Geotechnical Investigation of the Bamenda-Bambili Road (13 Km) in View of Rehabilitation and Widening

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Abstract:- The fast deformation and disintegration of pavements in tropical zones are seemingly becoming permanent phenomenon and these issues are increasing evidenced in most regions of Cameroon. This work focuses on the rehabilitation and widening of the Bamenda-Bambili road, Northwest Region of Cameroon. There was a preliminary field survey through the deflection and penetrometer tests at different sections along this road stretch. It was an experimental and observatory study adopting the quantitative and qualitative methods. Samples were collected and transported to the laboratory, where identification tests were carried out. The data obtained from the field survey and identification tests were used to redesign the road. The result of deflection survey shows that deformation ranges from 5/100 mm to 118/100 mm and belongs to Q2 and Q5 which corresponds to maintenance and rehabilitation respectively. Further results showed that an average resistance of 0.5 MPa was found at a depth of 5 m. The physical analysis of the in-situ material showed an average moisture content of 20%, specific gravity of 2 and plasticity index of 25.1%. The grain size analyses of subgrade soils showed fine lateritic soils (A-2–7, A-6, A-7-5 and A-7-5) according to the HRB classification. The mechanical analysis showed an average maximum dry density of 1.7 t/m^3 and an average bearing capacity of 20.6% which correspond to Class S3. These materials were recommended to be use in sub-base of the pavement for T2. The traffic, T3 was previewed for the widening of this road, thereby necessitating the stabilisation of the base course. The design with Alizé software showed a total thickness of 50 cm subdivided into three layers; wearing course in bituminous concrete (5 cm), base layer ameliorated with 0/31.5 (25 cm) and lateritic sub-base (20 cm). The study found that pavement failure along the Bamenda-Bambili road could be due to ageing and high-water content of the sub grade soils and suggest that holistic and objective survey and planning be done along this road to improve it state

***Keywords**:- Bamenda-Bambili Road, Geotechnical Investigation, Rehabilitation, Redesigns.*

I. INTRODUCTION

Roads are one of the important assets of every nation serving as a means of transport for people and goods but also providing avenues for such service apparatus as water, electricity and sewerages (Emeasoba *et al*., 2013). The provision of good quality road infrastructure usually brings about the growth of new businesses and attraction of firms to less developed areas. Generally, roads start undergoing degradation immediately it is put into service, hence there is need to take necessary measures to control and curb with this degradation and increase the life span of the road. In this light, most nations have adopted geotechnical studies prior to road constructions to improve their transport systems. Geotechnical investigations involve identifying geological and geotechnical hazards as well as materials that could be used during road construction thereby designing resilient roads capable of withstanding natural challenges. Roads have demonstrated worldwide to be the most effective and preferred mode of transportation for goods and persons (Owolabi, 2012). Road transport gain popularity due to its ability to provide better accessibility through door-to-door services and its suitability for short haulage of passengers and freight

Constraints of road development have once been barriers to the economy of Japan, precisely in the Palau region. Palau had to establish the "Public Sector Investment Program (2003-2007)" in April 2003 as its national development plan for public sector. This Program gives the highest priority and urgency on the improvement/development project of metropolitan trunk roads. The majority of existing trunk roads in the capital area were constructed during the former Japanese administration era. Although these trunk roads are still in services, damages of roads are remarkably progressing in recent years, due to the insufficient road structures to cope with the traffic volume, which shows rapid growth in these years, and latest vehicle size, which becomes bigger and heavier than before. (JICA, 2004). The four islands, which form the metropolitan area, are connected by bridge and causeways. As each causeway attaches public utilities of water supply, sewer, power lines and telecommunications, these are the lifeline of metropolitan, and given a quite important role to Palau. When the old KB Bridge, connecting Aria area and Koror, was collapsed in September 1997, Palau Government dared to

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declare the state of emergency. The causeways are having structural troubles such as collapses of bank slope protection, pavement depressions caused by the fill material loss due to suction behaviour of tide, or self-collapse of culverts installed in causeways, which are beyond the affordable scale of repair by Palau (JICA, 2004)

In Nigeria, road transport is the most affordable and efficient means of transport to the majority of people as other modes of transport are either too expensive or not fully developed, thus resulting to a rise in the construction of roads. Consequently, there is excessive axle loads on the majority of the roads. (Owolabi, 2012). Nigerian roads are predisposed to structural failure after few years of performance and often before reaching design age. Komolafe (2006) stated that, “the state of Nigerian roads stands out like a sore thumb and their national picture is simply scandalous”. Reconstruction and rehabilitation of roads in Nigeria without even care to investigate the causes of the perpetual failure is very common. Akintorinwa et al., (2011) reported that several causes such as geotechnical, geological, geomorphological, hydrological, design, material selection, construction practices, maintenance and usage-factor can influence the performance of pavement structures. Gidigasu, (1976) also reported that the prevalent deterioration and failure of Nigerian roads have been attributed to the indiscriminate use of lateritic soils without full knowledge of their limitations

According to the Note on sustainable maintenance strategy for earth roads in the Ministry of Public Works, Cameroon, (2021), the road network in Cameroon covers about 121,501.5 km of which about 7,786.5 km is paved, 113,716 km is unpaved (93.6% of the total road network) and 98,130 km being council roads. This limits trade exchanges between border countries and the link between the hinterland (high agricultural productivity areas) and the biggest cities. Consequently, this constrained the socio-economic development of Cameroon.

