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Geotechnical and Mechanical Characterization of Lateritic Soil Improved with Crushed Granite

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Abstract

Since many years, road infrastructures in West Africa are most often subject to premature degradations despite the large number of studies. This problem is often due to the poor control of the behaviour of materials used for the pavement, but also to the scarcity of good quality materials. Nowadays, with economic development, there is a necessity for road infrastructures of good quality. In this framework, the main objective was to study the vertical geotechnical variability of the gravelly lateritic soil from the Saaba site in Burkina Faso and to improve their performances by adding crushed granite. The results show that the physical properties of the soils are almost identical depending on the depth. However, a small difference in the mechanical properties was observed. Due to their poor characteristics, these materials cannot be used for the sub-base layer according to the pavement design guide for tropical countries, CEBTP [1]. In order to improve their geotechnical and mechanical characteristics, crushed granite of class 0/31.5 mm was added at different percentages: 20, 25, 30, and 35%. It appears that the plasticity index, the methylene blue value, as well as the optimal water content of the material decreased. The soaked CBR recorded a maximum relative increase of 164% (from 14 to 37%) with the addition of 20 to 30% of crushed granite. With the addition of 20 to 30% of crushed granite, Young's modulus and unconfined compressive strength also showed a clear increase of 309% (from 80 to 327 MPa) and 140% (from 0.72 to 1.73 MPa). By comparing the results with the CEBTP specifications, the addition of 30% of granites at 95% compactness allows the materials to have a CBR that exceeds the value of 30% and can be used in the sub-base layer of road pavement. The addition of 30% granite allows the materials to record an unconfined compressive strength higher than 0.5-1.5 MPa, which corresponds to lateritic soil suitable for sub-base layer according to Messou [2]. After the addition of 30% granite, the materials record a Young's modulus greater than 300 MPa and can be used as a base layer. The assessment of the improvement of mechanical performance simultaneously based on the CBR, the Young's modulus, and the compressive strength showed the contradictory evolution of the results from these different parameters. A discussion was made on the relationship between these parameters.

Keywords: Litho Stabilization; Lateritic Soil; Geotechnical Properties; Pavement Structure; Vertical Geotechnical Variability.

1. Introduction

Lateritic soils are very common in sub-Saharan Africa. They are easy to extract and have a relatively low operating cost. Therefore, they are commonly used in construction, especially in road construction. The borrow pits of this material are superficial and show large variability depending on the localization and the depth. In fact, the formation of this soil

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results from complex weathering processes specific to hot and humid tropical regions. However, the usage of these materials is subjected to meeting certain criteria defined by existing technical documents. The CEBTP [1] recommended their use in road construction when they meet the required criteria, such as plasticity index which should be lesser than 12 for a use as base layer, CBR index which must be greater than 80% for a use as base layer.

Most of the lateritic borrow pits meeting these criteria have been widely exploited in recent years. This results in the scarcity of lateritic soils with good geotechnical characteristics. Moreover, these lateritic materials, even if they meet all standardized geotechnical and mechanical tests at the time of pavement construction, may not be durable due to the deterioration of the materials through further chemical weathering in situ [3, 4]. Improvement techniques have been introduced in order to improve the geotechnical characteristics of these soils, such as chemical stabilization by adding hydraulic binders. In this context, many studies have been conducted, such as those of Autret [5], Millogo et al. [6], Boumekik et al. [7], Consoli et al. [8], Oyediran and Ayeni [9]. This method allows obtaining lateritic soil with very good performance, but it is very expensive and increases the execution time of the work on the sites. In addition, the use of cement induces environmental pollution. In fact, cement is not only an imported material in some countries, but also its production is responsible for 5–7% of global emissions of carbon dioxide (CO_2), and CO_2 makes up 65% of greenhouse gases. Therefore, the use of local materials should be strongly promoted in order to attenuate global change.

In Burkina Faso, alongside the widely present lateritic soils, there is the existence of aggregates from igneous rocks such as granite. Contrary to cement, which is imported from abroad, these materials are very common in this country. In this context, the combination of these two materials would be an adequate solution. This technique is called litho-stabilization. There are some works in the literature that have been done using this technique [10–19]. The stabilization of lateritic soil by the addition of crushed aggregates was applied in Burkina Faso by Lompo [10] on the road section of Ouagadougou-Yako and by Toé [11] on the road section of Ouagadougou-koupèla. Toé [11] reported a relatively slight increase in the CBR of 15% (from 64% to 73%) of lateritic material after the addition of 30% crushed granites of class 0/25 mm. The bearing capacity of the mixing in situ increased by 92% (from 75 to 39). He has shown that stabilization has a more significant effect on the in situ bearing capacity of the lateritic soil.

Ndiaye et al. [12] studied the effect of the addition of dune sand to lateritic soils from Senegal. The aim was to reduce the plasticity index of the lateritic soil while maintaining the value of the CBR index and verifying that the grain size curve remains within the specified range for use as a sub-base layer or base layer of a road at low traffic. Moreover, they found that the addition of 10% dune sand to lateritic soils results in a 20% decrease in the plasticity index and the increase of 21% of the CBR index. In addition, the study [12] reported the modification of the particle size distribution. Jjuuko et al. [13] investigated the addition of crushed rocks (10 to 50%) of class 0/37.5 mm to the lateritic soils of Uganda. The study showed that the maximum dry density and optimum moisture content of the resulting mixtures respectively increased and decreased with the higher content of aggregate. They found that an addition of 50% achieves a CBR value greater than 80%, thus allowing the use of this material as a base layer. In addition, they showed that the use of this stabilization method is less expensive by 26.1% compared to stabilization using lime. Sudla et al. [14] investigated the physical and mechanical properties of marginal lateritic soil and crushed slag blends at various replacement contents to evaluate them as an engineering fill material. After the addition of 50% of crushed slags, the maximum dry density and the CBR significantly increased from 2.16 t/m^3 to 2.43 t/m^3 and 19% to 45%, respectively. The author found that with a minimum of 10% crushed slag replacement content, the physical and mechanical properties of mixtures meet the requirements for engineering fill materials (PI less than 20%; CBR greater than 10%). However, this study did not assess the unconfined compression nor the Young's modulus.

Hyoumbi et al. [15] conducted a study on improving the physical and mechanical properties of Bafang laterite in Cameroon by adding basanite crushed aggregate 0/5. The authors found that the percentage of fine, the plasticity index, the methylene blue value, as well as the optimum water content decreased with the increasing content of crushed aggregate. The CBR and the unconfined compressive strength, respectively, increased by 105% (from 22% to 45%) and 135% (from 1.7 MPa to 4 MPa) with the addition of 50% of crushed basanite. The addition of 30% aggregate is sufficient to reach the value of CBR of 30% and unconfined compressive strength of 0.5 MPa, which allows the use of lateritic soil as a sub-base layer. Ahouet & Elenga [16] studied the mixture between a lateritic soil from Congo and crushed alluvial gravel of class 0/31.5. This study showed that the CBR increased by about 110% with the addition of 30% of alluvial gravel. The plasticity index of the mixture passed from 18.8% for the raw material to 0% after the addition of 40% of alluvial gravel. Corrêa et al. [17] evaluated the viability of using the fine residue of quartzite mining tailings and stone powder from dunite mining in Brazil as aggregates mixed with lateritic clayey soil for the treatment of base and sub-base layers. The fine fraction of the quartzite and dunite powder resulted in a reduction of the optimum water content from 26.6% to 16.5% and an increase in the apparent maximum dry density, which varies from 1.51 t/m^3 to 1.77 t/m^3 after the addition of 50% fine residue of quartzite mining tailings and stone powder to the raw material. The use of appropriate proportions of these materials (60% soil + 40% quartzite and 50% soil + 50% dunite powder) allowed the use of lateritic soils as a base or sub-base layer according to the criteria of the MCT (Miniature, compacted tropical) methodology from Brazil. However, this study did not include the mechanical test.

Tony et al. [18] studied the effects of the addition of quarry dust on the properties of lateritic soil mixes from India. They found that the optimum proportion of quarry dust on the properties of lateritic soil mixes was 40% and 60%, respectively. The optimum dry density showed an increase of 11.61%, while the shear properties decreased with the addition of quarry dust. However, the effect of stabilization with the addition of crushed materials on the geo-mechanical behaviour of laterite is still less studied. Gidigasu [19] showed that the addition of crushed waste rock and spent carbide to poor lateritic soil significantly improved its geotechnical characteristics. Based on the CBR and grading specifications, the content of 40% waste of crushed rock with 7% spent carbide is recommended as the optimum to stabilize poor lateritic soil for the base structure of the road despite its slightly high Atterberg's limits and CBR swell.

Most of these studies on litho stabilization rarely assessed the impact of the addition of crushed stone on mechanical parameters such as compressive strength and Young's modulus. Indeed, the impact of the addition of crushed stone was mostly evaluated based on the parameters of the soil identification, such as the plastic limit, the compaction parameters, and the CBR. However, the Young's modulus is the main parameter that allows the design of pavement structures in tropical Africa. This Young's modulus, E, is often estimated from the empirical relation given by the CEBTP guide, E = 5*CBR.

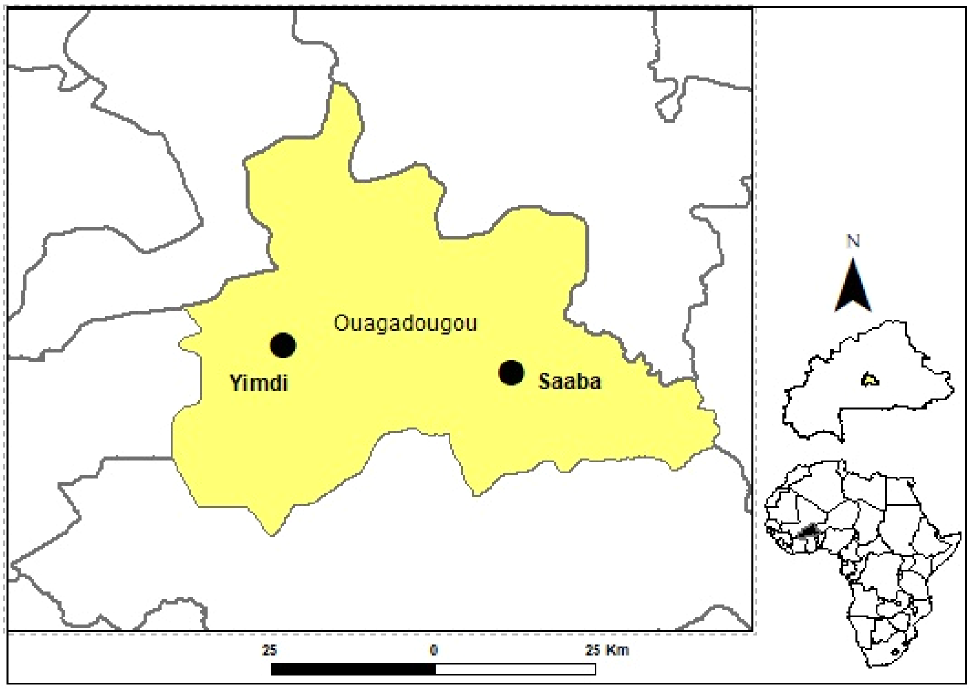
Therefore, the purpose of this work is to study the geotechnical variability of the Saaba borrow pit depending on the sampling depth on the one hand. On the other hand, it studies the effect of the addition of crushed granites of class 0/31.5 on the geotechnical and mechanical parameters, especially the CBR, the Young's modulus, and the unconfined compressive strength of the lateritic soils of Saaba.

