Geotechnical Properties of Lateritic Soil Stabilized with Cement and Corn Cob Ash

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Abstract: This study investigates the stabilization of laterite soil with blends of cement and corn cob ash (CCA) at different percentages. Lateritic soil contains a significant amount of clay materials; its strength and stability cannot be assured under load especially in the presence of moisture. The presence of high plastic clay in lateritic soil causes soil cracks and damage on road pavements, road ways, building foundations, or any civil engineering construction projects. Due to deficiencies in properties of laterite soil for highway construction, there's need to improve its properties through stabilization process. This study examines the stabilization of laterite soil using a mixture of corn cob ash (CCA) and cement at different percentages. A central composite design (CCD) was employed to optimize the mixture proportions of CCA and cement and the responses were unconfined compressive strength, (UCS) and California bearing ratio (CBR). The result of natural soil had an average value of the liquid limit, plastic limit and plasticity index as 22.7, 6.5 and 16.2 respectively, while the linear shrinkage was 11%. The results of the numerical optimization revealed that the optimal mixture consisted of 2% CCA, 3% Cement, 82.9% CBR Unsoaked, 31.2% CBR Soaked, 352 N/mm^2 UCS at 7 days, 357 N/mm^2 UCS at 7+7 days, 419.1 N/mm^2 UCS at 14 days and 795.9 N/mm^2 UCS at 28 days. The stabilized soil samples exhibited significant improvements in unconfined compressive strength, durability, and reduced plasticity. The use of CCA as a supplementary cementitious material also offers environmental benefits by reducing waste and promoting sustainable construction practices. The findings of this study demonstrate the potential of CCA and cement stabilization for improving soil properties and provide valuable insights for future applications in geotechnical engineering.

Keywords: *Lateritic Soil, Corn Cob Ash, Optimization, Rice Husk Ash, Stabilization*

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1. Introduction

The commonest material for the construction of the subgrade and subbase in the tropics is lateritic soil (Mai-Bade et al., 2021; Federal Ministry of Works and Housing, 1997). This lateritic soil does not have adequate engineering properties to really carry the expected wheel load, especially high traffic density (Amu et al., 2020). Therefore, the need to improve the engineering properties of soil has been of great concern to the civil engineers especially in Nigeria where the cost of locating, excavating and hauling of alternative suitable soil may be too expensive (Ishola et al., 2024).

Soil improvement is achieved through either the stabilization or modification of the unsuitable or unstable soil (Bidur et al., 2024; Bamforth et al., 2021). Laterite soils are sedimentary rock deposits arising from the disintegration of rocks into smaller particles, is one of the most common and readily available road building materials that can be sourced locally in Nigeria (Matawal and Tomorin, 1996; Abdulkarim & Umar, 2020).

According to Morakinyo et al. (2022), lateritic soils are generally used for road project in Nigeria. The soil in its original state, possesses low bearing capacity and overall low shear strength due to its high clay content. If lateritic soil contains a significant amount of clay materials, its strength and stability cannot be assured under load especially in the presence of moisture (Emmanuel et al., 2021). The presence of high plastic clay in lateritic soil, causes soil cracks and damage on road pavements, road ways, building foundations or any civil engineering construction projects.

Due to deficiencies in laterite soil suitable for highway construction, there is need to improve the properties of existing soils through stabilization processes. The process of blending and mixing materials with soil to alter certain properties of the soil is referring to as stabilization. According to Ayodele and Sunday (2024), the process may comprise of blending of soils to achieve a desired gradation or the mixing of commercially available additives that may improve the gradation, texture or plasticity or act as a binder for cementation of the soil. There are three purposes of stabilization, that include; strength improvement, dust control and soil water proofing (Amu et al., 2020).

Also, Nigeria remains one of the nations with very low waste management collection system, as a result of that, the poor management often constitutes a threat to the environment. Study has shown that a lot of these wastes such as rice husk (Adetoye et al., 2023; Sani and Adetoye, 2024), sawdust ash (Sarkin-Shanu et al., 2024), waste glass powder (Abdulkarim et al., 2025), eggshell powder (Adanu et al., 2024) to mention but a few could be useful when recycled. For example, when incinerated to ashes they possess pozzolanic properties which can serve as partial replacement of cement (Ahmad et al., 2021).

Corn cob is the hard-thick cylindrical central core of maize (on which are borne the grains or kernels of an ear of corn). Adesanya and Raheem (2009) described corn cob as the agricultural waste biomass obtained from corn; which is the most significant cereal crop in sub – Saharan Africa. Plateau precisely is considered as one of the states in Nigeria with the highest rate of corn farming and production.

Ordinary Portland Cement (OPC) is one of the most effective and commonly used stabilizers of soils. According to Afolayan et al. (2022), cement manufacturing causes 5-7% of the total CO_2 emissions and its manufacturing process is one of the major causes of global warming.

Consequently, it is imperative that the world moves towards a partial or total replacement of ordinary Portland cement (OPC) as a soil stabilizer, this has been the reason why many researchers have carried out several researches on some agricultural wastes due to their pozzolanic content to test their effectiveness as stabilizing agents that can be used to further improve the geotechnical properties of soils and other engineering materials other than OPC.

2. Relevant Literatures

2.1. Lateritic Soil

According to Oriaje et al. (2021), laterite is a soil and rock type rich in iron and aluminum and occur mostly in tropical and sub- subtropical regions. Laterite in itself contains a substantial amount of clay minerals hence its strength and stability cannot be guaranteed under load especially in the presences of moisture. Laterite original has low bearing capacity and low strength due to high content of clay. When laterite soil contains large amount of clay, its strength and stability cannot be guaranteed under load in the presence of moisture. Crack and damage on pavement, road way, foundation and other civil engineering works/construction result to the high plasticity in clay material present in the laterite (Matawal and Tomorin, 1996; Abdulkarim & Umar, 2020). The laterite soil classification is shown in Table 1.

Table 1: Soil Classification

Names	Sesquioxide ratio
Laterite	1.33
Lateritic Soil	1.33 - 2.00
Non-lateritic, tropical soil	2.0

2.1.1. Mode of Formation

Laterite are formed from leaching of parent sedimentary rock (Sandstone, Clay, Limestone); metamorphic rocks (Schists, Gneisses, Peridotites); and mineralized proto-ores (Charles and Okechuwku, 2020; Onyeka and Osegbowa, 2020). An essential feature for the formation of laterite soil is the repetition of wet and dry seasons. Rocks are leached by percolating rain water during dry season. these ions form solute salt compounds which dry on the surface; these salts are washed away during next wet season.

