Expansive Soil Stabilization Using Lime and Bottom Ash to Increase the CBR Value: A Case of Cililin Soil

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ABSTRACT

Soil stabilization plays a crucial role in enhancing the bearing capacity of soils to support structural loads, particularly in road construction projects involving soft soils with inherently low strength. This study investigates the feasibility of using bottom ash, a coal industry waste, as a soil stabilizing agent. Utilizing bottom ash not only aims to improve soil mechanical properties but also addresses environmental concerns associated with coal waste disposal. Laboratory tests were conducted to evaluate the index properties (specific gravity, Atterberg limits, and grain size distribution) and engineering properties (compaction characteristics, swelling behavior, and unconfined compressive strength) of soil treated with varying proportions of bottom ash. The results demonstrate that the addition of bottom ash significantly enhances the soil's bearing capacity and reduces swelling potential, indicating its suitability for application as a subgrade layer in pavement construction. The addition of 13% lime combined with varying percentages of bottom ash significantly improved the soil's properties, with CBR values increasing to between 11% and 15%, compared to the untreated soil. This study highlights the potential of bottom ash as an effective and sustainable soil stabilizer, contributing to both infrastructure development and industrial waste management.

Keywords: Bottom Ash, Coal Waste, Expansive Soil, Properties of Soils, Shrinkage, Soil Stabilization

1. INTRODUCTION

Coal combustion produces a huge amount of solid waste in the form of bottom ash (BA) and fly-ash (FA), which needs to be utilized rather than disposing of, otherwise, it will produce environmental pollution and also it is harmful to humans in the form of respiratory problems [1]. Coal combustion generates solid waste in the form of fly ash and bottom ash. Fly ash consists of fine particles that rise to the top of the combustion furnace and function as filler material. In contrast, bottom ash is the coarser residue that settles at the bottom of the furnace. Both types of coal combustion waste are classified as hazardous and toxic materials, requiring proper management before disposal in designated sites [2]. One such disposal site is located in the Cililin area of West Bandung Regency, near residential neighbourhoods. The proximity of this site to inhabited areas raises concerns about environmental pollution and health risks to local communities, particularly due to coal dust exposure, which can adversely affect the respiratory system when inhaled over prolonged periods.

Expansive soil is characterized by volume changes in response to variations in moisture content [3], a phenomenon particularly influenced by seasonal fluctuations common in Indonesia. The expansive soil is a problematic type of soil in a construction project, especially for the construction of a base ground of a building because it contains montmorillonite minerals with the largest surface area and is very easy to absorb water [4]. These volume ch`anges can cause significant damage to structures built on such soils, including roads and buildings. The swelling behaviour of expansive soil occurs in two main stages: the crystalline stage and the osmotic stage [5]. The crystalline stage involves cation hydration triggered by water absorption, followed by the osmotic stage, during which cation transfer is driven by osmotic pressure resulting from hydration in the crystalline stage. This process leads to soil expansion that can uplift and damage structures.

The results of the study [6] indicate that the greater the addition of bottom ash, the higher the CBR value. Another research [7], shown that stabilization (fly ash, bottom ash, or FABA) results in changes in physical and mechanical residual soil properties with an increase in the CBR value in addition to fly ash at 10% with a CBR value = 4.02%, bottom ash at 20% with a CBR value = 9.49% and FABA at 20% with a CBR value = 14.32%. Through a series of laboratory tests [8], with variations in the composition of the fly ash mixture between 15% to 45% and bottom ash between 15% to 25%, a significant increase in the CBR value was obtained compared to the original soil without treatment. Beside the changes on CBR values, soil with the addition of 0% and 20% Fly ash and Bottom ash at variations of 0%, 10%, 20% and 30% showed a decrease in the value of the soil plasticity index from 21.083% to 7.511% [9]. This study aims to evaluate the feasibility of using bottom ash, a byproduct of coal combustion, as a stabilizing material for expansive soil. Soil Stabilization is a soil improvement effort making possible to improve soil that has a low bearing capacity [10]. Soil stabilization aimed to improve the nature properties of soil foundation with the additional materials in increasing shear

strength [11]. It often demonstrates a high plasticity index (PI), low unconfined compressive strength (UCS), and low California Bearing Ratio (CBR), making it inadequate for construction purposes without treatment [12]. The key research question is: Can bottom ash improve the engineering properties of expansive soil to make it suitable for construction applications? Specifically, the objectives are to analyze the physical and mechanical properties of expansive soil treated with various proportions of bottom ash and to assess its potential use as a subgrade material in road construction.

2. METHODOLOGY

This research utilized a quantitative methodology, involving experimental testing carried out at the Soil Mechanics Laboratory of Politeknik Negeri Bandung. The collected soil samples were visually classified as expansive soils.

2.1 Index Properties Testing

The index properties of the soil were determined through a series of laboratory tests, including specific gravity measurement, atterberg limits determination, grain size distribution analysis, and evaluation of expansive soil characteristics.

