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COUPLED EFFECT OF HYDRATED LIME AND GLASS WASTE FINE STABILIZERS ON THE MECHANICAL PROPERTIES OF LATERITIC SOIL

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Abstract

This study has investigated the coupled effect of hydrated lime (L) and glass-waste fine (GF) stabilizers on the mechanical properties of two different lateritic soil samples (specimens A and B). The stabilizing materials L and GF were mixed together ("LGF") at equal percentages of 2%, 4%, and 6% of the mass of the soil samples and thereafter used as the soil stabilizer. Compaction tests, the California Bearing Ratio (CBR) test, and the Unconfined Compressive Strength (UCS) test were conducted to determine the coupled effect of the stabilizing materials. The results obtained show that the maximum dry density (MDD) for the stabilized soils is 1.63kg/m³ at 2% LGF and 1.70 kg/m³ at 4% LGF versus 1.54kg/m³ and 1.61kg/m³ for the untreated soil for both samples, respectively. For the CBR, it increases to 62% at 4% LGF and 72% at 6% LGF, while the UCS value after curing for 24 hours increases by 30.60% and 40.63% for samples A and B, respectively.

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Key words

- Glass Waste Fine,
- Hydrated Lime,
- Compaction,
- CBR,
- UCS.

1 INTRODUCTION

In Nigeria, laterite soil is found in almost every part of the country; it is usually used as a road construction material. Most of the time this soil possesses some properties that are detrimental to the civil engineering infrastructures placed on it, therefore making it problematic when encountered or used in civil engineering works. The presence of clay minerals such as kaolinite, illicite and montmorillonites in laterite soil has been attributed to the poor strength and durability performance shown by the structures placed on it when used as a construction material, especially in the presence of moisture (Puppala and Pedarla, 2017). Laterite soil behaves similarly to expansive soil; in the presence of clay minerals, i.e., it swells when it absorbs water and shrinks when dry, thus affecting the soil's stability. Therefore, as a result, geotechnical engineers have sought better ways to improve the properties of this type of soil through soil stabilization (Konkol et al., 2019; and Lin et al., 2022). Abdeldjouad et al. (2019) define soil stabilization as the physical or chemical alteration of the natural properties of soil to meet engineering needs. Similarly, Ayininuola and Oladotun (2016) define it as the process of using cementitious materials to improve the engineering properties of the soil.

Several attempts towards stabilizing lateritic soil with poor engineering properties by using materials with cementitious properties have been attempted by various researchers. For instance, Wang et al. (2017) uses jute fibers to improve engineering soil; Yang et al. (2020) reinforce the soil with waste rubber tyres; Abdeldjouad et al. (2019) used agricultural waste base geopolymer as a stabilizer, while Al-Swaidani et al. (2016); and Khemissa and Mahamedi (2014) used lime and cement to stabilize weak soil. However, among the various soil stabilizers used by researchers, cement and/or lime have not only been the most recommended or widely used soil stabilizers but have also been reported to improve the properties of stabilized soil better through cation exchange, flocculation, and a pozzolanic reaction mechanism when used alone or in combination with another source binder (Rao and Rajasekaran, 1996). For example, when Ikeaguwuani et al. (2019) stabilized subgrade soil with sawdust ash (SDA) and lime, an improvement in the strength properties of the stabilized soil was reported with the addition of 4% lime and 16% SDA. Also, when the geotechnical properties of lateritic soil were improved with metakaolin and lime as cement substitutes by Amusan et al. (2024), a significant improvement in the plasticity index of the stabilized soil was observed. A decrease in the maximum dry density (MDD) of the stabilized soil was also reported as the percentage of the mixture of lime and metakaolin increased. Similarly, when the evolution of the strength in lime-stabilized rammed soil was optimized (Baldovino et al. 2019), the addition of 9% and 11% lime to both stabilized soil samples improved the unconfined compressive strength (UCS) respectively.

However, with a recent increase in the price of lime and cement, as well as the attribution by various scholars of the cement industry as one of the major contributors to climate change causing global warming due to the depletion of the ozone layers, alternative soil stabilizers if available or a reduction in the quantity of lime and/or cement used has become necessary. Alrubaye et al. (2022) reported that about 5% of the global carbon dioxide (CO_2) emissions released into the environment is due to the utilization of fossil fuel in the production of cement and/or lime. Aside from the high cost and emissions of CO_2, the use of lime and/or cement also produces other harmful oxides such as sulphur and dust, which affect the quality of the air and also have a deleterious impact on the environment. Similarly, Higgins (2007) opined that the extraction of limestone used in the production of lime and/or cement is causing the depletion of natural resources, while a large amount of electricity (about 7500 J/ton) and heat are consumed in their production. Therefore, a reduction in the volume of cement and/or lime in any geotechnical engineering works through partial replacement with more sustainable materials is becoming more important.

