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Geotechnical Characterization and Stability Analysis of Excavation Slopes in Road Works: Case of the Works of the Seguela-Touba/Biankouma-Sipilou Road Development and Asphalt Project

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Abstract

This study analyzes the geotechnical properties of the cut slopes along the Biankouma-Sipilou road project and the Kabakouma and Sokourala spurs in order to identify the causes of embankment instability. The authors performed particle size analyses, Atterberg limit tests, modified Proctor tests, and CBR tests on samples taken at different depths from three specific sites: Pk1 + 760 - PK2 + 020, Pk4 + 500 - PK4 + 960, and PK9 + 460 - PK9 + 800. The results highlight specific soil horizons, including lateritic gravels, clayey sands, and deep clay layers ranging from low to high plasticity. For each study site, the soil horizons exhibit, from top to bottom, high to very low bearing capacity and stability parameters, reaching 7% and 9% respectively, for a dry density at the OPM of approximately 1.5 t/m^3 and 1.59 t/m^3 for slopes Pk4 + 500 - PK4 + 960 and PK09 + 460 - PK09 + 800. Slopes with unstable depths underwent stabilization techniques.

Keywords

Characterization, Geotechnical Engineering, Stability Analysis, Cut Slope, Road Construction

1. Introduction

The development strategy in Côte d'Ivoire currently relies on modernizing transport infrastructure to improve accessibility to different areas of the country. Western

Côte d'Ivoire has strong economic potential, but unfortunately, its road network is underdeveloped [1]. This strategy is based primarily on two pillars:

- The first focus is the conservation, maintenance, and upgrading of existing infrastructure;
- and the second focus is the paving of new roads as part of economic and social development.

It is within this second axis that the State of Côte d'Ivoire has included in its National Development Plan (NDP) [2] the development and paving of the Biankouma-Sipilou road, including the Soukourala and Kabacouma spurs (80 km). This study area has very rugged terrain, and most of the mountain and valley slopes are exposed to a high risk of erosion [3], because along the entire route, two types of soil are found: highly desaturated ferralitic soils derived from hypersthene granites, and highly and moderately desaturated ferralitic soils derived from granites [4]. Previous studies conducted by [5] show that rising temperatures, changing rainfall patterns, and extreme weather events affect road embankments. These are severely impacted by these climatic hazards.

Indeed, the ferrallitic weathering of granite, under a hot and humid climate, transforms the hard rock into loose materials (sand) and iron oxides. This process generates plasticity through the hydrolysis of feldspars into clay and mechanical strength through cementation due to iron oxides.

During road construction projects, several difficulties arise, such as flooding of construction sites by rain, landslides, and, above all, the instability of embankments in general due to the plasticity of the soil layers. Embankment stability plays a crucial role in many civil engineering and geotechnical projects. A natural or artificial embankment is an inclined surface of the ground that can be formed by natural processes or result from human intervention.

In the field of geotechnics, slope stability analysis and management are essential for several reasons:

- Unstable slopes can lead to landslides, endangering human lives and infrastructure;
- Preventing instability avoids significant costs related to damage and repairs;
- Proper slope management ensures the longevity of structures and infrastructure.

The stabilization of slopes contributes to the preservation of structures, hence the theme submitted to our study: "geotechnical characterization and stability analysis of cut slopes in road works, case of the works of the development and paving project of the Séguela-Touba/Biankouma-Sipilou road".

The overall objective of this work is to study the stability of cut slopes in road construction. More specifically, it aims to analyze the geotechnical parameters influencing slope stability and to propose solutions to enhance the resilience of unstable slopes.

2. Material and Methods

2.1. Presentation of the Study Area

The project is located in western Côte d'Ivoire, in the Tonkpi region, specifically in the departments of Biankouma and Sipilou. It involves the development and asphalt paving of the Biankouma-Sipilou road, including the Kabacouma and Sokourala spurs. The project crosses the following localities: Biankouma, Worodougou, Sipilou, Kabacouma, and Sokourala. The site plan is shown in **Figure 1**. The study area features highly desaturated ferralitic soils derived from hypersthene granites, and highly and moderately desaturated ferralitic soils derived from granites [4]. The hydrographic network is dominated by tributaries of the Cavally River and the N'Zi River to the south. The terrain in this area is very rugged, with a mountainous landscape that includes the region's main peaks, such as Mount Tonkpi and the Dent de Man.

