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Stabilization of Pavement Subgrade Clay Soil Using Sugarcane Ash and Lime

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Abstract: Soft to medium clay soil possesses major sources of damages to the pavement layers overlying them because of their potential failure under moisture changes and external heavy traffic load. In such situations, soil stabilization methods can be used to improve the soil properties and satisfy the desired engineering requirements. This study presents the use of sugarcane bagasse ash (SBA) and lime as chemical stabilizers for a clay soil subbase. Sugarcane bagasse ash and lime are used individually and as mixtures at varying percentages to stabilize a clay soil from Taxila, Pakistan. Various geotechnical laboratory tests such as Atterberg limits, compaction test, and California Bearing Ratio (CBR) are carried out on both pure and stabilized soils. These tests are performed at 2.5%, 5%, and 7.5% of either SBA or lime by weight of dry soil. In addition, mixtures of lime and SBA in ratios of 1:1, 2:1, 3:1, 1:2, and 1:3 are used in 5%, 7.5%, and 10% of dry soil weight, respectively. Results indicate that soil improved with 7.5% SBA showed a 28% increase in the liquid limit, while soil mixed with 2.5% lime in combination with 7.5% SBA showed an increase of 40% in the plastic limit. For the plasticity index, the soil mixed with 7.5% SBA showed an increase of 42%. Moreover, 2.5% lime in combination with 2.5% SBA showed the best improvement in soil consistency as this mixture reduced the soil plasticity from high to low according to the plasticity chart. Furthermore, 2.5% SBA in combination with 5% lime demonstrated the largest improvement on the CBR value, which is about a 69% increase above that of the pure soil. Finally, the cost analysis indicates a promising improvement method that reduces pavement cost, increases design life, and mitigates issues of energy consumption and pollution related to SBA as a solid waste material.

Keywords: sugarcane bagasse ash (SBA); agricultural waste; lime; clay soil; CBR; pavement subgrade; cementitious materials

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1. Introduction

Different soil stabilization techniques including chemical and mechanical stabilization have been investigated over the last five decades [1–4]. Chemical improvement techniques include different binders such as lime, cement, fly ash, and sugarcane bagasse ash. Koukouzas et al. [1] conducted an extensive and thorough literature review about different soil stabilization, considering different binders and mixing techniques. For chemical stabilization, Koukouzas et al. [2,3] classified fly ash as a high Ca content (CaO: 10–35%) which provides high potential for self-cementing properties and stabilization of soils [4]. Fly ash is usually combined with cement for soil stabilization, with most common cement–FA ratios being 1:1 and 1:4 [5], and with appropriate mixture percentage for soil stabilization ranging between 10% and 20% of dry weight of soil. These percentages lead to an increase in bearing capacity and reduce the necessary pavement thickness and cost.

The surge in industrial agricultural production has led to a significant increase in the generation of residues and ashes. For instance, sugarcane production in Brazil is projected to double from 570 million tons in 2008 to an estimated 1000 million tons in 2020 [6].

Similarly, Australia is expected to produce around 10 million tons of sugarcane by the year 2025 [7]. This emphasizes the urgent need for large-scale landfills to safely dispose and manage non-combustible residues and ashes. Sugarcane bagasse, which is the leftover fibrous material after sugar extraction from processed sugarcane, is a readily available resource from the sugar-refining industry. The locally generated bagasse and those from sugar factories present a problem of handling due to the bulk of the material. When left in an open space, it ferments and decays, therefore necessitating the safe disposal of this solid waste pollutant. Also, when the pollutant is inhaled in large doses, it can cause a respiratory disease known as bagassosis (Laurianne) [8]. Alternatively, this bagasse is usually used as a fuel in cogeneration boilers to generate steam for both sugar production and electricity generation. It undergoes controlled burning at temperatures ranging from approximately 700 °C to 900 °C to maximize its fuel potential. This process results in the formation of bagasse ash characterized by its high content of amorphous silica, low carbon content, and a notably large specific surface area [9]. There is a growing need for public awareness and proper management practices to ensure the safe handling of these materials and prevent any detrimental effects on the environment [10,11]. Soil treatment using this bagasse ash could be a safe way to dispose this waste material and reduce its health and environmental hazards.

The requirement for economical, sustainable, and environmentally friendly materials has increased the interest in using natural fiber in soil stabilization and improvement [12]. Natural fibers have shown good potential when used as reinforcement in different soil and clay adobe bricks [13,14]. One of these natural fibers are sugarcane fibers which are the biproduct of the squeezed sugarcane plants. The potential use of natural fibers in construction applications like compacted clay bricks were investigated by Alavéz-Ramírez et al. [15]. In these cases, [16–18], the natural fibers act as a mechanical stabilizer by adding more interlocking between the soil particles, which in turn increases the strength of the soil containing these fibers. In this process, the natural fiber yarns are randomly distributed throughout the weak soil prior to compaction and the subsequent casting of the adobe bricks. This mechanical process is different from the chemical process which depends on the chemical interaction between the ashes and soil. The global annual production of sugarcane has increased from 933 million Mt in 1985 to 1859 million Mt in 2023 [19]. Brazil is the world's highest producer of sugarcane, followed by Pakistan, India, China, Thailand, Mexico, Australia, and Cuba. For Arab countries, sugarcane is mostly grown in Egypt, with the largest Arab countries producing sugarcane, followed by Sudan, as well as in limited areas in Iraq and Morocco. This huge production of bagasse material could end up at landfills or land spaces if not used in useful engineering applications.

Various studies have shown that the usage of sugarcane ash could be an effective and environmentally friendly approach to strengthen weak subgrade clay soil in road construction. Osinubi et al. [20] performed an experimental program to identify the effect of waste sugarcane bagasse ash on the geotechnical properties of different soil types. They concluded that a small amount of sugarcane ash (i.e., 2%) improved the strength and bearing capacity of the stabilized soil sample. However, the results by Osinubi et al. [20] imply that the SBA cannot be used as a standalone soil stabilizer and requires an activation agent. In another laboratory study by Anupam et al. [21], the potential application of industrial waste materials such as bagasse ash, in enhancing the stability of clay soil specifically for road construction application was investigated. The findings revealed significant improvements in both shrinkage limit and California Bearing Ratio (CBR) values when bagasse ash content was increased from 0% to 25%. However, it was noted that this increase in bagasse ash was accompanied by a reduction in the dry density of the improved soil. Dang et al. and Hasan et al. [22,23] experimentally reported that the implementation of sugarcane ash in addition to lime as strengthening material resulted in a significant increase in bearing capacity and a remarkable reduction in the shrinkage limit of treated soil. These studies provided a practical waste management solution and effective stabilization techniques of weak subgrade soil in road construction.

Experimental study by Silvani et al. [24] used sugarcane bagasse ash (SBA) in a swelling soil treatment. The swelling potential of the improved soil was assessed by a series of one-dimensional swelling tests involving sand–bentonite blends (B–S blend) compacted at different dry densities, with varying levels of SBA substitution. Results revealed that the B–S blends with 12.5% of SBA or more resulted in a swelling factor lower than 0.5% when tested in a one-dimensional swelling test. Madhu et al. [25] used SBA as a filler to enhance the mechanical properties of glass fiber polymeric composites. As compared to unfilled glass fiber composites, the inclusion of 5% bagasse ash in the glass fiber composites increased both the tensile and compressive strength by 11% and 4%, respectively. Mora-Ruiz et al. [26] studied the mechanical and physical properties of an unsaturated compacted mixture of clay soil and sugarcane ash. The study reported that an addition of 8% SBA to the soil decreased the plasticity index by 20% and increased the unconfined compressive strength by 15%. Using a series of experimental tests, Dang et al. [27] reported the effect of bagasse ash (BA), lime (L), and their combination (BA–L) on expansive soil mechanical properties. Results revealed that increasing the additive (BA–L) content significantly enhanced maximum compressive strength by 800% and CBR by nine times. In addition, it reduced the mixture swell potential by 100%, and improved soil compressibility by 83%. Moreover, the effect of the improvements was more prominent in the soil treated with a mixture of bagasse ash and lime (BA–L) compared to soil treated with bagasse ash or lime, individually.