2.2 Soil Sample Collection

Soil samples were obtained from seven distinct locations, each identified by specific geographic coordinates Cililin 1 (6°55'41.15"S, 107°28'26.81"E), Cililin 2 (6°55'31.87"S, 107°28'14.83"E), Cililin 3 (6°55'31.49"S, 107°28'15.41"E), Cililin 4 (6°55'40.36"S, 107°28'27.19"E), Majalaya (7°4'14.81"S, 107°47'20.16"E), Margaasih (6°57'45.94"S, 107°32'25.69"E), and Maroko (6°54'29.5"S, 107°26'32.7"E).

2.3 Soil Classification

Following the determination of index properties, the soil samples were classified according to the AASHTO and Unified Soil Classification System (USCS) standards.

2.4. Assessment of Expansive Potential

The expansive potential of the soils was evaluated and categorized based on the classification criteria proposed by Skempton [13].

2.5 Engineering Properties Testing

Selected soil samples underwent further testing to determine their engineering properties, which includes standard proctor compaction test, measurement of swelling potential, California Bearing Ratio (CBR) test, as well as unconfined compression test.

2.6. Soil Stabilization

Soil stabilization was carried out by preparing mixtures of soil with bottom ash and lime. The lime content was maintained at 13%, to achieve the target improvements without causing excessive stiffness or brittleness, and it balances cost-effectiveness and environmental considerations.The proportion of bottom ash was varied at 10%, 15%, 20%, and 25%.

2.7. Mixtures Materials

Limestone used for soil stabilization is a powder (lime) or crushed aggregate form, with the density around 2.3 - 2.7 g/cm³. Chemically, lime modifies clay minerals and induces cementing reactions, and mechanically, crushed limestone improves soil texture and load capacity.

3. RESULTS AND DISCUSSION

3.1 Index Properties

Soil classification plays a fundamental role in determining the bearing capacity of subgrade soils used in road construction. The classification of soil provides essential information about its physical and engineering properties, such as particle size distribution, plasticity, compressibility, permeability, and shear strength—all of which directly influence the soil's ability to support loads without excessive deformation or failure.

Coarse-grained soils generally exhibit higher bearing capacities compared to fine-grained soils like silts and clays. This is largely due to their better drainage characteristics and higher shear strength, which enable them to withstand heavier loads with minimal

settlement. Well-graded granular soils tend to have low void ratios and high friction angles, contributing to enhanced load distribution and resistance to deformation. In contrast, fine-grained soils, particularly those with high plasticity and high clay content, pose significant challenges for road bearing capacity. Expansive clays, for example, undergo volumetric changes with moisture fluctuations, leading to swelling and shrinkage that compromise the stability and integrity of road structures. Such soils usually have low shear strength and high compressibility, resulting in higher settlement under load and reduced load-bearing capacity. The presence of silt and clay fractions also reduces soil permeability, leading to water retention and weakening of the soil structure under saturated conditions. Accurate soil classification is essential for assessing the bearing capacity of subgrade soils and determining the necessary design and treatment measures in road construction. Recognizing the limitations and strengths of each soil type allows engineers to optimize pavement structures, improve load distribution, minimize maintenance, and extend the lifespan of road infrastructure. The soil used in the index properties testing was collected from various locations, with the selected samples exhibiting prominent cracks in dry conditions. Subsequently, the soil with these characteristics was subjected to index properties in an undisturbed condition.

3.1.1 Index Properties Testing

The soil specific gravity testing was conducted in the laboratory, referring to the "SNI – 1964 – 2008", with the laboratory test results shown in Table 1. The specific gravity values obtained range between 2.17 – 2.57. According to Wesley, the tested soil is classified as clay, indicating a significant presence of organic material.

Table 1 Soil Specific Gravity Testing Result

Cililin 1	Cililin 2	Cililin 3	Cililin 4	Margaasih	Majalaya	Maroko
2.33	2.38	2.53	2.55	2.17	2.57	2.52

3.1.2 Atterberg Limit

The result of the Atterberg Limit tests, comprising the liquid limit and plastic limit of the soil samples, were conducted in accordance with "SNI 1996:2008" and "SNI 1967:2008:" standards. The result is presented in Table 2:

Table 2 Results of Soil Atterberg Limit Testing

Soil Sample	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (PI)
Cililin 1	42.12	20.37	20.75
Cililin 2	53.4	42.3	11.04
Cililin 3	109.2	49.7	59.56
Cililin 4	70.7	35.9	34.82
Maroko	52,35	42.57	9.60
Margaasih	57.1	52.40	4.69
Majalaya	50.7	32.1	18.60

3.1.3 Grain Size Analysis

The result of the Atterberg Limit tests, comprising the liquid limit and plastic limit of the soil samples, were conducted in accordance with "SNI 1996:2008" and "SNI 1967:2008:" standards. The result is presented in Table 3.

