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CLAY SOIL STABILISATION USING POWDERED GLASS

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Abstract

This paper assesses the stabilizing effect of powdered glass on clay soil. Broken waste glass was collected and ground into powder form suitable for addition to the clay soil in varying proportions namely 1%, 2%, 5%, 10% and 15% along with 15% cement (base) by weight of the soil sample throughout. Consequently, the moisture content, specific gravity, particle size distribution and Atterberg limits tests were carried out to classify the soil using the ASSHTO classification system. Based on the results, the soil sample obtained corresponded to Group A-6 soils identified as 'fair to poor' soil type in terms of use as drainage and subgrade material. This justified stabilisation of the soil. Thereafter, compaction, California bearing ratio (CBR) and direct shear tests were carried out on the soil with and without the addition of the powdered glass. The results showed improvement in the maximum dry density values on addition of the powdered glass and with corresponding gradual increase up to 5% glass powder content after which it started to decrease at 10% and 15% powdered glass content. The highest CBR values of 14.90% and 112.91% were obtained at 5% glass powder content and 5mm penetration for both the unsoaked and soaked treated samples respectively. The maximum cohesion and angle of internal friction values of 17.0 and 15.0 respectively were obtained at 10% glass powder content.

Keywords: Atterberg limits, Subgrade material, Stabilisation, Compaction, California bearing ratio.

1. Introduction

Clay soils exhibit generally undesirable engineering properties. They tend to have low shear strength which reduces further upon wetting or other physical disturbances. They can be plastic and compressible; expand when wetted and shrink when dried. Some types expand and shrink greatly upon wetting and drying, thereby, exhibiting some very undesirable features. Cohesive soils can creep

Nomenclatures

G_s	Specific gravity
M_1	Mass of empty can, g
M_2	Mass of can and wet sample, g
M_3	Mass of can and dry sample, g
m_1	Mass of specific gravity bottle, g
m_2	Mass of specific gravity bottle with 50 g of soil sample, g
m_3	Mass of specific gravity bottle with water and soil sample, g
m_4	Mass of specific gravity bottle filled with water, g

Greek Symbols

γ	Density, kN/m^3
γ-max	Maximum density, kN/m^3
τ_F	Maximum shear at failure

Abbreviations

CBR	California bearing ratio
LL	Liquid limit
LS	Linear shrinkage
MC	Moisture content
MDD	Maximum dry density
OMC	Optimum moisture content
PI	Plasticity index
PL	Plastic limit

over time under constant load, especially when the shear stress is approaching its shear strength, making them prone to sliding. They develop large lateral pressures and tend to have low resilient modulus values. For these reasons, clays are generally poor materials for foundations. Their properties may need to be improved upon in some cases by soil stabilisation.

Stabilisation is the process of blending and mixing materials with a soil to improve the properties of the soil. The process may include the blending of soils to achieve a desired gradation or the mixing of commercially available additives that may alter the gradation and improve the engineering properties of soil, thus making it more stable. This study seeks to determine the geotechnical properties of clay soil stabilised with broken glass through laboratory tests. The disposal of wastes produced from different industries has become a great problem. These materials pose a threat to the environment because they can result in pollution in the nearby locality since they are majorly non-biodegradable.

In recent years, applications of industrial wastes have been considered in road construction both in industrialised and developing countries. Utilization of such materials is based on technical, economic and ecological criteria which are crucial for a country like Nigeria which normally provides a good environment for both the manufacture and importation of glass materials. However, Nigerian cities and towns are currently facing serious environmental problems arising from poor solid waste management. The rate of solid waste generation in Nigeria has increased with rapid urbanization. Solid waste is generated at a rate which has grown beyond what the capacity of the city authorities can handle. This has resulted in a poor solid-waste management system that portends serious environmental crisis in most Nigeria towns and cities. The components of solid waste in Nigeria consist mainly of polythene bags, pieces of clothes, foodstuffs, plastic and paper materials. Others are metals, tins, bottles and glass materials.

Traditionally soil, stone aggregates, sand, bitumen, cement among others are used for road construction. However, the cost of sourcing for and processing such materials in adequate quantity and quality is on the increase. Hence, a lot of research is on-going to identify and develop alternative materials for highway construction and industrial waste products are one of such alternatives. If these materials can be developed and suitably utilised in highway construction, the attendant pollution and disposal problems may be greatly reduced.

Consequently, this study assesses the use of broken (waste) glass in powdered form as stabiliser for clay soil. The underlying objectives are to identify the waste glass materials to be used process the waste glass into suitable form for addition to the soil, assess its performance as a stabiliser by measuring its effects on clay soil (through a comparison of the properties of the soil with and without the addition of the broken glass) and determine appropriate quantities of the broken glass required for adequate stabilisation of the clay soil.