2. Materials and Methods

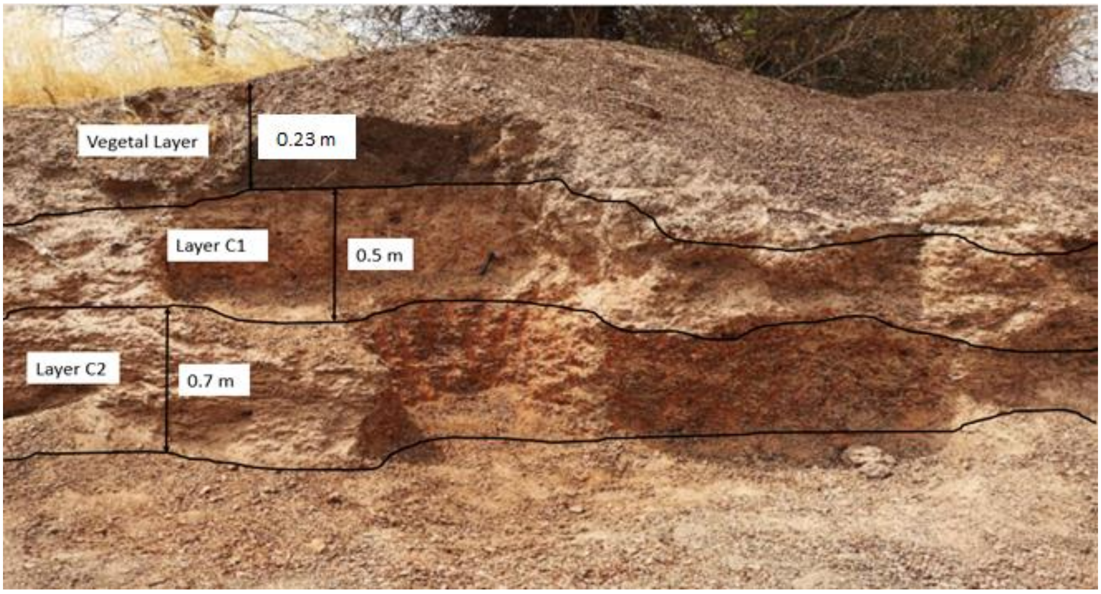
2.1. Materials

The lateritic soil was collected from the borrow pit of Saaba in the region of Ouagadougou (Burkina Faso) in the province of Kadiogo (12° 16 '37.5'' North and 1° 21' 12.1'' West), as located on Figure 1-a. The surface area of the site is around 0.3 km². The landscape is typical of the central plateau of Burkina Faso, generally flat with Sahelian-Sudanese vegetation. The altitude is around 308 m with rendering bedrock sparse outcrops. The geological map of the region indicates that the bedrock is constituted from Paleoproterozoic granitoids of the Leo shield and is covered with yellow to red acidic soils. This location does not ensure that superficial soils are genetically linked to the underlying bedrock. Indeed, the lateritic cover can be highly subjected to remobilization. There is the presence of colluvial deposition or glacis that may conceal the underlying bedrock or the in-situ weathering profile developed from the bedrock.

The superficial soils are lightly cemented gravels with sands and fine fraction, characterizing the lateritic soils. The cementation is soft, forming an endured layer, easily destroyed to give loose materials. The geological profile, observed in situ was subdivided into two quite distinct lateritic layers (denoted C1 for the top layer and C2 for the under layer) after stripping of the vegetal layer of about 0.23 m (Figure 1-b). This distinction between the different layers was made on the basis of observation of the texture, color and structure of the materials.



(a)



(b)

Figure 1. a- Location of Saaba and Yimdi quarry in the region of Ouagadougou, b- lithographic profile of the lateritic soil from Saaba

The first layer is about 0.5 m thick. Its particles are reddish gray (5YR5/2) [20] in color and crumbly when crushed by hands. The second layer is about 0.7 m thick. Its particles are reddish yellow (5YR7/6) [20] and not very friable. Two different materials were separately sampled from the first layer (C1) and the second layer (C2), at different points in each layer, using a pick and a shovel. Several quantities of the materials were collected and stored in respective bags. These samples were separately used to study the lateritic profiles (Figure 1). The third material (CM) was obtained by mixing the material of C1 and C2 layers.

The crushed granite aggregates were collected from the Yimdi quarry, located in the outskirts of Ouagadougou (12°18'55.4'' North and 1°40'12.749'' West), as located on Figure 1-a. The granites were supplied by a local company after crushing and screening. The crushed granites have grain size which varies between 80 µm to 31.5 mm and angular shape. The quality of the aggregates was assessed by some tests, such as the particle size distribution, Los Angeles, sand equivalent tests and specific density (Table 1).

Table 1 . Properties of the crushed granite 0/31.5

	Effective Particle size (mm)			% Passing on 80 µm	Uniformity Coefficient	Curvature coefficient	Specific density (t/m^3)	SE (%)	LA (%)
	d_{10}	d_{30}	d_{60}						
Crushed Granite 0/31.5	0.4	6.3	16	6.9	40	6.2	2.9	73	25

PI: Plasticity Index, SE: Sand Equivalent, LA: Coefficient of Los Angeles

2.2. Experimental Methods

The tests were carried out to characterize the lateritic materials, from the two layers and their mixture (C1, C2 and CM), and the crushed granite aggregates. The tests were also carried out to characterize the mixtures of lateritic materials and crushed granites. The mix design, with respect to the dry mass, consisted of 80% lateritic materials (GL) and 20% granites (G) (80% GL + 20% G), and similarly 75% GL + 25% G, 70% GL + 30% G, 65% GL + 35% G. These mixtures were made for each of the materials from C1, C2 and CM. Figure 2 details the mix design.

2.3. Tests of the Physicals and Compaction Properties

The particle size analysis was carried out according to the French standard NF EN ISO 17892-4 [21] on soils with a diameter greater than 80 µm in order to determine the percentage by weight of passing materials according to their diameter. The tests on the Atterberg limits were carried out on materials passing through a 0.4 mm sieve in accordance with standard NF EN ISO 17892-12 [22]. The particle size distribution and the Atterberg limits tests were carried out on lateritic materials alone and on the mixtures of crushed granite and the lateritic materials. The methylene blue tests were carried out on the lateritic materials passing on the 5 mm sieve and the 80 µm sieve, in order to better understand the clay activity of these materials, in accordance with the standard NF P94-068 [23]. The specific density was determined according to the method prescribed by the standard ASTM D5550 [24] for gravels materials. The Proctor compaction test allowed to determine the maximum dry density (MDD) and optimum water content (OWC) of a material in accordance with standard NF P94 -093 [25]. The test was carried out on materials passing on 20 mm sieve. The materials were compacted in five layers at the rate of 56 compaction strokes per layer in a CBR (Californian bearing ratio) mold, in accordance with standard NF P94 -093 [25]. The compaction and the methylene blue tests were carried out on the lateritic materials and the mixture of the lateritic materials and the crushed granite.

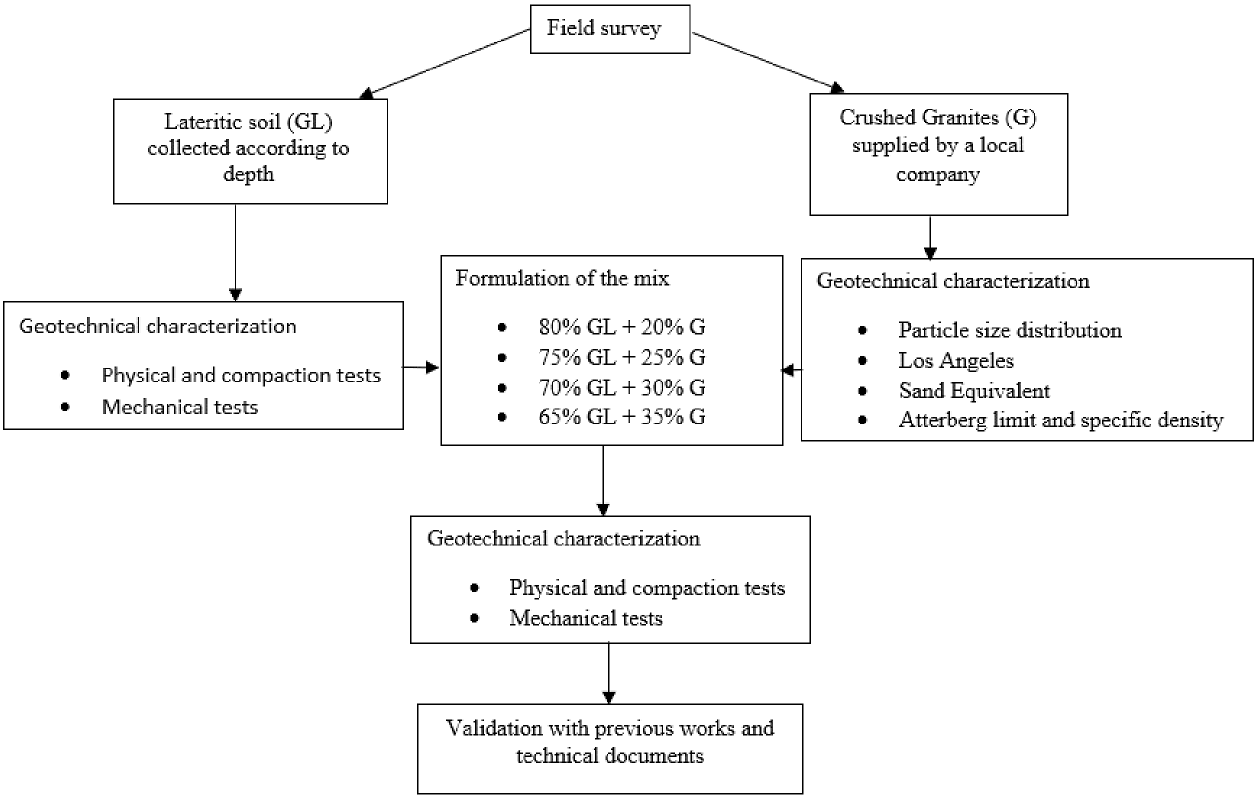


Figure 2. Experimental method

2.4. Tests of the Mechanical Properties

The CBR tests were carried out on the lateritic materials and on the materials mixed with crushed granites, referring to the standard NFP94-078 [26]. The tests were carried out on materials passing on the 20 mm sieve. Two types of CBR were determined, namely the immediate CBR index (IPI), in order to assess the bearing capacity of the soil during the work phase, and the CBR index after four days of immersion in water, in order to assess its bearing capacity in the long term. In addition, the values of CBR were determined at different levels of compaction energy; i.e., at 95% and 98% of the maximum dry density (section 2.2) for both CBR indices. This test consists of measuring the forces to be applied to a cylindrical punch to make it penetrate, at constant speed, into a test piece of material compacted at the desired energy. The values of the forces that caused the two conventional deformations (2.5 mm and 5 mm) on the test materials are respectively related to the values of the forces which caused the same deformations on the reference material.

The compression tests were carried out on the materials from C1, C2 and CM and on lateritic materials mixed with crushed granites. The test specimens (the height of 32 cm and the diameter of 16 cm) were compacted at different energies: 90%, 95% of the maximum dry density of laterite soils or others and three specimens for each sample. After the compaction, all test specimens were placed in airtight bags and kept in a closed chamber for 28 days. The compression tests allowed to determine the values of the compressive strength (Rc) of materials, as well as their modulus of elasticity (E). These values are most often used to assess the improvement of the mechanical performances of materials. The tests of the compressive strength and modulus of elasticity were respectively carried out referring to the standards NF EN 13286-41 [27] and NF EN 13286-43 [28]. The compressive strength, Rc (MPa), was determined using Equation 1; where, Fmax (N) is the maximum applied compression force at failure and S (mm^2) is the cross-sectional area of the test piece. The elastic modulus, E (MPa), was determined using Equation 2; where, D (mm) is the diameter of the test piece and ε is the longitudinal deformation of the test piece when the applied force, F = 0.3xFmax.

$$R_c = \frac{F_{max}}{S} \tag{1}$$

$$E = \frac{1.2 * F_{max}}{\pi * D^2 * \varepsilon} \tag{2}$$

3. Results and Discussions

3.1. Physical and Compaction Properties of the Lateritic Soils Before and After the Addition of Aggregates

The grain size distribution and the main characteristics of the three lateritic materials (C1, C2 and CM) are shown respectively in Figure 3 and Table 2. The three lateritic materials are all of class A-2-6 according to HRB [29] classification and B6 according to GTR [30] classification. The three materials have gravelly texture with fine fractions. The fine contents (<0.08 mm) of the materials from C1 and C2 are practically similar around 30%. The specific density is 2.73, 2.67 and 2.75 for C1, C2 and CM respectively. Their plasticity index (PI) (17.9% for C1, for 21.6% and 21.4 for CM) and their liquidity limit (LL) (33% for all the materials) show that the fine fraction is inorganic clay of medium plasticity, according to Casagrande diagram [31]. These results are confirmed by the results of the methylene blue tests (on the two fractions: passing on 5 mm and 80 µm sieve) which show the values between 0.2 and 1.5, hence the fine fraction soils, are sensitive to water: it has a siltier than clayey behavior according to GTR [30].