According to Khandelwal et al. [28], SBA can offer a cost-effective, eco-friendly, and sustainable approach to pavement subgrade enhancement. In an experimental study, Pradeep et al. [29] stated that an addition of 20% bagasse ash to expansive soil reduced the plasticity index by 38.5%. However, an addition of 20% bagasse ash mixed with 5% lime results in 80% reduction in plasticity index. In addition, mixing 5% of lime alongside with 15-20% bagasse ash notably decreased the soil swelling potential. The enhanced unconfined compressive strength in lime-modified mixtures is attributed to the cementitious bonding between the clay, lime, and bagasse ash. Teddy et al. [30] studied the Atterberg limits and mechanical properties of expansive soil stabilized with lime and sugarcane ash. They concluded that the addition of SBA–L mixture to expansive soil decreased the Maximum Dry Density (MDD) by 16% and increased the optimum moisture content (OMC) by 90%. This advancement offers potential advantages for stabilizing expansive soil by enhancing the compaction during wet conditions.

Kavak et al. [31] studied the long-term performance of clay soil that had been improved with two different percentages of lime. The samples were prepared and cured in a highly humid room for either 1 month or 10 years. The UCS of the improved soil increased by eight times that of the short-term strength test, while the 10 years' strength increased by 21 times the initial value of the UCS. Based on these results, Kavak et al. [31] suggested that the pozzolanic reactions with lime stabilization may continue in the long term for up to 10 years. Field studies by Péterfalvi et al. [17] documented that a minimum lime stabilization depth of 25–35 cm was required for satisfactory subgrade performance. Based on finite element analysis, Nagrale and Patil [18] estimated the pavement life with subgrade stabilization, revealing a few substantial improvements. The estimated improvement in pavement life for lime, fly ash, and fiber stabilization of subgrade soil increased by 6.49, 4.37, and 3.26 times, respectively. Ghanizadeh et al. [16] investigated the effect of subgrade soil stabilization on the performance and life extension of flexible pavements. The results suggest that the optimum percentage of maximum pavement life are 3% lime for subgrade soil type CL, 6% lime for subgrade type CH, and 15% CFA and CKD for both subgrade soil types. The maximum pavement life increase occurred in the section with the highest stabilization thickness.

In summary, it can be concluded that using sugarcane bagasse ash by itself or mixed with cement or lime improves the properties and consistency of weak soils. However, there is insufficient research on how the SBA waste affects the behavior, strength, and consistency of expansive soils in the presence of hydrated lime. Application of sugarcane bagasse

ash (SBA) in subgrade stabilization not only prevents environmental pollution but also serves as a potential pozzolanic and alumina silicate binder material. Due to its pozzolanic specifications, SBA can effectively replace cement and lime in highway construction and reduce the thickness of different pavement layers. The special pozzolan action of the SBA material is due to the existence of a substance called non-crystalline silica. This feature makes it a good match with hydrated lime; therefore, a mixture of the two could trigger chemical reactions such as a cation exchange, pozzolanic reaction, and cementation effect.

Despite the potential benefits of using SBA in soil stabilization, the existing studies primarily concentrate on evaluating the mechanical properties of soils, such as unconfined compression strength and California Bearing Ratio (CBR) [27,32]. However, for swelling soil such as clay and silty clay soil, the Atterberg limits, index properties, and compaction characteristics are significant in affecting soil classification and strength properties. Therefore, the aim of this study is to determine the suitability of SBA as a stabilizer to improve the engineering properties of silty clay soil in combination with hydrated lime. These environmentally friendly stabilizers can replace the traditional additives such as cement, lime, fiber, or bitumen. The paper addresses the following two problems related to soil stabilization: (1) the effect of SBA, lime, and the SBA–lime mixture on the Atterberg limits, soil classification, and compaction properties; and (2) the effect of stabilizing agents on the measured CBR values at different compaction efforts and percentages of stabilizing agents (lime and SBA). Finally, results of this investigation are used to conduct cost analysis of a hypothetical flexible pavement section constructed on stabilized and unstabilized soil.

2. Materials Properties and Sample Preparation

2.1. Materials Properties

Details of the material properties used in the current investigation including pure soil, sugarcane bagasse ash, and lime, and their mechanical and chemical properties, are presented in this section. A soil sample was collected from the subbase of a road construction site located in Taxila, Pakistan. Soil slumps were crushed with a rubber hammer to loosen the soil particles and remove any organic debris. All tests were conducted at the geotechnical lab located at the University of Engineering and Technology Taxila. Particle size analysis, Atterberg limits, specific gravity, water content, and soil classification were investigated for the natural soil sample following the associated ASTM standards. Physical properties of the natural soil are listed in Table 1, while the chemical properties of the same soil tested by Dang et al. (2016) [22] are presented in Table 2a. Figure 1a,b shows sample of the soil used with some tests that are conducted during this investigation. The soil was classified as Silt with high plasticity (MH) according to Unified Soil Classification System—USCS (ASTM 2018b) [33]. Following AASHTO (1982) [34] classification standard, the soil is classified as A5 (Silty-clay soil). Soils under this class are generally of poor engineering properties and needs to be stabilized if it is subjected to external stresses. In addition, soil with a high percentage of silt content usually exhibits relatively high shrinkage limits, as shown in Table 1. This may lead to a possible volumetric change due to shrinkage and swelling under the variation in water content.

Sugarcane bagasse ash (SBA) was collected from a local sugar manufacturing industry located in outskirts of Taxila city. It was collected at a controlled burning temperature of 600 to 700 °C, depending upon the moisture content of the sugarcane bagasse used to produce the ash. This calcination temperature setting produced an increase in the pozzolanic activity index as stated by [38,39]. Furthermore, de Paula [40] suggested that the optimal temperature for the production of pozzolanic sugarcane ash is 600 °C, and at this temperature, the predominantly amorphous silica with a good pozzolanic activity index would be produced. Raw sugarcane fiber and treated sugarcane bagasse ash (SBA) are shown in Figure 2a,b, respectively. The bagasse ash obtained was pulverized and passed through sieve No. 40, removing the unburnt and large-sized particles, and then through sieve No. 200 to obtain the final bagasse ash stabilizer. Table 2b depicts the chemical composition of the SBA used in this investigation [22]. Hydrated lime (calcium hydroxide)

and sodium chloride (NaCl) were also used in this study and obtained from a local market. Example of the hydrated lime used in this study is shown in Figure 2c.

(a) 3000 gm silty clay soil (b) soil sample and liquid limit test (c) Compaction test (d) CBR test

Figure 1. Soil used, liquid limit test, compaction, and CBR tests.

Table 1. Properties of the natural soil sample.

Characteristics	Value (%)	Standard
Gravel content (diameter > 2 mm)	0.06	ASTM D6913 (2017) [35]
Sand content (0.06 mm < diameter < 2 mm)	10.97	
Silt/clay content (diameter < 0.06 mm)	88.97	
Natural water content	30.76	
Linear shrinkage	21.67	ASTM D4318 (2018a) [36]
Plastic limit	38	
Liquid limit	65	
Plasticity index	27	
Specific surface area (m^2/kg)	1475	
Specific gravity	2.64 ± 0.02	ASTM D5550 (2014) [37]
Classification of the soil, AASHTO (USCS)	A5 (MH)	ASTM D2487 (2018b) [33]

Table 2. Chemical composition of sugarcane ash (SCA) and soil sample.

(a) Soil sample (Dang et al., 2019) [22]			(b) Sugarcane ash (SCA) (Hasan et al., 2016) [23]		
Components	**Abbreviation**	**Content (%)**	**Components**	**Abbreviation**	**Content (%)**
Magnesium Oxide	MgO	2.20	Magnesium Oxide	MgO	1.98
Aluminum Oxide	Al_2O_3	23.0	Aluminum Oxide	Al_2O_3	5.96
Silicon Dioxide	SiO_2	61.8	Silicon Dioxide	SiO_2	77.5
Calcium Oxide	CaO	4.50	Calcium Oxide	CaO	2.40
Iron Oxide	Fe_2O_3	8.50	Iron Oxide	Fe_2O_3	5.3
			Sulfur Trioxide	SO_3	0.9
			Potassium Oxide	K_2O	3.25
			Sodium Oxide	Na_2O	0.53
			Titanium Dioxide	TiO_2	0.38
			Phosphorus pentoxide	P_2O_5	1.10

(a) Sugarcane Bagasse (b) Sugarcane bagasse ash (c) Hydrated lime

Figure 2. (**a**) Sugarcane bagasse, (**b**) sugarcane bagasse ash (SBA), and (**c**) hydrated lime, used in this study.

