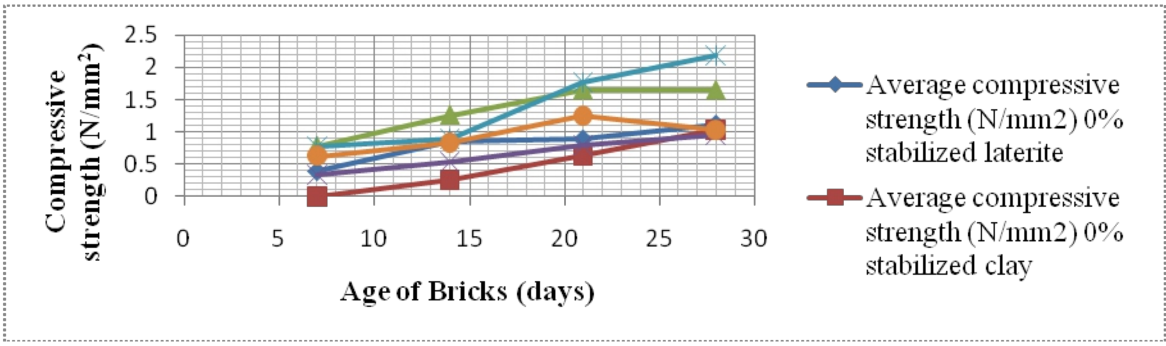

Fig 6 Graph showing comparison of compressive strength development for all percentages of stabilization for both clay & laterite samples

4.0 Conclusion and Recommendations

4.1 Conclusion

Due to the fact that insufficient data has existed to document the advantages of some CEB construction methods over others, expansion and increased use of this ancient technology has been slow. By promoting the development of CEB manufacturing techniques, including improved soil selection and more advanced types of manufacturing methods and stabilization techniques, CEBs can have a significant and sustainable impact to the housing needs of the developing nations in tropical climates. Properly stabilized CEBs, with appropriate climate-based technological modifications and adaptations, can be an optimal choice for constructing several types of structures in parts of the world that have been historically unable to utilize this construction material.

In the light of the results of experimental test reported in this study it can be concluded that laterite soil is more suitable than clay for CEB production. The dry compressive strength values obtained for all the stabilized earth bricks were within specified limits and suitable for single storey dwelling. Apart from bricks without stabilizer all other bricks had dry compressive strength in excess of the minimum strength requirements for masonry in most codes, confirming their suitability for two or three storey buildings. There was a general reduction in water absorption as Ordinary Portland cement content increases. The earth bricks showed a remarkable improvement in their durability properties (water absorption and abrasion). As the quantity of Ordinary Portland cement in the soil bricks increases, the ability of the soil blocks to resist abrasion also increased appreciably. Furthermore, laterite bricks also tend to have high water exclusion property as the quantity of Ordinary Portland cement increases making it suitable as a masonry unit. From the initial rate of water absorption test it can be deduced that Ordinary Portland cement has low permeability of moisture.

It is appropriate to conclude that the use of Ordinary Portland cement will improve significantly the strength and durability properties of compressed earth bricks. The use of Ordinary Portland cement alone should be used sparingly especially lower percentages and in poor soils. In other to obtain optimum compressive strength of earth bricks, Ordinary Portland cement should be used as admixture.

4.2 Recommendations

From the outcome of this current study the following recommendations are made:

a. Block press be used in molding the bricks rather than using the proctor hammer in an improvised mould.
b. Ordinary Portland cement should at an appropriate percentage should be used as a stabilizing agent in that it will improve significantly the strength and durability properties of compressed earth bricks
c. In selecting suitable soil for compressed bricks production, earth from different locations should be collected and investigated for their suitability rather than selection based on availability and social acceptance.
d. In reality, rain storm is the commonest wearing agent of wall surfaces. This implies that in subsequent studies wind driven rain simulation test be used to test for abrasion resistance test.
e. Again other test such as chemical test and tension should be carried out to determine the resistance of the earth bricks to other weaknesses of earth bricks.

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FACTORS THAT AFFECT EFFECTIVE FENESTRATION REQUIREMENTS IN BUILDING ENVELOPES

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Abstract

The aim of this study was to appraise the factors that affect effective fenestration requirements in building envelopes. The following objectives were to: identify major factors that affect fenestration in building envelopes; determine the extent of impact of each of these factors that affect fenestration and assessment of thermal performance per fenestration/ventilation feature. The study employed a survey technique in eliciting information on the subject matter. A total of one hundred and fifty professionals were interviewed and questionnaires administered on them and retrieved by the researcher. A 4- point Likert scale was adopted with a maximum score of 4.0 and a criterion mean score of 2.50. Different combinations of natural fenestration/ ventilation features were also considered and their performance analyzed in the course of this study. The findings of these focus group interviews, questionnaires and personal observations were correlated with existing mathematical relationships of the major factors that affect fenestration and design solutions were made. The study found out that the major factors that affect fenestration in building envelopes are, in accordance with their ranks: thermal conductance/ u-values of components of the envelope, TC/U-V (1); solar exposure, SE (2); total area of glazing elements, TAGE (3); solar heat gain, SHGC (4); orientation of glazed elements, OGE (5); climatic zone, CLZ (6); total floor area, FLA (7); projections and shades, P & S (8). The issue of fenestration is as old as the concept of building, itself. Thermal comfort in buildings is paramount in design and construction considerations. At the design stage, effective fenestration is affected by: thermal conductance/ u-values of components of the envelope; solar exposure; total area of glazing elements; solar heat gain; orientation of glazed elements; climatic zone; total floor area; projections and shades. A methodical design approach was recommended for effective fenestration in building envelopes.

Keywords: Building envelope, fenestration, ventilation, features, thermal efficiency, energy, design.

major role in setting the interior atmosphere and defining the external appearance of a structure (Barry, 2011). It is also used to create daylighting in building envelopes. Whether side daylighting, top daylighting, or a combination is used in a particular building project, the design challenge of balance and control is critical to a successful outcome (Frontczak &Wargocki, 2011). Daylighting has direct impacts on things beyond the provision of natural light and views. If the light is too intense or creates too much of a contrast within a space, then it will be regarded as uncomfortable glare that is not welcome by the occupants (Corgnati et al, 2011). The constant exposure of materials and finishes to sunlight can also cause colors to fade and materials to break down. However, during warm weather or in buildings that tend to require more cooling than heating, increased daylighting can bring an unintended penalty of too much solar heating, thus making people less comfortable, causing more air conditioning to run, and consume more energy—all the opposite of our original design intentions (Attia et al., 2013). The rising rate of thermal discomfort reported annually by building occupants which has been making occupants of very beautiful homes to find thermal comfort outside their homes is very worrisome. This problem is as a result of poor computation of fenestration requirements for the various building envelopes.

The aim of this paper, therefore, was to appraise fenestration in buildings and determine appropriate methods of computation of its effectiveness. The following were the objectives of the paper:

a. Identification of the major factors that affect fenestration in building envelopes;
b. Determination of the extent of impact of each of these factors on the effectiveness of fenestration and
c. Assessment of the thermal performance per ventilation feature.

2.0 Methodology

This study employed a survey technique in eliciting information on the subject matter. A total of one hundred and fifty professionals were interviewed and questionnaires administered on them and retrieved by the researcher. A 4- point Likert scale was adopted with a maximum score of 4.0 and a criterion mean score of 2.50. The findings of these focus group interviews, questionnaires and personal observations were correlated with existing mathematical relationships of the major factors that affect fenestration and recommendations were made.

The following combinations of natural fenestration/ ventilation features were also considered and their performance analyzed in the course of this study: Night Purge (NP); Operable Windows (OW); Clerestory Windows and Vented Skylights (CWVS); Wind Capture Facades (WCF); Night Purge/ Wind Capture Facades (NP/WCF); Night Purge/ Clerestory Windows and Vented Skylights (NP/CWVS); Night Purge/ Wind Capture Facades/ Clerestory Windows and Vented Skylights (NP/WCF/CWVS) in accordance with Ruth (2016).

3.0 Results andDiscussion

3.1 Factors that affect fenestration in building envelopes

Table 1: Major Factors that Affect Fenestration in Building Envelopes

S/N	Factor	Mean Score	Rank
1	Solar exposure	3.25	2
2	Climatic zone	3.10	6
3	Projections and shades	2.96	8
4	Solar heat gain	3.15	4
5	Total area of glazing elements	3.20	3
6	Orientation of glazing elements	3.12	5
7	Thermal conductance / u-values components of the building envelope	3.54	1
8	Total floor area of buildings	3.0	7

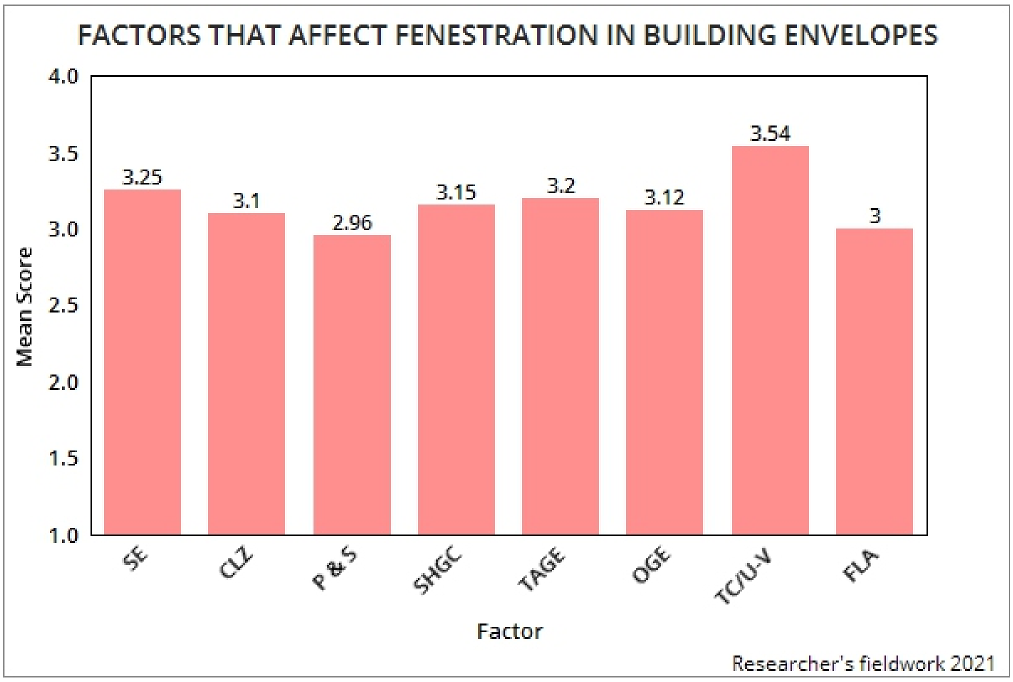


Fig. 1: Major factors that affect fenestration in building envelopes

From Table 1 and fig.1, the major factors that affect fenestration in building envelopes are: solar exposure; climatic zone; projections and shades; solar heat gain; total area of glazing elements; orientation of glazed elements; thermal conductance/ u-values of components of the envelope; total floor area.

3.2 Ranking of Factors that affect Fenestration in Building Envelopes

From Table 1, it could be seen that the major factors that affect fenestration in building envelopes are, in accordance with their ranks: thermal conductance/ u-values of components of the envelope, TC/U-V (1); solar exposure, SE (2); total area of glazing elements, TAGE (3); solar heat gain, SHGC (4); orientation of glazed elements, OGE (5); climatic zone, CLZ (6); total floor area, FLA (7); projections and shades, P & S (8).

3.3 Thermal Performance

Table 2: Thermal Performance per Ventilation Feature

Ventilation Feature	Thermal Performance (watt/s)	Per unit/ time	Rank
Night Purge (NP)	408.3	0.87	6
Operable Windows (OW)	357.4	0.76	7
Clerestory Windows and Vented Skylights, (CWVS)	569.6	1.21	5
Wind Capture Facades, (WCF)	633.0	1.35	4
Night Purge/ Wind Capture Facades, (NP/WCF)	709.0	1.51	3
Night Purge/ Clerestory Windows and Vented Skylights (NP/CWVS)	744.9	1.59	2
Night Purge/ Wind Capture Facades/ Clerestory Windows and Vented Skylights	873.7	1.87	1

(NP/WCF/CWVS)			

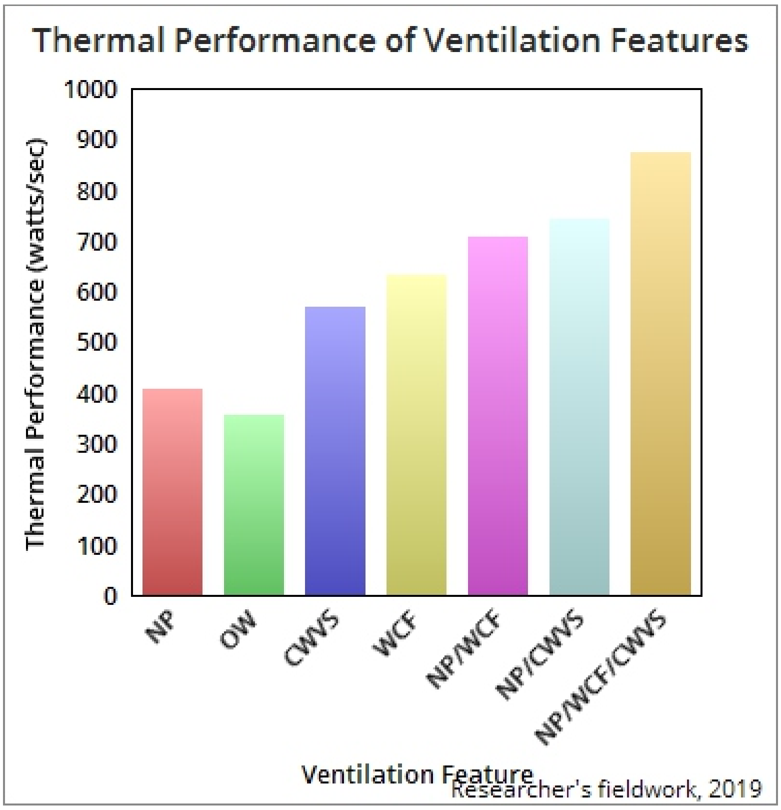


Fig.2: Thermal Performance of Ventilation Features (source: Table 4.15)

Table 2 shows the thermal performance of the various natural ventilation features, as determined by the researcher. The best performance was exhibited by a combination of Night Purge/ Wind Capture Façade/ Clerestory Window and Vented Skylights with a performance rating of 873.7 watt/sec. This was closely followed by the combination of Night purge/Clerestory Window and Vented Skylights with a performance rating of 744.9 watt/sec. A combination of Night purge and Wind capture façade had a performance rating of 709.0 watt/sec. The Wind Capture Façade, alone had a thermal performance of 633 watts/sec. The natural ventilation feature that had the least thermal performance was the Operable Window System with a value of 408.3 watts/ sec.