In recent times, human activities have continued to generate high volumes of solid waste that have become problematic to handle, due to a lack of proper waste management. This waste has not only caused environmental nuisances but have also served as a potential source of health-related issues. According to Attom et al. (2019), industrial waste, agricultural waste, vehicular tires, and hospital waste, as well as household-generated wastes such as glass, plastic, and metallic waste are major sources of commonly generated solid waste. Therefore, the effective utilization of these solid wastes by engineers has become necessary to achieve a sustainable environment. Glass, which is one of the major non-biodegradable wastes produced worldwide, is often used in construction industries as various building components as well as in other industries to make millions of packaging bottles, and, as the use of glass continues to increase, so does the volume of glass waste generated. According to the United Nations, about 200 million tonnes per volume of solid waste are disposed yearly, with glass waste amounting to about 7% of the total volume (TobÇu and Canbaz, 2004). Similarly, Ganiron (2014) reported that glass makes up 7% of the total weight of the municipal solid waste disposal of the United State annually, with only 20% recycled, while the remaining 80% are left unused. This therefore makes the reuse of glass waste fines essential in order to achieve the sustainable development goals (SDG) of circular economies and smart cities.

Although numerous studies on the use of solid glass waste as a partial replacement of lime in soil stabilization have been conducted by many researchers, only a few of the studies conducted have reported their effectiveness on the mechanical properties of soil. Also, results from past studies have continually reported varying combinations of the mixture of glass waste fines and lime to attain maximum strength with stabilized soil. However, with this remarkable improvement in the partial replacement of lime with glass waste fine materials, the volume of lime materials that is still in use when combined together as a soil stabilizer is still often compared to glass waste fines. Therefore, this research work tends not only to improve the mechanical properties of lateritic soil, but also to obtain a maximum proportion of L and GF when combined to stabilize soil. In trying to ascertain the equal percentages of L and GF that can be used as a soil stabilizer to obtain the maximum strength of the stabilized soil, this will create an opportunity for further study to incorporate more sustainable cementitious materials that will continuously lower the percentage of lime when mixed with GF.

2 MATERIALS AND METHODS

2.1 Materials

2.1.1 Soil Samples

The soil samples used in this study were collected at two different locations, namely, Site A: the Akinyele local government area, Ibadan, and Site B: the Faculty of Technology, University of Ibadan, Ibadan, Nigeria. Disturbed soil samples were collected at a depth of 1.5m below the ground's surface from a trial pit after removing the topmost soil. They were thereafter stored in sacks and then appropriately labelled and transported to the laboratory for a geotechnical analysis. Also, representative samples of about 30g of both samples A and B were collected and stored separately in a known weight moisture content can on site and then oven-dried at a temperature of 1050 C in the laboratory for 24 hours to determine the natural moisture content (NMC) of both soil samples. The transported soil samples were separately air-dried for 24 hours, pulverized and then sieved through a standard 2 mm sieve before been used for the test. This was in accordance with the standard procedures of ASTM E 1727 (2020), FMWH (1997), and BS 1377 (2022).

The decision to select two soil samples for this study was taken to investigate the effect of the different clay contents on the soil-lime-glass waste fine reaction process. An X-ray fluorescence analysis (XRF) and an X-ray diffraction (XRD) analysis were conducted on both samples to assess the chemical and mineralogical components of the soils. Table 1 presents the physical properties of samples A and B, while the chemical and mineral compositions of the soil samples are shown respectively in Table 2.

From the physical properties in Table 1, both soil samples can be seen to be slightly acidic as the pH values for sample A and B fall between the pH value range of 6.10 to 6.50 respectively. Also, based on ASTM D854 (2023), both soils can be seen to be clay-to-silty clay soil as the specific gravity of both soil samples ranges between 2.67 to 2.9. Similarly, Table 2 shows the percentage of the silica (SiO_2), iron (Fe_2O_3) and alumina (Al_2O_3) oxides present in both samples. Both soil samples A and B contain these oxide compositions in higher amounts with sample A having a total of 86.96% and sample B having 90.45% respectively. On the contrary, the mineral composition of both soil samples from the XRD analysis shows variations in the amount of the minerals present in the soil samples. Sample A has a higher amount of kaolinite minerals of about 42% and a trace of about 7% montmorillonite minerals, while sample B has no montmorillonite minerals, and the quantity of kaolinite content is only 20%. Although the illite quantity in sample B is much higher compared to sample A, there are no significant differences in the quantity of other minerals present for both samples A and B.

