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Evaluating the Effect of Agro-based Admixture on Lime-treated Expansive Soil for Subgrade Material

Abstract

An experimental study was carried out on an expansive soil treated with lime and bamboo ash (BA), an agro-based admixture, to determine its suitability as subgrade material. Preliminary tests conducted on the natural black cotton soil indicated that the soil cannot serve as subgrade material in the design of flexible pavement as the values obtained from the preliminary tests failed to meet the recommendations of the Nigeria general specification for flexible pavement design. Tests were carried out to determine the compaction and strength characteristics of the stabilised soil using an optimal 4% lime content (selected from the range of 0-10% based on plasticity index value) with various percentages of BA (4, 8, 12, 16 and 20%). After stabilisation, an optimal value of 4% lime and 12% BA, added by weight of the air dried soil showed that the compaction and strength characteristics of the soil improved significantly due to chemical actions experienced between the admixtures and the soil. Furthermore, whilst the resilient modulus (M_R) of the natural soil decreased with increasing deviatoric and bulk stresses, the M_R of the stabilised soil improved significantly with increase in deviatoric and bulk stresses. The scanning electron microscope analysis of the specimen admixed with the optimal value and subjected to 28 days curing clearly showed that cementitious compounds were formed between the admixtures and the soil. Finally, polynomial models were developed for the unconfined compressive strength and peak M_R of the stabilised soil. The obtained coefficient of determination depicted a reasonable predictive capability.

Keywords: Bamboo ash; black cotton soil; lime; resilient modulus; stabilisation

Introduction

Pavement construction on black cotton soil (BCS) used as subgrade material has often been fraught with numerous challenges because of the perennial shrink-swell behaviour associated with the soil. Results of several studies have shown that the soil has very poor engineering properties and cannot be used effectively as a construction material either as subbase or subgrade (Simon *et al.* 1975, Osinubi 2000, 2006, Lekha *et al.* 2015). It is an expansive soil that undergoes seasonal changes in volume depending on the prevailing weather condition. It exhibits swelling and shrinkage characteristics in the presence of moisture (Fredlund and Rahardjo 1993). This often leads to significant loss in shear strength of the soil, increased permeability and excessive settlement during and after the construction of buildings (Ola 1978, Osinubi *et al.* 2010, Soltani *et al.*2017, Etim *et al.* 2017). The predominant clay mineral present in the soil is montmorillonite (Morin 1971, Ola 1978, 1981, Osinubi *et al.* 2010, Mudgal *et al.* 2014) and this clay mineral, with an expandable lattice is responsible for the high level of expansiveness associated with the soil.

Due to this inherent detrimental quality of the soil, researches have been ongoing to proffer solution to remedy the weak engineering properties that characterise the soil. The volume change instability exhibited by the soil is often remedied by stabilising the soil with different types of additives to improve its geotechnical properties in an economically viable manner, so that the soil can be used adequately as subbase or subgrade material for road construction (Balogun 1991). The commonest additives used in practice for the stabilisation of soils are usually lime and cement, however, the soaring cost of these additives has led researchers to explore alternative means that are relatively inexpensive to stabilise weak soils.

Stabilisation of BCS often brings about desirable and improved properties in the soil when the right additive and techniques are adopted. The method and additive to be used for stabilisation of the soil is the prerogative of the geotechnical engineer because the soil varies in its composition. Stabilisation is used primarily to alter the properties of the soil. This alteration in the properties of the soil could be a change in the soil gradation, modification of its physical or chemical properties done with the sole intent of making the soil stable. A stabilised soil will have improved bearing capacity, shear strength, durability and permeability characteristics (El-Rawi and Awad 1981, Joel and

Agbede 2011). Due to the heterogeneous nature of soils, it is difficult to recommend a single stabiliser that can best improve the properties of the soil particularly for the construction of pavements. This has prompted researchers to attempt stabilising the soil with different additives.

A recent study on the stabilisation of BCS by Etim *et al*. (2017) using lime and iron ore tailings (IOT) as stabilisers revealed that with a combination of 8% lime and 8% IOT, there was significant improvement in the soil. A California bearing ratio (CBR) value of 50% was observed when the stabilisers were added to the BCS that was classified as A-7-6 and CH using the American Association of State Highway and Transportation Officials (AASHTO) classification system and the Unified Soil Classification System (USCS) respectively. The only drawback with the use of this admixture is in the durability of the mixture because the stabilisers did not adequately meet the specification recommended by Ola (1974). Osinubi *et al.* (2016) evaluated the strength characteristics of BCS using locust bean waste ash (LBWA), an agricultural waste produced from the incineration of locust bean husks. Different compactive efforts were used for the compaction tests and it was observed that with a percentage of between 10 to 12% LBWA content, the soil properties improved significantly. However, like the study performed by Etim *et al.* (2017), the durability test also failed to meet the stated criteria. In the study carried out by Amadi (2014) on the enhancement of durability by stabilising BCS with cement kiln dust (CKD), it was observed that by adding between 8 to 16% CKD content, the soil became durable. The specimens used for the durability tests were effectively assessed by subjecting them to a very long period of soaking. The CBR swell tests were also performed on them. Improvement in the soil properties with a higher CKD was attributed to the increase in cementitious compound formed between the CKD and the soil. AbdEl-Aziz and Abo-Hashema (2013) applied a calcined clay waste material known as Homra and lime for the stabilisation of a clayey subgrade. The results of the study showed that the additives achieved peak values of about 94% reduction in swell potential, 500% increase in CBR and 750% increase in the soil shear strength. The study further suggested that the use of homra significantly reduces the quantity of lime required for the clay subgrade stabilisation. In a similar study by Ashango and Patra (2014), the occurrence of long term reaction between Portland slag cement (PSC) and rice husk ash (RHA) was observed to improve the strength properties of a clay subgrade. The result of the study clearly depicted higher strength development with curing, as peak values of unconfined compressive strength (UCS) and CBR were observed after a maximum curing period of 30 days and additive percentages of 7.5% PSC and 10% RHA. A recent study by Ikeagwuani *et al.* (2019) utilised a combination of sawdust ash (SDA) and lime to improve the geotechnical properties of BCS for subgrade application. The results of the study revealed that a combination of 4% lime and 16% SDA optimally improved the geotechnical properties of the soil, which was attributed to the long term pozzolanic reaction that occurred between lime and SDA with curing. The occurrence of pozzolanic reaction between the additives was consolidated by the changes in the soil microfabric in the form of agglomeration and flocculation. In another recent study by Amulya *et al.* (2018), geopolymerization process was found highly efficient in term of strength and durability characteristics of the stabilised clay soil. Alkali activation was applied to leach out aluminum and silicate ions from the alumino-silicate rich fly ash (FA) and ground granulated blast furnace slag (GGBFS) used for the stabilisation process. The improved soil, using alkali activated additives, were found to be more resistant to wet-dry cycles and passed the durability test in comparison with soils stabilised with GGBFS and FA, which failed. Other applications of additives for expansive soil stabilisation have been well documented (Ikeagwuani and Nwonu 2019).

Bamboo and Bamboo Ash

Bamboo is a member of the plant family, Proaceae. A woody perennial grass plant that grows very fast, and also, takes little time to mature. It could grow to its maximum height in about three months and can be cultivated in most parts of the world, especially the temperate, subtropical as well as the tropical regions (Netravali and Pastore 2014), particularly Nigeria. There are over a thousand species of bamboo in the world. The roots of the bamboo known as rhizomes can spread rapidly producing more shoots and thus can survive for many years. This is seen as a disadvantage if it is planted in the garden. However, this very inherent and perceived negative quality of bamboo can be harnessed into good use as the bamboo can be produced in very large quantity. The stem of the bamboo plant called bamboo culm have long been used in most civil engineering construction works especially for scaffolding and props. Lately, it has been found useful in flooring known as bamboo parquet flooring. As a result of the aforementioned industrial processes, which deploy an annual quantity of about 20 million tons of bamboo (Savastano *et al.* 2012), annual production of bamboo waste becomes significant. Most of the produced bamboo wastes are discarded in landfills or sometimes burnt openly, which could be environmentally hazardous (Karade 2010, Villar-Cocina *et al.* 2011).

Various attempts have been made to utilise bamboo in various forms for ground engineering applications. The use of bamboo splints as reinforcement in rammed earth blocks/walls has been reported in literature. The utilisation of bamboo for this application has shown improvement in seismic resistance, load capacity, ductility, as well as reasonable bond strength (Gao *et al*. 2009; Tripura and Sharma 2014).

Improvement in geotechnical properties of soil beds have also been reported in the use of bamboo for soil reinforcement applications. Bamboo straw, carefully woven into planar and three-dimensional materials have been used as geocells, geotextiles, geogrids and has been shown to exhibit superior performance in terms of bearing capacity, settlement, lateral displacement, and tensile strength (Sivakumar Babu and Vasudevan 2008, Khatib 2009, Jiang *et al.* 2010, Othman 2010, Hegde and Sitharam 2015). More recent application of bamboo is in form of randomly distributed fibre inclusions used for fibre-reinforced soil applications. The application of bamboo fibre for soil reinforcement has been reported to show significant improvement in both soaked and unsoaked CBR. Maximum increments between 170-220% have been observed for varying fibre lengths and diameters (Brahmachary and Rokonuzzaman 2018). However, the forgoing discussion of the applications of bamboo in ground engineering is limited in application due to durability issues associated with bamboo as a biodegradable material and thus, requires treatment for application. Utilisation of bamboo in form of ash for soil stabilisation is a more robust approach, which not only serves to surmount the challenge associated with its biodegradability, but also encourages an environmental-friendly utilisation of bamboo waste.

Bamboo, which consist of the leaves, branches and culm, burnt into powdery form is the bamboo ash (BA) used in this study. Generating the BA from bamboo plant is not a difficult process. This explains, aside from its availability, cheapness, and environmental hazard mitigation, why it was adopted as an alternative to supplement lime in this study so that its potential as a veritable stabiliser for BCS can be effectively assessed. Its chemical properties are similar to that found in most wood, however, it contains higher percentage of both ash and silica contents than wood (Tomalang *et al.* 1980). The use of wood ash as a standalone stabiliser has not shown any promising result for the stabilisation of expansive soil (Okagbue 2007). This necessitated the combination of BA and lime in this study for BCS stabilisation.