II. STATEMENT OF THE PROBLEM

Road transport development has been driving mechanism to the progress or improvements in developmental activities in Cameroon such as improve socio-economic and touristic activities, spatial interactions, and the development of secondary towns as well as the effective distribution of its agricultural products both at the national and international level. Although at the early stage of development most of the roads often look so reliable and fit for service delivery, in the long run there is always an unexpected degradation of these roads which stemmed most of the aforementioned activities. Most of the roads are characterised by collapse, potholes and sinking which delayed movements and animates peak traffic jam especially during rush periods of the day. Plethora of measures has always been implemented by the government to improve the conditions of these roads but they yield temporal successes and everything become futile. The Bamenda-Bambili stretched remains one of these roads that is highly sensitive to rapid degradations. Despite the measures put in place by the Cameroon government in the domain of geotechnical investigation of road in view of rehabilitation or widening (Bamenda-Bambili), it is not meeting up with its demands in terms of quality (Guide for geotechnical studies in view of rehabilitation of roads and bridges, MINTP, 2018).

These are explained by the inadequate geotechnical investigations resulting from insufficient geotechnical experts, poor sampling techniques and procedures, insufficient equipment, poor supervision, limited funding, and over or under designing of structural elements which eventually contribute immensely to project cost over-run, and poor performance. This usually leads to the abandonment of the projects as well as the increase in the project’s duration.

III. LITERATURE REVIEW AND FRAMEWORK

Geotechnical engineering is the study of the engineering behaviour of the ground (Charles, 2005). This discipline adopts the principles of soil mechanics and engineering geology to investigate surface and underground conditions of soil aggregates to determine. The stability and durability of civil engineering structures (for instance, buildings, highways, dams and bridges) are dependent on the stability of soil used for foundation or as construction materials (Laskar and Pal, 2012). Earlier studies on geotechnical investigation according to Nwankwoala and Amadi (2013); Avwenagha *et al* (2014); and Nazir (2014) have shown that engineers, project managers, and other built environment practitioners use information acquired from geotechnical investigation to design and effectively manage these projects to meet project constraints (time, cost, quality). According to Adepelumi *et al.* (2009), it enhances the knowledge of the character of the soil aggregate which bears the load to be transferred by the proposed structure. According to Feld (2005), the geotechnical investigation usually comprises of site geological survey, topography survey, geophysical survey, in-situ testing, and laboratory testing.

➢ *Phases of Geotechnical Investigation*

Geotechnical investigation is a continuous process, lasting throughout the project development process. Figure 1 shows the interrelationship between the geotechnical investigation and the project cycle.

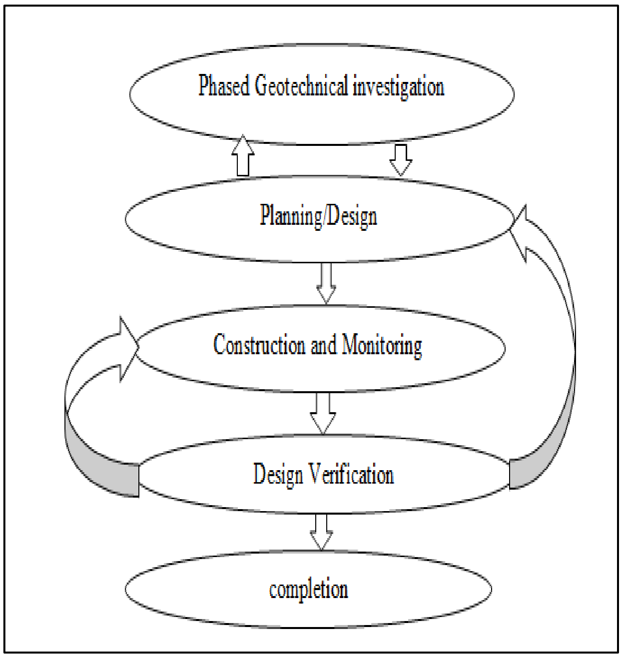


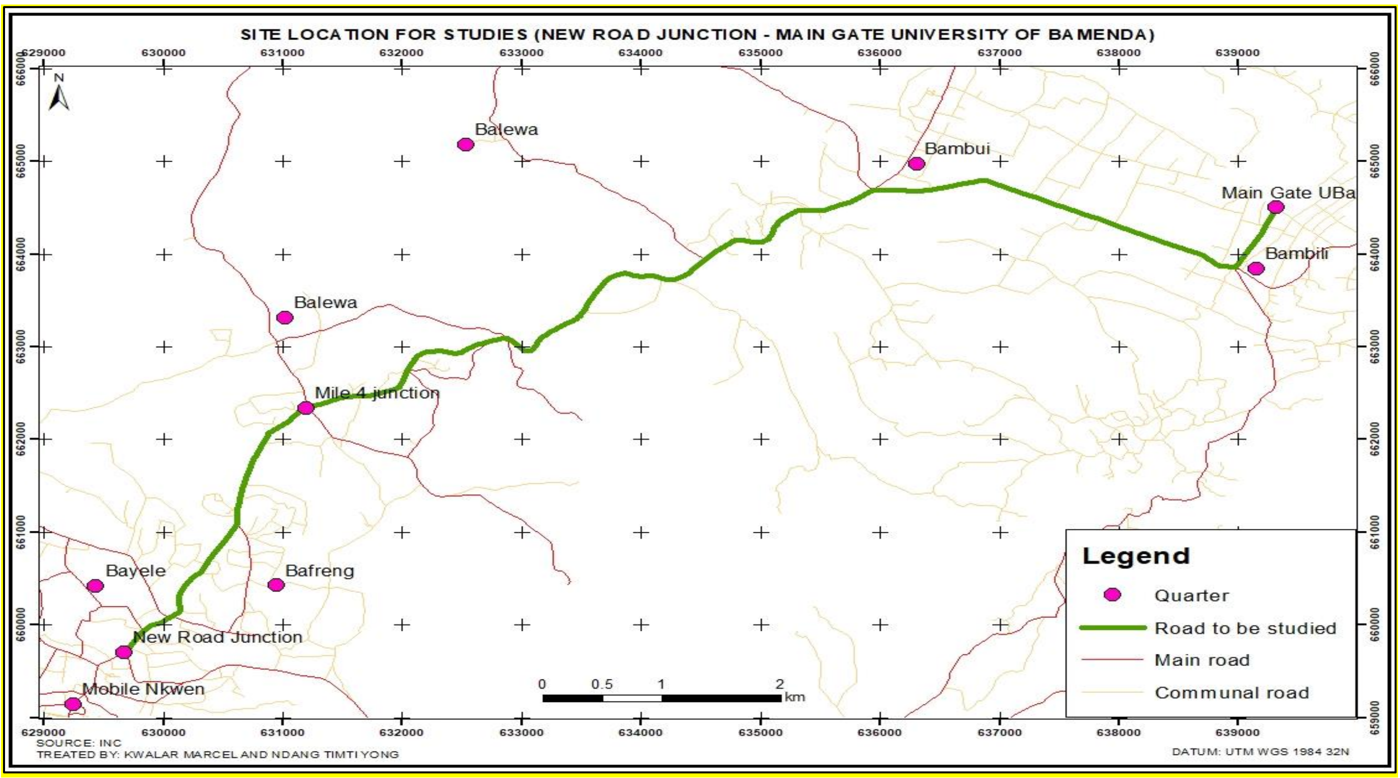
Fig 1: Phased Geotechnical Investigations with Project Development Process
(Adopted from Hung et al., 2009)