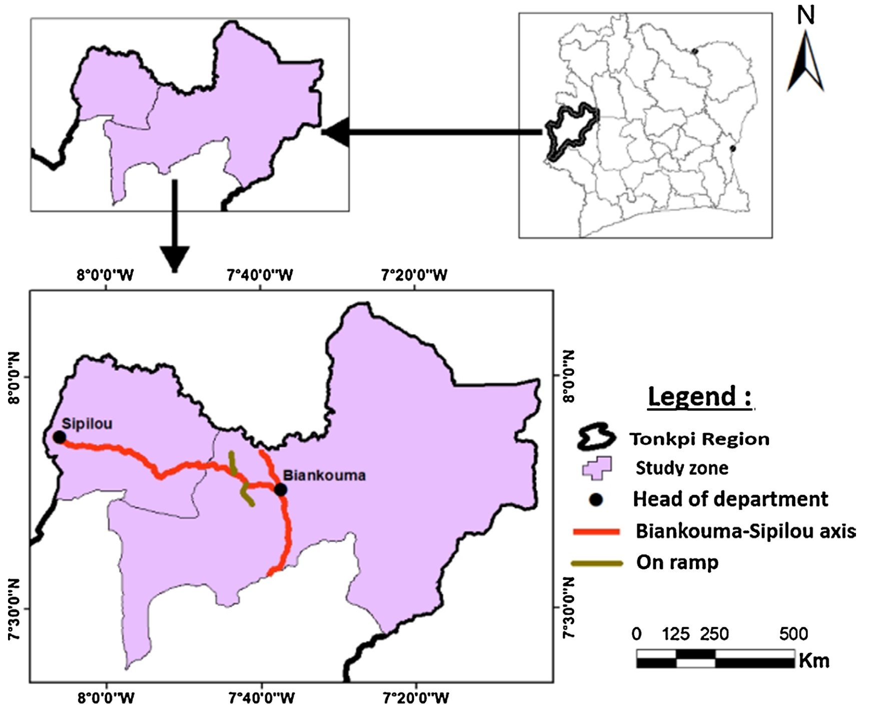

Figure 1. Project location plan (Source: LBTP, 2020. Provisional report of the geotechnical study of the culvert platform and foundations, 60p.).

2.2. Materials Sampling

The materials used in this study were collected from different sections of the site's embankments using standard construction equipment, including a backhoe, pickaxe, shovel, and sampling bags. Three sampling sites were selected: site PK1 + 760 - PK2 + 020, where a 9-meter-high cut embankment was found; and sites PK4 + 500 - PK4 + 960 and PK9 + 460 - PK9 + 800, with an embankment approximately 5 meters high after the topsoil was removed. Material samples were collected from four horizons at the first site and three horizons at the second and third sites. **Figure 2** shows the different cut embankments.

2.3. Study Method

After the materials have been collected and transported, they are dried and then

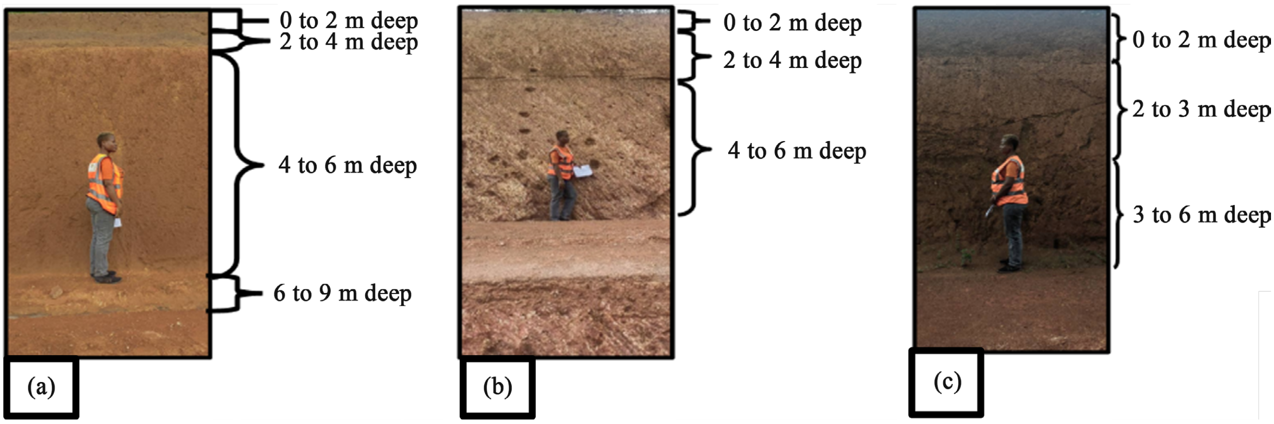

Figure 2. Excavation slope (a): Pk1 + 760 - PK2 + 020, (b) Pk4 + 500- PK4 + 960, (c) PK9 + 460-PK9 + 800.

subjected to geotechnical tests (particle size analysis, Atterberg limit, modified Proctor and CBR bearing capacity).

2.3.1. Particle Size Analysis

The test is carried out according to standard NF EN 933-1 [6], which consists of sieving the solid soil particles using a column of sieves with mesh sizes of 80, 63, 50, 40, 31.5, 20, 16, 10, 5, 2, 1, 0.5, 0.2, and 0.08 mm. The cumulative percentage passing (CP) is calculated from the following Equation (1):

$$\%\text{CP} = 100 - \%\text{CR} \tag{1}$$

$$\text{And} \quad \%\text{CR} = \frac{M_1}{M_{Tot}} \times 100 \tag{2}$$

%CR is the percentage of cumulative rejections from the sieves.

