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Experimental Investigation and Multi Parameter Strength Optimization of Crushed Stone with Sustainable Lateritic Soil Blends: A Case Study in Rwanda

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ABSTRACT

The rising cost of conventional stabilizers like cement and lime has intensified the search for cost-effective, locally available alternatives in road construction. This study investigates the use of abundant lateritic soils in Rwanda to enhance the strength of crushed stone for base course applications. Laboratory investigations were conducted on crushed stone blended with lateritic soil as a binder at proportions of 0%, 4%, 6%, 10%, and 12%. A comprehensive suite of geotechnical tests, including Sieve Analysis, Atterberg Limits, Modified Proctor Compaction, California Bearing Ratio (CBR), and Direct Shear tests, was performed. The results indicated a significant improvement in the geotechnical properties of the blended material. The optimum lateritic soil content was found to be 10%, yielding the highest CBR value of 130% and a maximum dry density of 2.181 g/cm³ at an optimum moisture content of 9.1%. A trade-off between shear strength parameters was observed: cohesion increased, and the internal friction angle decreased with increasing lateritic soil content. The study concludes that a 10% lateritic soil blend provides a sustainable, cost-effective, and technically sound solution for optimizing the strength of crushed stone base courses in Rwanda.

Keywords: Atterberg Limit; Base Course; Binder; California Bearing Ratio; Compressive Strength; Crushed Stone; Fluconazole; Lateritic Soil; Stabilization; Soil Blends; Shear Strength.

1.0. Introduction

1.1. Background

Road infrastructure serves as a critical catalyst for socio-economic development (Khanani et al. 2021; Nawir et al. 2023; Tafida et al. 2024; Mbereyaho et al. 2025), facilitating trade, mobility, and access to essential services such as healthcare, education, and markets (Elburz and Cubukcu, 2021; Shi et al. 2024). In developing nations like Rwanda, where economic growth is closely tied to agricultural productivity and market accessibility, the quality and extent of road networks directly influence poverty reduction and regional development. Rwanda's Vision 2050 strategy explicitly emphasizes the development of robust, sustainable infrastructure as a cornerstone for national transformation (Perez-Guzman et al., 2023; Malbila et al. 2026; S. Daga-ang et al., 2026), recognizing that efficient transport systems are indispensable for achieving middle-income status. In Rwanda, road transport dominates as the primary mode for both passenger and freight movement, carrying over 90% of domestic cargo and the majority of passenger traffic. The national road network, systematically categorized under Law No. 55/2011 into National, District Class 1, District Class 2, and Specific roads, forms the structural backbone of the country's transport system. Despite substantial investments and ongoing development initiatives through organizations like the Rwanda Transport Development Agency (RTDA), a significant portion of the district road network remains unpaved or inadequately maintained. This infrastructure gap creates a pressing need for innovative, cost-effective construction methodologies that can accelerate pavement provision while maintaining stringent technical standards and ensuring long-term durability (Abosede et al., 2020).

1.2. Research Objectives

1) To investigate the influence of lateritic soil content on the geotechnical properties of crushed stone base materials.

2) To determine the optimal blend proportion that yields maximum strength and performance.

3) To evaluate the compaction behavior of stabilized mixtures.

4) To analyse the strength characteristics using CBR and shear strength parameters.

5) To assess the sustainability and economic viability of lateritic soil stabilization.

2.0. Literature Review

The base course represents a fundamental structural layer in flexible pavement for long-term design and durability requirements (Wu et al. 2017; Mshali and Steyn, 2020; Mousavi et al. 2021; Mir et al., 2025), strategically designed to possess sufficient strength and stiffness to effectively distribute traffic-induced stresses to the underlying subgrade. This load-distribution function is critical for preventing premature pavement failure and ensuring satisfactory performance throughout the design life. Conventionally, high-quality crushed stone aggregates have been the material of choice for base course construction due to their excellent mechanical properties and drainage characteristics (Ma et al. 2022; Gao et al. 2023; Sujatha and Deepa Balakrishnan 2023). However, the extensive reliance on these premium materials within Rwanda's rapidly expanding construction industry has precipitated increased material costs (Dedek et al. 2012; Ntitanguranwa et al. 2018), growing scarcity due to intensive quarrying operations, and environmental concerns related to landscape degradation and ecosystem disruption. This economic and environmental challenge necessitates the exploration of innovative techniques to stabilize and enhance locally available, marginal materials to meet technical specifications without prohibitive expense. The concept of mechanical stabilization through blending offers a promising solution, particularly in regions with abundant alternative materials that can be optimized through scientific proportioning. Lateritic soils (Yamus et al. 2019; Guimarães et al. 2025; Ukwizagira et al. 2025), which are abundantly available along road corridors throughout Rwanda's tropical landscape, present a particularly promising, sustainable alternative for mechanical stabilization applications. These reddish, iron-rich weathered materials, formed through intensive laterization processes under tropical conditions, offer valuable fine-grained fractions that can potentially enhance the properties of coarse aggregates when blended in optimal proportions.

The fundamental principle underlying this research draws an interesting parallel from advanced materials science. Co-crystal engineering in pharmaceuticals allows for the creation of new solid forms with improved properties without altering the chemical identity of the active ingredient. Analogously, in geotechnical engineering, the strategic "blending" of soils and aggregates can create a new composite material with superior mechanical properties compared to its individual components through optimized particle packing, enhanced interlocking, and improved stress distribution (Gettu et al. 2021; Gan et al. 2025). As illustrated in Figure 1, this study applies this

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fundamental principle to systematically optimize the strength characteristics of crushed stone by blending it with varying proportions of lateritic soil, aiming to determine the precise optimal proportion that maximizes performance in road base courses under Rwandan conditions while promoting sustainable construction practices. The research addresses a critical knowledge gap by providing quantitative, experimentally validated data on the optimization of local material blends specifically for Rwandan geological and climatic conditions. By establishing the optimal lateritic soil content that enhances crushed stone performance, this study contributes to sustainable infrastructure development, cost reduction in road construction, and the promotion of circular economy principles in the construction sector. The findings have significant implications for road agencies, construction companies, and policymakers seeking to balance infrastructure development with environmental sustainability and economic efficiency.

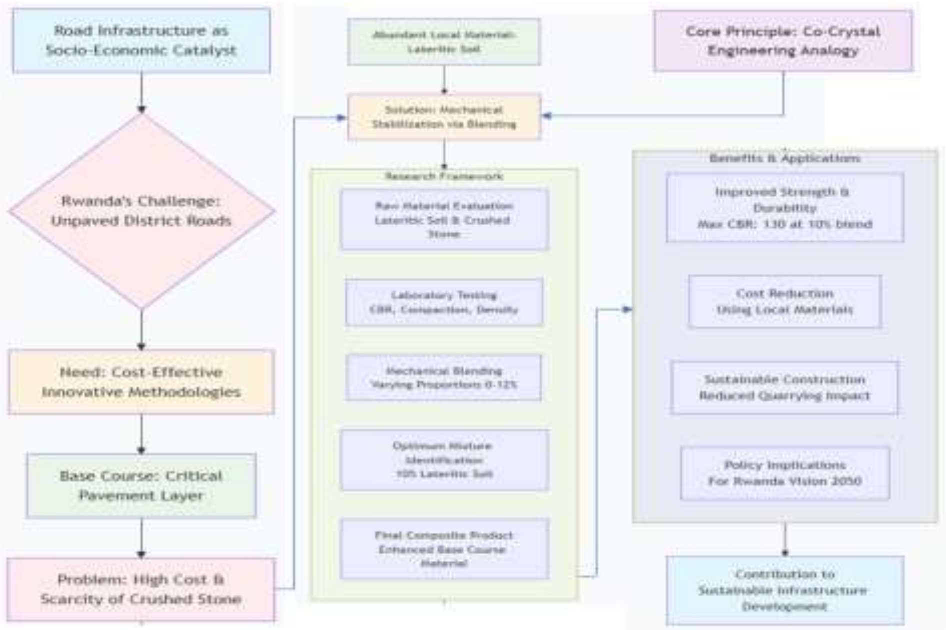

Figure 1. Conceptual framework for optimizing road base course materials using local lateritic soil in Rwanda.