Table 3 Result of Grain Size Analysis Testing

Soil Sample	% Clay	%Grain Size Loose 200
Cililin 1	42.12	93,82 %
Cililin 2	37.85	98.30 %
Cililin 3	42.31	97.91 %
Cililin 4	51.35	97.98 %
Maroko	19.93	92.10%
Margaasih	18.40	97.34%
Majalaya	61.26	98.82%

3.1.4 Swelling Potential Analysis

The expansive degree analysis was conducted based on the methods proposed by Skempton [13] The test results are presented in the Table 4: Based on the expansive activity analysis, the results indicate the presence of expansive activity in the soil samples. The Cililin 4 sample was selected for further testing due to its proximity to the research object and ease of collection. The Cililin 3 and Maroko samples were excluded from the engineering properties testing due to the limited amount of available soil.

Table 4 Swelling Potential Analysis

Soil Sample	% Clay	Plasticity Index	Activity	Swelling Potential	
				Skempton (1953)	Chen, Raman (1967)
Cililin 1	42.12	20.75	0.46	Low	Medium
Cililin 2	37.85	11.04	0.30	Low	Low
Cililin 3	42.31	59.56	0.73	Low	High
Cililin 4	51.35	34.82	0.66	Low	Medium
Maroko	19.93	9.60	0.48	Low	Low
Margaasih	18.40	4.69	0.30	Low	Low
Majalaya	61.26	18.60	0.33	Low	Medium

3.2 Soil Classification

3.2.1 Classification Based on AASHTO

The AASHTO classification is conducted to determine the soil type based on the results of the Atterberg limit tests. Figure 1 presents the soil classification according to AASHTO for all tested soil samples. Based on the results of the laboratory tests and subsequent analysis, the soil samples were classified under the AASHTO system as clays and silts. These soil classifications are typically deemed unsuitable for use as base materials in embankment construction due to their inherent low shear strength, high plasticity, and proneness to volumetric changes under varying moisture conditions, which can adversely affect the structural stability and performance of the embankment.

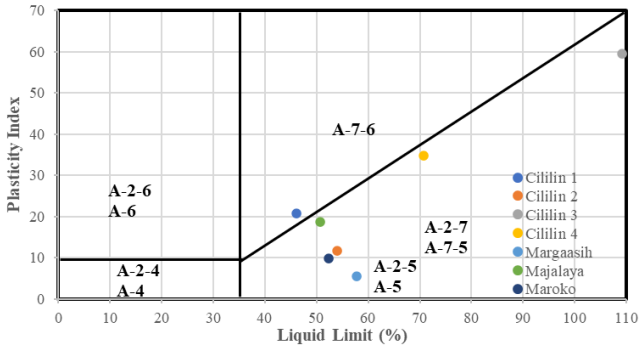

Figure 1 Soil Distribution Graph Based on AASHTO

3.2.2 Classification Based on USCS

The USCS classification aims to determine the soil type based on the results of the Atterberg limit tests and grain size analysis. Figure 2 present the soil classification according to USCS.

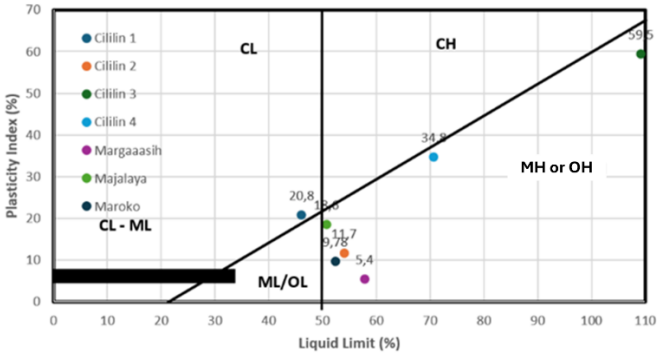

Figure 2 Soil Distribution Graph Based on USCS

3.3 Engineering Properties Testing

The Engineering Properties Testing was conducted on the Cililin 4 soil sample, which was selected based on its expansive potential.

3.3.1 Standard Compaction

The standard compaction test is intended for common embankment soil. The test refers to SNI 03-1742-1989, with the test result as presented in Table 5, Figure 3, and Figure 4. The test results indicate an increase in Optimum Moisture Content (OMC) and a decrease in Maximum Dry Density (MDD) in soil mixtures with the addition of lime. This phenomenon occurs due to the flocculation and agglomeration of clay particles induced by lime, which results in the formation of larger aggregates and increased interparticle voids. These larger voids require more water to lubricate the particles and achieve effective compaction, leading to a higher OMC. Simultaneously, the increased void spaces reduce the overall packing efficiency of the soil, causing a decrease in MDD. This behaviour aligns with soil stabilization theory, where chemical reactions between lime and clay minerals alter the soil structure, increasing workability but reducing density.