Lateritic soils are of low to medium plasticity. The lateritic soils strength decreased with increase in depth beneath hardened crust. When lateritic soil contains a large amount of clay materials its stability and strength and stability cannot be guaranteed under load in presence of moisture. lateritic soil consists of high plastic clay, the plasticity of the soil may result to cracks and damage on pavement, roadways, building foundations or any civil engineering construction projects (Das et al., 2019)

The common values on the properties of laterite are given in Table 2.

Table 2: Properties of Laterite

Property	Values
Natural moisture content (%)	20.13
Liquid limit %	52.23
Plastic limit %	34.77
Plasticity index %	14.46
Specific gravity	2.22
Maximum dry density Mg/m^3	1.92

Optimum Moisture Content %	17.39
AASHTO classification	A-7-5
USCS classification	SM
Colour	Reddish-brown

Source: Oriaje et al. (2021).

2.2 Effect Pozzolanic Waste Material on Lateritic Soils Stabilization

These agricultural wastes are known to be pozzolanic in nature; this is further confirmed by a lot of literatures that their ashes are used as stabilizing agents to improve engineering materials for road construction and other engineering purposes.

Uche and Ahmed (2013) investigated the effects of Millet Husk Ash (MHA) on index properties of marginal lateritic soil at Maikunkele area of Minna, Nigeria. The study was carried out in accordance with BS 1377 (2002)

Amu et al. (2011) examined the consistency limits of lateritic soils modified with coconut shell ash (CHA) with a purpose of obtaining a cheaper and effective road stabilizer. They reported that geotechnical properties were further improved at 4% addition of CHA, with a reduction of nearly 18-46% in plasticity index of the lateritic soil. Another study by Oluremi et al. (2012) concluded that the addition of CHA increased the plastic limit while it decreased the plasticity index of the soil. The liquid limit also decreased from 0 to 4% addition of CHA. Both researchers concluded with possibility of using not more than 4% CHA to stabilized lateritic soil for good modification of its consistency limits.

Okafor and Okonkwo (2009) carried out some studies on the effects of the rice husk ash (RHA) on some geotechnical properties of lateritic soil obtained from Obukpa, Enugu State, Nigeria. The soil was replaced with RHA from 0-12.5% by mass of the soil at interval of 2.5%. The results of the study showed a decrease in liquid and plastic limits as the rice husk ash increased from 0 to 7.5% but these properties increased with further addition of the RHA. In addition, increasing ash contents also led to a continuous increase in plasticity index and linear shrinkage limit of the stabilized soils.

3. Materials and Methods

This research assessed the effect of corn cob ash (CCA) blended with Portland cement on the geotechnical properties of stabilized lateritic soil to be used for road construction. The soil samples were collected from Shendam while the corn cob was obtained from Kerang, Plateau State, Nigeria.

3.1. MRI Characteristics and Their Correlations

Dried Corn Cobs, lateritic soils and cement are basically the materials used for this study. Soil testing was carried out to assessed the influence and suitability of the cement stabilized lateritic soil mixed with corn cob ash in relation to its strength and other geotechnical properties.

3.1.1. Corn Cob Ash (CCA)

This research assessed the effect of corn cob ash (CCA) blended with Portland cement on the geotechnical properties of stabilized lateritic soil to be used for road construction. The soil samples were collected from Shendam while the corn cob was obtained from Kerang, Plateau State, Nigeria.

3.1.2. Soil

The soil used for this study was obtained from a borrow pit along Shendam-Yelwa road (7.1oN, 3.3oE) North-Central Nigeria via method of disturbed sampling. The location was chosen due to vast availability of lateritic soil in the area. The sample was taken to the geotechnical laboratory of Department of Civil Engineering, Abubakar Tafawa Balewa University Bauchi, where it was air-dried, stored and made readily available for characterization and geotechnical properties tests in accordance with BS 1377:2000.

3.1.3. Cement

The soil used for this study was obtained from a borrow pit along Shendam-Yelwa road (7.1oN, 3.3oE) North-Central Nigeria via method of disturbed sampling. The location was chosen due to vast availability of lateritic soil in the area. The sample was taken to the geotechnical laboratory of Department of Civil Engineering, Abubakar Tafawa Balewa University Bauchi, where it was air-dried, stored and made readily available for characterization and geotechnical properties tests in accordance with BS 1377:2000.

3.2. Methods

Elementary tests such as: Natural Moisture Content, specific gravity, particle size analysis and Atterberg limits test were carried out on the natural soil sample for the purpose of determining its index properties and general classification of cement which was the main stabilizing material and thoroughly mixed with the soil sample to determine the optimum cement requirement of the sample. All the tests were carried out in the Civil Engineering Laboratory of Abubakar Tafawa Balewa University, in accordance with the standard procedures of BS 1377: 2000.

3.2.1. Determination of natural moisture cintent of the soil (compaction test)

In this test, the moisture tin and lid number were recorded and weighed, the mass of empty tin with its lid as (M1), the moisture soil was placed in the moisture tin and the lid secured. The mass of the moisture tin (containing the moisture soil) with the lid were determined and recorded as (M2), the lid was removed and the moisture tin (containing the moisture soil) placed in the oven set at 105^{O}C for 24 hours. It was then removed carefully but securely, and the lid was replaced on the moisture tin using gloves, and allowed to cool to room temperature. The mass of the moisture tin and lid (containing the dry soil) were weighed and recorded as (M3). The moisture Can and lid were emptied, cleaned and kept, the moisture content was calculated by the Equation (1)

$$Moisture\ content\ (w)\ =\ \frac{M_2\ -\ M_3}{M_3 - M_1}\ -\ M_1\ X\ 100 \qquad \ldots (1)$$

Where,
M_1 = mass of empty can
M_2 =mass of moist tin (containing the moisture soil)
M_3 = mass of moist tin with dry soil
w = moisture content in percentage

3.2.2. Specific gravity

Specific gravity of a substance denotes the number of times that substance is heavier than water. In case of soils, specific gravity is the number of times the soil solids are heavier than equal volume of water. In this test, an empty clean bottle was weighed and recorded as M_1, 10g of the soil were poured into the bottle and weighed as M_2. It was bed filled with water, shaken well to remove air bubbles, allowed to settled and measured the next day as M_3, the empty bottle was filled with air free water and weighed as M_4. The specific gravity was calculated from the specific gravity formula in Equation (2).

$$Specific\ gravity\ Gs\ =\ \frac{(M_2 - M_1)}{(M_4 - M_1) - (M_3 - M_2)} \qquad \ldots (2)$$

M_1=weight of empty bottle.
M_2= weight of bottle + sample
M_3= weight of bottle +sample +liquid
M_4 = weight of bottle with liquid full.