The schematic outlines the research approach from problem identification (unpaved roads, material scarcity) to solution development (mechanical stabilization via blending), through systematic experimental phases (material evaluation, testing, blending optimization), to final outcomes (enhanced material properties, cost reduction, sustainability benefits) that contribute to national infrastructure goals.

3.0. Methodology

3.1. Materials Collection and Characterization

Lateritic soil samples were systematically collected from an active road construction site in the Huye District of Southern Province, Rwanda, to ensure representativeness of local geological conditions and practical relevance to actual construction practices. The sampling followed a randomized grid pattern across the exposed lateritic profile to capture inherent material variability, with composite samples obtained from multiple locations within the site to ensure homogeneity. The crushed stone aggregate was sourced directly from a local quarry within the same geological formation to maintain consistency with the materials typically used in Rwandan road projects. Prior to

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blending operations, both materials underwent comprehensive initial characterization to establish baseline engineering properties. The lateritic soil was subjected to natural moisture content determination, specific gravity testing using the pycnometer method, and visual classification to confirm its lateritic characteristics, including distinctive reddish coloration and sesquioxide enrichment. The crushed stone aggregate was visually inspected and confirmed to be predominantly composed of sound and durable granite-gneiss rock fragments, typical of local quarry products in the Rwandan geological context. Preliminary tests confirmed the aggregate's general conformity with standard specifications for base course materials, providing a reference point for evaluating blend improvements. Preliminary tests confirmed the aggregate's general conformity with standard specifications for base course materials, providing a reference point for evaluating blend improvements. The systematic methodology for material collection and characterization is summarized in Figure 2.

3.2. Experimental Design and Sample Preparation

A rigorous experimental matrix was designed with five distinct blend proportions: 0% (control), 4%, 6%, 10%, and 12% of lateritic soil by dry weight of the crushed stone. The selection of these specific percentages was strategically designed to capture the full transition from stone-dominated to fines-influenced behavior, enabling precise identification of the optimal proportion at which performance metrics peak. The 0% blend served as the control specimen to benchmark improvement, while the progression to 12% allowed observation of potential performance degradation beyond the optimum. For each blend proportion, the materials were meticulously mixed in a dry state using mechanical mixers to ensure consistent distribution and prevent segregation. The calculated amount of distilled water was added incrementally to achieve the target compaction moisture content, with mixing continuing until visual homogeneity was achieved throughout the sample. This systematic approach ensured reproducibility and reliability of test results across all blend variations.

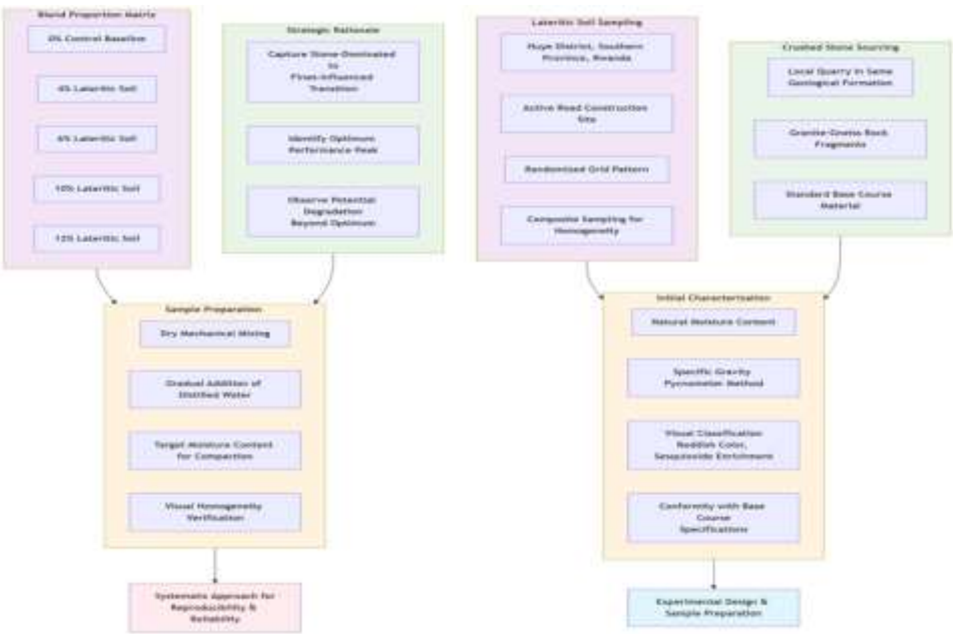

Figure 2. Schematic representation of materials collection, characterization, and experimental design methodology.

Figure 2 above shows the systematic approach, beginning with a representative sampling of lateritic soil from Huye District and crushed stone from a local quarry, followed by comprehensive material characterization. The experimental design incorporates five strategically selected blend proportions (0%, 4%, 6%, 10%, 12%) to capture performance transitions, with detailed sample preparation protocols ensuring reproducibility and reliable identification of optimal mixture proportions.

3.3. Laboratory Testing Protocol

A comprehensive suite of geotechnical tests was conducted at the RP-Huye Soil and Material Testing Laboratory in Rwanda, in accordance with the international standards detailed in Table 1. The Modified Proctor Compaction test (AASHTO T99) was employed to establish the precise moisture-density relationship for each blend, determining the critical parameters of Maximum Dry Density (MDD) and Optimum Moisture Content (OMC). The California Bearing Ratio (CBR) test was performed on specimens compacted to their maximum dry density at their respective optimum moisture content, with results meticulously recorded for three different compaction energies (10, 25, and 55 blows) to assess the material's sensitivity to varying compaction efforts encountered in field applications. The Direct Shear Test was conducted under three different normal stresses (50, 100, and 150 kPa) to accurately determine the fundamental shear strength parameters, cohesion (c) and angle of internal friction (φ), for each blend proportion, providing crucial insight into the mechanical behavior transformation resulting from the blending process. Complementary tests, including Particle Size Distribution, Atterberg Limits, Sand Equivalent (Tertsea et al., 2024), Los Angeles Abrasion, and Aggregate Crushing Value, provided a comprehensive characterization of both individual materials and their blends, enabling a holistic assessment of the stabilization effectiveness.

Table 1. Laboratory Tests and Corresponding Standards.