Resilient Modulus

An inherent soil property that governs the elastic theory is the resilient modulus (Russell and Hossain 2000). It is simply the ratio of the cyclic deviator stress to the cyclic recoverable strain. Stabilised BCS is often used as subgrade material, which incidentally is a key component in pavement design. Adequate knowledge of the resilient modulus (M_R) of stabilised BCS is highly important in the design of flexible pavement. Because of the critical role it plays in pavement design, AASHTO suggested a replacement of the CBR with the M_R of the soil during design of pavement (AASHTO 1986, 1993). Interestingly, its use in the design of pavement is gradually gaining momentum in several parts of the world but in some third world countries particularly, Nigeria, the CBR method for the design of flexible pavement is still widely used. Consequently, from literature surveys, virtually no research study on M_R of stabilised subgrade expansive soil has been undertaken in Nigeria. More so, in general, only very few researches have been conducted on stabilised soils, to predict the values of the soil M_R (Mamatha and Dinesh 2017). Few studies from survey of literature have presented predictive models for M_R of stabilised soils and are subsequently discussed in brief.

Various Kernels of support vector machine were used by Heidaripanah *et al.* (2016) to predict the M_R of lime-stabilised subgrade soils. The dataset were trained using polynomial kernel model, radial basis function kernel model and linear kernel model and the respective R^2 values obtained after evaluation are 0.9821, 0.7095 and 0.7926. Mamatha and Dinesh (2017), developed a multiple linear regression model for prediction of the M_R of lime-stabilised BCS. The model equation is as shown in Equation (1).

$$M_R = \frac{K_1 \theta^{k_2} \left(\frac{\gamma_s}{\gamma_{opt}}\right)^{k_3} (CP)^{k_4} (L)^{k_5}}{\tau_{oct}^{k_6} \left(\frac{\omega_s}{\omega_{opt}}\right)^{k_7}} \tag{1}$$

Where M_R= resilient modulus; $\theta =$ bulk stress; τ_{oct} = octahedral shear stress; γ_s= unit weight; γ_{opt}= maximum unit weight; ω_s= moulding water content; ω_{opt} = optimum water content; CP=curing period in days; L=lime content in%; K_1, k_2, k_3, $k_{4,}k_5$, k_6, k_7 = regression constants. The R^2 value obtained using the model is 0.875. However, the drawback of this model is that it predicts the M_R of the soil to be zero when no curing is done. This necessitates more research on M_R of stabilised subgrade soils.

Conversely, several predictive models have been proposed to predict the M_R of natural soils. The constitutive models used for M_R prediction are often based on the state of stress of the soil. The most common models include the octahedral model proposed by Shackel (1973), the bilinear model proposed by Thompson and Robnett (1976), the semi-log model proposed by Fredlund *et al.* (1977), the power model proposed by Moossazadeh and Witczak (1981), and the hyperbolic model proposed by Drumm *et al.* (1990). The M_R of subgrade is usually affected by several factors and these include the soil physical condition, the type of soil and the current stress level of the soil (Li and Selig 1994, Wang and Li 2011). The stress levels are often represented in terms of confinement (that is confining pressure or bulk stress) and/ loading (in form of the deviatoric stress or octahedral shear stress). Determination of the M_R of subgrade is conducted by repeated load tests with a combination of varying deviator stresses and confining pressures as specified in AASHTO T307-99 (2007) guide.

In this study, the influence of confinement was represented in terms of the bulk stress, while that of loading was represented in terms of deviatoric stress. The effect of the BA additive on the aforementioned resilience characteristics was investigated in this study, in addition to other geotechnical properties of the stabilised soil. The lime used in this study was solely for modification of the natural BCS, which served as a pretreatment measure in determining the optimum lime content.

Materials and Method

Materials Used

Black Cotton Soil

The soil sample used in this study was obtained from a single location in the Ngurore - Numan area of Adamawa State in Nigeria. Numan, which is one of the local government areas in Adamawa and a semi-arid region, lies within latitude 9°29’10’’N and longitude 12° 02’36’’E of the Nigeria geographical map (Ikeagwuani 2016). A location map of the Numan area is shown in Figure 1. Samples were collected at depths of not less than 1 meter below the ground surface using the disturbed sampling method. The samples, which were in hardened and caked state because they were collected during the dry season, were pulverised using a pestle. Pulverisation of the samples was carefully done to avoid breaking the grains of the samples. The properties of the natural BCS are presented in Table 1. The BCS is classified as A-7-6 (13), using the AASHTO soil classification system and as CH soil using the USCS (Murthy, 2002). The percentage of fines obtained as 82% far exceeded 35%, which is the limit that Nigeria general specification (1997) placed on the grading requirement for subgrade. The liquid limit and plasticity index obtained were quite high and exceeded the subgrade limit of the Nigerian general specification (1997), while the soaked and unsoaked CBR values obtained are low. The Nigeria general specification (1997) recommends a soaked CBR of between 5-11%, while the unsoaked CBR should never be less than 10%. These determined properties of the soil are indications that the soil cannot serve as a subgrade material which means that the soil has to be stabilised before it can be used as subgrade material.

Bamboo

The bamboo used for this study was obtained from timber market in Nsukka local government area of Enugu state, Nigeria. Nsukka can be located on the geographical map of Nigeria using the coordinate 6°51’24”N latitude and 7°23’45”E longitude (Ikeagwuani 2016). The bamboo was then burnt under controlled conditions to produce BA.

Lime

Quicklime was used for this study. Its principal sources are rocks with calcium carbonate as their primary chemical composition (Ikeagwuani 2016). The lime was obtained from Ogige market in Nsukka town and the chemical composition is shown in Table 2.

Methodology

Production of Bamboo Ash

In the preparation of the BA, dual-phase ignition method was used. In the first phase, known as the carbonisation phase, the bamboo was collected, washed to remove any impurities and then cut into smaller sizes of about 0.4m or less to facilitate rapid burning and drying. The cut bamboo was allowed to dry under the sun for a week to remove any absorbed moisture. The dried bamboo was placed in a kiln and burnt at a temperature of 400°C. In the second stage, which is the carbon elimination stage, the carbonised bamboo was inserted into the heating chamber of the electric furnace and heated to a temperature of about 700°C until it was reduced to ash. The process described here can be scaled up to produce larger quantity of ash using a solid fuel combustion system such as the pulverised fuel combustion system, which is capable of maintaining high temperatures. Such a system is capable of sustaining high combustion temperatures of up to 600°C and basically executes solid fuel combustion via four major processes including moisture removal, thermal decomposition (which leads to the release of volatile matter), combustion of volatile matter and char combustion (Kleinhans *et al*. 2018).

The resulting BA from the heating chamber was allowed to cool for duration of 24 hours in the furnace, then brought out and sieved through BS No 200 sieve before being placed in watertight bags to prevent the ingress of moisture until usage. A small fraction of the sample was used for the determination of its oxide composition using XRF analysis. The oxide composition of the BA with that of the BCS is presented in Table 2.

From Table 2, it can be seen that both the natural BCS and the BA have a high content of silica. However, the BA has higher silica content. Using the classification by ASTM C618-12a, the requirement for the BA to be classified as a Class F pozzolana is satisfied. With silica content of 85% and magnesia less than 5%, it can be asserted that the BA has similar chemical compositions to that of silica fume (Heba 2011), a supplementary cementing material, which is sometimes used in the manufacturing of concrete to improve the strength and durability of hardened concrete.

Experimental Procedure

Atterberg's Limits: Atterberg's limits test for both the soil–lime and soil-BA mixtures were performed using the method described in BS1377, part 2 (1990). The lime was added in steps of 2% beginning from 0 to 10% by weight of the dry soil, while the BA was added in steps of 4% from 4 to 20% by weight of the dry soil. The Casagrande's apparatus was used for this experiment and the specimens were tested without curing. The mixture was mixed manually and thoroughly to achieve a homogeneous mixture, and then covered for about one hour to mellow before testing. The value of lime from the mixture with the least plasticity index was used for the compaction, strength and resilience tests in this study since the lime was solely used as a pretreatment for the soil.

Compaction: British standard light (BSL) compactive effort as described by BS1377, part 4 (1990) was employed in this study for the determination of the moisture-density relationship of the soil-BA and soil-lime-BA mixtures. Compaction of the mixtures was done in three layers with 25 blows each using the standard Proctor mould, as the mixtures were being thoroughly and manually mixed. The test was performed on the mixture immediately after mixing. The BA was added in steps of 4% starting from 0 to 20% by weight of the air-dried soil sample for both the soil-BA and soil-lime-BA, while for the soil-lime-BA mixtures, the lime content was kept constant at the percentage obtained from the Atterberg's limits test that produced the least plasticity index.

California Bearing Ratio: This test was performed in accordance with BS 1377, Part 4 (1990). Both soaked and unsoaked CBR were carried out in this study using BSL compaction at optimum moisture content (OMC) and maximum dry density (MDD). The soaked CBR was carried out to determine its worst possible condition in the field. Curing of the specimens used for the CBR test was done for 6 days in a humidity controlled room and they were immersed in water for 24 hours at room temperature. The curing period used was in accordance with the requirements of the Nigeria general specification (1997). The admixtures were also added in a similar fashion to that of compaction for soil-lime-BA mixtures.

Unconfined Compressive Strength: Determination of the UCS was done in accordance with BS 1377 part 4 (1990). Manually and thoroughly mixed soil-lime-BA mixtures were used for the UCS. The specimens, which were compacted using BSL compaction at OMC and MDD, were cured for 7, 14 and 28 days in a humidity controlled room.

After curing, the samples were subjected to axial load in the UCS test machine in the laboratory. To obtain accurate and reliable results, the numbers of specimen used were increased to four for each experiment and the average values of each of the mixtures were noted.

Durability: Determination of the durability of the sample was carried out by using the method described by Ola (1974) because it is favourable for tropical regions, unlike the ASTM (1992) method which specifies the wet-dry and freeze-thaw test that are suitable for the temperate region (Ola 1974, Osinubi *et al.* 2016). The durability test was performed by measuring the resistance to loss in strength and this was obtained by finding the ratio of the UCS of specimens that were wax-cured for 7 days, then dewaxed top and bottom and later immersed in water for duration of 7 days to that of specimen wax-cured for 14 days (Osinubi 2016, Etim *et al*. 2017).

X-ray Fluorescence: The ARL-XRF Advantx 1200 model was used to determine the oxide composition of the natural BCS and the BA. Preparation of the samples was done by adding a binder, BORAX, in the ratio of 4:1. The binder was mixed thoroughly with the sample. An even and uniform mix was obtained by placing the mixture in a Herzog vibrating cup miller whose rotating speed was 8rpm. The mixture was then loaded onto an aluminium cup that was later placed in a pelletising machine. The machine was operated by allowing upward and downward movement at 6rpm. Once pelletisation of the mixture was achieved, it was removed and placed into the cassette of the XRF equipment. The cassette was locked manually by turning it in a clockwise direction. This action prevents the mixture to be analysed, from falling off or being scattered in the goniometer when the analysis is going on. Both the cassette point and loading point were in a position facing the goniometer position and this was done for ease of analysis.