3.2.3 Atterberg Limits

The Atterberg limits test was conducted to determine the liquid, plastic limit, the plasticity index and shrinkage limit which are described below;

Liquid limit

Sample, that pass 425μm BS sieve size were prepared on a glass plate with distilled water and stored for 24 hours in an air-tight container. The soil samples were removed and remixed for at least 10 minutes on the glass plate, the cylindrical metal cup (55 mm diameter by 40 mm deep) was filled up with the homogeneous mixture of the soil with the aid of a palette knife, excess was trimmed off and the top smoothen carefully not to trap any air bubbles. The cup was placed on the plunger and the cone pointer was lowered to touches top of the soil and the dial gauge reading set at zero, the cone was released by pressing the knob for 5 second and dial reading was recorded. Two samples were taken for moisture content determination. The tests were repeated at different moisture content. The penetration and moisture content were plotted on linear scale and the penetration at 20 mm gave the liquid limit. Values of penetration and moisture content for the samples are shown in the result obtained.

Plastic limit (PL)

A portion of the sample used for the determination of the liquid limit was used for the plastic limit. About 20g of the sample was rolled with hand, the ball was divided into two parts of about 10g, and each of these further divided into four equal parts. The soil sample was rolled with finger in to 6 mm diameter threads and each thread rolled between finger tips on a clean glass plate with sufficient pressure to reduce the diameter to 3 mm. The moisture content of the threads was taken as the plastic limit.

Plasticity Index (PI)

Plasticity index was calculated from the liquid limit and plastic limit test conducted, and is defined as the difference between the liquid limit and the plastic limit as shown in Equation 3.

$$PI\ =\ LL\ -\ PL \qquad \ldots (3)$$

Where
PI is plasticity index of the soil (%)
LL and PL are liquid limit and plastic limit of the soil expressed in %

Shrinkage Limit (SL)

A portion of the sample that was used for the determination of liquid limit was used to filled up the shrinkage mould which is 140mm in length and was smoothen and put in the oven for the determination of the shrinkage limit.

3.2.4. Classification of Soil

The soil was classified using the AASHTO classification system ASTM D3282-09 and Unified Soil Classification System ASTM D2487-11.

3.2.5. Compaction Test

The compaction test standard proctor (SP) was conducted in accordance with BS 1377: Part 4: (1990) and BS 1924: Part 2: (1990) for the natural and modified soil respectively. A 3 kg of oven dry soil sample that passes through 20 mm BS sieve size was mixed with 6-8 % water and put into the compacting mould of 105 dia. It was compacted with 2.5 kg compaction rammer, sliding freely in a tube and falling freely at the height of 300 mm drop. The compaction was done in three (3) layers; each layer receives 27 blows of the 2.5 kg compaction rammer. When it was completed, the top of the soil was trimmed, the mould containing the compacted soil weighed and recorded. The moisture content sample was taken from the top and bottom of the soil for moisture content determination. The soil was broken and mixed again with more water for compaction; the procedures were repeated for five times from which the optimum moisture content (OMC) and maximum dry density (MDD) were determined by plotting a graph of dry density versus moisture content. The procedure was repeated for the soil using requisite levels of modified blend.

The compactive effort in the case of the Modified Proctor (MP) compaction test consisting of the energy derived from a 4.5 kg rammer that is falling freely through a height of 450 mm onto five equal layers of the soil. Each layer was given 27 blows of the rammer. The procedure of the modified proctor compaction test is the same as that of the standard proctor test with the exception of the size of the rammer, height of fall and number of blows Equations (4) and (5) will be used to calculate the bulk and dry densities respectively.

$$\rho_b = \frac{M_2 - M_1}{v} \quad \ldots (4)$$

$$\rho_d = \frac{100 \,.\, \rho_b}{100 + w} \quad \ldots (5)$$

Where;

ρ_b = Bulk density (g/cm^3)
M_1 = mass of empty mould (g)
M_2 = mass of mould + wet soil (g)
W = moisture content (%)
ρ_d= dry density

3.2.6. California Bearing Ratio Test

The California bearing ratio (CBR) test is an empirical test developed by the California State Highway Department for the evaluation of subgrade strengths. In the test as given in BS 1377: 1990: part 4:7.4, a specimen which is 127 mm in height and 152 mm in diameter was compacted into the CBR mould. The specimens were prepared in five layers and heavy rammer was used to give 56 (fifty-six) blows onto each layer. The load required to cause a circular, 49.65 mm in diameter, to penetrate the specimen at a specified rate of 1.25 mm per minute were measured. From the test results, the CBR value were calculated. This was done by expressing the corrected values of force on the plunger for a given penetration as a percentage of a standard force. Mathematically the CBR is expressed as in Equation 6.

$$CBR = \frac{P_t}{P_s} X\ 100 \quad \ldots (6)$$

Where:
P_t = Corrected unit (or total) test load Corresponding to the chosen penetration curve.
P_s = Corrected unit (for total) standard load taken from standard code.

