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Characteristics and stabilization of Mbiama soil for sustainable road construction in Niger Delta area, Nigeria

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Abstract

This study investigates the characterization and stabilization of Mbiama soil with a view to its application in sustainable road construction. Laboratory analyses were conducted to determine the grain size distribution and Atterberg limits of the soil. In addition, the maximum dry density and corresponding optimum moisture content were established. The soil's activity and modified free swell index were also evaluated, alongside its penetration resistance using the California Bearing Ratio (CBR) test. To enhance the engineering performance of the soil, cement and calcium chloride were employed as stabilizing agents. Different proportions of calcium chloride were applied to control swelling behavior. For improvements in compaction characteristics and strength, a fixed calcium chloride content of 8% was combined with varying percentages of cement. The results indicate that soil swelling becomes negligible at calcium chloride contents of 8% and above. Furthermore, at this salt concentration, the soil exhibits behavior similar to fine sand when the cement content exceeds 5%. A significant increase in penetration resistance was also observed at cement contents of 5% and above in combination with 8% calcium chloride. Under these conditions, a CBR value of 40.63 was obtained, indicating that the treated soil is suitable for use as a subbase material in road construction.

Keywords: *Expansive soil stabilization, Mbiama soil, Calcium chloride, Cement stabilization, California Bearing Ratio (CBR), Swell control, Sustainable road construction.*

1.0 INTRODUCTION

The importance of soil in engineering applications cannot be overstated, as it constitutes the fundamental material upon which most civil engineering works are executed. Consequently, geotechnical engineers, alongside architectural engineers and geologists, frequently encounter significant challenges when structures are founded on problematic soil formations, particularly expansive soils.

Expansive soils are those that undergo considerable volume changes in response to variations in moisture content—swelling when wet and shrinking upon drying [1]. These soils are typically rich in clay minerals, especially montmorillonite (a smectite mineral), which exhibits pronounced shrink–swell behaviour. Other clay minerals such as kaolinite, illite, mica, vermiculite, and chlorite are also present in soils but generally exhibit less severe effects on engineering structures. Expansive soils commonly originate from the weathering of fine-grained extrusive igneous rocks and montmorillonite-rich sedimentary formations such as shales and mudstones [2], [3], US[4].

Their occurrence in southeastern Nigeria has been linked to the weathering of pyroclastic rocks in the Abakaliki area and shaly formations within the Asu River Group, Ezeaku, Awgu, Mkporo, Mamu, Nsukka Formation, and Imo Shale [5], [6]. These studies further indicate that the proportion and characteristics of montmorillonite in such soils are strongly influenced by the nature of the parent rock under humid tropical conditions.

The engineering implications of expansive soils are global in scale, as they are responsible for widespread damage to infrastructure, including highways, buildings, underground utilities, embankments, and hydraulic structures. According to Kerran [7], the United States Department of Housing and Urban Development (HUD) estimated losses of approximately $9 billion due to expansive soils as far back as 1981. Similarly, Jones and Holtz [8] reported that the annual damage caused by expansive soils in the United States exceeds the combined effects of floods, hurricanes, earthquakes, and tornadoes, while [9] estimated the annual cost to surpass $10 billion. These soils induce structural distress primarily through cyclic swelling and shrinkage, which leads to cracking, deformation, and eventual failure of engineering works [10], [11]. Their high water absorption capacity, attributed to montmorillonite content, further exacerbates these effects [12].

Where removal of unsuitable soil is impractical, improvement of its engineering properties through appropriate ground treatment methods becomes necessary. Craig [13] emphasizes that soil stabilization provides a viable alternative in such cases. Methods of treatment include restricting moisture ingress, replacing the problematic soil, or enhancing its strength through mechanical or chemical stabilization techniques ([14], [15]).