2. Background Literature

Soil stabilisation is the alteration of soils to enhance their physical properties. It can increase the shear strength of a soil, control its shrink-swell properties and improve its load bearing capacity. Soil stabilisation can be utilized on roadways, parking areas, site development projects, airports and many other situations where sub-soils are not suitable for construction. It can also be used to treat a wide range of subgrade materials varying from expansive clays to granular soils as well as improve other physical properties of soils such as increasing their resistance to erosion, dust formation or frost heaving.

Historically, engineers have long been aware of the stabilizing effects of various materials in earth works. The first and by far, the most extensive and successful application of stabilisation was developed by the French engineer, Henry Vidal, in the late 1950's. Vidal's system was known as 'Reinforced Earth', which consists of placing steel reinforcing strips at predetermined intervals within the fill mass for the purpose of providing tensile or cohesive strength in a relatively cohesionless material [1].

Vidal developed the idea for reinforced earth while visiting a sandy beach on the Mediterranean. He toyed with the sand, arranging it in piles which quickly slid down forming cones with an angle of repose that always remained the same. He then placed rows of pines needles tending more towards the vertical. Essentially, he reinforced the sand so that the internal friction between the sand and the pine needles held the sand in place. This theory was verified in 1965 when he designed and built the first reinforced earth embankment. In the reinforced earth concept, the steel strip reinforcement resists the forces that develop in the soil mass by means of transfer through friction between the soil grains and the reinforcement. If reinforcement is properly designed and placed, it is possible to avoid share failure so that the entire mass behaves like a cohesive solid capable of withstanding both internal and external forces.

2.1. Clay soil formations

Clay minerals are typically formed over long periods of time by the gradual chemical weathering of rocks (usually silicate-bearing) by low concentrations of carbonic acid and other diluted solvents. These solvents are usually acidic and migrate through the weathering rock after leaching through upper weathered layers. In addition to the weathering process, some clay minerals are formed by hydrothermal activity.

Clay deposits may be formed in places such as residual deposits in soil but thick deposits usually are formed as the result of a secondary sedimentary deposition process after they have been eroded and transported from their original location of formation. Generally, clay soils have poor drainage characteristics which are largely dependent on the infiltration rate, which can be changed by adding larger particles of organic matter or pea gravel to the soil.

2.2. Principles of soil stabilisation

Stabilisation is defined as the process of improving the soil aggregate properties by blending in materials that increase the load bearing capacity, firmness and resistance to weathering or displacement. It can be defined as the process of altering the soil properties by mechanical or chemical means thereby improving the desired engineering properties of such soils. There are three purposes for soil stabilisation namely strength improvement, permeability control and enhancement of soil durability and resistance to weathering.

2.2.1. Mechanical stabilisation

Mechanical stabilisation is achieved by compaction of interlocks of soil-aggregate particles. The grading of the soil-aggregate mixture must be such that a dense mass is produced when it is compacted. Uniformly mixing the material and compacting the mixture can accomplish mechanical stabilisation. As an alternative, additional aggregates may be blended before compaction to form a uniform, well-graded, dense soil-aggregate mixture after compaction. The choice of methods should be based on the gradation of the material.

Soil compaction is the process of increasing the density of soil by packing the particles closer together causing a reduction in the volume of air. The volume of water, initially, remains unchanged. Soil water acts as a lubricant increasing compaction when a load is imposed on the soil. By packing primary soil particles such as sand, silt, clay and soil aggregates closer together, the balance between solids, air-filled and water-filled pore spaces is dramatically altered.

According to Bowles [2], compaction usually eliminates the largest soil pores. A large portion of the initial soil air is forced out of the upper plant root zone, and the channels of greatest continuity and least resistance to air movement, water movement and root penetration are destroyed. Under comparable conditions, soils with a range of soil particle sizes (such as fine sandy loam) are generally more compactable than sandy soils of uniform particle size. The primary function of the portion of a mechanically stabilised soil mixture that is retained on a no. 200 sieve is to contribute internal friction. However, the best aggregates are those that are made up of hard, durable, angular particles with gradation that makes them more easily compacted.

2.2.2. Chemical stabilisation

This method deals with improving the engineering properties of soil by adding chemicals or other such materials and it is generally cost effective. These additives react with the soil usually clay minerals, with subsequent precipitation of new and insoluble minerals, which bind the soil together [3]. There are various categories of these chemical admixtures namely cementing agents, modifiers, water proofing agents, water retaining agents, water retarding agents and miscellaneous chemicals. In addition, their characteristics are vastly different from the others and each has its particular use as well as limitations.

Considering the cementing agents first, the materials often used are Portland cement, lime, mixture of lime and fly ash, and sodium silicate. Portland cement has been used extensively to improve existing gravel roads as well as stabilise the natural subgrade soils. Other admixtures that have come into extensive use in recent years are lime and fly ash admixtures. Fly ash is a by-product of blast furnaces and is generally high in silica and alumina. However, the quantity of fly ash required for adequate stabilisation is relatively high, making its use restricted to areas with availability of large quantities of fly ash at relatively low cost.