Tab. 1 *Physical Properties of soil samples A and B*

Properties	Sample A	Sample B
Colour	Reddish brown	Light brown
pH	6.42	6.05
Specific gravity	2.82	2.70
NMC (%)	25.8	16.2

Tab. 2 *Chemical and mineral composition of soil samples A and B*

Chemical properties	Sample A (%)	Sample B (%)
Silica (SiO_2)	54.50	60.24
Alumina (Al_2O_3)	26.82	18.55
Iron Oxide (Fe_2O_3)	5.64	11.66
Calcium oxide (CaO)	1.00	0.30
Magnesia (MgO)	0.80	1.20
Potash (K_2O)	11.24	8.05
Mineral composition (XRD Analysis)	**Sample A (%)**	**Sample B (%)**
Illite	16	38
Kaolinite	42	20
Quartz	22	25
Feldspar	11	10
Montmorillonite	07	-
Calcite	04	06

2.1.2 Glass Waste Fines

The broken glass bottle waste used in this study was collected from a drink distributor outlet at the Bodija market in Ibadan. The broken bottles were carefully rinsed to remove any dirty materials and impurities attached to the bottles; thereafter, they were sun-dried, manually crushed into pieces, and then milled by a locally fabricated milling machine into a powder form. The glass fines obtained were then sieved though a standard 425µm aperture sieve to increase their surface area for better reactivity. The sieved glass fines obtained were then used in the stabilization of the soil samples coupled with the hydrated lime. The XRF analysis of the glass waste fines is presented in Table 3.

Tab. 3 *XRF Analysis of the glass waste fines*

Compound	Weight, %
SiO_2	60.1
CaO	14.2
Fe_2O_3	7.0
MgO	6.3
Al_2O_3	11.6
Other oxides	0.8

The XRF analysis indicates that the silica, iron and alumina content in the GF are 60.1%, 7.0% and 11.6%, respectively. Nath et al. (2018) classified any materials containing about 70% silicates (SiO_2), aluminates (Al_2O_3), and oxides of a certain metal as pozzolanic material. Similarly, Walczak et al. (2015) opined that glass waste can also be used as a substitute for sand as a result of its high SiO_2 compounds; the SiO_2 compounds can also significantly increase the compressive strength. In addition to the use of glass waste as a substitute for sand, its utilization as a soil stabilizer can help reduce unused glass waste (Rahim et al., 2015).

2.1.3 Lime

The lime used in this study is a hydrated lime, which is also referred to as calcium hydroxide ($Ca(OH)_2$), slaked lime or calcium hydrate; it was manufactured by Buxton Lime Co., UK, and purchased locally from an accredited chemical store in Ibadan, Nigeria. Hydrated lime was selected because of its ability to neutralize the presence of any acidic substances present in the soil that may affect the reaction process of the soil-lime-glass waste fine due to its strong alkaline nature. Also, when hydrated lime is mixed with water, it is slightly soluble and forms a strong alkaline solution, thereby aiding the stabilization reaction process. Hydrated lime also can react with CO_2 to form calcium carbonate ($CaCO_3$). The basic physical and chemical properties of the hydrated lime are presented in Table 4 (Buxton (2014)), while the chemical composition of the hydrated lime according to Dang et al. (2016) is presented in Table 5. Based on the chemical composition, the presence of a high percentage of CaO lime constituent can be attributed because of its effectiveness as a cementitious material (Nath et al. 2018).

Tab. 4 *Physical and chemical properties of the hydrated lime (Buxton 2014)*

Properties	Value
Colour	White or off-white (beige) fine powder
Odour	Odourless
pH	12.4
Melting point	> 450^0 C
Relative density	2.24
Solubility in water	1844.9 mg/L
Explosive limits	Non-explosive

Tab. 5 *Chemical composition of hydrated lime (Dang et al. 2016)*

Properties	Weight, %
SiO_2	1.8
Al_2O_3	0.5
Fe_2O_3	0.6
CaO	72.0
MgO	1.0
CO_2	2.5
LOI	24

2.3 Methods

2.3.1 Preparation of the samples

The methods adopted in the preparation of the stabilized soil involved replacing each of the soil samples with a varying percentage of the stabilizing materials (L and GF); they were first mixed together in the same proportion of the dry weight of the soil samples and then manually mixed with both soil samples A and B in the absence of water. After a homogeneous mix was achieved for both mixtures, water at the Optimum Moisture Content (OMC) obtained for the unstabilized soil samples was then added to each mix proportion and then mixed thoroughly together again. The mixtures were then cured separately in a desiccator at room temperature for 24 hours. This was to allow reactions between each of the soil samples with the varying percentage of LGF respectively before the tests were conducted. A total of eight (8) samples of varying percentages of LGF stabilizers were prepared; these included the unstabilized soil samples A and B.

2.3.2 Sieve Analysis

To determine the grain size distribution of both soil samples A and B, the wet sieving method was adopted due to the high percentage of clay/silt content present in both soil samples. According to this method, 100g of each soil sample were weighed and washed separately under running water in a 0.075 mm sieve until there were no traces of clay or silt content left in the soil specimen. The samples were then oven dried at 105^oC for 24 hours. Thereafter, the oven-dried soil was then passed through a set of standard sieves stacked in decreasing order of sieve sizes using a mechanical sieve shaker. Then the samples retained in each sieve were weighed and recorded. This was in accordance with the recommendation of BS 1377 (2022).