Geotechnical investigation is usually carried out in phases, many scholars highlight the phases involved as preliminary investigation or desk study, detailed investigation, and investigation during construction (Baecher and Christian, 2003; Zumrawi, 2014; Albatal *et al.*, 2014; Myburgh, 2018). The initial phase involves carrying out a desk study or acquiring geological information about the region. Myburgh (2018) notes that the desk study involves review of existing records, detailed study. After the initial phase, a detailed investigation is carried out to obtain data through in-depth exploration, sampling, measurement, physical examination, laboratory tests, and analyses of both surface and subsurface soils. Although this phase may be regarded as the costliest, it is however, the most cost-effective phase of the investigation process by reducing the potential for unforeseen ground risks. The investigation during construction phase is mainly aimed at enhancing previous findings of preceding phases of the investigation (Myburgh, 2018). This investigation is carried out during earthwork or construction of foundation; therefore, it is imperative that geotechnical investigations be conducted and supervised by a qualified and experienced professional to guard against the observation by (Charles, 2005).

IV. RESEARCH APPROACH

➢ *Location of Study*

The Bamenda-Bambili road stretch (Figure 3.1) is part of the National Road number 11 and is in Mezam division of the North West Region. The road passes through two subdivisions namely: Bamenda III and Tubah Subdivisions. With a total length of approximately 13.425 km, the road stretch starts from New road junction (PK 0+00) to main entrance into the Bamenda University campus, Bambili (PK 13+425). It is an existing paved road with a carriageway of approximately 5-6 m, with high degradations and disintegrated section.



Map 1: Bamenda-Bambili Road Stretch
Realized by Bamenda Municipal Council (2023)

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V. RESEARCH METHODOLOGY

The study adopted an experimental and descriptive design blinding the quantitative and qualitative research techniques. The experimental design helped the researcher to collect soil samples from various targeted areas which were taken to a laboratory for analysis. The descriptive approach eases the observatory process which helped the researcher to be acquainted with the nature and the deteriorating condition of the road. The blinded techniques (Quantitative and qualitative) helped the researcher to tap the advantages enshrined in them and to reduce their inherent biases.

Soil sampling along the Bamenda-Bambili road was done both on the paved and unpaved section. On both sections, soils samples were collected from trial pits of dimension 1x 0.8 x 1.2 m (length, width and depth), excavated using spades and dig axes, at 250 m interval, along the road stretch, alternating from the left to right ends of the road. This alternation was to get a better representation of the platform or sub grade. For the unpaved section, sampling did not necessarily follow the contours of the paved section such that less pits (47) were made for the unpaved section compared to the paved section (55). The sample collection points were marked using a GPS. Figure 3.7 and Figure 3.8 presents the soil sampling points for the paved and unpaved road sections respectively. The collected soils samples were placed in clean plastic bags of approximately 40 kg (sealed to preserve their natural water content) each and transported by a pick-up vehicle to the laboratory for determining geotechnical characteristics.

As concerned the borrowed materials, eight samples of lateritic soils were collected from two lateritic borrow pits, one in Nkwen and the other in Mankon. For each borrow pit, four 1.5 m deep trial pits, spaced 50 m from each other were made and soil samples obtained. Three (03) types of sand (Mbatu, Bafut and Mbengwi) were sampled in a sand park at Amour Mezam, Mile 2 Nkwen. GPS Coordinates of the trial pits at GTHS Nkwen and Mankon are shown in Table 3.3. The samples were placed in clean plastic bags, sealed to conserve the natural water content and transported to the laboratory. Gravel 4/10 and 10/14 were also sampled from Dreamland and Kendely quarries, located in Mankon Bamenda and transported to the laboratory. For the lateritic borrow pits, the thickness of the soil layer (minus the dark organic top soil) observed along road cuts were used to estimate the volume of material using a visual estimated surface area (Length x width). To take into consideration the cost effectiveness of road constructions, all borrowed materials were sampled from borrowed pits or material parks less than 15 km to the road stretch.