Once the analysis was completed, which is usually in about twenty minutes, the raw data were collated automatically and this was followed by the manual results.

Resilient Modulus: The M_R was determined in accordance with AASHTO-T307-99 (2007) using the repeated load triaxial test. Cylindrical soil specimens of 100mm height by 50mm diameter were prepared at the OMC and MDD. After the mixing of the soil and admixtures, the prepared samples were cured for 24 hours to ensure even distribution of moisture before testing as specified for fine-grained soils by the Long-term pavement performance protocol P46 (LTPP 1996). The samples were preconditioned with 1000 cycles using deviatoric stress of 27.6kPa and confining pressure of 41.4kPa (1000 cycles was chosen to better simulate the event occurring between compaction and heavy traffic loading in the Nigerian condition). Each cycle consists of load duration of 0.1sec followed by a cycle duration usually called rest period of 0.9sec. Different combinations of five deviatoric stresses and three confining pressures were applied at 100 cycles for each sample for a total of 15 sequences and this was adopted in the test. The last five cycles were obtained and averaged to calculate the M_R of the sample. The tests were ended once the total vertical permanent strain exceeded 5%.

Results and Discussion

Plasticity Characteristics

Atterberg's Limits

The result of the Atterberg's limits of the natural BCS stabilised with various percentages of lime is shown in Table 3 for determination of the optimum lime content. It was observed that the specimen with 4% lime content gave the least plasticity index value. As the lime was added to the soil, an exothermic reaction between lime and the soil takes place. This is brought about by the di-valent calcium ions (Ca^{2+}) from the lime replacing the ions which are not strongly held in the diffuse double layer in what is known as cation exchange (Ola 1977; Firoozi *et al.* 2017). The reaction can be represented as shown below

$CaO + H_2O \longrightarrow Ca(OH)_2$

$Ca(OH)_2 \longrightarrow Ca^{2+} + 2(OH)$

$Ca^{2+} + OH^{-} + SiO_2 \longrightarrow CSH$

$Ca^{2+} + OH^{-} + Al_2O_3 \longrightarrow CAH$

The thickness of the double layer and the zeta potential tends to decrease according to O' Flaherty (1988). The effect results in the flocculation as well as the agglomeration in the soil-lime mixture into coarse granular particles, making them more friable and workable and as a result, compaction becomes convenient. This result is as expected since it is already known that lime reduces the plasticity index of expansive soils through short term cation exchange capacity (Diamond and Kinter 1965, O'Flaherty 1988, Boardman *et al.* 2001, Firoozi *et al.* 2017). This result is in agreement with that of other researchers (Wubshet and Tadesse 2014, Ikeagwuani 2016) who also worked on stabilisation of BCS using lime.

Furthermore, the effect of BA addition on consistency limits of the natural BCS is presented in Figure 2.The trend shows an abrupt drop in both the liquid limit and plastic limit, before an overall increase occurred. The initial drop can be attributed to the desiccation of water absorbed by the natural soil for hydration reaction in the presence of the additive. However, the subsequent observed increase was due to increased water requirement needed for coating and adequate lubrication of fine particles of the BA with higher surface area. These changes in liquid limit and plastic limit caused a continuous drop in the plasticity index, which ultimately modified the soil from clay of high plasticity to low plasticity clay. The result obtained for the soil-BA mixtures are satisfactory, based on the requirements of the Nigerian general specification (1997), for suitable subgrade material which stipulates maximum values of liquid limit and plasticity index at 50% and 30% respectively.

Compaction Characteristics

Figures 3 and 4 show the relationship between the various percentages of the additives with OMC and MDD.

Optimum Moisture Content

The natural BCS when compacted was found to have OMC of 18%, which is relatively high. This high OMC, which was observed, is because of the moisture being absorbed by the soil, and this resulted in its swelling. This swelling is expected since it is well known that BCS is an expansive soil with montmorillonite as the dominant clay mineral (Ola 1978, 1981, Mudgal *et al.* 2014). For soil-BA mixtures, there was a steady increase in the OMC, with higher BA percentages. This is due to higher water requirement during hydration for proper coating and lubrication of particles of the soil-BA mixtures with higher surface area for formation of a homogenous mix.

However, for the soil-lime-BA mixtures; with the addition of 4% BA plus 4% lime, which were added by weight of the dry soil sample, an initial decrease in the OMC was observed from the curve. This reduction in OMC could be attributed to the self-desiccation process of the soil-lime-BA mixtures in exhausting the absorbed water in the soil during the hydration reaction due to adequate supply of Ca^{2+} from the lime, with attendant drop in moisture level. The resulting effect is partial saturation of the clay soil peds, which became prevalent in form of an abrupt decrease in moisture level causing the soil OMC to drop lower than that of the natural BCS in which the initially absorbed water was still intact. But on further addition of BA, an increase in the OMC was observed, which can be attributed to the increased water requirement needed for hydration reaction. The increase was marginal owing to the soil-lime-BA mixtures, gradually forming resistance against moisture to prevent further penetration of water thus increasing the OMC only slightly, due to the BA filler effect on the flocculated and agglomerated particles. The trend is consentient with previous studies (Osinubi *et al.* 2009b, Ikeagwuani 2016).

Maximum Dry Density

It is pellucid from the trend in Figure 3 that the MDD of soil-BA mixtures gradually dropped with increase in BA content. This reduction could be attributed to replacement of the soil particles by particles of BA, which are lighter and finer (with higher surface area), which required more water for proper coating and lubrication of the soil-BA mixtures (as evident from the OMC trend) in order to form a homogenous mix. In addition, the soil-BA particles only aggregated, without formation of friable flocs which makes compaction effective. The trend is similar to that obtained by Ikeagwuani (2016), who used sawdust ash for stabilisation of BCS.

On the other hand, the curve in Figure 4 shows that the value of the MDD increased from 15.1kN/m^3 to a peak value of 17kN/m^3 at 12% addition of BA. After the peak MDD was attained, its value began dropping gradually. The gradual increase in MDD can be explicated in terms of the BA filler effect and the short term cation exchange process that occurred, with adequate supply of higher valence cation from the lime. This ultimately results in the flocculation as well as the agglomeration of the soil-lime-BA mixture into coarse granular particles, which are more friable and workable, making compaction expedient with a resultant increase in the MDD achieved. The BA contributed to the increase in the MDD by filling up the voids created due to flocculation and agglomeration. The drop in MDD could be adduced to the completion of the cation exchange process within the soil hydrous layer due to exhaustion of the available lime for floc formation in the soil-lime-BA mixtures. The trend is in consonance with previous works done by other researchers (Osinubi *et al.* 2016, Ikeagwuani *et al.* 2017).

Strength Characteristics

California Bearing Ratio

The result of both the soaked and unsoaked CBR test conducted is presented in Figure 5. The curves show a similar trend for both soaked and unsoaked CBR values. The CBR for the soaked specimen increased from an initial value of 4.2% for the natural soil, to attain a peak value of 8.8%, which is about 110% improvement; while the unsoaked CBR increased from an initial value of 8.2% for the natural soil to a peak value of 15.6%, which is about 90% improvement after treatment with the additives. The CBR values increased with increase in BA content, attaining its peak value when 20%BA plus 4% lime were added by weight of the air-dried soil. These peak values satisfy the condition for use as a subgrade material based on the requirements in the Nigeria general specification (1997).

The increase in the values of the CBR could be adduced to the improved soil gradation due to the formation of a tough and coarse granular soil structure in form of friable floccules (which resulted from the cation exchange process during hydration), as well as the BA filler effect. This ensured that the soil-lime-BA mix achieved higher MDD, which impacted on the mechanical strength of the natural BCS. The soil-lime-BA matrix structure was ultimately transformed into a tough water-resistant granular skeleton, capable of withstanding higher load (Firoozi *et al.* 2017).

In addition, the presence of moisture with adequate supply of calcium and silica from the additives for the soil-lime-BA mixtures in the CBR test ensured that the pozzolanic reaction occurred progressively (Al-Mukhtar *et al.* 2012) and higher values of CBR were obtained. This result is also in agreement with those of other researchers (Osinubi *et al.*2011, Osinubi *et al.*2012, Wubshet and Tadesse 2014, Karatai *et al.* 2016, Osinubi *et al.* 2016), who have also worked on the stabilisation of BCS using agro-based and traditional chemical additives.

Unconfined Compressive Strength

Figure 6 shows the variation of the UCS with varying percentages of soil-lime-BA mixture subjected to different curing ages (7, 14 and 28 days). The curves show a similar trend, in which there was a gradual build up of the UCS of the soil with increasing percentages of the admixture for all curves until it got to a fixation point. The peak was attained at 12% BA content, after which there was a marginal drop.

The increase in strength observed could be attributed to the cementitious compounds (Diamond and Kinter 1965, O'Flaherty 1988, Firoozi *et al.* 2017), calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH), responsible for the binding properties observed in the mix that were formed within the mixture due to long-term pozzolanic reaction.

With higher curing days, the strength improved further as with time, more cementitious bonds are formed. However, the drop observed after the fixation point could be attributed to full consumption of calcium in the soil-lime-BA mixture, which is required to sustain the pozzolanic reaction. The trend is in agreement with the results obtained by Osinubi *et al.* (2009a), Anupam *et al.* (2014) and Etim *et al*. (2017).

Durability

Variation of resistance to loss in strength of the soil-lime-BA mixture is shown in Figure 7. According to Ola (1974), the maximum allowable loss in strength for specimens with 7 days curing in dry and then soaking for another 4 days is 20%. However, the specimens used for the durability assessment in this study were soaked for 7days as recommended by the Nigeria general specification (1997). The recorded resistance to loss in strength for all specimens unfortunately fell short of the 80% requirement. However, the specimen with 12% BA and 4% lime content had the optimal value of 39%, which shows a promising result because the soil was cured for 7 days soaking as against the 4 days soaking period recommended by Ola (1974). The trend is similar to that reported by Etim *et al*. (2017)

Resilience Characteristics

The resilience behaviour of the soil was represented in terms of deviatoric and bulk stress levels, since their models are recommended for the design of flexible pavement according to AASHTO guide (Georgees *et al.* 2018).