The next category of stabilisation includes the water proofing materials. Foremost among these are bituminous materials used for coating the soil grains so as to retard or completely stop absorption of moisture. Bituminous stabilisation is best suited for sandy soils or poor quality base course materials and its benefit is derived by driving off the volatile constituents of the bitumen just prior to compaction.

2.3. Reinforced fiber stabilisation

Early civilisation witnessed the use of straws, plant roots, soil bricks and cob wall to improve soil properties even though their mechanism may have not been fully understood. However, modern geotechnical engineering has focused on the use discrete fibers in reinforcing soils which is still a relatively new technique in geotechnical projects. The concept of fiber-reinforcement in geotechnical projects originally involved the use of plant roots as reinforcement. Most researchers reported that plant roots increase the shear strength of the soil and, consequently, the stability of natural slopes.

Al-Khafaji and Andersland performed triaxial tests on kaolinite clay reinforced with cellulose pulp fibers [4]. The shear strength under various testing conditions (undrained, consolidated drained and consolidated undrained) increased with increasing fiber content and the mode of failure changed from brittle to plastic. The ductility of the specimen was also found to increase with increasing fiber content.

Al-Joulani evaluated the use of waste fiber materials such as scrap tire rubber, polyethylene and polypropylene fiber for the modification of clayey soils under unconfined compression, shear box and resonant frequency tests [5]. It was discovered that waste fibers improve the strength properties and dynamic behaviour of clayey soils.

Gray carried out series of laboratory unconfined compression, splitting-tension, three-point-bending and hydraulic conductivity tests on kaolinite clay reinforced with fiber, and reported that randomly distributed fibers increase the peak unconfined compressive strength, ductility, splitting tensile strength and flexural toughness of kaolinite clay [6]. The contribution of fiber-reinforcement was found to be more significant for specimens with lower water contents. Some researchers have studied the use of fibers to improve the ductility of cement-stabilised soils.

Consoli et al. reported that fiber-reinforcement increases the peak and residual shear strength of cement-treated soil and changes their brittle behaviour to ductile behaviour [7]. Consoli et al. reported similar behaviour when using fibers with soils stabilised with cement or fly ash [8]. The behaviour of fiber-reinforced uncemented soil was different from that in fiber reinforced cemented soil. Increasing fiber content was shown to increase the peak axial stress and decreases the stiffness. In addition, the distribution of the fibers in all parts of the soil was more effective than layer distribution.

2.4. Engineering properties of fiber reinforced soil

The addition of fiber reinforcement in the sand and clay specimens was reported to cause a substantial increase in the peak friction angle and cohesion values. The shear strength envelope for the clay specimens is described by a combination of curvilinear and linear sections. The friction angle at low confining pressures was found to be slightly larger than that at higher confining pressure. The phenomenon was explained as an effect of dilatancy which increases the interface shear strength between fiber and soil. This effect is more pronounced at low confining stresses than at high confining stresses.

Previous research on the equivalent shear strength of fiber-reinforced soil has focused on quantifying the effect of fiber content and aspect ratio. Several predictive models have been proposed. These include a load transfer model that requires parameters obtained with non-conventional testing of soil-fiber composites, a strain energy approach that uses energy concepts and a statistical model based on the regression analyses of previous test results. A recently proposed discrete design methodology used concepts derived from limit equilibrium, and requires independent characterization of soils and fibers [9]. However, additional experimental results are needed to validate the proposed design models. The accuracy of the prediction of these models also relies on a proper understanding of the mechanism of interface and interactions between the fibers and soils.

2.5. Influence of fibers on soil compaction

Fletcher and Humphries reported results of compaction tests on silty clay soil specimens reinforced with fibers [10]. It was concluded that the presence of fibers decreases the ability of soil to densify. Unlike the case of sandy gravel reported by Alobaidi and Hoare, the test results showed that increasing the fiber content caused a modest increase in the maximum dry unit weight [11]. The optimum water content was found to decrease with increasing fiber content. Prabakar and Sridhar reported similar results [12]. The results of compaction test on palm fiber reinforced silty sand showed that the maximum dry density decreases and optimum moisture content increases with increasing fiber content [13]. According to Terrel et al. [14], the soil mixture in its untreated form shows a lower dry density and higher optimum moisture content than the untreated soil for a given compaction effort.

Maher and Ho studied the effect of fibers on the hydraulic conductivity of a kaolinite-fiber composite and showed that its inclusion increased the hydraulic conductivity of the composite which became more pronounced at higher fiber contents (up to 4% by weight) [15]. Despite the increase, the hydraulic conductivity of the composite was still low enough to be considered for some landfill applications and acceptable to satisfy the requirements for landfill cover design.

