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The stabilization of expansive soil using sugarcane straw ash (Bagasse ash) and lime

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Abstract: Expansive soils pose significant challenges to civil engineering projects due to their high moisture-induced volume changes, resulting in detrimental effects on foundations, structures, and infrastructure. This study investigates the effectiveness of utilizing bagasse ash and lime (calcium hydroxide) as stabilizers for expansive soil. Bagasse ash (BA) is an abundant industrial waste from the sugar production industry and is an active pozzolan. The geotechnical characteristics of the expansive soil (control sample) were identified as clayey sand, containing an appreciable amount of organic and inorganic silts and clays. The ratios for investigation consisted of various lime and bagasse ash (2%, 4.5%, and 7%) at a ratio of 1:3, respectively. The maximum dry density and optimum moisture content of the control sample were calculated as 12.91 kN/m^3 and 30%, respectively, and were consistent with values observed in similar studies. Increasing stabilizer contents resulted in a decrease in density across all ratios. Free swell and Atterberg's results displayed that the 7%L-21%BA was the most effective ratio at reducing the swelling potential of the control soil. The CBR value of the control sample was 5.22% and indicated that in its current state, it would not be suitable for road subgrade applications, as they require at least 10%. With the addition of 7%L and 7%L-21%BA admixture, the CBR values increased to 19.17% and 17.12%, respectively. Furthermore, the addition of 7%L and 7%L-21%BA to the soil greatly reduced cohesion by 78.6% and 65.8%, respectively. However, it increased the internal friction angle by 32.9% and 46.9%, thus increasing its shear strength and bearing capacity.
Keywords: *Expansive soil, Soil stabilization, Sugarcane straw ash, Sustainability.*

1. Introduction

Expansive soils can pose a significant challenge to construction and infrastructure, leading to substantial damages, greater maintenance costs and endangering end users. Expansive soils have the tendency to volumetrically swell and shrink exponentially, upon moisture variations in the soil due to the presence of excessive swelling clay minerals such as montmorillonite, smectite and illite. Such minerals have crystal lattice structures consisting of weakly bound individual layers that permits the easy penetration of moisture between the layers, causing expansion [1].

This problematic soil is often the cause of distress and damages to various structural foundations through phenomenon such as ground heave and differential settlement. Ground heave is the upward movement of expansive soil when introduced to moisture, whereas differential settlement is the uneven/non-uniform settling of the load bearing, overlying structure. From excessive stress applied from such phenomena, overstressed pavements and building foundations can undergo distress such as cracking, faulting, pumping and punchouts [2].

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In 2018, European Soil Data Centre (ESDAC) [3] had conducted multiples surveys and prepared a geological map of the Fiji Islands as shown in Figure 1. This map displayed coastal areas including Nausori, Navua, Sigatoka, Nadi and Lautoka which had consisted of large deposits of unconsolidated silt, clay, and gravel. All of the above areas have major rivers and creeks which are responsible for sediment deposition at river mouths and surrounding areas [3].

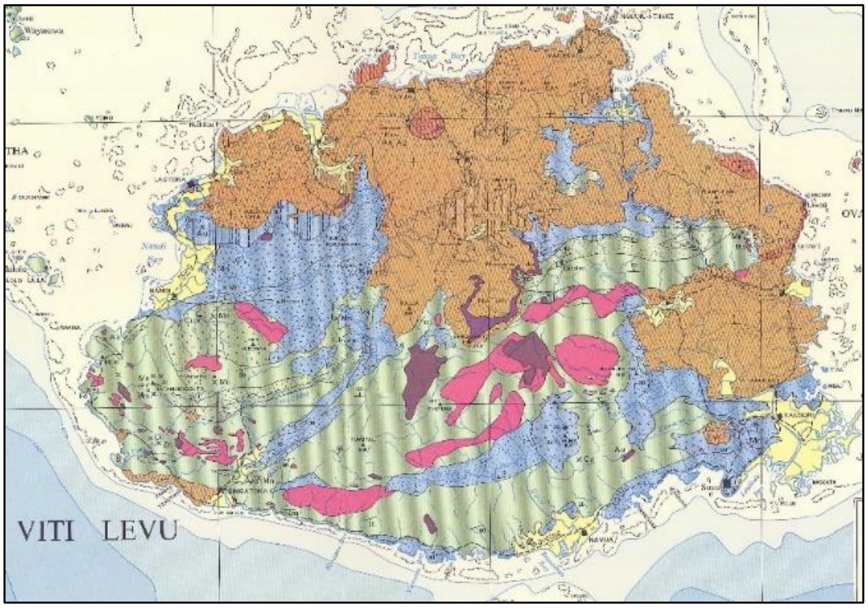

Figure 1.
Geological map of Fiji - yellow regions are those with high quantities of unconsolidated silt, clay and sand [3].

Expansive soils are often treated through the implementation of chemical admixtures to reduce the swelling potential the clay minerals. This study aims to utilize bagasse ash as the primary stabilizer of expansive soils with lime as the secondary stabilizer. Lime as a standalone stabilizer is renowned for its efficient soil stabilization capabilities. However, through investigative research this study aims to implore on the possibility of bagasse ash and lime mixture providing improved performance over a standalone lime stabilizer. Fiji, due to its subtropical climate with adequate rainfall and humidity, is the largest sugar-producing country in the South Pacific. As of 2022, Fiji, nation-wide, had produced over one million metric tonnes of sugarcane. Fiji's sugar industry operates solely through a single company, Fiji Sugar Corporation (FSC). FSC owns and operates multiple sugar mills, in location; Lautoka, Ba and Labasa, and in 2022 had an annual sugar production of 109,554 tonnes [4]. The remnants of sugar production, post crushing, is a fibrous residue called sugarcane straw or bagasse. Bagasse ash is a pozzolanic material, containing high quantities of minerals such as silica and alumina. The use of bagasse ash in soil stabilization is cost effective and environmentally friendly, and it is currently disposed of in municipal landfills. Bagasse ash, similar to fly ash, is also renowned for potential as a partial replacement in blended cements due to its pozzolanic reactivity. As bagasse ash is a disposed of waste material, its use in soil stabilization practices would be highly cost effective. Use of such methods has potential to be utilized in rural areas with limited resources and access to more expansive soil stabilization methods.

2. Literature Review

2.1. Effect of Particle Size on Pozzolanic Reactivity

The particle size of pozzolans has been found to have a significant effect on their pozzolanic reactivity. A study conducted by Janjaturaphan and Wansom [5] investigated the effectiveness of bagasse ash particle size on the pozzolanic reactivity in cement. A conductometric method was employed, that observes a conductivity drop due to the consumption of calcium ions (Ca^{2+}) and hydroxide by the pozzolanic reaction. The faster the conductivity drop, the higher the pozzolanic activity. Another method employed in this study was comparing the compressive strength indices of cement mortars admixed with BA of varying particle size. The results displayed a direct increase in pozzolanic reactivity and compressive strength, after both curing periods, upon a decrease in particle size. A similar study by Ranganath, et al. [6] investigated the effect of fly ash particle size on the pozzolanic reactivity in cements. The study found that pond ash, the remnants of coal firing, cannot be classed as reactive for larger particles and must be below 75 μm to be regarded as reactive and pozzolanic.

2.2. Determination of Additive Ratio

In chemical soil stabilization, the quantities of additives to the untreated soil is paramount in achieving desired geotechnical properties. Insufficient additive ratios may result in insignificant improvements to soil quality; however excessive additions of extra compounds may act as a detriment. Optimizing the ratio of bagasse ash to lime is the primary objective in treating expansive soil and avoiding damage to structures, such as foundation, road and pavements distress and cracking. In a study by Hendrych, et al. [7] evaluating the effects of the additives, bagasse and lime, on old municipal landfill soils which were prone to severe settlement, poor shear strength and high permeation. Through testing, it was concluded that the optimal ratio of bagasse to lime to attain high dry density and compressive strength was 3:1, respectively. Another study conducted by Barasa, et al. [8] testing the treatment expansive soil, tested bagasse to lime ratios; 1:4, 2:3, 3:2 and 4:1 and determined their strength and shrink-well potentials. It was found that the ratio 4:1 corresponded with a CBR value of 36% that conformed to the standard value of 20 for road sub base application by the road design manual part III. The 4:1 ratio had also displayed negligible swelling. The mixture of said components at the given ratio, in the presence of clay and moisture, facilitated pozzolanic reactions between the silica present in the BA and clay and the calcium from the lime. Similarly, Hasan, et al. [9] evaluated strength of 28 day cured expansive soil samples with increasing BA percentages (6.25%, 12.5% & 18.75%) and a constant lime quantity percentage (6.25%) across all samples. The 18.75% BA and 6.25% lime mixture (3:1) exhibited the greatest unconfined strength test ratio.