2.3.2. Atterberg Limit

This analysis is carried out according to the NF EN ISO 17892-12 standard [7], which consists of determining the change in consistency of a soil (liquid state, plastic state, and solid state). The parameters determined are the plasticity index (I_P) obtained from Equation (3), the liquid limits (W_L) according to Equation (4), and the plastic limits (W_P) according to equation 5.

$$I_P = W_L - W_P \tag{3}$$

$$W_L = \omega \left(\frac{N}{25} \right) 0.121 \tag{4}$$

$$W_P = W_L - I_P \tag{5}$$

2.3.3. Proctor Essai

The test is carried out in accordance with standard NF P 94-093 [8]. The Proctor compaction characteristics of a material are determined using the modified Proctor test, which is suitable for road projects. This test involves compacting samples in a mold using a rammer. It determines the optimal water content and maximum dry density of a soil during compaction for a given compaction energy.

The weight of the water (Pe) is determined, then the water content of the material is calculated using the following equation:

$$W\%nat = (Pe/Ps) \times 100 \quad (6)$$

The equipment used for the Proctor test is shown in Figure 3.

2.3.4. CBR Essai

The CBR test is performed following the modified Proctor test according to French standard NF P94-078 [9]. The thoroughly homogenized sample is placed in a CBR mold and compacted into five (5) layers, each 2.5 to 3 cm thick, using a modified Proctor rammer. After compaction and immersion of the samples in water, they are subjected to the pressure of the CBR apparatus corresponding to the following indentations : 0.25 mm - 0.50 mm - 0.75 mm - 1.0 mm - 1.50 mm - 2.0 mm - 2.5 mm - 3.0 mm - 3.50 mm - 4.0 mm - 4.50 mm - 5.0 mm - 6.0 mm - 7.0 mm - 8.0 mm - 9.0 mm. Figure 4 shows the equipment used for the CBR test.

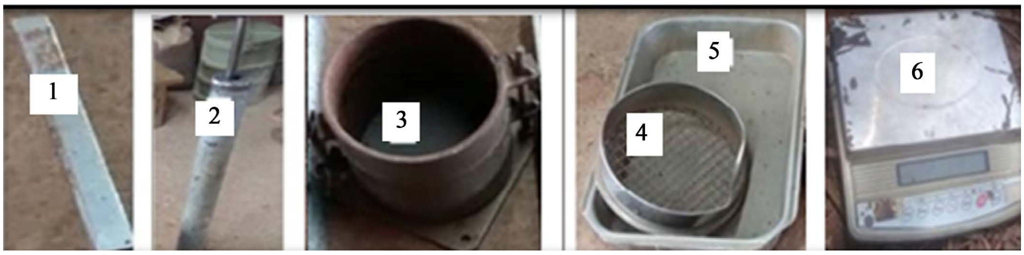

Figure 3. Materials for the proctor test: 1. Razor blade; 2. Proctor rammer; 3. Proctor mold; 4. 20 mm diameter sieve; 5. Tray; 6. Balance.

Figure 4. Equipment for the CBR test 1: CBR mold; 2: Water tank; 3: CBR press.

3. Results

3.1. Particle Size Analysis of the Different Slopes

The results of the particle size analysis carried out on the different horizons of the reworked soils relating to the slopes PK 01 + 760 - PK 02 + 020; PK4 + 500 - PK4 + 960 and PK9 + 460 - PK9 + 800 are presented in Figure 5.

The results in Figure 5(a) show that at depths of 0 - 2 m, 2 - 4 m, 4 - 6 m, and 6 - 9 m, the percentage of fines is 15.80%, 37.27%, 58.53%, and 81.35%, respectively. The percentage of fines increases with the depth of the layers. The maximum diameter (Dmax) at these different depths is less than 50 mm, with a sieve size of 0.08 mm, which is also greater than 35%. However, the 0 - 2 m layer has a fines percentage below 35%. According to standard NF P 11-300, based on the classification of materials usable in the construction of embankments and subgrade layers for road infrastructure, the slope PK01 + 760 - PK02 + 020 is class A according to the Road Earthworks Guide (GTR) [10]. The particle size at these different depths ranges from 0.080 mm to 60 mm. This slope consists of fine sandy and gravelly soils poor in fine particles in the 0 - 2 m layer.