2.3.3 Atterberg Limit Test

The difference between the LL, PL, and PI of the two lateritic soil samples before and after stabilization were ascertained using the Atterberg limits. To create a homogenous paste for the LL, 100 g of soil that had passed through a 0.075 mm sieve was thoroughly combined with an equal amount of lime and glass fines with water. After that, the final paste was put into the Casagrande apparatus cup, and the number of drops, or blows, needed to close the groove created in between the paste was noted. Each mixture's moisture content was then ascertained. For the PL, however, a portion of the paste made during LL was rolled out on a glass plate until a thread-like mixture with a diameter of about 3 mm was achieved. The moisture content at this point was also obtained by placing it in an oven at 105^oC for 24 hours. These activities were performed in accordance with ASTM D4318 (2000).

2.3.4 Compaction Test

In compliance with BS 1377-2 (2022), standard compaction tests were conducted on the air-dried soil samples, which were combined with different proportions of lime and glass waste fines at the optimal moisture content (OMC) derived from a compaction test of the unstabilized lateritic soil. In a 1000 cm^3 mold, each of the three layers was subjected to 27 blows from a 2.5 kg rammer. The sample's dry density and moisture content were measured; for each mixture, the maximum dry density (MDD) and optimal moisture content (OMC) were determined.

2.3.5 California Bearing Ratio (CBR) Test

The BS 1377: Part 4 (1990) method was used to determine the CBR, which is an index of soil strength and bearing capacity. After mixing and curing of the air-dried soil-lime-glass combination with the same amount of water as indicated by its OMC, it was put in the CBR mold in three layers, each of which was compacted using 63 blows from a 4.5 kg rammer. After determining the mass of the mold and the compacted soil-lime-glass mixture, the CBR machine was used with a seated load of roughly 2.5 kg at a penetration rate of 1.25 mm/min. The load's penetration was measured every 0.25 mm until failure occurred.

2.3.6 Unconfined Compressive Strength (UCS) Test

The Unconfined Compressive Strength (UCS) tests in this study were performed in accordance with ASTM D2166-06 (2010) and were conducted on LGF treated and untreated soil samples. The sample specimens used were prepared at the maximum dry density (MDD) and optimum moisture content (OMC) obtained from the compaction test. After the preparation of the samples, as previously explained, each of the soil samples mixed with varying percentages of LGF were placed in three (3) layers in a UCS split mould of a 38 mm size diameter and 76 mm height, with each layer receiving 14 blows using a tamping rod and then carefully removed and cured by sealing in a polythene nylon for 24 hours at room temperature before conducting the test on the UCS machine. During the test, a force-measuring device measured the compression force, and axial deformation gauges recorded the changes. The load applied to the specimen was recorded at intervals of 0.25 mm. Upon deformation, the test continued until either the maximum axial stress was reached or the axial strain reached 20%. Upon either of these instances, the tests were terminated.

3 RESULTS AND DISCUSSION

3.1 Presentation of the Results

Table 6 shows the results of the index properties and mechanical tests that were carried out on the unstabilized soil samples. The table shows that both samples were classified as ML soil in accordance with the USCS soil classification. This classification shows that both samples are fair to poor materials when used as subgrade materials; stabilization is therefore required to improve the mechanical properties of the samples.

Tab. 6 *Mechanical properties of the unstabilized lateritic soil samples*

Properties (Test)	Sample A	Sample B
Fine content, %	32	58
Silt content, %	28	17
Clay content, %	40	25
USCS Classification	Low Plasticity Silt (ML)	Low Plasticity Silt (ML)
AASHTO Classification	A-7-6	A-7-5
OMC (%)	17.3	14.5
MDD (g/cm3)	1.54	1.61
CBR (%)	32	35
UCS (kPa)	41.6	38.4

3.2 Grain Size Distribution

The sieve analysis results obtained for the unstabilized soil samples show the percentage passing Sieve No. 200 was 68% and 42% for samples A and B, respectively. These results did not satisfy the specification limits of 35% or less for a road as specified by FMWH (1997). It can be concluded that both soil samples are poor materials for sub-grade and sub-base courses. They showed that the samples had a higher percentage of silt and clay, which have expansive soil properties, i.e., they swell with the addition of water and shrink when water is dried.

3.3 Atterberg Limits

As previously mentioned, the liquid limits, plastic limit, and plasticity index of the soil samples with various stabilizer variations are included in the Atterberg limit results.