In the study area, the presence of expansive soils has resulted in visible structural failures, including cracks and distortions in roads, buildings, and drainage systems, as illustrated in Figures 1 to 3. These defects significantly reduce the durability and service life of such structures. Expansive soils are particularly unsuitable for construction due to their susceptibility to swelling, shrinkage, and strength reduction ([12]).

Whitlow [17] classifies soils based on plasticity, with liquid limits less than 35% indicating low plasticity, 50–70% high plasticity, 70–90% very high plasticity, and values above 90% representing extremely high plasticity soils. Swelling potential is commonly assessed using parameters such as plasticity index (PI), liquid limit (LL), and clay activity, while expansion characteristics are evaluated using linear shrinkage and free swell indices.

The soil encountered in this study is over-consolidated and contains a significant proportion of expansive clay minerals, predominantly montmorillonite. It is typically dark grey to reddish-grey in colour. Located within the coastal belt of the Niger Delta, the study area experiences a tropical climate with distinct wet and dry seasons. During the rainy season, water absorption by the expansive clay minerals leads to substantial volumetric expansion. Numerous lightweight masonry structures in the region have suffered damage due to differential heave. However, structural failures cannot be attributed solely to soil conditions; other contributing factors include inadequate design, poor-quality construction materials, and insufficient supervision during construction.

Observed structural damage varies in severity, ranging from minor hairline cracks to extensive cracking and, in extreme cases, complete structural collapse. The pattern of cracking is influenced by moisture movement within the soil. A dome-shaped heave occurs when moisture migrates from the edges toward the centre of a structure, whereas a dish-shaped (saucer) heave results from moisture movement from the centre toward the perimeter, as illustrated in Figures 1.2 and 1.3.

In addition to soil conditions, defects may arise from poor structural design and substandard construction materials. Materials used in construction vary widely in quality, and brittle materials such as unreinforced concrete and masonry are particularly susceptible to damage caused by soil heave.

Skempton [18] classified clays based on their activity index, defined as the ratio of plasticity index to clay fraction. According to this classification, clays are categorized as inactive (activity < 0.75), normal (0.75–1.25), or active (> 1.25). The free swell index has long been used to evaluate the expansiveness of fine-grained soils ([19], [20]); however, it may yield unreliable (negative) values for soils rich in kaolinite [21]. To address this limitation, the modified free swell index method was developed ([22]), and its classification system is presented in Table 1 [23].

Hooper and Marr [24] provided guidelines for the use of soils in road construction based on California Bearing Ratio (CBR) values. They recommended CBR ranges of 5–20 for subgrade materials, 20–50 for subbase layers, and 50–80 or higher for base courses, as summarized in Table 2.

The primary aim of this study is to characterize and stabilize Mbiama soil for its application in sustainable road construction. To achieve this, the following specific objectives are defined:

- To determine the physical properties and Atterberg limits of the natural soil
- To evaluate the activity, swelling potential, and CBR of the natural soil
- To assess the activity, swelling potential, and CBR of the stabilized soil

Figure 1: Building abandoned at Mbiama because to cracks

Figure 2: Failed portion of road at Mbiama

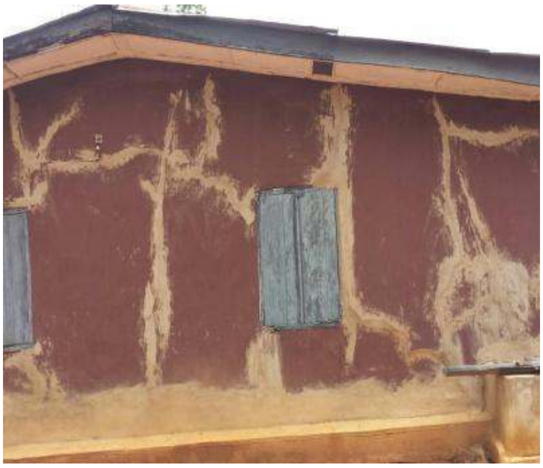

Figure 3: Patched cracks on the wall due to Expansive soil at Mbiama

2.0 Methodology

Soil samples used in this study were obtained from Mbiama town in Bayelsa State, Nigeria. The sampling location lies approximately at latitude 5°04'08.1" N and longitude 6°26'15.3" E. The sample was collected at a depth of 1200 mm below the ground surface to ensure representative subsoil conditions.