3.2.7. Unconfined Compressive Strength (UCS)Test

This test is particularly important in establishing the stabilization potential of an additive and for subsequent evaluation of the material for use as a pavement material. This test was conducted in accordance with British standard, BS 1377:2022: part 7. The natural soil sample and stabilized soil sample were compacted in 1000 cm^3 mould at their respective optimum moisture content. The UCS test involved subjecting a cylindrical specimen under a steady increasing axial load until failure is achieved. The procedure was repeated for each soil blend with corn cob ash and cement. The unconfined compressive strength (UCS) was calculated using Equations 7 and 8. Equation 8 was used to calculate stress at a particular strain. For each sample the highest value of the stress was taken to represent the stress of that sample.

$$\sigma = \frac{R \times C_r \times (100 - \varepsilon\ \%) \times 100}{100 \times A_0} \quad \ldots (7)$$

$$\varepsilon = \frac{\Delta l}{l_0} \quad \ldots (8)$$

Where,

ε = Strain (%), Δl = Amount of compression at any stage (mm), R = Load ring reading at strain A, C_r= Mean calibration of load ring, l_0= Initial length of specimen (mm), A_0= Initial sectional area (mm^2) σ = Compressive stress at strain, ε

4. Results and Discussion

4.1. Characterization of Natural Soil

The resulting physical properties of the soil from tests are presented in Table 3. The particle size distribution curve is presented in Figure 1.

Table 3: Preliminary Results of the Natural Soil

Soil Properties	Value
Natural Moisture Content w, (%)	17.5
Specific gravity (G)	2.54
Liquid limit (%)	22.7
Plastic Limit (%)	6.5
Plasticity index (%)	16.2
Linear shrinkage (%)	11
Maximum dry density BSL (Mg/m^3)	1.64
Optimum moisture content BSL (%)	11.0
Unconfined compressive strength (kN/m^2)	215
Unsoaked Californian bearing ration CBR (%)	65.6
Soaked Californian bearing ration CBR (%)	29.1
% Passing BS Sieve No.200	56.4
AASHTO Classification	A-6
USCS Classification	CL
Colour	Light Brown

The preliminary test results of the natural soil show characteristics typical of lateritic clay, with a plasticity index of 16.2%, classification as A-6 under AASHTO and CL under USCS, and 56.4% passing the BS No. 200 sieve. The soil exhibited moderate unconfined compressive strength of 215 kN/m^2 and a significant reduction in California Bearing Ratio from 65.6% unsoaked to 29.1% soaked, indicating moisture sensitivity common to lateritic soils. These properties confirm the need for stabilization to improve strength and reduce volume change for use as pavement material.

The natural soil has an average value of the liquid limit, plastic limit and plasticity index as 22.7%, 6.5% and 16.2% respectively, while the linear shrinkage is 11%, hence the soil is classified as A-6 in accordance with the American Association of State Highway and Transportation Officials (AASHTO M 145-2012) soil classification system and CL in accordance with the Unified Soil Classification System (ASTM D 2487-2011). 56.4% passing sieve No. 200 means it's fine-grained, which explains the plasticity and the need for stabilization.

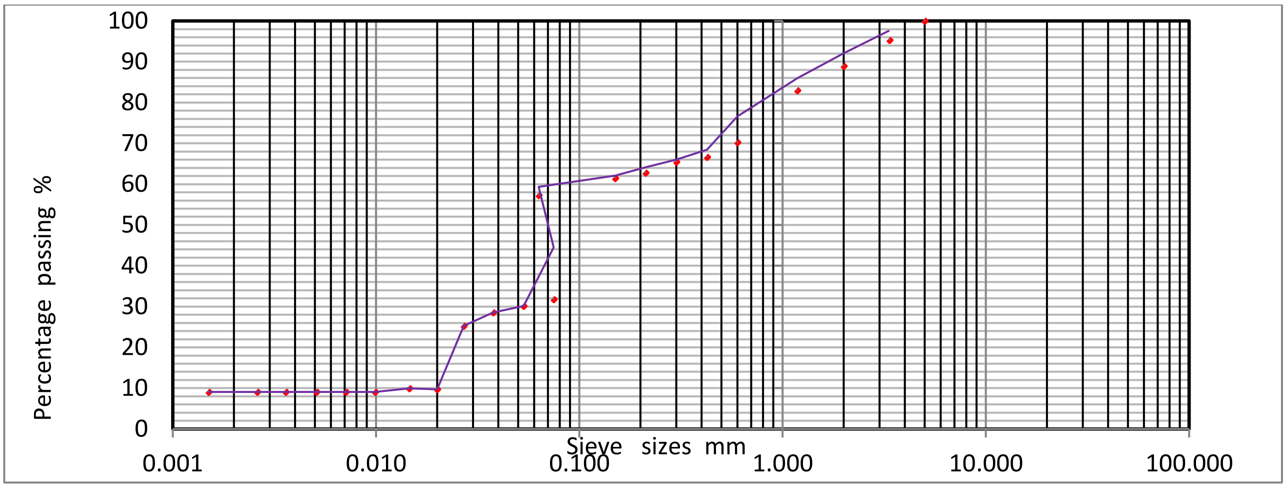


Figure 1. Particle Size Distribution Curve of the Natural Soil

4.2. Oxide Compositions of the Natural Soil

The oxide composition for the untreated soil sample obtained from the XRF analysis is presented in Table 4.

Table 4: Oxide Compositions of the Natural Soil from the XRF Analysis

Oxide	Concentration (%)
Silicon oxide (SiO_2)	55.9
Aluminum oxide (AI_2O_3)	22.5
Iron oxide (Fe_2O_3)	14.3
Calcium oxide (CaO)	0.42
Potassium oxide (K_2O)	1.42
Manganese oxide (MnO)	0.17
Barium oxide (BaO)	0.13
Zinc oxide (ZnO)	0.02
Copper (II) oxide (CuO)	0.04
Chlorine (Cl)	0.48
Titanium oxide (TiO_2)	1.89
ZrO_2	0.25

The results obtained from the XRF showed that the main oxide compositions of the soil comprised of Al2O3 –22.7 %, SiO_2 – 55.9 %, Fe_2O_3 – 14.3 %, and CaO- 0.42 %. The silica sesquioxide (SS) ratio of iron and aluminum was obtained using the Equation 9.

$$SS = \frac{S_iO_2}{Al_2O_3 + Fe_2O_3} \quad \dots (9)$$

The S-S = $\frac{55.9}{22.7+14.3}$ = 1.51. The S-S ration of 1.51 falls within the typical range for laterite soils. Laterite soils are characterized by a high sesquioxide ratio ranging from 1.3 to 2.0 Amu, (2020**)** and Emmanuel *et al.* (2021).

4.3. Oxide Compositions of Cora Cob Ash (CCA)

The XRF of corn cob ash is presented in Table 5.