2.2. Sample Preparations

Dry weights of the different components (soil, SBA, lime, SBA–lime) were weighed and manually mixed using a spoon until a uniform consistency was obtained. The dry weights of all the components followed the percentages presented in Table 3 for different sample mixtures. Tap water was then added at predefined percentages based on the type of tests that need to be conducted (i.e., Atterberg limits, compaction, or CBR) and mixed with the dry components till a homogeneous mixture was reached. The mixtures were then stored in plastic bags for 24 h for better water content distribution throughout the soil specimens. A day after wet mixing, the different samples were prepared for the index tests, compaction, and CBR tests. After the molding process had finished, the set specimen and mold were weighed with 0.01 g accuracy to check the dry unit weight (γ_d). Simultaneously, two small portions of the mix were taken to verify the molding moisture content. Following mixing, samples were subjected to standard geotechnical tests, including index tests, compaction, and California Bearing Ratio (CBR) tests. These tests were conducted to determine key parameters such as the optimum moisture content, maximum dry densities of different admixture combinations, and CBR of both treated and untreated soil samples.

Table 3. Summary of soil samples mixtures used in this study.

Mix No.	Bagasse Ash (SBA) (%)	Hydrated Lime (%)	Mixture Ratio (SBA:L)	Total Additive Content (%)	Notes
1	0	0	0:0	0	Natural soil
2	0	2.5	0:2.5	2.5	
3	0	5.0	0:5.0	5.0	
4	0	7.5	0:7.5	7.5	
5	2.5	0	2.5:0	2.5	
6	5.0	0	5.0:0	5.0	SBA
7	7.5	0	7.5:0	7.5	
8	2.5	2.5	1:1	5.0	
9	2.5	5.0	1:2	7.5	
10	2.5	7.5	1:3	10	
11	5.0	2.5	2:1	7.5	
12	7.5	2.5	3:1	10	

3. Testing Program

3.1. Atterberg Limits

Atterberg limits indicate the consistency of fine-grained soil, and the soil can be in a solid, semisolid, plastic, or liquid state depending on its moisture content. Each state has a unique consistency, behavior, and different engineering characteristics. The significance of the liquid limit is that it indicates the ability of the soil to easily deform under small external

stress. Soils with a low liquid limit are generally more brittle and prone to cracking, while soils with a high liquid limit are more ductile with a resistance to cracking. Determination of the plastic limit (PL) is as important as the liquid limit to determine the plasticity index (PI) of the soil. The significance of the plastic limit is such that it indicates the ability of the soil to support a load without deforming permanently. Soils with a low plastic limit are generally more compressible and may undergo large deformations under external load; however, the opposite is also true. Liquid and plastic limits were tested using Casagrande's liquid limit apparatus, Figure 1b, as per the procedures clearly described in ASTM D 4318 (2018a) [36]. Variations in the volume with water content at different Atterberg limits, and the stress–strain behavior of fine-grained soil, are schematically shown in Figure 3.

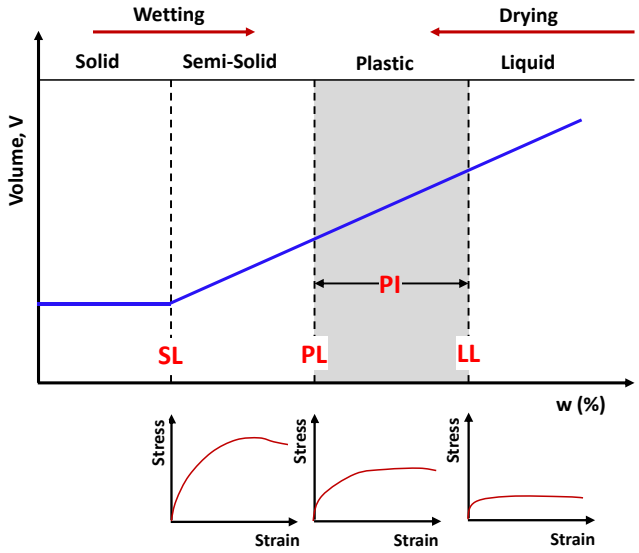


Figure 3. Volume change vs. water content at different consistency limits of fine-grained soil, and stress–strain in each soil state.

3.2. Compaction Tests

Compaction is the densification of soil by a reduction in air voids, while the volume of solids and water content remain unchanged. In the modified compaction test (ASTM D698-78, 2012) [41], 3 kg of oven-dried soil, passed through a No. 200 sieve (i.e., 0.075 mm sieve), was mixed with different percentages of sugarcane bagasse ash, lime, or a mixture of both additives with percentages prespecified according to Table 3. Water was added to achieve a desired water content percentage, based on the mixture's natural moisture content. The soil-stabilizer mixture was then placed in a 4 in mold, compacted in five layers with the 4.5 kg hammer, and subjected to 25 evenly distributed blows per layer (Figure 1c).

Compaction tests were conducted and analyzed following Connelly et al. [42] to determine the maximum dry unit weight (M.D.U.W.) and optimum moisture content (O.M.C.) for the purely untreated soil. Subsequently, various quantities of additives, as specified in Table 3, were meticulously mixed with the pure untreated soil. The resulting blended mixtures were compacted using the same procedure employed for the untreated soil. The dry unit weight of each mixture was obtained and utilized for subsequent geotechnical assessments. It should be noted that swelling potential and volumetric changes were not measured after compaction as this issue is beyond the scope of the current study.

3.3. California Bearing Ratio (CBR) Tests

California Bearing Ratio (CBR) tests are performed to determine the suitability of soil as road subgrade layers, in addition to its reflection of the ultimate soil bearing capacity. The CBR is estimated from the relationship between the force of the plunger and the penetration depth when the plunger penetrates the soil at a standard specified rate. Both treated and untreated specimens were compacted using a fixed-volume mold measuring 7 inches in

height and 6 inches in internal diameter. This was performed at the optimum moisture content (O.M.C) and maximum dry unit weight (M.D.U.W.), following the testing protocol outlined in the Standard Test Method for CBR [43] and shown in Figure 1d. To prevent changes in the moisture content, the soil specimens were securely wrapped with plastic film and stored in a controlled environment with regulated temperature and humidity. After preparation and curing, a 2.5 kPa annular surcharge was applied on top of the soil samples, followed by CBR testing without soaking. Penetrations vs. load values are plotted on a graph and corrected following the procedure specified in the test standard. Corrected stress values at penetration depths of 0.1 and 0.2 inches are divided by standard stresses of 1000 psi (6.9 MPa) and 1500 psi (10.3 MPa), respectively, and presented as percentages. The CBR values are reported based on the greater values observed at 0.1 inch (2.5 mm) or 0.2 inch (5 mm) penetration depths.

4. Results and Discussion

This section presents and discusses the test results of the experimental program conducted in this research paper. First, results of the consistency tests and Atterberg limits are presented and compared for both pure and stabilized soil. Based on the measured plasticity index and liquid limit, pure soil and stabilized soil are classified using the plasticity chart. Second, compaction properties, including optimum moisture content and maximum dry densities, are presented for pure and different stabilized soil samples. Finally, results from the CBR tests are presented and discussed for different soil samples as well. All these results are presented in Tables 4–6, which illustrate the effect of different percentages of lime or SBA in the first three columns of each table. The next three columns of each table present the effects of lime–SBA mixtures on soil properties. If a stabilized soil sample has a 7.5% lime–SBA mixture and is located under the column with SBA = 2.5%, this means that it has 5% lime. If the same soil has a 7.5% lime–SBA stabilizer but is located under the column with lime = 2.5%, this means that the SBA in this soil is 5%. The same notions are applied for other percentages of lime–SBA mixtures. It should be noted that in many cases, multiple tests (index tests, CBR) have been conducted and then the average values of the results were reported in the tables.