Table 1: GPS Coordinate of the Trial Pits

Reference Pk	**Ref**	**GPS coordinates (32N)**		
		X	**Y**	**Z (m)**
Borrow Pit at GTHS Nkwen	Pit 1	629744	661157	1380
	Pit 2	629821	661227	1374
	Pit 3	629662	661253	1372
	Pit 4	629790	661364	1382
Borrow pit Mankon	Pit 1	622378	664166	1426
	Pit 2	622414	664250	1431
	Pit 3	622676	664231	1427
	Pit 4	622350	664218	1430

Source: Fieldwork (2023)

A. *Fieldwork and Procedures on Onsite Geotechnical Study of the Subgrade*

➢ *Deflection Test Using the Benkelman Beam.*

Using a spacing interval of 50 m along both the right and left sides of the road stretch, a total of two hundred and sixty (260) deflection tests was carried out on the paved section. The NFP 98-200-2 standard was used wherein the Benkelman beam with its comparator is placed on a paved section, in between the dual rear tires of a 13-ton loaded truck. The truck is moved forward by 2.7 m and the vertical displacement recorded is read on the sensor or dial gauge. In addition to the deflection measurements, we described the degradation of the road from visual inspection along the roadway, following regular procedures. This combination of the deflection measurements with visual inspection is necessary when proposing solutions for treatment of the roadway.

➢ *Dynamic Cone Penetrometer (DCP) Tests (ASTM D6951-03).*

This test evaluates the strength and the thicknesses of the underlying layers. At an interval of 4.3 km, three DCP tests were conducted at points PK 0+100, PK 6+400 and PK 12+500. At each drilling point, the tar (5 cm) was removed and sounding rods of the DCP were drilled into ground using an 8 kg hammer. The number of blows for every 10 cm penetration of the cone was recorded which was used to measure the strength and thickness of the underlying soils layers. For cases where more than 30 blows were counted for a less than 10 cm penetration of the sounding rod, refusal was considered.

➢ *Heavy Weight Penetrometer Test*

The heavy weight penetrometer test was done to determine the resistance of the soil and the depth at which foundations can be implanted. Two (02) heavy weight penetrometer tests were carried out on the abutment of the Mile 4 bridge, Nkwen Bamenda at PK 2+610, where there is

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the occurrence of seasonal flash flood. The NFP 94-115 standard was used wherein sounding rods were drilled into the ground using a 64 kg hammer and the number of blows for every 20 cm penetration recorded.

- *The Characteristic of the Equipment used are as Follows*

- ✓ Mass of hammer………………. ……………...64 kg
- ✓ Mass of anvil + guide + train of rod ……………. 14 kg
- ✓ Mass of one rod……………………………………7 kg
- ✓ Mass of load cell ………………………………0.89 kg
- ✓ Section of the load cell …………………………20 cm^2
- ✓ Height of fall……………….................................70 cm

The cone tip resistance (R_p) is calculated using the Hollandaise formula, which is expressed as follows. Equation 1

$$R_p = \frac{M^2.H}{A.e(M+P)} \ldots\ldots\ldots 1$$

Where:
R_p is the cone tip resistance in kg/cm².
M is the mass of the hammer,
P is the weight of anvil, guide, and train of rods,
H is the height of fall of the hammer
A is the area of the load cell,
e is the refuse that is penetrated by a rod in cm per hit of the hammer.

B. Laboratory Work

All laboratory tests were carried out at the Bambuiy Engineering Geotechnical laboratory, Bamenda, Cameroon. Test procedures were done according to the French norms as specified in Table

Table 2: Laboratory Tests with Applicable Standards Carried Out on the Sampled Material

Test	Standard	Soils	Sand	Aggregate
Natural water content	NF P 94 050		x	x
Grain size analysis	NF P 94-056			
Atterberg's Limits	NF P 94-051		x	x
Specific gravity	NF P 94-054		x	x
Proctor test	NF P 94-093		x	x
CBR (subgrade and borrow pits sample)	NF P 94-078		x	x
Sand Equivalent test	NF P 18-598	x		x
Micro Deval	NF P 18-572	x	x	
Los Angeles	NF P 18-573	x	x	

Source: Fieldwork (2023)

The sampled materials (soils, sand and aggregates) were washed and dried in the oven at 105 degrees for a period of 24 hours to enable all the samples to be completely dry. The samples were properly mixed and washed to remove all lumps in the sand and the soils. The aggregate (4/6 and 10/14) were air dried before its characterization. Table 3.5, summaries the field and laboratory work executed on paved and unpaved section.

Table 3: Field and Laboratory Works Executed on Paved and Unpaved Section

Field investigation/Laboratory test	Existing Pavement		Zones of Enlargement	
	Executed	Quantities	Executed	Quantities
Measure of deflection (Benkelman beam)	260	260	--	--
Visual degradation of the road	13	13	--	--
Trail pits	55	55	47	47
Identification	55	55	47	47
CBR	55	55	47	47
Modified proctor test	55	55	47	47

Source: Fieldwork (2023)

C. Data Analysis

The specific gravity of the solids soil was calculated using equation 5.

$$G_S = \frac{W0}{W0 + (WB - WA)} \ldots\ldots\ldots 2$$

Where:
W_0 = weight of sample of oven-dry soil, g = W_{PS} - W_P
W_A = weight of pycnometer filled with water
W_B = weight of pycnometer filled with water and soil

D. Modified Proctor Test

- *Aim*

In carrying out the modified proctor test, we are interested in determining the maximum dry density and the optimum moisture content that can transmit maximum force of a structure to the soil safely, when compacted.

- *Requirements*

- Soil sample of about **7 kg**;
- A cylindrical mould for compaction;
- Mixing tools, such as trowels, spatula, spoon, etc;
- A cylindrical transparent and calibrated water cylinder;
- An open tray;

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- A manual rammer for compaction;
- A 5 mm sieve;
- An electronic balance.

➢ *Procedure*

- Weigh a quantity of air-dry soil equivalent to about 8 kg, and thoroughly mix the material with **125 mL** of water;
- Attach the mould, with collar, to the base plate and place the mould on a uniform, rigid surface that can withstand the force of the blows to be applied;
- Compact each of the 5 layers by **55 uniformly distributed blows** from the rammer, with the drop weight falling freely. In operating the manual rammer, care should be taken to hold the rammer vertical and avoid rebounding the rammer drop;
- Remove the extension collar from the mould. Remove the exposed compacted soil with a knife and carefully trim the surface even with the top of the mould by means of a straight edge. Any cavities formed by large particles being pulled out should be carefully patched with material from the trimmings;
- Remove the mould with the compacted specimen therein from the base plate, weigh the mould plus wet soil to the nearest gram, and record the weight on the data sheet;
- Take a representative specimen of the material from each of the two parts (above and below) and determine the water content of each;
- The procedure on the third point and the later are repeated. Compact enough test specimens over a range of water contents to establish definitely the optimum water content and maximum density.