Table 5: Oxide Properties of CCA

Oxide	CCA (%)
SiO_2	62.8
Al_2O_3	5.36
Fe_2O_3	3.72
CaO	1.83
MgO	0.62
SO_3	0.22
Na_2O	0.12
K_2O	0.17
P_2O_5	0.05
LOI	11.9

The X-ray fluorescence analysis of corn cob ash (CCA) presented in Table 5 shows that SiO_2 is the dominant oxide at 62.8%, with Al_2O_3 and Fe_2O_3 at 5.36% and 3.72% respectively. The sum of $SiO_2 + Al_2O_3 + Fe_2O_3$ is 71.88%, which exceeds the 70% minimum required by ASTM C618 for Class N natural pozzolans. The CaO content is low at 1.83% and LOI is 11.9%, indicating a predominantly siliceous ash with moderate carbon content. This oxide composition confirms that CCA has pozzolanic potential and can react with lime released during cement hydration to form additional cementitious products, which is why it’s effective for soil stabilization

4.4. Compaction Characteristics

The strength characteristics of engineering soils are important for assessing the OMC and MDD of soil structures. The OMC and MDD results are presented in Table 6.

Table 6: OMC and MDD Result

Run	Factor 1 A: CCA %	Factor 2 B: Cement %	OMC %	MDD (Mg/m^3)
1	2	3	9.5	1.78
2	2	1	9.2	1.84
3	4	3	11.5	1.78
4	4	2	9.9	1.80
5	4	2	9.9	1.80
6	6	3	11.7	1.76
7	6	1	12.6	1.88
8	4	2	9.9	1.80
9	2	2	9.3	1.84
10	4	1	9.9	1.80
11	6	2	12.6	1.88
12	4	2	9.9	1.80
13	4	2	9.9	1.80

The optimum moisture content of 9.2% and the maximum dry density of 1.84 mg/m3 was achieved at 97% natural soil, 2% corn cob ash and 1% cement.

4.5. California Bearing Ratio (CBR)

California bearing ratio (CBR) it’s a measure of soil’s load bearing capacity and its ability to resist deformation under load. The section below provides more information on the Californian bearing ratio (CBR).

The result of the CBR Soaked and Unsoaked is presented in Table 7.

Table 7: Result of CBR

Run	Factor 1 A: CCA %	Factor 2 B: Cement %	Response 1 CBR Unsoaked %	Response 2 CBR Soaked %
1	2	3	84	32
2	2	1	52	20
3	4	3	78	30
4	4	2	72	28
5	4	2	72	28
6	6	3	76	32
7	6	1	58	23
8	4	2	72	28
9	2	2	71	26
10	4	1	56	21
11	6	2	74	29
12	4	2	72	28
13	4	2	72	28

Table 7 gave the detailed presentation of the factors and responses for both soaked and unsoaked CBR of the soil treated with CCA. Highest CBR (Unsoaked) value of 84% was obtained at 2% CCA and 3% cement. 4% CCA and 3% cement also gave 78%. The detailed presentation of the factors and responses for the soaked CBR of the soil treated with CCA and cement showed that the maximum CBR (Soaked) value were obtained at 2% CCA and 3% cement and 6% CCA and 3% cement. The maximum CBR (Soaked) value of 32% was gotten which was above the minimum standard of 30% as specified by Nigerian General Specification, (2016).

The ANOVA results are presented in Table 8 – 9.

Table 8: Anova for CBR Unsoaked

Source	Sum of Squares	Df	Mean Square	F-value	p-value
Model	988.59	5	197.72	166.08	< 0.0001
A-CCA	0.1667	1	0.1667	0.1400	0.7194
B-Cement	864.00	1	864.00	725.76	< 0.0001
AB	49.00	1	49.00	41.16	0.0004
A^2	0.6905	1	0.6905	0.5800	0.4712
B^2	69.05	1	69.05	58.00	0.0001
Residual	8.33	7	1.19		
Lack of Fit	8.33	3	2.78		
Pure Error	0.0000	4	0.0000		
Cor Total	996.92	12			

Table 8 showed the Model F-value of 166.08 which implied that the model is significant. P-values less than 0.05 indicated that the model terms were significant. In this case B, AB, B^2 were significant model terms and values greater than 0.1 indicated model terms that were not significant.

Table 9: ANOVA for CBR (Soaked)

Source	Sum of Squares	df	Mean Square	F-value	p-value
Model	166.08	5	33.22	64.37	< 0.0001
A-CCA	6.00	1	6.00	11.63	0.0113
B-Cement	150.00	1	150.00	290.69	< 0.0001
B^2	7.57	1	7.57	14.66	0.0065
Residual	3.61	7	0.5160		
Lack of Fit	3.61	3	1.20		
Pure Error	0.0000	4	0.0000		
Cor Total	169.69	12			

In Table 9, the Model F-value of 64.37 implied that the model is significant. The P-values less than 0.05 indicated significant model terms. In this case, A, B, B^2 were the significant model terms while values greater than 0.1 indicated insignificant model terms.

The result of the Fit statistics for Soaked and Unsoaked CBR is presented in Table 10.

Table 10: Fit Statistics for Unsoaked CBR

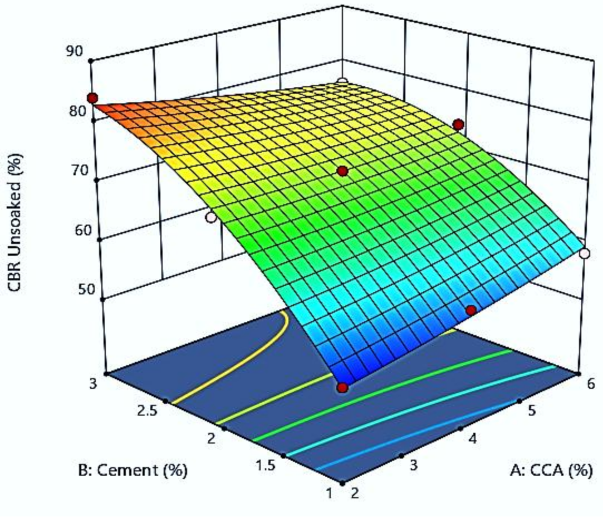
Parameters	Unsoaked CBR Value	Soaked CBR Value
R^2	0.9787	0.9916
Adjusted R^2	0.9635	0.9857
Predicted R^2	0.8252	0.9149
Adeq Precision	24.5894	41.8213

In Table 10, for Unsoaked CBR value, the Predicted R^2 of 0.9149 is in reasonable agreement with the Adjusted R^2 of 0.9857; i.e. the difference is less than 0.2. Adeq Precision ratio of 41.821 indicated an adequate signal. For Soaked CBR value, the Predicted R^2 of 0.8252 is in reasonable agreement with the Adjusted R^2 of 0.9635; i.e. the difference is less than 0.2. Adeq Precision ratio of 24.589 indicated an adequate signal.