2.3. Shrink-Swell Potential of Stabilized Soil

The shrink-swell potential of soil refers to its tendency to undergo changes in volume in response to changes in moisture content. The study by Hasan, et al. [10] investigated the shrink-swell potential of untreated and treated soils with varying BA-lime contents. It was observed that free swell ratios (FSR) for all treated samples were lower than the untreated sample, and further decreased as the BA and lime contents increased. Furthermore, the BA and lime combinations had been more effective in reducing the FSR than the lime alone. This was due to the trivalent and divalent cations present in BA, which had increased the rate of cation exchange and flocculation. Additionally, BA is non-expansive in nature and as its replacement rate of clay soil increases, the FSR values subsequently decreased. Similarly, Ewa, et al. [11] investigated the effect of stabilizers BA, lime stone dust (LSD) and BA-LSD combinations on the consistency limits. The results displayed a decrease in the shrinkage limits upon increase the stabilizer contents, which implies that the samples had become less expansive than the untreated soil.

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2.4. Effect of Bagasse Ash on Soil Compaction

The degree of compaction of a soil is reflected by its maximum dry density (MDD) and its corresponding optimum moisture content (OMC). Ewa, et al. [11] through compaction tests of samples with limestone dust (LSD), BA and LSD-BA mixtures, showed LSD and LSD-BA combinations obtained a higher MDD than the BA stabilizer, indicating that BA would not be effective as a standalone stabilizer. However, the LSD-BA combination attained the highest MDD, demonstrating that BA is effective if admixed with another compound. The increased density was attributed to the replacement of higher specific gravity (S.G.) soil with the low specific gravity BA. As stabilizer content increased, the OMC decreased due to the low water attraction properties of the BA pozzolan and non-plastic LSD particles. They concluded that the implementation of stabilizers allows for compaction to occur at a lower moisture content. Hasan, et al. [10] found that the MDD's of the lime-BA combinations decreased as their content percentage increased. All admixed samples exhibited a lower MDD than the control sample, and may be due to the BA and lime having a lower specific gravity than soil and being tested at a much higher OMC that's required for them to achieve their MDD.

2.5. Durability of the Stabilized Soil

The durability of a soil is its ability to withstand and perform under freeze thaw and wetting-drying cycles. Humidity, droughts and excessive moisture are the primary environmental factors causing cracks and erosion in problematic soils. In the study by Hasan, et al. [10] expansive soil treated with BA and lime has been tested for durability according to a method proposed by Basha, et al. [12]. Hasan, et al. [10] had examined the strengths via the unconfined compressive strength (UCS) test, for a sample with 6.25% lime only and another sample with a mixture of 6.25% lime and 18.75% BA. The wetting-drying cycles had continued for ten iterations and had revealed that the BA-lime sample had exhibited significantly greater residual strength of the soil after the final cycle, than the sample with lime alone. A set of samples were test for UCS which had been placed in potable water for one week. This procedure was to simulate effects heavy rain on the soil. The 12.5% and 18.75% BA had exhibited UCS values of 728 kN and 742 kN respectively, while the 6.25%L sample had 507kN. The implementation of BA in expansive soil, displayed a greater resistance to the detrimental effects of saturation. Another indicating characteristic of soil durability is the percentage mass loss. A loss in mass is attributed the brushing of stabilized material off the carbonated exposed surface. Jha, et al. [13] conducted a study on the effects of class F fly ash on lime stabilized soil. The durability tests consisted of soil samples increasing in BA% (25, 35 & 45%) with constant lime percentages 4%, 7% and 10%, and curing up to 180 days. It was observed that at 4% L and 25% BA, there was 10.31% mass loss and increased for tests with 35% and 45% BA. It was further observed that for BA compositions 25% and 35%, the 4% L samples could not satisfy the durability requirements for mass loss, even after 180 days of curing. This was due to there being an insufficient quantity of lime present in the mix, which then delayed the pozzolanic reaction responsible for the formation of the cementitious compounds. When the lime content was increased to 7%, the mass loss across all fly ash compositions met the durability requirements within 56 days, and only took 28 days when lime was increased to 10%. This demonstrated the induction period is dependent on availability of lime for the pozzolanic reaction.

2.6. Effect of Curing on Strength

Curing time refers to the duration treated soil takes to achieve a change in its mechanical properties. The curing time for treated soils is dependent on factors including the quantity of stabilizing agents, moisture content and soil temperature. A study by Amadi and Osu [14] investigated the effects of curing time on the strength development of black cotton soil, stabilized with 10% quarry fines (QF) and varying quantities (0-16%) of cement kiln dust (CKD). Similarly, to bagasse ash, the cement kiln dust composition consists of alumina and silica oxide and other components such as calcium carbonate, and calcium oxide. Through unconfined compression testing, the 16% CKD sample had exhibited the higher axial strength before failure across all curing periods. However, each soil composition sample displayed a common trend, in which the maximum strength was achieved after 28 days of curing. This indicated that soil strength is dependent on curing time. This was attributed to the time dependant nature of the pozzolanic reactions in which cementitious compounds are formed.

2.7. Applications of Bagasse Ash

This BA is rich in minerals and nutrients such as calcium, potassium, magnesium, and silica, making it a valuable resource for a wide range of applications. In addition, it has been found to contain traces of heavy metals such as zinc and copper, which can be extracted for use in various applications. Bagasse ash has found its use in various fields such as agriculture, construction, and environmental remediation. Its versatility and sustainable nature have made it a preferred alternative to synthetic materials, especially in developing countries where sugarcane cultivation is prevalent. BA has gained widespread attention as a potential pozzolanic material for partially replacement of cement, due to its relatively high content of amorphous silica. BA has already been widely applied as a pozzolanic material in concrete, in countries like India [15]. According to Bahurudeen, et al. [16] through performance testing of sugarcane bagasse ash blended cement in concrete, it had displayed improvements to compressive strength, flexural strength, and split tensile strength. The researchers also noted that the use of BA improved the workability and durability of the concrete, as it increased the resistance to the ingression of moisture, oxygen and chloride. In a similar study by Cordeiro, et al. [17] it had showed that the production of high-performance concrete was possible through the incorporation of bagasse ash in cement. Additional applications of BA have shown their potential as a partial alternative for aggregates in concrete. Due to the high silica content and analogous properties of fine aggregates like sand, in a study by Sales and Lima [18] BA was used in Sao Carlos, SP, Brazil as sand aggregate replacer and the mortar produced with BA instead of sand showed improved mechanical properties. These mortars were later used in civil construction site across the regions in Sao Carlos.

Another utilization of BA as is its application in stabilizing soil. Since BA works as a pozzolanic material, it was used by Jamnongwong, et al. [19] and Jamsawang, et al. [20] in their study of the possibility of replacing ordinary Portland cement (OPC) with BA to stabilize soft clay. The study showed that 20% of OPC has to be replaced with BA to improve the mechanical properties. BA, as a lone stabilizing agent was displayed in a study by Jamsawang, et al. [20] where the formation of cementitious compounds was present in the soft clays from different parts of Thailand. Bagasse ash has also been utilized in the strengthening and stabilization of earth blocks in India. In recent years, compressed earth blocks (CEBs) have gained popularity as a sustainable and affordable building material, particularly in low-income countries. However, the stability of CEBs is a concern, as they are prone to cracking and erosion. To address this issue, Singh, et al. [21] investigated the effects of BA and wheat straw on the stability of CEBs. The study involved producing CEBs with varying percentages of BA and wheat straw and subjecting them to a series of tests to determine their compressive strength, water absorption, and resistance to erosion. The results of the study showed that the addition of BA and wheat straw had a significant effect on the stability of CEBs. The compressive strength of the CEBs increased with the addition of BA and wheat straw, and the CEBs with the highest percentages of these additives had the highest compressive strength. The water absorption of the CEBs decreased with the addition of BA and wheat straw, indicating that these additives can improve the water resistance of CEBs. Additionally, the CEBs with BA and wheat straw had higher resistance to erosion compared to the control CEBs. The researchers also investigated the microstructure of the CEBs using scanning electron microscopy (SEM) and X-ray diffraction (XRD). The results showed that the addition of BA and wheat straw resulted in a more compact and homogeneous microstructure of the CEBs, which contributed to their improved stability [21]. A proof of concept was conducted by the Indian Institute of Technology Roorkee, with a project involving the building of a low-cost, energy-efficient house using compressed stabilized earth blocks made with BA. The house was designed to be earthquake-resistant and to withstand extreme weather conditions.

2.8. Environmental Impacts

The implementation of BA and lime as problematic soil stabilizers can offer significant improvements to environments vulnerable to soil erosion. Although BA and lime are relatively unreactive materials that are chemically basic in nature, improper application of said materials in soil stabilization can serve as detriments to the environment. Bagasse ash contains appreciable concentrations of heavy metals such as magnesium, phosphorus and sodium, as can leach into groundwater and nearby waterways and contaminate them [22]. Lime has no significant negative impacts on the environment; however, its production calcination process occurs at high temperatures that outputs vast amounts of carbon dioxide into the atmosphere.