Table 5 Results of Standard Compaction Testing

Sample	*Maximum Dry Density, MDD(gram/cm³)*	*Optimum Moisture Content,OMC (%)*
Soil	*1.290*	*28.4*
Soil + 13% CaO	*1.280*	*33.4*
Soil + 13% CaO + 10% Bottom Ash	*1.326*	*30.6*
Soil + 13% CaO + 15% Bottom Ash	*1.337*	*30.3*
Soil + 13% CaO + 20% Bottom Ash	*1.357*	*30.0*
Soil + 13% CaO + 25% Bottom Ash	*1.363*	*29.8*

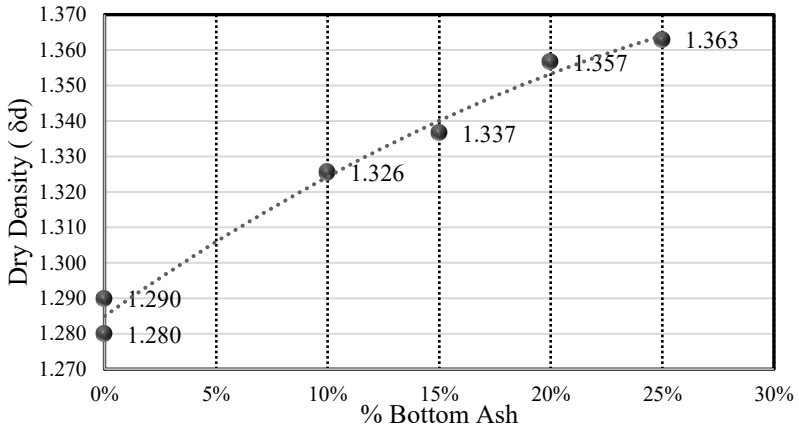

Figure 3 Graph of Dry Density Variation

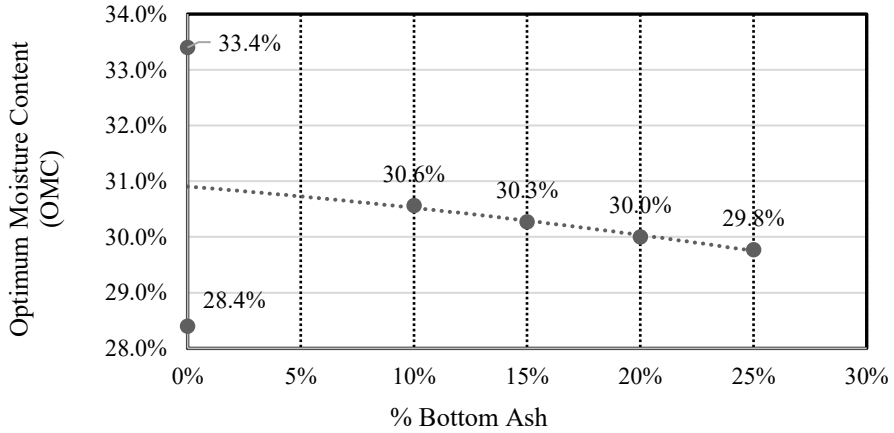

Figure 4 Graph of OMC Variation

In contrast, the addition of bottom ash results in a trend of increasing MDD and decreasing OMC. This can be attributed to the granular nature of bottom ash, which acts as a physical filler that occupies the voids between soil particles, thereby improving soil gradation and packing density. The decrease in OMC indicates that less water is required to reach maximum compaction, as the voids

are more efficiently filled, and the soil matrix becomes denser. According to basic soil mechanics principles, better particle size distribution and reduced void ratio enhance dry density and reduce the water demand during compaction. This contras behaviour between lime and bottom ash highlights the fundamental difference in their stabilization mechanisms—lime modifies the soil chemically and changes the particle arrangement through agglomeration, while bottom ash enhances the soil physically by filling voids and improving density.

3.3.2 Swelling

The swelling test is conducted to determine the percentage of soil expansion due to water saturation. The testing procedures and calculations refer to "SNI 6424:2008", with the test results presented in Table 6: A decrease in soil expansion was observed with each addition of stabilizing additives. The first mixture, which included lime, showed a significant reduction in soil swelling. This can be attributed to the chemical reaction between lime and the clay minerals present in the soil, particularly those with high activity such as montmorillonite. Lime induces cation exchange and pozzolanic reactions [14], which lead to the formation of cementitious compounds like calcium silicate hydrates (CSH) and calcium aluminate hydrates (CAH). These reactions result in stronger and more stable interparticle bonds, reducing the soil's affinity for water and thus its tendency to expand upon wetting. Additionally, the lime treatment alters the soil fabric by flocculating clay particles, reducing the double-layer thickness and limiting the water absorption capacity, which further contributes to reduced swelling.

In contrast, the addition of bottom ash led to a less pronounced reduction in soil expansion. This is because bottom ash primarily functions as a non-reactive filler material. It does not chemically alter the clay minerals but instead improves the soil's physical structure by filling voids and improving particle packing. This denser packing limits the space available for moisture ingress and reduces the free swell potential to some extent. However, without a strong chemical interaction, its effect on expansive behaviour is relatively limited compared to lime.