Table 4. Effect of lime, SBA, and lime–SBA mixtures on the liquid limit (LL) of stabilized soil.

Lime or SBA (%)	Liquid Limit (L.L.)		Lime–SBA (%)	Liquid Limit (L.L.)	
	Lime	SBA		Lime = 2.5%	SBA = 2.5%
0	64.5	64.5	**0**	64.5	64.5
2.5	54.6	81.7	**5**	43.9	43.9
5	51.1	79.4	**7.5**	47.0	59.8
7.5	51.0	82.4	**10**	51.9	56.9

Table 5. Effect of lime, SBA, and lime–SBA mixture on plastic limit (PL) of stabilized soil.

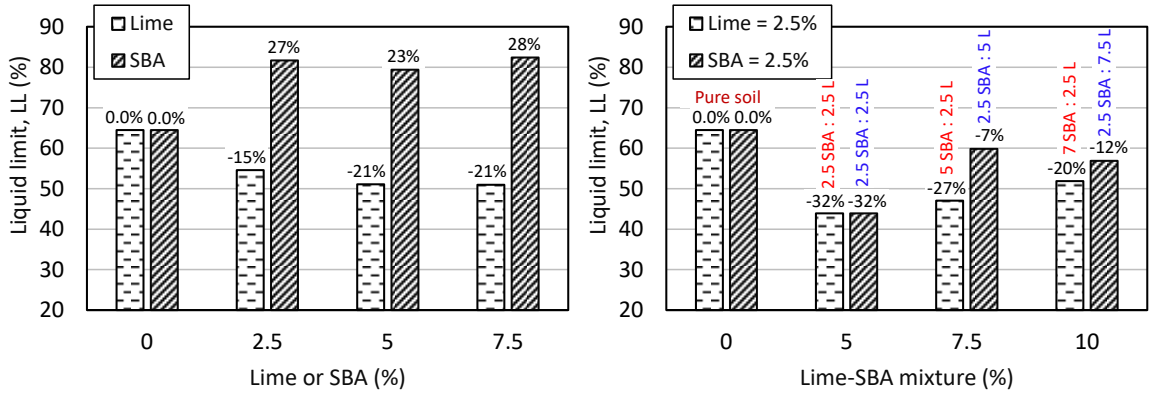
Lime or SBA (%)	Plastic Limit (PL)		Lime–SBA (%)	Plastic Limit (PL)	
	Lime	SBA		Lime = 2.5%	SBA = 2.5%
0	37.96	37.96	**0**	37.96	37.96
2.5	38.44	47.85	**5**	48.98	48.98
5	38.22	45.5	**7.5**	46.11	49.66
7.5	40.54	48.85	**10**	53.1	48.42

Table 6. Effect of lime, SBA, and lime–SBA mixture on plastic index (PI) of stabilized soil.

Lime or SBA (%)	Liquid Limit (PI)		Lime–SBA (%)	Liquid Limit (PI)	
	Lime	SBA		Lime = 2.5%	SBA = 2.5%
0	26.5	26.5	**0**	26.5	26.5
2.5	16.2	33.9	**5**	9.4	9.4
5	12.9	33.9	**7.5**	12.1	10.2
7.5	10.5	37.6	**10**	6.2	8.5

4.1. Index Properties and Classifications

Results of the index properties tests (LL, PL, and PI) on pure soil and the soil stabilized with different percentages of SBA, lime, and SBA–lime mixtures are presented in this section. All tests are conducted according to ASTM standard specifications for each test. The liquid limit results of all soil mixtures are shown in Table 4, and a comparison between the liquid limits of different stabilized soil samples is shown in Figure 4. The liquid limit for pure soil is measured to be 64.5%, and it decreases consistently with increasing lime content. Adding 5% to 7.5% of lime decreases the LL by about 21%. In contrast, the addition of SBA to the soil increases the LL significantly, as shown in Figure 4a. The liquid limits of the soil samples stabilized with different percentages of SBA–lime mixtures are shown in Figure 4b. Reduction in LL due to the addition of an SBA–lime mixture, regardless of the stabilizer content percentage, is clear. Furthermore, the effect of the lime content on the mixture prevails over that of the SBA content. As a conclusion, the effect of SBA content on the liquid limit is more prominent and positive compared to the effect of lime, which reduces LL. Finally, the SBA–lime mixture plays a negative role by reducing the liquid limit. It is well known that soil with a low liquid limit is generally more brittle and prone to cracking, while soils with a high liquid limit are more ductile and resistant to cracking but with high compressibility and swelling potential.



(**a**) Lime or SBA (**b**) Mixture of SBA–Lime

Figure 4. Effect of the (**a**) lime or SBA and the (**b**) mixtures of lime–SBA on the liquid limit (LL) of stabilized soil.

Results of the plastic limit (PL) on pure and stabilized soil with different additive contents are presented in Table 5. Variations in PL versus the different percentages of lime, SBA, and lime–SBA mixture content are shown in Figure 5. Individually adding different percentages of lime or SBA resulted in an increase in PL, as shown in Figure 5a. In addition, the effect of SBA on increasing PL is more significant compared to that of lime. The largest percentage increase in PL is 29% at 7.5% SBA content, while an increase of 6.8% in PL occurs at 7.5% lime content. The addition of the lime–SBA mixture appears to significantly influence the PL compared to the individual influences of lime or SBA, as shown in Figure 5b. It can be seen from the figure that a mixture of 2.5% lime and 2.5% SBA increases the PL by 29%. However, the largest increase in PL occurs at 10% lime–SBA mixture, with 2.5% lime and 7.5% SBA. Finally, the mixture with 2.5% SBA consistently increases the PL by a minimum of 27%, regardless of the percentage of lime in the mixture. The significance of the plastic limit is such that it reveals the ability of the soil to support a load without permanent deformation. Soils with a low plastic limit are generally more compressible and may undergo large deformations under external load.

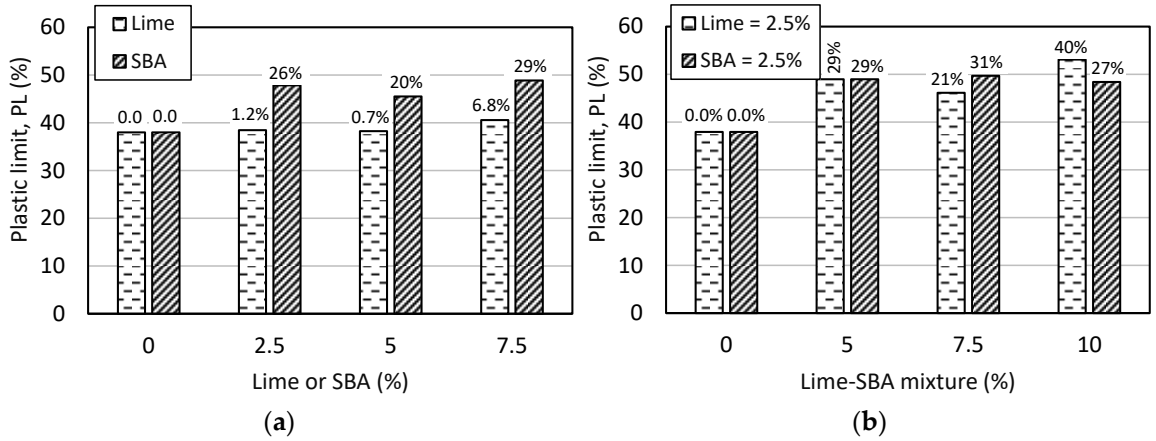


Figure 5. Effect of (**a**) lime or SBA and the (**b**) mixture of lime–SBA on the plastic limit (PL) of pure and stabilized soil.