3. Methodology

All geotechnical tests performed have been in strict accordance with American Society for Testing and Materials (ASTM) approved test equipment as well as standard ASTM procedures.

3.1. Soil Preparation

The expansive soil consisted of large quantities of red clay and possessed a high in-situ moisture content. Due to clay soil having a high-water absorption and retention capacity, prior to use in any test procedures, had to be oven dried at 110°C for 24 hours to remove all moisture. Once the sample had been oven dried, it had to be manually crushed beforehand sieving to the required particle sizes. All CBR, standard proctor, UCS, direct shear and durability tests require a maximum particle size of 2.5 mm, whereas the Atterberg's limit and free swell test require a particle size of 75 μm.

3.2. Sieve Analysis Procedure

The sieve analysis was performed for both the control soil sample and the bagasse ash sample in order to obtain their respective particle size distribution. In accordance with ASTM standards, the samples were prepared through light particle size adjustment and placed into the stack of sieves, No. 4, No. 10, No. 20, No. 40, No. 60, No. 140 and No.200, in order of decreasing sieve size. The sieve stack was placed within the sieve shaker (Figure 2), before being shaken for 15 minutes. The mass retained in each sieve, after shaking, is tabulated to obtain the percentage finer and plotted against the particle size to obtain the particle distribution curve. Through visual observation of the curve as well as D_{10}, D_{30} and D_{60} results, the soil and ash sample were then classified according to their degree of gradation.

Figure 2.
Sieve shaker with stack of sieves from sieve No.4 to No. 200.

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3.3. Standard Proctor Test Procedure

The standard proctor test is a compaction test conducted to obtain the maximum achievable dry density (MDD) of the soil sample and its optimum moisture content (OMC). The control, untreated, sample is conditioned to 10% moisture content, and was compacted in the mould with a 2.5 kg rammer, in three layers. The weight of the sample was taken in its mould before being extruded and its moisture recorded. The process was repeated for 6 trials until there was a significant decrease in sample density, with the moisture content increasing by approximately 5% each trial. From the results, the compaction curve was plotted and the MDD and corresponding OMC was obtained. Each soil and stabilizer ratio were then conditioned to the OMC of the control sample, and by employing the same compaction process, their respective dry densities were obtained. To allow for effective comparison, all admixture ratios were conditioned to the OMC of the control sample for other tests including CBR, UCS, direct shear and standard Proctor tests.

3.4. CBR Test Procedure

The CBR test for the untreated expansive soil sample, was performed in accordance with ASTM D1883-21. The soil was conditioned to the optimum moisture content of 30%, which was obtained previously from the standard proctor test. The conditioned soil was then compacted in three layers, with 56, 2.5 kg rammer blows per layer. The mould was then placed in the CBR test apparatus and the surcharge weights were placed on top of the mould. The surcharge plates are used to simulate additional stress/load the soil would experience in field conditions from overlying layers or structures. The test apparatus then applied a load via a piston to the mould at a rate of 1.25 mm/min. The load readings were taken from the proving dial at penetration depths ranging from 0.5 mm to 12.5 mm, at intervals of 0.5 mm. The proving dial readings were converted to axial loads via the apparatus calibration factor, 1.175. The standard load at 2.5 mm and 5 mm are 70 kg/m^2 and 105 kg/m^2, respectively. The CBR value at 2.5 mm is generally taken as the representative value for the sample and should be greater than the CBR value at 5mm. CBR tests were conducted on the untreated sample and the optimum ratios obtained from the UCS test.

3.5. Atterberg's Limit Procedure

3.5.1. Liquid Limit

The Casagrande apparatus and grooving tool were cleaned and dried before use. About 250 gm of air-dried soil, passed through the No. 40 sieve, was mixed well on the glass plate with water to form a thick homogeneous paste. Portion of the soil mass conditioned to thick homogeneous paste was placed in the brass cup levelled parallel to the base. The soil was divided into two equal parts using the grooving tool, the division was made in the direction perpendicular to the crank of the brass cup. The crank of the Liquid Limit device was turned at a rate of about 2 revs per second. The soil from the two sides of the cup began to flow towards the center. The number of blows, N, for the groove in the soil to close was 36, a moisture sample was collected from the soil in the cup in a moisture can and the moisture content (ω) of the soil was determined. All the soil paste was removed from the cup and placed in a dish. The soil in the dish was mixed with water and the second attempt was made with N of 28 blows. Steps were repeated for the remaining 2 trials and results were recorded. Water was added to the soil paste in the evaporating dish and mixed thoroughly aiming to decrease the N value. A log graph was plotted between water content (ω) and the number of blows N in the log scale. From the straight line flow curve, the moisture content (ω) corresponding to 25 blows was determined and the value represented the liquid limit of the soil.

3.5.2. Plastic Limit

A sample of about 40g of soil was obtained from the material that passed the 425 μm sieve. The soil sample was prepared until the soil mass became plastic enough to be shaped into a ball. A 30g of the

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prepared sample was taken, and a soil mass ball was formed. The sample was rolled on the palm of two hands until the heat generated dried the soil mass sufficiently, causing cracks to appear. From this sample, 10g was taken and rolled into threads with a diameter of 3.0 mm. The rolling was stopped when the rolled soil mass started to crumble. Some of the crumbled soil was placed in drying cans for water content determination. The procedure was repeated for another three trials. The average water content obtained, represented the plastic limit (PL) of the control sample.

3.6. UCS Test Procedure

Each ratio was conditioned to 30% moisture content and compacted in 3 layers, requiring 25, 2.5 kg rammer blows per layer. Each ratio had two molds for the different curing periods; 14 and 35 days. At the end of the curing periods, the samples were released from the molds as seen in Figure 3 and tested under a compression machine to obtain the maximum unconfined compressive strength values.

Figure 3.
UCS sample of treated soils after 14 and 35 days curing.

3.7. Free Swell Index Test Procedure

The free swell index (FSI) test for the control sample and the mixture were performed in accordance with the ASTM D4546-14 standard test methods for one dimensional swell or collapse of soils. The test began with two oven-dried soil samples, each weighing 40 grams and passing through a 500-micron sieve. Each soil sample was carefully poured into separate glass, graduated cylinders with a capacity of 100 ml. One cylinder was filled with kerosene, while the other was filled with distilled water, both up to the 40 ml mark. Gentle shaking and stirring with a glass rod were employed to remove any entrapped air in the cylinders. The samples were then allowed to settle in their respective cylinders, with a minimum waiting period of 24 hours to ensure the attainment of an equilibrium state. Final volumes of the soils in each cylinder were recorded. The entire procedure was followed and repeated for the weight replacement of lime to soil and bagasse to soil.

3.8. Direct Shear Test Procedure

To find the shear strength of both the treated and untreated soil the direct shear box method was employed where the shearing strain was increased at a steady rate. In this method, a soil sample weighing approximately 120 g was conditioned to reach a 30% moisture content and kneaded for even distribution. It was then compacted in three layers and placed in a shear box of 60 mm by 60 mm with non-porous and porous plates. A 4 kg load was applied, and the vertical deformation was recorded. The direct shear machine is initiated with a deformation speed of 1 mm/minute, and horizontal deformation and proving ring readings were recorded at 15-second intervals until deformation stabilized. Afterward, the normal load was released, and the process was repeated for various soil samples under normal loads of 8kg and 12kg. This series of steps allowed for the evaluation of shear strength properties of the soil samples like cohesion and internal friction angle. For each normal load, only the maximum shear stress

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was taken into account. The series of steps were repeated for treated soil, except the optimum ratios' samples were cured for 7 days inside the shear box for every normal load of 4kg, 8kg and 12kg, before they were tested.

4. Results and Discussion

4.1. Sieve Analysis

Sieve analysis is a standard test conducted to determine the particle size distribution of soil or other alternative materials. The degree of gradation of a sample can have a direct influence on its mechanical properties such as compaction, shear strength, permeability, solubility etc., as well as, it aids in the classification of the sample. From weighted percentages of the mass of material passing through each sieve, the particles distribution curve for both materials were obtained. The degree of gradation can be ascertained from visual observation of the curve as well as from the uniformity coefficient (C_u) and coefficient of curvature (C_c). For the control soil, the C_u obtained from Figure 4, is between 4-6 and the C_c is between 1-3, it is classified as a well graded soil sample. This indicates that the sample has the potential for higher shear strength due to lower void ratio, improved compaction and drainage and greater interparticle cohesion. Due to the percentage of mass retained; it was further concluded that the soil sample was primarily comprised of clayey sand. In Figure 5, due to the BA having a C_u below a value of 3, and C_c value less than 1, it is classified as poorly graded. The poor gradation is also reflected in the nature of the curve. This indicates that the BA must undergo particle size readjustment through further crushing before using in any additive mixtures. A further reduction in particle size was required to maximize the utilizable yield. According to literature studies [23, 24] the particle size must be at least 0.15 mm, in order to enhance the pozzolanic reaction.

