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Preventing Rapid Deterioration of Road Pavements: Optimal Use of Lateritic Gravel Soils from Western Senegal

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Abstract

This study covers 25 samples of lateritic materials, in the following granular classes (0/20, 0/25, 0/31.5, 0/40 and 0/50 mm). The samples are taken from the borrow pits of Lam-Lam, Mont Rolland, Pout, Ngoundiane and Sindia, all located in the Thiès region of western Senegal. The materials were tested to determine their classes and behaviors. The quality of these materials used in pavements is declining. Pavements constructed using lateritic materials often deteriorate prematurely. The cause is not well understood. Considering their composition and how their granularity influences their geotechnical characteristics would ensure their sustainable and optimal use. This publication aims to prevent the rapid deterioration of pavements. It recommends using lateritic materials with their optimal grain size. The methodology consists of three steps. First, samples with different grain sizes are identified using classification tests. Second, the samples are classified. Finally, their bearing capacity is qualified to determine their suitability for road pavement and to identify the class with the best geotechnical characteristics for resilient use. The results of the analyses reveal that the lateritic gravels studied are classified as G3 and I2 according to GTR 2023. On the one hand, these classes are equivalent to the GM (silty gravel and silty gravel with sand) and GW-GM (well-graded gravel with silt) classes of ASTM 2000. On the other hand, behavioral parameters change more markedly with granular classes than with borrow pits. This led to identifying the 0/31.5 mm granular class as the optimal class within the considered set. Therefore, to obtain lasting pavement layers with optimal bearing capacity, it is necessary to eliminate particles in lateritic gravel greater than 31.5 mm in size. As part of this study, the authors have put forth proposals to enhance the existing definition of lateritic gravel materials and their respective road classes.

Keywords

Optimal Use, Granular Classes, Lateritic Gravel, Pavement, Thiès

1. Introduction

The use of lateritic soils in road pavement is based primarily on the geotechnical classification and behavior parameters [1]. The determination of compaction characteristics and California Bearing Ratio (CBR) is based on standards applied to the 0/20 mm granular fraction during laboratory testing [1]-[3]. However, when road materials are used on site, the particle size may exceed 50 mm [1]. This implies the existence of ignored risk factors that are increasingly amplified by the scarcity of lateritic materials with good geomechanical properties [4]-[7]. These non-conformities can significantly contribute to the persistent phenomenon of the early deterioration of flexible and semi-rigid pavement structures, which have an increasingly shortened real service life [8]-[10].

Several solutions have been proposed, including litho-stabilization [5] [6], the use of reversible modulus or other compaction methods [11]-[13] and hydraulic binder upgrading or stabilization [5]. Nevertheless, the quality of the materials used for pavements is declining, causing pavements to deteriorate prematurely. The cause of this deterioration is not well understood. Considering their composition and how their granularity influences their geotechnical characteristics would ensure their long-lasting and optimal use.

At the same time, high-quality lateritic materials are wasted in the lower, undemanding layers of pavement [4]. The environment suffers from scattered extraction of borrow pit materials, which disrupts landform suitability and drainage efficiency, occasionally causing flooding.

Therefore, it is necessary to propose an optimal and resilient use of these materials that will be extracted in a rational manner to meet specific utilization needs. This study focuses on determining the nature and behavior parameters of five borrow pits (Lam-Lam: LL, Mont Rolland: MR, Pout: PT; Ngoundiane and Sindia: SD), successively located from north to south and from east to west in the Thiès region of Senegal. The samples are divided into five granular classes (0/20, 0/25, 0/31.5, 0/40, and 0/50 mm), named 1 to 5 respectively. These samples are then identified and characterized to determine the optimal class for sustainable use in pavement. These resources are currently in high demand for road and highway construction projects in Senegal.

Classifying materials into granular categories provides a classification system that is valid both in the laboratory and on the construction site. This involves adapting experimental conditions to the realities of construction sites and determining how each material class will be used. The aim of this approach is to contribute to the prevention of early deterioration of road pavements and to the rational use of lateritic materials. Thus, the use of lateritic gravel materials will be adapted to usage requirements, becoming more environmentally friendly while ensuring the construction of more sustainable pavements in countries where these materials are available [14].

2. Materials and Methods

2.1. Sampling Exploration

Samples of uncontaminated material were taken from stockpiles and from the bench faces at the locations shown in **Figure 1**. Ten rectangular bags, each measuring 55 × 95 cm^2 and closed at the base, were collected from each of the five sites.

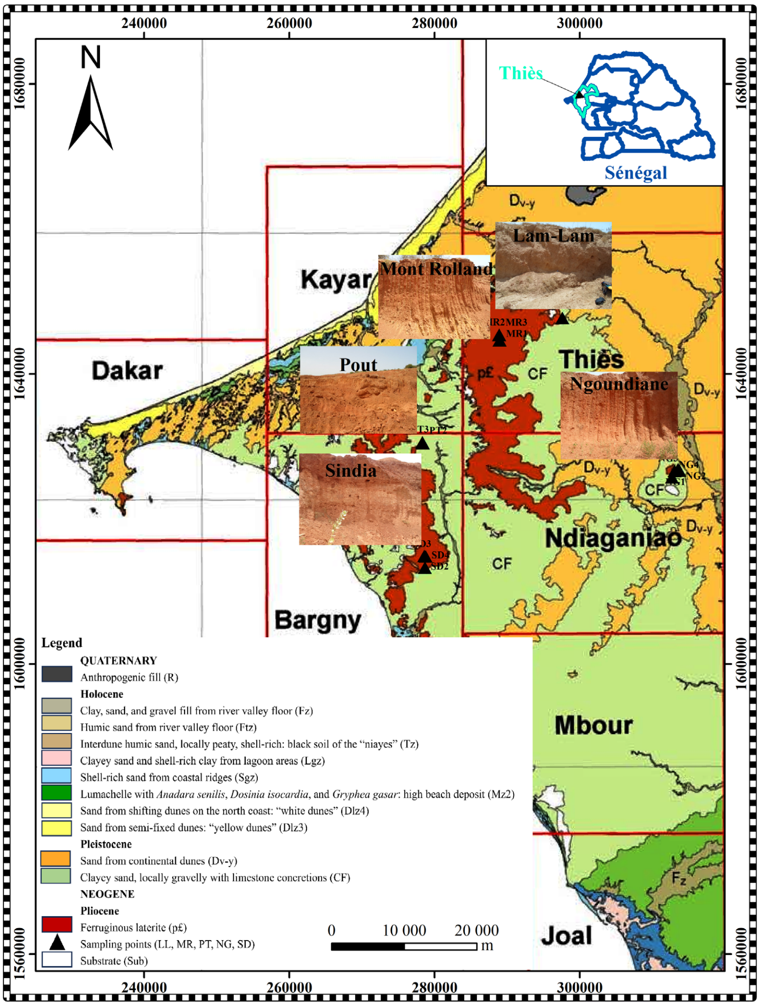

Figure 1. Locations of the Lam-Lam, Mont Rolland, Pout, Ngoundiane, and Sindia borrow pits (Roger *et al.* [15], modified).

The lateritic materials have a texture marked by gravelly nodules in a fine to sandy matrix, as presented in **Figure 2**, and as indicated by Autret [16] [17]. These soils are categorized as lateritic, comprising particles of variable shape and size [18] [19].

Figure 2. Uncontaminated material from Ngoundiane.

2.2. Distribution of Reserves

The borrow pits vary in thickness in terms of both the total and usable thickness of the profiles (**Table 1**). The most abundant lateritic material reserves (Horizon B) are found in descending order at Mont Rolland, Sindia, Ngoundiane and Pout, and are much less abundant at Lam-Lam, where the sand cover (Horizon A) is thickest. On average, they are around 500 cm thick (**Table 1**), with a fairly large mean absolute deviation of 85.33 that is primarily due to the data from Lam-Lam.

Table 1. Profile and horizon thicknesses.

Borrow pit	Profile thickness (cm)	A horizon (cm)	B Horizon (cm)
Lam-Lam	231.33	63.33	168.00
Mont Rolland	599.13	56.52	542.61
Pout	561.60	58.50	503.10
Ngoundiane	543.40	110.50	432.90
Sindia	501.15	58.50	442.65
Mean	487.32	69.47	417.85
Mean absolute deviation	85.33	13.68	83.28
Coefficient of variation	30%	33%	35%

However, according to Diop *et al.* [20], the altitude levels of the horizons are not uniform (**Figure 3**). The most heavily used borrow pits (SD and PT in **Figure 3**) have the most rugged terrain. To account for expansion, 1 m^3 of spread material corresponds to 0.5 m^3 of compacted material [21]. These particularities, along

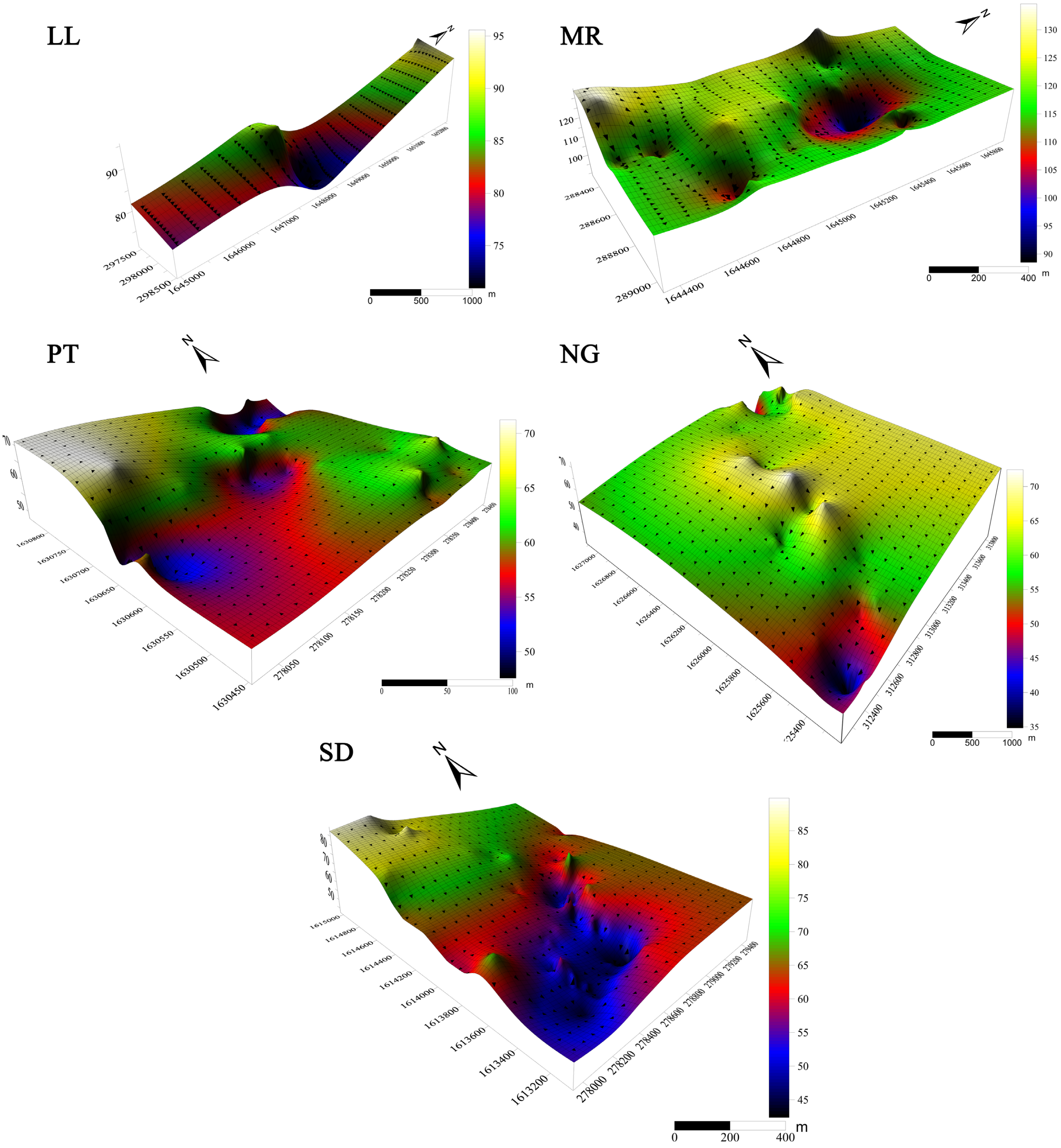

Figure 3. Digital terrain models showing the variability in elevation and operating levels of the Lam-Lam (LL), Mont Rolland (MR), Pout (PT), Ngoundiane (NG) and Sindia (SD) borrow pits.

with the potential for horizontal extension, need to be taken into account when estimating the remaining material reserves.

2.3. Laboratory Analysis

2.3.1. Sampling in the Laboratory

The materials from the five borrow pits were collected during periods of rain and drizzle. The samples were transported to the laboratory in covered bags to prevent the leaching of fine particles and the dissolution of colloidal oxides and hydroxides.

After mixing, drying, and quartering in the laboratory, samples were taken for specific identification and characterization analyses.

Upon arrival, the wet materials were cured in the open air with normal ventilation to promote drying and the separation of non-pisolitic granular particles. After curing, the prototypes obtained by sieving were successively mixed, quartered, and sampled. Each prototype consisted of one of the five granular classes (0/20, 0/25, 0/31.5, 0/40, and 0/50 mm) from a specific borrow pit.

Although, according to current standards [22], lateritic are considered not to have reached the threshold of 25% granular particles with a diameter greater than 20 mm for behavioral tests. Nevertheless, all samples were sufficiently rich in coarse particles to exceed this threshold.

2.3.2. Soil Classification

Soil classification parameters depend on granularity and clay content.