Table 6 Swelling Test Results

Sample	*Swelling Potential* (%)	*Swelling Preassure* (kPA)
Soil	3.80	91.38
Soil + 13% CaO	0.51	26.36
Soil + 13% CaO + 10% Bottom Ash	0.46	26.05
Soil + 13% CaO + 15% Bottom Ash	0.30	16.67
Soil + 13% CaO + 20% Bottom Ash	0.25	15.69
Soil + 13% CaO + 25% Bottom Ash	0.20	14.43

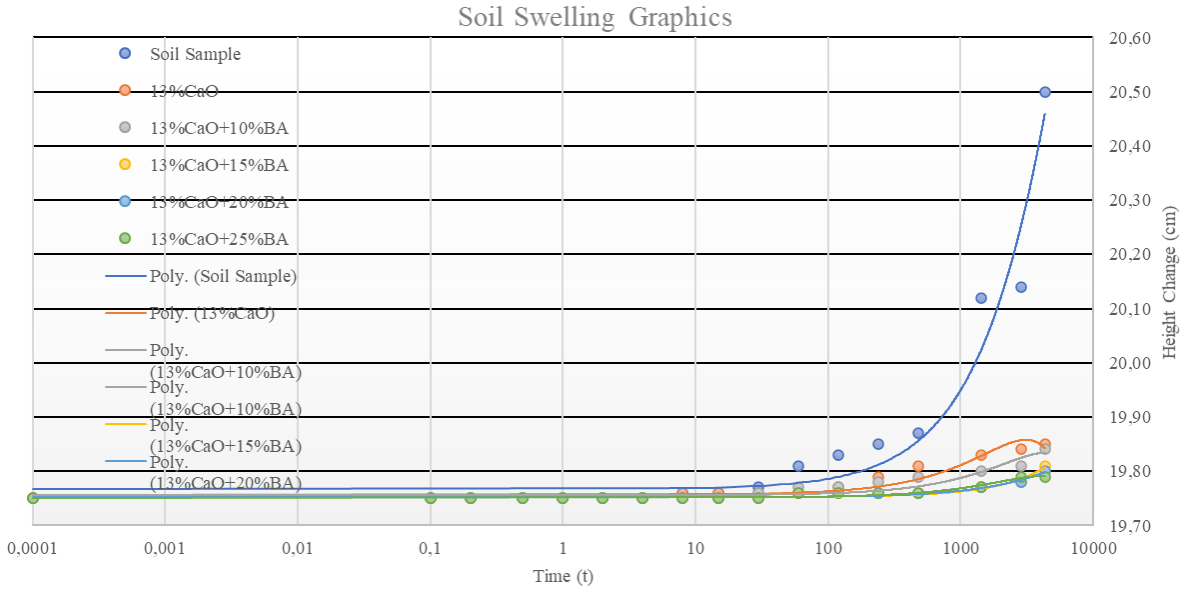

Figure 5 Soil Swelling Graphics

3.3.3 California Bearing Ratio (CBR)

The California Bearing Ratio (CBR) test was conducted in accordance with SNI 03-1744-1989, and the result shown in Table 7 and Figure 6. This test aims to determine the CBR of soil compacted in the laboratory at its optimum moisture content. CBR represents the ratio between the penetration load of a material and the standard load at the same depth and penetration rate.

Table 7 California Bearing Ration Test Results

Sample	CBR *Design Unsoaked* (%)	CBR *Design Soaked* (%)
Soil	6.91	4.04
Soil + 13% CaO	20.74	10.96
Soil + 13% CaO + 10% Bottom Ash	22.86	11.39
Soil + 13% CaO + 15% Bottom Ash	23.89	13.80
Soil + 13% CaO + 20% Bottom Ash	26.77	15.26
Soil + 13% CaO + 25% Bottom Ash	27.3	15.61

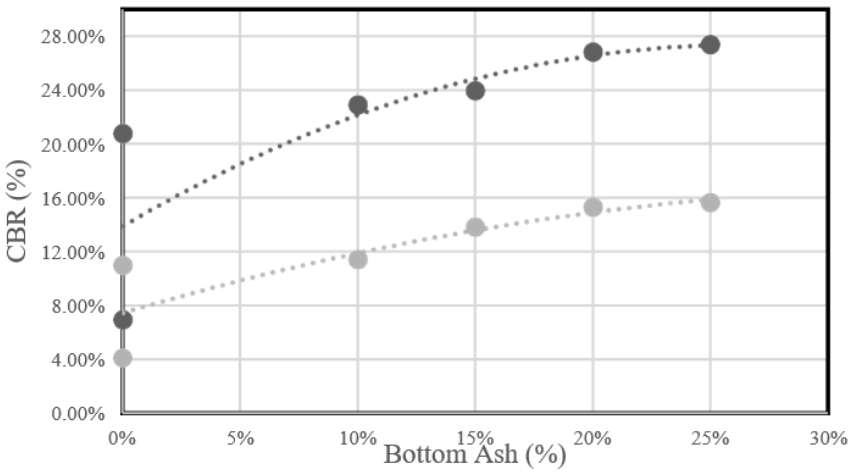

Figure 6 Graph of CBR Improvement

The increase in CBR values in the lime mixture was highly significant, with the 13% CaO mixture showing a 297% increase compared to the soil in the unsoaked condition. This result is attributed to the reaction between the soil and lime, which forms strong bonds between clay sheets. The addition of bottom ash resulted in a less significant improvement, indicating that bottom ash does not substantially enhance soil properties.