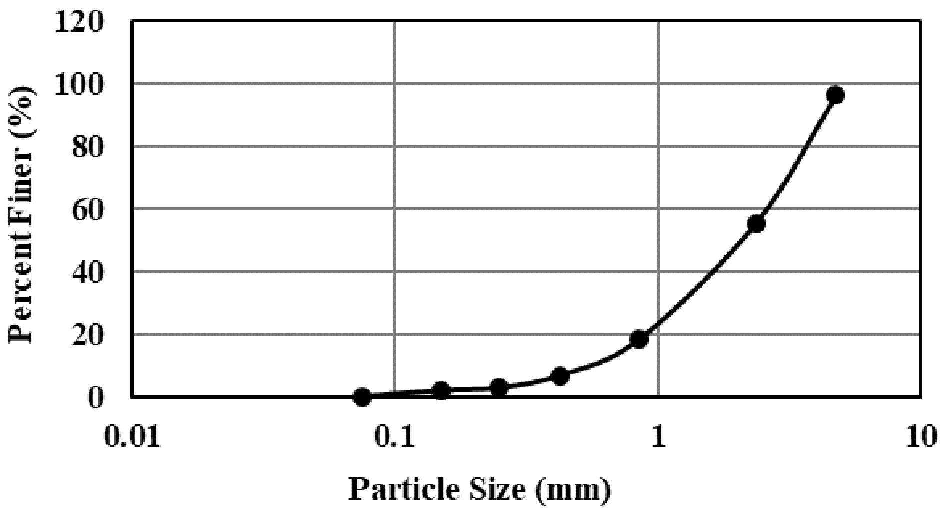

Figure 4.
Particle size distribution of control soil sample.

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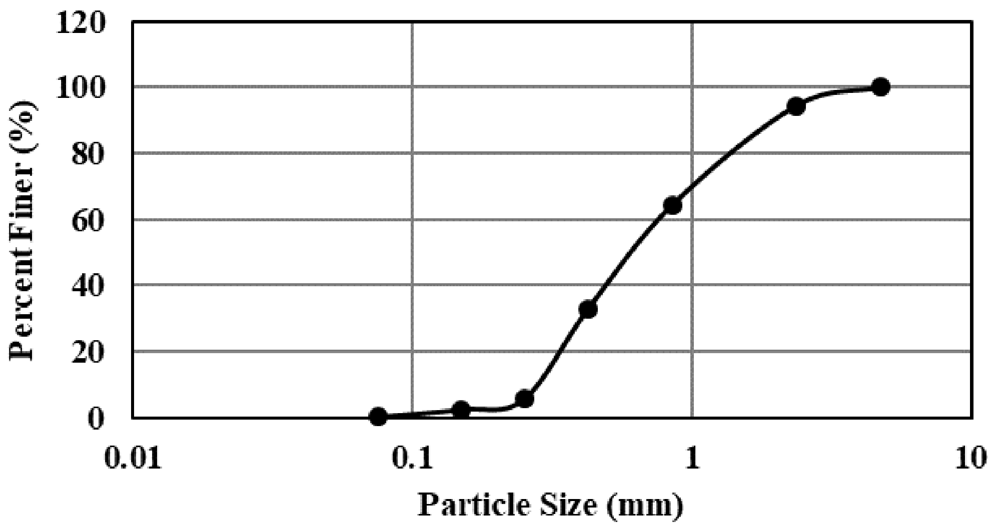

Figure 5.
Particle size distribution of bagasse ash.

4.2. Standard Proctor

The degree of soil compatibility is measured according to its maximum achievable dry density at optimal conditions. The standard proctor test was conducted on an untreated soil sample and the results from said test are displayed in the Figure 6. The maximum dry density (MDD) of the control sample, equated to 12.91 kN/m^2. The MDD value corresponded to an optimum moisture content of 31.04%. At low moisture contents, soil is stiff and resistant to compaction, however by increasing the moisture content it lubricates the soil particles, allowing for particle readjustment and resulting in a denser configuration. At moisture contents exceeding the OMC, there is an observable decrease in the maximum dry density. This is attributed to excess water increasing the pore water pressure. This counteracts the effective stress of the soil particles, which reduces the interlocking and load bearing capacity. The weakened interparticle bonds, effectively reduces the frictional resistance between particles, causing them to easily rearrange under external forces and leading to poor compaction and decreased dry density.

As the OMC exceeded 30%, it indicated that the sample contained significant quantities of high plasticity clay. High plasticity clays require relatively high moisture contents due to their high particle cohesion and shear strength. The standard proctor test was further conducted on all the admixture ratios, conditioned to 30% moisture content. In Figure 7, observable reductions in the densities of the treated soils corresponded to an increase in weight replacement. The greatest decrease in density was observed in the L-BA ratios. The density reduction may be attributed to flocculation between clay particles and additives which forms coarser particles, thus increasing the void volume and in turn lowering the density [25]. Additionally, due to increasing replacement of clay soil of higher specific gravity (Control - S.G. = 2.72) with stabilizers of lower specific gravity, (Lime - S.G. = 2.48) and (BA - S.G. = 1.26), would also contribute to the lowered density.

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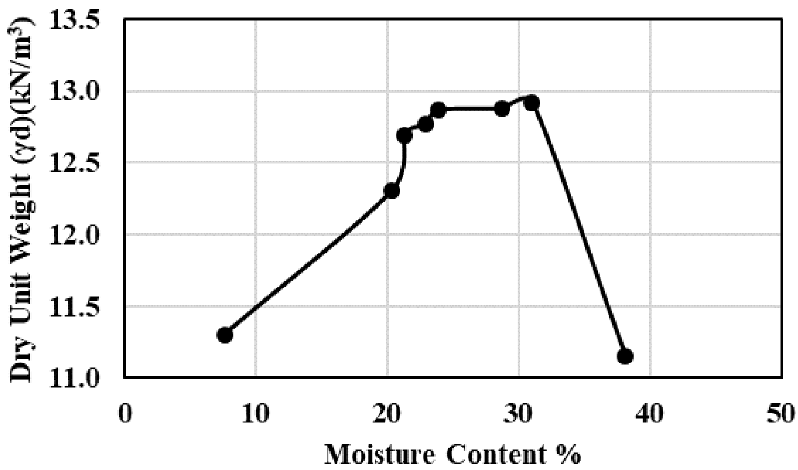

Figure 6.
Compaction curve showing MDD and corresponding OMC of untreated soil.

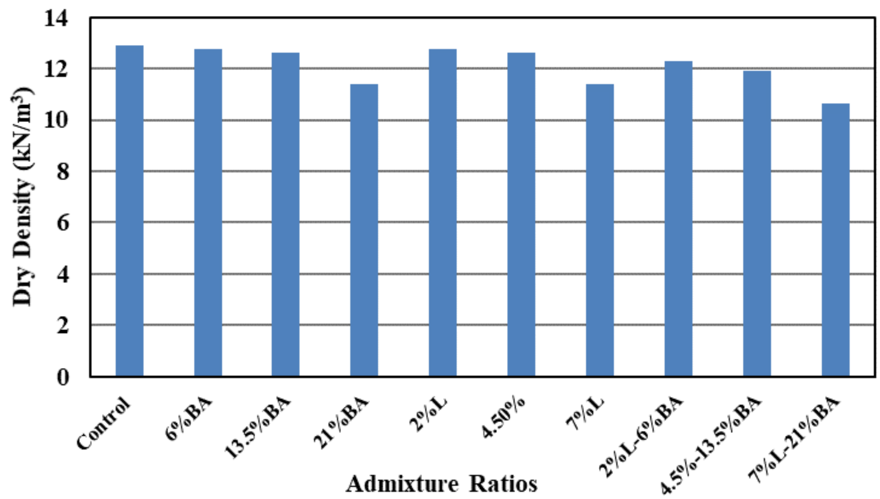

Figure 7.
Relationship between moisture content against dry unit weight.

4.3. Free Swell Test

The free swell index (FSI) is a property used to measure the potential swelling characteristics of expansive soils (volumetric changes), particularly clay soils, when they come into contact with water. It provides information about the volume change that a soil may undergo upon wetting. From the tests results shown in Figure 8, and comparison to the Degree of Expansivity given in Table 1, the control sample exhibited an FSI of 36%, thus classifying it as highly expansive. The tests results displayed significant reduction in the FSI upon addition of BA and lime. Figure 8 shows that BA was more effective at reducing the expansivity than lime, due to its non-expansive, hydrophobic characteristics. Lime-bagasse ratios were most effective at reducing the expansion potential due to the cation exchange between the admixtures and soil, causing flocculation. Through flocculation, flocs are formed, creating a more stable soil structure, which is more resistant to volumetric change upon moisture influxes [26]. The 7%L-21%BA ratio was most effective at hindering the swelling potential, as it reduced the FSI of the control soil by 85.5%, while the addition of 21%BA and 7%L reduced the swelling potential by

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80.4% and 69.7%, respectively. It must be noted that the free swell test is relatively rudimentary and would require further investigation to obtain more accurate data on the swelling potential of the treated and untreated soil.