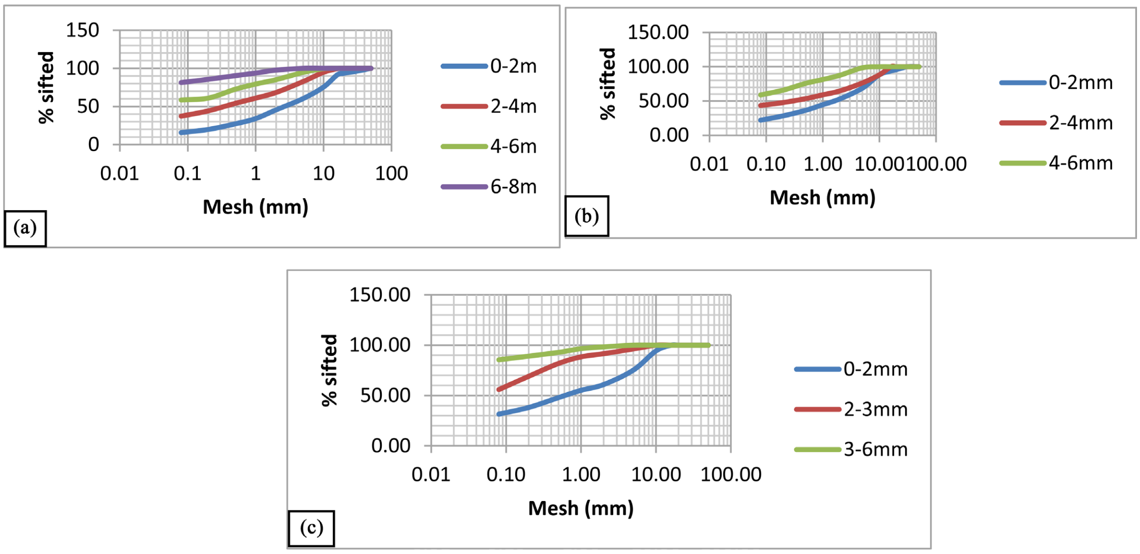

Figure 5. Grain size distribution curve of the slope horizons: (a) PK 01 + 760 - PK 02 + 020; (b) PK4 + 500 - PK4 + 960 and (c) PK9 + 460 - PK9 + 800.

Regarding **Figure 5(b)**, the particle size distribution curve results show that at depths of 0 - 2 m, 2 - 4 m, and 4 - 6 m, the percentage of fines is 22.04%, 43.47%, and 58.87%, respectively. The percentage of fines also increases with the depth of the layers. The maximum diameter (Dmax) at these three depths is also less than 50 mm, with a sieve size of 0.08 mm, which is also greater than 35%, except for the 0-2m layer, which has a fines percentage less than 35%. Since the characteristics of slopes PK04 + 500 - PK04 + 960 are almost identical to those of the first slope, we can assume that they belong to the same class, *i.e.*, class A.

As for **Figure 5(c)**, the particle size distribution curve shows that at the different depths of 0 - 2 m, 2 - 3 m, and 3 - 6 m, the percentage of fines is 31.50%, 55.89%, and 85.49%, respectively. For a depth ranging from 3 - 6 m, the percentage of fines increases with increasing layer depth. The maximum diameter (Dmax) at these different depths is less than 50 mm, with a sieve size of 0.08 mm, which is also greater than 35%, except for the 0 - 2 m layer, which has a fines percentage less than 35%. Since the characteristics of slopes PK09 + 460 - PK09 + 800 are practically identical to those of the first and second slopes, we can assume that these three layer depths are class A.

3.2. Atterberg Limit

Tables 1-3 present the different results on liquidity limits, plasticity limits and plasticity indices.

Table 1. Values of the Atterberg limits of the embankment PK 01 + 760 - PK 02 + 020.

Depth (m)	Atterberg limits (%)		
	W_L	W_P	I_P
0-2	42.80	21.25	21.55
2-4	50.81	24.00	26.81
4-6	52	25.69	26.31
6-9	62.75	30.73	32.02

Table 2. Values of the Atterberg limits of the embankment PK04 + 500 - PK04 + 960.

Depth (m)	Atterberg limits (%)		
	W_L	W_P	I_P
0-2	46.70	24.27	22.43
2-4	52.80	26.09	26.71
4-6	56.40	28.07	28.33

Table 3. Values of the Atterberg limits of the embankment PK09 + 460 - PK09 + 800.

Depth (m)	Atterberg limits		
	W_L	W_P	I_P
0-2	57.20	29	31.50
2-3	55.90	29.14	55.89
3-6	60.70	29.58	85.49

The results in **Tables 1-3** show that the liquid limits (W_L), plastic limits (W_P), and plasticity indices (I_P) increase with depth. For W_L, the values range from 42.80% to 62.75%, from 46.70% to 56.40%, and from 57.20% to 60.70%, respectively. For W_P, the values range from 21.25% to 33.75%, from 24.27% to 28.07%, and from 29% to 29.58%. And the I_P values range from 21.55% to 32.02%, from 22.43% to 28.33%, and from 31.50% to 85.49%, respectively. These I_P results are presented in the Casagrande diagram [11] see **Figures 6-8** in order to determine the nature of the materials of the different slopes sampled.

The Casagrande diagram shows that the slope PK01 + 760 - PK02 + 020 exhibits slightly plastic clay soils at a depth of 0 - 4 m and highly plastic clay soils at a depth of 4 - 9 m. The slope PK04 + 500 - PK04 + 960 exhibits slightly plastic soils at a depth of 0 - 2 m and highly plastic soils at a depth of 2 - 6 m. As for the slope PK09 + 460 - PK09 + 800, the soils are all highly plastic clay, regardless of the sampling depth.