1) Gradation Analysis

The particle size distribution, or gradation analysis, as described in Standard NF EN 933-1 [23], allows drawing a distribution curve of granular particles, which are divided into size-defined fractions, with relative proportions of dry mass that vary according to the granular fractions and the proportions of sieve passing [24].

The granulometric characteristics of granular materials are represented by two coefficients: one for uniformity and one for curvature, according to standard NF EN ISO 14688-2 [3].

2) Atterberg Limits

The Atterberg limits test determines the clay content of soils and is the second classification parameter, complementing granularity. For the plasticity index (PI) to be easily interpreted, the weight proportion of the 0/400 μm fraction must exceed 12%, and the PI must be greater than 7 [1] [25]. The test is performed according to the NF P 94-051 instructions [26]. PI is more effective than the MBV or MBM (Methylene Blue Value or Mass) for assessing the clay content of medium to very clayey soils [1] [25].

3) Determination of the Methylene Blue Value or Mass

MBV (or MBM) determination is a relatively new test that provides a complementary parameter to granularity and plasticity in soil classification. The MBV depends heavily on the abundance and specific surface area of fine particles. It assesses a sensitivity of soil to water through the amount of methylene blue that can be adsorbed by its sandy, silty, and clayey particles.

Unlike the previous standards, NF P 94-068 [27] and [1], which required the MBV to be expressed in terms of g/100 g, the test is carried out in accordance with standard NF EN 933-9 [28], which requires the results to be expressed in methylene blue mass (MBM) in g/kg. MBV or MBM takes precedence over PI when the latter is less than 12%.

2.3.3. Tests to Determine Laterite Behavior

In geotechnical engineering, laboratory studies are conducted to evaluate the suitability of materials for compaction and their bearing capacity, which are assessed using the modified Proctor test and the bearing capacity test, respectively. The results of these tests must meet the specifications for the intended use.

1) Determining Compaction Characteristics

The Proctor test involves dynamic compaction, and is used to evaluate the compactability of a material. Compactability depends on the ability of the material to densify by tightening and rearranging its mineral particles. Since the materials studied are gravel-rich and intended for road use, the preferred test is the Modified Proctor (MP) delineated in the standard NF EN 13286-2 [29].

2) Qualification of Soil Bearing Capacity

The bearing capacity of a material used in a pavement refers to its ability to support the traffic load. The soaked CBR (SCBR) method, described in the standard NF EN 13286-47 [30], is particularly well suited to pavement design in tropical countries. The materials are soaked for four days after compaction. There are various techniques for assessing bearing capacity in laboratories and on construction sites.

2.3.4. Permeability and Swelling Sensitivity of the Lateritic Materials

The 0/20 mm fraction of materials from the five sites is employed to assess the vulnerability of the materials to water and swelling in the laboratory.

1) Determination of Linear Swelling after Soaking

In accordance with standard NF EN 13286-47 [30], measuring the swelling of a compacted and soaked specimen, during the SCBR method provides information on the type and amount of clay in a material.

2) Determination of Water Absorption

The water absorption of the lateritic samples was measured after they were compacted to 95% of the Optimal Compaction Energy (OCE). Then the samples were soaked for four days.

3) Determination of Material Permeability

Soil permeability is the capacity of a soil to allow an incompressible fluid, such as water, to flow through it [31]. Permeability is measured by Darcy's hydraulic conductivity (k), which is expressed in terms of flow velocity. Since lateritic are assumed to have low permeability, the test is carried out according to standard NF P 94-512-11 [32]. The samples are first saturated for 24 hours to achieve maximum permeability at full saturation [33].

3. Results

Samples were studied by borrow pit: Lam-Lam (LL); Mont Rolland (MR); Pout (PT); Ngoundiane (NG); and Sindia (SD). They are also studied by grain size class: 0/20 mm (class 1); 0/25 mm (class 2); 0/31.5 mm (class 3); 0/40 mm (class 4); and 0/50 mm (class 5).

Descriptive statistics is used to briefly provide an overview of the results. The mean is a central reference parameter supplemented by the mean absolute deviation, the coefficient of variation, and the range, which provide a more complete picture of dispersion.

3.1. Geotechnical Identification of Untreated Lateritic Material Samples

Classifying lateritic materials helps prevent long-term changes in their composition.

3.1.1. Gradation Test

This test is carried out in two stages, before and after compacting road materials.

The results shown in the grain size curve represent the proportions of passing distributed among the granular fractions that make up lateritic materials. Depending on the nature and effect of compaction, there are varying proportions of gravel (63 to 2 mm), sand (2 to 0.063 mm), silt (0.063 to 0.002 mm) and clay (≤0.002 mm); these names correspond to the respective particle size fractions according to standard NF EN ISO 14688-2 [3].

1) Before Compaction

As shown in **Table 2**, the gravel proportions range from 59.52% (MR4) to 85.50% (LL4). The arithmetic mean is 78.20%. The dispersion of these proportions is characterized by a mean absolute deviation of 3.86%.

The sand content of the samples ranged from 5.36% (NG5) to 22.60% (MR4) with arithmetic mean of 9.26%. The dispersion of values around the mean is shown by a mean absolute deviation of 2.65%. Sand accounts for less than one-third of the analyzed samples.

The fines content of the samples ranges from 7.26% (LL4) to 18.82% (SD4), with a mean of 12.54%. According to the data in **Table 2**, dispersions are characterized by a mean absolute deviation of 2.95%, a coefficient of variation of 28%. Fines make up less than a fifth of the particles in the analyzed samples.

The granular fraction of fines consists of two sub-fractions: silts and clays. The proportion of silt ranges from 7.15% (LL4) to 18.62% (SD4), with a median of 11.59% (MR2) and an arithmetic mean of 12.54%. Dispersion is characterized by a mean absolute deviation of 3.0%. Silts account for less than one-fifth of the mass of these lateritic materials.

The clay content ranges from 0.11% (LL4) to 0.31% (MR4), with a mean of 0.18%. **Table 2** shows that the dispersion is reflected in a mean absolute deviation of 0.04%.

Discontinuous granulometry occurs when certain dimensions of the conventional sieve column are absent. Otherwise, the granulometry is continuous [3].

Uniformity coefficient (Cu) ranges from 60 (PT5) to 315 (NG3), while the curvature coefficient (Cc) ranges from 1.4 to 50.7.

The 0/2 mm fraction studied from a pedological point of view is composed of 55% - 90% sand (2 mm ≥ Ø < 50 μm), 10-45% silt (50 ≥ Ø < 2 μm), and less than 1% clay (Ø < 2 μm).

Table 2. Proportions of granular fractions in analyzed samples.

Sample name	Proportions (%)							
	63 ≥ Ø < 2 mm	2 mm ≥ Ø < 63 µm	Ø ≤ 63 µm	63 ≥ Ø < 2 µm	Ø ≤ 2 µm	2 mm ≥ Ø < 50 µm	50 ≥ Ø < 2 µm	Ø ≤ 2 µm
LL1	82.00	8.54	9.46	9.32	0.14	62.62	36.70	0.68
LL2	81.58	9.84	8.58	8.45	0.13	68.04	31.38	0.58
LL3	79.10	8.26	12.64	12.45	0.19	54.81	44.37	0.82
LL4	85.50	7.24	7.26	7.15	0.11	64.92	34.44	0.64
LL5	81.94	8.98	9.08	8.94	0.14	64.73	34.62	0.64
MR1	80.44	5.80	13.76	13.52	0.24	65.57	33.68	0.74
MR2	82.44	5.76	11.80	11.59	0.21	68.81	30.52	0.67
MR3	76.96	6.64	16.40	16.11	0.29	64.66	34.58	0.76
MR4	59.52	22.60	17.88	17.57	0.31	85.10	14.58	0.32
MR5	76.52	8.36	15.12	14.85	0.27	71.42	27.97	0.62
PT1	84.14	7.40	8.46	8.31	0.15	70.68	28.68	0.64
PT2	75.02	13.68	11.30	11.10	0.20	76.94	22.56	0.50
PT3	80.76	10.36	8.88	8.72	0.16	76.28	23.21	0.51
PT4	80.20	10.76	9.04	8.88	0.16	76.64	22.85	0.51
PT5	78.84	12.86	8.30	8.15	0.15	81.03	18.56	0.41
NG1	81.44	7.48	11.08	10.94	0.14	69.24	30.28	0.48
NG2	77.84	6.64	15.52	15.32	0.20	58.79	40.57	0.64
NG3	77.52	6.84	15.64	15.44	0.20	59.32	40.04	0.63
NG4	79.88	6.00	14.12	13.94	0.18	58.63	40.73	0.64
NG5	80.60	5.36	14.04	13.86	0.18	56.01	43.31	0.68
SD1	68.64	12.74	18.62	18.43	0.19	67.43	32.15	0.42
SD2	76.68	8.44	14.88	14.72	0.16	63.19	36.34	0.48
SD3	79.68	8.62	11.70	11.58	0.12	69.03	30.56	0.40
SD4	66.82	14.36	18.82	18.62	0.20	69.78	29.83	0.39
SD5	80.92	7.88	11.20	11.08	0.12	68.04	31.54	0.41
Mean	78.20	9.26	12.54	12.36	0.18	67.67	31.76	0.57
Mean absolute deviation	3.86	2.65	2.95	2.92	0.04	5.71	5.63	0.11
Coefficient of variation	7%	41%	28%	28%	29%	11%	23%	23%

2) After Compaction

The gravel proportions decreased significantly (Table 3), ranging from 55.52% (SD1 Ap) to 78.07% (NG5 Ap), with an arithmetic mean of 66.14%. The dispersion is characterized by a mean absolute deviation of 4.95%. Compared to the

Table 3. Proportions of granular fractions compacted to the OCE of the lateritic samples.

Sample name	63 < Ø ≥ 2 mm	2 < Ø ≤ 0.063	Ø ≤ 0.063	0.063 ≥ Ø < 0.002	Ø ≤ 0.002
LL1 Ap	67.45	15.86	16.68	16.43	0.25
LL2 Ap	62.61	20.30	17.09	16.83	0.26
LL3 Ap	64.82	17.86	17.32	17.06	0.26
LL4 Ap	60.04	20.96	19.00	18.71	0.29
LL5 Ap	67.80	16.65	15.55	15.31	0.24
MR1 Ap	68.59	13.59	17.82	17.50	0.31
MR2 Ap	71.96	15.78	12.26	12.05	0.22
MR3 Ap	69.42	15.19	15.38	15.11	0.27
MR4 Ap	61.20	17.52	21.28	20.91	0.37
MR5 Ap	61.53	18.81	19.66	19.31	0.35
PT1 Ap	65.78	21.70	12.52	12.30	0.22
PT2 Ap	62.93	24.40	12.67	12.44	0.23
PT3 Ap	69.88	18.92	11.20	11.00	0.20
PT4 Ap	72.43	18.22	9.35	9.18	0.17
PT5 Ap	65.63	21.67	12.71	12.48	0.23
NG1 Ap	70.57	13.49	15.94	15.74	0.20
NG2 Ap	69.85	13.48	16.67	16.45	0.21
NG3 Ap	73.11	13.19	13.70	13.53	0.18
NG4 Ap	76.41	12.28	11.31	11.17	0.15
NG5 Ap	78.07	12.07	9.86	9.73	0.13
SD1 Ap	55.52	20.68	23.80	23.55	0.25
SD2 Ap	57.40	19.60	23.00	22.76	0.24
SD3 Ap	56.44	20.04	23.52	23.27	0.25
SD4 Ap	64.17	17.13	18.70	18.50	0.19
SD5 Ap	59.88	21.38	18.75	18.55	0.20
Mean	66.14	17.63	16.23	15.99	0.23
Mean absolute deviation	4.95	2.83	3.38	3.35	0.04
Coefficient of variation	9%	19%	26%	26%	24%

pre-CBR data, the range of values narrowed further, while the mean absolute deviation and coefficient of variation increased, justifying the wider dispersion. The variation in this granular fraction indicates the resistance to crushing of gravels.

Sand percentages increased slightly, ranging from 12.07% (NG5 Ap) to 24.40% (PT2 Ap), with a median of 65.78% (PT Ap) and a mean of 66.14%. The distribution was characterized by a mean absolute deviation of 2.83%. The dispersion parameters decreased significantly for the coefficient of variation and slightly for the range, except for the mean absolute deviation, which increased slightly. The variation in this fraction and in the fines involved in particle tightening highlights the fragmentability and shock resistance of gravel particles since sand is shock-resistant due to its siliceous petrography.

The post-compaction fines range from 9.35% (PT4 Ap) to 23.80% (SD1 Ap). The mean is 16.23%. The dispersion around the mean is indicated by an absolute mean deviation of 3.38%. The proportion of fines increased following compaction.

These fines play a decisive role in the behavior of lateritic materials. The mass of silty particles also increased after compaction, ranging from 9.18% (PT4 Ap) to 23.55% (SD1 Ap) with a mean of 15.99%. The dispersion is indicated by a mean absolute deviation of 3.35%. The range of silt increased with its mean absolute deviation, while its coefficient of variation decreased. The variation in the percentage of silt depends on the contribution of gravel, which is crushed as a result of compaction.

The clay proportion increased slightly, ranging from 0.13% (NG5 Ap) to 0.37% (MR4 Ap) with a median of 0.23% (PT5 Ap). The mean absolute deviation was negligible, and equal to 0.04 %. Of the dispersion parameters, only the coefficient of variation decreased slightly. The variation in clay particle percentages following compaction allows to evaluate the clay content of gravel and its potential impact on the texture of lateritic material samples.

3.1.2. Clay Content and Activity of Lateritic Materials from the Thiès Region

The clay content of road materials is evaluated using the plastic limits and the MBV that are also employed to classify soils such as lateritic soils.