The plasticity index (PI) is the range of moisture contents over which the soil deforms or behaves plastically (i.e., a measure of soil plasticity). The PI is calculated as the difference between the LL and the PL (i.e., PI = LL − PL). Soil with high PI values generally tends to have a large percentage of clay, while those with a lower PI tend to be majorly silt. In addition, it is always better to reduce the range over which the soil behaves plastically, by reducing the PI. Results of the PI calculated for the pure and stabilized soil are presented in Table 6 for different percentages of lime, SBA, or lime–SBA mixture. Figure 6 shows a variation in the PI at different percentages of lime or SBA (Figure 6a), and lime–SBA mixture content percentages (Figure 6b). It is clearly indicated in Figure 6a that the addition of lime leads to a significant reduction in PI, with an increase in the reduction percentage as the percentage of lime content increased. This agrees with the previous results by Holtz [44] who reported a considerable decrease in the plasticity of soil when treated with lime. In contrast, the addition of SBA led to a significant increase in PI, the percentage of which increased with the SBA content. So, it can be concluded that SBA is not effective in improving the PI of silty clay soil, and lime is preferable as an additive if the PI is the main target for improvement. A large reduction in the PI could be reached if the SBA is mixed with lime at any percentage, as shown in Figure 6b. Any percentage of the lime–SBA mixture reduces the soil's PI by about 54% to 76%, which is a huge improvement in the soil plasticity index. It is well understood that a soil with a high plasticity index PL generally shows a low shear strength response.

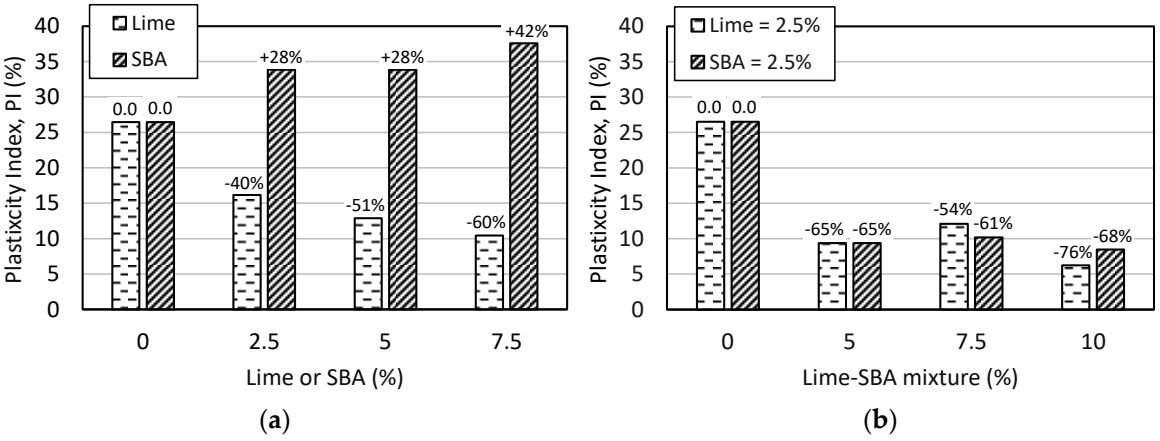


Figure 6. Effect of (**a**) lime or SBA and (**b**) the mixture of lime–SBA on plasticity index (PI).

Figure 7 presents the classification of pure and stabilized soil based on plasticity chart that is the major classification tool in the unified soil classification system—USCS (ASTM 2018b) [33]. The pure soil classification was silt with high plasticity (MH) as stated before and as shown by the black circle in Figure 7. Adding SBA to the soil (i.e., SBA + soil) moved the samples towards the top right corner of the chart. This means that the increase in SBA within the soil increases both the liquid limit and plasticity index, resulting in a stabilized soil with higher plasticity compared to pure soil. Therefore, it can be concluded that the addition of SBA increased the plasticity of the stabilized soil. On the contrary, the addition of lime (i.e., lime + soil) plays a role contrary to that of SAB, which moved the stabilized soil samples to the bottom left corner of the plasticity chart, Figure 7. This means that the increase in lime in the soil reduced both soil plasticity index and liquid limit, which in turn reduced the overall soil plasticity. It can be seen from Figure 7 that the mixture of 2.5% lime and 2.5% SBA has the most positive effect in reducing the plasticity of the soil. In fact, this percentage of the stabilized mixture changed the classification of soil from high plastic silt (MH) for pure soil to low plastic silt (ML) for improved soil, as shown in the plasticity chart.

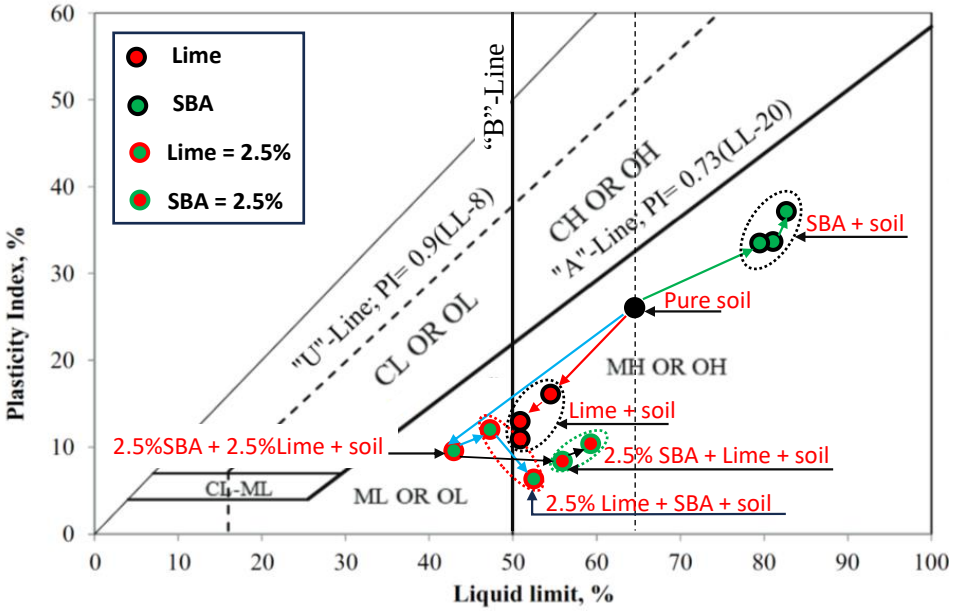


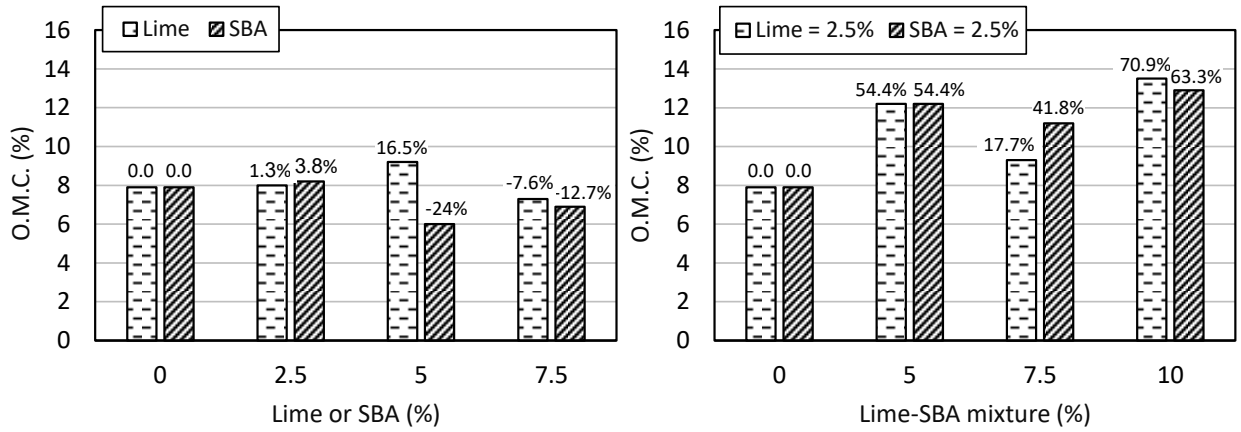
Figure 7. Classification of pure and stabilized soil using the USCS plasticity chart [33].