E. Analysis and Calculation

Increasing Wc will increase γ_{dry} up to a certain limit (Optimum moisture Content,) after this limit, Increasing Wc will decrease γ_{dry}; Knowing the wet unit weight and the moisture content, the dry unit weight can be determined from Figure 2.

$$\gamma_{dry} = \frac{\gamma_{wet}}{1 + \frac{w_c(\%)}{100}}$$

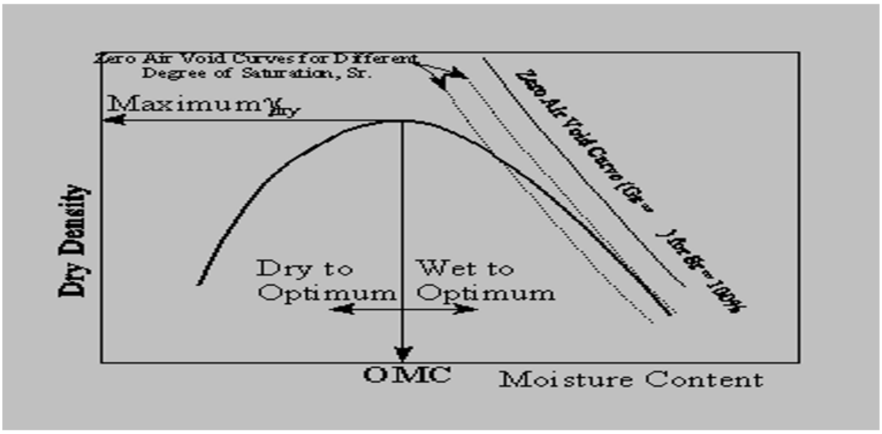

Fig 2: Moisture Density Relationship/Dry Density Formula

F. Califonian Bearing Ratio Test (CBR)

➢ *Aim*

The aim of this test is to determine the bearing capacity of the soil.

➢ *Principle*

This test is mostly carried out on sides where a high degree of compaction is needed as in roads and airports. This consists of determining the bearing capacity of a soil by experiment to:

- Establishing the classification of soil;
- Determine the thickness of road layers (as CBR increases the thickness of the road layer reduces).

➢ *Requirements*

- Three CBR standard moulds with base plate, Extension collar, and disc;
- Paper seals;
- A proctor hammer;
- Loads of moulds;
- Hydraulic press;
- A cylindrical piston;
- Graduated cylinders;
- Containers for mixing of material;
- A balance;
- An oven;
- A chronometer;
- A special spoon to fill materials in the mould.

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- *Procedure*

This test is mostly carried out after a proctor test has been conducted and the maximum dry density γdmax and optimum water content $\mathbf{W_{opt}}$ obtained and the volume of the mould known **V=2300 cm^3**. From here one could proceed to determine the quantity of materials needed for the test that will fill the 3 moulds at their different compaction energies as follows (equation 6):

γd_{max} **=2.03, W_{opt}=13.2%, V=2300 cm^3**

$$Pmat = \frac{\gamma d(w\%+100)V}{100} \text{...3}$$

With this known we can begin the test;

- Take a good quantity of soil such that when dried in an oven the weight will give **16 kg** as required;
- Put this material in the oven for **24 hrs.**
- Remove and sieve as in proctor;
- Take **16 kg** of it ready for the test and put in a container;
- Weigh each of the moulds including it base plate;
- Put water in to that material in the container and mix thoroughly observing because the water could be slightly over due to approximation sand scattering the lumps;
- Start with the mould chosen for the **25 blows;**
- Weigh each mould plus under and note their weights;
- Place the base plate in place and put the disc on it, fit the mould to it, place the seal on the disc to prevent adherence and fit the extension collar;
- Fill the mould in five layers and each layer receives two of the special spoons full and **25 blows**;
- After the fifth layer removes the extension collar and cut the excess material to level full the mould;
- Remove the under and remove the disc and the seal, weigh the mould with it content plus the under;
- Tie the mould to the disc now upside-down, now the space for the disc which was under is now up to receive the surcharge or load;
- Place the paper seal marking the numbers of blows on it and fit the surcharge this mould is now ready for immersion in water but all shall be immerged at the same time;
- From the rest of the material in the container take sample for the water content to deduce it;
- Fill the other moulds for the 55 and the 10 blows respecting their various blows per layer respectively in like manner;
- Immerge the three moulds now in water for 4 days;
- Remove from water and allow it to drain after removing the surcharge and you take the weight of each;
- Take the spacemen to the hydraulic press;
- Bring the piston nearest to the surface of the material in the mould and put it on. Compact for about **0.01 mm** to bring the pointer to zero;
- Maintain the pointer at zero and start reading the settlements corresponding to the following displacement 0.2 mm, 0.4 mm, 0.6 mm, 1 mm, 1.5 mm, 2.5 mm, 3 mm, 3.5 mm 4 mm, 4.5 mm, 5 mm, 6 mm. These displacements are measured at a rate of **1.27 mm/min**;
- Remove the material from each mould and pick out samples for water content from the print of the piston on top of the mould and from the bottom.

- *Analysis and Calculation*

Knowing the humid weight, water content and the volume of the mould we could then calculate the dry density (γd)

$$\gamma d = \frac{Ws}{V} = \frac{\text{dry weight}}{\text{Volume}}, \text{ With } Ws = \frac{100xP_h}{100+w} \text{.......................4}$$

The CBR is calculated from Stresses corresponding to settlement values of 2.5 mm and 5.0 mm i.e. 70 and 105 bars of pressure.