2.6. Powdered glass stabilisation

This involves the addition of broken glass powder to soil so as to improve its engineering performance. Glass is totally inert and therefore non-biodegradable. It degrades in a manner similar to natural rock. As an inert construction material, it can increase the strength of various road building elements. Glass has been experimented on as a substitute aggregate in asphalt concrete. Crushed glass has also been used as an aggregate for sub-base.

Glass is an amorphous non crystalline material, which is typically brittle and optically transparent. The familiar type of waste glass materials are drinking vessels and windows, however, most of the readily available waste glass material is soda-lime glass composed of about 75% silica ($Si0_2$), Na_2O, CaO and several additives.

3. Methodology and Materials

The materials used in carrying out this project are powdered glass, cement, clay soil and water. Glass is an amorphous non crystalline material which is typically brittle and optically transparent. The familiar type of waste glass materials found around are drinking vessel and windows, most of the readily available waste glass materials are soda-lime glass, composed of about 75% silica ($Si0_2$) plus Na_2O, CaO, and several additives. This material is added to clay soil in its powdered form for soil stabilisation.

Cement can be described as a material with adhesive and cohesive properties, which make it capable of holding material fragment into a compacted aggregate. It is manufactured from limestone and is added to an expansive soil to improve its engineering properties. It may be formed in place as residual deposits in soil while larger deposits usually are formed as the result of a secondary sedimentary deposition process after they have been eroded and transported from their original location of formation. Lastly, water is a universal solvent. The water used is obtained from bore holes and is free from suspended particles like organic matter and silt which might affect the hydration process of cement.

3.1. Collection, processing and composition of materials

The glass bottles used for this study were obtained from Cocacola's Bottling depot, Oba-Ile, Akure, Ondo State. They were washed, dried, broken down manually into smaller sizes with the use of hammer and passed through sieve number 400 to produce the glass in powdery form. However, the clay soil was collected from the bank of the stream located at Baydock also in Akure, after removing the top soil and excavating to a depth of about 0.7 m, for both disturbed and undisturbed samples. Figure 1 shows a sample of the glass powder obtained from the process.

Fig. 1. Sample of Glass Powder Used.

The chemical composition of the glass material used is as follows; 76% of $Si0_2$, 11.50% of Al_20_3, 11.6% of Na_20 while other constituents accumulate to 0.9%. In terms of physical properties, it has a specific gravity of 2.5-2.9, tensile strength of 27- 62 MPa, softening point of 1500- 1750° C and hardness of 5 to7.

3.2. Laboratory tests and analysis

Various tests and analysis were carried out to examine the effects of the glass powder on the clay soil namely particle size distribution analysis, specific gravity test, Atterberg limits test, compaction test, California Bearing Ratio test and Direct Shear test were carried to the investigate the effect of glass. Based on these tests, the required quantity of glass for effective stabilisation of the clay soil was determined.

3.2.1. Particle size distribution analysis

The particle size distribution expresses the size of particles in terms of percentage by weight of the soil passing each sieve. The procedure involves oven drying the clay soil sample for 24 hours allowing it to cool, and soaking also for 24 hours. The sieve 75 μm was then used to wash and sieve the soil which was then oven dried and its resulting weight was recorded. The sieves were arranged according to the aperture size and the reweighed sand was poured into the set of sieves and shaken vigorously for 10minutes. The sieve was left for a while for the sample to settle, the sand retained on each sieves was weighed and recorded and the corresponding percentage retained and passing were calculated. A graph of the percentage passing was plotted against the sieve sizes.

3.2.2. Natural moisture content

This test was used to determine the amount of moisture content present in the soil as a percentage of its dry mass. The empty can was weighed to the nearest 0.1 g (and represented as M_1) after which a considerable amount of wet sample was placed therein and weighed (represented as M_2). Thereafter, it was placed into the oven to dry for 24 hours, removed and weighed (represented as M_3). The moisture content (MC) was calculated as a percentage of dry soil mass by using Eq. (1).

$$MC = \frac{M_2 - M_3}{M_3 - M_1} \times 100\% \tag{1}$$

3.2.3. Specific gravity test

The specific gravity of a soil sample can be defined as the weight in air of a given volume of soil particles to the weight in air of an equal volume of distilled water of about 40°C in temperature. The procedure for its determination involved emptying, drying and weighing the specific gravity bottle (to give m_1) into which 50 g of the soil sample was introduced and weighed (to give m_2).