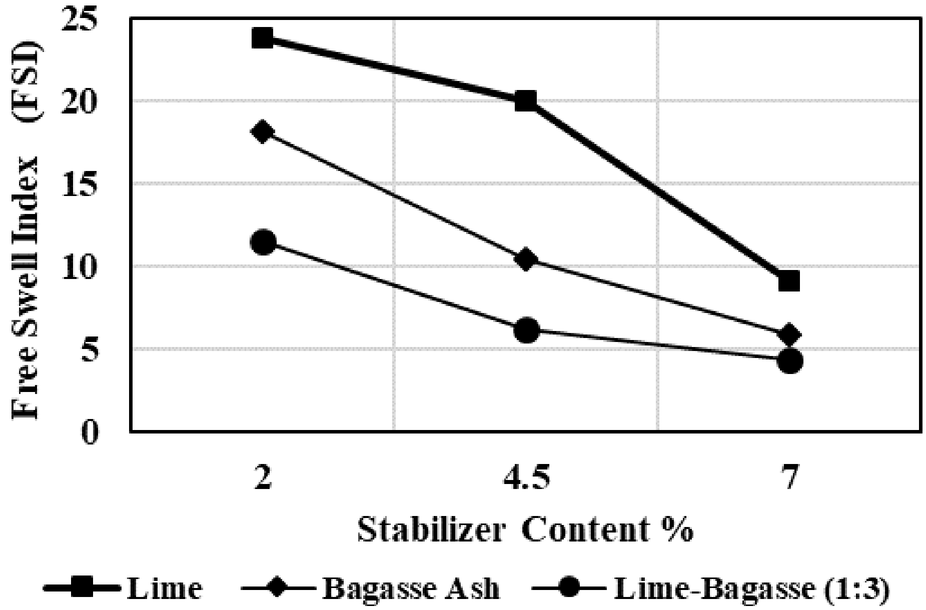


Figure 8.
Effect of stabilizers on the FSR of soil.

Table 1.
Degrees of expansivity [27].

Degree of swelling	FSI (%)
Low	<20
Moderate	20-35
High	35-50
Very high	>50

4.4. Atterberg's Limits

Atterberg's limits tests comprise of three important limits, known as the plastic limit, liquid limit and shrinkage limit. The Atterberg's limits are used to classify soils based on their behavior and consistency, and to assess their suitability for various engineering applications. The liquid limit and plasticity index are used in classifying soils according to the Unified Soil Classification System (USCS). The liquid limit is an indicator of the soil's ability to deform under shearing forces. The plastic limit is an indicator of the soil's ability to retain its shape when molded. The two values also give the soil's consistency, which is the soil's tendency to undergo volume change and its susceptibility to deformation. According to Figure 9, the control sample is classified as MH & OH. The control sample used by the project team had a Liquid limit (LL) of 75.21% and a Plasticity Index (PI) of 10.21%. This indicates that at 75.21% the soil loses its structural integrity and begins to behave as a fluid. MH and OH both fall under silts and clays classifications. MH are inorganic silts, micaceous or diatomaceous fine sandy/silty soils, and elastic silts. OH are organic clays of medium to high plasticity and organic silts.

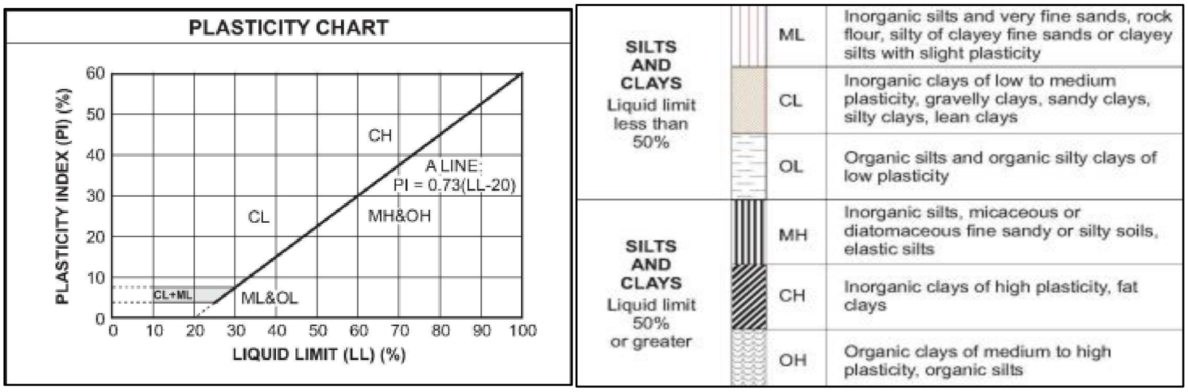


Figure 9.
USCS plasticity chart and USCS fine soil description [27].

Figures 10, 11 and 12 display the influence of stabilizer additions to the Atterberg's limits. Observations of the data revealed that the Liquid Limit (LL), Plastic Limit (PL) and the Plasticity Index (PI) decreased upon the increasing increments of each stabilizer's quantity. The LL for L, BA and L-BA ratios in Figure 10, displayed an average decrease from the control sample of 6.14%, 11.92%, 13.46%, respectively. The L-BA displayed the greatest reduction in the liquid limit due to the cation exchange brought on by the action of the divalent calcium-silicate ions. BA had the second largest percentage drop in LL. Due to the hydrophobic nature of the BA, it tends to serve as a desiccant, decreasing the elasticity of the soil. This reduction in the liquid limit suggests that these additives have a drying effect on the soil, effectively reducing the moisture content required to transition from a plastic to a liquid state. The PL for L, BA, L-BA decreased by 6.18%, 10.15%, 12.45%, respectively, while the PI for L, BA, L-BA decreased by 9%, 16.35% and 20%, respectively. In Figure 11, there was a decrease in the plastic limit across the different ratios when the additives were introduced. Between the three stabilizers, L ratios exhibited the lowest percentage decrease in PL, while L-BA ratios had the largest percentage drop, specifically the 7%L-21%BA ratio. This could be due to the calcium ions present in lime, which reduce the cohesive qualities of the soil and further reduces its tendency to distort [28]. The biggest drop in plastic limit is due to soil particles being replaced by non-plastic particles of bagasse ash and ionic exchange of lime and clay minerals of the soil. This combined action results in flocculation and agglomeration of the clay particles, thus reducing the plasticity and the expansivity of the treated soil. This was evident in Figure 11, whereby the highest ratio of the admixture (7%L-21%BA), was the most effective at reducing the expansivity and the plasticity of the soil by having the lowest plastic limit. The plasticity index exhibited a decrease across the three different ratios when the additives were increasingly added to the soil, as shown in Figure 12. The availability of calcium and silicates for cation exchange may be connected to the decline in plasticity index. This phenomenon can also be explained by the combined effect of pozzolan-made, non-plastic ash particles partially replacing plastic soil particles. A drop in the plasticity index implies that soil qualities have improved. This indicates that the soil's plasticity index decreased, which can be seen as a positive outcome for engineering purposes, as it suggests improved workability and reduced susceptibility to volumetric changes upon drying and wetting. Therefore, the 7%L-21%BA ratio was identified as the most effective ratio at reducing the expansion potential. This finding was also reflected in the free swell test.

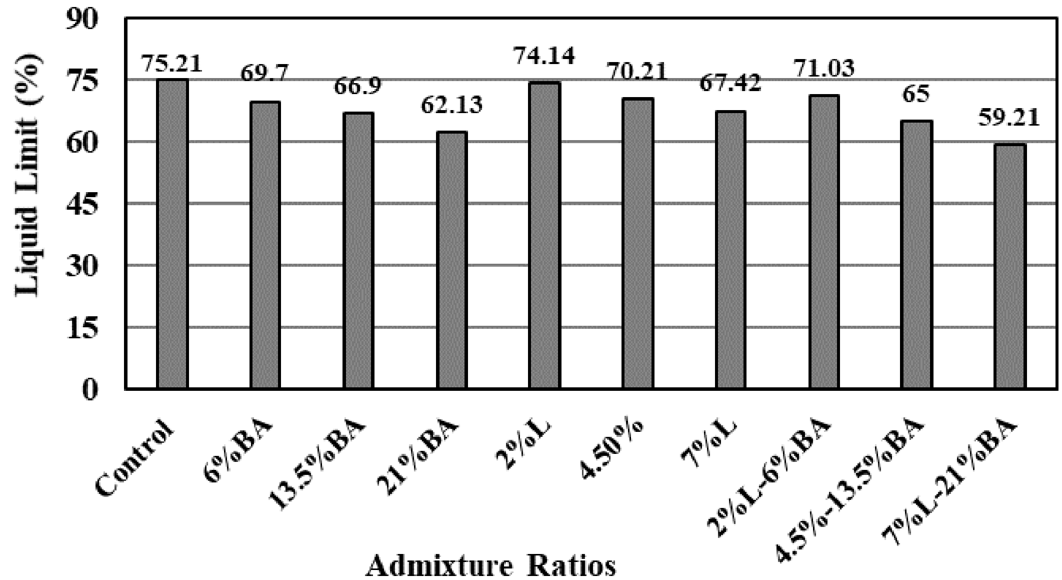


Figure 10.
Effect of stabilizers on liquid limit.