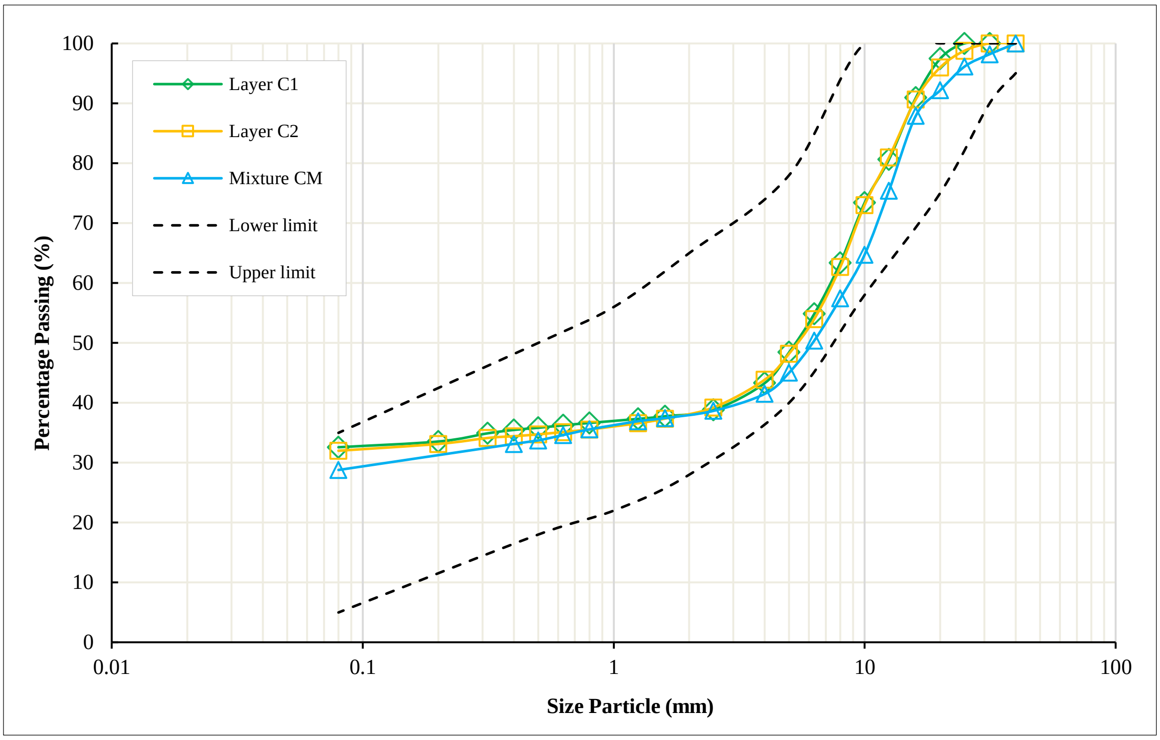


Figure 3. Grain size distribution of the three lateritic soils (C1, C2 and CM) and comparison with the CEBTP [1] criteria for the application in sub base layer

Table 2. Physical and compaction properties of lateritic soil

	% passing on diameter, d			Plasticity and clay content				Compaction characteristics			Specific density (t/m^3)	Classification	
	d<80 µm	d<2 mm	d<0.4 mm	LL (%)	PI (%)	MBV (d<5 mm)	MBV (d<80 µm)	OWC (%)	MDD (t/m^3)	CBR		GTR	HRB
C1	32.6	38.3	35.5	33	17.9	0.9	1.02	11.2	2.02	11	2.73	B6	A-2-6
C2	32	38.3	34.4	31	21.6	0.63	0.77	10.9	2.03	16	2.67	B6	A-2-6
CM	29	38	33.1	32	21.4	0.7	0.83	10.4	2.06	17	2.75	B6	A-2-6

d: diameter (mm), LL: Liquid Limit, PI: Plastic Index, MBV: Methylene Blue Value, OWC: Optimum water content, MDD: Maximum Dry Density, CBR: Californian Bearing ratio after 4 days soaked in water, HRB: American standard Highway Research Board, GTR: French guide for road earthworks.

The maximum dry density (MDD) is 2.03, 2.03 and 2.06 t/m^3, the optimum water content (OWC) is 11.9, 10.4, and 10.9% respectively for C1, C2 and CM. The MDD is greater than 1.8 t/m^3 which is the minimum value required for use as a pavement layer; while the optimum water content is lower than 13% which is_the maximum acceptable value for use as a pavement layer. In the present case, the materials C1, C2 and CM reached the CBR indices at 95% of compactness far less than 30%; i.e., 11%, 16% and 17%, respectively (Table 2). The soaked CBR index are low and similar for CM and C2. According to the CEBTP [1], the bearing class of C1 is S3; while C2 and CM are of class S4. The high bearing class of C2 and CM compared to C1 can be explained by the methylene blue values and the optimal water content which are 1.02, 0.69, and 0.83, 11.2 %, 10.4 % and 10.9% respectively for C1, C2 and CM. This can be related to the clay content of the materials C2 and CM which is significantly lower than that of C1. The MBV is lower for C2 and CM than for C1, which suggests that they contain lower content clay than C1, therefore less sensitive to water. These values (11% for C1, 16% for CM and 17% for CM) are lower than the value obtained by Millogo et al. [6] which is around 43% for the laterite of Sapouy located in the south of Burkina Faso; and by Ki et al. [32] which are 65% and 58 % respectively for the laterite of Dedougou and Badnogo in Burkina Faso. The lateritic soils [6, 32] had a lower content of fines (<0.080 mm) < 15% and a plasticity index less than 12, which shows a low content of clay minerals.

According to the CEBTP [1] the three materials C1, C2, CM are similar, despite the apparent difference from in situ observations. The three materials meet several criteria for use in sub-base layer. Indeed, for a material to be used as a sub-base layer, its grain size curve must fit into the limits proposed by the CEBTP (Figure 3), its plasticity index must be less than 25% and its density greater than 1.8 t/m^3 (Table 2). However, there is one criterion that these materials do not meet, namely the CBR index, which should be greater than 30. These materials cannot be used in sub-base layer in their current state.

The particle size distribution of the crushed granite aggregates shows that the fines (<80 µm) content is 6.9%, which is lower than the maximum allowed value of 10% (Figure 4). The particle size curve fits into the limits proposed by CEBTP [1] or the use in road construction. The sand equivalent gives a value of 73% which is far greater than the minimum required values of 30% for lower traffic and 40% for medium traffic [1]. The Los Angeles test gives a value of 25% which is less than the maximum accepted value of 30%. The specific density of this material is 2.9 t/m^3 (Table 1). The crushed granites are therefore suitable for use in road construction.

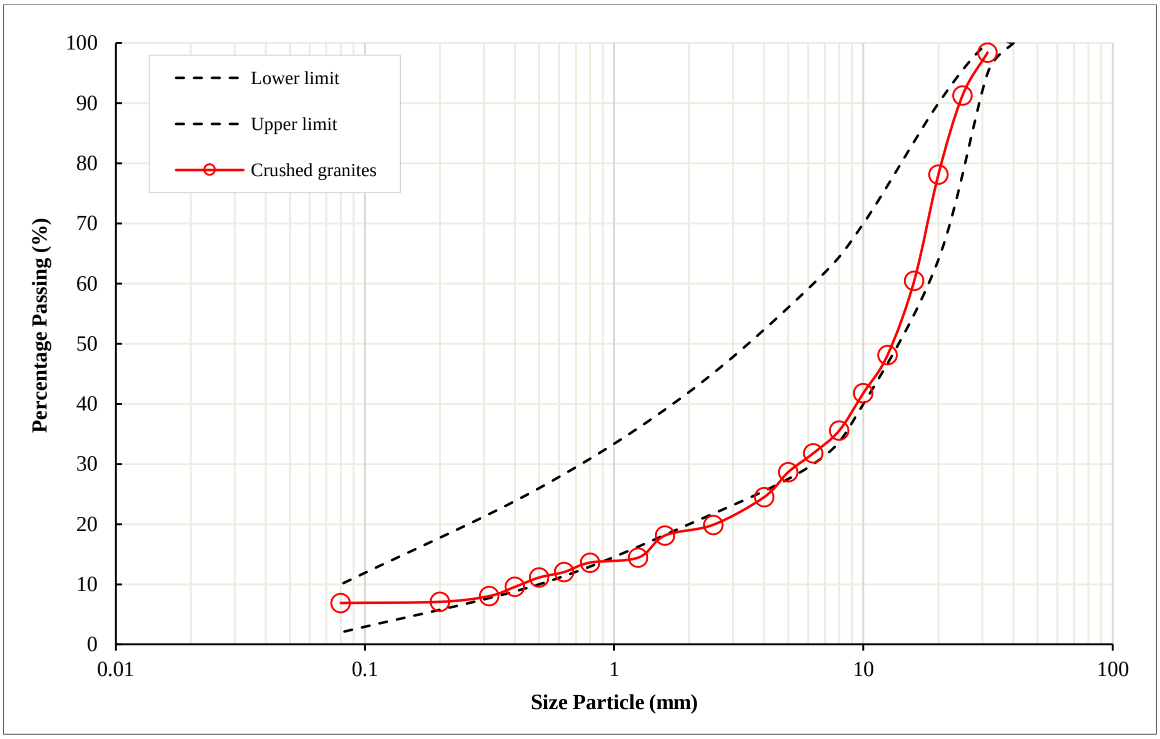


Figure 4. Grain size distribution of the crushed granite compared with upper and lower limits of the normalized aggregates established by CEBTP [1]

Figure 5 shows the particle size curves of the lateritic materials C1, C2 and CM before and after addition of crushed granite: 20%, 25%, 30% and 35%. It is noted that the addition of crushed granites aggregates increases the granular skeleton, which shifts downward as the content of aggregates increases. In fact, the percentage passing through the 2 mm sieve decreased from 37.8% to 31.4% before and after addition of 35% of granites for C1. For C2, it decreased from 37.3% to 29.6% before and after addition of 35% of granites. For CM, it decreased from 24.4% to 25.3% before and after addition of 35% of aggregates. However, the granular class does not change after addition of aggregates, i.e. class A-2-6 for C1, class A-2-6 for C2 and class A-2-6 for CM. All the curves remain in the boundaries proposed by CEBTP [1] for the useful materials in sub-base layer.