Test Name	Standard Code
Particle Size Distribution	BS 1377-2:2022
Atterberg Limits	ASTM D4318
Modified Proctor Compaction	AASHTO T99
California Bearing Ratio (CBR)	AASHTO T193
Los Angeles Abrasion	ASTM C131
Aggregate Crushing Value (ACV)	BS 812-110
Direct Shear Test	ASTM D3080

4.0. Results and Discussion

4.1. Particle size distribution and sand equivalent

The particle size distribution of the base course material was characterized through sieve analysis conducted in strict accordance with British Standard BS 1377:1975 (Methods of Test for Soils for Civil Engineering Purposes). The resulting gradation curves, presented in Figure 3, were evaluated against the project's specified gradation envelope. The analysis confirmed that the material's gradation fully complies with the contractual requirements. As

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illustrated, the particle size distribution curves for all tested variants, (a) the unmodified stone base, (b) the stone base mixed with 6% lateritic soil, and (c) the stone base mixed with 12% lateritic soil, consistently fall within the acceptable limits prescribed by the standard. This indicates effective control over the blend of course and fine fractions.

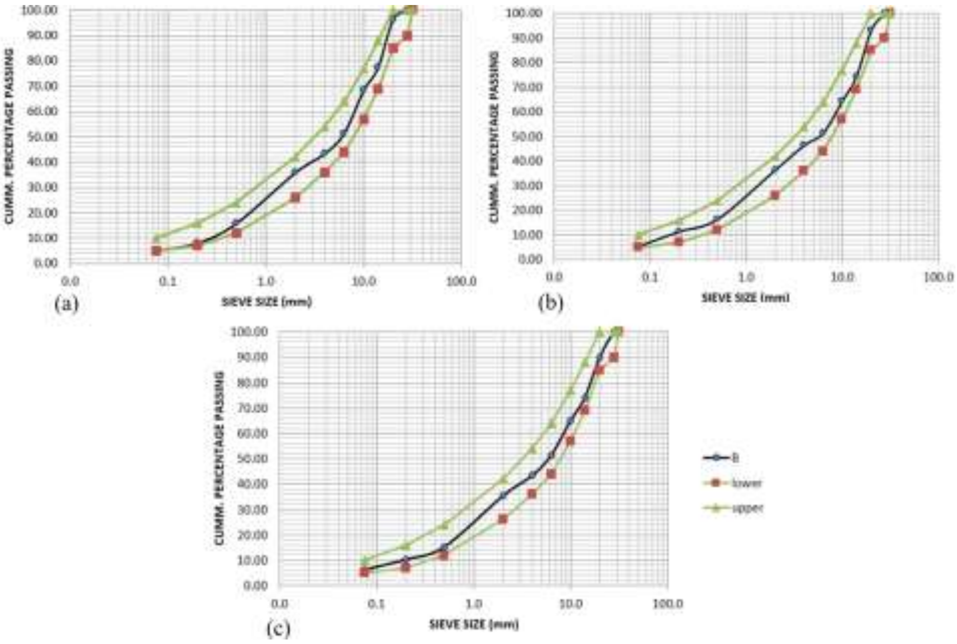

Figure 3. Particle size distribution curves of crushed stone base material with and without lateritic soil stabilization, determined through sieve analysis in accordance with BS 1377.

Sieve analysis results conducted according to BS 1377:1975 show gradation curves for (a) unmodified stone base material, (b) stone base mixed with 6% lateritic soil, and (c) stone base mixed with 12% lateritic soil. All tested variants fall within the specified gradation envelope, confirming compliance with contractual requirements and effective control of course- and fine-fraction blending.

Quantitative assessment of the gradation was performed by calculating two key engineering parameters: the uniformity coefficient (C_u) and the curvature coefficient (C_c), Equations (1) and (2). The calculated values for these coefficients align with the benchmark ranges for well-graded granular materials as recommended in BS 1377:1975. A high C_u value signifies a broad range of particle sizes, while a C_c A value between 1 and 3 confirms a balanced, non-gap-graded distribution. This combination of parameters objectively verifies that the base course material is well-graded. The sieve analysis results substantiate that the granular material, both in its native state and when blended with lateritic soil, possesses an optimal gradation. This ensures adequate shear strength, permeability, and compaction characteristics, thereby confirming its suitability for use in the structural layers of road construction.

$$C_u = \frac{D_{60}}{D_{10}} \qquad (1)$$

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} \qquad (2)$$

where C_u is the uniformity coefficient, $C_c$is the curvature coefficient, and D_{10}, D_{30} and D_{60} are Particle diameters at which 10%, 30%, and 60% of the material is finer, respectively.

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4.2. Atterberg Limits

The Atterberg limits of the constituent lateritic soil (Omoniyi et al., 2014; Faluyi & Adetoro, 2018; Udoh, 2025) and the composite base course material were determined using the standard thread-rolling and Casagrande cup methods. The results, detailed in Tables 2 and 3, characterize the materials' plasticity and workability. Table 2 presents the test results for the pure lateritic soil material. The liquid limit (LL) was derived from a flow curve using tests at 15, 23, and 42 blows, corresponding to water contents of 45.1%, 43.1%, and 36.4%, respectively. The moisture content interpolated for the standard 25 blows was 42.9%, which is reported as the Liquid Limit. The Plastic Limit (PL) was determined as the average water content (30.1%) from three trials (30.4%, 30.8%, 29.2%) in which the soil thread crumbled at a 3 mm diameter. Consequently, the Plasticity Index (PI = LL - PL) is 12.8, classifying the soil as having low to intermediate plasticity. Table 4.7 presents the results for the composite blend of crushed stone base and 12% lateritic soil. The liquid limit was established from tests at 16, 21, 26, 31, and 34 blows, yielding water contents of 26.9%, 22.5%, 24.9%, 23.8%, and 23.5%, respectively. The average of these values, 24.31%, is reported as the Liquid Limit. Critically, the material exhibited non-plastic behavior; repeated attempts to roll the moistened mixture into a 3 mm thread resulted in crumbling, rendering a measurable Plastic Limit unattainable. Therefore, the Plastic Limit is recorded as Non-Plastic (NP). In accordance with geotechnical convention, the Plasticity Index (Sridharan & Nagaraj, 2000; Nagaraj et al., 2012; Yao et al., 2018; Koffi Judicaël AGBELELE et al., 2022) is considered equal to the Liquid Limit (PI = 24.3) when PL = 0.