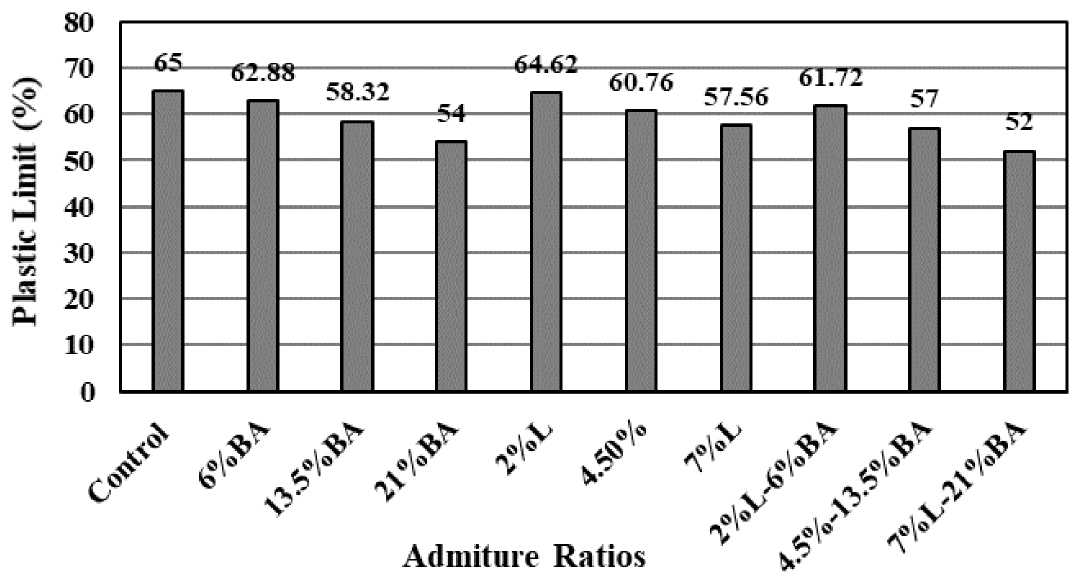


Figure 11.
Effect of stabilizers on plastic limit.

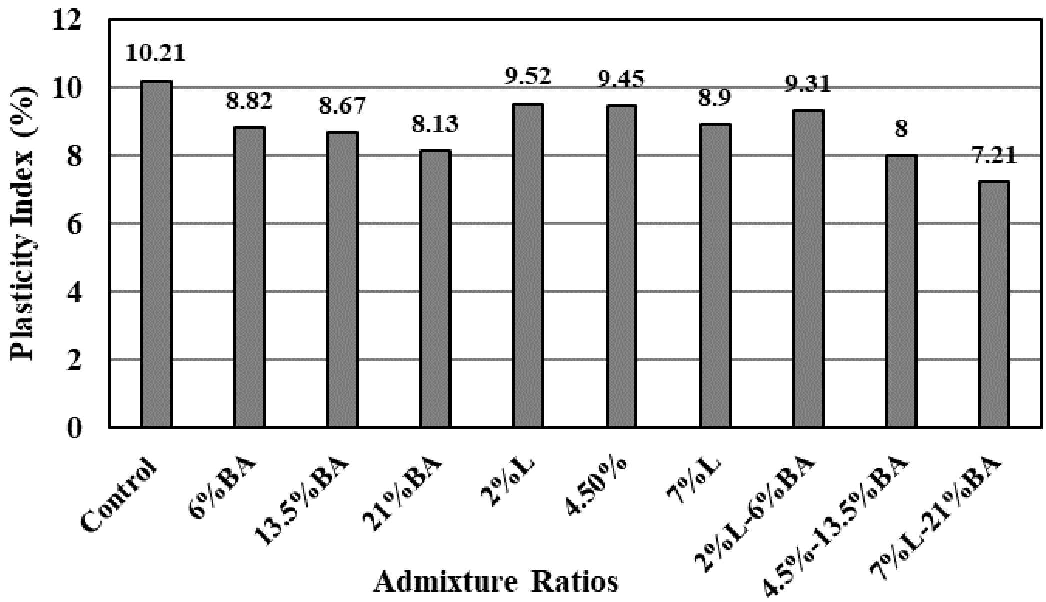


Figure 12.
Effect of stabilizers on plasticity index.

4.5. UCS Test

UCS strength for the lime only and L-BA mixtures for both curing periods increased exponentially as the weight replacement ratios increased as seen in Figure 13. In Figure 13, BA-only samples displayed a decrease in the UCS strength upon increasing weight replacement. This is because the BA does not possess inherent cohesive properties. Thus, increasing the weight replacement percentage will lower the overall cohesion between soil particles and in turn, lower the UCS strength of the soil. Unlike the BA ratios, the L and L-BA ratios displayed an increase in compressive strength upon curing from 14 and 35 days. The 7% lime exhibited a UCS strength of 3.54 kN after 35 days curing, which was the highest amongst all additive ratios. This performance was expected as lime is a renowned standalone stabilizer. The 7%-21% L-BA ratio displayed the highest mixture ratio strength of 2.86 kN after 35 days of curing. This value was significantly lower than the 7% L, and may have been attributed to an incomplete pozzolanic reaction due to insufficient moisture being available for hydration. This observation was substantiated by the moisture content readings taken at the end of the 14 day and 35 day curing periods, as shown in Figure 14. The moisture content of the 7%-L at the end of 14 and 35 days was 16.02% and 14.99%, respectively. Whereas, the moisture content for the 7%L-21%BA at 14 and 35 days was 14.93% and 13.32%, respectively. The BA ratios had the highest mean variance in moisture content, between the 14 and 35 day curing periods, of 26.83%. This variance was attributed to high evaporative losses due to the reduction in cohesion in the BA admixed soils. The L-BA ratios had the lowest mean variance in MC of 7.05%, which indicated that the cured samples had undergone some degree of pozzolanic reaction, causing a reduction in permeability. This reduction in permeability inhibits evaporative losses. The results showed that the UCS strengths of the ideal ratios at 35 days, displayed a minor increase when compared to the 14 day period. This indicated that 14 days is a sufficient curing period for the pozzolanic reaction to occur and achieve suitable strength. Similar findings were observed in other reports, where the UCS values for all L-BA ratios increased exponentially between curing periods of 7 and 14 days, after which they began exhibiting sublinear growth [25, 29].