4.0 Conclusion and Recommendations

4.1 Conclusion

Thermal comfort in buildings is paramount in design and construction considerations. At the design stage, effective fenestration is affected by: thermal conductance/ u-values of components of the envelope; solar exposure; total area of glazing elements; solar heat gain; orientation of glazed elements; climatic zone; total floor area; projections and shades. In terms of the fenestration/ ventilation features, all the combinations had a satisfactory performance, especially that of Night Purge/ Wind Capture Façade/ Clerestory Window and Vented Skylights with a performance rating of 873.7 watt/sec. However, a satisfactory natural fenestration/ ventilation may not be achieved despite correct orientation, cross-ventilation, overhanging eaves and perforated walls in the absence and/or of the knowledge of the pattern of air flow for the district where the building is located.

4.2 Recommendations

The following criteria should be considered while designing fenestration features in building envelopes:

i. Net floor area of each storey of the proposed building measured within the enclosing walls;

ii. The total area of the glazing elements of each storey. This should not be greater than 15% of the net floor area;

iii. The Climatic Zone applicable to the location of the building envelope;

iv. The relevant constants for conductance and SHGC applicable to the identified climatic zone;

v. The maximum conductance and solar heat gain (SHGC) values that the glazing in each storey shall not exceed. Maximum conductance is given by = (net floor area of storey) x (constant, CU) and maximum Solar heat gain = (net floor area of storey) x (constant, CSHGC);

vi. The aggregate conductance value for the glazing in each storey should be obtained by adding together the conductance of each glazing element, where the conductance of a single glazing element is calculated as follows:

Conductance = (area of glazing element) x (U-value of glazing element).

That is, the aggregate conductance value of glazing elements should be obtained from (A1xU1) + (A2xU2) + (A3xU3) + (A4xU4) + (A5xU5) +...

vii. Size of openings should be between 20% and 35% of wall area and openings should be on the North and South walls and at body height, on windward side. Direct sunlight should be excluded from openings;

viii. Walls and floors should be heavy and have over 8 hours time lag;

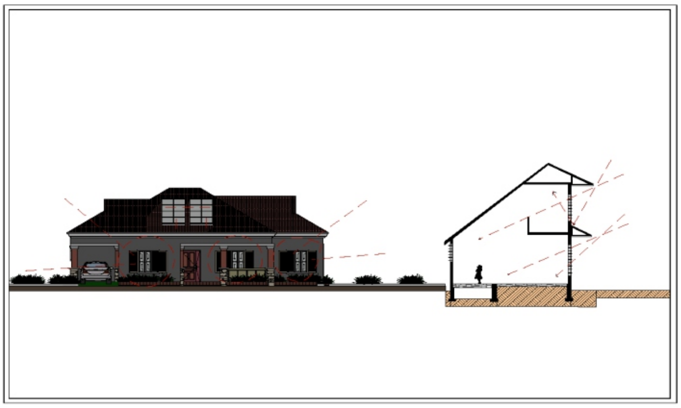
ix. Roofs should be heavy and have over 8 hours time lag;

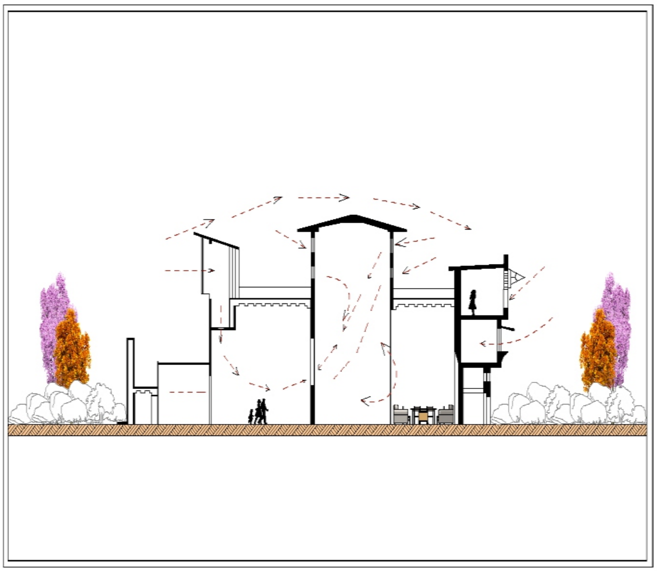
x. When there is a conflict between orientation for wind penetration and orientation for sun exclusion, the former should take precedence because the latter could be achieved by the use of shading devices over windows;

xi. Combinations of natural fenestration/ ventilation features such as Night Purge (NP); Operable Windows (OW); Clerestory Windows and Vented Skylights (CWVS); Wind Capture Facades (WCF); Night Purge/ Wind Capture Facades (NP/WCF); Night Purge/ Clerestory Windows and Vented Skylights (NP/CWVS);

Night Purge/ Wind Capture Facades/ Clerestory Windows and Vented Skylights (NP/WCF/CWVS) should always be considered during the design of building envelopes.

Appendices





Appendices.1-3: Convectional distribution of natural air in available fenestration in building envelopes employing different fenestration features (Source: Edokpolo, 2020)

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MODAL FREQUENCY RESPONSE OF REINFORCED CONCRETE TO CORROSION -INDUCED DAMAGE

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Abstract

The deterioration of reinforced concrete structures as a result of corrosion of rebar has become a serious issue in the study of structural reliability. Corrosion-induced damage causes a shift in the mechanical and modal signatures of the concrete structure, especially the modal frequency. This paper describes the impact of corrosion-induced damage on modal frequency of reinforced concrete under natural climate environment in the tropical region. Two sets of concrete specimens were prepared. One set has zero sodium chloride content, while the other set has 1 % sodium chloride content to accelerate corrosion. The specimens were exposed to different exposure conditions after 28 days normal concrete curing in water. The modal frequency response and the corrosion severity of the reinforced concrete specimens were determined at time intervals. The results show that it took about four months for the modal frequency to show a significant response to the corrosion-induced damage under corrosive environment. The findings of the study indicate that corrosion-induced damage, which distorts modal frequency, is environment dependent. Also, corrosion severity increases with exposure duration, and exposure condition influences the degree of the corrosion severity increase. The information could be utilised to develop a framework to monitor the service life of concrete structure.

Keywords. Modal frequency response, Corrosion severity, Corrosion-induced damage, Reinforced concrete, Modal parameter.

1.0 Introduction

Modal frequency is the frequency at which structures naturally tend to vibrate when subjected to a disturbance. It is one of the modal parameters that reveal the dynamic characteristics of a concrete structure through modal analysis. Corrosion and other environmental factors can alter dynamic characterises of civil engineering structures. Any variation of the physical properties of a given structure affects the modal parameters and influences the performance of the structure (Huynh et al, 2003).

The degeneration of reinforcing steel bar in concrete under an environment liable to corrosion has been a worrying factor for centuries, especially to the construction industry. The rebar in concrete experiences loss of mass and cross-sectional area as a result of corrosion. The rust of the rebar leads to weakening of the bond at the concrete-rebar interface, and eventually concrete cracking (Shetty et al., 2012). The properties of the concrete shift from the normal gradually, until structural failure occurs. However, corrosion of rebar in concrete is environment dependent (Hu et al., 2015).

Several studies have been done to quantify the effect of corrosion on the properties of steel structures (Saad-Eldeen et al., 2012; Jurišic and Parunoy, 2015) and reinforced concrete structures (Bhaskar et al., 2015; Wang and Liu, 2010; Li and Yang, 2011; Bhargava et al., 2007). Mitra et al., (2010) proposed a fuzzy logic based approach to assess the condition of corrosion-distressed reinforced concrete buildings. Cheung et al., (2009) developed a comprehensive numerical simulation technique to evaluate the corrosion performance in reinforced concrete bridge structures exposed to chloride environment. Bhargava et al., (2006) proposed an analytical model to estimate the time needed for a reinforced concrete under the influence of corrosion to experience concrete cover cracking and weight loss of the reinforcing bar. Verma et al., (2013) proposed a ten-point condition rating system to estimate the residual service life of deteriorated reinforced concrete structures. They opined that the rating system was able to appraise the present condition of the concrete structure based on the measured values of the concrete cover, the carbonation depth, and the chloride content at reinforcement depth.

Ortega and Robles (2014) conducted a study on the residual life of reinforced concrete structures under corrosion attack. They proposed a first natural frequency based non-destructive technique to predict the remaining life of the reinforced concrete beam that has different cracking level.

Choine et al., (2013) investigated a three-span integral concrete bridge to identify the influence of corrosion-induced deterioration on the seismic vulnerability of the bridge. The reinforcing steel in the bridge columns and the deck were subjected to chloride-induced corrosion for the three-dimensional non-finite element seismic fragility analysis. The result indicates that the bridge columns were highly sensitive to the system fragility.

Razak and Choi (2001) assessed the consequence of corrosion on the modal frequency and damping of reinforced concrete beam through measurement of the concrete crack width and spalling. The measurements were taken at time intervals to monitor the trend of the modal frequency and damping due to the concrete deterioration. It was shown that the modal frequency trend was sensitive to the decomposition state of the beam.

This paper describes the impact of corrosion-induced damage on the modal frequency of reinforced concrete specimen exposed to the natural climate environment in the tropical region, Johor in Malaysia. Malaysia is known for its warm, high humid, and low pressure environment. Furthermore, the influence of exposure duration on corrosion severity was determined. The study is essential for adequate characterisation of the influence of warm, high humid, and low pressure environment on modal frequency and corrosion induced damage.

1.1 Statement of the Problem

Corrosion of steel in concrete reduces the cross-sectional area and mass of the steel in concrete, and weakens the bond at the concrete-steel interface. The condition could lead to failure and collapse of concrete structures. Consequently, the effect of corrosion of reinforcing steel on the properties of concrete has been receiving enormous research attention in the recent times.

Although there are studies in the past on influence of corrosion-induced damage on the signatures of concrete specimen, most of the studies were conducted at controlled climatic conditions. The determination of the response of modal frequency of reinforced concrete to corrosion-induced damage in the uncontrolled climatic condition requires research attention.

1.3 Research Aim and Objectives

The aim of the research is to measure modal frequency response of reinforced concrete to corrosion-induced damage. Specifically, the research seeks to determine:

i. The effect of corrosion-induced damage on modal frequency of reinforced concrete exposed to the tropical rainforest environment.

ii. The influence of exposure duration on corrosion severity of reinforced concrete exposed to the tropical rainforest environment.

1.4 Research Questions

The study was guided by the following research questions:

i. What is the effect of corrosion-induced damage on modal frequency of reinforced concrete exposed to the tropical rainforest environment?

ii. How does exposure duration influence corrosion severity of reinforced concrete exposed to the tropical rainforest environment?

2.0 Materials and Methods

The materials and methods were discussed under the following subheading, namely; specimen preparation, specimen exposure, and data mining.

2.1 Specimen Preparation

Two sets of concrete specimen with water to cement ratio of 0.6 were prepared. Set 'A' has no sodium chloride (NaCl), while set 'B' concrete has 1 % sodium chloride. Nine numbers of 100 x 160 x 600 mm^3 of prism and thirty-three numbers of 200 mm long x 100 mm diameter cylinder were made from set 'A'; while twelve numbers of 100 x 160 x 600 mm^3 of prism and thirty-nine numbers of 200 mm long x 100 mm diameter cylinder were made from set 'B'. They were cast into moulds and demoulded after 24 hours. Three number specimen of the prism and twelve number specimen of the concrete cylinder made from set 'A' were cured in clean water to serve as control. Other concrete specimens were cured in 3.5% NaCl water solution for 28 days before exposure to the environment. The materials for the concrete were itemised and presented in Table 1.

Table 1. Quantity of materials for concrete (per I m^3) production

Material	Quantity (Kgm^{-3})
Cement	350
River sand	717.6
Crushed granite	1122.4
Water	210
Sodium chloride	3.5

2.2 Exposure Condition

The control specimens were kept in the laboratory at a controlled temperature and relative humidity of 23 ±2 °C and 55 ±5 % respectively. The specimens from set 'A' were of two groups, A1 and A2. The A1 specimen was immersed in 3.5 % NaCl water throughout the monitoring period, while the A2 specimen passed through 7 days wet and dry cycle between the NaCl solution and the natural unsheltered environment.

The specimens from set ‘B’ were four groups, namely; B1, B2, B3, and B4. Specimen B1 was immersed in 3.5% NaCl water throughout the monitoring period. Specimen B2 was subjected to 7 days wet and dry alternate process and in a 3.5% NaCl solution. Specimen B3 was exposed to the unsheltered natural environment. While specimen B4 was exposed to the sheltered natural environment.

2.3 Data Mining and Analysis

The modal frequency response of the simply supported concrete prism was mined through the DEWESoft instruments. The concrete prism has supports at points A and E. The modal hammer was used to excite the prism at designated points (A, B, C, & E), while the acceleration sensor was mounted at point D. Figure 1 shows the modal test setup.

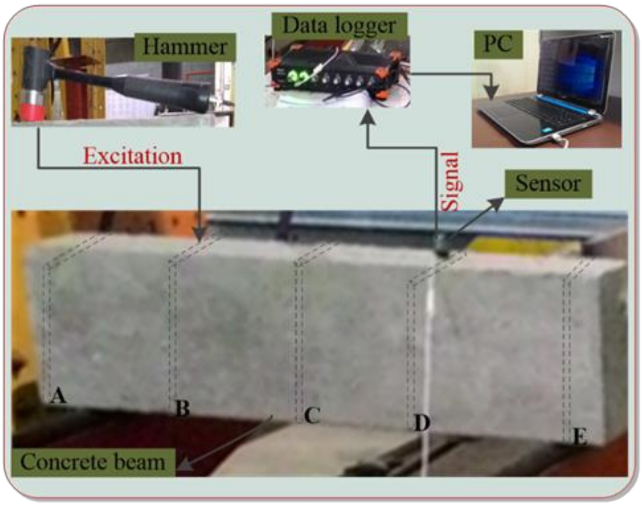

Figure 1. Modal test set-up

Also, the elastic modulus of the concrete cylinder, which is a parameter of modal frequency was deduced using the test setup shown in Figure 2. The data were used to calculate corrosion severity C_s (%) using Eq. 1, thus

$$C_s = \frac{E_o - E_c}{E_o} \times 100 \qquad (1)$$

where, E_o is the elastic modulus of intact concrete, and E_c is the elastic modulus of corroded concrete.

Figure 2. E-modulus test set-up

3.0 Results and Discussions

The first modal frequency response as a function of time is shown in Figure 3. It indicates that it took about four months for the modal frequency parameter to show a significant response to the corrosion-induced damage under a corrosive environment.