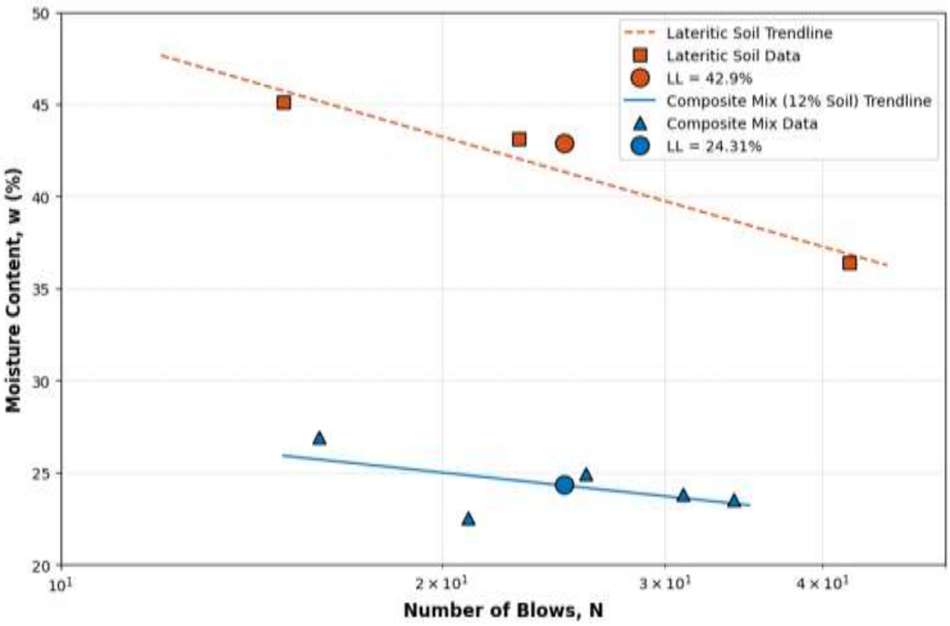


Figure 4. Comparison of Atterberg limits for the pure lateritic soil and the composite base material (crushed stone with 12% lateritic soil).

The key finding is the transition of the composite material from a plastic state to a non-plastic state. This indicates that the 12% addition of lateritic soil provides fines for particle bonding without imparting detrimental, water-sensitive plasticity to the granular matrix. This characteristic is highly favorable for a base course, as it suggests the material will maintain stability and reduced susceptibility to strength loss under variable moisture conditions. To visualize the significant change in material behavior induced by stabilization, a direct comparison of the key Atterberg parameters is presented in Figure 4. The figure quantitatively compares the liquid limit (LL), plastic limit (PL), and plasticity index (PI) of the pure lateritic soil stabilizer with those of the composite base course mixture containing 12% lateritic soil.

The composite mixture shows a 43% reduction in liquid limit. It exhibits non-plastic behavior (PL = 0%), confirming the transition from a cohesive, plastic soil to a favorable granular material suitable for road base construction.

This transition to a non-plastic state for the composite mixture is a critical finding (Karinski et al., 2017; Petrella et al., 2025). It demonstrates that the introduction of the lateritic soil at 12% does not impart detrimental plastic fines to the granular matrix. Instead, the mixture retains favorable, non-cohesive engineering properties, confirming its suitability for improving bonding while maintaining the essential drainage and strength characteristics required for a road base course. The experimental data presented in Table 2 reveal that the pure lateritic soil material exhibits plastic characteristics, with a Liquid Limit (LL) of 42.9%, a Plastic Limit (PL) of 30.1%, and a Plasticity Index (PI) of 12.8%, classifying it as a low-plasticity material. In stark contrast, Table 3 documents a fundamental transformation in the engineering behavior of the 12% stabilized blend. The composite material showed a significantly lower Liquid Limit of 24.3% and, most importantly, was classified as Non-Plastic (NP) as it consistently failed the thread-rolling test, preventing the determination of a measurable Plastic Limit. This demonstrates that the introduction of the lateritic soil at the optimum proportion does not impart detrimental plastic fines to the granular matrix. Instead, the mixture retains favorable, non-cohesive engineering properties, confirming its suitability for improving particle bonding and internal friction while maintaining the essential drainage characteristics and shear strength required for a high-performance road base course. The elimination of plasticity in the final composite is a key indicator of successful mechanical stabilization, ensuring the material will not exhibit undesirable volume changes or strength loss with variations in moisture content.

Table 2. Atterberg limits of the lateritic soil material, showing liquid limit (LL), plastic limit (PL), and plasticity index (PI) values obtained in accordance with ASTM D4318.

Parameter / Trial	**1**	**2**	**3**	**Average/Result**
Liquid Limit Test				
Number of Blows (N)	15	23	42	–
Moisture Content, w (%)	45.1	43.1	36.4	–
Estimated w at N=25 (%)	42	43	39	–

Liquid Limit, LL (%)	–	–	–	42.9
Plastic Limit Test				
Moisture Content, w (%)	30.4	30.8	29.2	30.1
Plastic Limit, PL (%)	–	–	–	30.1
Plasticity Index, PI (%)	–	–	–	12.8

Note: LL determined from the flow curve; value of 42.9% is the moisture content at 25 blows.

The Plasticity Index (PI) was calculated as the numerical difference between the Liquid Limit (LL) and Plastic Limit (PL) as per the standard definition:

$$PI = LL - PL \qquad (3)$$

Table 3. Atterberg limits of the composite base material (crushed stone + 12% lateritic soil).

Parameter / Trial	**1**	**2**	**3**	**4**	**5**	**Result**
Liquid Limit Test						
Number of Blows (N)	16	21	26	31	34	–
Moisture Content, w (%)	26.9	22.5	24.9	23.8	23.5	–
Liquid Limit, LL (%)	–	–	–	–	–	24.31
Plastic Limit Test						
Status	*	*	*	*	*	Non-Plastic (NP)
Plastic Limit, PL (%)	–	–	–	–	–	0 (NP)
Plasticity Index, PI (%)	–	–	–	–	–	24.3

A measurable plastic limit could not be determined; the material consistently failed the thread-rolling test. The stark contrast in plasticity characteristics between the pure lateritic soil and the 12% composite blend, as quantified in Tables 2 and 3, validates the core hypothesis of this research. The transformation from a plastic soil (PI = 12.8%)

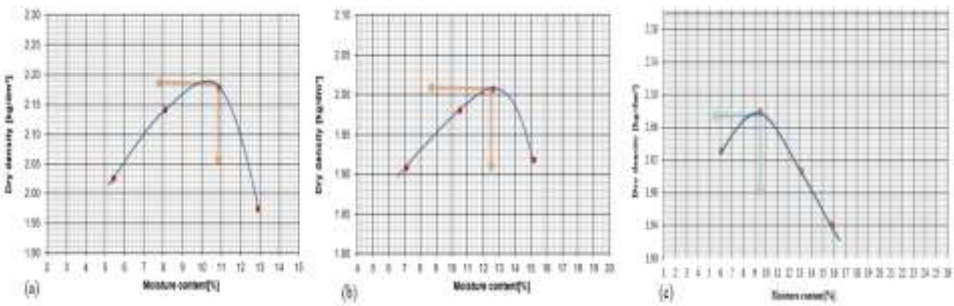
to a non-plastic composite material confirms that the granular skeleton of the crushed stone effectively dominates the mixture's behavior, diluting the cohesive influence of the lateritic fines. This non-plastic state is highly desirable for base course applications, as it ensures dimensional stability under fluctuating moisture conditions and promotes free drainage, critical properties that prevent softening, rutting, and other moisture-related pavement failures. The successful elimination of plasticity while incorporating the lateritic soil demonstrates that mechanical stabilization through optimal blending can enhance material properties without introducing the detrimental characteristics typically associated with clayey fines.