Water was then added to the sample in the glass jar to $^1/_3$ of its real height and stirred vigorously till the sample particles were in suspension. This was allowed to stand for 30 minutes before water was added to $^2/_3$ of the glass jar and kept for 24 hours after which it was filled to the glass jar brim and weighed as (m_3). Thereafter, the bottle content was poured out and cleaned. In addition, the jar was filled with water to the brim and its resulting weight was determined as (m_4). The specific gravity (G_s) was calculated by using Eq. (2).

$$G_S = \frac{m2 - m1}{(m4 - m1) - (m3 - m2)} \tag{2}$$

3.2.4. Atterberg limits tests

The Atterberg limits are basic measures of the nature of fine-grained soil, depending on the water content of the soil, it may appear in four states: solid, semi-solid, plastic and liquid. In each state, the Consistency, Behaviour and Engineering properties of a soil is different. Thus, the boundary between each state can be defined based on a change in the soil's behaviour. The Atterberg limits can be used to distinguish between silt and clay, and it can distinguish between different types of silts and clays. The different Atterberg limits are liquid limit, shrinkages limit and plastic limit test.

The procedure for determining the liquid limit (LL) involved measuring out 200 g of soil sample passing the 42 μm BS test sieve, pouring it on the metallic trays, adding a little quantity of water to it and mixing thoroughly to obtain a paste that was not too thick nor too watery. It was then placed in the Casagrande's device, levelled and divided with the grooving tool. The number of blows at which the divided part became closed was recorded. A portion of this soil was put in a can to determine its moisture content. The experiment was repeated while gradually increasing the amount of distilled water added. Then, the relationship between the moisture content and the corresponding number of blows was plotted. The moisture content corresponding to 20 blows was considered as the liquid limit of the soil.

The procedure for determining the plastic limit (PL) involved moulding and rolling the already thoroughly mixed sample with the palms to a threadlike shaped stick of about 3 mm diameter. The plastic limit was indicated by the moisture content corresponding to the point at which the stick first crumbled. Consequently, the plasticity index (PI) was calculated by using Eq. (3).

$$PI = LL - PL \tag{3}$$

Determination of the linear shrinkage (LS) involved obtaining the moisture content below which no further volume changes of a mass occurs. The procedure involved mixing thoroughly a portion of the clay sample passing the 425 μm BS sieve with water and placing it in a cleaned and greased shrinkage mould. The surface of the soil was levelled and the bowl was placed in an oven for 24 hours. Thereafter, the corresponding reduction in the length of the sample was measured and used in calculating the linear shrinkage based on Eq. (4).

$$LS = 1 - \frac{\text{Length of oven dried specimen}}{\text{Initial length of specimen}} \text{ x } 100 \tag{4}$$

3.2.5. Compaction test

This test determines the maximum (practical) dry density (γ-max) and optimum moisture content (W opt) of the soil with and without additives. The results are subsequently used in the preparation of CBR specimens. Compaction tests provide important information about the soil quality at a site which can be used to determine the most favourable building sites, the maximum load the soil can withstand and the appropriateness of the site for building. In order to investigate the effect of powdered glass on clay soil with addition of (base) cement throughout, the mix proportions of clay soil, cement and powdered glass as shown in Table 1 were used in the tests.

Table 1. Mix Proportions Used for Tests.

S/N	Clay soil (%)	Cement (%) by Mass of Soil	Glass Powder (%) by Mass of Soil
1	100	15	0
2	100	15	1
3	100	15	2
4	100	15	5
5	100	15	10
6	100	15	15

The test procedure involved pouring the oven dried clay soil on a tray and breaking it down into smaller and fine particles. Then 3000 g of this broken material was weighed and poured on a metallic tray. It was then mixed thoroughly with 60 ml of distilled water, placed in the mould (in 3 layers, each 50 mm thick) and subjected to 25 blows using the standard rammer (of 2.5 kg falling through

30 cm). The top surface of the layer was scraped before placing the subsequent layers of loose soil. After compacting the third layer, the level of the compacted soil was slightly above the top of the mould. The collar was then removed and followed by trimming of the soil with a straightedge and determination of its mass.

Two samples were taken, one from the top and the other from the bottom of the mould, and their respective moisture contents were determined. This was followed by extruding the sample from the mould and breaking it up into a loose state. Another 60 ml of water was added and the same series of steps were repeated until the mass of the compacted soil in the mould fell. Thereafter, a graph of moisture content versus dry density was plotted and the maximum dry density (MDD) and optimum moisture content (OMC) corresponding to standard doctor compaction test were determined. The calculations under the compaction test were carried out using Eqs. (5) and (6).

$$\text{Wet density;}\quad (\gamma - wet)\text{g/cc} = \frac{\text{weight of soil in compaction mould}}{\text{volume of mould}} \tag{5}$$

$$\text{Dry density;}\quad (\gamma - dry)g/cc = \frac{\gamma - wet}{1+\frac{\text{MC}}{100}} \tag{6}$$

3.2.6. California bearing ratio test

The California Bearing Ratio Test (CBR Test) is a penetration test used for evaluating the bearing capacity of subgrade soil for design of pavements. It is carried out on natural or compacted soils under soaked or un-soaked conditions and the results obtained are compared with the curves of standard tests to indicate the soil strength. The test is performed by measuring the pressure required to penetrate a soil sample with a plunger of standard area which is then divided by the pressure required to achieve an equal amount of penetration on a standard crushed rock material. The test is as described in [16, 17].