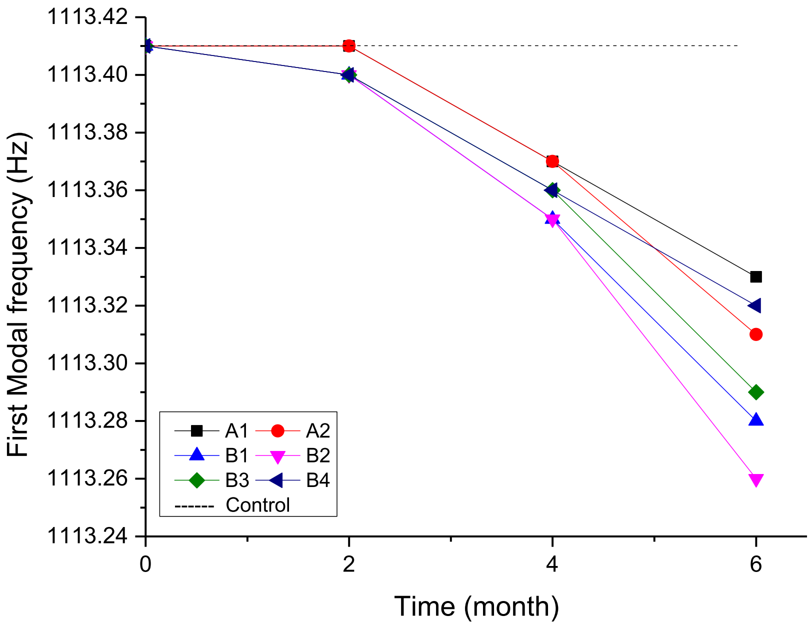

Figure 3: Modal frequency as a function of time

The analysis of the result revealed that the 7 days wet and dry alternate process (B2 exposure condition) has the highest effect on the first modal frequency for the concrete mixed with sodium chloride solution. This was followed by the B1 exposure condition, the B3 exposure condition and the B4 exposure condition. The result also showed that concentration of corrosive agent such as sodium chloride, as well as exposure condition affects modal frequency, as evidenced by the higher reduction rate recorded by the A2 specimen against the B4 specimen after the 5th month of exposure duration. Furthermore, the first modal frequency response trend line for the specimen B3, which was exposed to the unsheltered natural climate environment, deviated sharply from that of specimen B4 (exposed to a sheltered natural climate environment) after the fourth month of exposure. It implies that the fluctuating condition of the unsheltered natural climate of the region aids reduction of modal frequency of reinforced concrete. The findings of the study indicate that corrosion affects modal frequency of reinforced concrete especially in a friendly corrosive environment like the tropical rainforest. It agrees with the submission of Ismail and Egba (2017) that corrosion and climate affect behaviour of concrete.

The surveillance of the corrosion severity as a function of time is presented in Figure 4. It reveals that corrosion severity increases with exposure duration. However, exposure condition influences the degree of the corrosion severity increase. The result indicated that the B2, B1, B3 and A2 concrete specimens exhibited corrosion severity of 0.0069, 0.0058, 0.0054 and 0.0043 respectively, after six months of exposure duration. In addition, the B4 and A1 concrete specimens exhibited corrosion severity of 0.0042 and 0.0037 after six months of exposure duration.

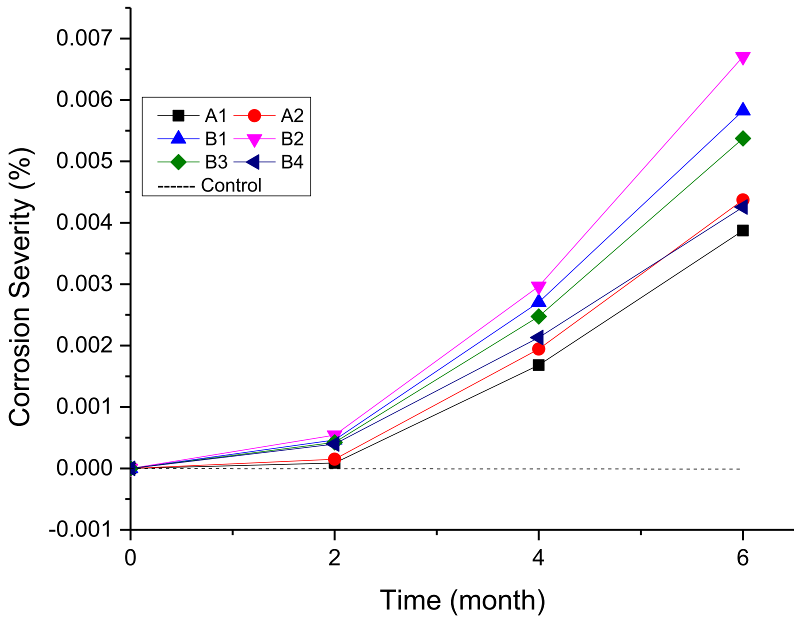

Figure 4: Corrosion severity as a function of time

The findings of the study indicate that elastic modulus, which is a function of corrosion severity is sensitive to corrosion-induced damage. The discovery is in line with the submission of Razak and Choi (2001) that modal frequency trend and elastic modulus are sensitive to the decomposition state of concrete beam.

4.0 Conclusion and Recommendations

The paper describes the impact of corrosion-induced damage on the modal frequency of reinforced concrete prism exposed to the natural climate environment in the tropical region, Johor Malaysia. The following submissions were made from the study namely:

i. Corrosion reduces modal frequency of concrete structures. It takes about four months for the modal frequency parameter to show a significant response to the corrosion-induced damage under corrosive environment of the tropical rainforest.

ii. Corrosion severity increases with exposure duration; however, exposure condition influences the degree of the corrosion severity increase.

iii. The natural climate of Johor, Malaysia promotes corrosion activity, as long as the concrete specimen is contaminated with corroding agents. The natural climate of the region exhibits similar behaviour of the wet and dry cycle process of accelerating corrosion, since its weather conditions fluctuate incessantly.

The following recommendations were made:

i. The response of other modal parameters like the mode shape and modal damping to corrosion-induced damage should be investigated to gather a comprehensive data of the influence of corrosion on dynamic signatures of concrete structures.

ii. A framework of modal parameter response to corrosion-induced damage to monitor service life of concrete structures should be developed.

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Effect of Rice Husk Ash -Based Geo -polymer on Some Geotechnical Properties of Lateritic Soil

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ABSTRACT

The effect of rice husk ash (RHA) based geo-polymer on lateritic soil was studied. The geo-polymer was activated using alkaline activators (NaOH and Na_2SiO_3) and applied in step concentrations of 0, 5, 10, 15, and 20% by dry weight of soil. Tests carried out include index properties, compaction (using British Standard light, BSL energy) and unconfined compressive strength (UCS). Statistical analysis of variance (ANOVA) was carried out using Microsoft Office Excel software. Test results indicated that the specific gravity of the natural soil decreased from 2.50 to a minimum value of 2.27 at 20% geo-polymer content. Also, a decline in the percentage of fines was observed with increase in geo-polymer content. Liquid limit initially increased from its natural value of 50% to a peak value of 54% at 15% geo-polymer content and thereafter decreased to 46% at 20% geo-polymer content. The plastic limit of the natural soil increased from 24.37% for the natural soil to 31.80% at 20% geo-polymer content. Plasticity index decreased with increase in geo-polymer content. Values of 25.63, 19.31 and 14.21% were recorded at 0, 10 and 20% geo-polymer content respectively. Maximum dry density (MDD) initially increased and thereafter decreased with higher geo-polymer content while Optimum moisture content (OMC) initially decreased and thereafter increased with higher geo-polymer content. OMC values of 17.8, 15.7, 15.5, 22.5, and 23% were recorded at 0, 5, 10, 15 and 20% geo-polymer content respectively. UCS values increased with increase in geo-polymer content. ANOVA analysis showed that geo-polymer has significant effect on the treated soil with exception of specific gravity and plasticity index results. Based on the result obtained, an optimal blend of 20% geo-polymer content improved the geotechnical properties of the soil and is recommended for geotechnical engineering application such as sub-base material for rural (light traffic) roads.

Key words: Geo-polymer; Lateritic soil; Rice husk ash; Compaction Characteristics; Unconfined compressive strength.

1. INTRODUCTION

City dwellings and rural settlements has experienced tremendous upsurge in population as well as industrialization over the years which has placed enormous pressure on existing natural resources including land. This fact has prompted several researches (Laxmikant *et al.,* 2011; Osinubi *et al*, 2015; Sani *et al*., 2018 and Kanyi *et al.,* 2017, 2019; Etim et al., 2017, 2020) into optimal utilization of these resources considering safety and economy for actualization of intended construction targets. The likelihood of erecting structures over soil with unfavourable conditions is inevitable due largely to the above reason. Researches (Basha *et al*., 2005; Alhassan, 2008; Moses *et al*., 2012; Osinubi *et al*., 2015; Rangan, 2014; Yohanna *et al*., 2016; Sani *et al*., 2018; Ishola *et al*., 2019; Ibrahim and Sa'eed, 2020; Sa'eed and Auustine, 2020) over the years however, have established more practical ways of improving the quality of soil over such areas to include utilization of different additives such as lime, cement and bitumen (which are considered effective traditional stabilizers, however, are usually very expensive) or agricultural and industrial waste materials having pozzolanic properties, geo-polymers as well as nano particles (Nano-silica) to improve the marginal properties of deficient soil for effective engineering applications.

According to Jeffrey *et al.* (2012), geo-polymer is a term first introduced by Davidovits in 1972, which is a product of geo-polymerization reaction involving alumina-silicate oxides (Si_2O_5, Al_2O_2) with alkali poly-silicates yielding polymeric Si–O–Al bonds. Geo-polymers find its application in various fields of human endeavour including construction industry due to its unique properties. Soe *et al.* (2018) reported that geo-polymer materials have been employed in the production of Formula 1 sports vehicles primarily due to its great resistance to fire and in production of concrete road such as runways at airports. Geo-polymers constitute mainly of alkaline liquids and source materials (Soe *et al.*, 2018).

Geo-polymers are made from source materials that are rich in silicon (Si) and aluminum (Al) (Rangan, 2014). These can be industrial by-product or agricultural waste materials such as fly ash, red mud, silica fume, rice-husk ash, etc. However, some pozzalanic materials may not be good for use as a stand-alone stabilizer because of their low calcium oxide content, hence the need for the introduction of alkaline activators to form geo-polymer. Rangan (2014) reported on the use of geo-polymer for engineering application in concrete work and have recorded positive results.

A lot of information on rice husk ash (RHA) ranging from its source, abundance, its environmental impacts etc is found in literatures (Musa and Muhammad, 2007; Alhassan, 2008; Nabanita *et al*, 2016). This study however, focuses on evaluation of the suitability of alkaline activated RHA-based geo-polymer for treatment of lateritic soil for use as road construction material.

2. MATERIALS AND METHODS

2.1. Materials

2.1.1 Soil: The disturbed soil sample used for this study was collected from Abattoir (9°52′50’’N and 8°53′20’’E) Jos, Plateau State Nigeria. The top soil was removed to a depth of 0.5m before the soil sample was collected in sacks and transported to the laboratory. The soil sample was then air–dried, pulverized and then sieved through several standard sieves for different types of tests with the largest sieve been BS No. 4 sieve (4.76 mm aperture).

2.1.2 Rice husk ash: The rice husk used for the soil improvement was sourced from Dadin-kowa village in Langtang South local government area of Plateau state, Nigeria. The Rice husk was collected in sacks; air dried and was burn completely in open heaps. The ash was then collected and sieved through 0.75 mm sieve.

2.1.3 Alkaline activators The Sodium Hydroxide (NaOH) and Sodium Silicate (Na_2SiO_3) were sourced from a chemical shop in Jos, Plateau State Nigeria. The NaOH and Na_2SiO_3 were added to prepared lateritic soil mixtures by the dry weight of the soil in the ratio of 1:1 as recommended by Gabriel (2018).

2.2. Methods

2.2.1 Index properties: index tests (Natural moisture content, Atterberg limits, specific gravity and particle size distribution) were carried out in accordance with BS 1377 (1990) and BS 1924 (1990) for natural and treated lateritic soil samples respectively. Soil samples were mixed with Geo-polymer at stepped concentrations of 0, 5, 10, 15 and 20% by dry weight of soil as recommended by Gabriel (2018).

2.2.2 Compaction: Test to determine the moisture – density relationships was carried out in accordance with BS 1377 (1990) Part 4 using the British Standard light (BSL) energy. The BSL compaction energy involves 3 layers each receiving 27 blows in a 1000 cm^3 compaction mould. The modified soil samples were gotten by mixing the soil with Rice Husk Ash-Geo-polymer at stepped concentrations of 0, 5, 10, 15 and 20% by dry weight of soil.

2.2.3 Unconfined compressive strength: The unconfined compressive strength (UCS) test was performed on the soil samples according to BS 1377(1990) using the British Standard light (BSL) energy level. The natural and the treated soil samples were compacted in 100cm^3 moulds at their respective OMCs. The compacted samples were sealed in a polythene bag and cure for 7 days. The samples were then placed centrally on the pedestal of the compression machine between the upper and lower platens. The machine was then adjusted so that contact is just made between the specimen, upper platen and the force measuring device. The axial deformation gauge was adjusted to read zero for a convenient initial reading, and the initial readings of the force recorded. The unconfined compressive strength was computed using equation 1.

$$\text{unconfined compressive strength (UCS)} = \frac{\text{Failure load}}{\text{Cross sectional area}} \qquad (1)$$

The same procedure was then repeated for RHA- based geo-polymer at stepped concentrations of 0, 5, 10, 15 and 20% by dry weight of soil.

3. RESULTS AND DISCUSSION

3.1. Physical Properties of the Soil Used

Preliminary investigations performed on the natural lateritic soil revealed that the reddish brown soil is fine-grained and has a natural moisture content of 12.3% with liquid limit (LL), plastic limit (PL) and plasticity index values of 50%, 24.4% and 25.6% respectively. The summary of the preliminary tests results is presented in Table 1 while Figure 1 showed the particle size distribution curve of the natural soil. The result showed that the soil contain high amount of fine fraction and is not well graded therefore required modification to meet construction requirements. On the basis of these properties, the soil is of low plasticity and below bar in terms of standard requirement for most engineered works especially road construction (Osinubi, 1998; Osinubi and Alhassan, 2008).

Table 1: Geotechnical properties of natural soil

Properties	Quantity
Natural moisture content (%)	12.30
Percentage passing BS No. 200 (%)	44.47
Liquid limit (%)	50
Plastic limit (%)	24.4
Plasticity index (%)	25.6
AASHTO classification	A-7-6(2)
USCS	CL
Specific gravity	2.50
Maximum dry density (Mg/m^3)	1.64

Optimum moisture content (%)	17.80
UCS (kN/m^2) 7 days	784
Color	Reddish brown

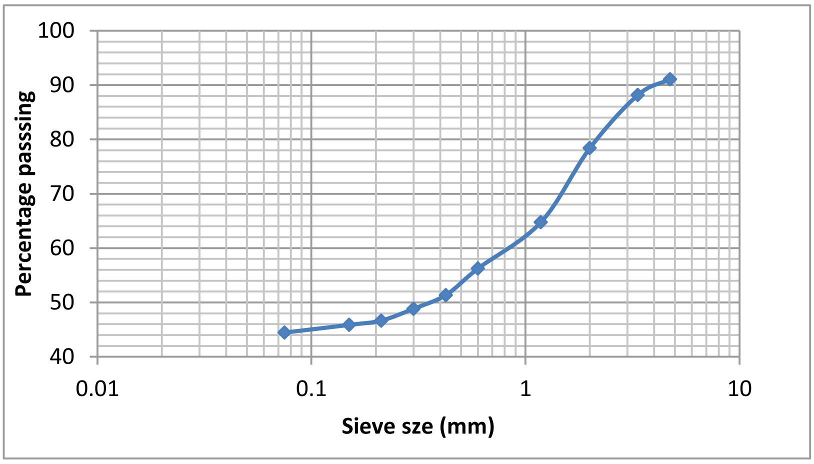

Figure 1: Particle size distribution curve of the natural soil

3.2. Specific gravity

The variation of specific gravity of lateritic soil treated with geo-polymer content is shown in Figure 2. It was observed that specific gravity of the lateritic soil generally decreased with higher geo-polymer content. The specific gravity of the natural soil decreased from its natural value of 2.50 to a minimum value of 2.27 at 0 to 20% geo-polymer content. The decrease could be attributed to low specific gravity of the additives replacing the soil particles with higher specific gravity thereby reducing the overall specific gravity of the resulting soil matrix. Similar behaviour was observed by Amadi, (2010).