Therefore, $\quad \mathbf{CBR} = \text{Max}\left[\frac{P(2.5mm)X100}{7o}, \frac{P(5mm)X100}{105}\right]$

where **P** is piston penetration or settlement.

G. Sand Equivalence Test

- *Aim*

We are interested in determining the percentage purity of sand to know how to formulate its dosage in whatever engineering practice we are using it for.

- *Requirements*

- Washing solution (**125 g/5 liters** of water);
- A **5 mm** diameter sieve;
- **120 g** of sand and a funnel;
- A transparent plastic bottle to contain the washing solution;
- A tripled leg stands of 1m high from the ground;
- A **15 cm** ruler;
- 3 plastic and transparent calibrated (at **150 mL** and **380 mL**) cylinders with a water tight cork;
- A piston;
- An agitating machine.

- *Procedures*

- The washing solution is poured into the three measuring cylinders to the **150 mL** mark;
- The sand is first sieved through a **5 mm** sieve and then measured into a **120 g** container;
- The sand is introduced to each of the three cylinders at intervals of **3 min** each;
- **10 min** after introducing the sand to the washing solution, the cylinder is corked and clamped horizontally on the agitating machine and is agitated for **15 sec** to wash properly the sand;
- It is then rinsed still with the washing solution to the second mark of the cylinder (the **380 mL** mark). Same is done on the other two samples, and then left to settle for **20 mins**;
- After 20 minutes, a ruler is used to measure the height of the sand and the impurities from the base (Ho) and from the base to the sand (H1) A piston is used in measuring the height of the sand for accuracy purposes (H2)

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VI. RESULTS

➢ *Deflection Test*

The deflection results on the pavement show a range of 5/100 mm -118/100 mm on the right side of the road moving from Bamenda to Bambili while the left side show a deflection between 10/100 mm and 110/100 mm. The right side of the road presents a stronger maximum deflection as well as a broader range of deflection values. Analysis of maximums on each traffic direction shows that for T4 traffic, the bearing capacity of the pavement layers are low, and the structural quality of the road oscillates between dubious and bad (CEBTP, 1984.

➢ *Delimitation of Homogeneous Sections and Classifications*

Based on the average maximum deflection recorded on both sides of the road, three deflection classes for homogenous sections were established (Figure 4.2) and the length of the road (also expressed in percentage; Figure 4.3) that falls under the various classes as follows:

- Dc<60/100 mm: 4,375 km (33%)
- 60/100 mm ≤ Dc 75 /100 mm: 1,987 km (15%)
- Dc>74/100: 6,888 km (52%)