3.3.1 Liquid Limit (LL)

The results obtained from the LL analysis and presented in Table 7 and Fig. 1 respectively show a gradual increase in the liquid limit for soil sample A from 48% to 57% as the percentage of LGF increases from 0% to 6%. Nath et al. (2018) show similar trends, noting that these beneficial changes in the engineering properties are mainly attributed to cation exchange, clay flocculation, agglomeration, and pozzolanic reactions between the stabilizing materials (LGF) and the soil samples. Similar trends were also observed when saw dust ash, rice husk ash, and cement are used as the stabilizing materials (Ogunribido, 2012; Sarkar et al. (2012a and b)).

For sample B, an initial increase in LL was observed at 2% LGF and a further increase in the LGF above 2% caused a reduction in the LL below the unstabilized soil value. The initial increase in the LL of sample B could also be attributed to cation exchange, clay flocculation, agglomeration, and pozzolanic reactions between the stabilizing materials (LGF) and the soil samples. While the decreasing trend in the LL values with an increase in the LGF above 2% can be attributed to the application of excess LGF to the soil, which resulted in pore fluid chemistry changes. Although, Gurumurthy and Rayudu, (2019) stated that the changes in pore fluid chemistry causes a physico-chemical interaction between the soil particles, as well as the particle groups. Thus, initiating a replacement of cations or changes in the thickness of the diffuse double layer. Also, as sample B does not have any montmorillonite, which is a water-retaining mineral, its tendency to absorb water to its pores is reduced, thereby leading to a decrease in the LL as the LGF increases.

Tab. 7 *Table showing the Liquid Limit Test Results*

LGF (%)	Sample A LL (%)	Sample B LL (%)
0	48	46
2	54	50
4	56	44
6	57	42

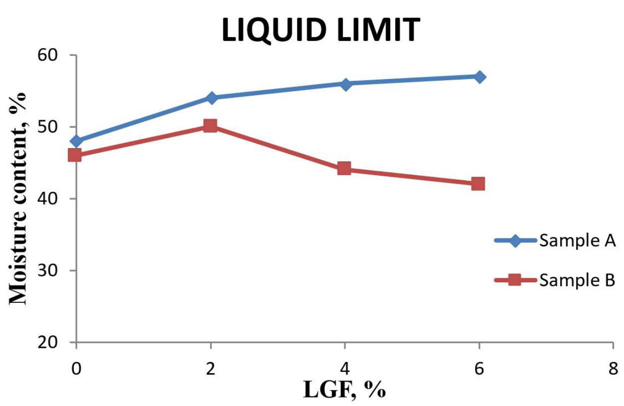

Fig. 1 *Graph of liquid limits at varying LGF contents*

3.3.2 Plastic Limit (PL)

The Table 8 shows the results obtained for the PL and depicted in Fig. 2. The data shows there is a rapid increase in the plastic limit of the soil sample from 29% to 39% as the percentage of LGF increases from 0% to 2% and thereafter remains constant up to 6% LGF for sample A. Whereas for sample B, the PL value at 0% LGF is 31% and decreases to 27% and 29% with the addition of 2% and 4% LGF respectively. At 6% LGF, the PL increases to 32%, which is the maximum PL value for sample B. For sample A, a percentage increase of about 34.50% in the PL was recorded. Similar results were obtained by Murmu et al. (2020) with 20% class F fly ash used as a soil stabilizer, which caused an increase in the PL of the soil by 36.6%. This increase could be attributed to the chemical interaction of the soil minerals with the lime and glass fine stabilizers in the presence of water. This chemical interaction therefore alters the ion interactions within the soil diffuse double layer, which in turn affects the soil water retention and consistency limit. However, for the higher value of PL for sample A compared to sample B, this can be attributed to the variations in the clay mineral components of the two soil samples. As sample A contains high percentages of clay minerals compared to sample B, the water retention of sample A is expected to be higher, thus leading to the maximum PL maximum value of 39% at 2% LGF as compared to the PL of 32% at 6% LGF for sample B.