1) Atterberg Limits

The plasticity index provides information on the clay content and plasticity of a soil.

a) Before CBR

The liquid limits of the analyzed lateritic materials range from 28% (LL4) to 48% (NG2), with a mean of 36%. According to the results of Table 4, the dispersion is characterized by a mean absolute deviation of 4%.

The liquid limit marks the point at which free water begins to exist, breaking the cohesion of particles in the material and causing them to disintegrate and move. This can be avoided with prior knowledge of the liquid limit (LL). Additionally, the LL and PI are used to classify soils in the Casagrande [34] plasticity chart.

The plastic limits range from 15% (LL4) to 24% (NG2), with a mean of 19%. The mean absolute deviation is 2%. Knowing the plasticity limit helps prevent the exposure of these materials to extreme hydrological conditions, which can cause soil to slide and infrastructure to degrade.

The PIs for the analyzed granular classes range from 11% (SD2) to 24% (NG2, NG4, and NG5), with a mean of 17. The dispersion of values around the mean is characterized by a mean absolute deviation of 3, as specified in **Table 4**.

The PI obtained before compaction can be used to predict the condition of lateritic materials after compaction, with respect to climatic variations, particularly rainfall.

Table 4. Clay content and activity parameters of the analyzed samples.

Sample name	LL before CBR	PI before CBR	Ac	LL after CBR	PI after CBR	MBM (g/kg)	ACB
LL1	32	16	0.23	31	11	0.75	0.11
LL2	34	16	0.21	32	13	0.86	0.15
LL3	32	16	0.26	28	12	0.84	0.10
LL4	28	13	0.26	30	12	0.65	0.10
LL5	30	14	0.32	26	12	0.78	0.12
MR1	38	18	0.31	36	13	0.82	0.11
MR2	35	17	0.24	34	13	0.73	0.11
MR3	36	17	0.35	36	15	1.63	0.21
MR4	36	18	0.29	34	15	0.92	0.29
MR5	39	22	0.46	37	16	0.98	0.16
PT1	31	14	0.17	32	11	0.61	0.10
PT2	31	14	0.19	28	9	1.00	0.20
PT3	30	13	0.26	28	11	0.80	0.16
PT4	32	16	0.25	31	14	0.83	0.16
PT5	35	18	0.44	32	15	0.92	0.22
NG1	44	21	0.34	41	20	0.80	0.17
NG2	48	24	0.75	46	23	0.85	0.13
NG3	44	21	0.42	46	20	0.77	0.12
NG4	46	24	0.37	41	20	0.94	0.15
NG5	46	24	0.62	43	23	0.74	0.11
SD1	34	16	0.25	36	18	1.15	0.27
SD2	31	11	0.18	39	19	0.89	0.19
SD3	34	17	0.40	35	21	0.78	0.19
SD4	38	16	0.24	40	20	1.38	0.35
SD5	35	17	0.41	38	21	0.64	0.15
Mean	36	17	0.33	35	15.90	0.88	0.17
Mean absolute deviation	4	3	0.10	5	3.77	0.15	0.05
Coefficient of variation	16%	21%	41%	16%	27%	26%	39%

The Skempton activity [35] of the samples ranges from 0.17 (PT1) to 0.75 (NG2). It gives an indication of the type of clay contained in soils with a certain plasticity.

b) After CBR

Overall, the compacted materials displayed a slight decrease in PIs ranging from 9 (PT2 Ap) to 23 (NG2 Ap and NG5 Ap) with a mean of 16. The dispersion is indicated by a mean absolute deviation of 4. The PIs decreased following the decrease in liquid limits, except for the PT1 Ap and NG3 Ap samples. As shown in **Table 4**, the LL4Ap and SD1-5Ap samples had increasing liquid limits, resulting in an increase in PIs.

The evolution of plasticity after compaction indicates whether the gravel content is clayey, as it has been partially crushed to combine with smaller particle fractions.

2) Determination of the Methylene Blue Mass of the 0/2 mm Fraction of Lateritic Materials from Thiès

For samples weighing 60 g, the volumes poured ranged from 20 ml (SD5) to 28 ml (LL2). The mean was recorded as 24 ml, with a mean absolute deviation of 1 ml.

The MBM ranged from 0.6 g/kg (PT1) to 1.6 g/kg (MR3), with a median and mean of 0.9 g/kg, and a mean absolute deviation of 0.1 g/kg (**Table 4**).

MBM represents the amount of blue that can be absorbed onto the accessible surfaces of soil particles. It helps to assess the specific surface area of the granular masses studied and prevents saturation, which can alter the cohesion of lateritic materials and promote crumbling of the material.

Differences in MBM among samples are often attributed to variations in the proportion of fines in the 0/2 mm fraction.

For the 0/20 mm reference granular class samples, the ACB ranges from 0.10 (PT1) to 0.27 (SD1). For samples from other granular classes, the ACB value varies from 0.10 (LL3-4) to 0.35 (SD4).

3.2. Proctor and CBR Design Tests on Untreated Lateritic Material Samples

Knowledge of compaction characteristics that are the optimum moisture content or OMC and the maximum dry density or MDD (OMC-MDD) is essential for road construction sites. A layer of compacted material must reach a specified percentage of dry density under site conditions, referenced against the MDD obtained in the laboratory. The CBR is a design parameter that is determined at 95% and 100% of the OCE. The higher of the two values is the result to be considered.

3.2.1. Experimental Determination of Compaction Characteristics by Proctor Test

The optimum Proctor modified compaction characteristics (OMC-MDD), for the lateritic gravel samples from Lam-Lam (LL), Mont Rolland (MR), Pout (PT), Ngoundiane (NG) and Sindia (SD), which have grain sizes of 0/20 mm, 0/25 mm, 0/31.5 mm, 0/40 mm and 0/50 mm respectively, are given in Table 5.

The 0/20 mm granular class is the only one standardized class. For this reference class, the OMCs values for the lateritic gravel samples range from 10.45 (MR1) to 13.09% (SD1). The samples LL1 and NG1 have close OMCs, of 10.81

Table 5. Behavioral characteristic and use of lateritic gravels in Lam-Lam, Mont-Rolland, Pout, Ngoundiane, and Sindia.

Sample name	OMC (%)	MDD (KN/m3)	CBR at 95 % OCE (%)	CBR at 100 % OCE (%)	Improved AGEROUTE class	Use
LL1	10.81	19.8	63	91	GL2	Sb[b]
LL2	12.58	19.2	58	86	GL1	Sb[b]
LL3	12.50	19.6	79	131	GL2	B[a]
LL4	11.43	19.5	58	70	GL1	Sb[b]
LL5	12.08	19.3	32	106	GL1	Sb[b]
MR1	10.45	19.6	89	128	GL2	Sb[b]
MR2	9.50	18.9	126	146	GL3	Sb[b]
MR3	10.07	20	90	163	GL3	B[a]
MR4	11.94	19.3	81	106	GL3	Sb[b]
MR5	10.63	20.2	76	87	GL2	B[a] T1[e]
PT1	11.44	19.4	77	88	GL2	Sb[b]
PT2	12.38	19.3	71	159	GL2	Sb[b]
PT3	11.59	19.5	84	154	GL3	Sb[b]
PT4	11.62	19.5	58	144	GL1	Sb[b]
PT5	11.46	19.2	74	230	GL2	Sb[b]
NG1	10.99	20.2	55	80	GL1	Sb[b]
NG2	10.99	20.4	44	65	GL1	Sb[b]
NG3	11.23	20.3	55	69	GL1	Sb[b]
NG4	11.09	20.1	37	57	GL1	Sb[b]
NG5	10.70	20.4	82	90	GL3	B[a]
SD1	13.09	18.3	25	29	GL0	Sg[c]/Ek[d]
SD2	14.33	18.3	16	24	GL0	Sg[b]/Ek[d]
SD3	12.50	18.7	28	49	GL0	Sg[b]/Ek[d]
SD4	11.83	18.5	30	40	GL1	Sb[b]
SD5	11.70	18.1	40	46	GL1	Sb[b]
Mean	11.56	19.42	61.11	97.59	-	-
Mean absolute deviation	0.76	0.5	20.70	39.41	-	-
Coefficient of variation	9%	3%	42%	51%	-	-

a: Base; b: Subbase; c: subgrade; d: Embankment; e: Traffic 1.

and 10.99%, respectively. For PT1, the OMC value is 11.44%.

Samples of other granular classes have OMC content values ranging from 9.50 (MR2) to 14.33% (SD2). Most samples have OMCs between 10 and 12%.

Samples of the 0/20 mm which is the reference granular class have maximum dry density values (MDD) ranging from 18.3 (SD1) to 20.2 kN/m^3 (NG1). The other samples in this class (LL1, MR1, and PT1) have similar MDDs: 19.8, 19.6, and 19.4 kN/m^3, respectively.

The MDDs of the other granular classes defined in this study range from 18.1 (SD5) to 20.4 kN/m^3 (NG2 and NG5).

3.2.2. Experimental Determination of the Bearing Capacity of Materials by CBR Test

CBRs obtained with the standard reference granular class (0/20 mm), at 95% of the OCE, range from 25 (SD1) to 89% (MR1). The other samples, LL1, PT1, and NG1, have non-negligible CBRs of 63, 77, and 55%, respectively.

For other granular classes, the CBR at 95% of the OCE ranges from 16% (SD2) to 126% (MR2).

Increasing compaction from 95% to 100% of the OCE increases the CBR value of these materials, regardless of the granular class (see Table 5).

3.3. Experimental Determination of the Durability Properties Related to the Interaction with Water

These properties were only measured for the five samples within the standardized 0/20 mm class.

3.3.1. Linear Swelling and Water Absorption of Compacted and Soaked Materials

The results of the relative linear swelling measurements are shown in Table 6.

The relative linear swelling values are 0.02% for samples MR1, PT1, and SD1; 0.08% for LL1 and NG1. These values are low, falling well below 1%.

Gomez-Gutierrez, Bryson, and Hopkins [36] demonstrated a correlation

Table 6. Linear swelling, water absorption and permeability of the Thiès materials.

Samples	Linear swelling (%)	Water absorption (%)	k (m/s)
LL1	0.08	7.68	9.14.10−6
MR1	0.02	11.53	1.14.10−5
PT1	0.02	7.60	1.00.10−5
NG1	0.08	8.74	1.02.10−5
SD1	0.02	9.56	9.14.10−6
Mean	0.04	9.02	9.76E−05
Mean absolute deviation	0.03	1.22	6.53E−06
Coefficient of variation	77%	18%	9%

between linear swelling and water absorption in clay-containing road materials.

The water absorption results are also displayed in **Table 6**. The absorption values range from 7.68% (LL1) to 11.53% (MR1). Absorptions for the other samples (PT1, NG1, and SD1) are 7.60%, 8.74%, and 9.56%, respectively.

3.3.2. Permeability of Compacted Materials

Table 6 shows the permeability coefficient k for the five samples of the standardized class 0/20 mm. Permeabilities range from 9.14×10^{-6} (LL1 and SD1) to 1.14×10^{-5} m/s (MR1). The k values for samples PT1 and NG1 are 1.00×10^{-5} and 1.02×10^{-5} m/s, respectively.

4. Discussion

Numerous studies have demonstrated the influence of the petrographic nature, granularity and mineralogical composition of road materials on their compactness and bearing capacity [20] [37]-[41].

Preventing early pavement deterioration is a pressing issue in scientific and technical research. This article focuses on analyzing how granular size variability influences the nature, condition and behavior of lateritic materials in tropical countries. The approach is intended to be comprehensive, with the use of several parameters, calculated in broader fields of study than those of Cisse *et al.* [40] and Diop *et al.* [39].

To compare the results to the current specifications, the following references will be used: the Catalog of New Pavement Structures and Pavement Design Guide in Senegal [42]; the Guide to Earthworks, Embankments, and Subgrade Layers [1] [25]; the Practical Guide to Pavement Design in Tropical Countries [43]; and the Catalog of Standard New Pavement Structures [44].

4.1. Soil Classification and Compaction Resistance of Lateritic Soils from the Thiès Region

The percentages of fines in the analyzed lateritic samples are below 35%. Consequently, fines do not dictate the behavior of these lateritic materials [1] [25]. Thus, the latter will thus be able to offer relatively superior strength, since they have a spread grading, where the voids left by gravelly particles are filled by sand and fines without excess or water saturation. The sand and fines in the analyzed soils do not comprise 70% of their mass [45]. These soils are not sandy in texture, so their permeability will be limited. In fact, after compaction, sand can be more representative if it is significantly incorporated into the gravel aggregates.

The gravel content, of the samples, exceeds 40%. These lateritic materials are therefore lateritic gravel soils [3], as illustrated in **Figure 4**. More specifically, they are gravelly, silty gravelly and sandy-loamy gravelly soils.

In addition to these characteristics typical of lateritic gravel soils, the samples from the Mont Rolland, Ngoundiane and Sindia borrow pits are divided into classes with percentages of fines below 15% and others above. The Lam-Lam and Pout samples have percentages below the threshold for separating gravelly and intermediate soils according to GTR 2023 [25].