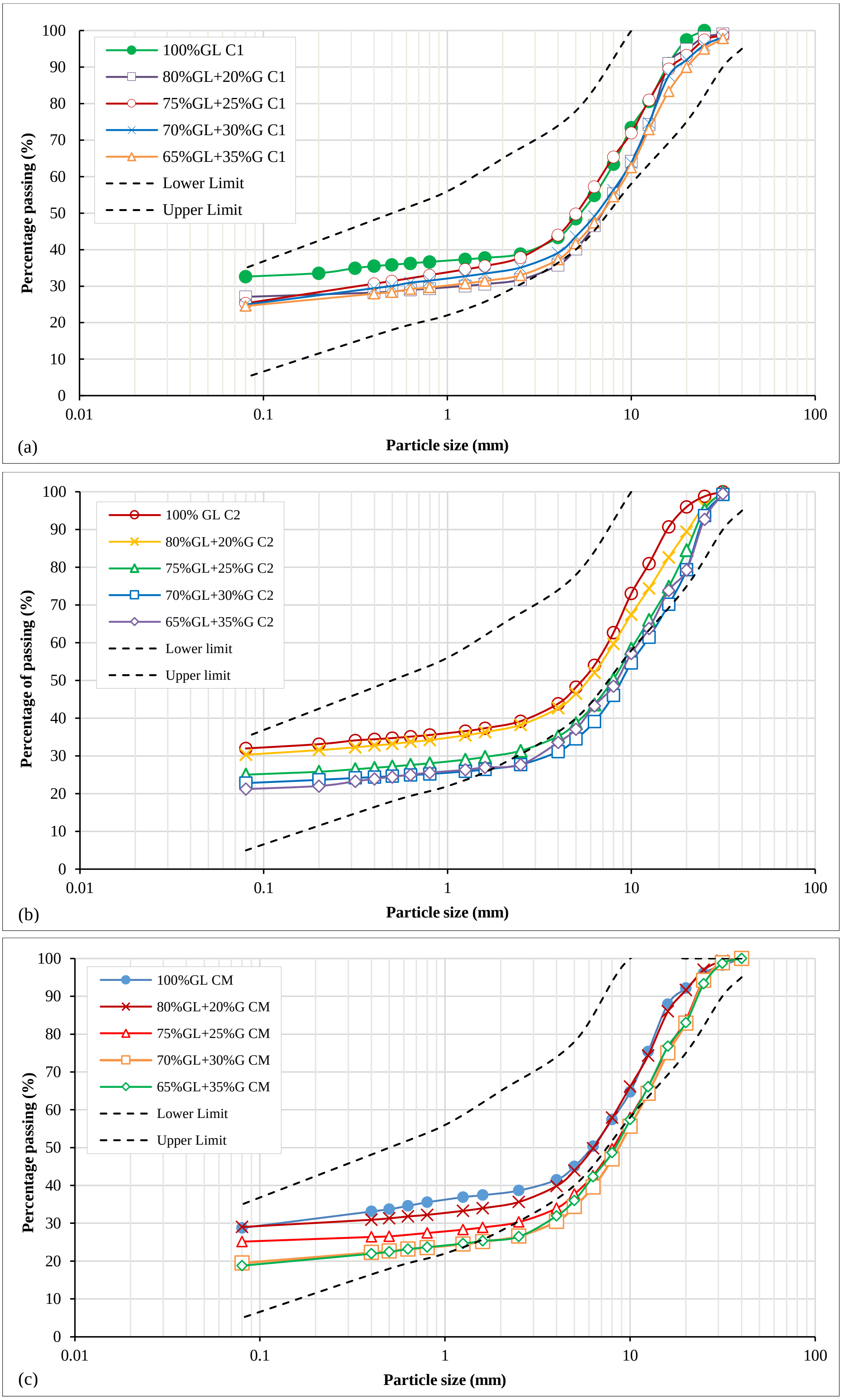


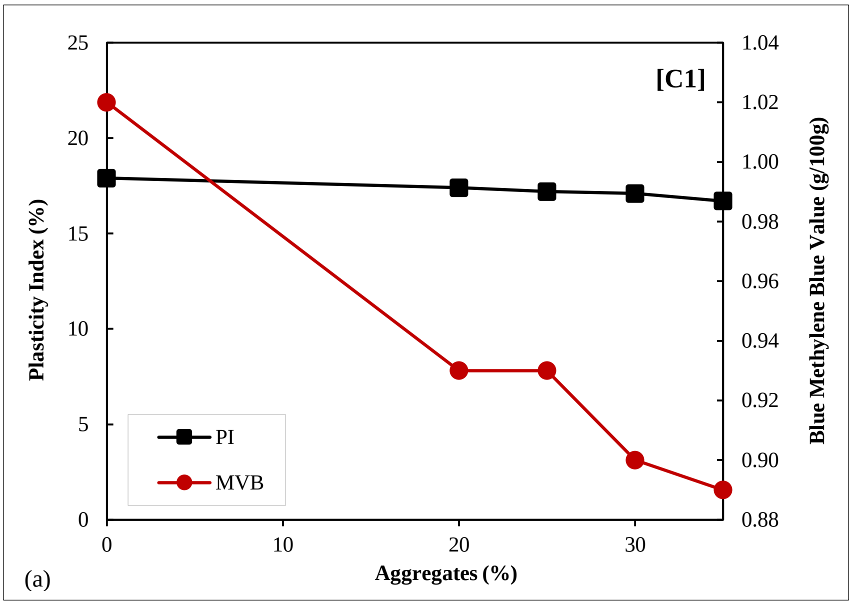
Figure 5. Particle size distribution curves of a) layer C1, b) layer C2, and c) CM before and after addition of crushed granites compared the criteria boundaries for application in sub-base layer established by CEBTP (1984)

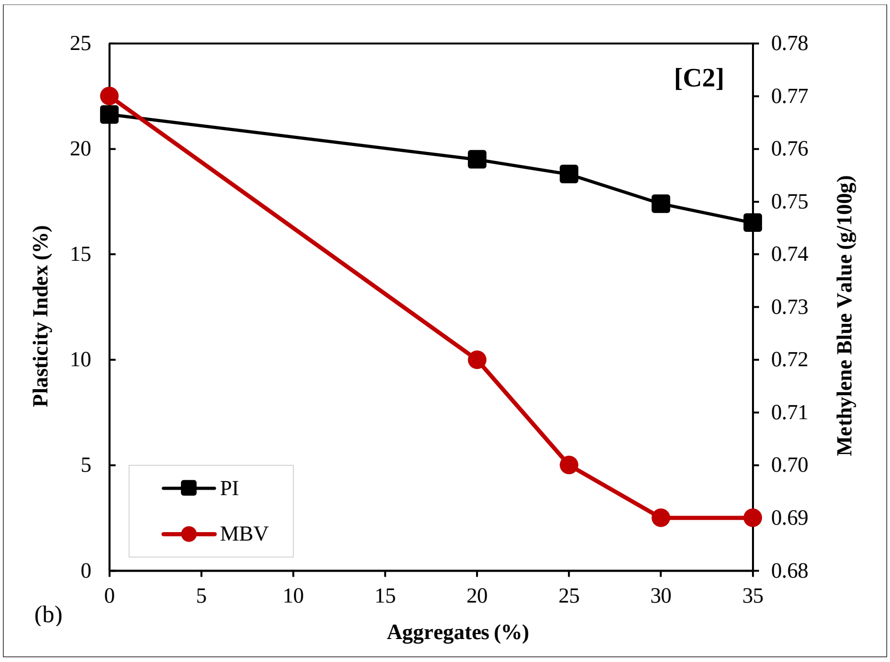
Tables 2 and 3 show the physical and compaction properties of the materials C1, C2 and CM, before and after addition of crushed aggregate. The plasticity index slightly decreases with the increase in the content of aggregate for all three lateritic materials (from 17.9% to 16.7% for C1, from 21.6% to 16.5% for C2 and from 21.4% to 16.6% (Figure 6). The clay activity (methylene blue value) also decreases with the increase in the percentage of crushed granite. Indeed, the methylene blue value decreases by 20% (from 0.9% to 0.72%) for C1, by 13% (from 0.63% to 0.55%) for C2 and by 17% (from 0.7 to 0.58) for CM (Figure 6). These phenomena could be attributed to the reduction of clay content as the non-plastic crushed granite content is increased. This results in a reduction in the water absorption capacity of the soil. The similar evolutions have been reported by various authors [12, 15, 19, 33]. Ndiaye et al. [12] found that the addition of 10% dune sand to lateritic soils from Senegal results in a 20% decrease in the plasticity Index (PI). Hyoumbi [15] showed that the PI decreased about 35% after the addition of 50% of crushed basanite of class 0/5. Issiakou [33] obtained an increase of 24% after addition of 10% of lateritic nodules of class 0/5 mm. Gidigasu [19] showed that the mix of lateritic soil with 80% of crushed waste rock decreased the PI by 40%. Even if there is a reduction in the plasticity index, it remains above 15, which is the minimum value required for a use as base layer according to the CEBTP. However, it remains below 20%, which is the maximum value for use as a sub-base layer according to the CEBTP.

Table 3. Physical and compaction properties of C1, C2 and CM before and after addition of crushed granites

	Plasticity and clay content					Compaction characteristics			Bearing capacity
% G	**LL (%)**	**PI (%)**	**MBV (d<5 mm)**	**MBV (d<80 µm)**	**f*PI**	**OWC (%)**	**MDD (t/m^3)**	**Specific density (t/m^3)**	**CBR**
					C1				
0	33	17.9	0.9	1.02	584	11.2	2.02	2.73	11
20	31	17.4	0.87	0.93	471	9	2.11	2.87	14
25	32	17.2	0.86	0.93	434	8.65	2.13	2.88	21
30	31	17.1	0.75	0.9	427	8.1	2.14	2.86	37
35	30	16.7	0.72	0.89	410	6.87	2.18	2.89	41
					C2				
0	33	21.6	0.63	0.77	691	10.9	2.03	2.67	16
20	36	19.5	0.59	0.72	592	10.01	2.09	2.94	23
25	36	18.8	0.57	0.7	471	8	2.13	2.94	36
30	33	17.4	0.58	0.69	397	7.61	2.14	2.82	45
35	36	16.5	0.55	0.69	349	7.18	2.12	2.84	24
					CM				
0	33	21.4	0.7	0.83	621	10.4	2.06	2.75	17
20	33	19.8	0.67	0.79	574	9.12	2.08	2.94	27
25	34	18.0	0.65	0.79	453	8.8	2.13	2.94	32
30	32	17.3	0.65	0.77	338	8.45	2.14	2.82	53
35	33	16.6	0.58	0.75	312	8.35	2.12	2.84	31

% G: the content of crushed granite (%), d: diameter (mm), LL: Liquid Limit, PI: Plastic Index, MBV: Methylene Blue Value, f*PI: the content of fine particle (< 80µm) multiply by plasticity index, OWC: Optimum water content, MDD: Maximum Dry Density, CBR: Californian Bearing ratio after 4 days soaked in water.





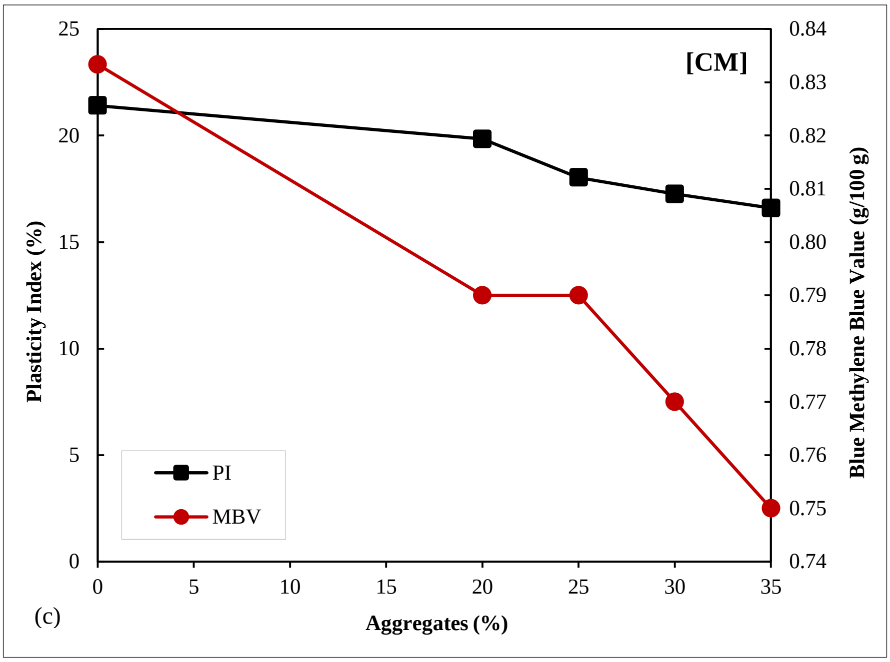


Figure 6. Variation of the plasticity index and the methylene blue value with the addition of aggregates, for the materials: a) layer C1; b) layer C2; c) CM

The compaction properties are also influenced by the amount of crushed granite added to the lateritic material. Figure 7 shows the variation of the maximum dry density (MDD) and the optimum water content (OWC) with the addition of the crushed granite for the three types of lateritic materials. It shows that the MDD increases from 2.02 to 2.18 t/m^3 for C1, from 2.03 to 2.12 t/m^3 for C2 and from 2.06 to 2.12 t/m^3 for CM and the OWC decreases from 11.2 to 6.87% for C1, from 10.9 to 7.18% for C2 and from 10.4 to 8.35% for CM. This trend is attributed to the specific density of granite (2.9 t/m^3), higher than that of the soil (2.75 t/m^3), which increases the MDD of the mix materials and decrease the clay content in the mixture, which resulted in the decrease of OWC required by the mixtures to reach the MDD [15, 16]. In fact, the specific density of the mixtures increases from 2.73 to 2.89 t/m^3 for C1; 2.67 to 2.84 t/m^3 for C2 and 2.75 to 2.84 t/m^3 for CM. This resulted in the decrease of the void index of the mixtures.