Laboratory characterization of the soil involved a series of standard geotechnical tests. Grain size distribution was determined using sieve analysis in accordance with Clause 9.3 of [25], complemented by sedimentation analysis (hydrometer method) following Clause 9.5 of [25].

Atterberg limits (liquid and plastic limits) were determined in line with Clause 4 of [25], while compaction characteristics were obtained using the Proctor compaction test as specified in Clause 3.4 of [26]. The swelling behaviour of the soil was evaluated using the free swell test procedure [19], and its strength properties were assessed using the California Bearing Ratio (CBR) test in accordance with Clause 7 of [26].

2.1 Determination of Physical Properties and Atterberg Limits of Natural Soil

For the grain size distribution by dry sieving, approximately 300 g of air-dried soil was passed through a stack of sieves ranging from 1.18 mm to 0.075 mm using a mechanical sieve shaker. The mass of soil retained on each sieve was measured and recorded. The fraction passing the 0.075 mm sieve was subjected to sedimentation analysis.

For the hydrometer test, 50 g of the fine soil fraction was dispersed in 125 ml of a 40 g/L sodium hexametaphosphate solution. The mixture was thoroughly stirred and allowed to stand for 10 minutes. Subsequently, an additional 75 ml of water was added, and the suspension was mixed for 5 minutes before being transferred into a 1000 ml sedimentation cylinder. Water was added to bring the total volume to 800 ml. The cylinder was then sealed and agitated by inversion 30 times within approximately 70 seconds to ensure uniform dispersion.

Immediately after agitation, a hydrometer was inserted into the suspension, and readings at the upper meniscus, along with corresponding temperature values, were recorded at intervals of 30, 60, 120, 180, and 240 seconds. The hydrometer was then removed, rinsed, and calibration readings (including meniscus correction, zero correction, and temperature) were obtained using a control cylinder containing only dispersing solution and water. The hydrometer was reinserted into the soil suspension, and additional readings were taken at extended time intervals of 8 minutes, 15 minutes, and 0.5, 1, 2, 4, 8, 16, 24, 48, and 96 hours.

The liquid limit test was performed by first air-drying and pulverizing the soil, followed by sieving through a 0.40 mm sieve. The passing fraction was divided into three portions of approximately 100 g each and mixed with varying amounts of distilled water. The samples were covered and allowed to equilibrate for 24 hours. Each sample was then remoulded to achieve uniform consistency and placed in the Casagrande apparatus. A groove was cut using the standard grooving tool, and the number of blows required to close the groove over a length of 12.5 mm was recorded.

Corresponding moisture contents were determined, and a flow curve was plotted to obtain the liquid limit at 25 blows.

For the plastic limit determination, about 25 g of soil from the liquid limit test (preferably the drier portion) was used. The sample was mixed with small amounts of water and kneaded until it exhibited slight cracking. It was then divided into two portions, each further subdivided into smaller specimens. These were rolled by hand on a glass plate into threads of approximately 3 mm diameter. The process was repeated until the threads crumbled uniformly at this diameter. The moisture content of the crumbled soil was then determined.

In the Proctor compaction test, approximately 3 kg of air-dried soil passing the 4.75 mm sieve was used. Water was added incrementally (starting with about 300 g), and the mixture was compacted in a 105 mm mould in three layers, each receiving 25 blows from a 2.5 kg hammer dropped from a height of 300 mm. After compaction, the mould was weighed to determine bulk density. The procedure was repeated with increasing water content until the dry density began to decrease. A plot of dry density versus moisture content was used to determine the maximum dry density (MDD) and optimum moisture content (OMC). CBR tests were conducted on similarly prepared samples in accordance with [26].