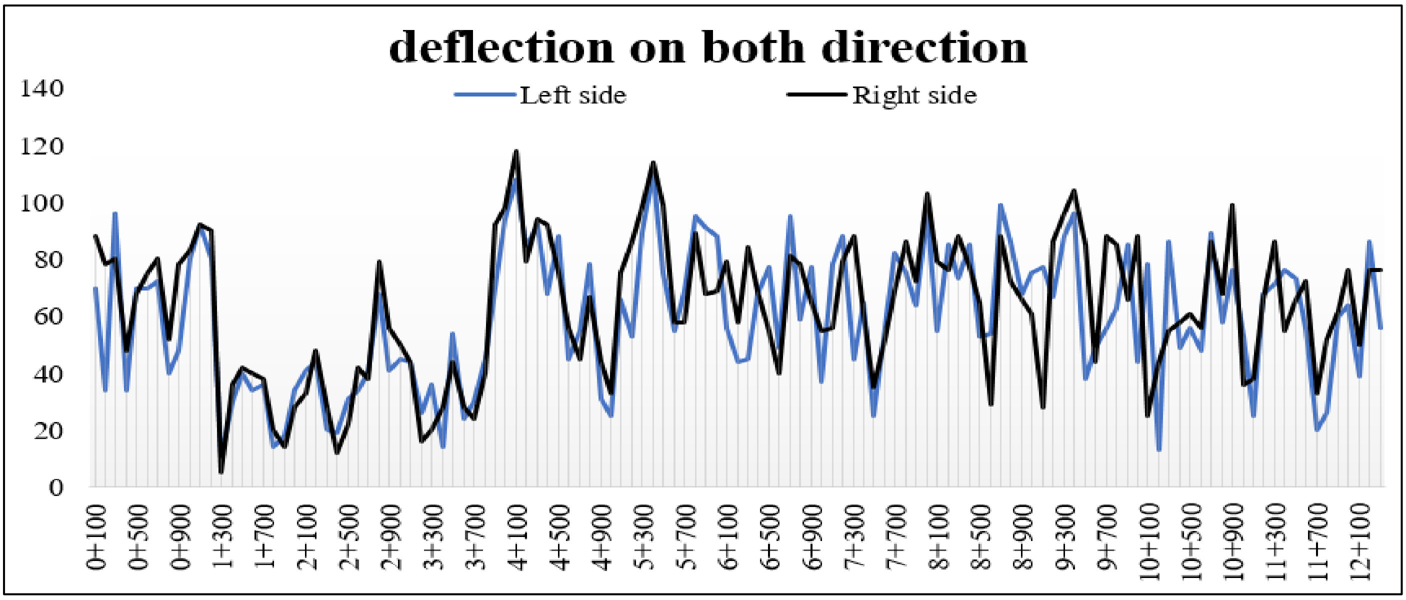

Fig 3: Deflection on Both Directions

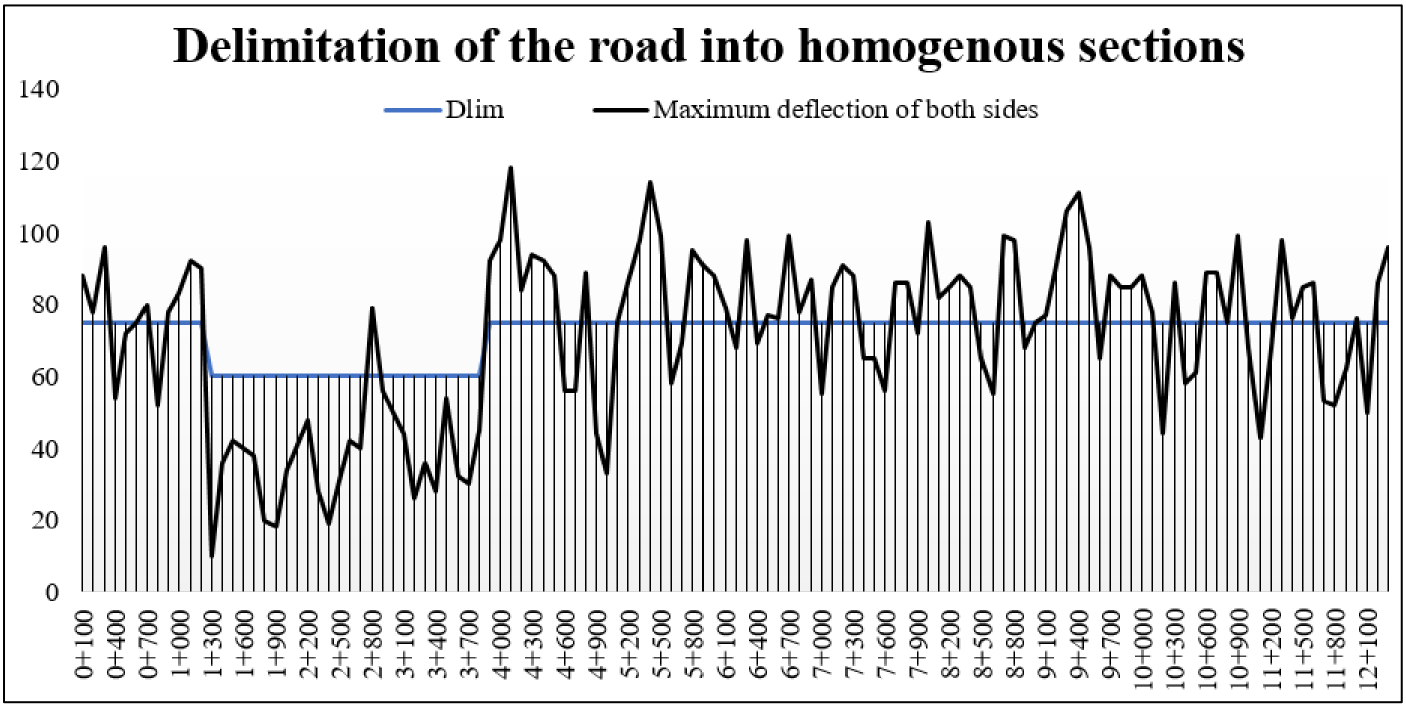

Fig 4: Delimitation of the Road in Homogeneous Sections

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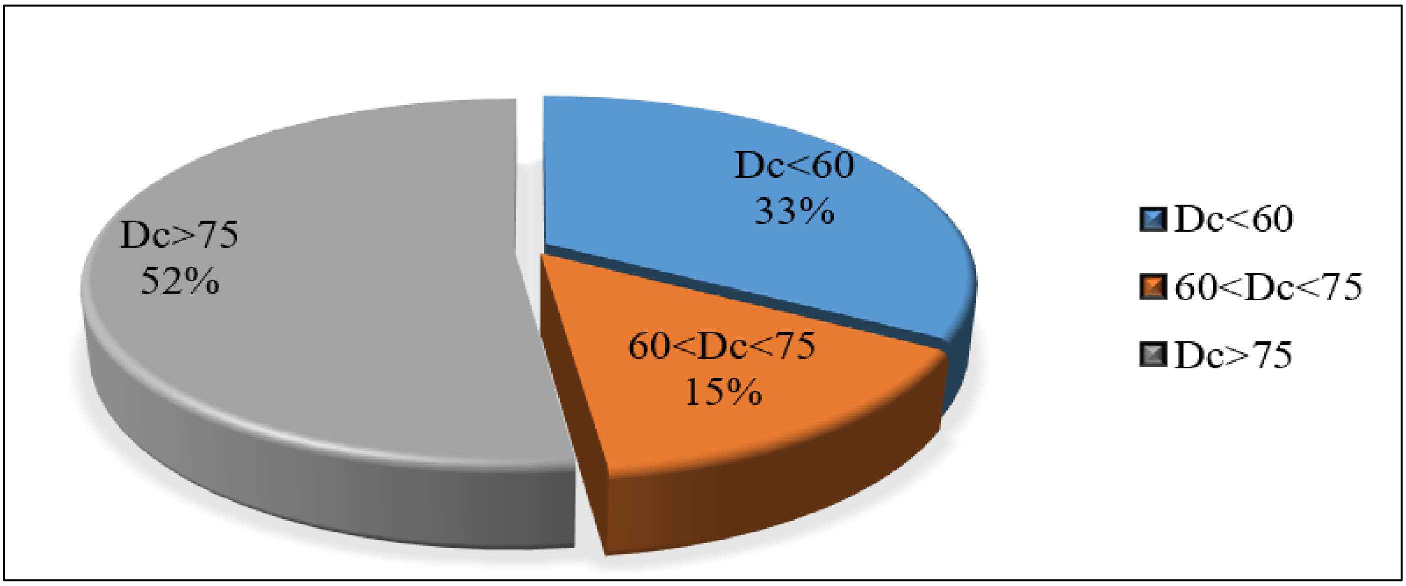

Fig 5: General Maximum Deflection

➤ *Result of Degradation Analysis on Homogeneous Sections*

The various degradations of the roadway have been inventoried and categorized. We have distinguished several types of disorders grouped into the following categories: cracking, disintegrations, and surface deformations.