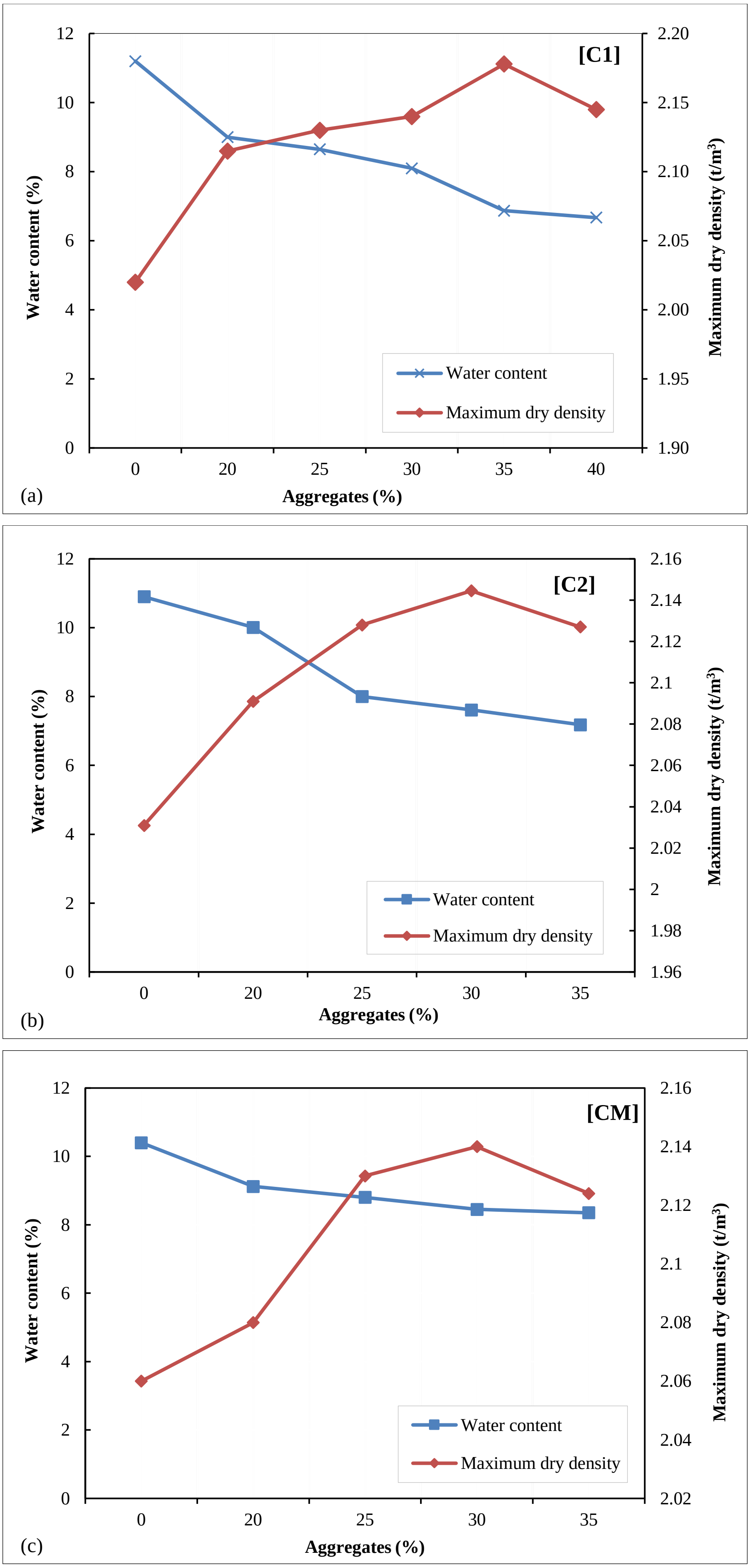


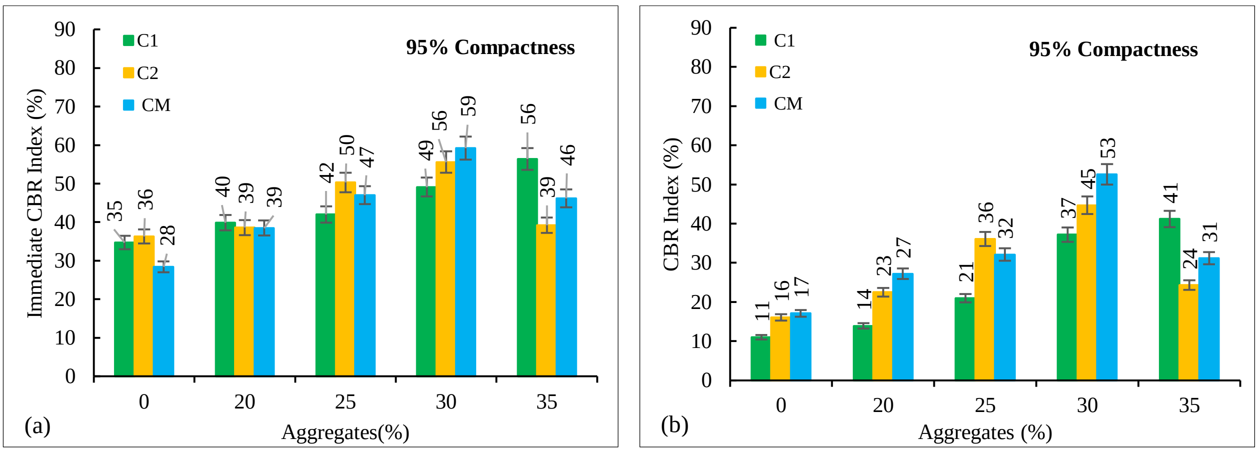
Figure 7. Variation of dry density and optimum water content with the addition of aggregates, for the materials: a) layer C1; b) layer C2; c) CM

The OWC decreased for the various lateritic materials with the percentage of crushed material. This reduction of the OWC is estimated at 40%, 34% and 20% for C1, C2 and CM, respectively from the raw material to a material mixed with 35% of granites. According to Chalermyanont & Arrykul [34], the OWC decreases with the decreasing amount of clay.

The MDD increased with the percentage of crushed aggregates. In fact, the density increases from 2.02 to 2.14 t/m^3, from 2.03 to 2.14 t/m^3 and from 2.06 to 2.14 t/m^3 respectively for C1, C2 and CM, before and after addition of 30% of granites. As the granular fraction increases, the voids created between the coarse particles become larger and the fines fill in between these voids, which leads to the increase of density and consequently the decrease of the void index. Table 3 shows the increase of the specific density which proves the decrease of the void index. However, the density of materials C2 and CM decreased beyond the addition of 30% due to the increase in the content of granites which exceeds the optimum content and therefore increases the voids in the material. Tony et al. [18] reported the same trend for a mixture of lateritic soil with quarry dust, where the MDD increased from 1.584 t/m^3 to 1.768 t/m^3 with 40 % of quarry dust, beyond which it decreased. The decrease of MDD is due to the change in the grain size of the mix from a well-graded material for percentages of laterite lower than 80% to a badly graded for a higher percentage of lateritic soil [18]. This suggests that C2 and CM pass from a well-graded material to a badly-graded material after addition of 35% of granites. In fact, Figures 5-b and 5-c show that the grain fraction 1/20 mm of materials C2 and CM get out of the boundaries recommended for usable in sub-base layer. On the contrary, the MDD continues to increase for C1 until 35% of the addition of granites. This could be explained by the fact that the lateritic materials C1 are more friable than C2 and CM. Therefore, C1 remains a well graded material after addition of 35% of granites, as proven by the particle size distribution curves. Indeed, the particle size distribution curve of C1 remains in the boundaries of the materials usable in sub-base layer (Figure 5-a) even after addition of 35% of granites. This increase of the MDD is more significant for C1, compared to other materials [35].

3.2. Mechanical Properties

Figure 8 presents the immediate CBR index (a, c) and the CBR index after four days of saturation by immersion in water (b, d) of the three lateritic materials (C1, C2 and CM) mixed with aggregates at different levels of compaction energies (95% and 98%). The value of the immediate CBR is higher than the value of the saturated CBR in all cases. The immediate CBR increases with the percentage of granites addition for the three lateritic materials. For C1, compacted at 95% of the maximum dry density (MDD), the immediate CBR index increases from the value of 35% before to the value of 56% after the addition of 35% of granites; i.e. a relative increase of 60% [(56-35)/35]. For C2, the immediate CBR index increases from the value of 36% to the value of 56% after addition of 30% granites; i.e. a relative increase of 55%. For CM, the immediate CBR index increases from the value of 28% to the value of 59% after addition of 30% of granites; i.e. a relative increase of 110%. The similar observation was made on the CBR indices after immersion in water. The relative increase of the CBR is about 273% for C1 after the addition of 35% of crushed granite, and about 180% and 139% for C2 and CM after addition of 30% of crushed granites. The value of the CBR (immediate and saturated) shows that the addition of granites increases the resistance of materials. The increase in the values of the CBR is directly related to the increase of the MDD (section 3.1) with the addition of granite [15, 16, 36, and 37]. Table 3 presents the variation of the product of fines content and plasticity index (f*IP) with the content of crushed granites. It can be noted that the values of f*IP decreases with the content of granites (from 584 to 410 for C1, from 691 to 349, from 621 to 312 for CM after addition of 35% of crushed granites to the raw lateritic soil) which shows that the fine content (<80 µm) decreased. The increase in crushed granites in the mix reduces the fines content and water holding capacity and lead to the improvement of the CBR values. The materials C2 and CM present higher CBR than C1 generally for percentages of crushed granites below 30%. This trend can be explained by the methylene blue values of C2 and CM which are lower than that of C1. The material CM reaches the highest values of CBR (53% with 30% crushed granite). The values of f*IP decrease 621 to 312 with the content of granites and reach lower values for CM than for C2. The lower is the values of f*IP, the higher is the CBR of the material [38], which may explain the higher values of CBR for CM.