2.2 Determination of Activity, Swell Potential, and CBR of Natural Soil

The swell potential of the soil was evaluated using the modified free swell index test. Two samples of 10 g each, previously oven-dried, pulverized, and passed through a 425 µm sieve, were placed in separate graduated cylinders containing 100 ml of distilled water. Air bubbles were removed by gentle agitation and stirring with glass rods. The samples were then allowed to settle undisturbed for 24 hours, after which the final volumes were recorded.

The modified free swell index (SI) was calculated as:

$$SI = \frac{V - V_s}{V_s} \quad (1)$$

where V is the volume of the swelled soil and V_s is the volume of the solid particles. The volume of solids is given by:

$$V_s = \frac{w_s}{G_s \gamma_w} \quad (2)$$

where W_s is the weight of soil solids, G_s is the specific gravity, and $_w$ is the unit weight of water.

The California Bearing Ratio (CBR) test was also conducted on the natural soil to evaluate its load-bearing capacity.

2.3 Determination of Activity, Swell Potential, and CBR of Stabilized Soil

Soil stabilization was carried out using cement and calcium chloride ($CaCl_2$) as stabilizing agents. These additives were introduced at varying proportions to assess their effects on soil properties.

For the free swell tests, only calcium chloride was used at concentrations of 1%, 3%, 5%, 8%, and 12% by weight of soil, without the addition of cement. For compaction and CBR tests, a fixed calcium chloride content of 8% was adopted, while cement was varied at 0%, 2%, 5%, 8%, 12%, and 15%.

All tests conducted on the natural soil, as described in Section 2.2, were repeated for the stabilized soil samples to evaluate the changes in activity, swelling potential, and strength characteristics.

3.0 RESULTS AND DISCUSSION

3.1 Results Presentation

The measured physical properties of the soil are summarized in Table 3.

The grain size distribution curve is presented in Figure 4, while the results of the Atterberg limits tests are illustrated in Figure 5. Compaction characteristics for cement contents below 8% are shown in Figure 6, whereas Figure 7 presents the results for cement contents of 8% and above. The corresponding values of maximum dry density (MDD) and optimum moisture content (OMC) are listed in Table 4. The California Bearing Ratio (CBR) test results are displayed in Figure 8. In addition, the modified free swell index (MFSI) values are presented in Table 5, with Figure 9 illustrating the relationship used to model swell behaviour.

3.2 Discussion of Results

The grain size analysis (Figure 4) indicates that the soil is predominantly clayey, with a clay fraction of approximately 31.5%. Based on the activity values presented in Table 3, the soil is classified as active, having an activity index greater than 1.25 [18]. The Atterberg limits results (Figure 5) show a liquid limit of 54.2% and a plastic limit of 10.6%, confirming the high plasticity nature of the soil.

The modified free swell index of the natural soil is 12.23 (Table 5), indicating a high swelling potential [23]. This level of expansiveness renders the soil unsuitable for direct use in road construction in its natural state. However, the addition of calcium chloride ($CaCl_2$) significantly reduces this swelling behaviour. At a calcium chloride content of 8%, the MFSI decreases to 2.43, which falls within the negligible swelling range (<2.5) [23]. This demonstrates the effectiveness of $CaCl_2$ in controlling soil expansiveness.

Figure 9 further illustrates that the reduction in MFSI follows an exponential trend with increasing calcium chloride content. The relationship between MFSI and $CaCl_2$ content can be expressed as:

$$MFSI = 11.76e^{-0.159x} \quad (3)$$

where x represents the percentage of calcium chloride. This relationship indicates that incremental additions of $CaCl_2$ result in progressively smaller reductions in swell potential.