3.3. CBR Bearing Capacity Characteristics of Embakment

The results of the CBR tests at 95% of the Modified Proctor Optimum (MPO) carried out after 4 days of immersion in water will be presented in summary tables.

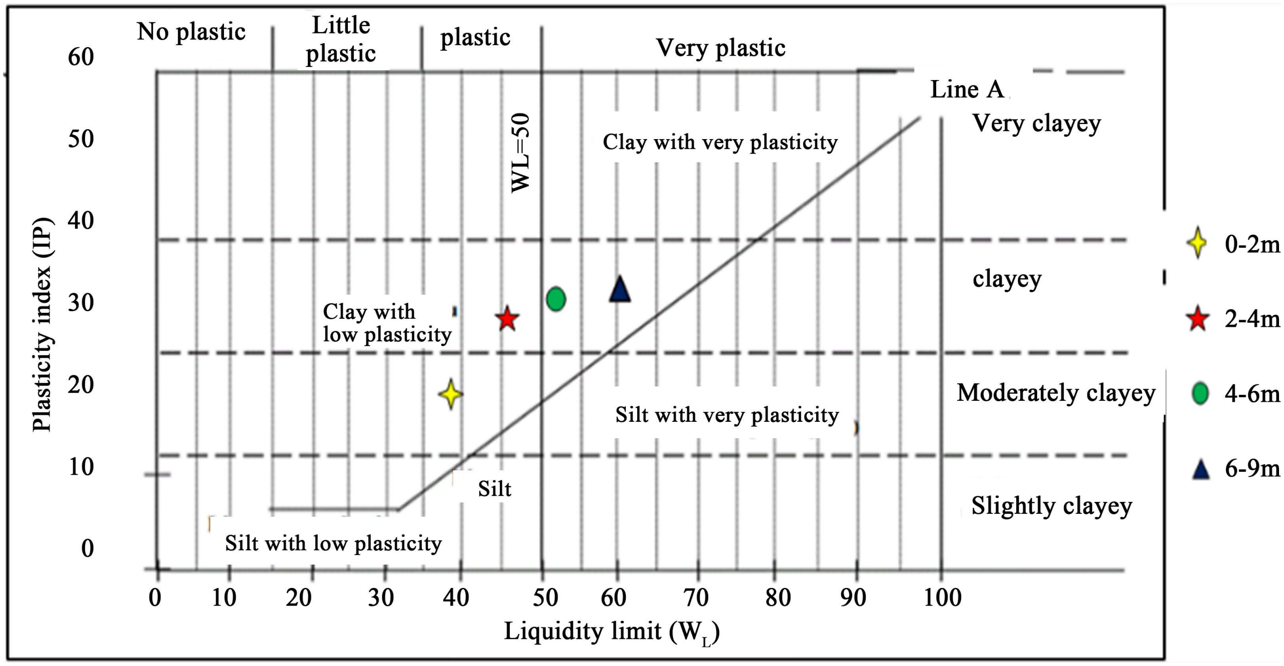

Figure 6. Position of the different depths of the disturbed soils of the embakment PK01 + 760 - PK02 + 020.