Effect of Stabilisation on Variation of Resilient Modulus with Deviator Stress

The variation of M_R with deviatoric stress, for various soil-lime-BA mixtures and at varying levels of confinement is presented in Figures 8 to 10. The figures show a similar trend for all confining pressures applied, and some key observations are pertinent from the figures. It is perspicuous that the M_R of the natural soil sample exhibited a monotonic decreasing sequence with increase in deviatoric stress. The respective values range between 178 to 123MPa, 154 to121MPa and 149 to 111MPa for confining pressures of 41.4kPa, 27.6kPa and 13.8kPa. However, on stabilisation, the M_R increased prosaically with increase in deviatoric stress and BA content, attaining its peak range of 227 to 278MPa, 231 to 269MPa and 218 to 258MPa for confining pressures of 41.4kPa, 27.6kPa and 13.8 kPa, respectively, at 12%BA content. This can be regarded as the stabilisation fixation point since the M_R ranges dropped gradually for higher percentages of BA content as shown in Figure 11.

The observed trend can be explicated in terms of the chemical changes which occurred in the soil-lime-BA mixtures. The strength development can be categorised in terms of early strength gain due to cation exchange process and minimal time-dependent strength gain due to pozzolanic reaction (Kang *et al.* 2014). As the admixtures are added to the soil in the presence of moisture, the divalent Ca^{2+} and trivalent Al^{3+} replaces lower valence ions present in the soil hydrous layer via cation exchange, which causes a compression of the diffuse double layer. This results in flocculation of particles as they become friable with increased pore surface tension, which increases the soil particles' cohesion, with resultant effect of improved workability and early strength gain (Kang *et al.* 2014). In addition, the BA filler effect on the friable floccules ensured that a higher density was imparted to the soil-lime-BA mixtures with the resultant early strength gain.

Pozzolanic reaction which causes long-term strength gain is dependent on the rate of formation of CSH and CAH gels, which are the major compounds responsible for the formation of a denser soil mix. However, the contribution of pozzolanic reaction on the increment of the soil M_R can be considered minimal as the soil samples were only cured for a day. Hence, enhanced strength gain due to higher curing period could not occur. Notwithstanding, the stabilised soil exhibited a stress hardening effect with incremental loading, as against the stress softening effect demonstrated by the load-induced failure trend of the natural soil. The drop in resilient behaviour, observed after the fixation point is attributable to drop in MDD achieved at higher percentages of BA addition in the soil-lime-BA mix, and consequently led to loss in resilience. The effect of stabilisation on variation of M_R with deviatoric stress obtained in this study is highly similar to that of Rasul *et al.* (2017), who applied cement and a combination of cement and lime as stabilisers on A-7-5 soil.

Effect of Stabilisation on Variation of Resilient Modulus with Bulk Stress

The overall effect of the stress state of the soil was represented in terms of the bulk stress, which is a function of both the deviator stress and the confining pressure. The specimen used for the M_R test was a cylindrical specimen, and for a cylindrical specimen, the bulk stress is given as shown in Equation (2)

$$\theta = \sigma_d + 3\sigma_3 \tag{2}$$

Where σ_3= confining pressure, representing the minor principal stress; and σ_d = deviator stress.

The variation of M_R with bulk stress, for various soil-lime-BA mixtures and at varying levels of confinement is represented in Figures 12 to 14. The figures show a similar trend for all confining pressures applied, and some key observations are vivid. The natural soil sample showed a straightforward drop in M_R with increase in bulk stress, which is pellucid from the figures and the range is same as that obtained for the deviator stress at all confinement levels. On stabilisation, the M_R rose monotonically with increase in bulk stress and BA content for all levels of confinement, attaining peak range equivalent to that obtained for the deviator stress at 12%BA content. This can also be regarded as the stabilisation fixation point since the M_R ranges dropped gradually for higher percentages of BA content and attributable to the drop in MDD achieved at higher percentages of BA as aforementioned.

The improvement in resilient behaviour with bulk stress and stabilisation can be explicated in terms of the enhanced stiffening effect imparted to the soil-lime-BA mixtures with cell pressure confinement. The enhanced stiffening is purportedly achieved due to the filler effect of the BA. This imparts a higher density, which causes minimisation of the recoverable deformation (Georgees *et al.* 2018) and thus yields higher resilient modulus for the stabilised soil. This increment in dry density is associated with a corresponding drop in moisture content. In view of this, studies on stabilised soils have presented results, which indicate that M_R improves at drier conditions. Tastan *et al.* (2011) obtained higher values of M_R for samples prepared at

OMC in comparison with those prepared at wet of OMC for soils stabilised with flyash, while Qian *et al.* (2014) reported decrease in M_R with increase in moisture content of lime stabilised soils. The observed trend for the stabilised soil is in consonance with that of Georgees *et al.* (2018).

Scanning Electron Microscope Analysis of Materials

The scanning electron microscope analyses of the materials used in this study are shown with their magnification in Figure 15. The micrograph of the natural BCS shown in Figure 15a reveals that the grains are made up of fine rounded particles in caked state, as the samples were collected during the dry season. These particles have void spaces within the interstices. Figure 15b, which is the micrograph of the BA, shows that BA is also made up of fine rounded particles like the BCS, but the particles are triturated and finer than the natural BCS used in this study. The particles are dispersed and loosely packed together. Figures 15c-15e show the stabilised BCS with 12% BA and 4% lime content after 28 days of curing at various image magnifications. The clumping of the mix together shows that cementitious products were formed between the soil and the admixtures. Consequently, a reduction in both intra and inter-aggregate pore sizes occurred. This resulted in the flocculation and agglomeration that took place in the mix after curing for 28 days.

The variation of CaO/SiO_2 for the stabilised soil after 28 days curing is shown in Figure 16. The trend shows an increase in CaO/SiO_2 with BA addition until the peak was attained at 12%BA, which is the stabilisation fixation point. The trend is associated with pozzolanic reactions which led to the formation of cementitious compounds as shown in the SEM results. Tastan *et al.* (2011) showed that increase in CaO/SiO_2 for expansive soils can be correlated with higher values of UCS and M_R. The trend in Figure 16 is consentient with that reported by Tastan et al (2011).

Phenomenological models

In order to describe the trend of the UCS results, the variation was approximated with a model. It is clear from UCS results that both curing time and BA content had significant effect on the trend. In view of this, phenomenological models were used as adopted by Jahandari *et al.* (2017) and Saberian *et al.* (2017), who respectively applied quartic and quadratic polynomials to describe the geotechnical properties of stabilised soils.

In this study, a degree two polynomial was obtained to best approximate the UCS and M_R of the stabilised soil through trial and error. The UCS was found to be dependent on the product of curing days (CD) and BA content (%), while the peak M_R was a function of BA content (%), and are both represented by Equations (3) and (4), respectively.

$$\mathrm{UCS} = \mathrm{a} \times (\mathrm{CD} \times \mathrm{BA})^2 + \mathrm{b} \times (\mathrm{CD} \times \mathrm{BA}) + \mathrm{c} \tag{3}$$

$$\mathrm{M_R} = \mathrm{d} \times (\mathrm{BA})^2 + \mathrm{e} \times (\mathrm{BA}) + \mathrm{f} \tag{4}$$

Where a (kPa/day^2), b (kPa/day), c, d, e, f (kPa) are all fitting parameters.

The quadratic functions, representing the relationships are as shown in Figures 17 and 11 for UCS and M_R respectively. From the figures, all the fitting parameters required for predicting the UCS and M_R have been determined. Furthermore, it can be seen that the UCS was fitted with R^2 value of 0.8023, while the M_R was fitted with R^2 value of 0.9955, representing fair and good predictive capability respectively, within the limits of the model input parameters. The implication of the R^2 value of 0.8023 for the UCS is that about 20% of the total variation is not accounted for by the model.

Conclusion

The experiments conducted in this study have shown that the natural BCS classified as A-7-6 using the AASHTO classification system had weak engineering properties and therefore cannot be used in the design of flexible pavement whether as subgrade, subbase, or even as a base course material. The study further revealed that with BA, which acted as a supplementary cementing material to the lime-treated BCS, there was significant improvement in the strength, compaction and resilience characteristics of the soil. However, the BA was unsuitable as a sole stabiliser based on the reduction in MDD recorded for the soil-BA mixtures. An optimal value of 4% lime and 12% BA added by weight of the air-dried soil was found suitable for the stabilisation of the soil to be used as subgrade material and then, polynomial models were developed to predict the UCS and M_R. The following salient points summarise the findings of this study.

- On addition of BA and lime, the compact nature of the soil greatly enhanced and improved the mechanical strength of the soil due to time-dependent chemical reactions which led to the formation of cementitious compounds, as suggested by the SEM.
- The improved denser soil matrix exhibited tremendous resilience with increase in both deviatoric and bulk stresses at various levels of confinement. The improvement is attributable to the short term cation exchange process due to the higher valence ions supplied by the admixtures, the BA filler effect and then slightly, due to pozzolanic reaction which increased the soil M_R at higher stress levels, as against the load induced failure which deteriorated the natural soil M_R under imposed stresses.
- The chemical changes which led to the formation of cementitious compounds and improved the strength characteristics of the soil were most plausible at 12% BA content, which exhibited the greatest resistance to strength loss.
- It is evident from the results of this study that BA had a positive influence in improving the performance of the understudied expansive soil as a subgrade

material due to its filler effect and also its contribution as silica amendment for cured specimens based on CaO/SiO_2 ratio of the stabilised soil.

- Finally, quadratic polynomial functions were found to be most plausible to model the behaviour of the UCS and peak M_R of the stabilised soil, with R^2 values of 0.8023 and 0.9955 for the UCS and M_R, which represent fair and good model fit respectively. With R^2 value of 0.8023, 80% of the variation in the UCS is accounted for by the model.

Declaration of Interest Statement

The authors declare that there are no known conflicts of interest that influenced this research work.