In order to carry out this test, the same mix proportions used under the compaction test were used again in this test. 6 kg of dry soil was mixed thoroughly with calculated quantity of water to obtain moist soil with the required moisture percentage. The soil was compacted in three different CBR moulds, each in 3 layers and subjected 25 blows each using the standard rammer (4.5 kg and falling through 30 cm). The top surface was scraped and levelled after compacting the third layer.

Sufficient surcharge mass was then placed on the soil surface to equal the actual or estimated mass of construction. The loading was applied at the rate of 1.25 mm/min. Readings of the load were taken at the following penetrations for both the top and bottom layers namely 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5 and 7.0. Immediately, after the penetration test, filter paper was placed on the compacted exposed surface (both at the top and bottom), closed with metallic cover to prevent direct influence of water and placed in soaking tank for 98 hours. Thereafter, it was removed and the corresponding soaked readings were taken at the same penetrations used under the unsoaked condition for both the top and bottom.

The readings of load intensity were plotted against the readings of penetration and a smooth curve was drawn through the points. The values of the load at penetration of 2.5 mm and 5.0 mm were expressed as percentages of standard loads of 70 kg/cm and 150 kg/cm respectively. The higher value out of these two was considered as the CBR. The CBR values were calculated by using Eqs. (7) and (8).

$$\text{CBR at 2.5 mm penetration} = \frac{\text{Actual Load in kg/cm taken by soil}}{\text{Standard Load at 2.5 mm penetration in kg/cm}} \tag{7}$$

$$\text{CBR at 5.0 mm penetration} = \frac{\text{Actual Load in kg/cm taken by soil}}{\text{Standard Load at 5.0mm penetration in kg/cm}} \tag{8}$$

3.2.7. Direct shear test

The simplest and easiest of the plane shear test is the direct shear test wherein a cubic sized soil sample either undisturbed or resounded is confined in a direct shear box which has two detectable halves of the same size. The soil samples in the shear mould were then loaded vertically under different vertical loads namely 5 kg, 10 kg and 15 kg in mass and then sheared off exactly halfway in between the sample.

The test was repeated two or more times on the same type of specimens under different vertical loading conditions, obtaining each time the maximum shear strength developed at failure. A graph of shear strength (maximum) versus normal loads was drawn. The y-axis represented the value of cohesion while the inclination of the straight line with the x-axis indicated the angle of internal friction. In order to carry out this test, the same mix proportions used in the compaction test were used again in this test.

The test involved placing plates on the top and bottom of the specimen to secure a good grip on it as well as placing the loading plate on the top of the box. The normal loading lever was adjusted horizontally (lever ratio 1:5). The dial gauge to measure both the vertical deformations and the horizontal displacements was adjusted and the readings were recorded. The required normal load was imposed on the specimen by loading the lever pan with the required mass. Also, the electric motor was switched on before commencement of the test while the readings of all dial gauges were taken at regular intervals till failure of the specimen. The test was repeated with two or three more specimens at different normal loads. A graph of the normal stress was plotted against the maximum shearing resistance from which the angle of shearing resistance and apparent cohesion were obtained. Equation (9) was used for calculating the maximum shear at failure.

$$\text{Maximum shear at failure } (\tau_F) = \frac{\text{Maximum shear resistance in kg}}{\text{Area of the specimen}} \tag{9}$$

4. Analysis and Discussion of Results

The natural moisture content, particle size distribution, specific gravity tests and the Atterberg limits tests were carried out to classify the clay soil while the compaction, California bearing ratio and direct shear tests were carried out to assess the effects of glass powder on the soil.

4.1. Natural moisture content

The natural moisture content (average) obtained for the clay soil sample was 21.70% as shown in Table 2.

Table 2. Moisture Content Test Results.

Test Sample	Mass of Empty Can (g) (M_1)	Mass of Can + Wet (g) (M_2)	Mass of Can + Dry (g) (M_3)	Moisture (g)	MC (%)
1	44.30	90.90	82.60	8.30	21.67
2	47.50	102.40	92.80	9.60	21.19
3	48.40	109.40	98.30	11.10	22.24
		Average MC (%)			21.70

4.2. Specific gravity

The specific gravity of the clay soil sample was determined as 2.64 as shown in Table 3.

Table 3. Specific Gravity Test Results.