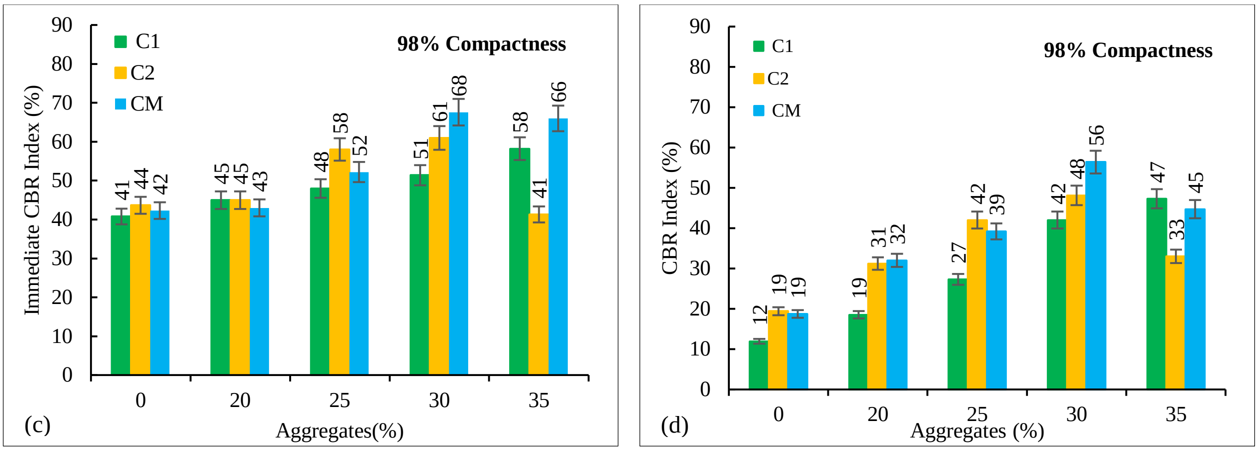


Figure 8. CBR index of lateritic materials C1, C2 and CM with respect to the percentage of granites: a) immediate CBR index at 95% (of the compactness), b) CBR index after 4 days of immersion at 95%, c) immediate CBR index at 98%, d) CBR index after 4 days of immersion at 98%.

Issiakou et al. [39] reported an increase in CBR of 27% for a laterite from Niger mixed with 10% of lateritic nodules of class 0/5 mm. Ahouet & Elenga [16] reported the same effects on the mixture of a gravelly lateritic materials and alluvial gravel of class 0/31.5 mm from the Bouenza region in Congo. The CBR increased by about 110% after addition of 30% gravel to gravelly lateritic materials. Jérémie [40] studied the laterite from northern Cameroon litho stabilized with palm nut shells of class 0/20 mm. The authors reported a 20% increase in CBR from the raw material and material mixed with 20% palm nut shells. Similarly, Toe [11] reported a relatively slight increase of the CBR (15%) of lateritic material after the addition of 30% crushed granites of class 0/25 mm. This value of the CBR is lower than the value reported in the present study; i.e. the increase of 273% for C1, 180% for C2 and 139% for CM, after addition of 30-35% crushed granite. In the present study, the values of CBR increased and reached higher values than in the previous studies [11, 16, 39, 40].This may be due to several factors such as the difference in the nature of the stabilization agent, the difference in the granular class and the specific density of these materials. Indeed, the stabilization agent in this study is crushed granite which has the specific density (2.9 t/m^3) greater than the specific density of lateritic soil (2.75 t/m^3) and increases of the mixture. The slight improvement in the CBR of the latter [11] can be explained by the fact that the addition of granites did not have an effect on the plasticity index and the content of fines which strongly influence the CBR [1, 38].

In the present study, the value of the immediate CBR of the materials is higher than the value of the saturated CBR for all the raw lateritic materials and lateritic materials mixed with crushed granites for all the compactness. Water greatly reduces the bearing capacity of the lateritic materials. The saturated CBR of the raw material decreases relatively by 71%, 57% and 55%, with respect to their immediate CBR, respectively for C1, C2 and CM at 98% of the MDD. At 95% of the MDD, the saturated CBR of the raw material decreases relatively by 68%, 47% and 32%, with respect to their immediate CBR, respectively for C1, C2 and CM. The degree of sensitivity to water of C1, C2 and CM is related to the methylene blue values (MBV) and the parameter f*IP. The MBV of C2 (0.77) and CM (0.83) are lower than that of C1 (1.02) and therefore are less affected by the presence of water. Furthermore, CM has lower value of f*IP (621) than C2 (691), which explains its lower sensitivity to water. However, the saturated CBR of the lateritic materials mixed with 30% of granites decreases only by 18%, 21% and 18%, with respect to their immediate CBR, respectively for C1, C2 and CM at 98% of the MDD. At 95% of the MDD, the saturated CBR of the lateritic materials mixed with 30% of granites decreases only by 24%, 20% and 10%, with respect to their immediate CBR, respectively for C1, C2 and CM. This shows that the addition of granites decreases the water sensitivity of the materials, which is in agreement with the evolution of the plasticity index. The material becomes less sensitive to water with the decrease of the content of fines which resulted from the addition of crushed granite. This is very important because most of the degradation of the roads is due to the combined effect of water sensitivity and traffic [41, 42]. This decrease of sensitivity to water with the addition of granite is very interesting to explore in the future studies, given that the properties of lateritic material are largely influenced by the water content.

However, the saturated and the immediate CBR index decrease beyond the addition of 30% of granites, for all soils except C1. This may be due to the shift from a well-graded material to a badly-graded material after addition of 30% of granites for C2 and CM as noticed in section 3.1. The decrease of these parameters is due to the high content of granite which reduces the compactness of the material. The compaction of material C1 produces more fines, given that it is very friable when crushed by hand, compared to the other materials C2 and CM. Therefore, the MDD of C1 continue to increase, with a continuous decrease of the OWC, even with 35% of granite. Similar results were reported by Hyoumbi et al. [15] and Jérémie [40].

The value of CBR shows that the lateritic material from the site of Saaba can be used as a sub-base layer after the addition of 25% of granites for the materials C2 and CM, and 30% of granites for C1, given that the value of their CBR index is greater than the required value of 30%. Indeed, the saturated CBR index is 36% and 32% respectively for the lateritic materials C2 and CM mixed with 25% granite and compacted at 95%, while it is 37% for C1 mixed with 30% of crushed granite compacted at 95%.

The uniaxial compressive tests have been carried out in order to better understand the mechanical behavior of the materials in terms of their compressive strength and elasticity modulus. Figures 9-a, and 9-b presents the evolution of the modulus of elasticity of materials C1 and C2 with the content of crushed granite (0, 20, 25, 30 and 35%) and compactness (90, 95%). It also presents the evolution of molding of OWC and the residual moisture content after the compression test. The modulus of elasticity increases with the content of the granite up to certain percentage (25% or 30%) for all compactness, beyond which it decreases. For C1 and C2, the maximum values of the modulus are respectively 327 MPa and 302 MPa for the compactness of 90% and 358 MPa and 308 MPa for the compactness of 95% A small variation (0.04% to 1.46 %) in the water content, from the OWC and residual moisture, is observed, which shows the good conservation of the samples. The results of the compaction tests showed an increase in the MDD for percentages of granites below 30%. Indeed, this is due to a better grain size distribution and therefore to a better packing of the particles. This same phenomenon explains the increase of the modulus of elasticity. Therefore, C1 and C2 can be used in base layer after addition of 30% of crushed granite compacted at 90% as their modulus of elasticity is greater than 300 MPa [43].

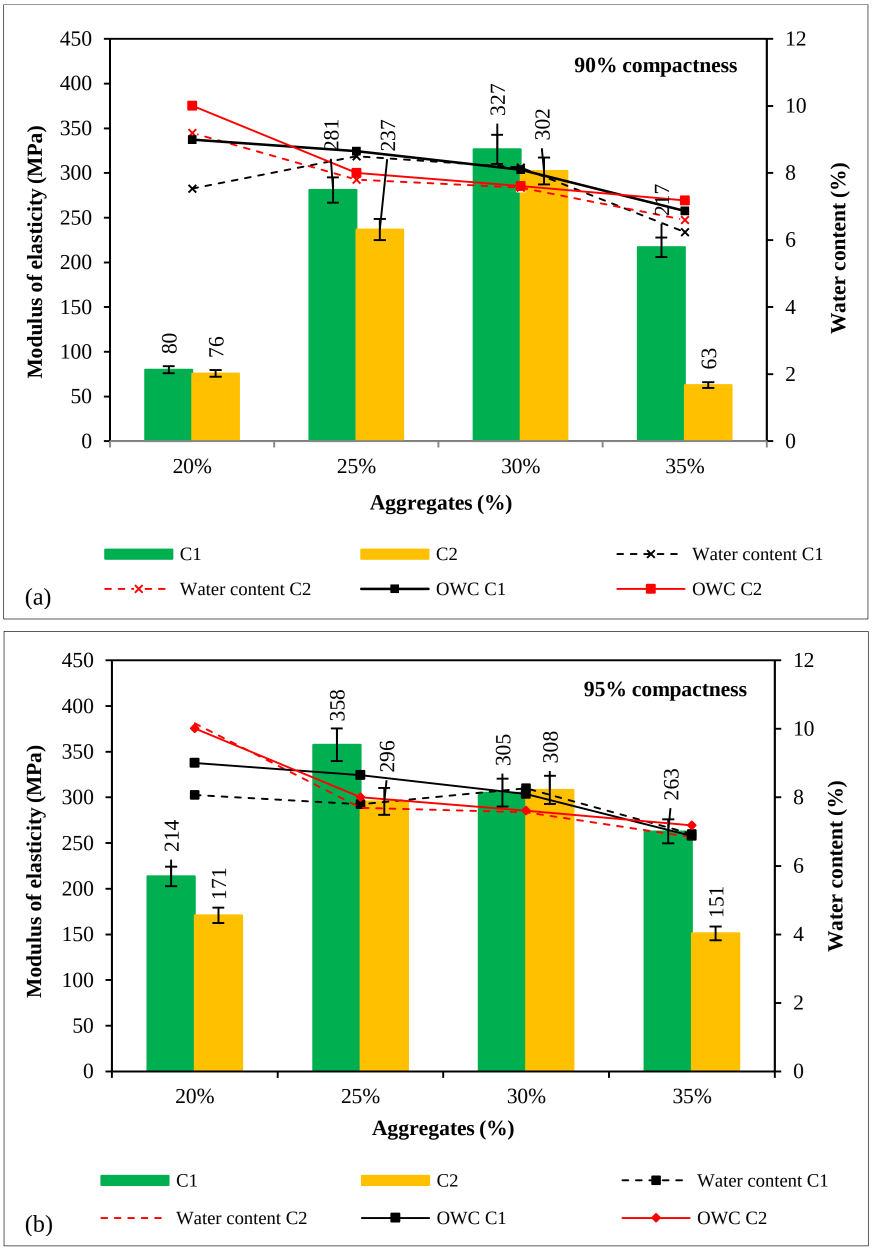
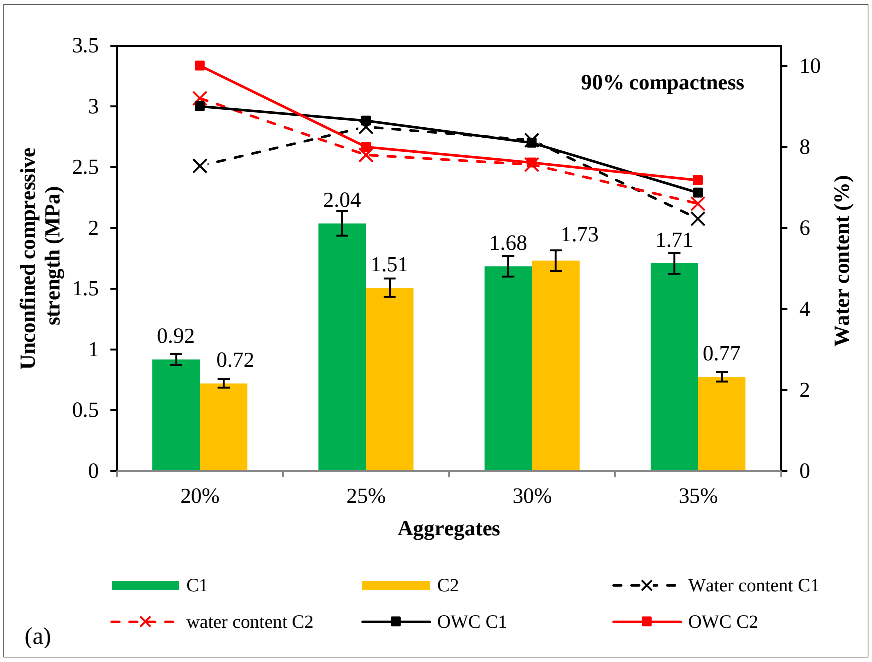


Figure 9. Modulus of elasticity of materials C1 and C2 with the addition of granite: a) compactness of 90%, b) compactness of 95%

Figures 10-a, and 10-b similarly presents the evolution of the unconfined compressive strength (UCS) of C1 and C2 for a compactness of 90% and 95%. The compressive strength increases with the content of crushed granite up to the maximum values beyond which it decreases, for all compactness. The UCS reaches the maximum value of 2.04 MPa for C1 and 1.73 MPa for C2 respectively mixed with 25% and 30% of crushed granite, for the compactness of 90%. For a compactness of 95%, the UCS reaches the maximum value of 2.05 MPa for C1 and 2.77 MPa for C2 respectively mixed with 30% and 25% of crushed granite. A small variation (0.04% to 1.46 %) of the water content, from the OWC and residual moisture, is observed, which shows the good conservation of the samples.