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Table 1. Geotechnical Properties of the Natural Soil Sample

Soil Property	**Sub-division**	**Description**
Specific gravity		2.75
Sand		18%
Fines	Silt	21%
	Clay	61%
Natural moisture content		12.4%
Liquid limit		70.4%
Plastic limit		24.9%
Plasticity index		45.5%
Shrinkage limit		14%
Optimum moisture content		18%
Maximum dry density		15.1kN/m^3
AASHTO classification		A-7-6 (13)
USCS classification		CH
California bearing ratio	Unsoaked	8.2%
	Soaked	4.2%
Unconfined compressive strength	7 days	400kPa
	14 days	750kPa
	28 days	1200kPa
Colour		Black
pH		7.01

Table 2. Chemical Composition of BCS, BA and Lime

Compound	Soil (%)	BA (%)	Lime (%)
Silicon oxide (SiO_2)	64.3	85	-
Iron oxide (Fe_2O_3)	3.6	0.56	-
Aluminium oxide (Al_2O_3)	5.1	6.56	-
Phosphorus oxide	-	0.30	-
Sodium oxide	1.06	-	-
Potassium oxide	1.52	5.30	-
Calcium oxide (CaO)	5.67	0.34	56.08
Magnesium oxide	2.31	0.64	≤0.5
Titanium oxide	1.86	-	-
Manganese oxide	14.79	0.48	-
Nitrogen oxide	-	-	≤0.004
Sulphur oxide	-	0.20	≤0.10
LOI	1.87	-	≤2
Minimum assay (after ignition)	-	-	98
Heavy metal (Pb)	-	-	≤0.005
Chloride	-	-	≤0.003
Iron	-	-	≤0.015
Ammonium precipitate	-	-	≤0.2
Acetate acid insoluble matter	-	-	≤0.05
Transparency test	-	-	Passed

Table 3. Atterberg's Limits for Soil-Lime Modification

Lime (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)
0	70.4	24.9	45.5
2	74	36.1	37.9
4	58	48.2	9.8
6	64	48.3	15.7
8	70	24.1	45.9
10	78.4	23.2	55.2

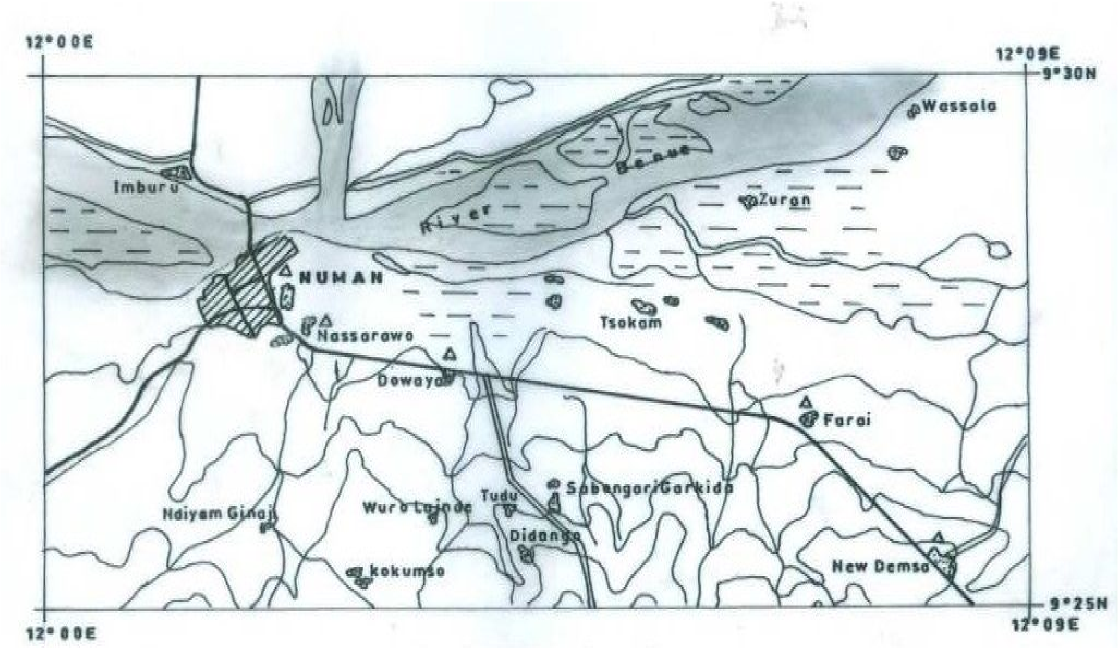


Figure 1

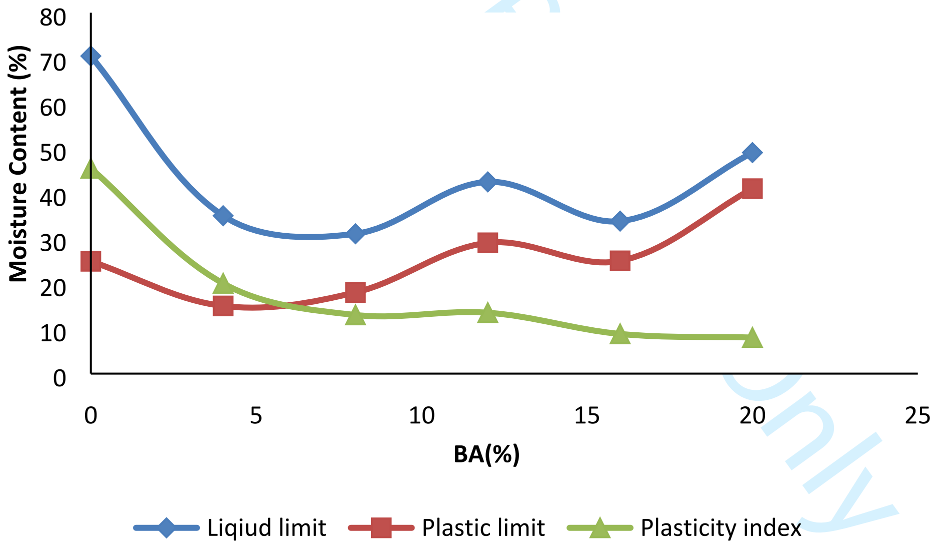
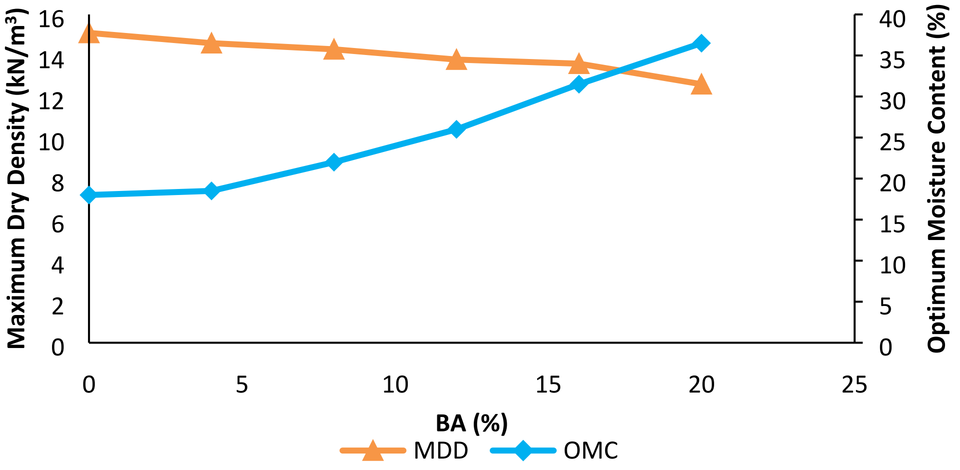


Figure 2



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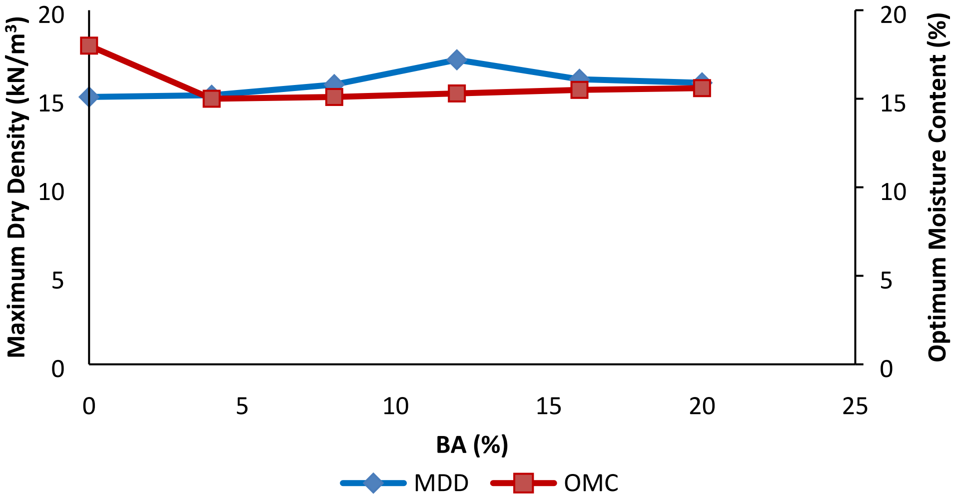

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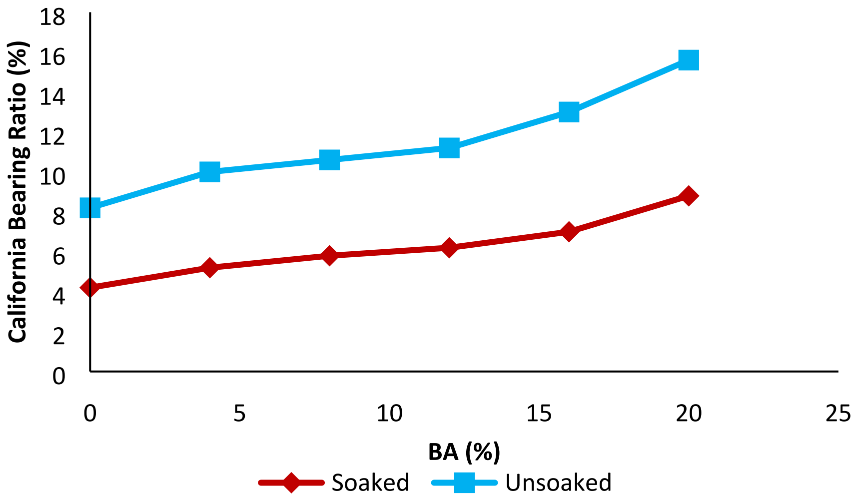

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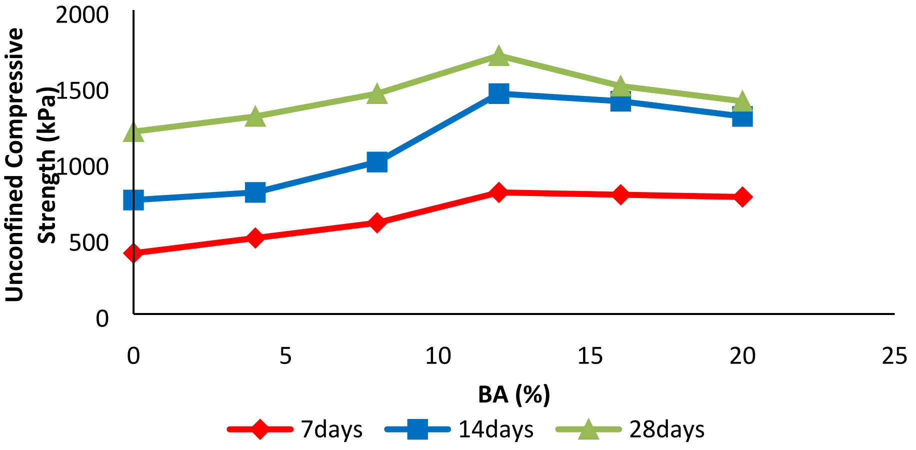