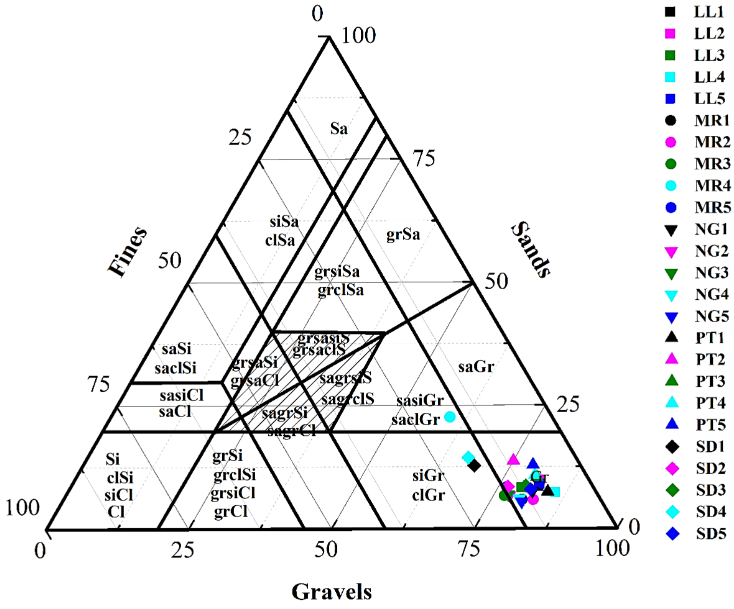

Figure 4. Grain size classification, according to standard NF EN ISO 14688 (Gr: gravel; Sa: sand; Si: silt and Cl: clay), of samples from LL: Lam-Lam, MR: Mont Rolland, PT: Pout, NG: Ngoundiane and SD: Sindia borrow pits.

Moreover, the 0/20 mm granular classes mostly have fines content below 15% (LL1, MR1, PT1 and NG1) and are often below 12% fines (LL1, PT1 and NG1). This can alter the gravelly texture of these samples from a granulometric point of view. In contrast, the 0/31.5 and 0/40 mm classes, which had the best geotechnical characteristics in previous studies [39] [40], mostly have fines content higher than 12%. Seven out of ten samples of these two classes have fines contents between 12 and 25%, which corroborates the gravelly texture of these samples (LL3, MR3, MR4, NG3, NG4, SD3, and SD4).

According to GTR 2023 [25], the soil samples fall into two granular classes: G (Gravelly), with fines contents between 0 and 15%, and I (Intermediate) with fines contents between 15 and 35%. The first class includes: LL1-LL5, MR1-2, PT1-5, NG1, NG4-5, SD2-3, and SD5. The second class includes the remaining 7 classes: MR3-5, NG2-3, SD1 and SD4.

4.1.1. Compaction Characteristics of Analyzed Lateritic Gravel Samples

Notable steps are sometimes present, particularly between the nominal sieve sizes of 0.063 and 0.0125 mm, and between 0.5 and 1 mm. These findings are corroborated by Van Ganse [46]. All particle size curves fall within the range defined by the specifications [42] [43], as illustrated in **Figure 5**.

According to the specifications reported by Hubert *et al.* [47], the curves are all well-spread (Cu >> 2) and discontinuous (Cu > 15) as displayed in **Figure 5** and presented in **Figure 6**.

All of the Cu values are above 6 (**Figure 6**). Therefore, all eighteen samples in

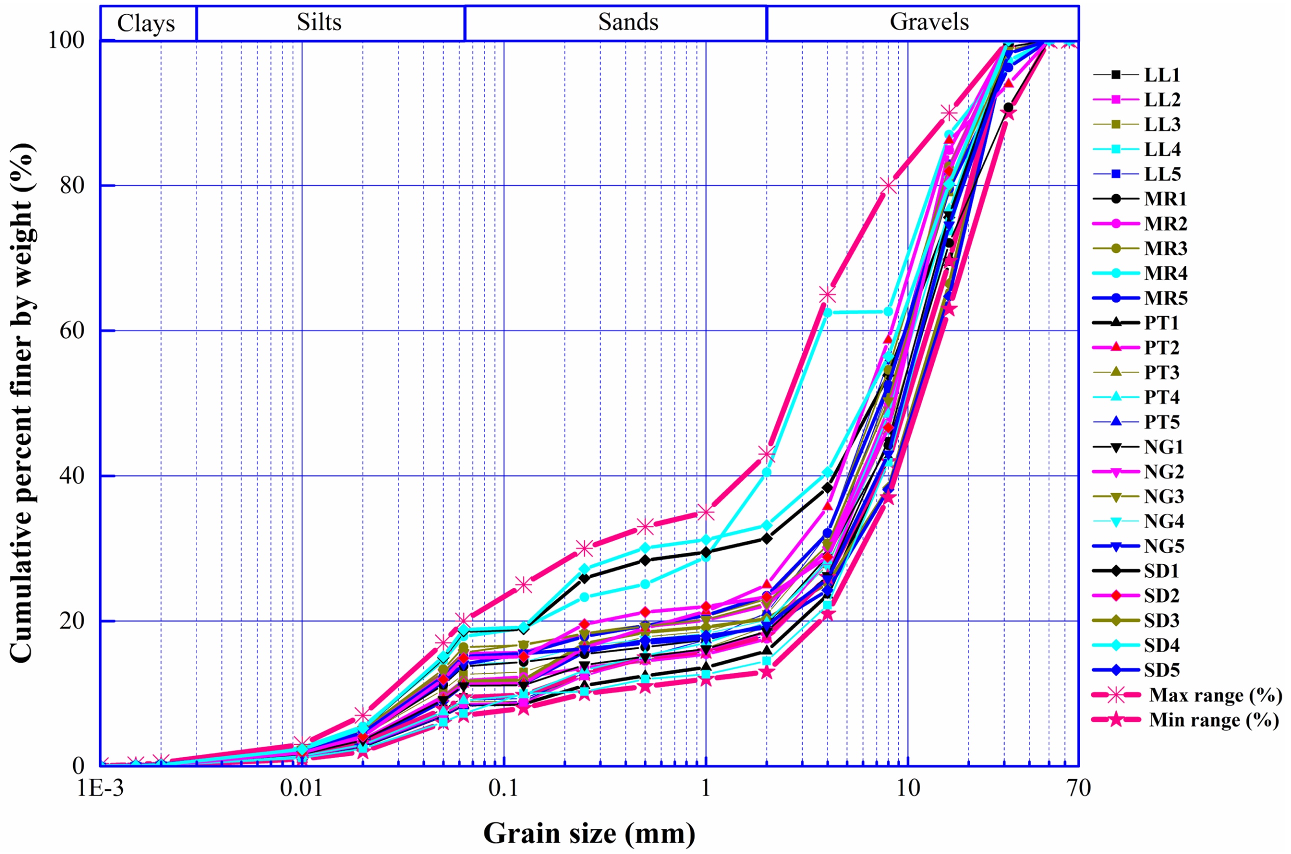

Figure 5. Pre-CBR particle size distribution curves for the studied granular classes.

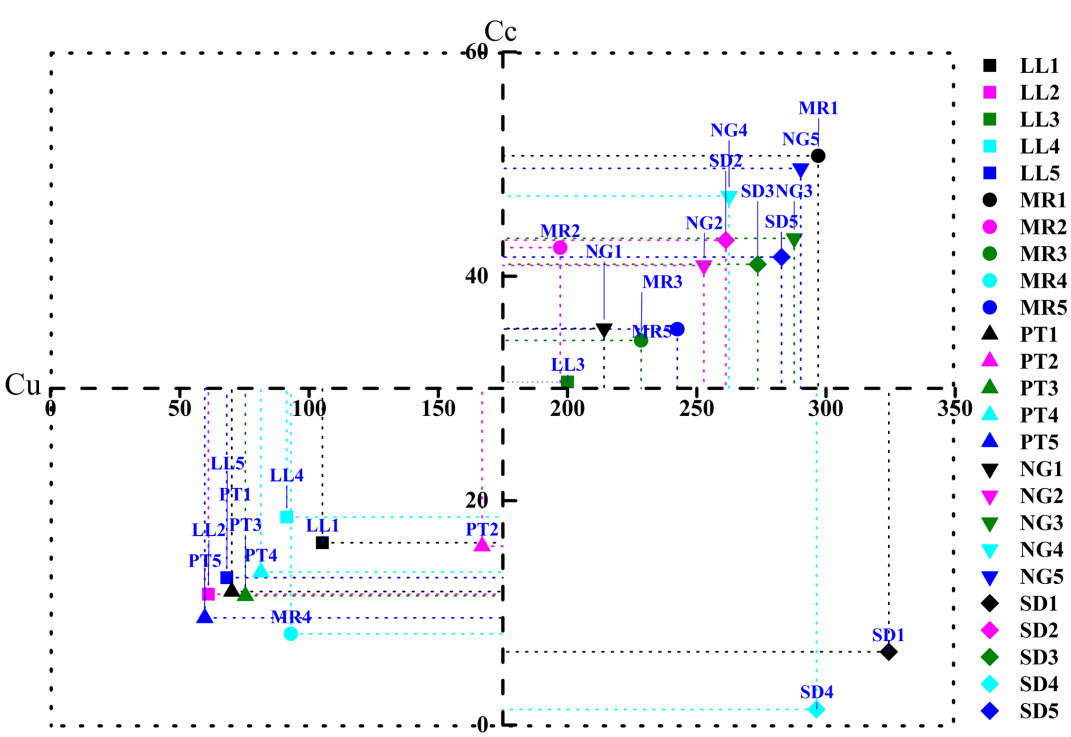

Figure 6. Hazen coefficients for the analyzed samples (Cu: Uniformity coefficient; Cc: Compliance coefficient).

class G are subclass G3.

The compaction characteristics of lateritic materials depend on their particle size, which depends on the pedogenesis of the soil, through chemical weathering, or laterization. This weathering essentially affects the composition, texture, and structure of the original formations [18].

Laterization leads to the formation of lateritic nodules, or pisolites. This evolutionary process begins with leached elements that are then concretized after clay is released and iron minerals are formed, which cement the residual quartz sand particles. These minerals have all been identified in these soils by [48]. The well-spread curves are therefore the result of the gradual evolution of cementation involving the formation of granular particles ranging from clay to gravel for a structure with distinct levels of organization [19] [49].

4.1.2. Texture of the Sandy and Fines Fractions of the Lateritic Soils from Thiès

Figure 7 shows the soil texture of the 0/2 mm arable fractions is sandy loam for most of the analyzed samples, sandy for classes MR4 and PT5, and light sandy loam for PT2 [50]. Fines in the samples are silty. These results corroborate the silty and sandy qualifiers used in the granulometric classification of some samples.

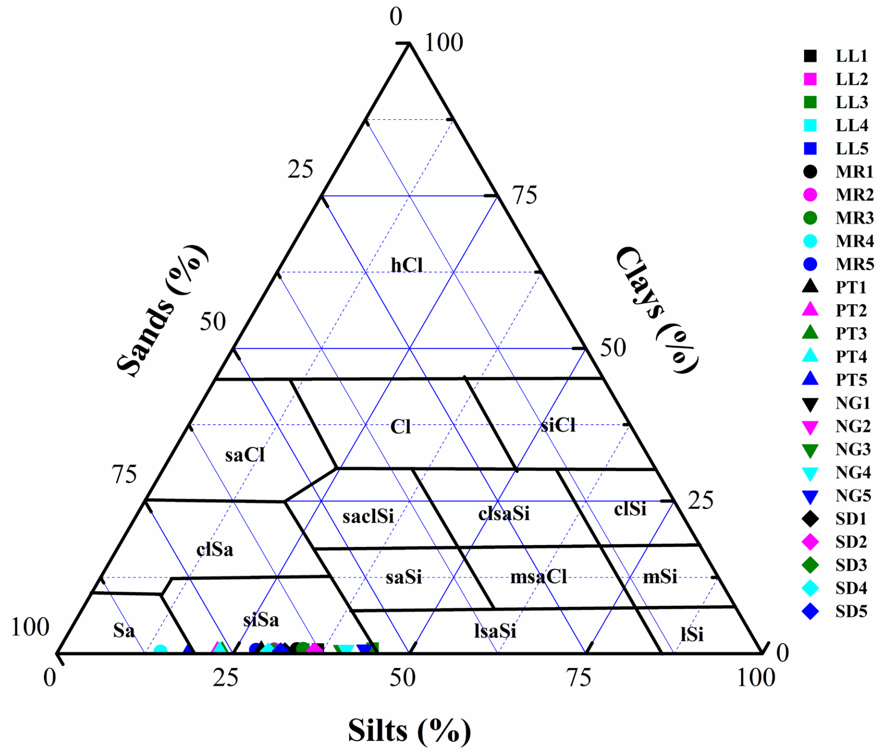

Figure 7. Textural triangle of sandy-loamy-clay fractions of lateritic gravels analyzed according to Jamagne [50] and Folk [51] (h: heavy, m: medium, l: light, Cl: Clay, Sa: Sand, Si: Silt).

4.1.3. The Effects of Compaction on the Granularity of Lateritic Materials

The effect of compaction is noticeable in all the analyzed granular classes, though it affects gravel more than other granular fractions. Nevertheless, the percentage of gravel by weight remains above 40% after compaction. The material will remain gravelly in the pavement layers after compaction. The gravel crushed has partly transformed into sand and fines to adequately fill the voids between the gravel particles [52]. This guarantees the continuity of the mechanical media of the pavement layers. This condition is necessary for properly transmitting vehicle weights to the pavement subgrade or existing soil.

Materials with fewer fines and sands have a purely gravelly texture (Gr) and more voids. These materials tend to have their gravel crushed more following compaction. Examples include samples LL2, LL4, PT1, SD2, SD3, and SD5, as presented in **Figure 8**. Conversely, samples with relatively more sand and silt (gravelly sandy-silty to silty), referred to as sasiGr and siGr according to NF EN ISO 14688 [3], are crushed less due to their compact texture. Examples include samples MR3, MR4, SD1, and SD4. The compaction of classes MR3, NG3, NG4, NG5, and SD4 caused a slight decrease in the relative percentage of fines in favor of sands. The clay content of soils is often linked to the abundance of fines.