4.2. Compaction Properties

4.2.1. Optimum Moisture Content (O.M.C.)

The influence of bagasse ash on the optimum moisture content (O.M.C.) is demonstrated for various mixtures, including soil–lime and soil–SBA in Figure 8a, and for soil–lime–bagasse ash in Figure 8b. Figure 8a indicates that in the case of the soil–lime mixture, the optimal moisture content increases as the lime content increases up to 5% and it decreases thereafter. For individual lime content, the largest increase in the O.M.C. is equal to 16.5% at 5% lime content. In the case of the soil–SBA mixture, the O.M.C. increased slightly, up to 2.5% SBA, and then significantly decreased thereafter. It can be seen from Figure 8a that the largest reduction in the O.M.C is equal to 24% at 5% SBA content. In addition, this reduction may be related to the decrease in dry unit weight which leaves more available voids for higher water content to occupy. It should be noted that it is better to reduce the O.M.C. in the compaction process, especially in the areas suffering from freshwater scarcity. For the lime–SBA–soil mixture, Figure 8b indicates that the O.M.C. increased significantly beyond the pure soil at all mixture percentages. The largest increase in O.M.C. was 70.9% at a mixture of 2.5% lime with 7.5% SBA. It should be recognized that the addition of lime

and SBA to the soil increases the surface area of the soil particles which needs more water to cover, which increases the O.M.C. during compaction.

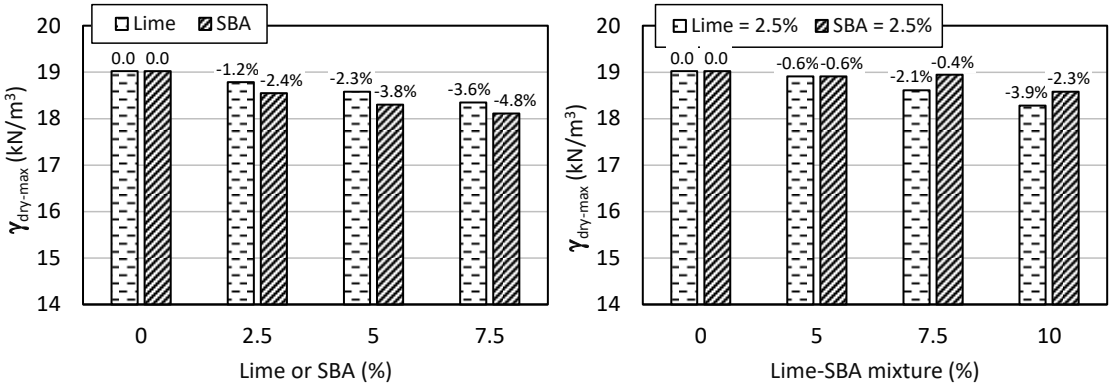


(**a**) Lime or SBA by weight of soil (**b**) Lime–SBA mixture by weight of soil

Figure 8. Effect of (**a**) lime or SBA and the (**b**) mixture of lime–SBA on optimum moisture content (O.M.C.).

4.2.2. Maximum Dry Unit Weight (M.D.U.W.)

Figure 9 depicts the influence of lime, sugarcane bagasse ash (SBA), and the mixture of both lime–SBA on the maximum dry unit weight ($\gamma_{dry\text{-}max}$) of the treated and untreated soil. The modified compaction tests were performed on the untreated soil to ascertain its maximum dry unit weight. The result indicates that the untreated soil achieved a M.D.U.W of 19.0 kN/m^3 at O.M.C of 7.9%. The addition of both lime, SBA, or lime–SBA mixture to the soil in varying percentages leads to reductions in the maximum dry unit weight. The value of the reduction in M.D.U.W. increases as the stabilizer percentage increases, regardless of the stabilizer type. Figure 9a specifically shows that the addition of 2.5% SBA decreases the M.D.U.W. by 2.4% compared to the untreated soil. This trend continues with higher percentages of SBA until 7.5%, which results in a 4.8% reduction in M.D.U.W., compared to untreated soil. For the same percentages of lime, the reduction in maximum dry unit weight is slightly less compared to the reduction resulting from SBA (Figure 9a). Incorporating 2.5% lime yields a 1.2% reduction in M.D.U.W., which is less than the 2.4% reduction that occurs due to 2.5% of SBA. Further, the largest reduction in M.D.U.W. occurs at 7.5% lime content and is equal to 3.6%.



(**a**) Percentage of Lime or SBA by weight of soil (**b**) Mixture percentage of lime–SBA by weight of soil.

Figure 9. Effect of (**a**) Lime or SBA, and the (**b**) mixture of lime–SBA on the maximum dry unit weight ($\gamma_{d\text{-}max}$).

For the mixture of lime and SBA (i.e., lime + SBA), Figure 9b shows an overall reduction in the maximum dry unit weight at any mixture and for any percentage. Furthermore, incorporating 2.5% lime with 2.5% SBA yielded a 0.6% reduction in M.D.U.W. compared to the pure soil. In addition, the mixture that incorporated a larger percentage of lime exhibited the smallest reduction in the dry unit weight as shown in cases with 2.5% SBA + 5% lime and 2.5% SBA + 7.5% lime. In these two cases, the reductions in M.D.U.W. were 0.4% and 2.3%, respectively. The largest reduction for the lime and SBA mixture was in the case of soil stabilized with 2.5% lime + 7.5% SBA, where the reduction was 3.9%.

4.2.3. General Comments on Compaction Properties

Figure 8 indicates that for all stabilizer percentages and mixes, the O.M.C. increases with increasing stabilizer contents, except for the mixes with 5% and 7.5% SBA, as well as 7.5% lime. This increase in O.M.C. was mostly attributed to the pozzolanic reaction of calcium in lime, or silica and alumina in the SBA, with clay fraction of the soil, according to conclusion by Ola [45]. This reaction results in the formation of calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH) which act as binding agents. Additionally, the fine particles of lime or bagasse ash possess a large surface area, necessitating extra water for wetting or water being absorbed by these particles. A similar increase in O.M.C. was noticed by Masued [46] for clay soil stabilized with SBA.

The lower specific gravity of bagasse ash and lime in comparison to pure soil particles further contributes to the observed decrease in maximum dry unit weight. This is added to the natural behavior of silty soil (kaolinite) during its mixing with water, where many of the particles are flocculated in face-to-face arrangements, according to Yong and Warkentin [47]. Each group of face-to-face particles are bonded together due to 'Van der Waals' forces to form a domain. As these domains encounter water, the water is adsorbed between the face-to-face particles and a slight swelling of the domain may occur. This leads to size growth of the flocculated groups of domains, which form apparent aggregates with the addition of more water. During compaction, an insignificant volume change is expected to occur, because the volume of air within the apparent aggregates is very small. However, densification occurs only with the translation and rotation of the aggregates (i.e., aggregates rearrangement). This explains why the maximum dry density does not increase with addition of lime or SBA stabilizer.

Wild et al. (1999) [48] stated that due to cation exchange and flocculation process, the grain size increases, leading to an increase in void ratio and a subsequent decrease in the maximum dry unit weight, further accompanied by an increase in the optimum moisture content. Neubauer and Thompson [49] concluded that lime treatment generally increases optimum moisture content by 2 to 4 percent and reduces dry unit weight by 3 to 4.5 percent. The benefit of this increased O.M.C. and the subsequent decrease in maximum dry unit weight (M.D.U.W.) is that it facilitates easier compaction of the soil when it is damp. Consequently, there is a reduced need to dry the soil to lower its moisture content before compaction, particularly in the field operations.

4.3. CBR Test Results

4.3.1. CBR–Compaction Effort Relation

In pavement engineering, the assessment of California Bearing Ratio (CBR) is routinely conducted as a part of the design phase when evaluating the suitability of construction materials for subgrade, subbase, and base layers. The CBR test is usually carried out on unsoaked and/or soaked conditions; however, in this study, the soaked condition was carried out as it represents the critical state of the tested soil strength required for flexible pavement design according to the ASTM standard D1883-21 (2021) [43]. Practically speaking, soaking accounts for the adverse moisture conditions from potential rainfall or flooding, and most CBR tests have adopted these procedures. Therefore, after the compaction process, samples are soaked in water for 96 h before the CBR penetration tests are conducted.