Masses (g)	Test 1	Test 2	Test 3
Mass of density bottle + Water (Full) = m_4	604.5	588	593.1
Mass of density bottle + Soil + Water = m_3	635.6	618	623.3
Mass of density bottle + Soil = m_2	358.9	304.5	315.7
Mass of density bottle = m_1	308.9	256.5	266.7
Specific gravity	2.65	2.67	2.61
Average specific gravity (G_s)		2.64	

4.3. Particle size distribution

Table 4 shows a breakdown of the particle size distribution analysis with the corresponding percentages retained on and passing through each of the sieves. This analysis showed that the natural soil sample comprised of 28% silt-size fraction and 72% sand fraction as indicated in Fig. 2. Based on the AASHTO Soil Classification System (Highway Research Board Classification system), this soil corresponds to group A-6 with ‘fair to poor’ drainage characteristic as well as a ‘fair to poor’ general rating as a subgrade material. Hence, the soil needs to be stabilised.

Table 4. Particle Size Distribution Analysis.

Sieve Diameter (mm)	Mass Retained (g)	% Retained	% Passing
4.75	0	0	100
2.36	0.3	0.23	99.77
1.700	0.4	0.30	99.47
1.18	1.3	0.98	98.49
0.600	4.3	3.24	95.25
0.500	6.5	4.90	90.35
0.425	1.1	0.83	89.53
0.212	53.7	40.47	49.06
0.150	35.1	26.45	22.61
0.075	27	20.35	2.26
pan	3	2.26	0.00

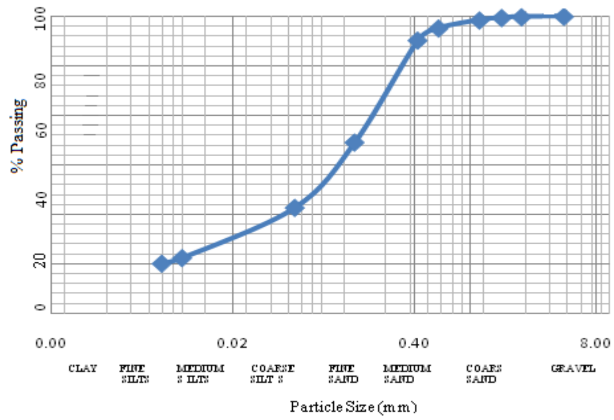

Fig. 2. Particle Size Distribution Chart.

4.4. Atterberg limits tests

The moisture content values obtained under the Atterberg limits test (comprising of the liquid limit, plastic limit and shrinkage limit results) are as shown in Tables 5(a), 5(b) and 5(c). The liquid limit, plastic limit and plasticity index of the natural soil sample were obtained as 24.73, 9.40 and 15.33% respectively while the shrinkage limit was obtained as 6.97%.

Table 5(a). Liquid Limit Results.

Liquid limit					
Test	**Number of blows**	**Mass of wet sample (g)**	**Mass of dry sample (g)**	**Moisture (g)**	**MC (%)**
1	48	7.7	6.9	0.8	11.59
2	28	9.2	7.3	1.9	26.03
3	22	8	6.2	1.8	29.03
4	14	8.2	6.2	2	32.26
Average MC (%)					24.73

Table 5(b). Plastic Limit Results.

Plastic limit				
Test	**Mass of wet sample (g)**	**Mass of dry sample (g)**	**Moisture (g)**	**MC (%)**
1	1	0.9	0.1	11.11
2	1.4	1.3	0.1	7.69
Average MC (%)				9.40

Table 5(c). Shrinkage Limit Results.

Shrinkage limit				
Test	**Initial length (L_o) cm**	**Final length (L_1) cm**	**Change in length (cm)**	**Shrinkage limit (%)**
1	14	13	1	7.14
2	14	13.2	0.95	6.79
Average shrinkage (%)				6.97

4.5. Compaction test

Table 6 summarizes the compaction test results for the clay soil with and without the powdered glass as well as cement base. These results when compared showed that the addition of these additives at varying quantities has a positive effect on the soil, as they all improve the dry density of this selective clay soil. However, the best result was obtained at 5% glass powder with 15% cement (base) content. Figure 3 shows that the MDD value increases from 25.37 kN/m^3 for the control sample up to 25.90 kN/m^3 for the sample containing 5% glass powder with 15% cement (base) and decreases afterwards.

Table 6. Summary of MDD and OMC Values at Different Additives Proportions.