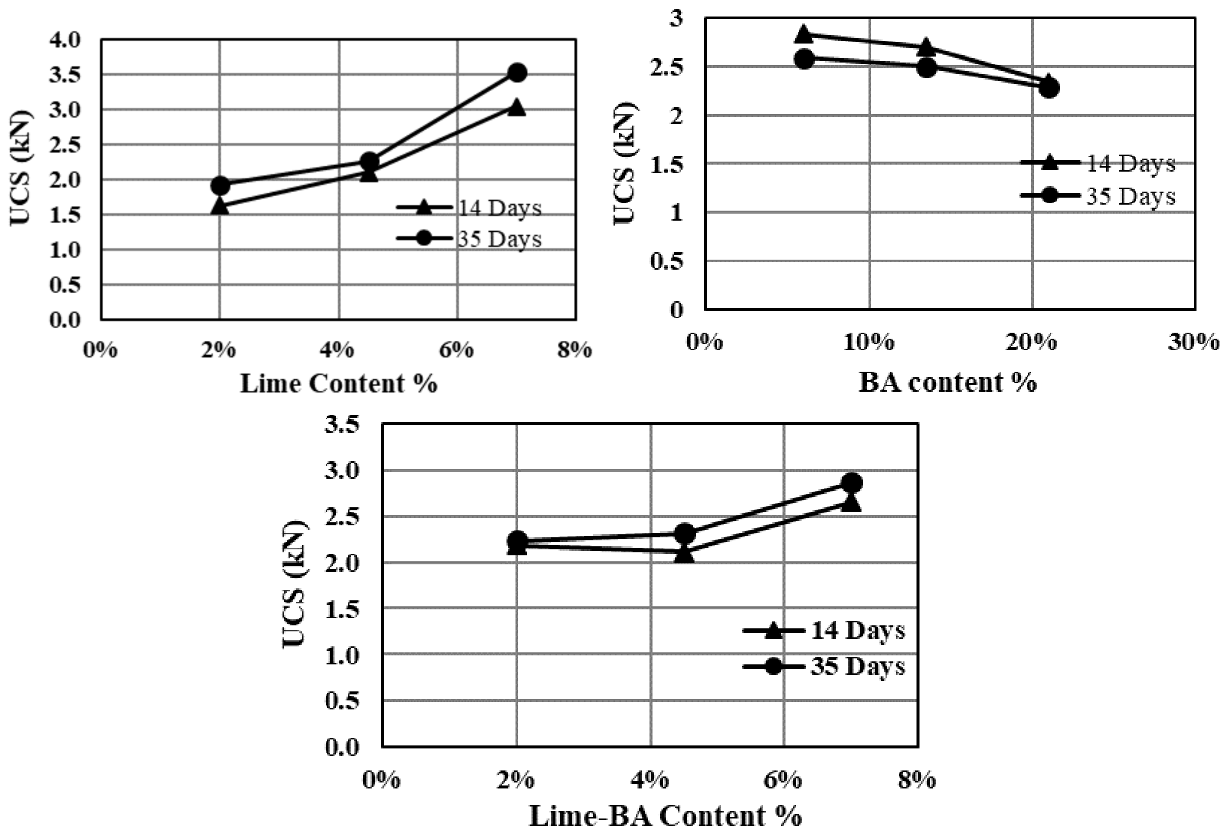
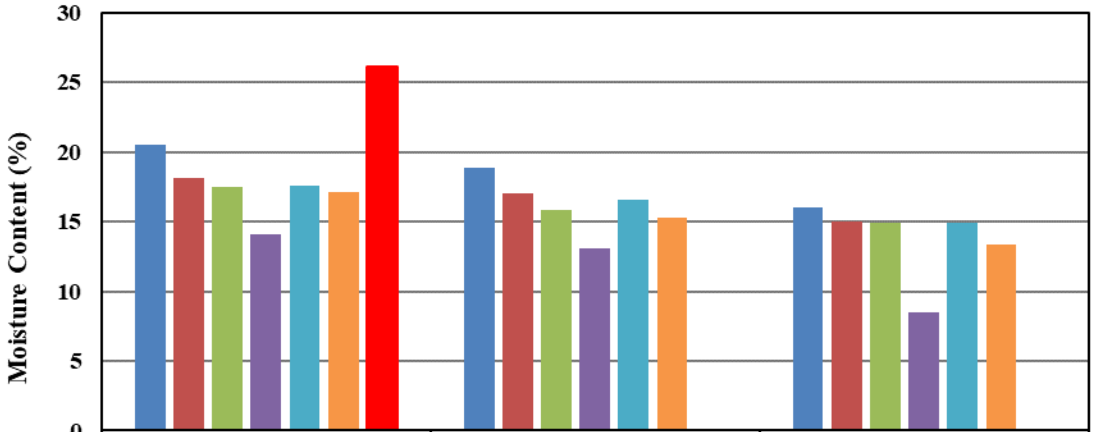


Figure 13.
UCS strength of the soil with lime stabilizer, BA stabilizer and Lime-BA combined stabilizers.



	2%L, 6%BA, 2%L-6%BA	4.5%L, 13.5%BA, 4.5%L-13.5%BA	7%L, 21%BA, 7%L-21%BA
L (14 Days)	20.52	18.83	16.02
L (35 Days)	18.16	17.01	14.99
BA (14 Days)	17.54	15.89	14.96
BA (35 Days)	14.07	13.12	8.48
L-BA (14 Days)	17.63	16.60	14.93
L-BA (35 Days)	17.16	15.33	13.32
Control	26.17		

Figure 14.
Moisture content readings from the UCS samples, post testing.

4.6. California Bearing Ratio (CBR)

The CBR test is a standard test conducted to determine the mechanical strength and load-bearing capacity of road subgrade and subbase materials. A CBR test was conducted on an untreated, control sample. The axial load per millimeter penetration depth is obtained by multiplying the proving dial division readings by the constant, 1.176 kg per one division. The load is then divided by the area of the penetration plunger, 19.63 cm^2. Figure 15 displays the stress-penetration curves for the control, 7%L and 7%L-21%BA ratios. The curves indicate the applied stress required for 0.5 mm penetration into the soil. The CBR value is a ratio of the axial load corresponding to penetration depth 2.5mm and 5mm, to the unit standard load, 70 kg/m^2 and 105 kg/m^2, respectively. The CBR values displayed validity, as they typically decrease with depth. At 2.5mm penetration, the stress is more concentrated within a smaller area, leading to higher localized resistance to penetration. At 5mm penetration, the stress is distributed over a greater area, causing a reduction in the localized resistance. The CBR value corresponding to 2.5 mm was 5.22% and the value at 5 mm was 4.34%. According to AASHO, the representative value for the sample is taken at 2.5 mm [30]. The CBR value, 5.22%, is unacceptable for road construction, as highway engineers are required to achieve a CBR value of at least 10% for urban and highway roads. In its current state, for implementation in road subgrades, geosynthetics must be utilized between the flexible pavement layers to act as reinforcement and increase drainage qualities. Another approach to increase the CBR is through chemical soil stabilization.

CBR tests were performed on the optimum ratios obtained from the UCS test; 7%L and 7%L-21%BA. A 7%L-21%BA was cured within the CBR test mold for one week, and the CBR value obtained was 5.56%, displaying minimal improvement from the untreated sample. After a two-week curing period, the same, 7%L-21%BA ratio yielded a CBR value of 17.12%. This improvement in the CBR values between curing periods indicated the importance of maturation on the pozzolanic reaction and the strength development. The 7%L ratio, after 2 weeks curing was identified as the most effective at

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improving the CBR value, as it increased to 19.17%. Further research is required to investigate whether the addition of BA to the mixture, is detrimental to the efficacy of the lime as a standalone stabilizer, or whether the moisture content was insufficient for the pozzolanic hydration between the specific lime and BA ratio.

As a demonstration, utilizing the CBR values and an assumed design traffic, the effect on road subgrade construction was investigated. The thickness of a road base course layer is a factor of the Design Traffic in Equivalent Single Axle (DESA) of the road and its CBR value. The DESA, utilizing the Weight-In Method (WIM) is determined as a function of the average number of daily heavy vehicles, the average axle groups of the vehicles and the average load on a single axle. Therefore, soils exhibiting poor/low CBR values must require thicker road subgrade and subbases. The construction of excessively thick subgrade layers is avoided as it promotes uneven and non-uniform compaction and moisture content throughout, leading to differential settlement. The compaction of such thick layers requires a substantial compaction effort, which can be time consuming and lead to increased construction costs. For explanatory purposes for the effect of admixtures to road construction processes, a hypothetical situation is utilized here. For a M1 class road (connecting large towns and provincial areas), the average expected axle groups per heavy vehicle is 2.8. Furthermore, the single axle load limit outlined by the Fiji Roads Authority (FRA) is 88,000 kN. Lastly, a daily heavy vehicle usage of 150 vehicles is assumed. Therefore, the DESA is 3.67×10^7. Using the empirical formula/design chart (Figure 16) for subgrade thickness, the subgrade thickness required for the untreated soil is 530 mm. Furthermore the 7%L and 7%L-21%BA reduced the required subgrade layer thickness to 240 mm and 258 mm, respectively. The treated soils satisfy the standard requirements for subgrade thickness, outlined by the FRA. The thickness of subgrades range between 100 mm - 300 mm, depending on the intended use and type of pavement (bound and unbound flexible and rigid pavements [31].

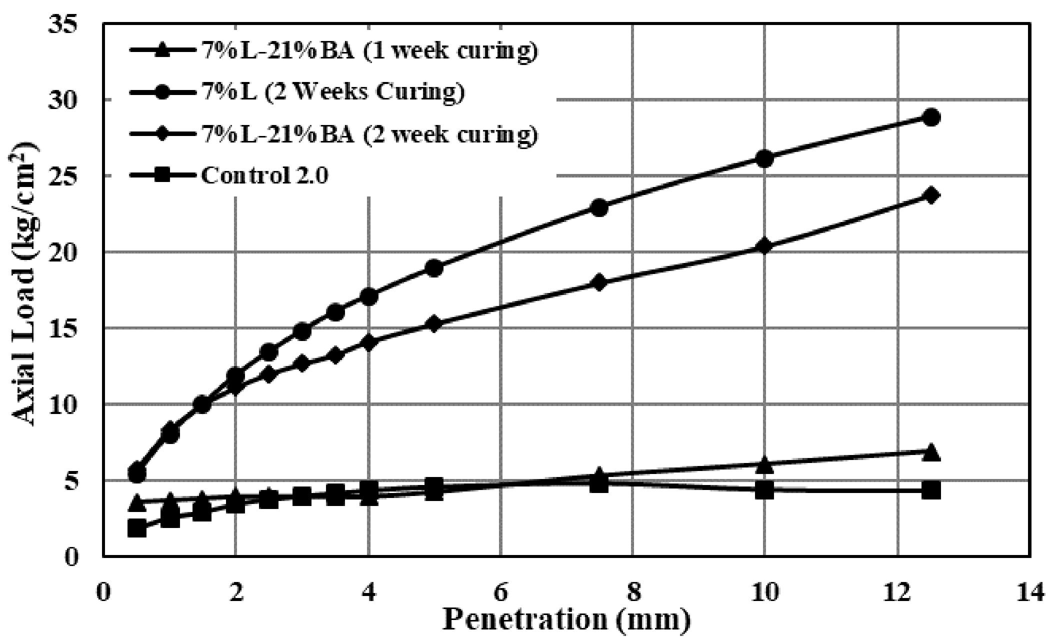

Figure 15.
CBR test for the control sample and optimum ratios.