The ASTM standard D1883-21 (2021) [43] specified that if CBR is required at optimum water content and around the maximum dry unit weight, three specimens from the soil should be prepared at O.M.C ±0.5% and compacted using the specified compaction method but with a different number of blows per layer for each specimen. The number of blows per layer should be varied as necessary to prepare specimens having unit weights above and below the desired value (maximum dry unit weight in this case). Typically, if the CBR for soil at the maximum dry unit weight is required, specimens compacted using 56, 25, and 10 blows per layer is satisfactory, and penetration shall be performed on each of these specimens. CBR test results of such practice are shown in Figure 10, for three specimens compacted at 65, 30, and 10 blows in the compaction mold which is 6.0 inch in diameter. The figure indicates that the penetration load increases as the compaction effort increases. Furthermore, for the range of penetration reached at these tests, there is a linear relationship between the penetration load, P (kN), and penetration depth, δ (in), which takes the form of P = m δ. The slope m is shown in Figure 9 to be equal to 31.7, 26.65, and 20.41 for samples compacted with 65, 30, and 10 blows, respectively.

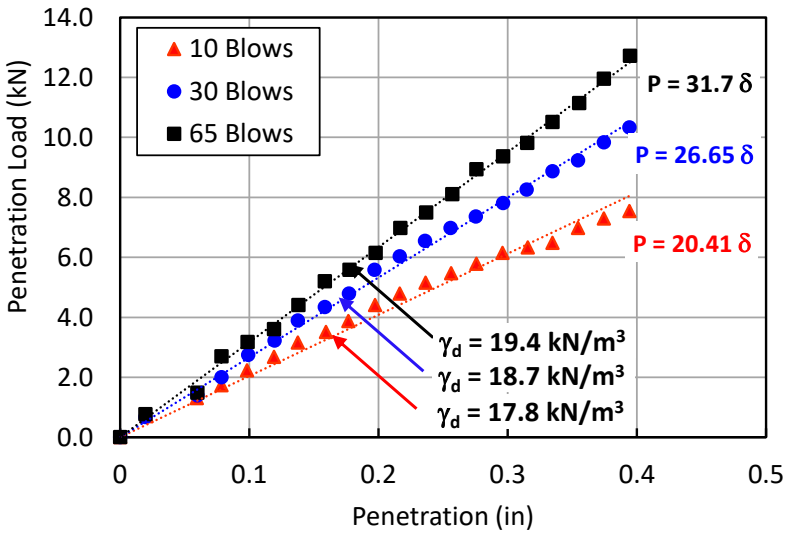


Figure 10. Load–penetration curve from CBR tests for samples compacted at three different compaction efforts.

The three slope values (m) extracted from Figure 10 are plotted in Figure 11a against the number of blows (N). A second order polynomial perfectly fits the relationship between m and N, while the correlation coefficient R^2 = 1.0 is developed. This polynomial can be used at any compaction effort in terms of the number of blows N to interpolate the slope m and formulate the equation P = m δ, which is specified for this compaction effort. This equation can be used to develop the P–δ curve associated with this compaction effort without physically conducting the CBR test in the lab. From the compaction properties, the relationship between the maximum dry unit weight for the three tests presented in Figure 10 and the CBR values are shown in Figure 11b. The figure clearly indicates that as $\gamma_{d\text{-}max}$ increases, the CBR values increase in a linear fashion with R^2 = 0.9982. The CBR values (in percentage) can be interpolated with excellent accuracy from the linear equation shown in Figure 11b, at any maximum dry unit weight. It should be noted that the curves and equations in Figure 11 are different for the different soil types or different percentages of lime or SBA.

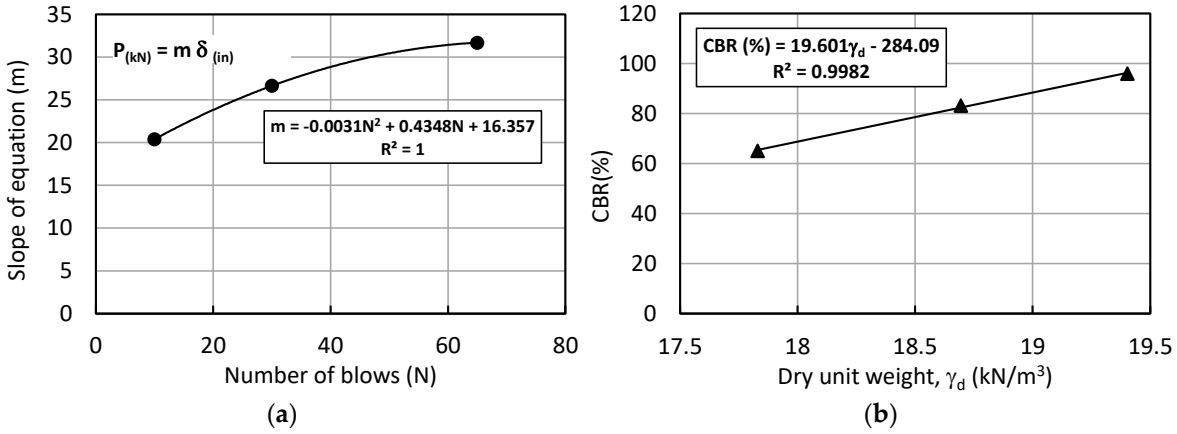


Figure 11. Effect of compaction effort and dry unit weight on CBR values (samples with 7.5% lime). (**a**) Slope (m) vs. number of below (N). (**b**) CBR vs. dry unit weight.

4.3.2. CBR of Different Stabilized Soils

Figure 12 presents the variation in CBR with different percentages of lime, SBA, and lime–SBA mixtures. In general, it can be stated that the CBR of fine-grained soil increases with lime percentage and decreases with SBA percentage (Figure 12a). A maximum of about 39% increase in CBR resulted from the addition of 7.5% lime, and about 41% decrease in CBR resulted from the addition of 7.5% SBA. Therefore, it is advised to use lime over SBA to improve CBR of the fine-grained soil. It should be noted that lime and SBA in Figure 12a are used individually as stabilizers. However, mixtures with different percentages of lime–SBA are used for soil stabilization, and the CBR resulting from these mixtures are shown in Figure 12b. It is clear from the figure that, regardless of the percentages of lime and SBA in the mixture, the CBR increases as the percentage of mixture added to the soil increases. The only exception is the soil mixed with 2.5% lime + 7.5% SBA, which has a negative effect on CBR value due to the larger portion of the SBA. The highest improvement on CBR occurs at 5% lime + 2.5% SBA, and this increases the CBR value by about 69% compared to untreated soil. The improvements in CBR and strength of treated soil when mixed with a mixture of lime and SBA are attributed to the development of cement-like compounds, specifically calcium silicate hydrates (CSH) and calcium aluminate hydrates (CAH). These compounds result from the interaction of calcium sourced from lime with the easily accessible silica and/or alumina found in both the soil and bagasse ash. Comparable to the products formed in Portland cement, CSH and CAH constitute the matrix responsible for reinforcing the stability of the improved soil layers.

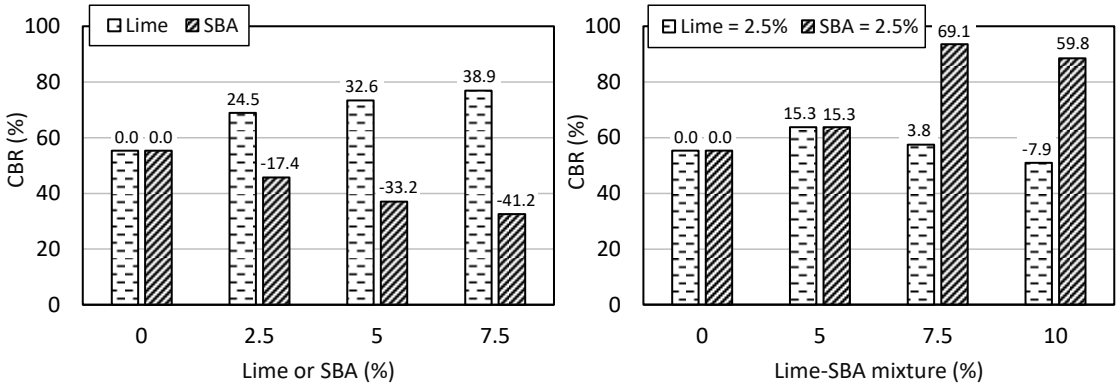


Figure 12. Effect of (**a**) lime or SBA and the (**b**) mixture of lime–SBA on CBR values.