4.3. Modified Proctor Compaction

The Compaction characteristics, defined by the relationship between moisture content and dry density, were determined for all material variants using the Modified Proctor method (AASHTO T 180). Tests were conducted in a standard Proctor mould with a volume of 2412 cm³ under dynamic compaction energy. The resulting compaction curves establish two key engineering parameters for each mix design: the maximum dry density (γ_d) and its corresponding optimum moisture content (w).

$$\gamma_d = \frac{\gamma}{1+w} \qquad (4)$$

The test results are summarized in Figure 5, which presents the dry density versus moisture content curves for: (a) the pure lateritic soil (murram) stabilizer; (b) the unmodified crushed stone base (0% stabilizer); and the stone base blended with (c) 4%, (d) 6%, (e) 10%, and (f) 12% lateritic soil by mass. The pure lateritic soil (Figure 5a) exhibited a maximum dry density of 2.17 g/cm³ at an optimum moisture content of 10.9%. The unmodified crushed stone (Figure 5b) achieved a lower γ_d of 2.007 g/cm³ at a higher w of 12.6%, indicative of its granular, free-draining nature. The introduction of lateritic soil significantly altered the compaction behavior. At 4% addition (Figure 5c), the maximum dry density increased to 2.078 g/cm³ while the optimum moisture content decreased sharply to 9.5%. This trend continued at 6% stabilizer (Figure 5d), yielding a γ_d of 2.16 g/cm³ at a w of 7.4%. The peak dry density was observed at the 10% blend (Figure 5e), with a value of 2.181 g/cm³ at 9.1% moisture content. However, at the highest stabilizer content of 12% (Figure 5f), the maximum dry density decreased slightly to 2.155 g/cm³ at a w of 8.1%, suggesting an optimal stabilizer content may have been exceeded. The addition of lateritic soil up to 10% consistently increased the maximum achievable dry density of the crushed stone base while simultaneously reducing its optimum moisture content. This demonstrates that the stabilizer effectively fills voids within the granular matrix, enhancing particle packing and requiring less water to achieve maximum density, a critical improvement for field compaction control and strength gain in road construction.



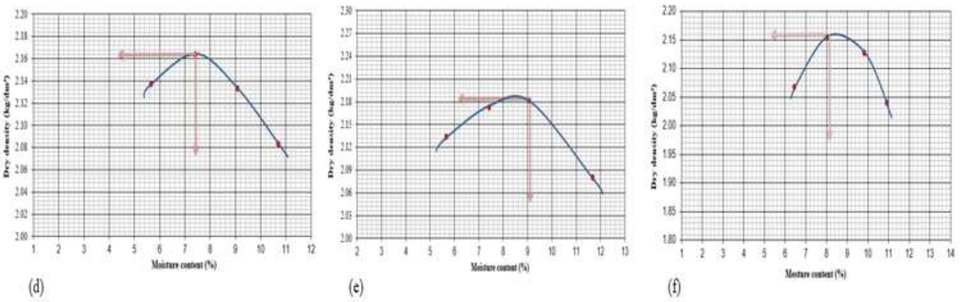


Figure 5. Modified Proctor compaction curves for the crushed stone base and its blends with lateritic soil.

The curves depict the relationship between dry density and moisture content as shown in Figure 5 above for: (a) pure lateritic soil (murram); (b) unmodified crushed stone (0% stabilizer); and crushed stone blended with (c) 4%, (d) 6%, (e) 10%, and (f) 12% lateritic soil. The peak of each curve defines the maximum dry density. γ_d and optimum moisture content (w). The results demonstrate that adding up to 10% lateritic soil increases. γ_d and reduces w, with the 12% blend showing a slight reduction in density. Testing was performed per AASHTO T 180.

The progressive changes in compaction parameters revealed by the curves in Figure 5 are quantitatively summarized in Table 4, which presents the maximum dry density (γ_d) and optimum moisture content (w) for each blend proportion. The tabulated data clearly show the engineering trade-off: as lateritic soil content increases from 0% to 10%, γ_d rises from 2.007 g/cm³ to a peak of 2.181 g/cm³, while w decreases from 12.6% to 9.1%, with the most significant moisture reduction occurring at the 6% blend (w = 7.4%). The subsequent slight decrease in γ_d to 2.155 g/cm³ at 12% stabilizer content, as recorded in Table 4, provides a critical quantitative benchmark indicating the threshold beyond which additional fines may begin to disrupt the optimal particle packing achieved at 10%. This numerical summary confirms that the addition of lateritic soil fundamentally modifies the compaction characteristics of the granular matrix, enhancing densification efficiency and reducing the water demand for optimal field compaction, key practical advantages for construction under Rwandan climatic conditions.

Table 4. Summary of Modified Proctor compaction test results for crushed stone lateritic soil blends, showing the variation of maximum dry density (MDD) and optimum moisture content (OMC) with increasing lateritic soil content (0–12%).

Lateritic Soil Content (%)	Maximum Dry Density (g/cm³)	Optimum Moisture Content (%)
0	2.007	12.6
4	2.078	9.5
6	2.161	7.4
10	2.181	9.1
12	2.155	8.1

The compaction characteristics summarized in Table 4 reveal a clear material-optimization trend. The increase in maximum dry density up to the 10% blend indicates improved particle packing and densification, where the lateritic soil fines effectively fill the voids between the crushed stone particles. The corresponding reduction in

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optimum moisture content, particularly notable at the 6% blend, suggests enhanced workability and lower water sensitivity, an advantageous property for construction in variable field conditions. The slight decline in dry density at 12% stabilizer content signals a transition from an optimally packed granular structure to one in which excess fines may begin to separate, potentially reducing stone-to-stone contact and shear strength. This quantitative data provides essential guidance for field implementation, establishing 10% lateritic soil as the target blend to achieve the highest compacted density and an optimal moisture-stability balance in base course construction, directly supporting the overarching goal of sustainable and efficient road development in Rwanda.

4.4. California Bearing Ratio (CBR)

The California Bearing Ratio (CBR) test was conducted to quantify this improvement. As detailed in Table 5, the results demonstrate that the addition of lateritic soil significantly enhances the engineering properties of the stone base, resulting in higher CBR values. This indicates a marked improvement in the material's bearing capacity, making it more suitable for supporting pavement layers. All tested mixing ratios successfully met the stringent performance criteria for base course construction. Specifically, the mixes achieved the required CBR threshold of 95% as specified by the Association Française de Normalization (AFNOR NF P 11-300). These standards mandate that materials used in base courses must exhibit a CBR value exceeding 80% when compacted to 100% of their maximum dry density. All stabilized blends in this study comfortably surpassed this requirement.

Table 5. Detailed California Bearing Ratio (CBR) test results for lateritic soil and crushed stone stabilized with varying lateritic soil contents (0–12%), evaluated under different compaction energies (10, 25, and 55 blows).

Lateritic Soil Content (%)	Mould No.	No. of Blows	Mass of Mould + Wet Material (g)	Mass of Mould (g)	Mass of Wet Material (g)	Volume of Mould (cm³)
Control	1	55	11370	6270	5100	2176
	2	25	11270	6270	5000	2176
	3	10	11170	6270	4900	2176
0	1	55	11970	7089	4881	2176
...	...	...	...	...	...	...