The model equations for both soaked and unsoaked CBR are presented in Equation 10 – 11.

$$Soaked\ CBR = 7.52 + 0.56\ CCA + 13.12\ Cement - 0.375\ CCA\ (Cement) + 0.086\ CCA^2 - 1.66\ Cement^2 \quad \ldots (10)$$

$$Unsoaked\ CBR = 15.7 + 2.58\ CCA + 39\ Cement - 1.75\ CCA\ (Cement) + 0.125\ CCA^2 - 5\ Cement^2 \ldots \quad (11)$$



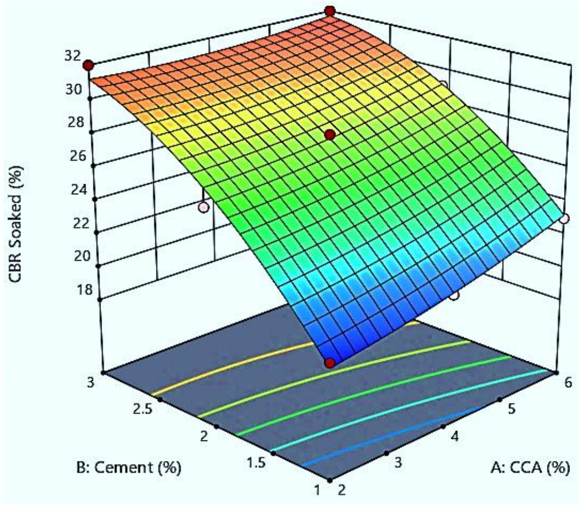


Figure 2. 3D Response Graph of Cement, CCA and Unsoaked CBR **Figure 3.** 3D Response Graph of Cement, CCA and Soaked CBR

Figure 2 showed the 3D response surface graph of interactions between CCA and Cement and the effect on Unsoaked CBR. The 3-D surface showed the correlation between the dependent variables (responses) and the independent variables (factors). The Figure 2 showed that increase in CCA and Cement caused increase in value of unsoaked CBR. Figure 3 shows 3D response surface graph of interactions between CCA and Cement and the influence on Soaked CBR. The 3-D surface showed the correlation between the dependent variables (responses) and the independent variables (factors). The Figure 3 shows that increase in CCA and Cement caused increase in unsoaked CBR value.

4.6. California Bearing Ratio (CBR)

The unconfined compressive strength of soil stabilized with cement and CCA and cured for 7 days, 7+7 days, 14 day and 28 days were conducted and results analyzed.

The result of the UCS of soil is presented in Table 11. The ANOVA results are presented in Table 12 – 13 and the Fit statistics in Table 14.

Table 11: Results of UCS

Run	Factor 1 A: CCA %	Factor 2 B: Cement %	Response 3 UCS at 7 Days kN/m^2	Response 4 UCS at 28 Days kN/m^2
1	2	3	349	809
2	2	1	162	605
3	4	3	342	782
4	4	2	322	740
5	4	2	322	740
6	6	3	334	784
7	6	1	270	660
8	4	2	322	740
9	2	2	295	700
10	4	1	185	640
11	6	2	325	755
12	4	2	322	740
13	4	2	322	740

In Table 11, soil treated with CCA showed higher UCS at 28 days than natural soil. UCS value of 349 kN/m2 was the maximum value obtained with soil stabilized with CCA at 7 days of curing with 2% CCA and 3% cement. The presence of CaO in the cement and silica oxide in CCA may be responsible for the strength gain which could be due to exothermic reaction between the CaO and lead to the formation of CSH according to (Shivaramaiah et al., 2020). UCS value of 809 kN/m2 was the maximum value obtained with soil stabilized with CCA at 28 days of curing. The best result was achieved using 2% CCA and 3% cement and conformed to standard (Ubani, 2020). The presence of CaO in the cement and silica oxide in CCA may be responsible for the strength gained which could be due to reaction between the CaO and silica oxide in the formation of CSH.

Table 12: ANOVA for UCS at 7 Days

Source	Sum of Squares	Df	Mean Square	F-value	p-value
Model	40566.08	5	8113.22	60.83	< 0.0001
A-CCA	2521.50	1	2521.50	18.91	0.0034
B-Cement	27744.00	1	27744.00	208.02	< 0.0001
AB	3782.25	1	3782.25	28.36	0.0011
B^2	5678.90	1	5678.90	42.58	0.0003
Residual	933.61	7	133.37		
Lack of Fit	933.61	3	311.20		
Pure Error	0.0000	4	0.0000		
Cor Total	41499.69	12			

In Table 12, the Model F-value of 60.83 implies the model is significant. P-values less than 0.05 indicate model terms are significant. In this case A, B, AB, B^2 are significant model terms.

Table 13: ANOVA for UCS at 28 Days

Source	Sum of Squares	Df	Mean Square	F-value	p-value
Model	41407.09	5	8281.42	66.54	< 0.0001
A-CCA	1204.17	1	1204.17	9.68	0.0171
B-Cement	36816.67	1	36816.67	295.81	< 0.0001
AB	1600.00	1	1600.00	12.86	0.0089
B^2	1250.21	1	1250.21	10.05	0.0157
Residual	871.22	7	124.46		
Lack of Fit	871.22	3	290.41		
Pure Error	0.0000	4	0.0000		
Cor Total	42278.31	12			

In the Table 13 the Model F-value of 66.54 implies the model is significant. P-values less than 0.05 indicate model terms are significant. In this case A, B, AB, B^2 are significant model terms.