Statistical examination of the tests result using One – way analysis of variance (ANOVA) for specific gravity is shown in Table 2. The result shows that the effects of geo-polymer on lateritic soil were not statistically significant ($F_{CAL} = 4.632 < F_{CRIT} = 5.318$).

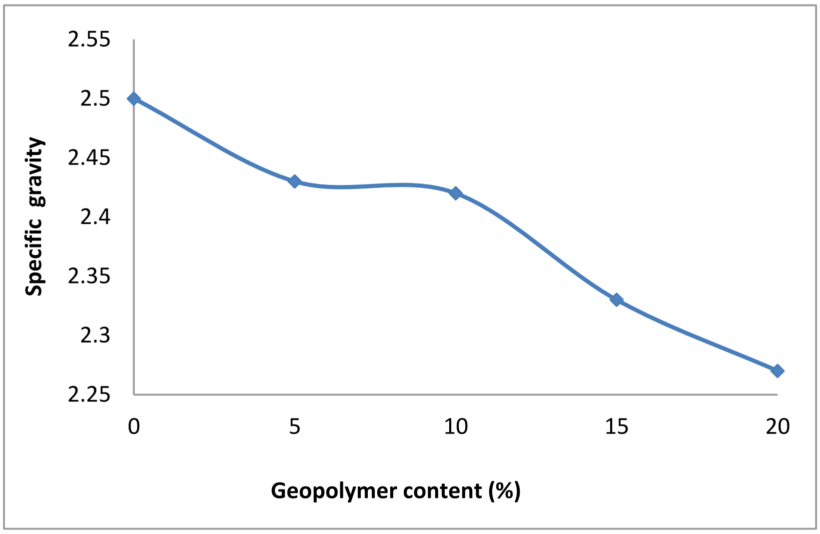

Figure 2: Variation of specific gravity of lateritic soil – Geo-polymer mixtures.

Table 2: One - way Analysis of Variance for Specific Gravity of Lateritic Soil - Geo-polymer Mixtures

Property	Source of variation	Degree of Freedom	F_{CAL}	P-Value	F_{CRIT}	Remark
Specific Gravity	Geo-polymer	1	4.632	0.063548	5.318	NS

NS=No significant Effect

3.3. Particle size distribution

The particle size distribution curve for soil treated with various percentages of geo-polymer is shown in Figure 3. It can be inferred that with increase in geo-polymer content (RHA, NaOH and $NaSiO_3$ mixtures), the particle size distribution curves shifted to the left at 5% geo-polymer, indicating increase in the percentage of fine. It then shifted to the right at 10, 15 and 20% respectively, indicating decrease in the percentage of fines. Generally, a decline in the percentage of fines was recorded with increase in geo-polymer content. Values of 44.47, 44.59, 38.77, 27.23 and 35.74% were recorded at 0, 10, 15 and 20% geo-polymer content respectively. Similar behaviour was reported by Phummiphan *et al.*, 2014. A little change was noticed in the coarser sizes; this may be due to modification reaction between the geo-polymer and clay minerals which facilitated the formation of heavier pseudo, sands particles.

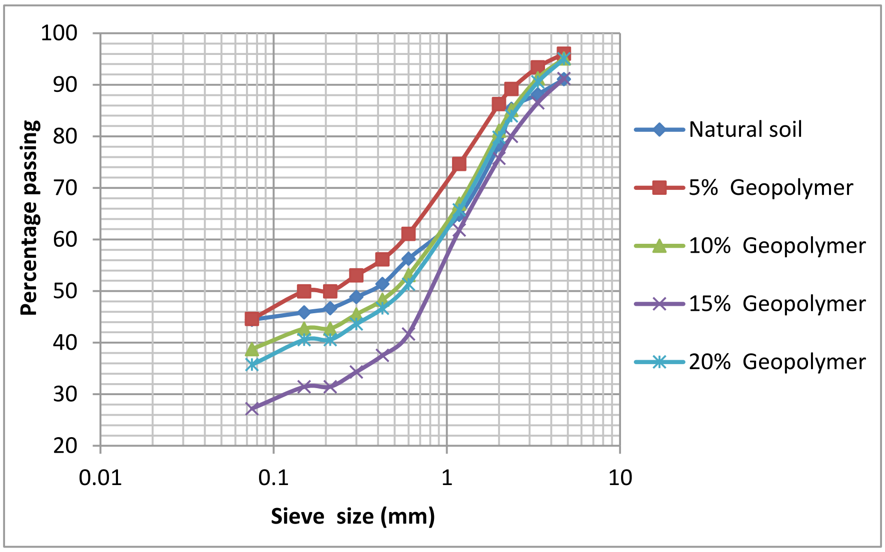

Figure 3: Variation of particle sizes of lateritic soil – Geo-polymer mixtures.

3.4. Atterberg limits

Liquid limit

The variation of liquid limit of lateritic soil - mixtures with geo-polymer content is shown in Figure 4. It was observed that the liquid limit initially increased from its natural value of 50% to a peak value of 54% at 15% geo-polymer content and thereafter decreased to 46% at 20% geo-polymer content. Similar behaviour of decrease was reported by Okafor and Okonkwo, 2009, Nabanita *et al.,* 2016 and Lestari *et al.,* 2019. The decrease may be due to flocculation and agglomeration arising from cation exchange reactions whereby Ca^{2+} in the additives reacted with ions of lower valence in the soil structure. The addition of RHA-geo-polymer introduced 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 (Nabanita *et al.,* 2016).

Statistical examination of the tests result using One – way analysis of variance (ANOVA) for liquid limit is shown in Table 3. Results shows that the effects of geo-polymer on lateritic soil were statistically significant ($F_{CAL} = 83.597 < F_{CRIT} = 5.318$).

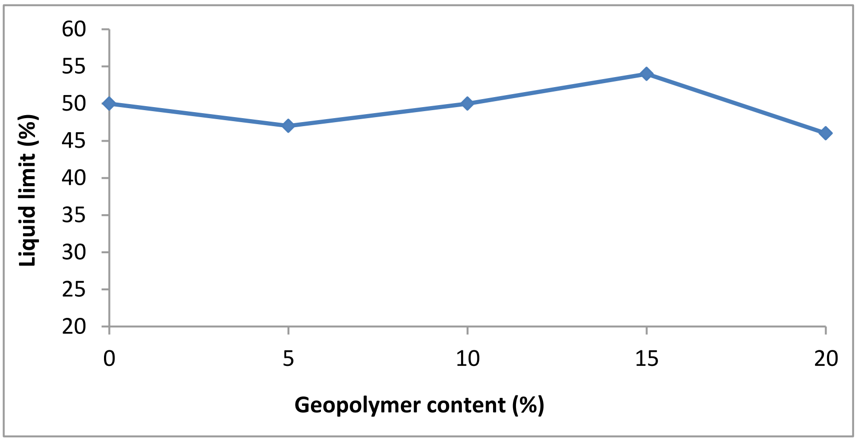

Figure 4: Variation of Liquid limit of lateritic soil – Geopolymer mixtures.

Table 3: One - way Analysis of Variance for Atterberg Limits of Lateritic Soil -Geo-polymer Mixtures

Property	Quantity measured	Source of variation	Degree of Freedom	F_{CAL}	P-Value	F_{CRIT}	Remark
Atterberg limits	Liquid Limit	Geo-polymer	1	83.597	1.65E-05	5.318	SS
	Plastic Limit	Geo-polymer	1	35.876	0.000327	5.318	SS
	Plasticity Index	Geo-polymer	1	0.593	0.46355	5.318	NS

SS= significant Effect; NS= No significant Effect

Plastic limit

The variation of plastic limit of lateritic soil – geo-polymers mixtures is shown in Figure 5. The plastic limit of the natural soil increased from 24.37% for the natural soil to 31.8% at 20% geo-polymer content. The observed trend may be due to cation exchange reaction that liberated adsorbed water particles in the soil leading to the flocculation and aggregation of the soil (Salahudeen and Sadeeq, 2016). The results recorded are in agreement with the findings reported by Amadi, (2010) and Lestari *et al.*, 2019

Statistical examination of the tests result using One – way analysis of variance (ANOVA) for plastic limit is shown in Table 3. Results shows that the effects of geopolymer on lateritic soil were statistically significant (F_{CAL} = 35.876>F_{CRIT} = 5.318).

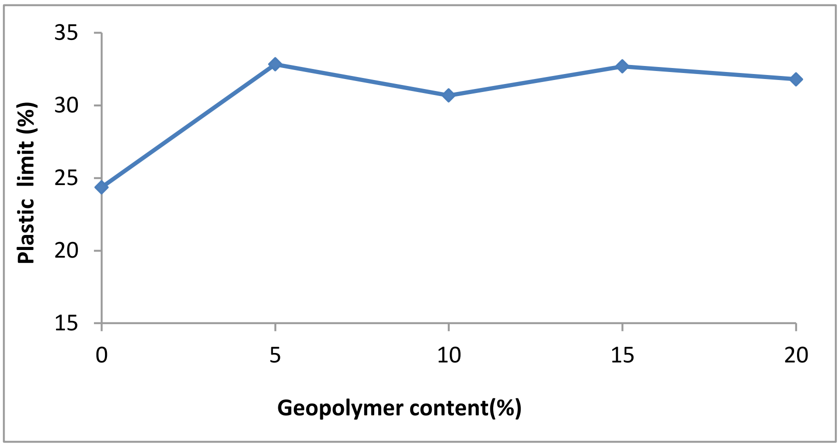

Figure 5: Variation of plastic limit of lateritic soil – Geopolymer mixtures.

Plasticity index

The result of the plasticity index test on lateritic soil treated with geo-polymer is shown in the Figure 6. From the results, it can be observed that the plasticity index decreased with increase in geo-polymer content relative to the natural value of 25.63%. Values of 14.18, 19.31, 21.31 and 14.21% were recorded at 5, 10, 15, and 20% geo-polymer content respectively. This trend is not in agreement with report of Salahudeen and Sadeeq (2016).

Statistical examination of the tests result using One – way analysis of variance (ANOVA) for plasticity index is shown in Table 3. Results shows that the effects of geo-polymer on lateritic soil were statistically not significant ($F_{CAL} = 0.596 < F_{CRIT} = 5.318$).

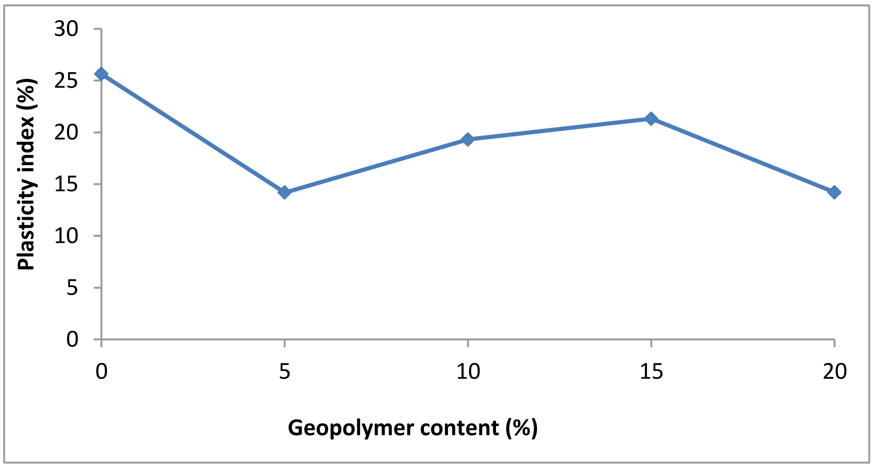

Figure 6: Variation of plasticity index of lateritic soil – Geopolymer mixtures.

3.5. Compaction Characteristics

Effect of additive on maximum dry density

The effect of activated RHA (geo-polymer) on the maximum dry density (MDD) of lateritic soil mixture is shown in Figure 7. It was observed that MDD values initially increased from 1.64 for the natural soil to a peak value of 1.76 Mg/m^3 at 5% geo-polymer content and thereafter decreased with higher geo-polymer content. MDD values of 1.75, 1.69, 1.62 Mg/m^3 were recorded at 10, 15 and 20% geo-polymer content respectively. Similar trend of increase in MDD values were reported by Mohamed and Nagaratnam (2018), Soe *et al.,* 2018 as well as Hanifi *et al*., 2019. The initial increase in MDD could be due to geo-polymer that occupied the void within the soil matrix and in addition, the flocculation and agglomeration of the clay particle due to exchange of ions. This is in agreement with the findings reported by Amadi (2010) as well as Mohamed and Nagaratnam (2018).

Statistical examination of the tests result using One – way analysis of variance (ANOVA) for maximum dry density is shown in Table 4. Results show that the effects of geo-polymer on lateritic soil were statistically significant (FCAL = 5.52619> FCRIT = 5.318).

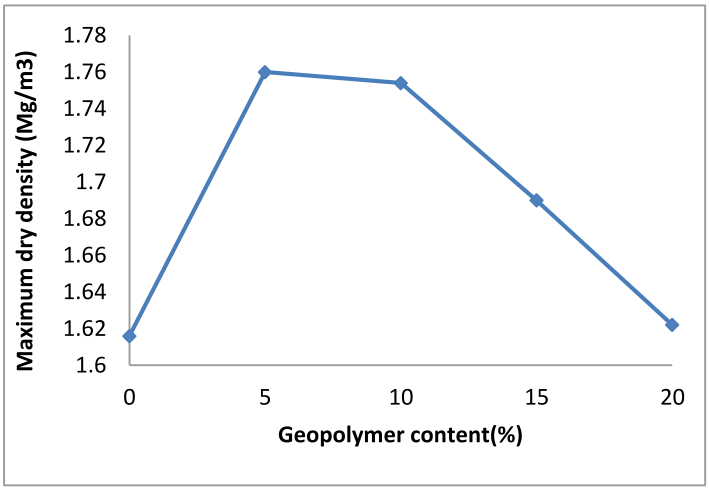

Figure 7: Variation of maximum dry density of lateritic soil – Geo-polymer mixtures.