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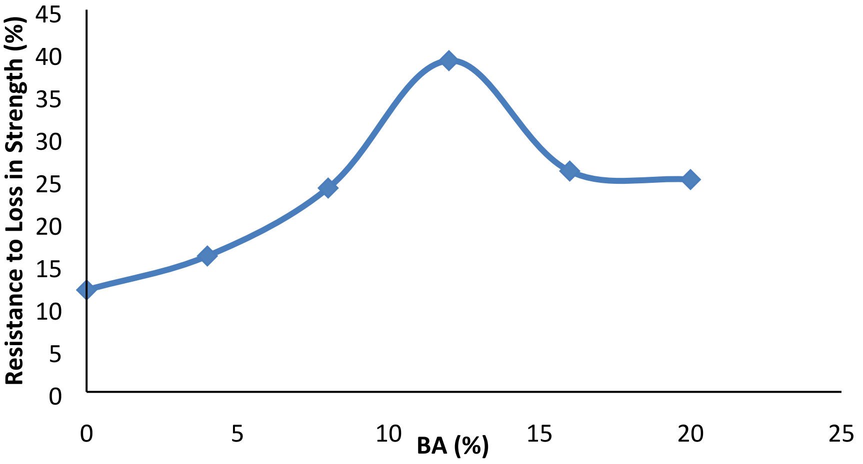

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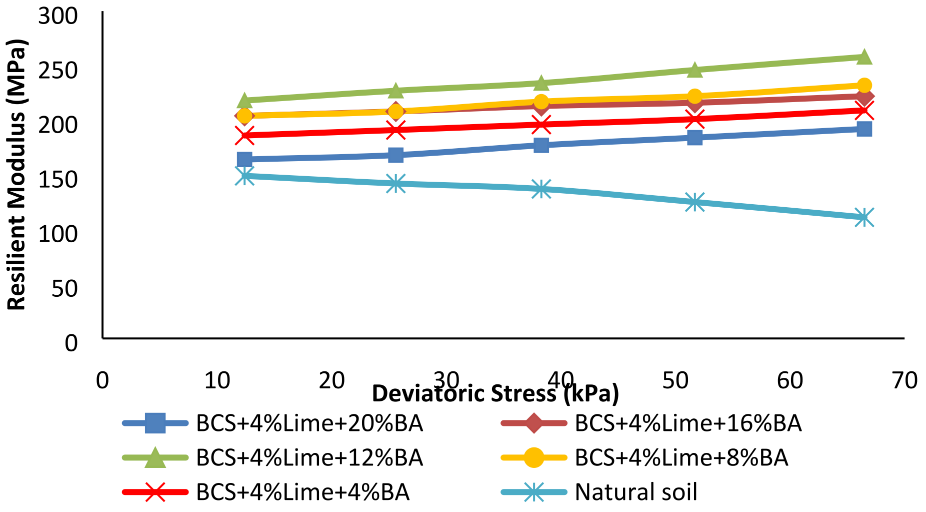

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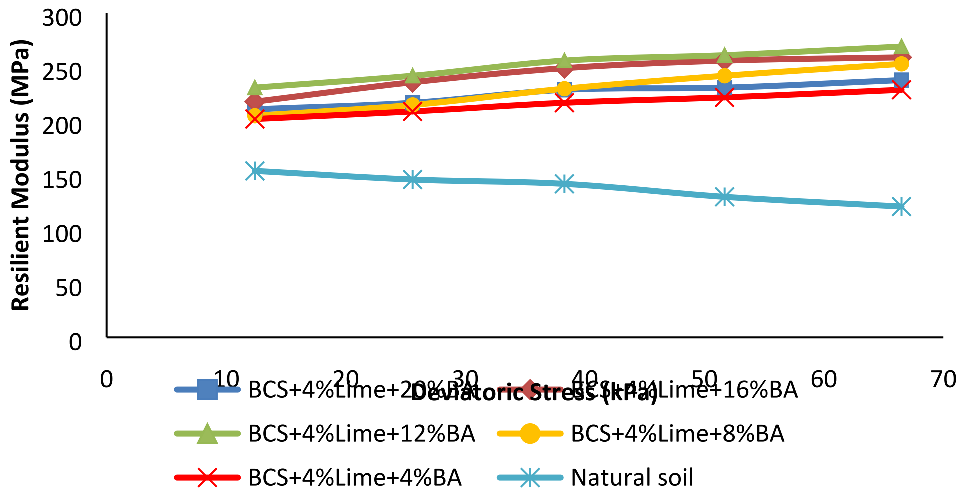

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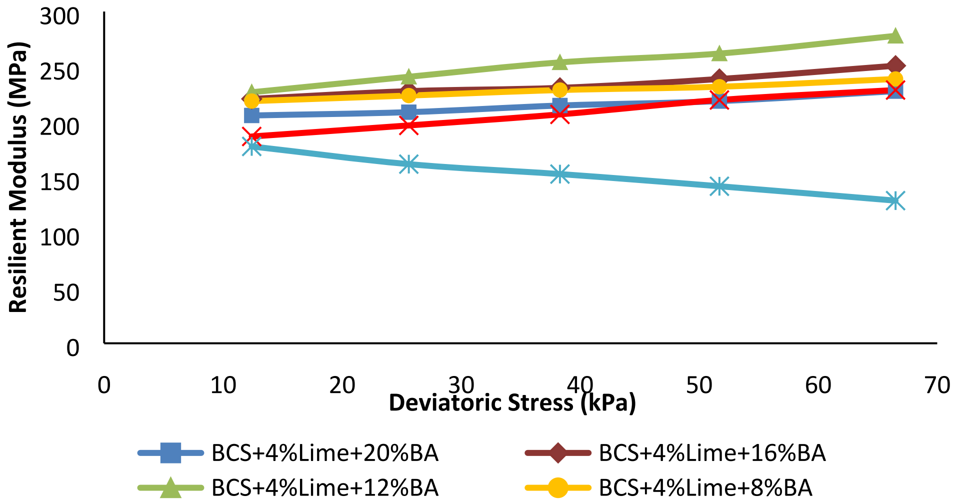

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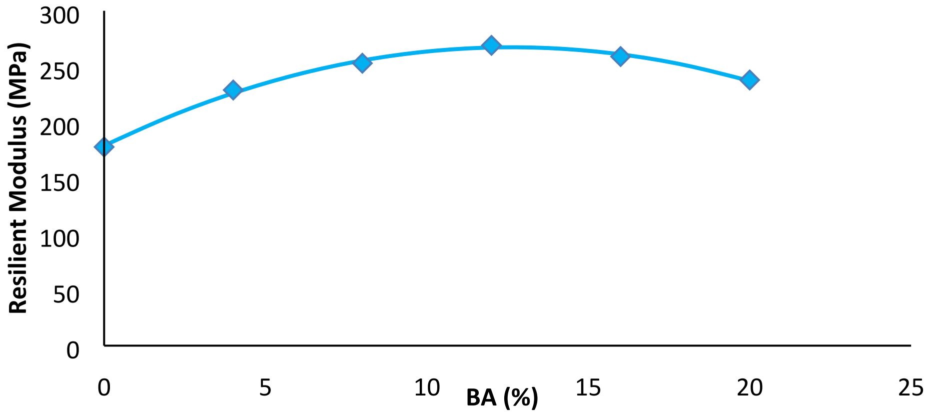

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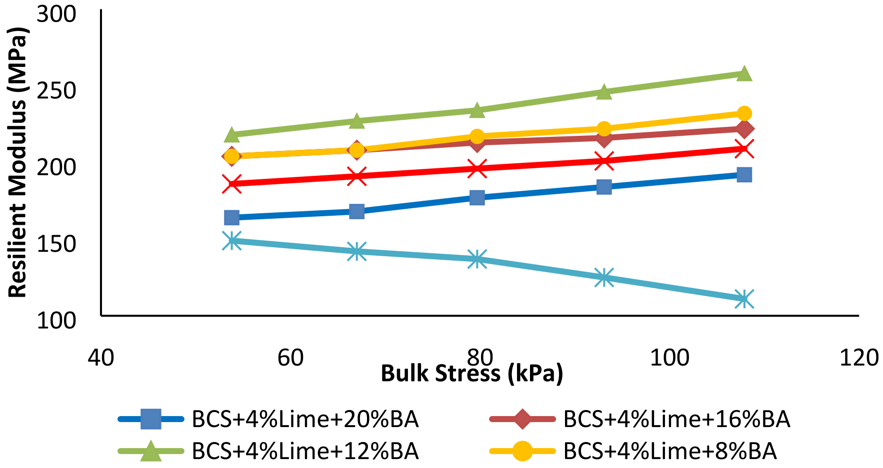

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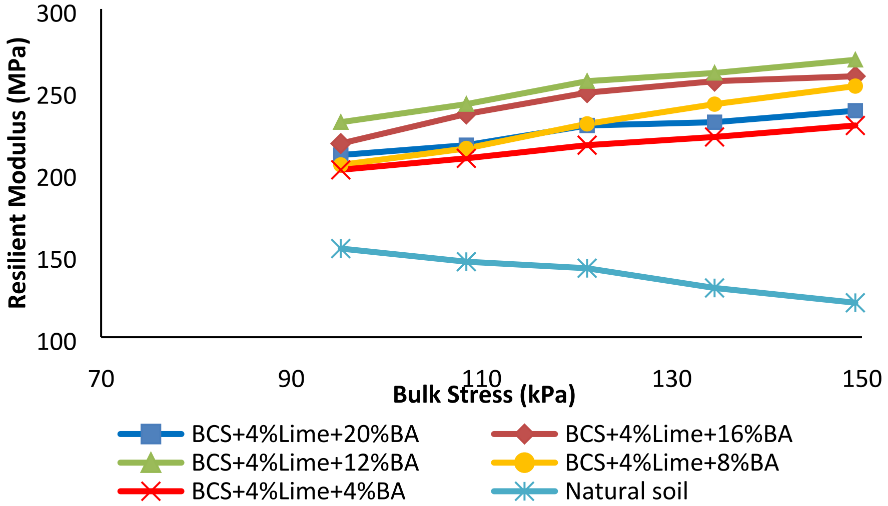
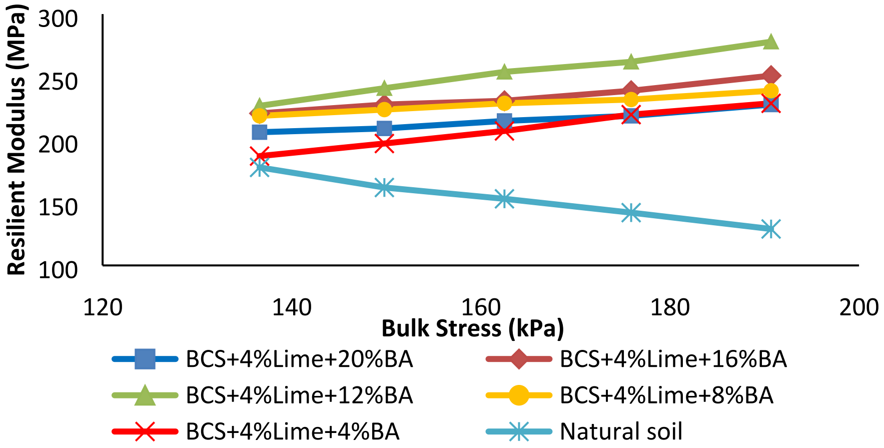