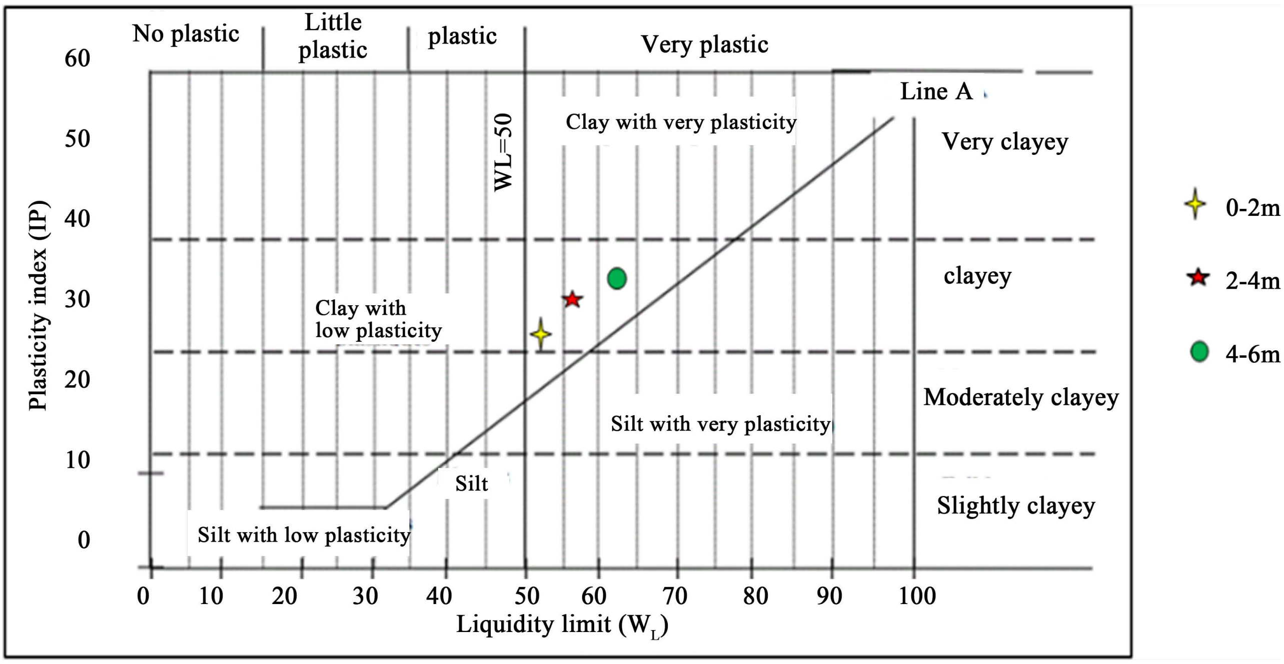

Figure 7. Position of the different depths of the disturbed soils of the embakment PK04 + 500 - PK04 + 960.

3.3.1. Resistance to CBR Approached at the Embakment PK01 + 760 - PK02 + 020

Table 4 presents the characterization parameters of the slope formations PK01 + 760 - PK02 + 020 as well as the CBR bearing indices.

The results are presented according to the different time horizons.

- Horizon A (0 - 2 m) corresponding to lateritic gravels exhibits the highest bearing capacity parameters with an average CBR of around 38.20% for a

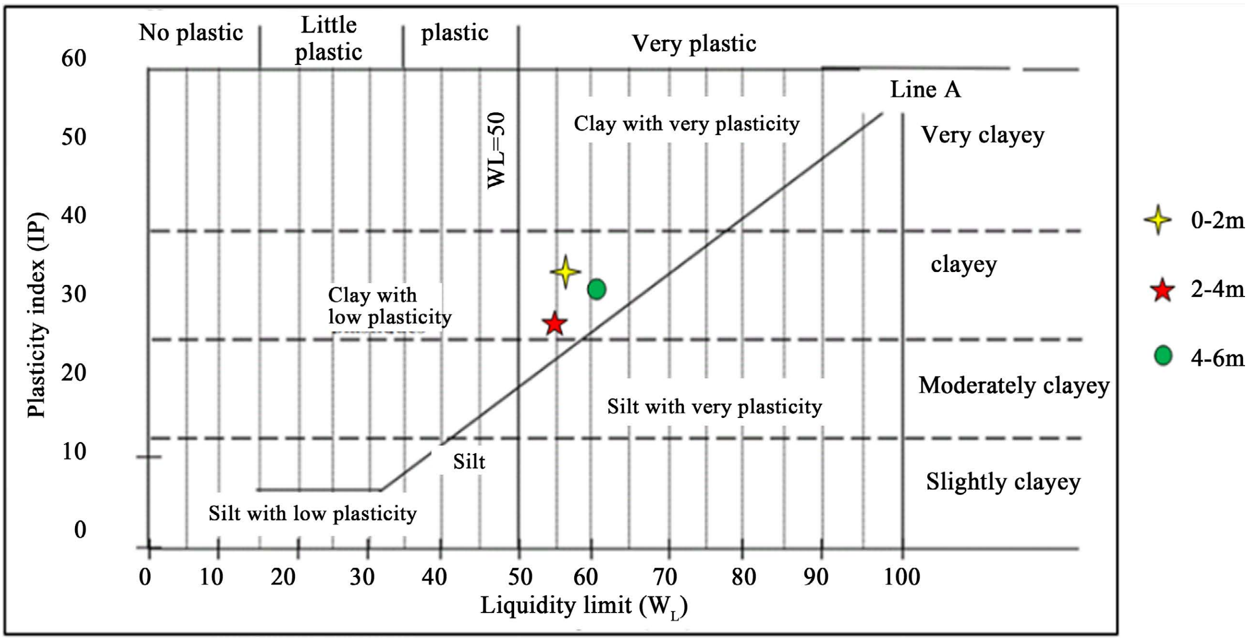

Figure 8. Position of the different depths of the disturbed soils of the embakment PK09 + 460 - PK09 + 800.

Table 4. Characterization parameters of the embakment formations PK01 + 760 - PK02 + 020.

Settings	Embakment Formation			
	A	B	C	D
Dry density MPO (T/m^3)	2.21	1.74	1.58	1.55
Water content MPO (%)	9.40	23.82	25.82	28.80
Water content of the material W (%)	2.04	6.84	6.80	3.72
Raw CBR Lift Index	38.20	25	12	7

density at MPO of around 2.21 T/m^3. These materials generally exhibit good mechanical performance, combining both the frictional character resulting from the presence of a significant percentage of gravelly elements with a certain cohesion resulting from the presence of fines at a level of 22%, thus ensuring a certain cementation between the gravelly elements of the material.