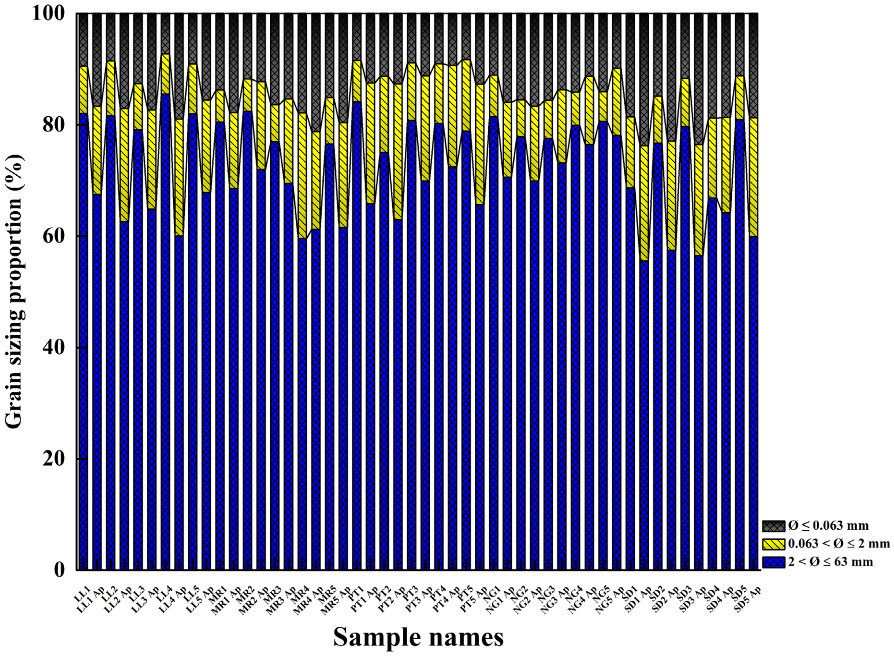

Figure 8. Effect of compaction on the texture of lateritic soils.

However, gravel must be resistant to crushing during compaction. To be suitable for road construction, the material must be brought to an optimum mechanical state of compactness and bearing capacity. Suitable compaction produces the particles needed to achieve optimum compactness. According to the CEBTP guide, the quantity of fines (≤0.08 mm) should increase by no more than 8% after compaction in order to authorize the use of lateritic gravel in road base courses. However, since the definition of fines has changed, this limit should change as well. Taking this specification into account, samples LL2, LL4, SD2, and SD3 would be disqualified for use in base courses, even if their CBRs reached 60%.

4.1.4. Influence of Granularity on the plasticity of Lateritic Gravels from the Thiès Region

According to GTR 2023 [25], the majority of the mortar or 0/400 µm fraction of lateritic gravel samples analyzed have PIs between 12 and 22, indicating moderately clayey soils for 20 samples. This fraction in samples MR5, NG2, NG4, and NG5 are clay soils with an PI value of at least 22. However, sample SD2 is low in clay, with a PI value of less than 12, according to GTR 2023 [25] and NF P11-300 standard [2].

The samples are also moderately plastic (5 ≤ PI < 15) to plastic (15 ≤ PI < 40), as reported by Berthaud, De-Buhan, and Schmitt [53]. These characteristics are confirmed by the Casagrande abacus (**Figure 9**).

The area occupied by laterite mortar above line A in this diagram reveals poorly to moderately plastic, and inorganic soils [34].

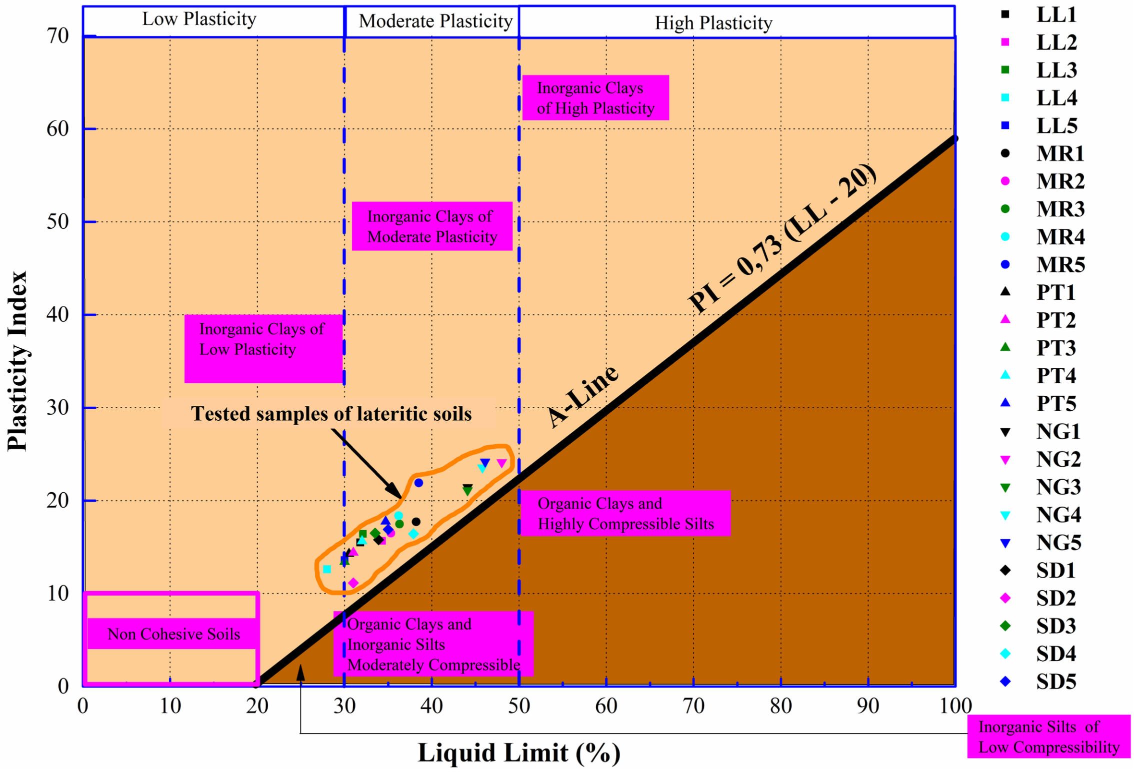

Figure 9. Classification in the plasticity chart of the tested samples ([2]; modified).

Consequently, when their moisture content averages over 36%, the mortar in these samples behaves like a viscous or liquid material.

Compaction of the lateritic gravel resulted in a decrease in the PI for 20 of the 25 analyzed samples, all of which stayed in the slightly to moderately clayey range. The exception was sample NG2, which stayed clayey (**Table 4**).

The remaining five samples are from Sindia and are still moderately clayey. However, unlike the others, compaction has increased their PIs. This increase can be linked to the mineralogical composition of the fines in the gravels of a soil, the predominance of fines over sand, which is notable in SD1, 2, and 3, or the relative proportion of sand and fines compared to the composition of the other samples. Diop *et al.* [48] invalidate the first hypothesis since kaolinite is the clay present in all the borrow pits. The second hypothesis is supported by Otçu *et al.* [54], while the third needs validation.

The maximum Skempton activity value for the samples is 0.749 (**Figure 10**). According to Skempton [35], the clay mineralogy of the samples is inactive. The kaolinite clay of these materials is favorable for the pedogenesis of the study sites [20]. Compared with other clays, kaolinite offers the lowest sensitivity to water on its specific surface area [47].

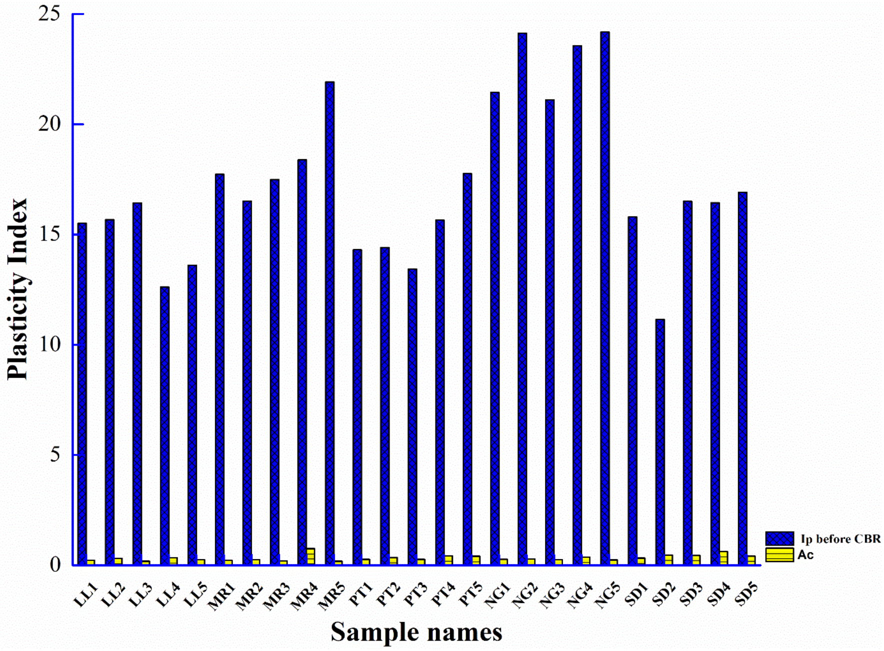

Figure 10. Plasticity indices and Skempton activity of analyzed samples.

Maintaining laterites in a medium state of plasticity by adjusting their moisture content is advantageous for improving their workability during implementation.

MBM is another clay parameter that depends on the abundance of electronegative and adsorptive clay particles. Almost all samples are insensitive to water (MBM < 1), except for MR3 (1 ≤ MBM < 2), which is slightly sensitive to water (**Figure 11**). This confirms the granulometry indicating clay proportions of less than 1%. The low presence of fines is confirmed by the low sensitivity to water and the presence of kaolinite identified in the mineralogical composition of the analyzed lateritic gravels [48]. The low A_{CB} values indicate that the clay fraction of these soils is inactive.

4.1.5. Classification of Lateritic Gravels from the Thiès Region

According to GTR 2023 [25], the lateritic gravel samples studied ultimately fall into classes G3 and I2, corresponding to gravelly and intermediate soils (**Figure 12**).

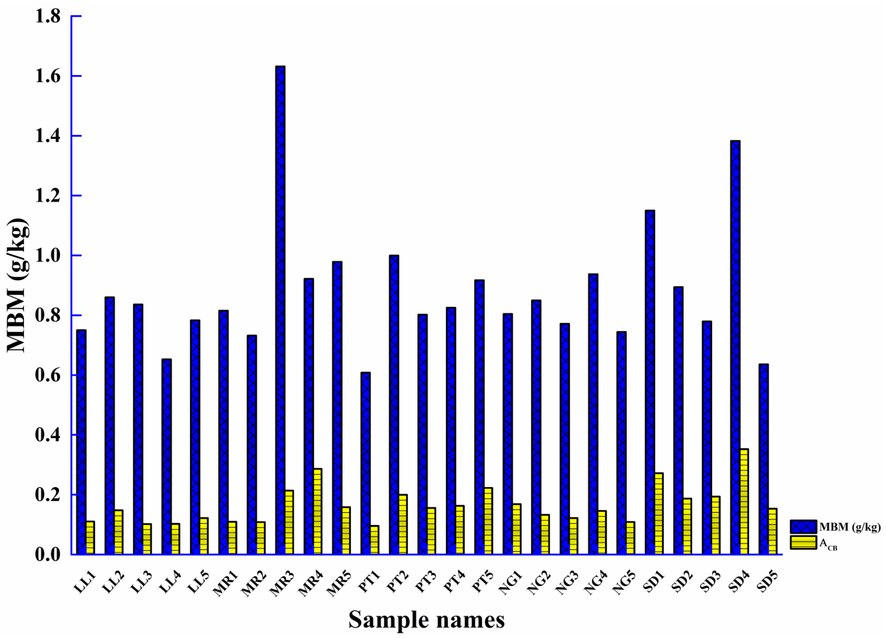

Figure 11. Blue mass and activities of the clay fraction in the samples analyzed.

According to ASTM 2000 [55], they are classified as GM (silty gravel and silty gravel with sand if sand ≥ 15%) and GW-GM (well graded gravel with silt). Seven of ten 0/20 and 0/31.5 classes are classified as G3, meaning they are gravelly soils that are better suited for road construction. The MBVs do not guide the classification, as the PIs exceed 12.

These results demonstrate that the variability of granular material classes from the same borrow pit reflects their nature and geotechnical behavior. Similar classes that are classified using the same classification parameter generally originate from the same borrow pit. This leads us to study intra-borrow segregation.

4.2. Compaction Quality Control and Use Guiding of Materials

Investigations are used to monitor and assess the implementation of road projects. According to Hubert *et al.* [47], they are carried out during the design and implementation phases, as well as for approval.

4.2.1. Compaction Quality Control

The quality of compaction can be assessed during the work using a nuclear gauge in accordance with the NF P 98-241-1 standard [56]. The measurement takes one minute, after which the device indicates wet density, moisture content, and dry density.

This control can also be carried out using the balloon density method of standard NF P 94-061-2 [57], though it is less precise. This method involves measuring the volume of a cavity created by extracting moist material. The maximum dry weight is then calculated after oven drying at 105˚C.