Lateritic Soil Content (%)	Mould No.	Wet Density (g/cm³)	Moisture Content (%)	Dry Density (g/cm³)	Reference Proctor Density (g/cm³)	Percentage Compaction (%)	CBR Index
Control	1	2.34	7.0	2.190	2.170	99.8	56
	2	2.30	8.1	2.126	2.170	97.8	44
	3	2.25	8.9	2.068	2.170	95.3	38
0	1	2.24	12.0	2.003	2.007	99.8	99
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The comprehensive results are synthesized in Figure 6, which details the CBR performance across the spectrum of mix designs. Panel (a) in Figure 6 establishes the baseline CBR of the lateritic soil stabilizer alone, with values ranging from 38 to 56 as shown in the first three rows of Table 5. Panel (b) presents the CBR of the unmodified crushed stone material prior to any improvement, corresponding to the "0%" stabilizer content data, where values range from 24 to 99. The subsequent panels illustrate the progressive effect of stabilization: panel (c) for a 4% lateritic soil addition (CBR: 47–105), panel (d) for 6% (CBR: 50–122), panel (e) for 10% (CBR: 50–130), and panel (f) for a 12% mix proportion (CBR: 58–100). Analysis of these results reveals a clear trend: the incorporation of lateritic soil substantially improves the CBR of the crushed stone base. The peak strength was attained at a 10% stabilizer content, suggesting this is the optimal mix proportion under the tested conditions for maximizing inter-particle bonding and structural integrity.

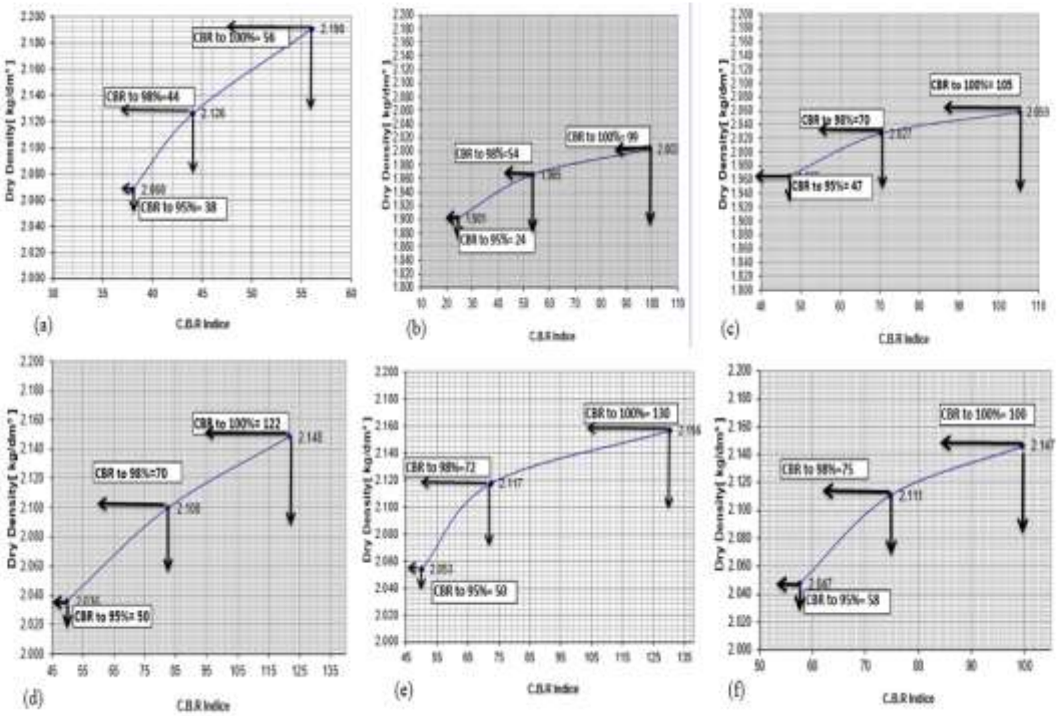

Figure 6. California Bearing Ratio (CBR) test results for crushed stone material stabilized with lateritic soil at various mix proportions.

(a) CBR of lateritic soil material alone; (b) CBR of unimproved crushed stone material; (c) CBR of crushed stone improved with 4% lateritic soil; (d) CBR of crushed stone improved with 6% lateritic soil; (e) CBR of crushed stone improved with 10% lateritic soil; and (f) CBR of crushed stone improved with 12% lateritic soil.

The quantitative CBR performance for each stabilizer content across the three standard compaction energies (10, 25, and 55 blows) is consolidated in Table 6. This tabular summary confirms the trend visualized in Figure 6, providing precise numerical evidence of the strength enhancement. The data shows that the 10% blend yields the highest CBR value of 130 under heavy compaction, followed closely by the 6% blend at 122. Notably, Table 6 also reveals the sensitivity of each blend to compaction effort; for instance, while the 0% (unimproved) material shows a drastic drop from 99 to 24 CBR with reduced compaction, the stabilized blends particularly at 10% and 12% maintain significantly higher minimum strengths (50 and 58, respectively) even at lower compaction energy. This

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underscores that the optimized blends not only achieve higher peak strength but also offer more consistent performance under variable field compaction conditions, a critical advantage for practical road construction.

Table 6. Summary of California Bearing Ratio (CBR) test results for crushed stone stabilized with varying lateritic soil contents (0–12%), measured at compaction energies of 10, 25, and 55 blows.

Lateritic Soil Content (%)	CBR @ 55 Blows	CBR @ 25 Blows	CBR @ 10 Blows
0	99	54	24
4	105	70	47
6	122	83	50
10	130	72	50
12	100	75	58

The strength trends detailed in Table 6 highlight a clear peak in CBR at the 10% lateritic soil content under heavy compaction. To further understand the multi-variable relationship governing this optimal performance, the interactions between the CBR index, the maximum dry density (MDD) from compaction, and the derived cohesion of each blend are visualized in Figure 7. This three-dimensional bubble plot represents CBR values as bubble sizes, directly correlating the data from Table 6 with the density results from Modified Proctor tests and shear strength parameters. The visualization in Figure 7 clearly illustrates how the 10% blend achieves an optimal balance positioned at the peak of both the density and cohesion axes resulting in the largest bubble size, which corresponds to the highest CBR of 130. This integrated graphical analysis confirms that the superior bearing capacity is not solely a function of density or cohesion alone, but rather the synergistic optimization of both properties through precise mechanical blending.

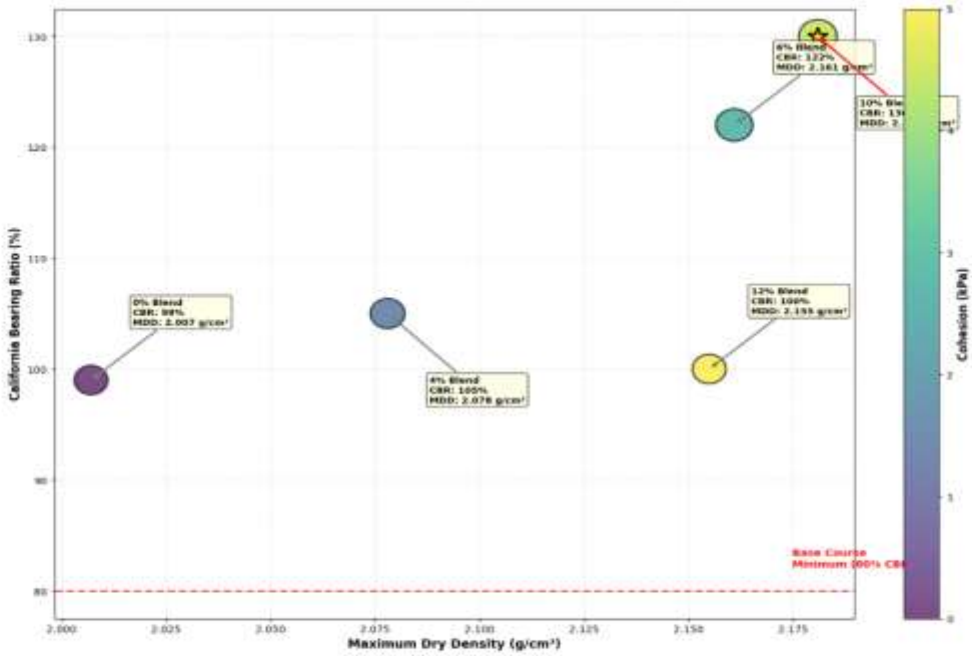

Figure 7. Integrated relationship between California Bearing Ratio (CBR), maximum dry density (MDD), and cohesion (c) for crushed stone stabilized with varying lateritic soil contents (0–12%).