3.3.4 Unconfined Compression Test

The Unconfined Compression Test aims to determine the shear strength of soil (q_u) under undrained, unconsolidated conditions. The test was conducted on disturbed/remoulded soil at optimum moisture content (OMC) according to "SNI 03-3638-1994," with the results presented in Table 8 and Figure 7. The lime mixture demonstrated a 190% increase in unconfined compressive strength compared to the remoulded soil. In contrast, the addition of bottom ash resulted in only a marginal improvement, with the 10% bottom ash mixture showing a 6% increase in strength compared to the 13% lime (CaO) mixture. This difference is mainly due to lime's strong binding properties, which promote chemical reactions that enhance soil cohesion, while bottom ash primarily acts as a filler that fills soil voids and provides limited chemical stabilization.

Table 8 Unconfined Compression Test Results

Sample	Unconfined Compression Strength, q_u (kg/cm^2)
Soil	0.68
Soil + 13% CaO	1.27
Soil + 13% CaO + 10% Bottom Ash	1.355
Soil + 13% CaO + 15% Bottom Ash	1.418
Soil + 13% CaO + 20% Bottom Ash	1.554
Soil + 13% CaO + 25% Bottom Ash	1.560

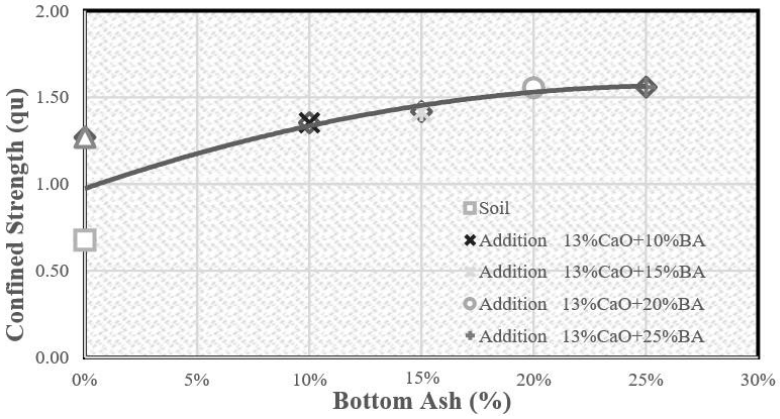

Figure 7 Graph of Soil Compression Strength Improvement

The comparatively low improvement in strength from bottom ash can be attributed to its limited pozzolanic activity, meaning it does not actively react with soil minerals to form cementitious compounds as lime does. Coal combustion wastes like bottom ash improve soil gradation and reduce void spaces but do not significantly increase strength on their own. From a practical standpoint, this indicates that bottom ash alone may not sufficiently enhance the load-bearing capacity required for road subgrade applications. However, it can serve as a useful additive when combined with reactive stabilizers such as lime, improving soil properties while contributing to sustainable industrial waste management.

3.4 Relationship Between Test Results

3.4.1 Compaction Curve Relationship

The compaction curve relationship aims to observe the relationship between changes in optimum moisture content and dry density among each stabilized mixture. Figure 8 show the compaction curve test result.

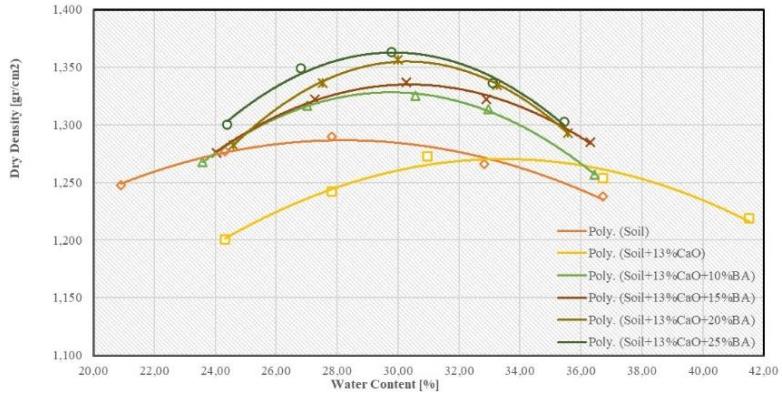

Figure 8 Compaction Curve

In the testing of soil and mixtures, an increase in optimum moisture content and a decrease in maximum dry density were observed in the soil-lime mixtures. This phenomenon can be explained by the flocculation and agglomeration of clay particles induced by lime, which causes the formation of larger soil aggregates and consequently increases the volume of voids within the soil matrix. These larger void spaces require additional water to achieve optimal compaction, thus raising the optimum moisture content while reducing the maximum dry density. Lime alters the soil structure by promoting particle binding rather than simply filling voids.