	Control (15% cement)	1% glass + 15% cement	2% glass + 15% cement	5% glass + 15% cement	10% glass + 15% cement	15% glass + 15% cement
MDD (kN/m^3)	25.37	25.79	25.87	25.90	25.67	25.32
OMC (%)	16.40	15.72	15.25	14.96	14.17	14.09

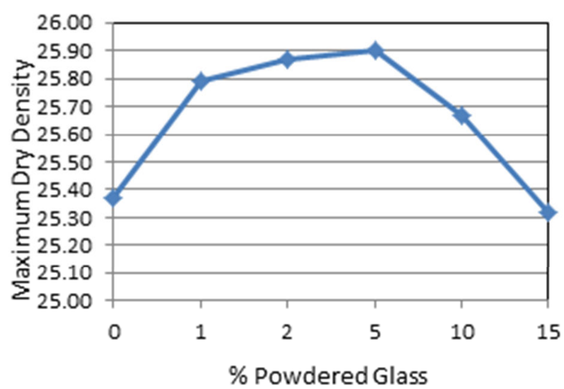

Fig. 3. Graph of Maximum Dry Density against Percentage Glass Powder.

4.6. California bearing ratio test

Tables 7(a) and 7(b) give a summary of the unsoaked and soaked CBR values for the control and treated samples at 2.5 mm and 5 mm penetrations. The soaked CBR values were obtained after 98 hours. It can be observed that the powdered glass treated samples have greater CBR values than the untreated (control) samples. In addition, both the unsoaked and soaked CBR values increase with increasing powdered glass content up to 5% by mass of the soil after which the values began to drop. Consequently, the highest CBR values for both the unsoaked and soaked treated sample of 14.90% and 112.91% respectively were obtained at 5% glass powder content, 15% cement (base) content and at 5 mm penetration.

Table 7(a). Summary of Unsoaked CBR Values for the Clay Soil Samples (Treated and Untreated).

Penetration (mm)	Control (15% cement)		1% glass + 15% cement		2% glass + 15% cement	
	Top (%)	Bottom (%)	Top (%)	Bottom (%)	Top (%)	Bottom (%)
5.0	8.48	9.17	8.54	9.63	12.27	14.33

Penetration (mm)	5% glass + 15% cement		10% glass + 15% cement		15% glass + 15% cement	
	Top (%)	Bottom (%)	Top (%)	Bottom (%)	Top (%)	Bottom (%)
5.0	14.90	10.32	14.04	14.90	10.32	14.04

Table 7(b). Summary of Soaked CBR Values for the Clay Soil Sample (Treated and Untreated).

Penetration (mm)	Control (15% cement)		1% glass + 15% cement		2% glass + 15% cement	
	Top (%)	Bottom (%)	Top (%)	Top (%)	Bottom (%)	Top (%)
5.0	89.98	88.26	94.00	89.98	88.26	94.00

Penetration (mm)	5% glass + 15% cement		10% glass + 15% cement		15% glass + 15% cement	
	Top (%)	Bottom (%)	Top (%)	Top (%)	Bottom (%)	Top (%)
5.0	109.01	112.91	63.70	109.01	112.91	63.70

4.7. Direct shear strength test

Table 8 shows the summary of cohesion and angle of internal friction values obtained from the direct shear test. These values increased as the powdered glass content was increased up to 10% by mass of the soil and decreased thereafter at 15%. The maximum values for both the cohesion and angle of internal friction of 15.0 and 17.0 respectively were obtained at 10% powdered glass content.

Table 8. Summary of Cohesion Values and Angle of Internal Friction of the Treated Soil Sample.

	Control (15% cement)	1% glass + 15% cement	2% glass + 15% cement	5% glass + 15% cement	10% glass + 15% cement	15% glass + 15% cement
Cohesion value	11.6	12.6	13.0	14.0	15.0	14.5
Angle of internal friction	9.5	11.0	13.0	14.0	17.0	15.0

5. Conclusion

This study has shown that the improvements in the properties of the clay soil obtained herein are more significant with the addition of the powdered glass. It seems that the percentage quantity of the powdered glass required achieving the best results in terms of the clay soil properties lies between 5% and 10% by mass of the soil. This is because the corresponding maximum values from both the compaction and CBR tests were obtained at 5% glass powder content while the maximum values from the shear strength test were obtained at 10% glass powder content.

Furthermore, it can be concluded based on the results obtained that powdered glass can be effectively used as a soil stabiliser since it was able to produce considerable improvements in the properties. Such improvements included an increase in the MDD value from 25.37 kN/m^3 for the control sample up to 25.90 kN/m^3 for the sample containing 5% powdered glass by mass of the soil, achievement of the highest CBR values of 14.90% and 112.91% obtained at 5% powdered glass content for both the unsoaked and soaked treated samples respectively as well as achievement of the maximum values of cohesion and angle of internal friction of 15.0 and 17.0 respectively obtained at 10% powdered glass content.

6. Recommendation(s)

It is recommended that further research should be carried out to determine the optimum amount of this additive for effective clay soil stabilisation, which apparently seems to have a value between 5% and 10% of powdered glass content. The effect of the powdered glass on other kinds of soils such as laterites should also be investigated to determine whether similar results will be obtained which will help to establish it as an all-round or general soil stabiliser.

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