Tab. 8 *Table showing Plastic Limit Test Results*

LGF%	Sample A PL (%)	Sample B PL (%)
0	29	31
2	39	27
4	39	29
6	39	32

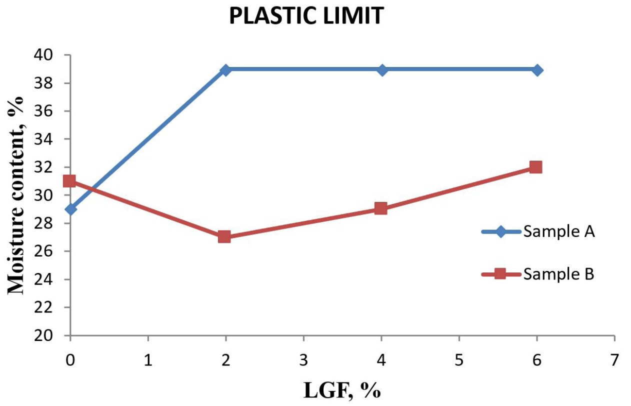

Fig. 2 Behavior of plastic limit under various LGF contents

3.3.3 Plasticity Index (PI)

The results obtained for the plasticity index are presented in Table 9 and Fig. 3, respectively. For sample A, the PI obtained for the unstabilized soil is 19%. However, the addition of LGF causes a decrease in the PI values below the unstabilized PI value for all percentages of the LGF addition. For soil sample B, the PI value obtained for the unstabilized soil is 25%. However, at 2% LGF, a slight decrease to 23% was obtained, while a significant decrease to 15% and 10% was obtained at 4% and 6% LGF, respectively. These results show that LGF has a significant effect on the plasticity index of the lateritic soil. The reduction in the PI for sample A can be attributed to the presence of montmorillonite minerals in the soil sample when reacting with lime. Jalal et al. (2020) opined that lime is known to provide the highest plasticity index (PI) reduction for montmorillonite soils.

Tab. 9 *Table showing Plastic Index Test Results*

LGF (%)	Sample A PI (%)	Sample B PI (%)
0	19	25
2	15	23
4	17	15
6	18	10

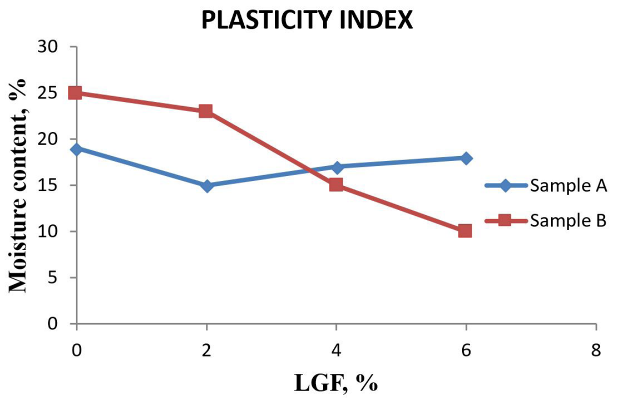

Fig. 3 Behavior of PI under varying LGF contents

3.4 Compaction

The results of the compaction characteristics for both samples are presented in Table 10. Also, Figures 4 and 5 show the behavior of the Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) at different percentages of the LGF contents. The dry density ranges between 1.54g/cm^3 at 0% LGF to a maximum value of 1.66g/cm^3 at 2% LGF, while the optimum moisture content ranged from 9.56% for the unstabilized soil sample A and continued to increase with the addition of the LGF stabilizers to 17.3% at 6% LGF for sample A. The addition of 2% LGF causes an initial increase in the MDD, while a further increase in the LGF causes a decrease in the MDD of the soil sample A. Similarly, at 0% LGF, sample B has a MDD value of 1.61g/cm^3, which then increases to 1.70g/cm^3 at 4% LGF; however, at 6% LGF, sample B decreases to 1.68g/cm^3.

The initial increase in the MDD values of both samples A and B can be attributed to the addition of LGF material of a higher specific weight, and the increase in the moisture content can be attributed to the additional water the LGF requires to begin the hydration processes (Oluwatuyi et al., 2018). Similar results were obtained by Osinubi (2006), who concluded that for clayey soil stabilized with lime, a higher OMC value is required. However, decreases in the MDD at 4% LGF for sample A and at 6% LGF for sample B can be attributed to the higher content of carbon in the GF, resulting in a mixture of a lower specific gravity (Rivera et al., 2020). Similarly, Kumar et al. (2019) attributed a decrease in MDD to the void spaces within the soil matrix created by the stabilizers after being mixed with the soil samples, as it enhanced the water and air entrapment in the matrix, thereby facilitating a slippery surface at the interface of the soil composite and leading to a decrease in MDD.

Tab. 10 *Table showing the MDD and OMC results*

LGF%	SAMPLE A		SAMPLE B	
	MDD (g/cm³)	OMC (%)	MDD (g/cm³)	OMC (%)
0	1.54	9.56	1.61	14.50
2	1.66	12.00	1.62	16.50
4	1.59	15.20	1.70	15.80
6	1.63	17.30	1.68	13.50