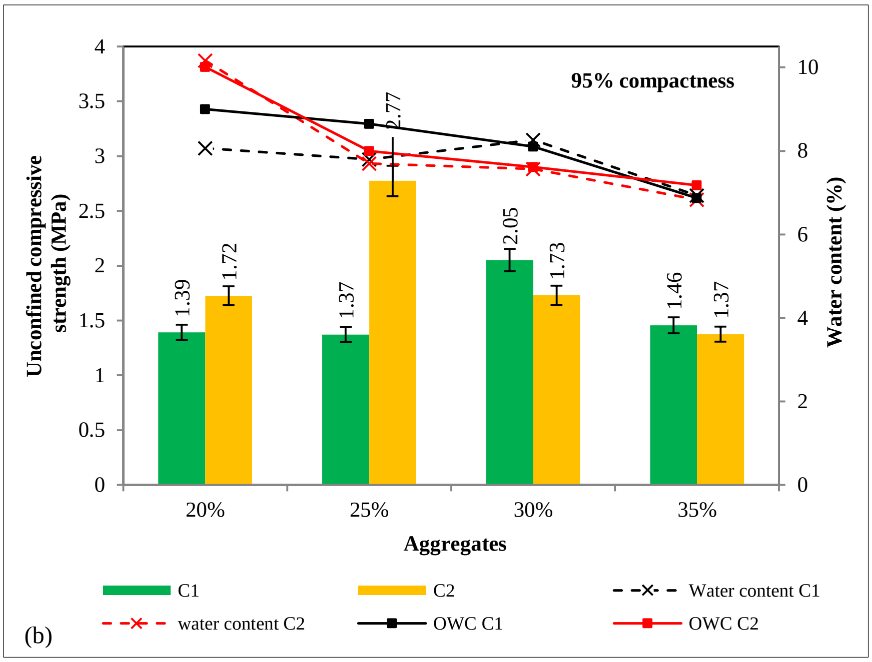


Figure 10. Unconfined compression strength of materials C1 and C2 with the addition of granite: a) compactness of 90%, b) compactness of 95%

The decrease in modulus and strength at certain content of granite resulted from the loss of compactness of the materials due to the high content of granite in the material which leads to a higher void index which is related to the specific density (Table 3). The values of UCS obtained in the present study are lower than those obtained by Hyoumbi et al. [15] on lateritic materials mixed with crushed basanites which varied from 1.7 MPa to 4 MPa. However, the maximum values obtained in the present study are higher than 1.26 MPa obtained by Millogo et al. [6], 1.22 MPa and 1.28 MPa obtained by Nzabakurikiza et al. [44], and 0.88 MPa and 1.22 MPa obtained by Onana et al. [45]. Consoli [8] also reported that the UCS increases from 1.12 to 1.81 MPa after an addition of 45% of sand to the raw material, enabling the use of this mixture for sub-base layer of pavement. According to Messou [2], a material can be used as in sub-base layer if its compressive strength is between 0.5 MPa and 1.5 MPa. Therefore, the materials from Saaba can be used as in sub-base layer.

The compressive test shows a certain contradiction with respect to the CBR index. Indeed, the values of CBR index are higher for the material C2 than for C1; while the value of the modulus and the compressive strength for C1 are higher than for C2. Regarding the CBR, the lateritic material C2 and CM can be used as materials for sub-base layer after addition of 25% of granite aggregates; while C1 can only be used after addition of 30% of aggregates at a compactness of 95%, according to the CEBTP [1]. Regarding the modulus and compressive strength, C1 and C2 can be used as materials for the base layer after addition of 30% of granites, at 90% of compactness.

4. Discussion

The present study on the characteristics of lateritic materials collected from different depths of the site of Saaba shows that the materials C1, C2 and CM have practically the same physical properties except for the methylene blue value, which is lower for C2 and CM. This has influenced the values of the CBR index of the raw materials, which show that C1 is of class S3 while C2 and CM are of class S4, therefore showing the differences in the mechanical properties. The addition of crushed granite to these materials shows an increase in the dry density with the content of granite and a decrease in the optimum water content, the methylene blue value, and the plasticity index. The addition of crushed granite improves the compactness and reduces the content of fines in the material, making it less sensitive to water.

Two trends have been highlighted by the mechanical properties: (1) The addition of crushed granites increases the bearing capacity of the materials, which is shown by an increase in the CBR, the modulus of elasticity as well as the compressive strength with the content of crushed granites. (2) The optimum addition is obtained with 30% of crushed granites for the CBR at 95% compactness, with 30% of crushed granites for the modulus at 90% compactness and with 25% of crushed granites for the modulus at 95% compactness. Various authors, such as Lompo [10], Toé [11], Ndiaye et al. [12], Jjuuko et al. [13], Sudla et al. [14], Hyoumbi et al. [15], Ahouet & Elenga [16], Corrêa et al. [17], Tony et al. [18], and Gidigasu [19], reported similar results. However, these authors rarely study the effects of adding crushed granite to lateritic soils on elasticity modulus and unconfined compression strength. Hyoumbi [15] reported that the maximum value of the unconfined compressive strength is reached at a 30% addition of class 0/5 mm crushed basanite, while the CBR continued to increase beyond the 30% addition. Toe [11] reported that the CBR of materials did not have a significant variation (11%), before and after the addition of granite, tested in the laboratory. However, the author [11] noticed that the gain in the bearing capacity of the material varied from 27 to 32% by performing the deflection tests on a test board formed of lateritic materials mixed with granite.

The present study confirmed that the CBR is a good parameter to evaluate the improvement of a soil by the addition of crushed granite, but the CBR-based evaluation remains insufficient when it is considered alone. It should be combined with the compression tests to better appreciate the level of improvement in the performance of materials and the optimal addition of crushed granite. This was evidenced by the fact that, at a compactness of 95%, the increase in the value of the soaked CBR after the addition of 20% to 30% of granites is 164% for C1 and 96% for C2, while the increase in Young’s modulus is only 67% for C1 and 80% for C2. In this study, the values of CBR overestimate the level of improvement in the mechanical performances of the materials compared to the Young's modulus. In fact, other studies [11, 15] have shown that the use of CBR alone as a parameter for assessment of the improvement of mechanical performance of soil can be misleading in estimating the effect of the addition of granite to the soil. This is due to the fact that the level of improvement of the CBR is not necessarily similar to that of the modulus. Therefore, the estimation of the modulus from the CBR ($E=5\times CBR$), commonly used to determine the thickness of the pavement layers in tropical Africa, is problematic.

This relationship has been discussed for a long time. Several studies showed that this relation can lead to the erroneous estimation of the modulus of lateritic soils [46, 47]. This report recommends that elasticity modulus should be determined from the unconfined compressive test or the indirect tensile test. This method seems to be more suitable because it allows to carry out the volumetric measurement of the strength of the materials and therefore the appropriate deduction of the elasticity modulus; unlike the CBR test which is a localized surface measurement.

However, a certain dispersion of the results of the elasticity modulus was noted in the present study, given the difficulty in achieving the reproducibility of the test specimens, due to the random distribution of the aggregates. It is recommended to increase the number of test specimens to achieve better results. Moreover, the elasticity modulus was determined from the deformation corresponding to 30% of the maximum applied force. This value of the deformations is relatively higher than the deformations caused by the traffic in the real conditions, such as 10^{-6} to 10^{-4}, as reported by Reiffteck [48]. Therefore, it would also be interesting to determine the elasticity modulus at the deformation corresponding to 10% of the maximum force.

5. Conclusion

The apparent difference observed between the materials C1 and C2 in the field was not observed in the geotechnical results of laboratory tests. Indeed, all the lateritic materials (layers: C1, C2 and CM) are classified in the same granular class (A-2-6) according to HRB, and B6 according to the GTR guide. They also have practically similar physical and compaction characteristics. However, a small difference in the mechanical properties was observed. The properties of the CM material are closer to those of C2 than expected. Therefore, the systematic mixing of materials from different layers without separating them by the geotechnical companies seems to be acceptable.

After the addition of crushed granites, there is a tendency to decrease the optimal water content (by 20 to 40%), the plasticity index (by 6.7 to 27%), and the methylene blue value (by 13 to 20%) for all lateritic materials.

The addition of crushed granite resulted in an increase in the maximum dry density and the CBR before and after immersion in water, the modulus of elasticity and the compressive strength. The values of the CBR index of materials C2 and CM mixed with granite are relatively greater than those of C1. The material C1 reaches a higher compressive strength and modulus of elasticity than C2. This suggests that it is not appropriate to systematically consider the CBR index as the only main parameter for the design and construction of road structures. It is recommended to assess the mechanical behaviour of materials, such as the elasticity modulus, and more specifically, under cyclic stress, in order to ensure which of these two parameters (CBR and modulus) is more constraining for the road design. In addition, it is recommended to mix the materials from the first two layers since the properties of the materials vary slightly depending on the depth. In this case, the material C1 from the first layer has the highest modulus. In this case, a more in-depth study of the mechanical parameters would allow us to decide which one of these three materials has the greatest improvement in modulus.

The optimum content of the addition of granite is 25% for C2 and CM and 30% for C1 at 95% of compactness in terms of the CBR index and the compressive parameters. It is also shown that the improved physical, compaction, and mechanical characteristics of the materials mixed with the granite allow for their applications in the sub-base layer of road structures. Litho-stabilization is therefore a technique that allows one to overcome the weakness in terms of the characteristics of lateritic materials.

The results of the compression tests are more dispersed than those of the measurement of the CBR index. This presents the limits in terms of using the compression test, which seems to be more suitable than the empirical relationship from the CBR, for determining the elasticity modulus. It is recommended to increase the number of test specimens to achieve better results. Other methods for determining the elasticity modulus, such as the triaxial test applying repeated cyclic loading as well as the Pundit test, should be carried out to better assess this key input parameter in the design of flexible pavements.

6. Declarations

6.1. Author Contributions

Conceptualization, M.T.M.M.; methodology, M.T.M.M., A.M., A.L.G., and A.P.; validation, M.T.M.M., A.M., A.L.G., and A.P; formal analysis, M.T.M.M.; resources, A.M. and A.L.G.; data curation, M.T.M.M.; Investigation, M.T.M.M., A.M., A.L.G., and A.P.; writing—original draft preparation, M.T.M.M.; writing—review and editing, M.T.M.M., A.M., A.L.G., and A.P.; visualization, A.M. and A.P.; supervision, A.M. and A.P. All authors have read and agreed to the published version of the manuscript.

6.2. Data Availability Statement

The data presented in this study are available in article.

6.3. Funding

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6.4. Acknowledgements

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6.5. Conflicts of Interest

The authors declare no conflict of interest.

7. References

[1] AIPCR. (1984). Review of the Practical Guide to Pavement Design for Tropical Countries. Experimental Center for Building and Public Works Research and Studies, Paris, France. (In French).

[2] Messou, M. (1980). Mechanical behavior of a lateritic gravel base course improved with cement: the case of roads in Côte d'Ivoire. Ph.D. Thesis, National school of ponts and chaussées, Paris, France. (In French).