Table 14: Fit Statistics for UCS at 7 Days

Parameters	7- Days Value	28-Days Value
R^2	0.9794	0.9775
Adjusted R^2	0.9647	0.9614
Predicted R^2	0.8042	0.8125
Adeq Precision	25.9485	25.1727

The 7- day Fit-statistics value showed that the Predicted R^2 of 0.8042 is in reasonable agreement with the Adjusted R^2 of 0.9647. Adeq Precision ratio of 25.95 indicates an adequate signal. In 28 days Fit-statistics values, the Predicted R^2 of 0.8125 is in reasonable agreement with the Adjusted R^2 of 0.9614; i.e. the difference is less than 0.2. Adeq Precision ratio of 25.17 indicates an adequate signal.

The model for UCS at 7 days and UCS at 28 days are presented in Equation 12 – 13. The 3-D response surface graphs are presented in Figure 4 – 5.

$$UCS\ at\ 7days\ =\ -159\ +\ 38.7\ CCA\ +\ 311\ Cement\ -15.4\ CCA\ (Cement)\ +\ 0.288\ Cement^2 - 45.3\ Cement^2 \quad \ldots (12)$$

$$UCS\ at\ 28\ days\ =\ 369 +\ 36.6\ CCA\ +\ 203\ Cement - 10\ CCA\ (Cement) - 1.194\ CCA^2\ - 21.3\ Cement^2 \quad \ldots (13)$$

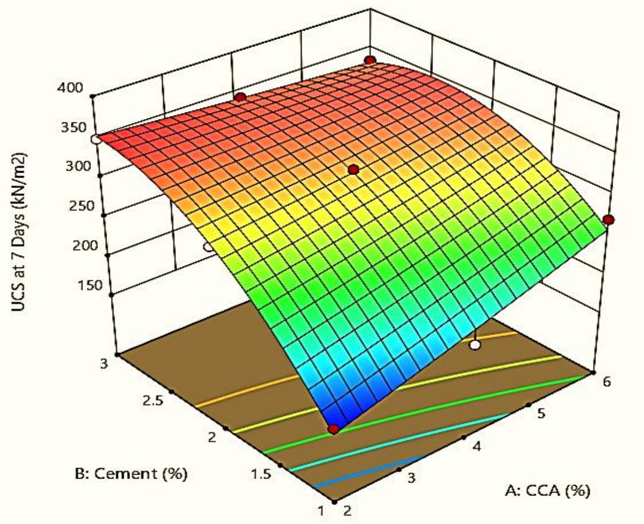


Figure 4. 3D Response Graph of Cement, CCA and UCS at 7 Days

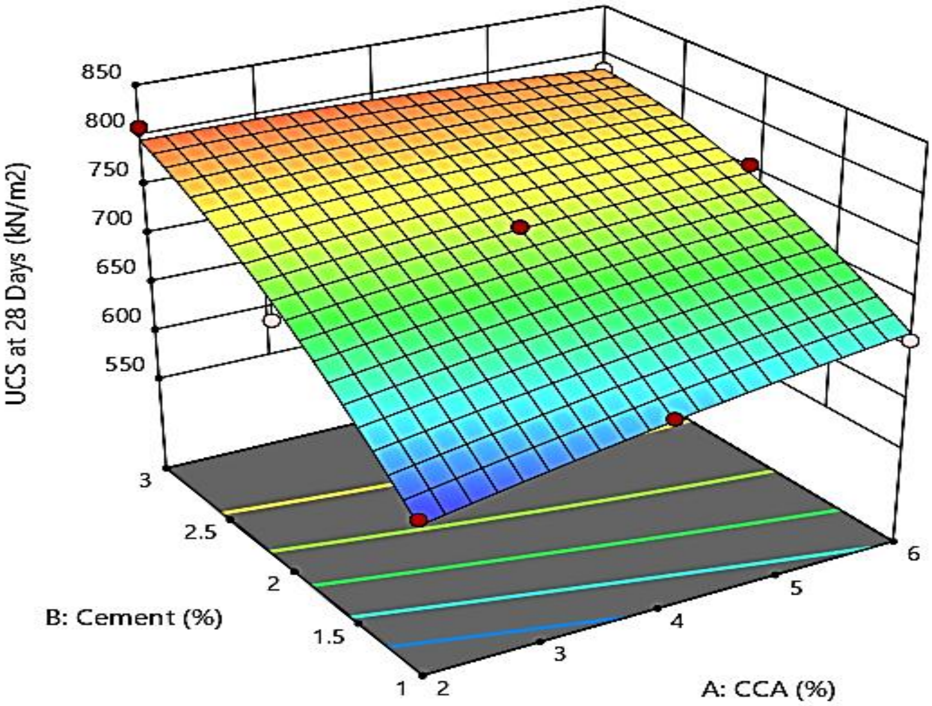


Figure 5. 3D Response Graph of Cement, CCA and UCS at 28 Days

Figure 4 shows 3D response surface graph of interactions between CCA and Cement and the influence on UCS at 7 days. The 3-D surface elucidates the correlation between the dependent variables (responses) and the independent variables (factors). The Figure 4 shows that increase in CCA and Cement cause significant increase in value of UCS at 7 days. Figure 5 shows 3D response surface graph of interactions between CCA and Cement and the influence on UCS at 28 days. The Figure 7 shows that increase in CCA and Cement caused slight increase in value of UCS at 28 days but reduction as the percentage of CCA increased.

4.7. Optimization by Numerical Method

The ramp plot graph showing optimized factors and responses are shown in Figure 6.