- Horizon B (2 - 4 m) corresponding to slightly gravelly clay whose formation generally presents average bearing capacity and stability parameters with a dry density at modified MPO of the order of 1.74 T/m^3 against a CBR index at 95% of MPO of the order of 25%.
- Horizon C (4 - 6 m), corresponding to reddish clay, is the dominant formation along the project axis. Based on identification parameters, these materials are characterized by an average fines content of approximately 59%. Overall, these materials exhibit low bearing capacity and stability parameters, with a modified Proctor optimum dry density of 1.58 T/m^3 compared to a CBR index at 95% of the MPO of approximately 12%.
- Horizon D (6 - 9 m) corresponding to the schistose alteration clay presents the

weakest bearing parameters with an average CBR of the order of 7% for a dry density at MPO of the order of 1.55 T/m^3, a liquid limit equal to 63% with a water content of 28.80%.

The results for the slope at PK01 + 760 - PK02 + 020 show that, from 0 to 6 meters deep, the materials constituting this slope are favorable to its stability, according to the values obtained. However, from 6 to 9 meters deep, the soil is loose with unfavorable characteristics that influence the stability of this embakment, with a CBR value below 10.

3.3.2. Resistance to CBR Approached at the Embakment PK04 + 500 - PK04 + 960

The different characterization parameters of the formations of the slope PK04 + 500 - PK04 + 960 are presented in Table 5.

Table 5. Characterization parameters of the embakment formations PK04 + 500 - PK04 + 960.

Settings	Embakment formation		
	A	B	C
Dry density MPO (T/m^3)	2.21	1.86	1.69
Water content MPO (%)	9.40	14	25.51
Water content of the material W(%)	2.04	3	6.50
Raw CBR Lift Index	38	27	16.01

The results in Table 5 show that:

- The A horizon (0 - 2 m), corresponding to lateritic gravel, exhibits high bearing capacity parameters with an average bearing capacity index of approximately 38% for a dry density at the MPO of approximately 2.21 t/m^3. The fines content is approximately 22%. This horizon, composed of lateritic gravel, demonstrates good mechanical performance due to the presence of large gravel elements interspersed with fine particles that act as cement between the larger elements. The values are virtually identical to the A horizon of the slope PK01 + 760 - PK02 + 020.
- Horizon B (2 - 4 m), corresponding to the slightly gravelly clay, is an intermediate formation between lateritic gravels and variegated clays. This material generally exhibits average bearing capacity and stability parameters, with a dry density at the MPO of approximately 1.86 t/m^3 and a CBR index at 95% of the MPO of 27%. This depth is characterized by a relatively high percentage of fines (43%), an average plasticity index of approximately 27%, and a liquid limit of 53%.
- The C horizon (4 - 6 m), corresponding to the yellowish-red variegated clay, exhibits relatively low but acceptable bearing capacity and stability parameters according to LBTP recommendations. Indeed, the dry density at the OPM (Oil Pressure Measure) is approximately 1.69 t/m^3, with a CBR index at 95% of the average OPM, which is approximately 16.01%. This clay is characterized by a very high percentage of fines (59%), a liquid limit of 56%, a plastic limit of

28%, and a plasticity index also of 28%.

An embakment is considered stable when the materials constituting it meet the following conditions:

- A fines content of 60% or less
- A maximum dry density of 1.50 or greater
- A liquid limit of 60% or less.

Based on the characteristics of slope PK04 + 500 - PK04 + 960, it is therefore deemed stable.

3.3.3. Resistance to CBR Approached at the Embakment PK09 + 460 - PK09 + 800

Analysis of the results from this Table 6 shows that for:

Table 6. Characterization parameters of the embakment formations PK09 + 460 - PK09 + 800.

Settings	Embakment Formation		
	A	B	C
Dry density MPO (T/m^3)	1.76	1.68	1.59
Water content MPO (%)	17.20	19.50	26.20
Water content of the material W (%)	6.29	6.70	2.90
Raw CBR Lift Index	25	16	9

- Horizon A (0 - 2 m) corresponding to the reddish, slightly gravelly clay

This material is characterized by an average fines content of approximately 32%, compared to an average plasticity index of 28%. The liquid limit is approximately 57%, while the plasticity limit is 29%. This material exhibits good bearing capacity and stability, with a dry density at 95% of 1.76 t/m^3 and a CBR index at 95% of 25%. The mechanical performance of this material is likely due to a good distribution of coarse elements, which constitute the gravel, with the fine clay particles, which act as a binder.

- Horizon B (2 - 3 m) corresponding to reddish lateritic clay

This reddish lateritic clay has a high percentage of fines, around 56%, compared to an average plasticity index of 27%. The liquid limit is also around 56%. This material exhibits low bearing capacity and stability, with a dry density at 95% of 1.68 t/m^3 and a CBR index at 95% of 16%. The poor mechanical performance of the material is likely due to a significant amount of fine particles, which are highly plastic clay. This clay is prone to swelling because it is very sensitive to water.