The compaction results (Figures 6 and 7) show a consistent increase in maximum dry density with increasing stabilizer content (cement combined with 8% $CaCl_2$). At lower cement contents (below 5%), the soil retains its clay-like characteristics. However, at cement contents of 5% and above, the soil exhibits behaviour more typical of granular materials such as sand. This transition can be attributed to the chemical reactions between the active clay minerals and calcium hydroxide released during cement hydration, leading to the formation of cementitious compounds (hydrates) that alter the soil structure.

From Table 4, the maximum dry density increases from 1.655 g/cm^3 at 0% cement to 1.84 g/cm^3 at 5% cement content. Up to this level, the soil shows improved compaction without any significant reduction in density with increasing moisture content, indicating enhanced structural stability.

The CBR results (Figure 8) reveal a near-linear increase in penetration resistance with increasing cement content at a constant $CaCl_2$ content of 8%. At 5% cement and 8% $CaCl_2$, a CBR value of 40.63 is obtained. According to established standards [24], this value falls within the range suitable for subbase materials in road construction. Therefore, the stabilized soil meets the required strength criteria for such applications.

From a practical standpoint, stabilizing Mbiama soil using 5% cement and 8% calcium chloride offers a cost-effective alternative to importing suitable construction materials from outside the area. However, for economic optimization, further investigation into the use of cement alone as a stabilizing agent is recommended. This approach may provide a more affordable solution while still improving both the swelling characteristics and mechanical properties of the soil.

Table 3: Index and Classification Properties of the Natural Soil

Natural Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Clay Content (%)	Activity	USCS Classification	AASHTO Classification
23.2	54.2	10.6	43.6	30	1.453	CH	A-7

Table 4: Compaction Characteristics of Soil at Different Stabilizer Combinations

Cement (%)	$CaCl_2$ (%)	Maximum Dry Density (g/cm^3)	Optimum Moisture Content (%)
0	0	1.655	21.2
0	8	1.728	19.8
2	8	1.795	18.6
5	8	1.84	21.1

Table 5: Modified free swell index result of the soil

$CaCl_2$ (%)	G_s	V_s (cm^3)	V_1 (cm^3)	V_2 (cm^3)	$V = \frac{(V_1+V_2)}{2}$ (cm^3)	$MFSI = \frac{(V - V_s)}{V_s}$	Swelling Potential ([21])
0	2.69	3.72	49.2	49.2	49.20	12.23	High
1	2.67	3.75	43.6	41.6	42.60	10.37	High
3	2.62	3.82	35.9	33.1	34.50	8.04	Moderate
5	2.57	3.89	23.5	22.6	23.05	4.92	Moderate
8	2.5	4.00	13.8	13.6	13.70	2.43	Negligible
12	2.41	4.15	13.2	12.8	13.00	2.13	Negligible
γ_w = 10 g/cm^3; W_s = 10g							

Table 5: Modified Free Swell Index (MFSI) of Soil at Varying Calcium Chloride Contents

$CaCl_2$ (%)	Specific Gravity (Gs)	Volume of Solids, Vs (cm^3)	V_1 (cm^3)	V_2 (cm^3)	Average Volume, V (cm^3)	MFSI = (V – Vs)/Vs	Swelling Potential
0	2.69	3.72	49.2	49.2	49.2	12.23	High
1	2.67	3.75	43.6	41.6	42.6	10.37	High
3	2.62	3.82	35.9	33.1	34.5	8.04	Moderate
5	2.57	3.89	23.5	22.6	23.05	4.92	Moderate
8	2.5	4	13.8	13.6	13.7	2.43	Negligible
12	2.41	4.15	13.2	12.8	13	2.13	Negligible