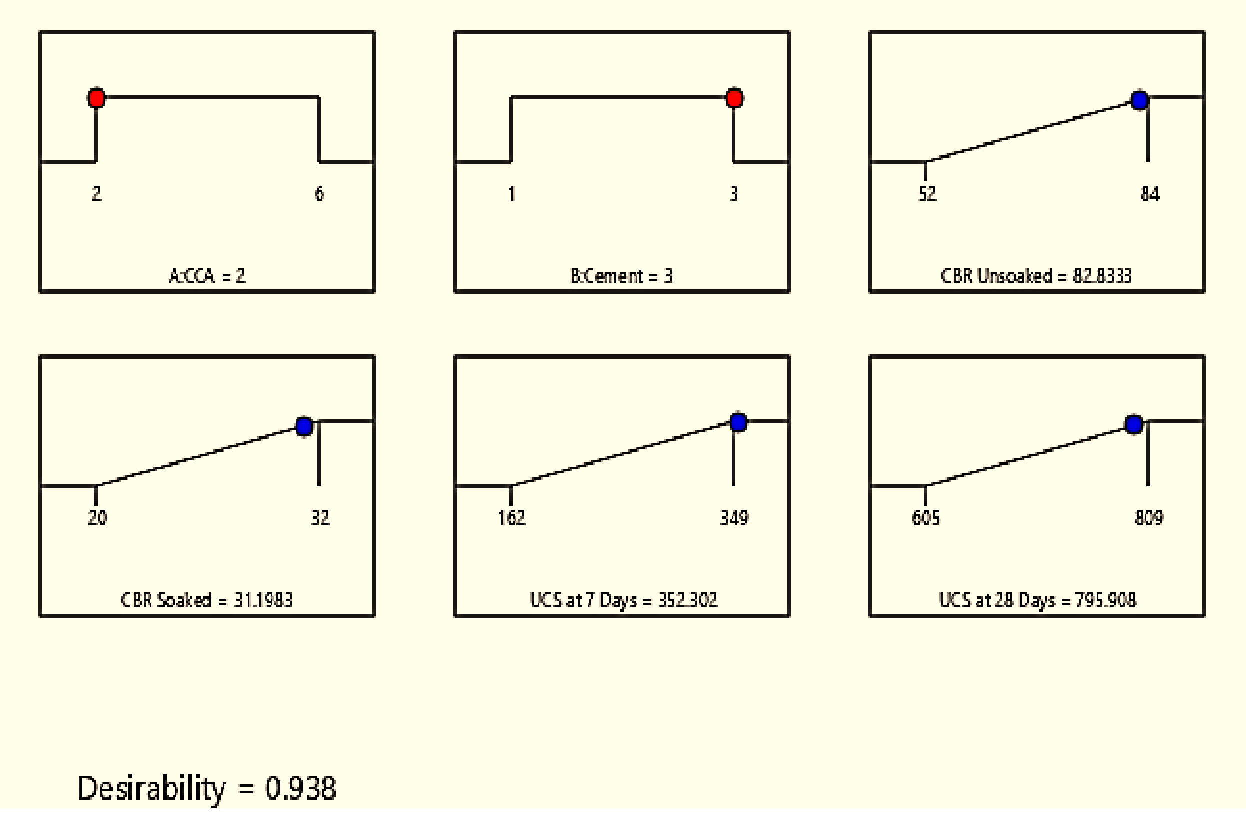


Figure 6: Ramp Plot Showing the Optimal Values for Factors and Responses

In Figure 6, the ramp plot shows the result of numerical optimization of the UCS and CBR properties of the stabilized soil with cement and CCA. The result revealed the best mix of CCA and cement in stabilizing laterite soil as follows; 2% CCA, 3% Cement, 82.9% CBR Unsoaked, 31.2% CBR Soaked, 352 N/mm^2 UCS at 7 days, 357 N/mm^2 UCS at 7+7 days, 419.1 N/mm^2 UCS at 14 days and 795.9 N/mm^2 UCS at 28 days. The result of CBR soaked and unsoaked, UCS at 28 days all conform to Nigeria General Specification (2016).

4.8. Durability Test

Durability tests are essentially in soil stabilization to ensure that the stabilized soil maintain its strength and stability over a long period of time, even under adverse environmental condition (Shivaramaiah et al., 2020). The durability result is presented in Table 15.

Table 15: Durability Test of Stabilized Soil

Run	Factor A: CCA %	Factor B: Cement %	UCS at 7 + 7 Days kN/m^2	UCS at 14 Days kN/m^2	Durability CCA %
1	2	3	362	422	85.8
2	2	1	251	340	73.8
3	4	3	350	420	83.3
4	4	2	335	380	88.2
5	4	2	335	380	88.2
6	6	3	340	410	82.9
7	6	1	293	355	82.5
8	4	2	335	380	88.2
9	2	2	310	370	83.8
10	4	1	262	345	75.9
11	6	2	345	390	88.5

12	4	2	335	380	88.2
13	4	2	335	380	88.2

The maximum durability observed from the UCS of soil with cement and CCA was 88.2% when 4% CCA and 2% cement was used. The resistance to loss should not be more than 20 % which is the maximum allowable durability index according to (Obeta et al., 2019). The result showed that only two replacements (2% CCA and 1% cement) and (4% CCA and 1 cement) were below the allowable durability index.

5. Conclusion

From the research work carried out, the following conclusions were drawn:

- The natural soil has an average value of the liquid limit, plastic limit and plasticity index as 22.7, 6.5 and 16.2 respectively, while the linear shrinkage is 11%, hence the soil was classified as A-6 in accordance with the American Association of State Highway and Transportation Officials (AASHTO M 145-2012) soil classification system and CL in accordance with the Unified Soil Classification System (ASTM D 2487-2011).
- The results obtained from the XRF showed that the main oxide compositions of the soil used for the research comprised of Al2O3 –22.7 %, SiO2 – 55.9 %, Fe2O3 – 14.3 %, and CaO- 0.42 %. The Silica Sesquioxide (SS) ratio of iron and aluminum is 1.51 falls within the typical range for laterite soils. The oxide content of corn cob ash shows silica oxide at 62.8%, alumina oxide at 5.36% and Ferric oxide at 2.72%. The addition of these oxides was 71.9% which satisfies ASTM C 618 and the CCA can be classified as a pozzolan.
- The compaction properties of the admixed soil with cement and CCA was achieved at 97% natural soil, 2% corn cob ash and 1% cement.
- Optimization of the UCS and CBR properties of the stabilized soil with cement and CCA were conducted and the result revealed the best mix of CCA and cement in stabilizing laterite soil as follows; 2% CCA, 3% Cement, 82.9% CBR Unsoaked, 31.2% CBR Soaked, 352 N/mm2 UCS at 7 days, 357 N/mm2 UCS at 7+7 days, 419.1 N/mm^2 UCS at 14 days and 795.9 N/mm^2 UCS at 28 days.
- Regression analysis models for the prediction of CBR (soaked and unsoaked) and UCS (7 days and 28 days) of the stabilized soil with cement and corn cob ash were all developed.

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