Conversely, the addition of bottom ash resulted in an increase in maximum dry density and a decrease in optimum moisture content. This trend suggests that bottom ash primarily acts as a granular filler, occupying the void spaces between soil particles and enhancing particle packing efficiency. By filling these voids, bottom ash reduces the soil's overall porosity and moisture-holding capacity, leading to denser soil mixtures that require less water to reach optimum compaction. Therefore, the contrasting effects of lime and bottom ash on soil compaction parameters reflect their fundamentally different mechanisms: lime chemically stabilizes and restructures the soil fabric, while bottom ash physically fills voids to improve particle packing and density.

3.4.2 Relationship between Swelling Test Results

Swelling Test Results produces data on swelling potential and swelling pressure, with the relationship between the test results as depicted in Figure 9. As presented in Figure 9, the percentage of swelling is directly proportional to the swelling pressure. This relationship indicates that as the soil undergoes greater volumetric expansion due to moisture absorption, a correspondingly higher swelling pressure is required to restrain or reverse this expansion and return the soil to its original volume. Swelling pressure arises because expansive clay minerals, such as montmorillonite, absorb water and increase in volume, generating internal stresses within the soil matrix. These stresses exert an outward force against any confining structure, which must be overcome by external pressure to control soil deformation. Therefore, the positive correlation between swelling percentage and swelling pressure reflects the inherent tendency of expansive soils to exert greater pressures as their volumetric strain increases. This behaviour is critical in foundation and pavement design, as insufficient resistance to swelling pressure can lead to structural damage, including cracking and uplift.

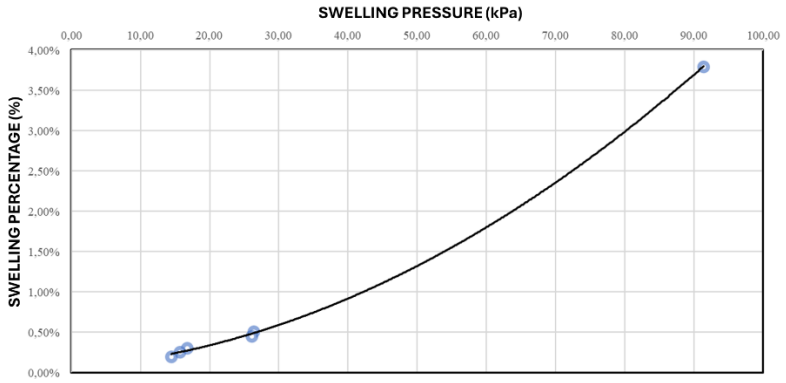

Figure 9 Relationship of Swelling Test Results

3.4.3 Relationship between Optimum Moisture Content and Void Ratio

The relationship between optimum moisture content and void ratio aims to review the changes that occur in the void ratio along with the addition of additives. Figure 10 shows that the void ratio of soil mixed with lime is higher than that of the untreated soil and other soil mixtures. This increase in void ratio can be attributed to the flocculation and agglomeration of clay particles caused by lime treatment. Lime induces chemical reactions that alter the soil fabric by binding fine particles into larger, more porous aggregates, which results in the formation of larger and more open void spaces within the soil structure. An increased void ratio signifies greater pore volume, which in turn leads to a higher optimum moisture content, as more water is required to fill these expanded voids to achieve maximum compaction.

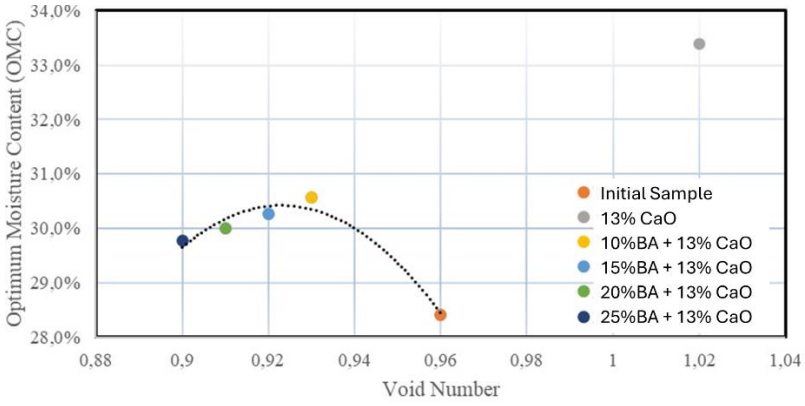

Figure 10 Relationship of Void Ratio and OMC

3.4.4 Maximum Dry Density with Unconfined Compression Strength and CBR

The relationship between maximum dry density and UCS with CBR aims to determine the effect of mixture density on UCS and CBR. Figure 11 and Figure 12 show that the increase in dry density is directly proportional to the California Bearing Ratio (CBR) and the unconfined compressive strength (UCS) of the soil. This relationship suggests that as the soil becomes denser, its ability to

resist external loads and deformation improves significantly. Higher dry density reflects a more compacted soil structure with reduced void ratios and improved particle interlocking. This denser configuration enhances frictional resistance and cohesion between particles, leading to greater shear strength and stiffness—key factors that contribute to higher UCS values.