- Horizon C (3 - 6 m) corresponding to the shaly weathered clay

The shaly weathered clay exhibits low bearing capacity parameters with an average CBR of approximately 9%, a dry density at the OPM of approximately 1.59 t/m^3, a liquid limit of 61%, and a water content at the OPM of approximately 26.20%. This material appears to be an extension of the slope PK01 + 760 - PK02 + 020, which is 6-9 m deep. A very high percentage of fines (85%) is observed, along with a high plasticity index of 31% and a very high liquid limit of 61%. The poor performance of this layer is likely due to the presence of weathered shaly rocks of very low hardness. These clays of schistous origin are very plastic and sensitive to water, therefore prone to swelling.

It should be noted that the slope between PK09 + 460 and PK09 + 800 is considered unstable based on the characteristics of its materials. From 0 to 3 meters deep, the materials exhibit characteristics favorable to slope stability. Beyond 3 meters, we find shale with unfavorable characteristics. The fines content is 85.49%, exceeding 60%, the CBR (Cellular Boundary Ratio) is 9, also below 10, with a dry density of 1.59 and a water content of 26.20%. These characteristics are unfavorable and compromise the stability of this slope.

The fines content remains above 35% for almost all three slopes. It is also noted that the maximum particle diameter of these formations is less than 50 mm. These soils belong to class A according to the classification in the French Road Earthworks Guide (GTR).

4. Discussion

The particle size analysis study shows that the percentage of fines obtained at different depths is greater than 20%. These results contradict those of [12] and [13], who obtained between 10% and 13% fines in lateritic gravels. Regarding the plasticity index, [14] and [15] obtained plasticity indices of 19.6 and 19.46, respectively. It should be noted that these authors worked with lateritic gravels for backfill materials used in subgrade layers. For these types of materials, the French Building and Public Works Laboratory (LBTP) [16] recommends a plasticity index of less than 20%. However, for excavated materials, the plasticity index should be greater than 20%, as recommended by the LBTP. Our results are consistent with those of Kouakou (2019), who also worked with this type of material.

For the CBR bearing capacity study of the PK01 + 760 - PK02 + 020 slope, the water content results determined during the work are identical to those of [15] [14] and [12]. Regarding dry density, Hien and Millogo obtained 2.28 t/m^3 and 2.19 t/m^3, respectively. These values are consistent with the results obtained for the lateritic gravels of the two slopes. However, concerning the dry density values for the lateritic gravels at the OPM, Mahamat (2012) obtained high dry densities ranging from 2.35 t/m^3 to 2.44 t/m^3, which differ from the results obtained during this work (2.21 t/m^3).

For the slope PK04 + 500 - PK04 + 960, the results of the bearing capacity index of the lateritic gravel studied on our slopes give a bearing capacity index greater than or equal to 38%, which is in accordance with the recommendations of the LBTP which advocates a bearing capacity index greater than 30. These results are consistent with those of [17] which recorded a minimum bearing capacity index at 95% of the OPM equal to 30 and [18] which obtained a maximum bearing capacity index at 95% of the OPM equal to 66.

For the slope between PK09 + 460 and PK09 + 800, the plasticity index used to characterize soil angularity presents unfavorable characteristics in terms of water sensitivity. This characterization gives these soils a clayey nature, ranging from slightly to highly plastic and deformable, with Plasticity Indices between 20% and 32%. These soils are of the clayey and clayey-gravelly type, without any trace of gravel.

According to [16], soils with a Plasticity Index (PI) greater than 15 are plastic. The GTR indicates that these formations belong to subclasses A1 and A2. LBTP confirms this result and adds that the plastic soils are only slightly deformable. However, the clayey nature is implicated in the issue of slope stability. The results of the soil samples are suitable for providing a good platform soil for road construction, with the exception of certain depths such as the depth of 6 to 9 m for slope 1 and 3 to 6 m for slope 3.

Regarding the minimum CBR values at 95% of OPM after 4 days of water immersion, permitted for raw materials used in road construction, are 30% and 80% respectively, according to [19]. Taking these geotechnical parameters into account, the samples studied are suitable for the construction of a pavement base course and for the integrity of cut embankments.

5. Conclusions

Based on the results concerning the stability of the different embankments studied, it appears that the embankment PK01 + 760 - PK02 + 020 exhibits materials favorable to its stability from 1 to 6 m in altitude, with acceptable CBR values. Beyond 6 m in depth, the soil becomes loose with characteristics unfavorable to its stability.

The embankment PK04 + 500 - PK04 + 960, at an altitude of 5 m, shows stable material characteristics. This is not the case for the embankment PK09 + 460 - PK09 + 800. From 0 to 3 m in depth, the materials of this embankment exhibit characteristics favorable to its stability. Then, beyond 3 m in depth, the characteristics become unfavorable, leading to instability.

Conflicts of Interest

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

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