4.4. Cost Analysis

This section describes the economic benefits from this study in terms of cost analysis of a hypothetical road constructed on natural subgrade compared with the construction of the same road on stabilized subgrade. The initial construction costs of the flexible pavement are calculated based on government rates for various tasks involved in its construction process, utilizing the CSR 2014 rates of NHA [50] for Lahore. This computation considered rates for seven different materials and all other associated construction expenses. Additionally, the life cycle of the pavement in both cases is assessed using AASHTO 1993 [51] standards. The flexible pavement design equation is as follows:

$$\log_{10} W_{18} = Z_R S_o + 9.36\,[log_{10}(SN+1)] - 0.20 + \frac{log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40+\left[\frac{1094}{(SN+1)^{5.19}}\right]} + 2.32\,log_{10} M_R - 8.07 \quad (1)$$

where:

W_{18}: Predicted number of 80 kN (18,000 lb.) ESALs.

Z_R: Standard normal deviation from table in AASHTO (1993) [51] for flexible pavement design.

S_o: Combined standard error of the traffic prediction and performance prediction.

SN: Structural number (an index indicative of the total pavement thickness).

ΔPSI: Difference between the initial design serviceability index and the design terminal serviceability index.

M_R: Subgrade resilient modulus (in psi).

A flexible pavement section of unit length (1 km) is considered to have a 3.7 m lane width and 1 m shoulders, and it is supposed to be designed at a reliability R = 95% which results in an associated standard normal deviation $Z_R = -1.645$. Based on the initial serviceability index and terminal serviceability index, ΔPSI is calculated as 1.7. Subgrade resilient modulus M_R, which is the most important parameter in this analysis, is directly related to the CBR using the famous Heukelom and Klomp [52] relationship as follows:

$$M_R = 1500 \times CBR \quad (2)$$

where M_R is the resilient modulus (psi) and CBR is California Bearing Ratio (%). Equation (2) is used for untreated soil, and M_R is determined from the CBR value to be 33,206 psi. For treated soil with the largest CBR value, which is achieved with a soil sample containing a mixture of 2.5% SBA with 5.0% lime contents, M_R is calculated to be 46,475 psi. Despite the availability of various relations between CBR and M_R, Equation (2) is used in this work as it is adopted by AASHTO for most practical applications. It is also known that the resilient modulus is dependent on the applied stress level, according to Rada and Witczak [53], and there is a wide variation in the resilient modulus value that can be obtained using the CBR, depending on the plasticity properties of the soil. After estimating the structure number SN from Equation (1), thicknesses of the flexible pavement layers are then calculated for the treated and untreated subgrade samples and shown in Table 7. It can be seen that the thicknesses of the individual layers reduced when calculated using the modified M_R for treated soil. This indicates that the thickness of the pavement layers is directly related to the M_R value of subgrade soil. For the thickness of stabilized subgrade soil, the Department of Transport and Main Roads determined that a lime-stabilized subgrade should ideally be at least 250 mm, preferably 300 mm, in thickness for optimal performance, based on a field study [54]. Similarly, field studies by Péterfalvi et al. [17] confirmed a minimum lime stabilization depth of 250 to 350 mm for satisfactory subgrade performance.

Table 7. Calculated thickness of pavement layer for untreated and treated soil.

Pavement Layer	Material	Layer Coeff.	Thicknesses, in (cm)	
			Untreated Soil	Treated Soil
Wearing course	Hot mix asphalt	α_1 = 0.44	D_1 = 2.0 in (5.08)	D_1 = 1.7 in (4.318)
Base course	Aggregate-Bituminous	α_2 = 0.30	D_2 = 5.0 in (12.7)	D_2 = 4.5 in (11.43)
Sub-base	Crushed stone	α_3 = 0.11	D_3 = 7.0 in (17.78)	D_3 = 6.0 in (15.24)

Regarding cost estimates, the initial site preparation cost which includes removal of vegetation and trees, and compaction of natural ground is same for both cases. Material, labor cost, equipment, overhead, and rehabilitation cost for the life of the pavement for both untreated and treated subgrades are considered. Detailed calculation of construction costs indicates that there is a 16% reduction in the cost of flexible pavement constructed on treated subgrade soil with a specified thickness compared to untreated soil. The design life of the pavement is also calculated for both cases, and it is found that design life of the pavement is increased from 12 years for the pavement on untreated subgrade to 25 years for the pavement on treated subgrade soil with thicknesses shown in Table 7. In conclusion, treating the subgrade with 2.5% SBA and 5% lime increases soil resilient modulus (M_R) and therefore, the thickness of the pavement layers reduces, which results in cost reduction and the longer design life of the pavement.

5. Conclusions

This study investigated the potential of utilizing sugarcane bagasse ash (SBA), a byproduct of the sugar industry, in combination with lime (L), to enhance the stability of weak fine-grained soil, aiming to contribute to environmental cleanliness and sustainability. The research involves a series of experimental tests, varying amounts of SBA, lime, and their mixture (SBA–L). Results demonstrate that these additives lead to significant improvements in various index and strength properties of stabilized soil, with the SBA–L mixture outperforming the effect of SBA and lime individually. Based on results analysis of experimental tests, the following conclusion can be summarized:

- Addition of lime or lime–SBA mixture reduces the soil liquid limit; however, addition of SBA individually increases the soil liquid limit. The largest reduction in L.L. occurs at a mixture of 2.5% SBA with 2.5% lime, and the largest increase in the L.L. occurs at 7.5% SBA content.
- The plastic limit (PL) increases as the lime, SBA, or SBA–lime mixture contents increase in the soil. The largest increases occur at 7.5% SBA and at a mixture of 2.5% lime with 7.5% SBA.
- The plasticity index (PI) decreases with the percentage of lime or lime–SBA mixture; however, it increases significantly with the increase in SBA content in the soil. This is the same trend as the liquid limit variation. To reduce the soil plasticity index, a mixture of lime and SBA should be added to the soil at any individual percentage. This mixture is expected to reduce PI by an average of 65% and may change soil classification from high to low plasticity soil.
- Adding lime, SBA, or mixture of lime–SBA leads to a decrease of up to 4.8% in the maximum dry unit weight and an increase in the optimum moisture content of up to 70%. The increase in the optimum moisture content is significant, while the decrease in the maximum dry unit weight is insignificant.
- A linear relationship between penetration load (kN) and penetration (in) exists in CBR tests at different dry unit weights, and the variation in CBR value with dry unit weight is also approximately linear.
- The CBR values of stabilized soil increase as the lime percentage or the mixture of lime–SBA percentage added to the soil increases, regardless of the percentages of lime and SBA in the mixture. The individual addition of SBA significantly reduces the CBR

value of the stabilized soil. The best improvement in CBR value occurs at a mixture of 5% lime with 2.5% SBA.

- The use of untreated soil as pavement subgrade yielded a projected design life of 12 years; however, if the subgrade soil is treated with a mixture composed of 5% lime with 2.5% SBA, it yields a project design life of 25 years, which is double the life of untreated subgrade. Furthermore, a cost–benefit analysis indicates a reduction in the overall costs of the project with the implementation of improvement measures for subgrade soil.
- The findings from this study are anticipated to significantly expedite the utilization of recycled sugarcane ash (SBA) in engineering projects, particularly in bolstering pavement subbase and in the production of sustainable road and railway embankment materials.

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