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4.5. Direct Shear Test

Direct shear tests were conducted to evaluate the fundamental shear strength parameters, internal friction angle (φ) and cohesion (c), of the crushed stone base material before and after stabilization with lateritic soil. The complete results are presented in Figure 8, which shows the shear stress–normal stress envelopes for all blend proportions and provides insight into the mechanical behaviour and strength evolution of the composite blends. Figure 8a presents the shear strength envelope for the unimproved crushed stone base material, which exhibited a high internal friction angle of 52.1° and negligible cohesion (c = 0 kPa), characteristic of clean granular materials where shear resistance is governed by particle interlocking and frictional contact. Figure 8b illustrates the results for the blend containing 4% lateritic soil. The addition of fines reduced the internal friction angle to 47.2°, while cohesion remained effectively zero (c = 0 kPa), indicating the initial influence of fine particles within the granular matrix. Figure 8c shows the direct shear test outcomes for the 6% lateritic soil mixture. The friction angle further decreased to 43.3°, with cohesion remaining at 0 kPa, suggesting that fine particles increasingly occupied voids and altered inter-particle contacts without developing measurable cohesive bonds. A significant shift in failure mechanism is observed in Figure 8d for the 10% lateritic soil blend. The internal friction angle decreased to 37.2°, while a measurable cohesion of 2.5 kPa was recorded for the first time, indicating that the optimum blend proportion promoted particle bonding and introduced true cohesive strength. Finally, Figure 8e details the shear parameters for the 12% lateritic soil mixture. The friction angle decreased to 34.0°, whereas cohesion increased to 5.0 kPa, representing the highest cohesive strength measured in this study. Overall, Figure 8 demonstrates that increasing lateritic soil content systematically reduces the internal friction angle while enhancing cohesion, transforming the shear strength mechanism from purely frictional to a combined frictional–cohesive behaviour.

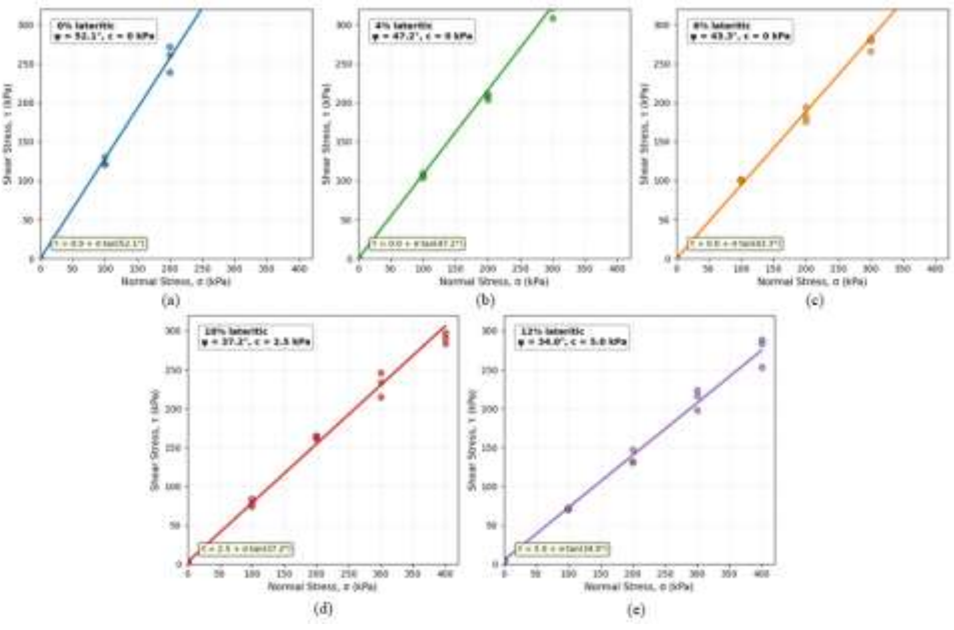

Figure 8. Direct shear test results for crushed stone base material stabilized with varying proportions of lateritic soil.

The figure presents shear stress versus normal stress envelopes for: (a) unimproved crushed stone (0% lateritic soil), φ = 52.1°, c = 0 kPa; (b) stone base with 4% lateritic soil, φ = 47.2°, c = 0 kPa; (c) stone base with 6% lateritic soil, $\varphi = 43.3°$, $c = 0$ kPa; (d) stone base with 10% lateritic soil, $\varphi = 37.2°$, $c = 2.5$ kPa; and (e) stone base with 12% lateritic soil, $\varphi = 34.0°$, $c = 5.0$ kPa. The results illustrate the systematic transition from purely frictional behaviour to cohesive-frictional composite response as lateritic soil content increases.

$$\tau = c + \sigma_n tan\emptyset \qquad (5)$$

Where τ is the Shear strength, c is cohesion, σ_n is normal stress, and $\emptyset$ is the angle of internal friction. Table 7 provides a synthesized summary and discussion of all key laboratory tests conducted on the crushed stone material before and after improvement with lateritic soil.

Table 7. Overall results and discussion of laboratory tests conducted.

Test	Test results of crushed stones before improvement	Test results of crushed stones after improvement with lateritic soil
Sieve Analysis	The material was excessively coarse, leading to reduced stability, increased shear risk, and difficult compaction.	The 12% lateritic soil blend produced a well-graded distribution curve closely matching the ideal Fuller curve specified for base course materials.
Los Angeles Abrasion	LA ratio = 29%	Not applicable (test performed on base aggregate only).
Sand Equivalent	74%	Not applicable (test performed on base aggregate only).
Aggregate Crushing Value	ACV = 23.9%	Not applicable (test performed on base aggregate only).
Atterberg Limits	Liquid Limit = 22%, material classified as Non-Plastic (NP).	At the 12% blend, Liquid Limit = 24.2%, material remained Non-Plastic (NP).
Modified Proctor	OMC = 12.6%, γ_{dmax} = 2.007 g/cm³.	Peak density achieved at 10% blend: γ_{dmax} = 2.181 g/cm³ at OMC = 9.1%.
California Bearing Ratio	CBR index = 99% (at 55 blows).	Maximum CBR index = 130% achieved with the 10% blend (at 55 blows).
Direct Shear Test	Internal friction angle (φ) = 52.1°, Cohesion (c) = 0 kPa.	The internal friction angle decreased while cohesion increased for all blends. Maximum cohesion (c = 5.0 kPa) was recorded at the 12% lateritic soil content.