These measurements are used to assess the following:

- The compactness of the material layer in place;

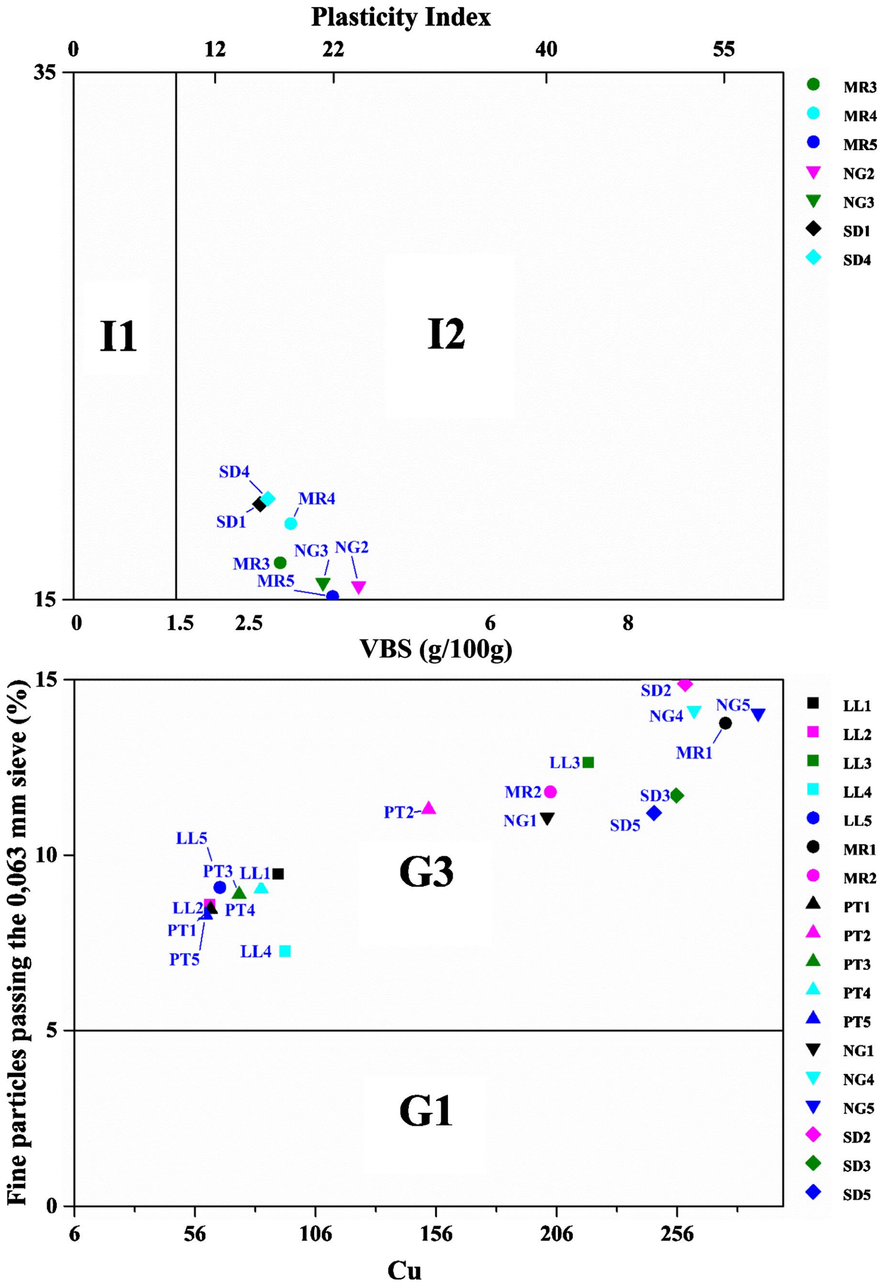

Figure 12. Classifications according to GTR 2023 [25] of samples coming from LL: Lam-Lam; MR: Mont Rolland; PT: Pout; NG: Ngoundiane and SD: Sindia borrow pits.

- The moisture content of the material, which must be within the correct processing range.

The contracting authority sets the values of these parameters and the acceptable margins of error, which are included in the Construction and Standard Specifications.

4.2.2. Orientation of Materials According to Site Conditions

Table 5 provides data that can be used to select the appropriate road materials for the different layers of a flexible pavement. The selection is based on classification parameters, traffic intensity, and the SCBR of samples analyzed at 95% of the OCE.

4.3. Sensitivity to Changes in Materials

The water absorption of the samples is less than 15% of the sample mass, with a linear swelling below 1% (**Figure 13**). This indicates the good compactness of the specimens and their low sensitivity to water. These values reflect the low representativeness of the clay, which is kaolinitic and non-swelling [4] [20].

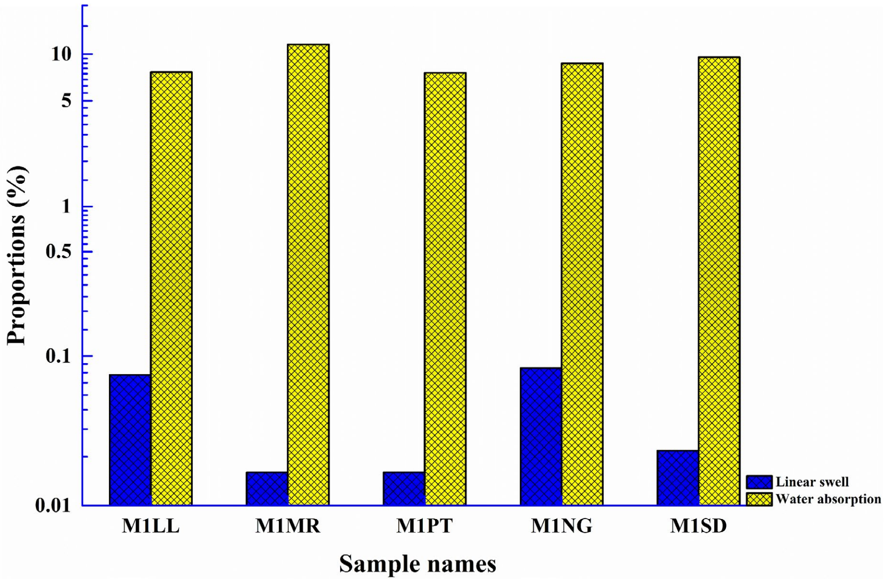

Figure 13. Representation of linear swelling and water absorptions of 0/20 mm class lateritic gravel samples during soaking.

Soil permeability is generally inherent to its mineralogical composition and granular texture [58]. However, the presence of organic matter and the activities of organisms can significantly increase permeability. The permeability of compacted road materials must be low to ensure the watertightness of the pavement.

According to the LCPC-SETRA and IDRRIM [1] and [25], road earthworks guide, lateritic gravels that are compacted to over 92% of the OCE normally offer significant impermeability when used in pavements. Consequently, they ensure the waterproofing of pavement layers and the drainage of surface water via appropriately sized overflows [59]. According to Casagrande and Fadum [45] [60], all of these compacted samples at 95% of the OCE fall into the category of impermeable soils formed by climate and vegetation with good drainage. The linear swelling coefficient of less than 1% and the low absorption of these materials attest to their inactivity and compliance with AGEROUTE and LCPC-SETRA [42] and [1] specifications.

Thus, compaction significantly closes pores, rendering the lateritic gravels of Lam-Lam, Mont-Rolland, Pout, Ngoundiane, and Sindia impermeable.

4.4. The Influence of Granularity on the Behavior of Lateritic Gravels from the Thiès Region

In road geotechnics, characterizing the behavior of lateritic materials generally involves determining their MDD, OMC, and bearing capacity.

To accurately measure the bearing capacity in a laboratory setting, it is crucial to ensure that the material being analyzed is equivalent to the material used on site rather than relying solely on compaction characteristics. According to Diop *et al.* [39], the latter have no real correlation with the bearing capacity of lateritic materials.

4.4.1. Overall Study of Borrow Pits

1) Compaction characteristics of materials

According to AGEROUTE and CEBTP ([42] and [43]), all the classes analyzed are usable in the sub-base course of flexible pavement, embankments, and subgrades under low traffic conditions (NE < 1.5 million) at MDD of lateritic gravels above 18 kN/m^3. According to CEBTP [43], only materials whose samples have a maximum dry density of at least 20 kN/m^3 will be usable in base courses under low traffic levels, such as MR3, MR5, and NG1-5.

2) California bearing capacity of the materials

According to **Figure 14** and **Table 5**, the 0/20 mm class of lateritic gravel can be used as select fill and subbase layers regardless of traffic. All of the materials except for the Sindia material (SD1) can also be used as a subbase for all types of traffic. Lateritic gravels from Lam-Lam (LL1), Mont Rolland (MR1), and Pout (PT1) are well-suited for use as base course material for flexible pavement with low traffic levels. Mont Rolland (MR1) is the only material that can be used in a base course for flexible pavement with moderate traffic (NE ≥ 1.5 million).

Among the other granular classes, the Sindia samples have the lowest SCBRs, except for the 0/50 mm class. Conversely, Mont Rolland samples have the highest CBRs, except for the 0/50 mm class (**Figure 14**).

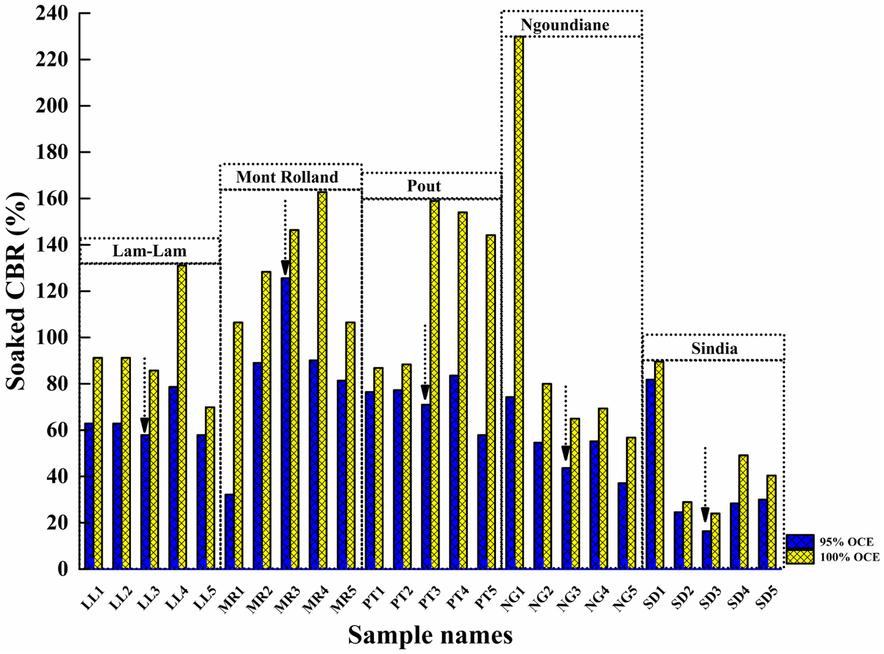

Figure 14. Representation of soaked CBRs of lateritic gravel samples from the Thiès region.

The 0/31.5 mm samples indicated by the arrows have acceptable CBRs and MDDs that are not dispersed (**Figure 14**).

In practice, the project owner in Senegal sets the maximum compactness limit for these materials at 98.5% of the MDD at OMC, for use in sub-base [42].

This dry density is not only a compaction characteristic, but also an indicator of the resistance of the soil skeleton to fragmentation. Therefore, it is clear that layers with a higher density will be the most stable.

Classification and behavioral data are used to relate the studied parameters.

Samples with the highest fines content are most sensitive to water (MR3, MR5, SD1, and SD4).

Samples with the highest fines content after compaction have the lowest MDD and CBR and the highest OMC (SD1, SD2, and SD3).

Conversely, they have the highest OMC. After compaction, the classes containing more gravel have the lowest sand content and the highest PI and MDD (NG2, NG3, NG5), which is consistent with the findings of Chaulagai *et al.* [52]. Furthermore, the classes with more sand give the lowest PI after compaction (PT2, PT1, LL4) for varying MBM (**Table 4** and **Figure 11**). It should be noted that classes distinguished by their preponderance or low attribution relative to a given parameter generally belong to the same borrow pit. This leads us to study the relationship between parameters within each borrow pit.

4.4.2. Partial Study of Borrow Pits

Previous studies of Diop [39] conducted on the Sindia site alone showed that the 0/31.5 mm grain size class, SD3, had the best geotechnical characteristics for pavement construction. The sampling points chosen for this study differ from those chosen in 2018 [39], despite being in the same borrow pit. The standards and specifications have improved, and the study area has expanded to include four additional sites in the Thiès region.

1) Geotechnical Properties of Lam-Lam Lateritic Gravels

According to the MBM, the classes most sensitive to water are the most demanding in terms of OMC to achieve the MDD (LL2, LL3, and LL5). Classes LL1 and LL3 are the densest and most load-bearing. LL1 is the densest and LL3 is the most load-bearing.

LL3 is more sensitive to water than LL2 and is the least active in blue. It is also the richest in fines and the second most demanding in terms of OMC after LL2.

However, the materials in this borrow pit are the least permeable and absorbent. Nevertheless, all other materials have low permeability and absorption. The comparison of the swelling, permeability, and absorption parameters is only for illustrative purposes.

2) Geotechnical properties of the lateritic gravels of Mont Rolland

The densest classes (MR3 and MR5) are the most sensitive to water. Additionally, the classes with the lowest OMC are the most load-bearing (MR2 and MR3).

The class that is the least sensitive to water, the least dense, the least demanding in terms of OMC, and that contains the fewest fines proves to be the most load-bearing (MR2). MR3 follows, being the densest after MR5, unlike MR2, which has the highest bearing capacity.

Mont Rolland material also exhibits the least swelling, similar to Pout material, though swelling is low for all materials analyzed.

3) Geotechnical Properties of Pout Lateritic Gravels

The densest and least water-sensitive classes, PT1 and PT3, require the least amount of water to reach maximum dry density (MDD) and have the highest bearing capacity. Class PT4 has the same maximum dry density as PT3, but it requires more water to reach MDD. It is also more clayey and richer in fines, and it has the lowest bearing capacity of all the characterized classes. PT3 is the least plastic and least sensitive to water, and it is the densest class, so it has the highest bearing capacity. PT1 follows it in terms of bearing capacity; it is slightly less dense and less sensitive to water.

4) Geotechnical properties of Ngoundiane lateritic gravels

The most load-bearing classes are the least water-sensitive and densest (NG3 and 5). The most water-sensitive and least dense class (NG4) is the least load-bearing. NG2 and NG4 are among the most plastic and the least dense. This may be because the greater the specific surface area of a material is, the greater its tendency to swell and the lower its density is. The greater vertical swelling of these borrow pit materials may support this idea. NG5 is the most load-bearing, followed by NG3, which has a higher load-bearing capacity than NG1, the standardized class.