Note: *γw = 10 g/cm^3, W_s = 10 g*

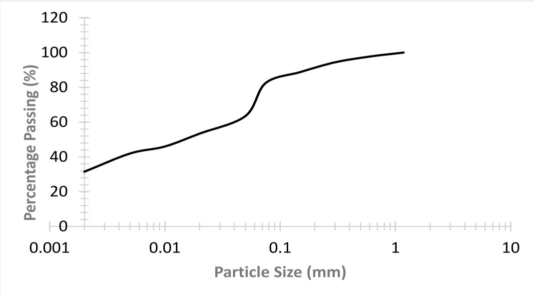

Figure 4: The result of grain size distribution test of the soil

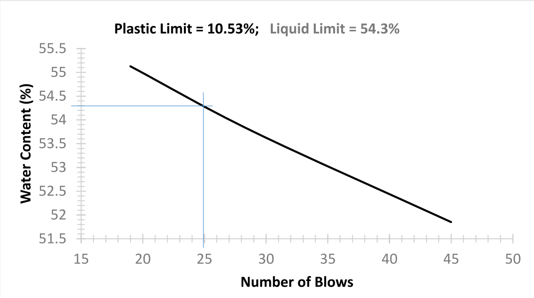

Figure 5: Result of Atterberg limit test of the soil

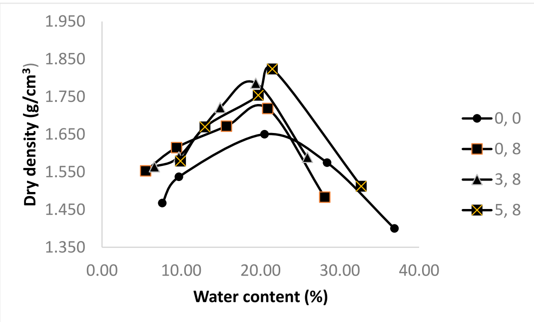

Figure 6: Dry density versus water content for various stabilizing agents contents (cement, $CaCl_2$: 0,0; 0,8; 2,8; 5,8)

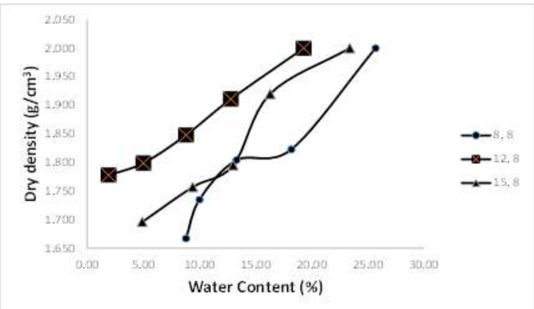

Figure 7: Dry density versus water content for various stabilizing agents contents (Cement, $CaCl_2$: 8,8; 12,8; 15,8)

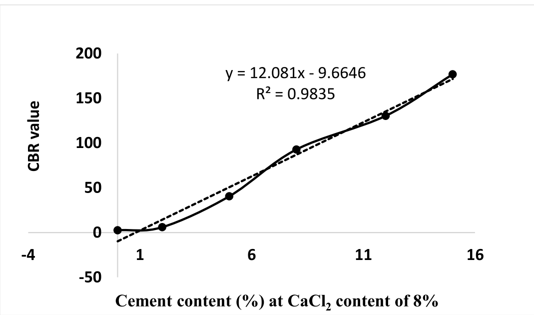

Figure 8: CBR value versus stabilizing agents contents (Cement, $CaCl_2$: 0,8; 2,8; 5,8; 8,8; 12,8; 15,8)

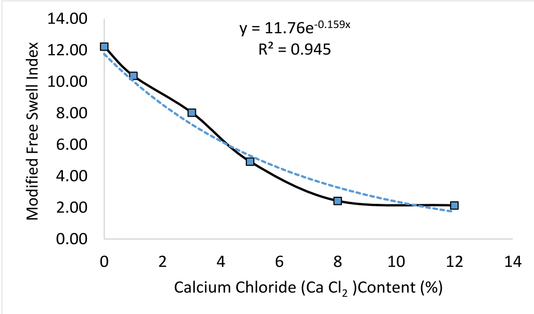

Figure 9: Modified free swell index versus Calcium Chloride contents

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