Figure 13



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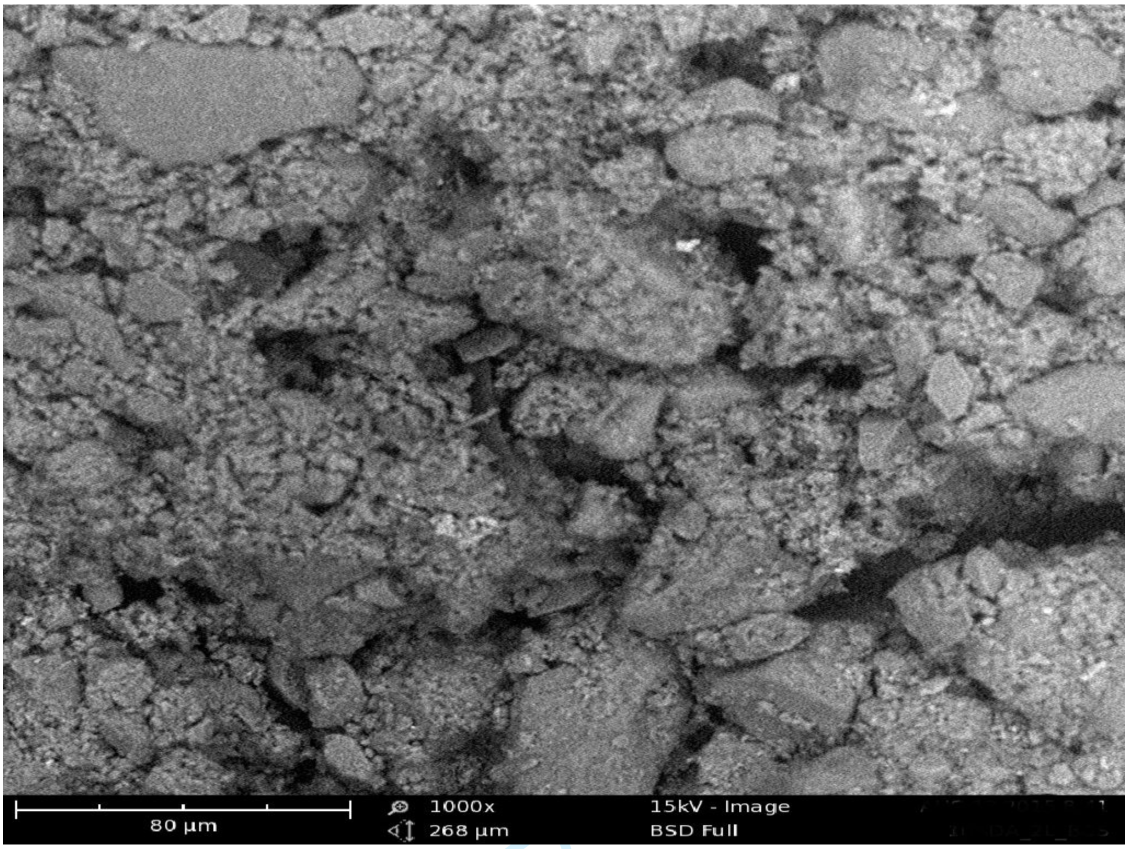
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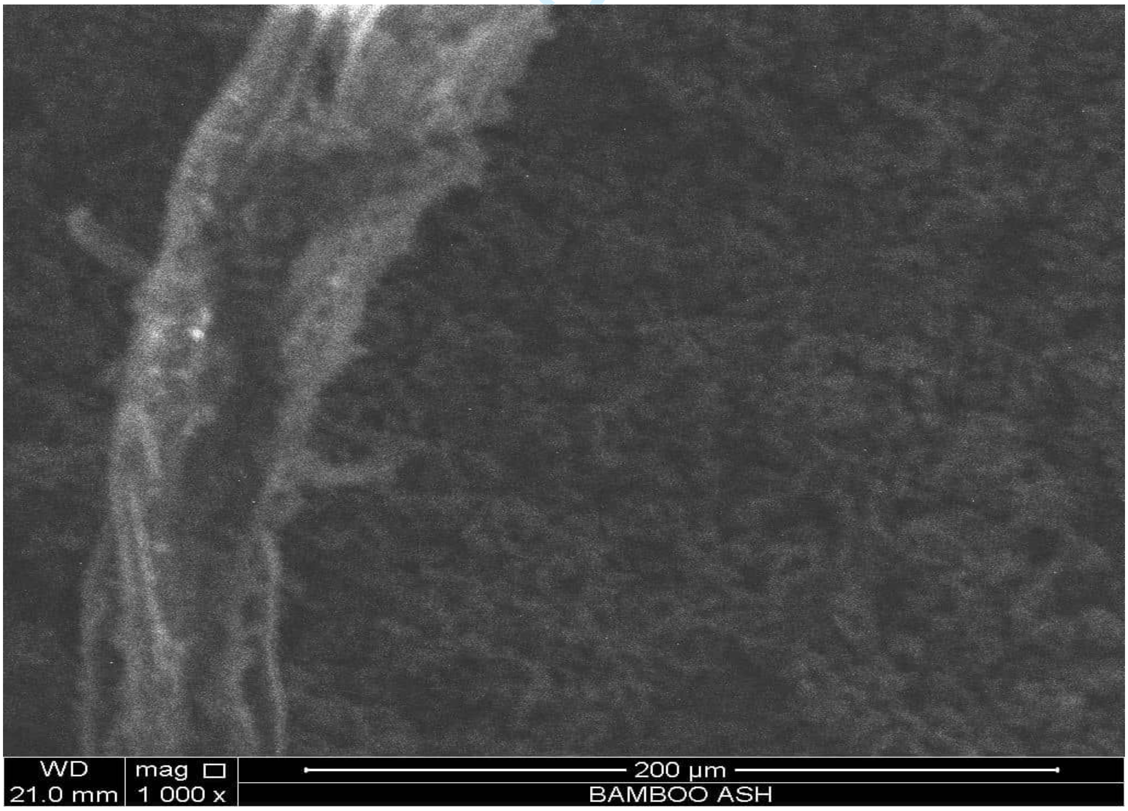
1000x
268 µm
15kV - Image
BSD Full

(a)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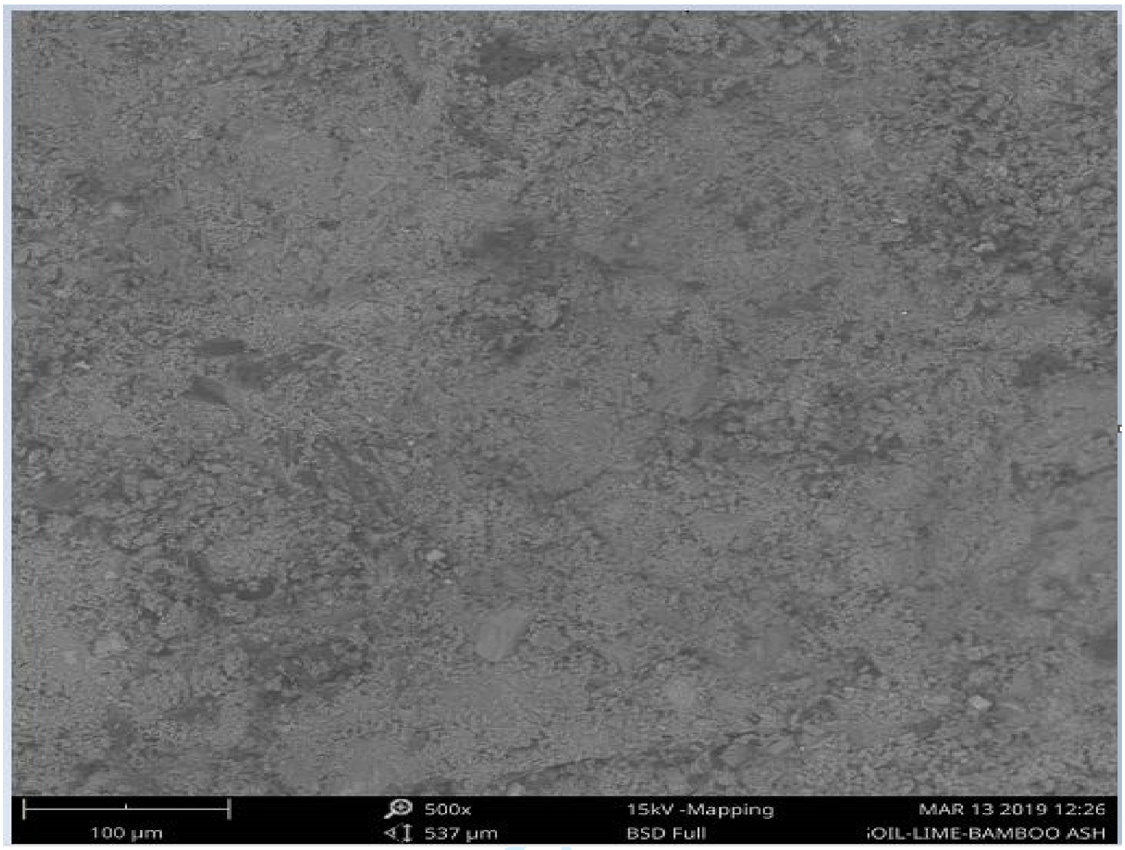
mag □
1 000 x
200 µm
BAMBOO ASH