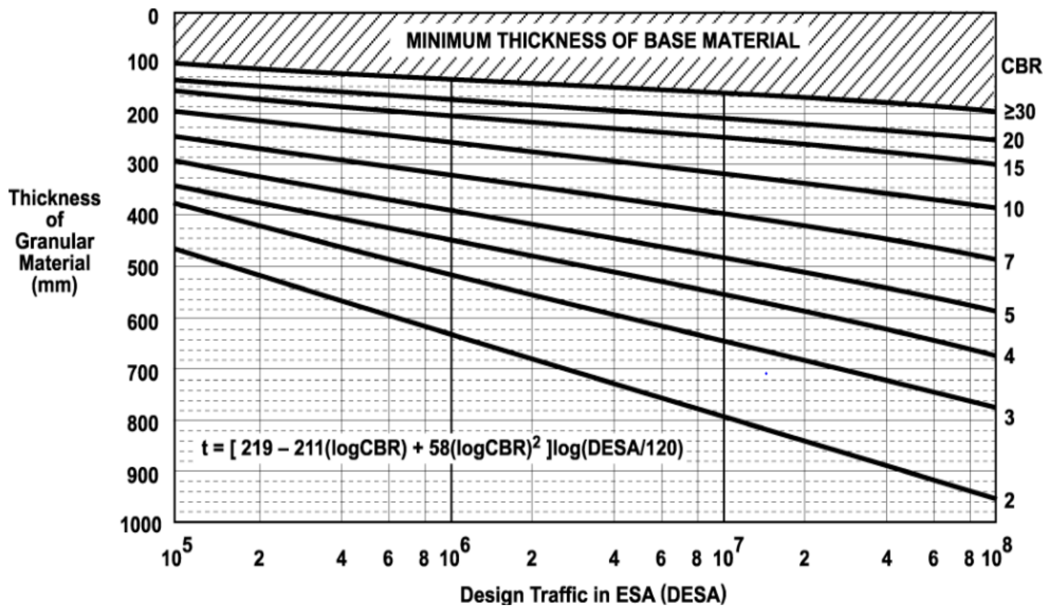

Figure 16.
Road subgrade thickness based on CBR value and DESA [31].

4.7. Direct Shear Test

The shear strength parameters of the soil, cohesion (c) and internal friction angle (Ø), are obtained as shown in Figure 17. The results of the direct shear test for the untreated soil showed that the cohesion (c) was 34.93 kN and the internal friction angle (Ø) was 33.45°. A 7%L-21%BA mixture which was cured within the direct shear box for two weeks and tested, displayed a lowered cohesion of 11.96 kN and increased internal friction angle of 44.48°. The 7%L ratio exhibited a further reduction cohesion, of 7.47 kN and an increase in internal friction angle, 49.16°. The lowered cohesion in the optimum ratios can be attributed to the calcium ions in lime, weakening the electrochemical bonds between soil particles, hence reducing cohesion and increasing plasticity. Furthermore, the presence of L and BA in soil aids in breaking down clay particles to promote flocculation, which leads to increased interlocking of soil particles and increased internal friction angle [32]. From the results, it can be concluded that the 7%L ratio presents the greatest shear strength. Improved soil shear strength parameters display a direct improvement towards embankment/slope stability and soil bearing capacity. To demonstrate the effects of the admixtures on shear strength parameters, the influences on foundation soil bearing capacity was investigated. An example of a typical bearing capacity application is discussed here. A square, 2 m x 2 m, isolated footing is supporting a column load, and has a depth limitation of 1.5 m, due to a high water table. Utilizing Terzaghi's bearing capacity equation; $q_u = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma$ [33] and the shear strength parameters (c and Ø), the effect on the bearing capacity can be observed in Table 2. From the table it can be seen that the 7%L-21%BA and 7%L increased the bearing capacity of the soil by 5607.3 kN and 7849.19 kN, respectively. An increased soil bearing capacity influences the design of shallow foundations by allowing for reduction in foundation size, reduced foundation depth requirements, an increase in load bearing capabilities and reduced settlement.

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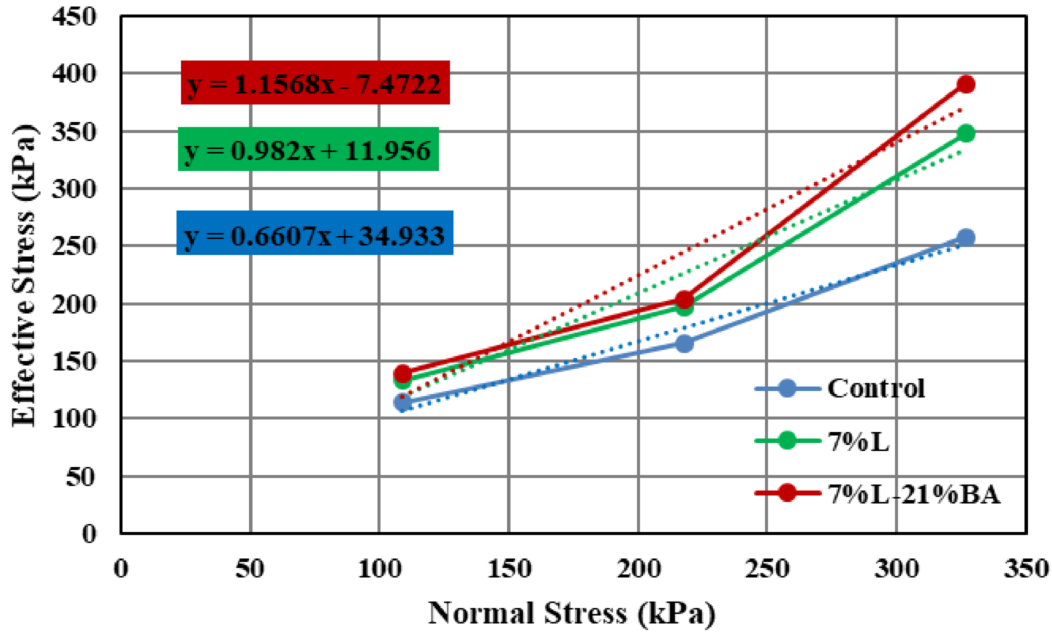

Figure 17.
Effect of optimum ratios on soil shear strength parameters.

Table 2.
Effect on treated soil shear strength parameters on bearing capacity.

	Control	7% L – 21% BA	7% L
N_c	49.68	161.708	298.71
N_q	33.72	160	306.516
N_γ	34.075	292.195	347.5
γ	12.91	13.8321	11.3947
Ø	33.35	44.48	49.16
c'	38	11.956	7.47
q_u (kN)	3459.11	9066.44	11308.3

5. Conclusion

In order to support sustainability and a clean environment, this study evaluated the potential of recycling bagasse ash in combination with lime for stabilizing expansive soils. The impact of combining BA, lime, and BA-L on the mechanical properties of expansive soils was examined through experimental tests that varied the admixtures concentrations and curing times. The following conclusions were drawn.

- The control soil was identified as well graded clayey sand, consisting of inorganic silts and organic clays, and classified as moderately-highly expansive. The CBR value of the control sample was also determined and it was concluded that it would be inadequate in road subgrade applications and would require significant stabilization.
- UCS tests revealed that the L and L-BA ratios attained the majority of their strength after 14 days curing. However, minor strength increases were observed up till 35 days curing.
- The 7% L had the highest compressive strength, followed by the 7%L-21%BA and then the 6%BA. Insufficient moisture content was the suspected cause of poor performance of the L-BA ratio.
- BA is ineffective as a standalone stabilizer. The 7%L and 7%L-21%BA ratios vastly increased the CBR values, however the 7%L ratio was slightly more effective.

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- The 7%L increased the internal friction angle of the control soil by 11.03°, concluding that it was the most effective ratio at increasing the strength parameters of the soil.
- L-BA ratios were the most effective at reducing the free swell ratio and the plasticity index, indicating a lowered susceptibility to volumetric changes.
- Employing bagasse ash in geotechnical applications is environmentally friendly, as it utilizes an otherwise disposed of waste material. The heavy metals present in the ash, when disposed of in dump yards and landfills, pose a threat and could lead to air and nearby water source pollution.

Transparency:
The authors confirm that the manuscript is an honest, accurate, and transparent account of the study; that no vital features of the study have been omitted; and that any discrepancies from the study as planned have been explained. This study followed all ethical practices during writing.

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