Table 4: One - way Analysis of Variance for Compaction Characteristics of Lateritic Soil - Geo-polymer Mixtures

Property	Source of Variation	Degree of Freedom	F_{CAL}	P-Value	F_{CRIT}	Remark
Maximum dry Density	Geo-polymer	1	5.526	0.046624	5.318	SS
Optimum moisture content	Geo-polymer	1	6.167	0.037911	5.318	SS

SS= significant Effect

Effect of additive on optimum moisture content

The variation of optimum moisture content (OMC) of lateritic soil mixture with geo-polymer content is shown in Figure8. It was observed that the OMC initially decreased from to a minimum value of 15.5% at 10% geo-polymer content and thereafter increased with higher geo-polymer content. OMC values of 22.5, and 23% were recorded at 15 and 20% geo-polymer content respectively. The initial decrease could be due to self - desiccation of the mixture during which all the water was used, resulting in low hydration. When no water movement to or from soil- Geo-polymer matrix is permitted, the water is used up in the hydration until too little is left to saturate the solid surfaces and hence the relative humidity within the paste decreases (Osinubi, 2001; Moses *et al.,* 2012; Salahudeen and Sadeeq, 2016).

Statistical examination of the tests result using one – way analysis of variance (ANOVA) for optimum moisture content is shown in Table 4. Results show that the effects of geo-polymer on lateritic soil were statistically significant (FCAL = 6.167> FCRIT = 5.318).

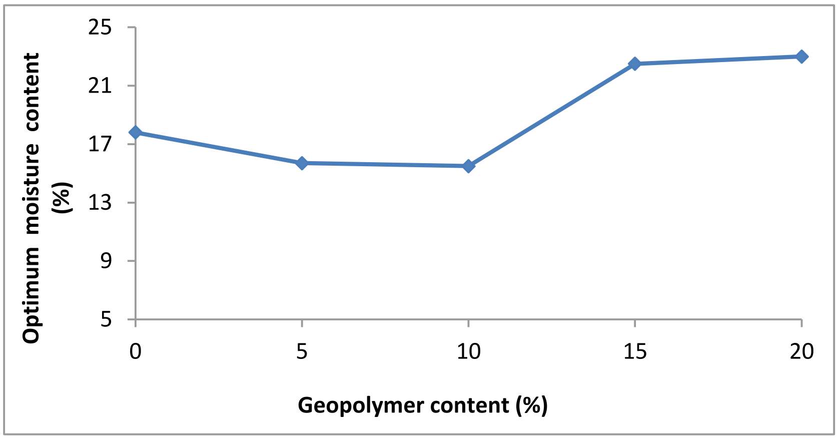

Figure 8: Variation of optimum moisture content of lateritic soil – Geo-polymer mixtures.

3.6. Unconfined Compressive Strength

The unconfined compressive strength (UCS) test is used in determining the amount of additive to be used in the stabilization of soil (Singh, 1991). It forms the basis for the evaluation of the design criteria for the use of soil as a pavement material (Ola, 1983). The variation of UCS of soil-geo-polymer mixtures cured for 7 days with BSL compaction effort is shown in Figure 9. It was observed that the UCS values increased with increase in geo-polymer content. Values of 785, 891, 1353, 1824 and 2105kN/m2 were recorded at 0, 5, 10, 15 and 20% geo-polymer content respectively. The increase could be attributed to the formation of secondary cementation compounds caused by availability of sufficient water that helped in the hydration reaction of RHA and the alkaline activator. Similar result was reported by Sani *et al.* (2018).

Statistical examination of the tests result using One – way analysis of variance (ANOVA) for unconfined compressive strength (UCS) test is shown in Table 5. Results show that the effects of geo-polymer on lateritic soil were statistically significant (F_{CAL} = 28.969< F_{CRIT} = 5.318).

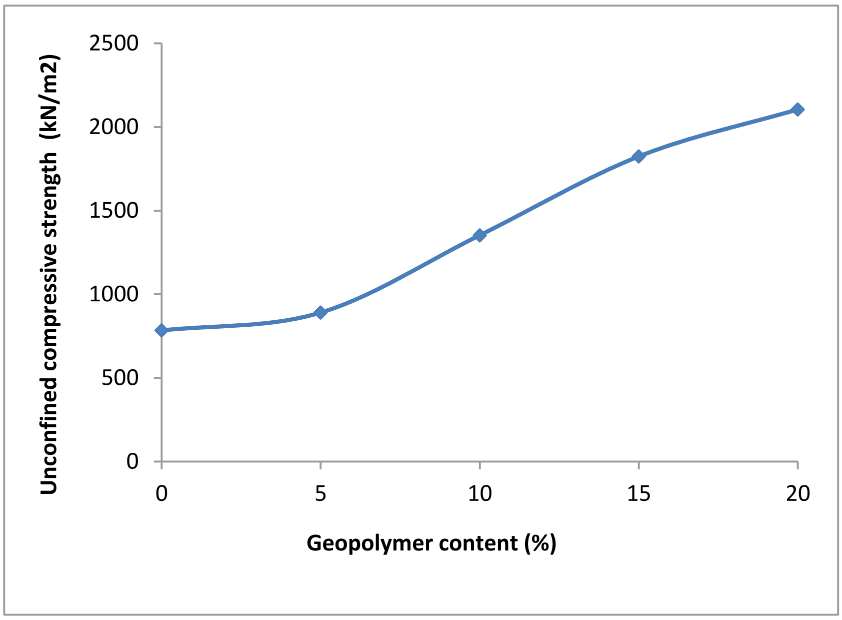

Figure 9: Variation of UCS of soil- geo-polymer mixture cured for 7 days

Table 5: One - way Analysis of Variance for unconfined compressive strength (UCS) of Lateritic Soil – Geo-polymer Mixtures

Property	Source of Variation	Degree of Freedom	Fcal	P-Value	Fcrit	Remark
Unconfined compressive strength	Geo-polymer	1	28.961	0.00066	5.318	SS

SS= significant Effect

4 CONCLUSION

Based on the results obtained, the following conclusions are drawn.

1. The specific gravity of the natural soil decreased from its natural value of 2.50 to a minimum value of 2.27 at 20% geo-polymer content, while a decline in the percentage of fines was recorded with increase in geo-polymer content.

2. Liquid limit initially increased from its natural value of 50% to a peak value of 54% at 15% geo-polymer content and thereafter decreased to 46% at 20% geo-polymer content. The plastic limit of the natural soil increased from 24.37% to peak value of 31.8% at 20% geo-polymer content while plasticity index generally decreased with increase in geo-polymer content.

1. MDD values initially increased for the natural soil to a peak value of 1.76 Mg/m^3 at 5% geo-polymer content and thereafter decreased with higher geo-polymer content. **MDD values of 1.64, 1.76, 1.75, 1.69, 1.62 Mg/m^3 were recorded at 0, 5, 10, 15 and 20% geo-polymer content respectively while** OMC initially decreased and thereafter increased with higher geo-polymer content.

2. **The UCS values increased with increase in geo-polymer content. Values of 785, 891, 1353, 1824 and 2105kN/m^2 were recorded at 0, 5, 10, 15 and 20% geo-polymer content respectively.**

3. Based on the study carried out, an optimal blend of 20%geoplymer content improved the geotechnical properties of the soil and is recommended for geotechnical engineering application such as sub-base material for rural roads.

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PREDICTING INVERTED ELECTRICAL RESISTIVITY FROM GEOTEC HNICAL ENGINEERING PARAMETERS

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ABSTRACT

Predictive Inverted Electrical Resistivity values were obtained from geotechnical engineering parameters using regression analysis tool. Key basic engineering parameters were also utilized against Resistivity. A total of 76 samples were obtained within FCT, Abuja and Electrical Resistivity was carried out at the corresponding locations through which the inverted electrical resistivities were determine and used in determining the model. In addition, Terrameter SAS 300C was used to carry out the measurement in the field. The models were developed from the data obtained from the field and laboratory results. Generally, the results show that the coefficient of determination ranges from no correlation to mild correlations. It can be seen that cohesion has the highest R^2 value of 0.40 which is considered to have a mild correlation with the model as Inv. R = $3E\text{-}07C^4 - 3E\text{-}05C^3 + 0.0009C^2 - 0.0146C + 0.1385$. It can be recommended that this model can be use to predict inverted electrical resistivity.

Keywords: Electrical Resistivity, Engineering parameters, Inverted Electrical Resistivity, Coefficient of Correlation.

1.0 INTRODUCTION

The study of the structure and composition of the earth's interior is called Geophysics. An applied branch of geophysics, which employed the physical methods (such as electrical, seismic, magnetic, electromagnetic and gravitational) on the Earth surface to measure the physical properties of the subsurface, along with the anomalies in those properties is said to be Exploration Geophysics (Mohd*et al.,* 2012). It is mostly used to detect the availability and position of economically useful mineral depositions; examples of such are ore minerals; fossil fuels and other hydrocarbons; geothermal reservoirs; and groundwater reservoirs. Typically, geophysical site investigation techniques can give a voluminous value and produce image of the subsurface without disturbing the subsoil physically. (Mohd*et al.,*2012).

Electrical Resistivity, Ground Penetrating Radar and Magnetism are examples of geophysical techniques that are convenient and use specific imaging equipment. Electrical geophysical is a non-destructive test, less time consuming and a low-cost method that help to know the resistivity of a soil (Hatta and Syed Osman, 2015). Geophysics enables engineers to predict the subsoil layer properties based on the soil resistivity.

A surface geophysical technique provides an alternative wide-area method for subsurface characterization and information regarding relevant material properties (Pazzi et al., 2018). Ground penetrating radar, electrical resistivity, and magnetism are examples of Geophysical Investigation Techniques which are convenient and are used in specific imaging equipment. The relationship between mechanical properties of sedimentary rocks and P-wave velocity were studied by (Altindag, 2012). Simple regression analysis and an already acquired data are used by Altindag 2012. The already acquired data were then subjected to multi-regression analysis. A lot of empirical equations that has high correlation coefficients were also derived, which are useful for rock engineering practitioners (Soupios*et al.,* 2005).

A little study has been carried out to correlate electrical resistivity of subsurface and geotechnical parameters of soil. Cosenza *et al.* (2006) established a qualitative and quantitative correlation between resistivity and CPT values, when 2D electrical resistivity surveys with Wenner electrode configuration were conducted. Sudha *et al.* (2009) uses 2D electrical resistivity at two different locations in India to investigate the relationship between electrical resistivity and SPT values. Also, investigation was carried out by Liu *et al.* (2008) on the electrical resistivity of soil-cement admixture, at different cement-mixing ratio, water content and curing time. Correlation of SPT and compressive strength shows a good result of soil-cement admixture with electrical resistivity. Oh and Sun (2008) concluded that electrical resistivity of soil has a good correlation with SPT values after a combined analysis of electrical resistivity and SPT for the assessment of earth filled dam were used. Geotechnical characterization through geotechnical and electrical data in India were carried out by Gautam and Sastry (2007). Correlation between water content and electrical resistivity of sand in Bhopal, India were also worked on by Bhatt and Jain (2014). Bery and Saad (2012) characterized the geotechnical site using the electrical resistivity method of approach in Malaysia. The main aim of this study is the correlation of Inverted Electrical Resistivity values with the geotechnical engineering parameters.

2.0 METHODOLOGY

2.1 Description of the Study Area

Soil samples were collected from about 76 different locations. The sites were located and data was collected at different parts of the Federal Capital Territory, Abuja depending on the availability of the data. Federal Capital Territory, Abuja is located on a latitude 9.072264 and longitude 7.491302

2.2 Experimental Procedure

2.21 Natural Moisture Content

The natural moisture content of the soil was determined according to BS 1377 (1990) Test 1 (A). An appropriate weight of the wet soil was placed in a container of known mass. The soil plus the container was weighed to the nearest 0.01g, after which they were kept in an oven and allowed to dry at a temperature of 105 to 110 ºC for a period of 24 hours. Immediately after drying, the container plus the dry soil was weighed again. The moisture content was then calculated using the relationship.

$$W = \frac{M_2 - M_1}{M_3 - M_1} \times 100 \qquad (1)$$

W - Moisture content (%)

M_1 - mass of container (g)

M_2 - mass of container + wet soil (g)

M_3 - mass of container + dry soil

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

2.22 Specific Gravity

The specific gravity test was conducted for the natural soil in accordance with Test 6 (B) BS 1377 (1990). About 25 g of soil sample passing 2 mm BS sieve size was oven dried at 105 ºC –110 ºC. The sample was allowed to cool in a desiccator and then placed in 50 ml density bottle with the stopper attached. The weight of the soil, bottle and stopper were measured. 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. The bottle was emptied and filled with distilled water alone, again the weight of the bottle, stopper and water were recorded. The entire procedure was repeated three times using 25 g of sample each time in order to obtain an average value for the specific gravity.

The specific gravity of soil particles is calculated using the expression:

$$G_S = \frac{W_2 - W_1}{(W_4 - W_1) - (W_3 - W_1)} \qquad (2)$$

Where G_s - Specific Gravity

W_1 - mass of density bottle, stopper (g)

W_2 - mass of density bottle, soil and stopper (g)

W_3 - mass of density bottle, stopper and distilled water and soil sample (g)

W_4 - mass of density bottle, stopper and distilled water only (g)

2.23 Atterberg Limits

The liquid limit, plastic limit, plasticity index and linear shrinkage tests were carried out according to BS 1377 (1990) Test 1 (A) for the natural soil samples.

2.24 Liquid Limit (LL)

The sample for this test was dried sufficiently and broken up; 200 g of the material passing through the 425µm BS test sieve was obtained. The sample was then placed on a clean glass plate, using the palette knife and the spatula the soil was thoroughly mixed with water until a homogenous paste was obtained. The paste was wrapped in an airtight cellophane bag for not less than 24 hours in order to allow moisture to penetrate through the sample. The paste was then placed on the glass plate and mixed thoroughly for some few minutes. Some of the remixed paste was placed in the brass cup of the Casagrande apparatus. After leveling the sample, a standard grooving tool was drawn through the paste along a diameter through the pivot.

The crank was turned to lift drop the cup at two revolutions per second. The number of blows that is required to close the groove for 13 mm distance was recorded after which about 10 g of the paste was immediately taken from the cup for the moisture content determination. These procedures were repeated by adding more water to the paste. Thus the moisture content for each test was increased, the soil flowed more easily and the number of blows for closure of the groove decreased. Four tests were conducted on each batch of the sample for number of blows ranging between 50 and 10.

A graph of moisture content versus number of blows was plotted to a logarithmic scale. The liquid limit was taken at the moisture content corresponding to 25 number blows.

2.25 Plastic Limi t (PL)

About 20 g of dried soil passing 425µm BS test sieve was placed on a flat glass plate and mixed thoroughly with water until the mixture was plastic enough and be shaped into a small ball. The paste was rolled between the finger and the thumb into 6 mm diameter threads, and then rolled between the fingertips and the glass plate with sufficient pressure to reduce threads to 3 mm diameter. The procedure was repeated severally until the thread crumbled. Immediately, the moisture content of the crumbled threads was determined. The procedure was repeated two to three times and the moisture content determined from the mean values of the tests.