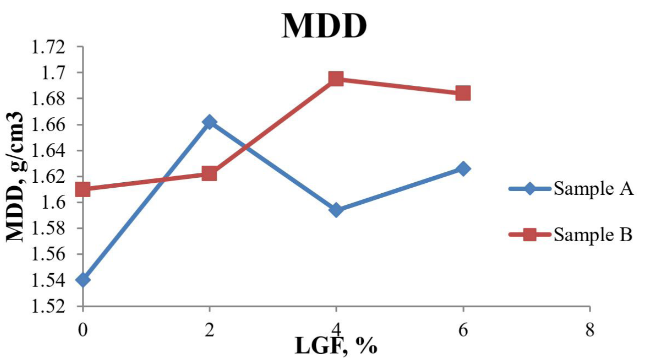

Fig. 4 *Behavior of MDD under varying LGF contents*

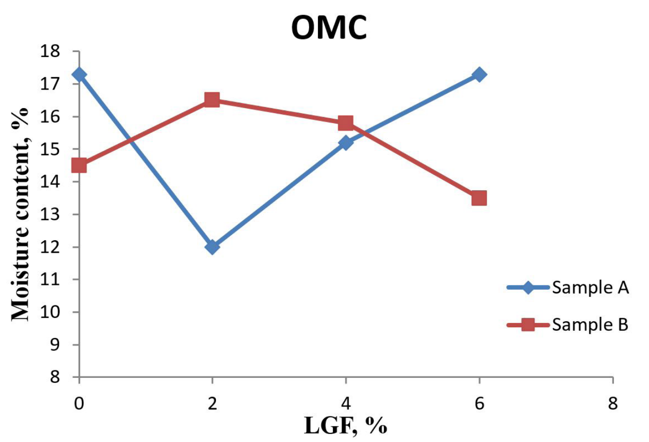

Fig. 5 *Behavior of OMC under varying LGF contents*

3.5 California Bearing Ratio (CBR)

The Table 11 presents the CBR (unsoaked) results under varying percentages of LGF. The CBR values increase with increases in the LGF content from 32% at 0% LGF to 62% at 4% LGF for sample A; above 4% LGF, the addition of LGF causes a decrease in the CBR of the soil with an increase in the LGF. Similarly, the CBR for sample B increases with an increase in the LGF from 35% at 0% LGF to 72% to 6% LGF respectively as shown in Fig. 6. Also, no significant difference was observed in the CBR value for both samples at 4% before the value of sample A decreased while sample B increases further at 6% LGF for both samples.

The increase in the CBR for both samples could be attributed to the ability of the lime acting as an activator due to its strong alkalinity to dissolve some silica from both soil samples A and B to participate in the reactions (Abdeldjouad et al., 2019), thus causing an increase in the bearing capacity of both samples. Similarly, the reaction of lime with the silica and alumina in the glass waste fines produces an appropriate amount of calcium silicate hydrate (C–S–H) gels and calcium aluminosilicate hydrate (C–A–S–H) gels that bind the clay particles and the GF, thereby increasing the bearing capacity of the soil samples (Wang et al. 2023). However, the decrease in the CBR value for sample A at 6% LGF can be attributed to the presence of montmorillonite clay minerals, which causes the LGFA mixture to absorb and retain more water in its pore spaces for its reaction process to be complete. The absorption of this excess pore water therefore leads to a decrease in the CBR value of the soil sample A at 6%. However, based on FMWH (1997), the CBR results obtained for both samples A and B at 4% LGF show a promising potential for LGF to act as a stabilizing material for improving the bearing capacity of the soil as subgrade and/or sub-base materials.

Tab. 11 *California Bearing Ratio Test results at varying LGF percentages*

LGF (%)	SAMPLE A CBR (%)	SAMPLE B CBR (%)
0	32	35
2	48	60
4	62	64
6	55	72

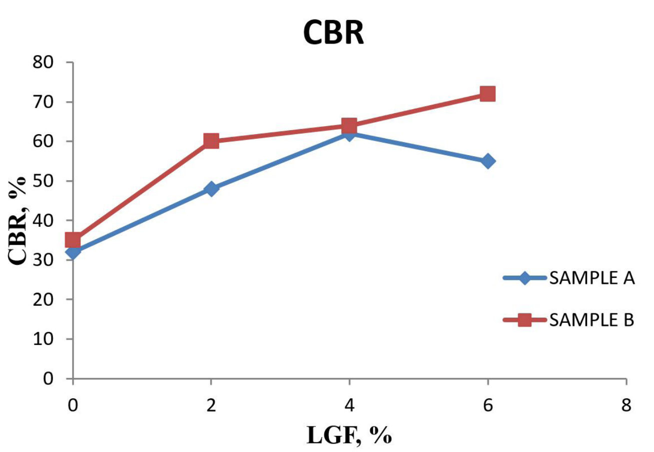

Fig. 6 *Effect of variations of LGF on CBR*

3.6 Unconfined Compressive Strength (UCS)

The effect of varying contents of LGF on the UCS of the lateritic soil after 24 hours of curing at room temperature is presented in Table 12 and depicted in Fig. 7 respectively. The value of UCS increased with an increase in the percentage of LGF stabilizer from 41.6kPa at 0% to 64.4kPa at 2% for sample A, and 38.4kPa at 0% to 50kPa at 6% LGF, respectively.