[3] Gidigasu, M. D. (1974). Degree of weathering in the identification of laterite materials for engineering purposes - a review. Engineering Geology, 8(3), 213–266. doi:10.1016/0013-7952(74)90001-5.

[4] Tockol, I., Massiera, M., Chiasson, P. A., & SALIHA MAIGA, M. (1994). Lateritic gravel in the Sahel countries: case of unpaved roads. 7th International congress International Association of Engineering Geology, 5-9 September 1994, Losoba, Portugal, 3423-3431. (In French).

[5] Autret, P. (1983). Laterites and gravelous lateritic. Institute of science and technology of equipment and environment for development, Paris, France. (In French).

[6] Millogo, Y., Traoré, K., Ouedraogo, R., Kaboré, K., Blanchart, P., & Thomassin, J. H. (2008). Geotechnical, mechanical, chemical and mineralogical characterization of a lateritic gravels of Sapouy (Burkina Faso) used in road construction. Construction and Building Materials, 22(2), 70–76. doi:10.1016/j.conbuildmat.2006.07.014.

[7] Boumekik, N. E. I., Labed, M., Mellas, M., & Mabrouki, A. (2021). Optimization of the Ultimate Bearing Capacity of Reinforced Soft Soils through the Concept of the Critical Length of Stone Columns. Civil Engineering Journal, 7(9), 1472–1487. doi:10.28991/cej-2021-03091737.

[8] Consoli, N. C., Párraga Morales, D., & Saldanha, R. B. (2021). A new approach for stabilization of lateritic soil with Portland cement and sand: strength and durability. Acta Geotechnica, 16(5), 1473–1486. doi:10.1007/s11440-020-01136-y.

[9] Oyediran, I. A., & Ayeni, O. O. (2020). Comparative effect of microbial induced calcite precipitate, cement and rice husk ash on the geotechnical properties of soils. SN Applied Sciences, 2(7), 1157. doi:10.1007/s42452-020-2956-0.

[10] Lompo, P. (1980). Materials used in road construction in Upper Volta. A non-traditional material “The lithostab”. 4th African Road Conference, 20-25 January 1980, Nairobi, Kenya, 20-25. (In French).

[11] Toe, J. M. (2007). Use of the chassis-assisted litho stabilization technique. A worksite experience. Presentation at the Conference of Young African Geotechnicians, Tunisia, 16-18. (In French).

[12] Ndiaye, M., Magnan, J. P., Cisse, I. K., & Cisse, L. (2013). Study of the improvement of laterites in Senegal by adding sand. Bulletin of the Roads and Bridges Laboratories, 123-127. (In French).

[13] Jjuuko, S., Kalumba, D., Bbira, S., & Bamutenda, J. B. (2014). Blending of marginally suitable lateritic soils for use in base construction. Proceedings of the IV South American young geotechnical engineers conference–IVSAYGEC, Bogota, Colombia.

[14] Sudla, P., Horpibulsuk, S., Chinkulkijniwat, A., Arulrajah, A., Liu, M. D., & Hoy, M. (2018). Marginal lateritic soil/crushed slag blends as an engineering fill material. Soils and Foundations, 58(3), 786–795. doi:10.1016/j.sandf.2018.03.007.

[15] Hyoumbi, W. T., Pizette, P., Wouatong, A. S. L., Abriak, N. E., Borrel, L. R., Razafimahatratra, F. N., & Guiouillier, T. (2019). Investigations of the Crushed Basanite Aggregates Effects on Lateritic Fine Soils of Bafang Area (West-Cameroon). Geotechnical and Geological Engineering, 37(3), 2147–2164. doi:10.1007/s10706-018-0751-0.

[16] Ahouet, L., & Elenga, R. G. (2019). Improvement of the geotechnical properties of lateritic gravel by adding crushed alluvial gravel 0/31.5. Applied and Engineering Sciences, 3(1), 1-6. (In French).

[17] Corrêa, F. A., Collares, A. C. Z. B., Collares, E. G., Ribeiro, R. P., & Reis, F. M. D. (2020). Evaluation of mixtures of lateritic clayey soil with quartzite and stone powder for road purposes. Transportes, 28(3), 228–237. doi:10.14295/transportes.v28i3.2088.

[18] Tony, N. C., Devdas, G., Raj, S. G., & Varghese, R. M. (2021). Compaction Characteristics of Red Earth and Quarry Dust Combinations. In Proceedings of the Indian Geotechnical Conference 2019, 191-201. Springer, Singapore. doi:10.1007/978-981-33-6346-5_17.

[19] Gidigasu, S. S. R., Lawer, K. A., Gawu, S. K. Y., & Emmanuel, E. (2021). Waste crushed rock stabilised lateritic soil and spent carbide blends as a road base material. Geomechanics and Geoengineering, 1–13. doi:10.1080/17486025.2021.1903096.

[20] Color, M. (1975). Munsell Soil Color Chart: Macbeth Division of Kollmorgen Corporation. Baltmore, Maryland.

[21] NF EN ISO 17892-4. (2018). Geotechnical investigations and tests: Laboratory tests on soils - Part 4: Determination of particle size distribution. French Association for Standardization (AFNOR), Paris, France. (In French).

[22] NF EN ISO 17892-12. (2018). Geotechnical investigations and tests - Laboratory tests on soils - Part 12: determination of liquid and plasticity limits. French Association for Standardization (AFNOR), Paris, France. (In French).

[23] NF P 94-068. (1998). Soils: reconnaissance and tests: Measurement of the methylene blue absorption capacity of a soil or a rocky material-Determination of the methylene blue value of a soil or a rocky material by testing to the job. French Association for Standardization (AFNOR), Paris, France. (In French).

[24] ASTM D 5550-06. (2006). Standard test: method for specific gravity of soil solids by helium gas Pycnometer. ASTM International, Pennsylvania, United States.

[25] NF P 94-093. (2014). Soils: reconnaissance and tests: Determination of the compaction references of a material-Normal Proctor test-Modified Proctor test. French Association for Standardization (AFNOR), Paris, France. (In French).

[26] NF P 94-078. (1997). Soils: reconnaissance and tests: CBR index after immersion. Immediate-Measure Bearing Index on sample compacted in the CBR mold. French Association for Standardization (AFNOR), Paris, France. (In French).

[27] NF EN 13286-41. (2003Mixtures treated and unbound with hydraulic binders - Part 41: Test method for the determination of the compressive strength of mixtures treated with hydraulic binders. French Association for Standardization (AFNOR), Paris, France. (In French).

[28] NF EN 13286-43. (2003). Mixtures treated and unbound with hydraulic binders - Part 43: Test method for the determination of the modulus of elasticity of mixtures treated with hydraulic binders. French Association for Standardization (AFNOR), Paris, France. (In French).

[29] National research Council, Highway Research Board. (1958). Proceedings of the annual Meeting-Highway Research Board, National Academy of Sciences-National Research Council, Washington, United States.

[30] French Road Earthworks Guidelines (GTR). (2000). Guide to Road Earthworks, construction of embankments and subgrades, booklets I and II, GTR SETRA-LCPC. (In French).

[31] Casagrande, A. (1948). Classification and Identification of Soils. Transactions of the American Society of Civil Engineers, 113(1), 901–930. doi:10.1061/taceat.0006109.

[32] Ki, B. I. J., Ba, M., Gueye, R., Hornych, P., & Sana, A. (2021). Effect of Water Content and Grains Size Distribution on the Characteristic Resilient Young's Modulus Obtained Using Anisotropic Boyce Model on Gravelly Lateritic Soils from Tropical Africa (Burkina Faso and Senegal). Open Journal of Civil Engineering, 11(01), 134–152. doi:10.4236/ojce.2021.111009.

[33] Issiakou, M. S. (2016). Caractérisation et valorisation des matérieux latéritiques utilisés en construction routière au Niger. Ph.D. Thesis, University of Bordeaux, Bordeaux, France. (In French).

[34] Chalermyanont, T., & Arrykul, S. (2005). Compacted sand-bentonite mixtures for hydraulic containment liners. Songklanakarin Journal of Science and Technology, 27(2), 313-323.

[35] Hassan, H., Taher, S., & Alyousify, S. (2020). Effect of Gravel Dust and Limestone Dust on Geotechnical Properties of Clayey Soil. The Journal of the University of Duhok, 23(2), 194–205. doi:10.26682/csjuod.2020.23.2.16.

[36] Ojuri, O. O., Adavi, A. A., & Oluwatuyi, O. E. (2017). Geotechnical and environmental evaluation of lime–cement stabilized soil–mine tailing mixtures for highway construction. Transportation Geotechnics, 10, 1–12. doi:10.1016/j.trgeo.2016.10.001.

[37] Okagbue, C. O., & Onyeobi, T. U. S. (1999). Potential of marble dust to stabilise red tropical soils for road construction. Engineering Geology, 53(3–4), 371–380. doi:10.1016/S0013-7952(99)00036-8.

[38] Gidigasu, M. D. (1983). Development of acceptance specifications for tropical gravel paving materials. Engineering Geology, 19(3), 213–240. doi:10.1016/0013-7952(83)90004-2.

[39] Issiakou, M. S., Saiyouri, N., Anguy, Y., Gaborieau, C., & Fabre, R. (2015). Study of lateritic materials used in road construction in Niger: improvement method. Civil Engineering University Meetings, May 2015, Bayonne, France. (In French).

[40] Madjadoumbaye, J., Kamdjo, G., Mbessa, M., Defo, F. E. H., & Tamo, T. T. (2013). Possibilities for Improving the Bearing Capacity of Laterite with a Vegetable Shell: the Shell of Palm Kernels. The Electronic Journal of Geotechnical Engineering (EJGE), 18, 1917-1928.

[41] Saberian, M., Li, J., & Setunge, S. (2019). Evaluation of permanent deformation of a new pavement base and subbase containing unbound granular materials, crumb rubber and crushed glass. Journal of Cleaner Production, 230, 38–45. doi:10.1016/j.jclepro.2019.05.100

[42] Silva, M. F. da, Ribeiro, M. M. P., Furlan, A. P., & Fabbri, G. T. P. (2021). Effect of compaction water content and stress ratio on permanent deformation of a subgrade lateritic soil. Transportation Geotechnics, 26, 100443. doi:10.1016/j.trgeo.2020.100443.

[43] Bagarre, E. (1990). Use of lateritic gravel in road engineering. Institute of science and technology of equipment and environment for development (ISTED). Available online: https://trid.trb.org/view/1018608 (accessed on March 2022) (In French).

[44] Nzabakurikiza, A., Onana, V. L., Ze, A. N., Mvindi, A. T. N., & Ekodeck, G. E. (2017). Geological, geotechnical, and mechanical characterization of lateritic gravels from Eastern Cameroon for road construction purposes. Bulletin of Engineering Geology and the Environment, 76(4), 1549–1562. doi:10.1007/s10064-016-0979-y.

[45] Onana, V. L., Ngo'o Ze, A., Eko, R. M., Ntouala, R. F. D., Nanga Bineli, M. T., Owoudou, B. N., & Ekodeck, G. E. (2017). Geological identification, geotechnical and mechanical characterization of charnockite-derived lateritic gravels from Southern Cameroon for road construction purposes. Transportation Geotechnics, 10, 35–46. doi:10.1016/j.trgeo.2016.12.001.

[46] AGEPAR Working Group PIARC Working Group PIARC AGEPAR. (2019). Methodological document for the establishment of a national catalog and pavement design guide. 2019R40EN - Technical Report, PIARC. (In French).

[47] Gansonré, Y., Breul, P., Bacconnet, C., Benz, M., & Gourvès, R. (2021). Estimation of lateritic soils elastic modulus from CBR index for pavement engineering in Burkina Faso. International Journal of Pavement Research and Technology, 14(3), 348–356. doi:10.1007/s42947-020-0118-9.

[48] Reiffsteck, P. (2002). New test technologies in soil mechanics-State of the art. PARAM 2002-international symposium identification and determination of soil and rock parameters for geotechnical calculations, Paris, September 2-3, 2002. (In French).