(b)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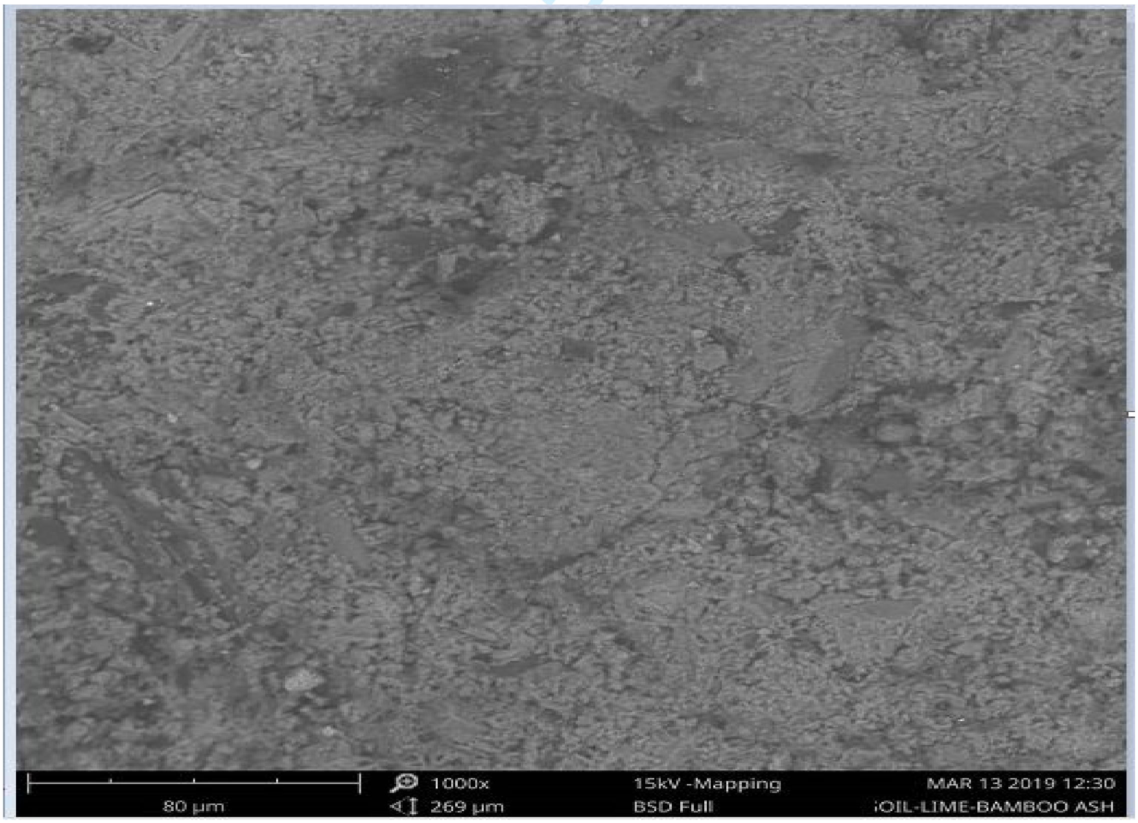
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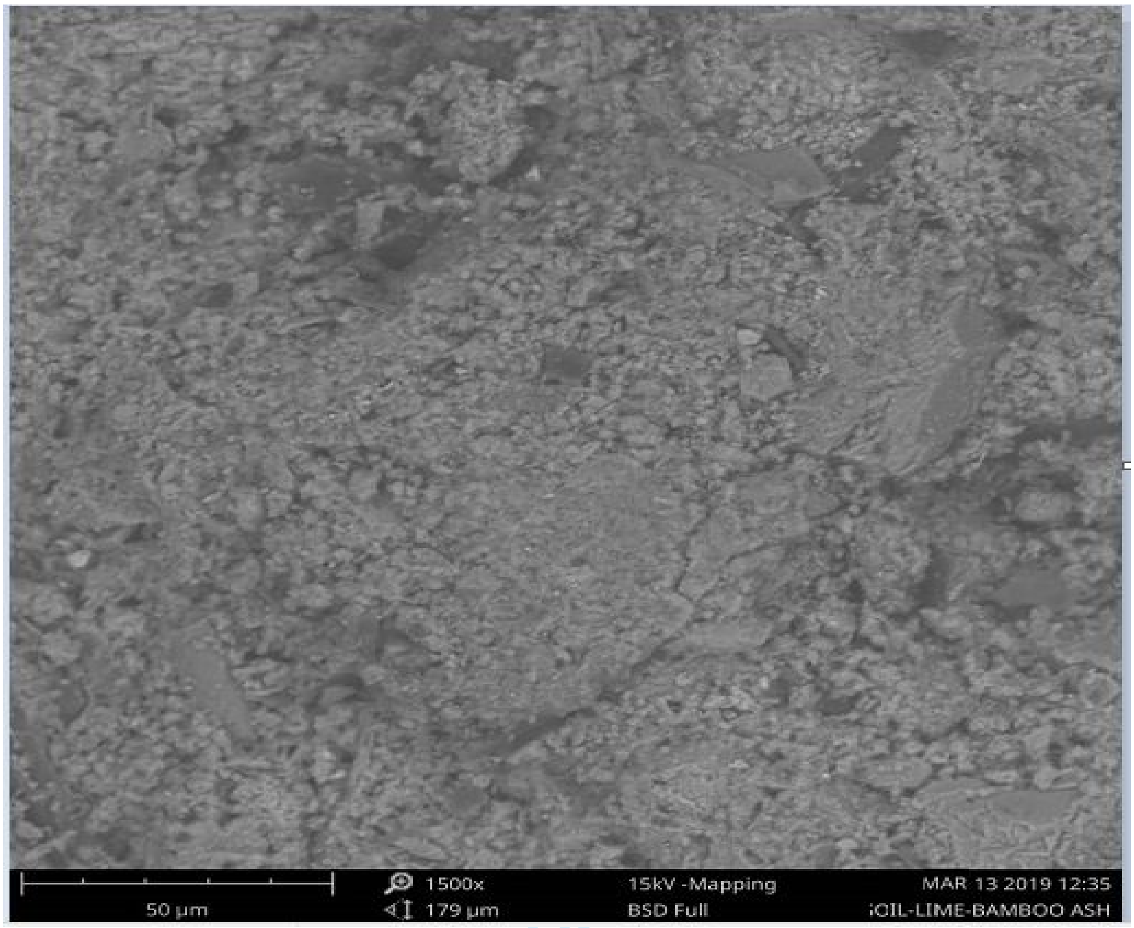

(e)

Figure 15

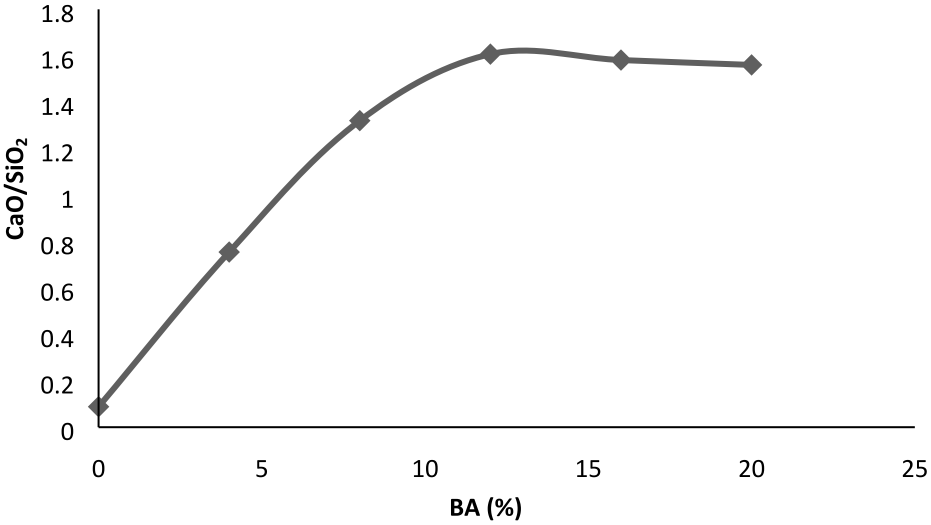

Figure 16

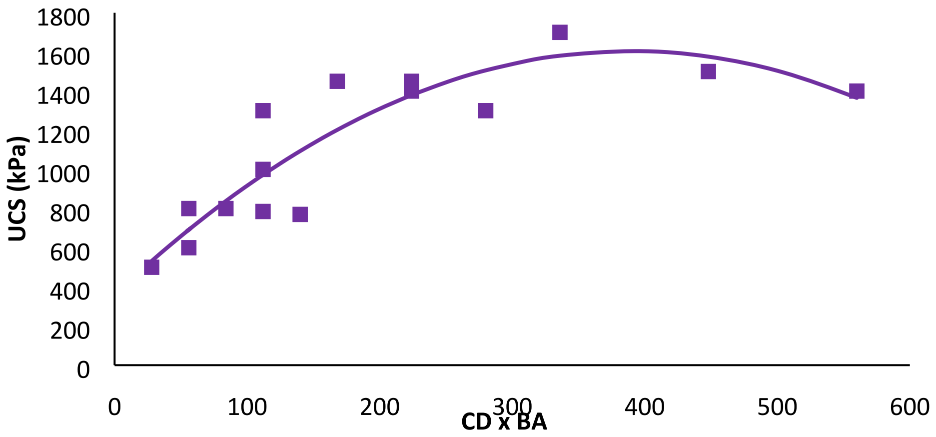
UCS (kPa)
1800
1600
1400
1200
1000
800
600
400
200
0
0
100
200
300
400
500
600
CD x BA

Figure 17

Figure Captions

Figure 1. Location of Numan area (modified after Obiefuna *et al.* 2010)

Figure 2. Atterberg limits for soil-BA mixtures

Figure 3. Moisture-density relationship for soil-BA mixtures

Figure 4. Moisture-density relationship for soil-lime-BA mixtures

Figure 5. CBR for soil-lime-BA mixtures

Figure 6. UCS for soil-lime-BA mixtures

Figure 7. Resistance to loss in strength for soil-lime-BA mixtures

Figure 8. Variation of M_R with deviatoric stress for soil-lime-BA mixtures at 13.8kPa confining pressure

Figure 9. Variation of M_R with deviatoric stress for soil-lime-BA mixtures at 27.6kPa confining pressure

Figure 10. Variation of M_R with deviatoric stress for soil-lime-BA mixtures at 41.4kPa confining pressure

Figure 11. Polynomial model for M_R

Figure 12. Variation of M_R with bulk stress for soil-lime-BA mixtures at 13.8kPa confining pressure

Figure 13. Variation of M_R with bulk stress for soil-lime-BA mixtures at 27.6kPa confining pressure

Figure 14. Variation of M_R with bulk stress for soil-lime-BA mixtures at 41.4kPa confining pressure

Figure 15. Micrograph of (a) natural soil (b) BA (c) stabilised soil with 12% BA plus 4% lime after 28days curing at 500X magnification (d) stabilised soil with 12% BA plus 4% lime after 28days curing at 1000X magnification (e) stabilised soil with 12% BA plus 4% lime after 28days curing at 1500X magnification.

Figure 16. Variation of CaO/SiO_2 with BA content after 28days curing

Figure 17. Polynomial model for UCS