2.26 Plasticity Index (PI)

The plasticity index was determined from liquid limit and plastic limit results for both the natural and stabilized soil using the expression:

$$PI = LL - PL \qquad 3$$

2.27 Particle Size Distribution (Wet Sieving).

The particle size distribution was conducted in accordance with BS 1377(1990) Part 2. A 200g of the soil was weighed, wet sieved to remove clay and silt particles using BS No. 200 (0.075 mm) sieve under tap water. Washing was done carefully to avoid damage to the sieves. After washing, the sample was dried in an oven set to 105^{0}C for 24 hours. After drying, the standard BS sieves were arranged in descending order of sieve size. The oven-dried sample was transferred into the top most sieves and shaken for at least 10 minutes on a mechanical sieve shaker. After sieving, the mass retained on each sieve was then weighed. The percentages passing each sieve size was calculated and plotted on a semi-log graph of percentage passing against sieve sizes.

2.28 Electrical Resistivity

The geophysical investigation involved the resistivity method using vertical electrical sounding technique. The principle of the resistivity method is that an electric current is passed into the ground with the aid of two electrodes (anode and cathode), and the resultant potential difference is measured with another set of two electrodes. The ratio of the potential difference to the current is displayed by the Terrameter as electrical resistance. The electrodes are then arranged in a straight line, about a center point symmetrically. A geometric factor in meters is calculated as a function of the electrode spacing. The resistance reading obtained by the Terrameter is multiplied by this to give an apparent resistivity value. The electrode spacing is increased progressively, keeping the center point of the electrode array at a fixed location.

Terrameter SAS 300C was used to carry out the resistivity measurements in the field.

The full Schlumberger configuration was adopted for the soundings. Maximum half-current electrode spread, (AB/2) was between 1 and 50 while the half-potential electrode separation (MN/2) was maintained between 0.3 and 5m. The separation between adjacent sounding centers varies from 180-200m, while the traverse lines were spaced usually at 1km intervals but sometimes according to accessibility.

4.0 RESULTS AND DISCUSSION

4.1 Introduction

The results of field tests conducted on the selected locations as apparent resistivity was converted to Inverted Resistivity were analyzed and correlations of Inverted Resistivity with geotechnical engineering parameters were also presented in accordance with the correlation coefficient of determination R^2. According to the values of R^2, the relationship between any two parameters are classified as ($R^2<0.30$) are considered to have no correlation, (R^2 of 0.30 - 0.499) are considered to be a mild relationship, (R^2 of 0.50 - 0.699) are considered to be a moderate relationship and, (R^2 of 0.70 - 1.0) are considered to be a strong relationship(Taylor, 1990).

4.2 Correlation of Inverted Resistivity with Geotechnical Engineering Parameters

4.2.1 Correlation between Inver ted Resistivity and Natural Moisture Content

The regression trend of Inverted Resistivity and Natural Moisture Content (EMC) is as indicated in Figure 1. The result presents a weak or no correlation between the Inverted Resistivity and Natural Moisture Content (EMC) with the coefficient of determination R^2 = 0.2358 is as shown in Figure 1.

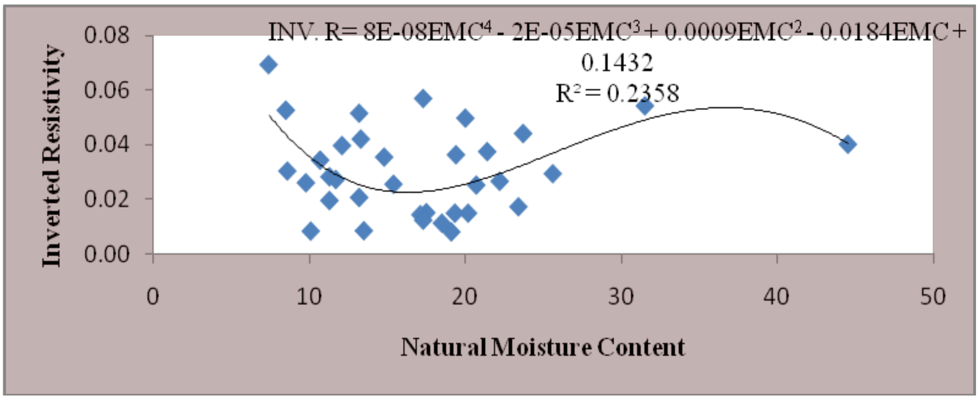

Figure 1: Correlation between Inverted Resistivity and Natural Moisture Content

4.2.2 Correlation between Inverted Resistivity and Liquid Limit

The regression trend of Inverted Resistivity and Liquid Limit (LL) is as shown in Figure 2. The result presents a mild correlation between the Inverted Resistivity and Liquid Limit (LL) with the coefficient of determination R^2 = 0.349 is as shown in Figure 2.

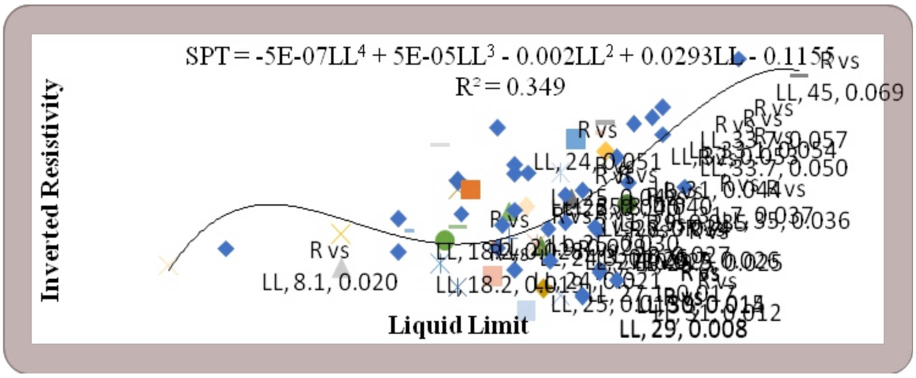

Figure 2: Correlation between Inverted Resistivity and Liquid Limit

4.2.3 Correlation between Inverted Resistivity and Plastic Limit

The regression trend of Inverted Resistivity and Plastic Limit (PL) is as shown in Figure 3. The result presents a weak or no correlation between the Inverted Resistivity and Plastic Limit (PL) with the coefficient of determination R^2 = 0.1564 is as shown in Figure 3.

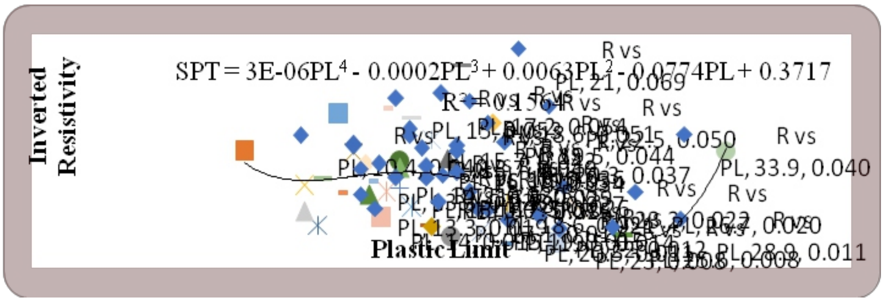

Figure 3: Correlation between Resistivity and Plastic Limit

4.2.4 Correlation between Inverted Resistivity and Plasticity Index

The regression trend of Inverted Resistivity and Plasticity Index (PI) is as shown in Figure 4. The result presents a mild correlation between the Inverted Resistivity and Plasticity Index (PI) with the coefficient of determination $R^2 = 0.3344$ is as shown in Figure 4.

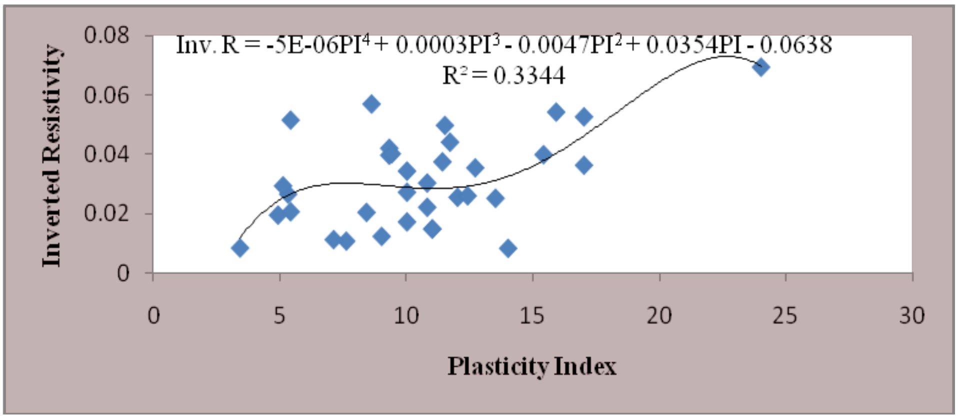

Figure 4: Correlation between Inverted Resistivity and Plasticity Index

4.2.5 Correlation between Inverted Resistivity and Sieve No. 200

The regression trend of Inverted Resistivity and Sieve No. 200 is as indicated in Figure 5. The result presents a weak or no correlation between the Inverted Resistivity and Sieve No. 200 with the coefficient of determination $R^2 = 0.1317$ is as shown in Figure 5.

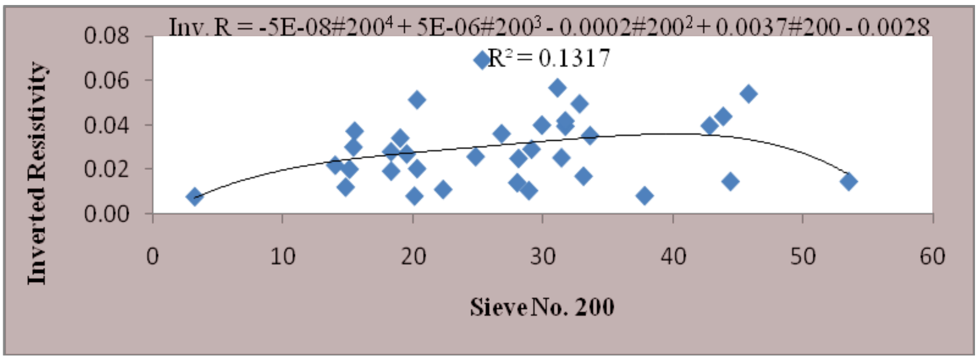

Figure 5: Correlation between Inverted Resistivity and Sieve No. 200

4.2.6 Correl ation between Inverted Resistivity and Cohesion

The regression trend of Inverted Resistivity and Cohesion (C) is as shown in Figure 6. The result presents a mild correlation between the Inverted Resistivity and Cohesion (C) with the coefficient of determination $R^2 = 0.4004$ is as shown in Figure 6.

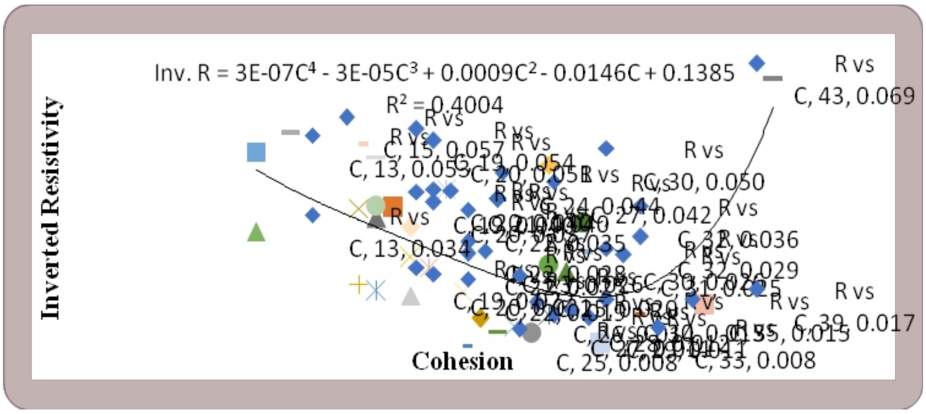

Figure 6: Correlation between Inverted Resistivity and Cohesion

4.2.7 Correlation between Inverted Resistivity and Angle of Internal Friction

The regression trend of Inverted Resistivity and Angle of Internal Resistance (**Φ°)** is as shown in Figure 7. The result presents a weak or no correlation between the Inverted Resistivity and Angle of Internal Friction (**Φ°)** with the coefficient of determination $R^2 = 0.110$ is as shown in Figure 7.

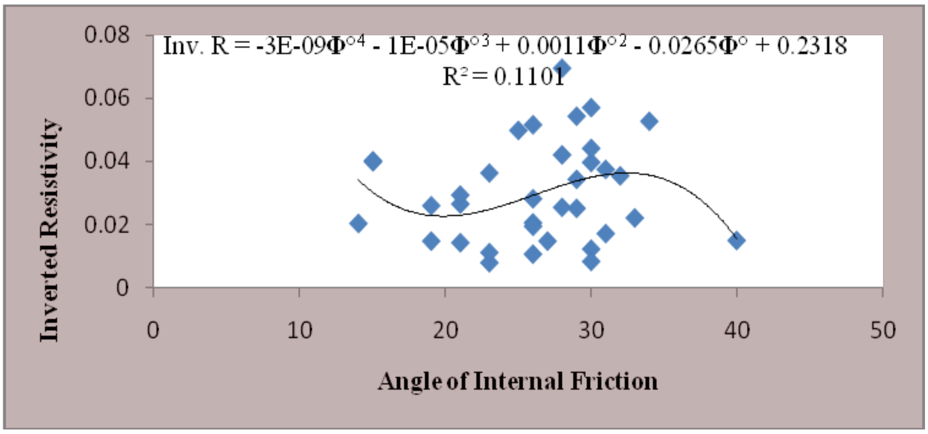

Figure 7: Correlation between Inverted Resistivity and Angle of Internal Friction

4.2.8 Correlation between Inverted Resistivity and Coefficient of Consolidation

The regression trend of Inverted Resistivity and Coefficient of Consolidation (C_V) is as shown in Figure 8. The result presents a weak or no correlation between the Inverted Resistivity and Coefficient of Consolidation (C_V) with the coefficient of determination $R^2 = 0.1455$ is as shown in Figure 8.

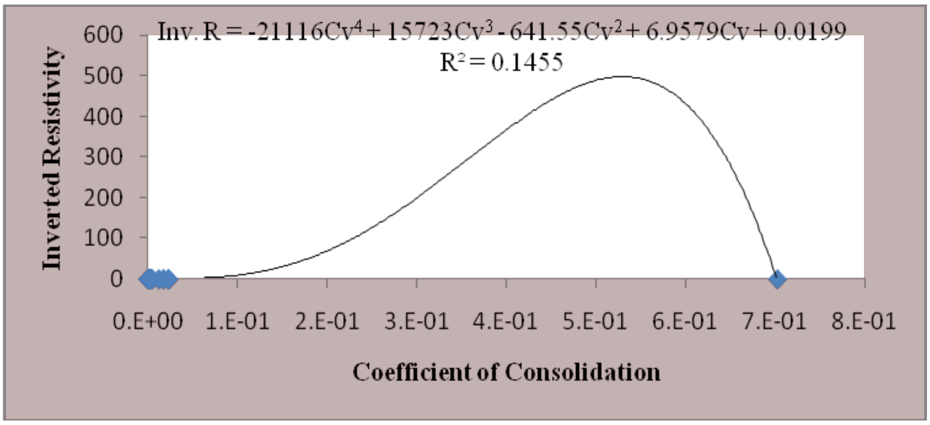

Figure 8: Correlation between Inverted Resistivity and Coefficient of Consolidation

4.2.9 Correlation between Inverted Resistivity and Coefficient of Uniformity

The regression trend of Inverted Resistivity and Coefficient of Uniformity (M_V) is as shown in Figure 9. The result presents a weak or no correlation between the Inverted Resistivity and Coefficient of Uniformity (M_V) with the coefficient of determination R^2 = 0.201 is as shown in Figure 9.