5) Geotechnical properties of lateritic gravels from Sindia

Classes SD4 and SD5 are among the clayiest and least demanding in terms of compaction moisture content. They also have the highest bearing capacity. SD5 is the least sensitive to water and has the lowest OMC to achieve its MDD. SD5 also has the highest bearing capacity of the classes in this borrow pit. SD4 follows SD5 in terms of bearing capacity and plasticity. Despite being denser than SD5, SD4 is the richest in fines and the most sensitive to water. SD3 is the densest of all the classes and ranks third in terms of bearing capacity. These materials are among the most absorbent and least permeable, showcasing two opposing yet relevant trends.

The slightly lower bearing capacity of class SD3 is unexpected, as it has the lowest water sensitivity and fines content after class SD5. It also has the highest maximum dry density. However, classes SD4 and 5 surpass it in terms of non-requirement in OMC and bearing capacity. Conversely, the studies of Diop *et al.* [39], showed the superior qualities of the 0/31.5 class over all the others. However, the materials are of retrograde quality due to the advanced operation of this borrow pit, which focuses primarily on higher quality sampling points.

4.4.3. Relevant Parameters for Assessing the Usability of Lateritic Gravel

Though MDD is an important specification parameter, water sensitivity, MBM, PI, fines content, and OMC are also significant for these materials. Materials with the highest density after compaction may be the most load-bearing, as seen in

Pout and Ngoundiane, or among the most load-bearing, as seen in Lam-Lam.

The 0/31.5 mm class stands out in all the results obtained, as it is the densest class in Pout and Sindia, and the second densest class in Lam-Lam, Mont Rolland, and Ngoundiane. It is also the most load-bearing class in Lam-Lam and Pout, as well as in previous studies in Sindia [39], prior to the depletion of high-quality sampling pockets. It is the second most load-bearing class in Mont Rolland and Ngoundiane. It is the third most load-bearing class in Sindia, despite being the densest in this borrow pit according to the present studies.

However, to ensure a long-term bearing capacity, pavement stability is essential. Since the mineralogy of materials does not change within the same borrow pit [48] pavement material stability depends largely on bearing capacity and MDD.

Based on the results presented in this study, the 0/31.5 mm class could be a better alternative, offering higher quality and being better suited for the optimal use of lateritic gravel, which often contains particles greater than 20 mm in diameter. Before making any decisions, it would be interesting to compare these two classes.

In accordance with AGEROUTE requirements [42], which have been improved here, the analyzed materials are lateritic gravels, types GLO, GL1, GL2 and GL3. Therefore, they can be used in different layers during pavement construction.

4.4.4. Proposals for a Definition Review of Lateritic Gravels

The gravel content of the studied samples exceeds 50% of the particle weight, with values not widely dispersed (**Table 2**).

Therefore, as a contribution to previous studies [3] [51] [61] [62], we can define lateritic gravel soils as consisting of more than 40% gravel, less than 30% sand, and less than 30% fines.

Road contractors in Senegal do not select materials with a CBR between 5 and 10%, as considered in the original CEBTP table [43]. Currently, a road material with a CBR ≥ 20% at 95% of the OCE is considered acceptable. Consequently, as part of this study, the authors revised the reference data to account for the experience of Senegalese road contractors and the lower thresholds encountered in lateritic soils.

They proposed two classes, GL0 and GL3, which are defined in **Table 7**, to expand the CEBTP [43] classification and incorporate the classes recommended

Table 7. Level and classes of use in road geotechnics of raw lateritic gravel soils (CEBTP and AGEROUTE classes ([42] [43], revised).

Soaked CBR (%) at 95 % of the OCE	Level of use of materials	Class
10 ≤ CBR <30	Select fill and subgrade for all types of traffic	GL0
30 ≤ CBR <60	Subbase for all types of traffic	GL1
60 ≤ CBR < 80	Base course, for low traffic	GL2
CBR ≥ 80	Base course, for moderate or heavy traffic	GL3

in the AGEROUTE catalog [42]. Including these proposed classes allows engineers to account for materials with a SCBR of less than 30%, which can be used for embankments and subgrades. Materials with a SCBR of at least 80% could also be added and considered usable without stabilization in the subbase course of a flexible pavement with low traffic. These proposals have been applied to the results of the present study, as shown in Figure 15.

4.4.5. Optimal Use of 0/20 and 0/31.5 mm Granular Classes of Lateritic Gravel in Pavements

The presence of fines is quite noticeable in the 0/20 and 0/31.5 mm samples, but is beneficial as long as the percentage remains below the usual threshold of 35%, and ideally below 25%. The materials LL1, LL3, MR1, PT1, PT3, NG1, and SD3 have fines percentages between 5 and 15%, corresponding to gravelly soils with low fines content [25]. The MR3, NG3, and SD1 samples have fines percentages between 15 and 35%, corresponding to gravelly soils rich in fines (Table 2 and Figure 12).

Samples PT1, PT3, and SD1 are moderately plastic (5 ≤ PI < 15). The remaining 22 samples are plastic (15 ≤ PI < 40). The clay content of all samples in these two soil classes is moderate (12 ≤ PI < 22). They correspond primarily to water-insensitive soils (0 ≤ MBM ≤ 1), except samples SD1 and MR3, which correspond to low-water-sensitive soils (1 ≤ MBM ≤ 2).

The MDDs of samples MR3, NG1, and NG3 are suitable for base courses (MDD ≥ 20 kN/m^3). Conversely, classes LL1, LL3, MR1, SD1, and SD3 have MDDs suitable for subbase courses (18 ≤ MDD < 20 kN/m^3).

The CBRs of the 0/20 and 0/31.5 mm granular classes, at 95% of the OCE, are quite comparable for the different borrow materials, with similar conditions of use in road construction (Table 5). Based on the results, the studied lateritic

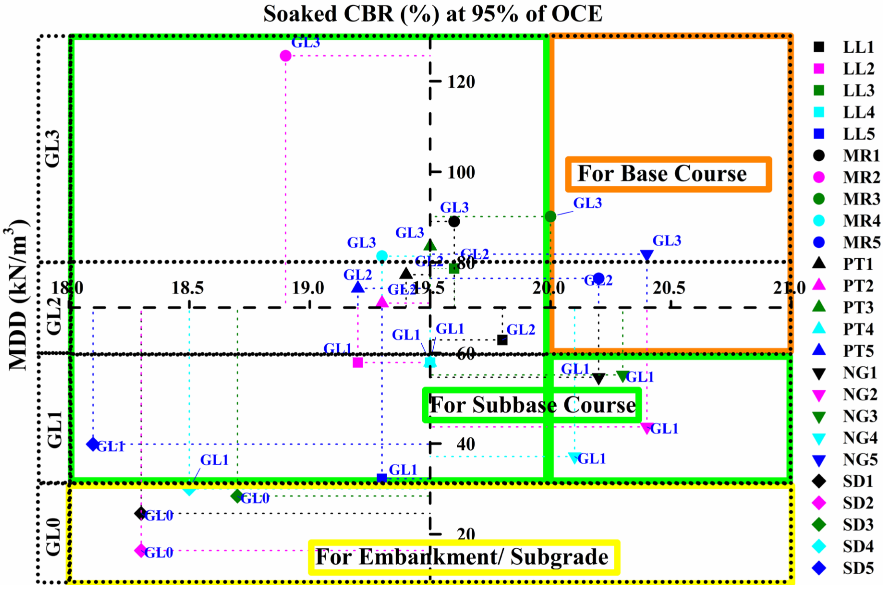

Figure 15. Use of lateritic gravel from the Thiès region in road construction.

gravels can be used for pavement, select fill, subbase, and base layers depending on their geotechnical characteristics, particularly their CBR (**Figure 15** and **Table 5**).

- SD1 and SD3 can be used as subgrade or embankment layers (Su/Ek) for all traffic (CBR ≥ 20%).
- NG1 and NG3 can be used in a subbase course (Su) for all traffic (CBR ≥30%).
- LL1 and LL3 can be used in a base course (BT1) for low-traffic areas (CBR ≥ 60%).
- PT1 can be used in a base course (BT1) for low-traffic areas (CBR ≥ 60%), while PT3 is suitable for a base course (B) in areas with moderate or heavy traffic (CBR ≥ 80%).
- MR1 and MR3 can be used in a base course (B) for moderate or heavy traffic (CBR ≥ 80%).

Based on their geotechnical characteristics, all of the analyzed lateritic gravel samples are suitable for use in subgrade, embankment, foundation, base, and subbase pavement layers (see **Table 7**).

According to the main experimental results recorded in **Table 5**, the 0/31.5 mm class can replace the standard 0/20 mm class for geotechnical laboratory tests that characterize the intrinsic and extrinsic properties of lateritic gravel for road construction.

In short, to optimize the use of lateritic gravels from the Thiès region in pavement, it is necessary to consider the relationships between relevant eligibility parameters of these materials, as well as their sensitivity to water and granulometric variability.

The 0/20 mm class, which is standardized, often has satisfactory parameters, particularly in terms of water sensitivity and gradation. However, it did not offer the highest load-bearing capacity of the five borrow pit materials analyzed. Except for the Lam-Lam borrow pit, it also did not have the highest MDDs.

With this in mind, research efforts will focus on using a granular class other than the 0/20 mm fraction to optimize the use of lateritic materials.

On the basis of the various results presented in this study, the 0/31.5mm class could be an alternative class offering better quality and better suited to the optimum use of lateritic gravels, which often have particles with diameters in excess of 20 mm.

Considering the behavior in the classification, in accordance with the requirements of AGEROUTE [42], the materials analyzed are lateritic gravels, type GL1 and GL2, with SCBR of over 30% and 60% respectively, in addition to other samples, of lower bearing capacity, with a SCBR of over 20% and usable in select fill and subgrade.

From a statistical point of view, the 0/31.5 mm granular class has the most stable mechanical characteristics and is the most relevant for use in base and subbase courses to prevent the early deterioration of pavements. Therefore, it is rational to use classes with the best densities and acceptable bearing capacities, particularly the 0/31.5 mm granular class, as raw lateritic material for flexible pavement subbases or stabilized materials for the base course of semi-rigid pavement. The other classes, which have inconsistent or inferior mechanical properties, can be used as select fill and subgrade layers to ensure road durability, optimize the use of lateritic gravel, and comply with specifications for pavement layers in tropical countries.

Therefore, the Modified Proctor test and the CBR test can be performed on 0/31.5 mm samples of lateritic gravel without modifying the experimental devices or standardized operating instructions. However, materials from all analyzed classes can be used in road geotechnics. To meet technical requirements, a combination of criteria is necessary, rather than just a single criterion.

5. Conclusions

The usefulness of road materials depends on two types of parameters: intrinsic, or those inherent to the material itself, and extrinsic, or those related to its behavior. The gravel lateritic samples exhibit fairly variable characteristics in terms of both nature and behavior. These characteristics vary not only between the five borrow pits at Lam-Lam, Mont Rolland, Pout, Ngoundiane, and Sindia, but also within each specified grain size class: 0/20, 0/25, 0/31.5, 0/40, and 0/50 mm. The nature of the lateritic soils in the 25 samples studied is confirmed as gravelly. The 0/2 mm granular fraction, including the sands, silts, and clays of the lateritic gravels, has a sandy, sandy loam, or loamy texture. The materials exhibit well-spread, discontinuous grading curves and compaction did not alter the gravelly texture of the samples. Therefore, as a contribution to previous studies, lateritic gravel soils can now be defined as composed of more than 40% gravel, less than 30% sand, and less than 30% fines. The studied lateritic gravel samples are moderately plastic to plastic and low to moderately clayey. They are also slightly sensitive to water. The samples fall into GTR 2023 classes G3 and I2, which correspond to gravelly (G) and intermediate (I) soils. These classes are equivalent to the ASTM GM (silty gravel) and GW-GM (well-graded gravel with silt) classes.

The results of the modified Proctor compaction and CBR design tests showed slight variations in the maximum dry density of the optimal compaction energy. CBRs vary significantly depending on granularity. These materials have suitable compaction characteristics for use in pavement layers.

To improve the CEBTP classification and incorporate the classes recommended in the AGEROUTE catalog, the GL0 and GL3 classes are defined in this study. The CBRs are variable and indicative of lateritic gravels of classes GL0, GL1, GL2, and GL3. These gravels are thus for one or more layers of flexible pavement.

The studied materials are waterproof, low-absorbent, and non-swelling. Therefore, they drain well and are not reactive to the hydrodynamic and chemical aggressiveness of water.

After comparing the 0/20 mm and 0/31.5 mm classes, it was determined that the latter exhibited superior behavioral characteristics. To put that in perspective, using the 0/31.5 mm class instead of the 0/20 mm class yields an average increase of over 10% in soaked CBR at 95% of optimum compaction energy, as well as nearly 1% of maximum dry density. However, to reap these advantages, an average of more than 3% more water is necessary to prepare the materials for optimal conditions. Using the optimal 0/31.5 class of lateritic gravel soils in the base and/or foundation layer of pavement will ensure durability and resistance to rapid deterioration.

6. Recommendation

Lateritic gravel soils may now be defined as consisting of more than 40% gravel and less than 30% each of sand and fines.

In order to classify lateritic gravels for use in select fill, subgrade, and base layers, the AGEROUTE classification system should include the classes proposed in this study: GL0 and GL3. GL0 has a soaked CBR between 10 and 30%, while GL3 has a soaked CBR of greater than 80%.

To optimize the use of lateritic gravel materials and prevent the rapid deterioration of pavements made from them, the 0/20 mm class can be substituted for the 0/31.5 mm class in laboratory without modifying the experimental setups or operating instructions.