4.6. Comprehensive 3D Visualization of Geotechnical Properties

To synthesize the complex relationships between the key geotechnical properties and identify the global performance optimum, an integrated multi-parameter analysis was conducted. The results of this analysis are visualized in Figure 9, which plots the four critical engineering parameters Maximum Dry Density (MDD), California Bearing Ratio (CBR), cohesion (c), and internal friction angle (φ) against the lateritic soil content. This

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comprehensive visualization reveals the non-linear and interdependent effects of stabilization. The plot clearly shows that MDD and CBR follow a similar trend, both peaking at the 10% lateritic soil blend (MDD = 2.181 g/cm³, CBR = 130%). In contrast, the shear strength parameters exhibit an inverse relationship: cohesion increases monotonically from 0 kPa (0% soil) to 5.0 kPa (12% soil), while the internal friction angle decreases correspondingly from 52.1° to 34.0°. The convergence of peak density and bearing capacity at the 10% blend, coupled with a balanced transition in shear behaviour (φ = 37.2°, c = 2.5 kPa), provides robust, multi-criteria validation that this proportion represents the global performance optimum for the composite material. This integrated approach moves beyond single-parameter optimization, offering a holistic understanding of the stabilization mechanism. It demonstrates that the optimal blend is not defined by maximizing any one property in isolation, but by achieving the most favorable synergy between density, strength, and shear characteristics for flexible pavement base course applications under Rwandan conditions.

The plot illustrates the variation of four key geotechnical parameters with increasing stabilizer content: Maximum Dry Density (MDD, red), California Bearing Ratio (CBR, teal), cohesion (c, blue), and internal friction angle (φ, green). The coordinated peak of MDD and CBR at 10% lateritic soil, alongside a balanced transition in shear parameters, identifies this blend as the global performance optimum for base course application.

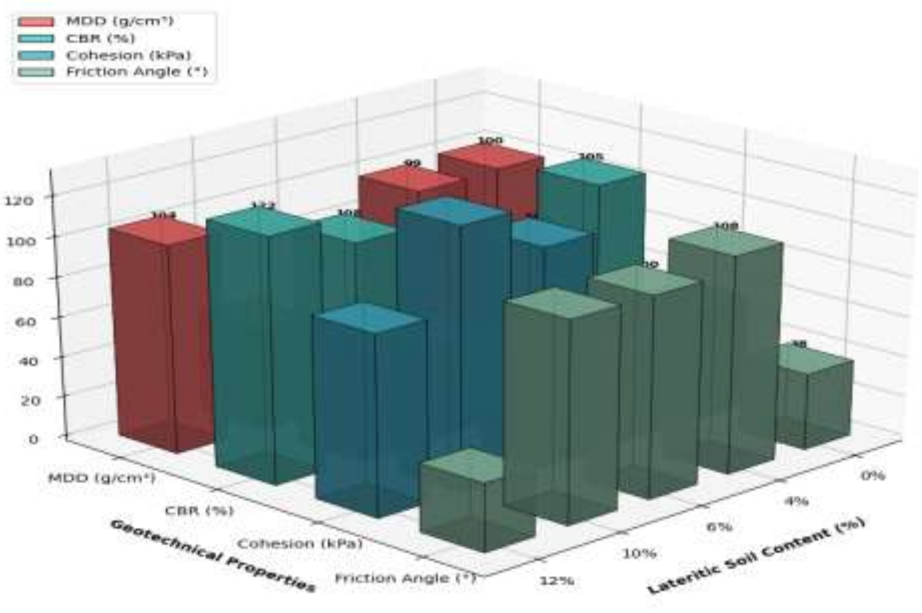

Figure 9. Integrated performance optimization of crushed stone stabilized with lateritic soil.

The experimental investigation evaluated the performance of crushed stone mechanically stabilized with varying proportions of locally available lateritic soil, demonstrating that optimized blending significantly enhances engineering properties for sustainable road base construction in Rwanda. The improvement is primarily attributed to modifications in particle size distribution (gradation). The untreated crushed stone was poorly graded and excessively coarse, resulting in large voids, reduced stability, and inefficient compaction. The addition of lateritic soil fines transformed the material into a well-graded composite, approaching the ideal packing condition. This optimized gradation enabled fine particles to fill voids between coarse aggregates, improving interparticle contact and structural integrity.

5.0. Conclusion and Future Recommendations

This study successfully investigated the mechanical stabilization of crushed stone base course material using locally abundant lateritic soil in Rwanda. Through a comprehensive laboratory testing program, the optimal blend proportion was quantitatively determined. The key findings demonstrate that the addition of lateritic soil significantly enhances the geotechnical properties of the crushed stone. The particle size distribution improved, evolving from poorly graded to well-graded with the addition of 12% soil. The compaction characteristics showed a clear optimum: the maximum dry density (MDD) peaked at 2.181 g/cm³, and the optimum moisture content (OMC) reduced to 9.1% for the blend containing 10% lateritic soil. This indicates superior particle packing and efficient void filling. Most critically, the California Bearing Ratio (CBR), a direct measure of strength and stiffness, reached its maximum value of 130% at the 10% blend, substantially exceeding the control specimen (99%) and typical specification requirements for base courses. Direct shear tests revealed the expected mechanical trade-off: cohesion increased while the internal friction angle decreased with higher soil content. The 10% blend represented the optimal balance of these shear strength parameters, coinciding with the peak CBR and MDD. Therefore, the study concludes that blending crushed stone with 10% lateritic soil by dry weight produces a composite material with optimized strength, density, and gradation. This blend provides a sustainable, cost-effective, and technically sound alternative for base course construction in Rwanda. It utilizes local materials, reduces reliance on scarce high-quality aggregates, lowers project costs, and promotes environmentally conscious construction practices aligned with Rwanda's Vision 2050. The findings offer practical guidance for engineers and policymakers seeking to improve the resilience and affordability of road infrastructure. The following Future recommendations would be useful in improving this research.

1. Field Validation and Piloting Projects on other researchers should concentrate on implementing this 10% lateritic soil crushed stone blend on a large scale under actual traffic.

2. Durability and Environmental Performance Assessment for Additional research on the improved blend's long-term durability under various climatic circumstances, including wet and dry cycles and erosion resistance to seasonal impact assessment.

3. Stabilization with Additives to increase strength, durability, and moisture resistance of the base course material, lateritic soil may be used in a bonded form with chemical stabilizers on industrial products like fly ash.

4. Material Variability Since lateritic soils varied between locations, future research should evaluate the performance of comparable mixes using lateritic soils in larger regions for applications based on design standards.

5. Advanced Mechanical and Microstructural Analysis comprehend the mechanical interaction between crushed stone and lateritic soil, with appropriate research using refined testing techniques, including triaxial tests and microstructural analysis, must be adopted.

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Competing Interests Statement

The authors have declared that no competing financial, professional or personal interests exist.

Consent for publication

All authors contributed to the manuscript and consented to the publication of this research work.

Author's contributions

All authors contributed equally to the Literature Review, Data Analysis, and Manuscript Writing.

Availability of data and materials

The data that support the findings of this study are available from the corresponding author upon reasonable request.

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