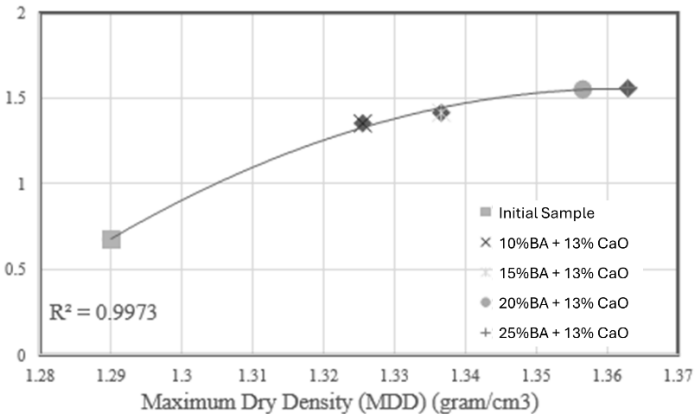

Figure 11 Comparison of MDD with Unconfined Compressive Strength

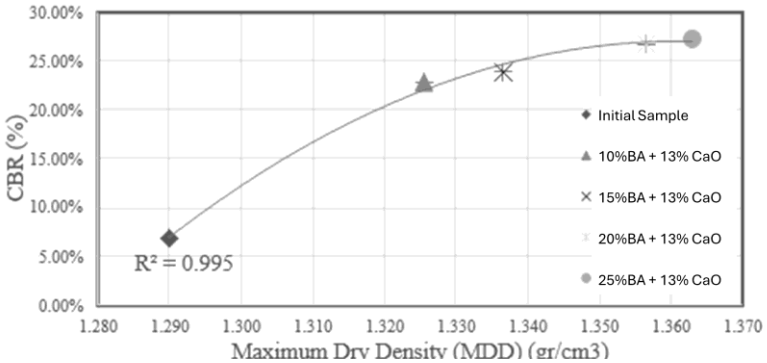

Figure 12 Comparison of MDD with Unsoaked CBR

Similarly, the increase in dry density also improves the CBR, which is a measure of the soil's load-bearing capacity. A well-compacted soil can distribute loads more efficiently and is less susceptible to settlement or deformation under applied stress. This is particularly important in pavement and subgrade design, where a higher CBR value indicates better performance and durability under traffic loads. Thus, the positive correlation among dry density, CBR, and UCS reflects fundamental principles of soil mechanics, where improved compaction results in enhanced mechanical behaviour and structural reliability of soil materials.

4. CONCLUSION

This study investigated the stabilization of expansive soil from Cililin using bottom ash and lime, yielding significant improvements in its geotechnical properties. Based on index property tests and soil classification systems, including AASHTO and USCS, the native Cililin soil was identified as an organic silty clay with moderate swelling potential. The soil exhibited a neutral pH of approximately 7, which broadened to a range of 6 to 9 upon the addition of bottom ash. Initially, the soil demonstrated a California Bearing Ratio (CBR) of 4.04% and a swelling potential of 3.8%, indicating its inadequacy for use as a subgrade material. However, stabilization with 13% lime combined with varying percentages of bottom ash significantly enhanced the soil's engineering performance. The treated mixtures showed improved CBR values ranging from 11% to 15% and a notable reduction in swelling characteristics, making the soil more suitable for supporting structural loads. Furthermore, based on the Bina Marga standard, which requires a minimum soaked CBR value of 7%–20% for subgrade applications, the stabilized soil mixtures in this study met the necessary criteria, confirming their viability as subgrade fill materials even under adverse conditions.

This study is limited by its focus on soil samples collected from specific locations within the Cililin area, which may affect the generalizability of the results to other regions or soil types. Additionally, the sample size and variation of soil and bottom ash compositions were constrained, limiting the scope of conclusions. Laboratory conditions may not fully replicate field conditions,

especially regarding curing times and environmental factors. Future studies should explore a wider range of soil types and geographical locations to validate the applicability of bottom ash and lime stabilization techniques. Long-term field trials are recommended to assess durability and performance under real environmental conditions. Furthermore, investigating the environmental impact of bottom ash disposal and stabilization, including leachate behaviour and soil-plant interactions, would provide valuable insights for sustainable practice.

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