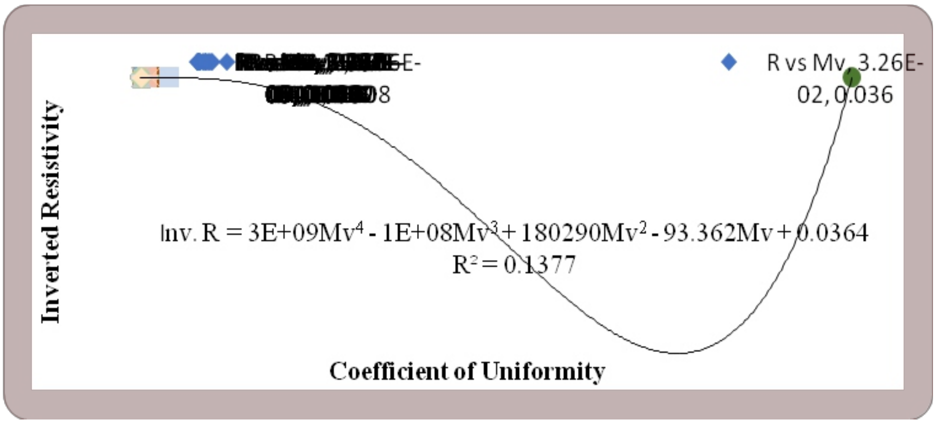

Figure 9: Correlation between Inverted Resistivity and Coefficient of Uniformity

4.2.10 Correlation between Inverted Resistivity and Spec ific Gravity

The regression trend of Inverted Resistivity and Specific Gravity (G_S) is as shown in Figure 10. The result presents a weak or no correlation between the Inverted Resistivity and Specific Gravity (G_S) with the coefficient of determination R^2 = 0.0823 is as shown in Figure 10.

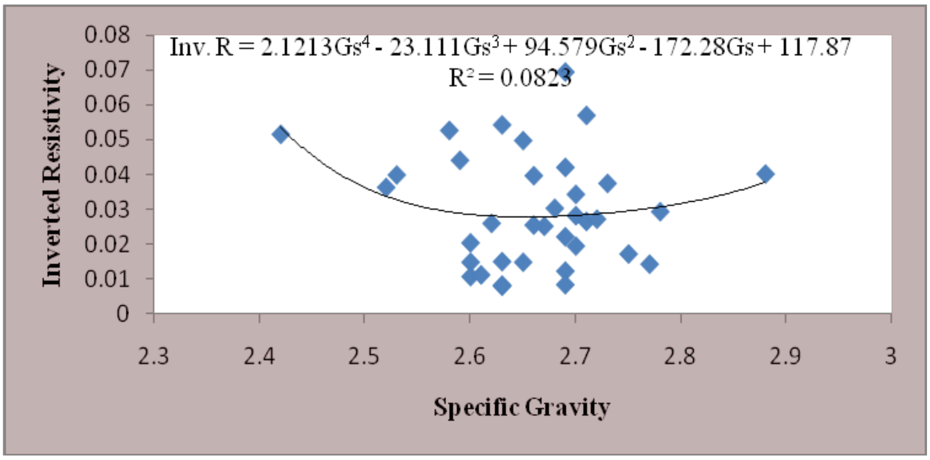

Figure 10: Correlation between Inverted Resistivity and Specific Gravity

4.2.11 Correlation between Resistivity and Bulk Density

The regression trend of Inverted Resistivity and Bulk Density is as shown in Figure 11. The result present a weak or no correlation between the Inverted Resistivity and Bulk Density with the coefficient of determination $R^2 = 0.2711$ as shown in Figure 11.

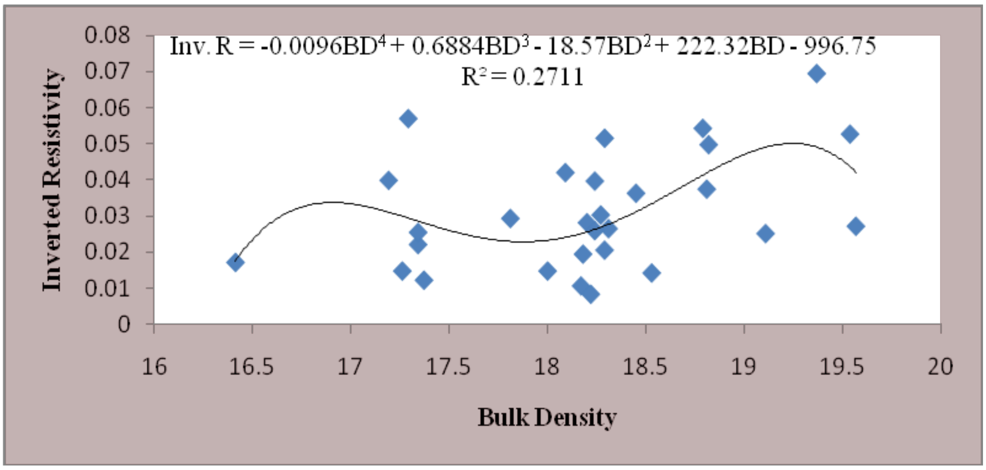

Figure 11: Correlation between Inverted Resistivity and Bulk Density

5.0 CONCLUSION

Predictive Inverted Electrical Resistivity from Geotechnical Engineering Parameters was evaluated by carrying out a simple regression analysis with all the basic engineering parameters. Relationship between Inverted Electrical Resistivity and other parameters demonstrate a 4th order Polynomial trend.

Generally, the results show that the coefficient of determination ranges from no correlation to mild correlations. It can be seen that cohesion has the highest R^2 value of 0.40 which is considered to have a mild correlation. This is preceded by Liquid Limit with R^2 value of 0.35 and Plasticity Index with R^2 value of 0.33. Also, Bulk Density, Natural Moisture Content, Plastic Limit, Coefficient of consolidation, coefficient of volume compressibility, Sieve #200, Angle of Internal Friction and

Specific Gravity have R^2 value of 0.27, 0.24, 0.16, 0.16, 0.14, 0.13, 0.11 and 0.08 respectively which are considered to have weak or no correlations according to Taylor 1990. It is therefore recommended that correlation of inverted electrical resistivity be made with cohesion that gave mild correlation of 0.40.

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The effect of Lime: Bitumen emulsion admixture on the plasticity and compaction properties of lateritic soil

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Abstract

This study presents the outcomes of laboratory assessment on the effect of Lime: Bitumen emulsion admixture on the plasticity and compaction properties of lateritic soil. The sourced soil material was air-dried and treated with up to 10% lime and bitumen emulsion by weight of the dry samples and compacted using British Standard Light compaction energy. The experimental assessment of the Atterberg limit (liquid limit and plasticity index) revealed a reducing trend with increasing lime-bitumen blend and this is an indicator of soil improvement. For the compaction response, the addition of lime-bitumen facilitated a decreased maximum dry density (MDD) and optimum moisture content (OMC). Based on the plasticity indices of the soil-additives, the blend of 4% lime - 10% bitumen enriched the plasticity tendency of the soil and this could be advantageous as surrogate materials to the conventional binding materials and it is recommended for use as sub-base material for pedestrian work ways or rural roads. Statistical package such as two-way analysis of variance (ANOVA) embedded in Microsoft excel software was employed for the testing of significant level of additives on the tested property.

Keywords: Atterberg limits; Compaction; Lime; Bitumen emulsion; Lateritic soil.

1.0 INTRODUCTION

Lateritic soils are highly weathered soils formed from materials with lower concentrations of oxides or hydroxides of iron and aluminium (Amu et al., 2011). Lateritic soil contributes to the general economy of the regions where they are found. Civil engineering studies on the usage of these materials have been in practises for ages and they have vast areas of applications in our society which includes: its use as foundation materials in civil infrastructural developments such as dams, retaining walls, air fields, pavements and waste containment applications (Lemougna et al., 2011; Sani et al., 2018; Osinubi et al, 2019, 2020). The use of indurated lateritic soil as a building material has been, and is still very common in Africa (Lemougna et al., 2011). Interestingly, a good amount of this soil material exists in our various communities but in most cases, it fails to match the minimum benchmark for use as construction material. This makes this deficient soil a good candidate for either soil replacement or improvement. Soil improvement can be justified when employed by its ability to overcome a deficiency and meet up with the engineering requirements for the construction purpose. Soil improvement has been regarded as a last resort for upgrading substandard materials where no economic alternative is available.

The studies of Zhang & Yi (2011), Latifi et al., (2013), Agashua & Ogbiye (2018) revealed that stabilizers like lime, bitumen and cement are good candidates for improvement of soil strength and reduction of soil settlement in soil re-engineering. However, the deployment of cement treatment were widely used for the improvement of the mechanical properties of soils, but due to the high cost of Portland cement, other low cost additives such as lime, rice husk ash, bitumen emulsion are now being considered and their effectiveness has to be determined. Recent advances made by soil researchers in the field of soil re-engineering have resulted to positive outcomes on the use of additives either as single stabilizer or multiple stabilizers in ameliorating deficient soil (Etim et al., 2019; Etim et al., 2020; Ogundipe, 2014; Attah et al., 2019; Attah et al., 2021a; Attah et al., 2021b; Alaneme et al., 2020a; Alaneme et al., 2020b; Alaneme et al., 2021; Attah et al., 2021c). Lateritic soils are often used as imported fill material for the prepared subgrade in many road projects (Okunnade, 2010, Amadi, 2010).

The studies of Nikraz (2016) and Kumar (2017) narrated the effectiveness of bitumen-cement and bitumen emulsion-ESP-CSA on lateritic soil materials. From their studies, it was established that bitumen and bitumen-additive facilitate firmness, versatility and penetrability of soil materials. In summary, the efficacy of bitumen emulsion, lime and Portland cement in stabilization of soils has been highlighted by researchers (Lesueur, 2000; Umar & Osinubi, 2003; Lesueur & Pott, 2004; Umar & Elinwa, 2005; Mutter, 2019) and has been found to be the most effective agent in reducing the swelling properties and water absorption capacity of soils. The current trend of soil re-engineering is the use either two or more additives in ameliorating deficient soil parameters and the aim of this practise is to reduce high consumption of traditional binders; thereby promoting the use of additives as surrogate materials and as well as lessening costs of construction projects. Hence, the definite objective of this study is to evaluate the effect of lime-bitumen emulsion blend on the plasticity and compaction properties of lateritic soil material.

2.0 MATERIALS AND METHODS

2.1 Materials

2.1.1 Lateri tic soil

Samples of lateritic soil were taken from borrow pit at a depth of 0.8m to 1.0m below the ground surface, to avoid organic matter, at Maitama Extension District (MED), Abuja. Nigeria.

2.1.2 Bitumen Emulsion

The bitumen emulsion used for this study was gotten from Kakatar Construction Company, Abuja.

2.1.3 Lime (Hydrated Lime)

The lime used for this study was sourced from Delta steel company limited, Ovwian-Aladja, Delta State, Nigeria. The elemental compositions of the materials (lime and unaltered soil) were made known via the aid of X-ray fluorescence experiment.

2.2 Methods

2.2.1 Index Properties

The particle size distribution of the virgin soil was conducted as per the details in Head (1982). The experimental procedures were executed by blending 0, 2, 4, 6, 8 and 10% Lime and 0, 2, 4, 6, 8 and 10% bitumen emulsion by dry weight of soil.

2.2.2 Atterberg limits

For the Atterberg limits test, the natural and modified soil mixtures passing through sieve No. 425μm were utilised and in accordance with BS 1377 (1990) and 1924 (1990) respectively.

2.2.3 Compaction

Compaction tests were carried out in accordance with BS 1377 (1990) and BS 1924 (1990) to determine the compaction characteristics of untreated soil and soil–lime – bitumen emulsion mixtures. Specimens were compacted with British Standard light (BSL) energy that involves a 2.5 kg rammer falling 300 mm onto three layers in a British Standard mould, each receiving twenty seven (27) blows each.

3.0 RESULTS AND DISCUSSION

3.1 *General characterization and classification of test material*

Displayed in Table 1 are the general properties of the unaltered soil. The results show that the reddish brown soil has natural moisture content, liquid limit, plastic limit and plasticity index of 6.7, 59, 21.6 and 37.4%, respectively. According to AASHTO (1986) and USCS (ASTM 1992), the tested soil falls under A-7-5(29) and CH group, respectively. The particle gradation is shown in Figure 1. Similarly, the elemental composition of the materials used (lime and soil) is displayed in Table 2 and property of the bitumen used is summarized in Table 3.

Table 1. Properties of the Natural Soil

Property	Quantity
Percentage passing BS No 200 sieve	68.5
Natural Moisture Content, %	18.4
Liquid Limit, %	44.4
Plastic Limit, %	21.6
Plasticity Index, %	22.8
Specific Gravity	2.26
AASHTO Classification	A-7-6 (14)
USCS	CH
Maximum Dry Density, Mg/m^3	1.67
Optimum Moisture Content, %	13.2
Colour	Reddish Brown

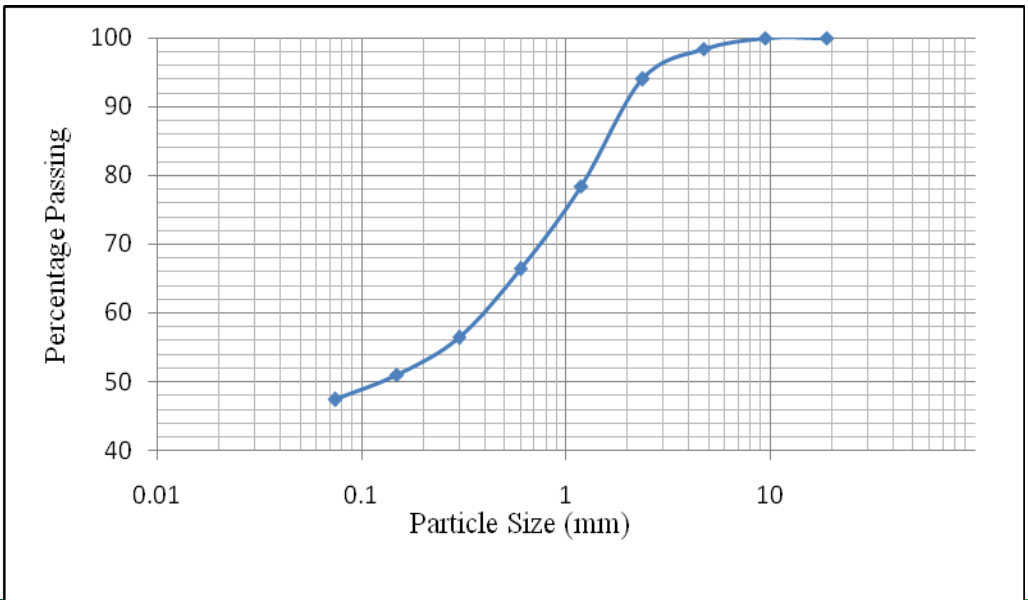

Fig. 1: Particle size distribution curve of lateritic soil.