Figure 7 shows that the UCS of both samples increases with an increase in the LGF content. At 2% LGF for sample A, the overall maximum UCS value of 64.4% was obtained, and an increase in LGF to 4% decreases the UCS value; then an increase in the UCS value was thereafter observed at 6% LGF. However, for sample B, a continuous steady rise in the UCS value as the LGF increased was observed. Also, for both soil samples, sample B with a lower kaolinite content showed the least amount of compressive strength compared to sample A, which signifies the effect of clay minerals on the strength of the LGF-stabilized soil samples.

As shown in Table 5, the high amount of the LOI (unburned carbon) (24%) in the GF may have a negative effect when mixed with soil in an alkali medium such as lime due to the tendency of unburned carbon to absorb a certain amount of the alkaline activator to remove alkali cations, thereby hindering the kinetics of the dissolution reactions of the precursor, condensation and polymerization of cementitious gels as a result of its large surface area (Lee et al. 2010), (Shearer et al. 2016). Also, the chemical composition of the soil as shown in Table 2 can also have an adverse effect on the development of the compressive strength of soil stabilized in an alkali medium such as lime due to the degree of laterization. According to Oluwatuyi et al. (2018), the degree of laterization is defined as the ratio of silicon to sesquioxide ($SiO_2/(Al_2O_3 + Fe_2O_3)$). Both soil samples A and B used in this study have a ratio of 1.68 and 1.99, respectively. These signify an advance chemical weathering process of the soil due to its high amount of iron and aluminum oxides. Similar studies have shown that the reactive Fe_2O_3 may precipitate much faster than Al or Si compounds under highly alkaline conditions to generate iron hydroxides or oxyhydroxides, which consume the OH- ions of the lime; this slows the dissolution of the remaining particles of the precursor material such as GF, thus reducing the number of nucleation sites of the oligomers that form the cementitious aluminosilicate gel (Rivera et al., 2019). This may be the reason for the slow steady rise in the UCS value for sample B. However, other studies have suggested that Fe^{+3} acts as a nucleation site since in silicate glasses, Fe^{+3} is known as a form of network due to a charge and ionic radius similar to those of Al^{+3} (Lemougna et al. 2013), (Choi et al. 2012).

Tab. 12 Unconfined Compressive Strength Test Results

LGF (%)	Sample A UCS (kPa)	Sample B UCS (kPa)
0	41.6	38.4
2	64.4	39.0
4	54.5	42.6
6	59.9	50.0

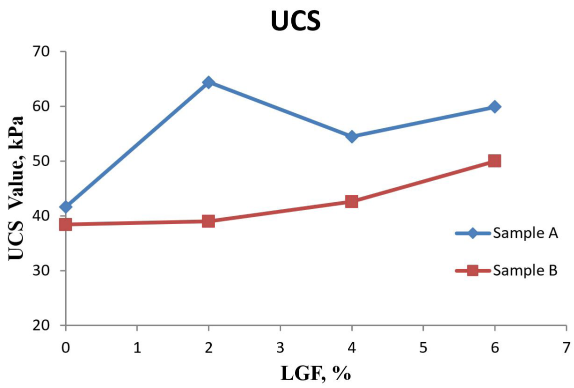

Fig. 7 Graph showing the effect of LGF on the UCS of lateritic soil

4 CONCLUSIONS

This study examined the combined effects of lime and glass waste fines (LGF) on the mechanical behaviour of two lateritic soils collected within the metropolis of Ibadan, Nigeria, with the aim of identifying the minimum lime content required, together with the LGF, to achieve an improvement in the optimum strength. Based on the experimental investigations conducted, the following conclusions can be drawn:

1. Chemical Reactivity
Both lateritic soils (samples A and B) contained high proportions of silicate oxides, which actively participated in the reaction with the LGF. This contributed significantly to the pozzolanic reaction mechanisms responsible for improving the strength.

2. Plasticity Characteristics
The presence of water-retaining minerals, particularly the 7% montmorillonite content in Sample A, was responsible for the increase in the liquid limit.

3. Compaction Characteristics
MDD was achieved at 2% LGF for Sample A and 4% for Sample B. The relatively high kaolinite content in sample A was likely contributed to its earlier attainment of optimum MDD.

4. Strength Performance (CBR)
The peak CBR values were recorded at 4% LGF for Sample A and 6% LGF for Sample B. Although Sample B performed better overall, the results demonstrate that lime effectively activated LGF, thus enhancing the pozzolanic reactions and resulting strength gains.

5. Strength Performance (UCS)
The maximum UCS values occurred at 2% and 6% LGF for both Sample A and Sample B respectively, with Sample A exhibiting relatively higher improvement. Both soils, however, showed a consistent upward trend in UCS as the LGF content increased.

Overall, the findings confirmed that the combined use of lime and glass-waste fine is effective for enhancing the compaction and strength properties of lateritic soils, with the optimal LGF content varying between the soil types due to the differences in mineralogical composition.

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