Considering the appropriate mineralogy and optimal granularity of lateritic gravel prevents premature deterioration of road pavement and allows the use of materials that have actually been tested in laboratories on construction sites according to their granularity. This approach also enables the optimal use of lateritic materials in different pavement layers for constructing load-bearing, stable, and durable pavements.

Looking ahead, it would be useful to expand this study beyond the composition and bearing capacity of pavement materials to include their geomechanical properties after stabilization, as well as the relationship between their geotechnical and compositional parameters. This would allow for more effective prevention of the rapid deterioration of road pavement.

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Conflicts of Interest

The authors declare no conflicts of interest regarding the publication of this paper.

References

[1] LCPC-SETRA (2000) Réalisation des remblais et des couches de forme. 2nd Edition,

SETRA/LCPC.

[2] AFNOR (1992) NF P11-300: Exécution des terrassements-Classification des matériaux utilisables dans la construction des remblais et des couches de forme d'infrastructures routières. LCPC, 21.

[3] AFNOR (2005) NF EN ISO 14688-2: Principes pour une classification: Dénomintion, description et classification. Principles for soil classification: Naming, description and classification. 13.

[4] Paige-Green, P., Netterberg, F. and Pinard, M.I. (2024) Testing of Lateritic Materials for Road Pavements. 42*nd Southern African Transport Conference*, Pretoria, 8-11 July 2024, 16. https://repository.up.ac.za/server/api/core/bitstreams/2264dd47-0bc7-4884-a3e1-fb1e766436ef/content

[5] Okonkwo, V.O., Omaliko, I.K. and Ezema, N.M. (2022) Stabilization of Lateritic Soil with Portland Cement and Sand for Road Pavement. *OALib*, **9**, 1-15. https://doi.org/10.4236/oalib.1108560

[6] Razali, R., Rashid, A.S.A., Razali, R., *et al.* (2022) Experimental Study on Mechanical Behavior of Laterite Soil Treated with Quicklime. *Journal of Mechanical Engineering*, **11**, 109-122. https://doi.org/10.24191/jmeche.v11i1.23592

[7] Ouedraogo, M., Bamogo, I., Sanou, H., Sanogo, S.S.S., Soré, S., Sanou, S.O., Aubert, J.-E. and Millogo, Y. (2023) Improvement of Geotechnical and Mechanical Properties of Laterite from Burkina Faso Using Sugar Cane Molasses for Use as Road Structural Layers. *Journal de la Société Ouest-Africaine de Chimie*, **52**, 32-43.

[8] Kumar, A. and George, V. (2018) Effect of Soil Parameters on Resilient Modulus Using Cyclic Tri-Axial Tests on Lateritic Subgrade Soils from Dakshina Kannada, India. *Geotechnical and Geological Engineering*, **36**, 3987-4000. https://doi.org/10.1007/s10706-018-0550-7

[9] Nagaraju, T.V., Sunil, B.M. and Rao, M.V. (2023) Utilization of GGBS-Based Geopolymer Lateritic Soils for Sustainable Pavements. In: Sil, A., N. Kontoni, DP. and Pancharathi, R.K., Eds., *Lecture Notes in Civil Engineering*, Springer, 429-439. https://doi.org/10.1007/978-981-19-4055-2_34

[10] Nagaraju, T.V., Bahrami, A., Prasad, C.D., Mantena, S., Biswal, M. and Islam, M.R. (2023) Predicting California Bearing Ratio of Lateritic Soils Using Hybrid Machine Learning Technique. *Buildings*, **13**, Article 255. https://doi.org/10.3390/buildings13010255

[11] Paige-Green, P. and Van Zyl, G.D. (2019) A Review of the DCP-DN Pavement Design Method for Low Volume Sealed Roads: Development and Applications. *Journal of Transportation Technologies*, **9**, 397-422. https://doi.org/10.4236/jtts.2019.94025

[12] FALL, M., Sarr, D., Ba, M., Berbinau, E., Borel, J., Ndiaye, M., *et al.* (2011) Evolution of Lateritic Soils Geotechnical Parameters during a Multi-Cyclic OPM Compaction and Correlation with Road Traffic. *Geomaterials*, **1**, 59-69. https://doi.org/10.4236/gm.2011.13010

[13] Fall, M., Ba, S., Sarr, D., Ba, M. and Ndiaye, M. (2011) An Alternative Method to the West African Compaction (WAC) Test Procedure. *Geomaterials*, **1**, 25-27. https://doi.org/10.4236/gm.2011.12004

[14] Odier, L., Millard, R.S., Pimentel, D.-S. and Mehra, S.R. (1968) Routes dans les pays en voie de développement. Éditions EYROLLES.

[15] Roger, J., *et al.* (2009) Notice explicative de la cartographie multicouches à 1/50 000 et 1/20 000 de la zone d'activité du Cap-Vert. DMG, 245.

[16] Autret, P. (1980) Contribution à l'étude des graveleux latéritiques traités au ciment. Ecole National des Ponts et Chaussées (ENPC), 297.

[17] Autret, P. (1983) Latérites et graveleux latéritiques. Institut des Sciences et Techniques de l'Équipement et de l'Environnement pour le Développement (ISTED), 38.

[18] Gidigasu, M.D. (1976) Laterite Soil Engineering-Pedogenesis and Engineering Principles. Developments in Geotechnical Engineering.

[19] Diome, F. (1996) Rôle de la structure du sol dans son fonctionnement hydrique. Sa quan-tification par la courbe de retrait. UCAD.

[20] Diop, B.O., Gbaguidi, I., Lo, P.G., Sène, S., Cisse, A., Ba, M., *et al.* (2025) Compositional Suitability Assessment and New Classification of Lateritic Soils for Road Construction: Case of Materials from the Thies Region in Senegal. *Journal of Geoscience and Environment Protection*, **13**, 71-88. https://doi.org/10.4236/gep.2025.131005

[21] CEBTP (1990) Utilisation des graveleux latéritiques en technique routière. ISTED.

[22] AFNOR (1999) NF P 94-093 Soils: Investigation and Testing—Determination of the Compaction Characteristics of a Soil—Standard Proctor Test—Modified Proctor Test. LCPC, 15.

[23] AFNOR (2012) NF EN 933-1: Détermination de la granularité: Analyse granulométrique par tamisage. 17.

[24] Head, K.H. (2006) Manual of Soil Laboratory Testing. 3e Edition, Whittles Publishing.

[25] IDRRIM (2023) Guide des terrassements des remblais et des couches de forme. Cerema.

[26] AFNOR (1993) NF P 94-051 Soil: Investigation and Testing—Determination of Atterberg's Limits-Liquid Limit Test Using Casagrande Apparatus—Plastic Limit Test on Rolled Thread. LCPC, 14.

[27] AFNOR (1998) NF P 94-068: Sols: Reconnaissance et Essais—Mesure de la quantité d'adsorption de bleu de méthylène d'un sol ou d'un matériau rocheux. Détermination de la valeur de bleu de méthylène d'un sol ou d'un matériau rocheux par l'essai à la tâche. 8.

[28] AFNOR (1999) NF EN 933-9: Quantification des fines—Essai au bleu de méthylène. 11.

[29] AFNOR (2005) NF EN 13286-2: Méthodes d'essai de détermination en laboratoire de la masse volumique de référence et de la teneur en eau—Compactage Proctor. 18.

[30] AFNOR (2004) NF EN 13286-47: Méthode d'essai pour la détermination de l'indice portant Californien (CBR), de l'indice de portance immédiate (IPI) et du gonflement. 11.

[31] Ogunsanwo, O. (1985) The Microfabric and Permeability of a Laterite Clay. *First International Conference on Geomechanics of Tropical Lateritic and Saprolitic Soils*, **1**, 397-402.

[32] AFNOR (2005) NF P 94-512-11: Reconnaissance et essais géotechniques—Essais de la-boratoire sur les sols—Partie 1: Détermination de la perméabilité au perméamètre à charge constante ou variable. 26.

[33] Saadi, L.A. (2003) Méthodologie de contrôle de l'homogénéité et de la perméabilité des barrières argileuses. Paris VI.

[34] Casagrande, A. (1948) Classification and Identification of Soils. *American Society of Civil Engineers*, **113**, 783-810.

[35] Skempton, A.W. (1953) The Colloidal Activity of Clays. 3*th International Conference*

of Soil Mechanics and Foundation Engineering, Switzerland, 16-27 August 1953, 57-61.

[36] Gomez-Gutierrez, I.C., Bryson, L.S. and Hopkins, T.C. (2011) Correlations between Geotechnical Properties and the Swell Behavior of Compacted Shales. *Geo-Frontiers* 2011, Dallas, 13 Mar 2011, 4119-4128. https://doi.org/10.1061/41165(397)421

[37] Mvondo-Ondoa, J. (1979) Rôle de la granularité et de la composition minéralogique sur la compactibilité et la résistance au cisaillement des mélanges de sable et d'argiles. Paris VI.

[38] Reiffsteck Ph., and Nguyen Pham P.T., (2005) Influence de la répartition granulométrique sur le comportement mécanique d'un sol. In: Japanese Geotechnical Society, Ed., *Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering*, IOS Press, 583-586. https://doi.org/10.3233/978-1-61499-656-9-583

[39] Diop, B.O., Gbaguidi, I., Lo, P.G., Cisse, A., Sène, S. and Ba, M. (2018) Grain Size Influence on the Compaction Aptitude and the Bearing Strength of the Gravel Lateritic Soils. *Geomaterials*, **8**, 63-76. https://doi.org/10.4236/gm.2018.84005

[40] Cisse, A., Lo, P.G., Diaw, E.B., Sene, S., Diagne, M., Gbaguidi, I., *et al.* (2016) Contribution to Improving the Quality of Road Base Layers: Concept of Optimum Maximum Diameterand Its Application to the Laterite of Sindia (Senegal). *Research Journal of Environmental and Earth Sciences*, **8**, 1-7. https://doi.org/10.19026/rjees.8.2696

[41] Guilloux, A. (1978) Influence de la pétrographie sur la compactabilité et la portance des sables fins traités (Région Parisienne). Université Pierre et Marie Curie, 133.

[42] AGEROUTE (2015) Catalogue de structures de chaussées neuves et Guide de dimension-nement des chaussées au Sénégal. MITTD.

[43] CEBTP (1984) Guide pratique de dimensionnement des chaussées pour les pays tropicaux, 2nd Edition, La documentation française.

[44] LCPC-SETRA (1998) Catalogue des structures types de chaussées neuves. SETRA. https://doc.cerema.fr/Default/doc/SYRACUSE/13990/catalogue-des-structures-types-de-chaussees-neuves

[45] Terzaghi, K., Peck, R.B. and Mesri, G. (1996) Soil Mechanics in Engineering Practice. 3rd Edition, John Wiley & Sons, Inc.

[46] Van Ganse, R. (1957) Proprietes et applications des laterites au Congo beige. Académie Coloniale.

[47] Hubert, B., Philipponnat, B., Payant, O. and Zerhouni, M. (2019) Fondations et ouvrages en terre. Éditions EYROLLES.

[48] Diop, B.O., Talla, K., Sylla, N.F., Ndiaye, N.M. and Ngom, B.D. (2023) Physical Analyzes Applied to Lateritic Soils of Western Senegal for Their Geochemical, Mineralogical and Morphological Identification. *MRS Advances*, **8**, 607-612. https://doi.org/10.1557/s43580-023-00591-5

[49] Brewer, R. (1964) Fabric and Mineral Analysis. John Wiley & Sons, Inc.

[50] Jamagne, M. (1967) Bases et techniques d'une cartographie des sols. *Annales Agronomiques*, **18**, Article 142.

[51] Folk, R.L. (1954) The Distinction between Grain Size and Mineral Composition in Sedimentary-Rock Nomenclature. *The Journal of Geology*, **62**, 344-359. https://doi.org/10.1086/626171

[52] Chaulagai, R., Osouli, A., Salam, S., Tutumluer, E., Beshears, S., Shoup, H., *et al.*

(2017) Influence of Maximum Particle Size, Fines Content, and Dust Ratio on the Behavior of Base and Subbase Coarse Aggregates. *Transportation Research Record: Journal of the Transportation Research Board*, **2655**, 20-26. https://doi.org/10.3141/2655-04

[53] Berthaud, Y., De-Buhan, P. and Schmitt, N. (2018) Aide mémoire mécanique des sols, 3e Edition, DUNOD.

[54] Otçu, N.Ü., Uzundurukan, S. and Kaplan, G. (2017) Determination of the Plasticity Index of Soils with Fine-Grained Soils Using Methylene Blue Test. *Journal of Geoscience and Environment Protection*, **5**, 165-181. https://doi.org/10.4236/gep.2017.53012

[55] ASTM (2000) Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), D 2487-0. ASTM International.

[56] AFNOR (1993) NF P 98-241-1: Essais relatifs aux chaussées—Mesure de la masse volumique des matériaux en place—Partie 1: Mesure ponctuelle de la masse volumique moyenne apparente par gammadensimètre à transmission directe. LCPC, 11.

[57] AFNOR (1996) NF P 94-061-2: Détermination de la masse volumique d'un matériau en place. LCPC, 9.

[58] Ganoumi, M.E. (2010) Eau dans le sol. Hassan II.

[59] SETRA (2006) Comprendre les principaux paramètres de conception géométrique des routes.

[60] Casagrande, A. and Fadum, R.E. (1940) Notes on Soil Testing for Engineering Purposes. Harvard University, 74.

[61] Millogo, Y. (2008) Étude géotechnique, chimique et minéralogique de matières premières argileuses et latéritiques du Burkina Faso améliorées aux liants hydrauliques: Application au génie civil (bâtiment et route). Université de Ouagadougou.

[62] Bagarre, E. (1990) Utilisation des graveleux latéritiques en technique routière. IFSTTAR.