Table 2. Oxide composition of materials

Oxide composition	Concentration (%)	
	Lime	Soil
Al_2O_3	-	30.15
CaO	69.14	0.30
SiO_2	-	36.05
V_2O_3	0.002	-
TiO_2	0.05	0.430
MnO	0.07	0.029
Fe_2O_3	0.288	27.15
ZrO_2	0.001	-
BaO	0.11	-
NiO	0.059	-
SrO	0.705	-
CrO_2	0.01	-
SO_3	-	0.48
Loss on ignition	29.36	4.55

Table 3: Properties of Bitumen emulsion and standard of tests used.

Test carried out	Results	Standards
Viscosity (sec)	100	BS 6313
Storage stability test 24 hr. (%)	0.5	ASTM D6927-05
Sieve test (%)	0.08	ASTM C 136
Residue by distillation (%)	65	ASTM D402 / D402M
Penetration at 25°C	140	AASHTO D T-49
Ductility at 25°C, min	45	ASTM D113
Solubility in trichloroethylene, %	98.0	ASTM D 2042 15

4.2 ATTERBERG LIMITS

4.2.1 Liquid Limit

The variation of liquid limit of the lateritic soil – bitumen emulsion mixtures with lime is shown in Fig. 2. Liquid limit of the lateritic soil decreased with lime and bitumen emulsion content. The liquid limit of the natural lateritic soil decreased from 50 to a value of 32% when treated with 4% lime/10 % bitumen emulsion and this could be linked with the alteration of soil fabric due to the incorporation of additives. The decrease could also correspondingly be attributed to the agglomeration and flocculation of the soil particles, as a result of cation exchange capacity at the surface of the soil particles (Moses & Afolayan, 2011, Etim *et al*., 2017). The decrease in the liquid limit could also be due to water proofing and adhesion properties of the bitumen emulsion. In this case, soil particles were coated by the bitumen emulsion; thus reduces the penetration and absorption of water by the soil which in turn, resulted in decrease of the soil liquid limit. Similar statement was made by (Mutter, 2019). Other studies by Lesueur (2000) and Lesueur & Pott, (2004) reported similar decreasing trends.

The two – way analysis of variance (ANOVA) test on the liquid limit result shows that the effects of lime and bitumen emulsion on lateritic soil were statistically significant (F_{CAL}=25.5> F_{CRIT}

= 2.6) for bitumen emulsion and (F_{CAL}=5.79 > F_{CRIT} = 2.6) for Lime. The effect of bitumen emulsion on the liquid limit result was more pronounced than that of lime (α =5 % and $p < 0.05$).

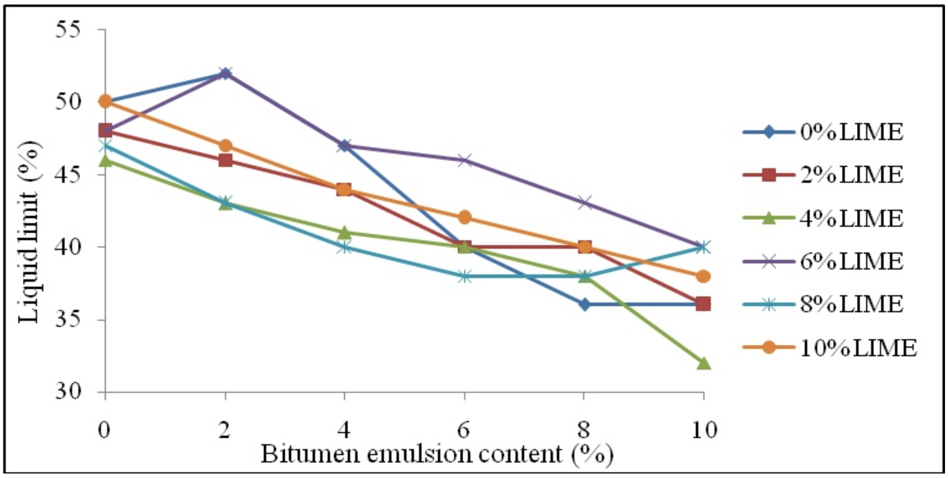

Fig. 2: Variation of liquid limit of laterite with lime and bitumen content

4.2.2 Plastic Limit

The variation of plastic limit of lateritic soil with bitumen emulsion and lime content is shown in Fig. 3. Plastic limit generally increased with increase in both lime and bitumen emulsion contents with exception in few cases. The trend of plastic limits is negligible so far as there is a reduction in PI which signifies soil improvement. The recorded increase could be probably due to the higher release rate of Ca^{2+}, Si^{++} and Al^{++} cations ion the modified soil with increase in the admixture concentration. Similar statement was reported by (Ramzi et. al., 2001).

The two – way analysis of variance (ANOVA) test on the plastic limit result shows that the effects of lime and bitumen emulsion on lateritic soil were statistically significant (F_{CAL}=5.08> F_{CRIT} = 2.6) for bitumen emulsion and (F_{CAL}=32.01> F_{CRIT} = 2.6) for lime. The effect of lime on the plastic limit was more pronounced than that of bitumen emulsion (α =5 % and $p < 0.05$).

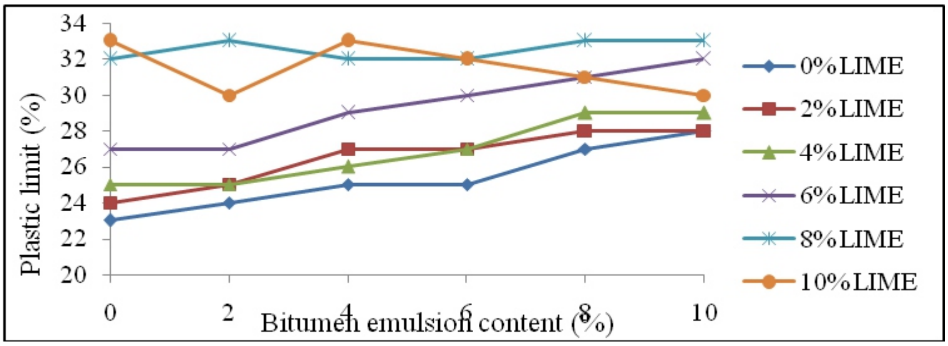

Fig. 3: Variation of plastic limit of soil – lime mixtures with bitumen content

4.2.3 Plasticity Index

The variation of plasticity index limit of lateritic soil with bitumen emulsion and lime content is shown in Fig. 4.The plasticity index significantly decreased with increase in both admixtures. The plasticity index of lateritic soil displayed a decreasing trend and its values ranged between 3 and 27%. It is important to note that the decreasing movement of plasticity index infers a considerable level of soil improvement. The decrease could be associated with the agglomeration and flocculation of the soil particles which is as a result of exchange of ions at the surface of the soil particles (Moses & Afolayan, 2011). The water proofing and adhesion behaviour of bitumen emulsion could be responsible for the decline in plasticity index of the treated soil (Lesueur, 2000; Lesueur & Pott, 2004; Mutter, 2019).

The two – way analysis of variance (ANOVA) test on the plasticity index result shows that the effects of lime and bitumen emulsion on lateritic soil were statistically significant (F_{CAL}= 27.55 >F_{CRIT}= 2.6) for bitumen emulsion and (F_{CAL}=11.07> F_{CRIT} = 2.6) for lime. The effect of bitumen emulsion in the plasticity index result was more pronounced than that of lime (α =5 % and $p < 0.05$).

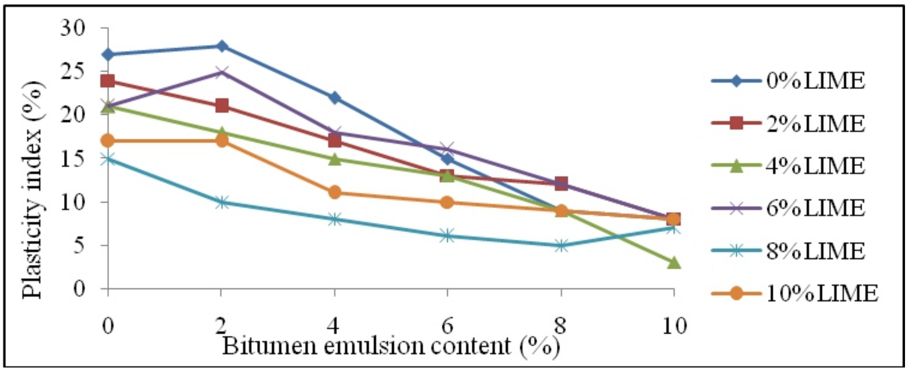

Fig. 4: Variation of plasticity index of soil – lime mixtures with bitumen content

4.3 Compaction Characteristics

4.3.1 Maximum Dry Density

The variation of maximum dry density (MDD) of soil-lime mixtures with bitumen emulsion content is presented in Fig. 5. Generally, MDD values decreased with higher lime content and also as the bitumen emulsion content increases. The MDD of the virgin soil decreased from 1.67Mg/m^3 to its lowest value of 1.49Mg/m^3 recorded at 6%lime-6%bitumen. Interestingly, the recorded decrease in the values of MDD is as a result of the increase in lime/bitumen particles coating the soil to form large aggregates which then occupy larger spaces within the soil and as well increase the fluid content within the soil matrix. The same trend of decrease in MDD was also reported by Hausmann (1990), Stephen (2006), Osinubi et al., (2007) and Sani (2012). Also specific gravity of additives significantly influences the MDD of treated soil. The decline in MDD with increase in additive content could be attributed to this effect as reported in literatures (Osinubi et al., 2015; Etim et al. 2017; Sani et al. 2018).

The two – way analysis of variance (ANOVA) test on the MDD result shows that the effects of lime and bitumen emulsion on lateritic soil were statistically significant (F_{CAL}=67.1 > F_{CRIT} = 2.6) for bitumen emulsion and (F_{CAL}=14.9 > F_{CRIT} = 2.6) for lime. The effect of bitumen emulsion on the MDD result was more pronounced than that of lime (α =5 % and $p < 0.05$).

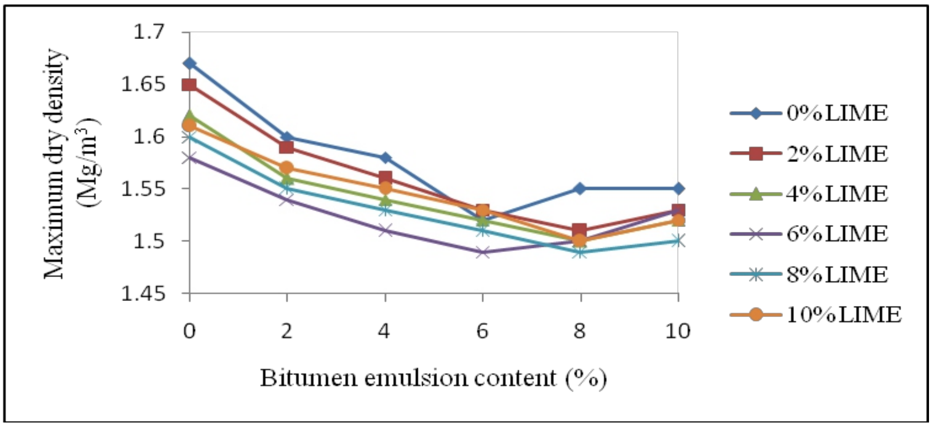

Fig. 5: Variation of maximum dry density of soil – lime mixtures with bitumen content

4.3.2 Optimum Moisture Content

The effect of bitumen emulsion on lateritic soil is for waterproofing and as binding agent while the lime helps in reducing the clay content of the lateritic by the agglomeration of the soil particles to form larger clods (Etim et al., 2017; Mutter, 2019). The effect of the combination of bitumen emulsion and lime on lateritic soil improves soil strength, workability, durability and dries wet soil (Lesueur, 2000; Lesueur & Pott, 2004; Etim et al., 2017; Mutter, 2019). The variation of optimum moisture content (OMC) of soil-lime mixtures with bitumen emulsion contents is shown in Fig. 6. Generally, the increment in lime content facilitated an increase in OMC values while the reverse was the case for soil-bitumen mixtures. The increase in OMC with increase in lime content is as a result of more water needed to dissociate Ca^{2+} and OH^- ions to supply Ca^{2+} for the cation exchange reaction (Etim et al., 2017).

The two – way analysis of variance (ANOVA) test on the OMC result shows that the effects of lime and bitumen emulsion on lateritic soil were statistically significant ($F_{CAL}=54.8 > F_{CRIT} = 2.6$) for bitumen emulsion and ($F_{CAL}=191.7 > F_{CRIT} = 2.6$) for lime. The effect of lime in the OMC result was more pronounced than that of bitumen emulsion ($\alpha = 5$ % and $p < 0.05$).

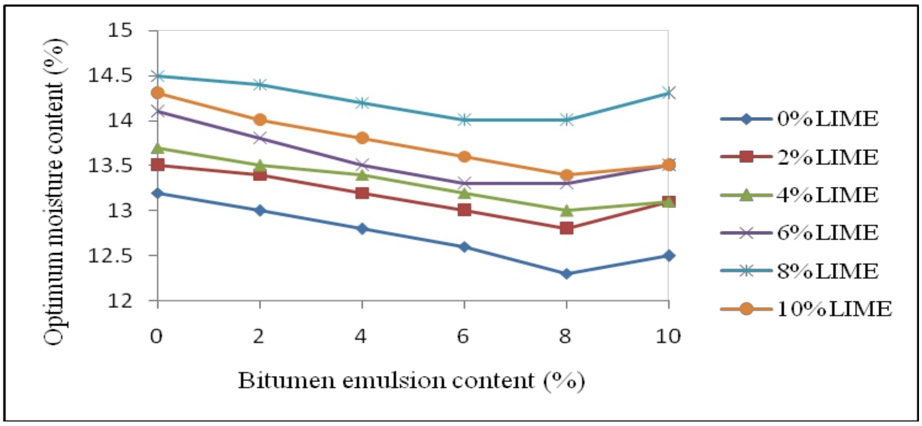

Fig. 6: Variation of optimum moisture content of soil – lime mixtures with bitumen content

5.0 CONCLUSION

The virgin soil belongs to the CH group using the Unified Soil Classification System and A-7-5(29) soil using the AASHTO classification system. Based on the results obtained, the following conclusions were drawn:

1. Both the liquid limit and plasticity index of the lateritic decreased with increase in lime and bitumen emulsion content while the plastic limit generally increased with increase in both lime and bitumen emulsion contents with exception in few cases.
2. For the compaction response (maximum dry density and optimum moisture content) reduced drastically with the increase with higher lime and bitumen emulsion content.
3. The plasticity indices of the treated soil after admixing with lime-bitumen at some point was far below 12% which denotes the least possible benchmark for a material to be measured as subgrade material as documented by the Federal Ministry of Works and Housing, FMWH.
4. Based on the plasticity indices, blending of 4% lime/10% bitumen emulsion improved the soil workability and is recommended as sub base material for pedestrian work ways or rural roads.

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