Table 5: Delimitation of Road Sections into Homogenous Section Based on Visual Degradation.

Zone	Degradation	Localization	Linear ml)
Zone 1	Transversal cracks Longitudinal cracks Pot holes	Pk0+000–Pk 1+200	1200
Zone 2	Cracks	Pk1+300–Pk3+800	2500
Zone 3	Transversal cracks Longitudinal cracks Pot holes Disintegration	Pk3+900–Pk13+250	9350

➤ *Structural Quality of Homogeneous Sections*

By applying the principles of characterization of the flexible pavements outlined above to our project, we have the following results.

Table 6: Characteristic of the Flexible Pavement

	Pk start	Pk end	Average deflection	Maximum admissible deflection value for T4	Observation	Quality Qi
Zone 1	0+100	1+200	78.2	60/100 mm	Cracks and deformed	Q5
Zone 2	1+300	3+800	37.2		Cracks non-deformed	Q2
Zone 3	3+900	13+200	79.6		Cracks and deformed	Q5

The low deflection from PK 1+300-PK 3+800 is due to the reinforcement of this stretch, given that the higher the deflection value, the less stable the subgrade and the lower the deflection value, the more stable the subgrade. The pictures of the deflection test are indicated below,

➤ *Dynamic Cone Penetrometer Test*

The DCP survey points (PK 0+100, 6+400 and 12+500) falls within zone 1 (PK 0+100) and zone 3 (Pk 6+400 and 12+500) of the homogenous sections established from the deflection test. The DCP results (Table 4.4) reveal a layer of soil with thickness ranging from 284 mm to 386 mm and CBR values of the first layer falling within class S5 materials. This result shows an irregular succession of two layers, the last one probably the subgrade soil of the pavement. The CBR value of the layers shall not be used in the design of the pavement given that they are influenced by the climate and probably highly consolidated and impermeable. However, it gives an indication of the nature and continuity of the layers encountered.

Table 7: Summary Results of DCP Along Bamenda-Bambili Road

Layer	**Road stretch Bamenda - Bambili**				
	N0 DCP		**1**	**2**	**3**
1st layer	***Pk***		***0+100***	***6+400***	***12+500***
	Thickness (mm)		**284**	**386**	**355**
	DCP index	Min	0.3	0.7	1
		Max	2.8	2	2.1
		Average	**1.6**	**1.4**	**1.6**
	CBR	Min	100	85.5	92.5
		Max	100	100	100
		Average	**100**	**93**	**96**
2nd Layer	Pk		***0+100***	***6+400***	***12+500***
	Thickness (mm)		**--**	**--**	**--**
	DCP index	Min	3.9	4	3.2
		Max	7.7	5.4	4
		Average	**5.8**	**4.7**	**3.6**
	CBR	Min	42.2	44.4	62.5
		Max	65.2	62.5	74.8
		Average	**54**	**53**	**69**

Table 8: Summary Results of DCP Along Bamenda-Bambili Road

Layer	**Road stretch Bamenda - Bambili**				
	N0 DCP		**1**	**2**	**3**
1st layer	***Pk***		***0+100***	***6+400***	***12+500***
	Thickness (mm)		**284**	**386**	**355**
	DCP index	Min	0.3	0.7	1
		Max	2.8	2	2.1
		Average	**1.6**	**1.4**	**1.6**
	CBR	Min	100	85.5	92.5
		Max	100	100	100
		Average	**100**	**93**	**96**
2nd Layer	Pk		***0+100***	***6+400***	***12+500***
	Thickness (mm)		**--**	**--**	**--**
	DCP index	Min	3.9	4	3.2
		Max	7.7	5.4	4
		Average	**5.8**	**4.7**	**3.6**
	CBR	Min	42.2	44.4	62.5
		Max	65.2	62.5	74.8
		Average	**54**	**53**	**69**

➤ *Heavy Penetrometer Test*

On the point before the bridge moving from Bamenda to Bambili (SPD1), a maximum resistance of 0.34 bars was observed at a depth of 4 m with refusal at a depth of 4.8 m. For the point after the bridge (SP2), a maximum resistance of 4.84 bars was observed at a depth of 5 m with refusal at a depth of 5.20 m. Given that bed rock occurs at different depth, an average of the two should be considered as the depth of anchorage of the foundation

Table 9: Summary Table for the Heavy Weight Cone Penetrometer Test

Location	**PK**	**Admissible constraint σ_{adm} as a function of anchorage depth**		**Admissible constraint σ_{adm} as a function of anchorage depth**		**Water Level**
		SPD 1		**SPD 2**		
		x	**y**	**x**	**y**	
		630817	661928	630815	661934	
Bamenda-Bambili	2+610	Depth (m)	σadm (Bars)	Depth (m)	σadm (Bars)	**0.8 m**
		1.00	0.42	1.00	0.42	
		1.50	0.78	1.50	0.98	
		2.00	0.39	2.00	0.39	
		2.50	0.55	2.50	0.55	
		3.00	0.73	3.00	0.37	
		3.50	0.34	3.50	0.34	

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		4.00	0.34	4.00	0.34	
		4.80	Bedrock	5.00	4.84	
				5.20	Bedrock	

Conclusion: Anchor the Foundation on the Rocky Bed at a Depth of 5.0 m
NB** The Reference Level Taken as the Natural Surface of the Existing Road.

To solve the problem of seasonal flash floods on this bridge, filling with 0/100 mm aggregate (drainage layer) followed by lateritic materials to a thickness of about 2.5 m to 3 m on both sides of the road was proposed. This filling should begin from the Guinness deport (mile 3) to the bridge and about 10 m after which will enable water to circulate freely under the bridge and the drainage layer. This solution has been applied successfully along the Metazem-Bamenda road stretch, precisely at Mile 4 Akum.

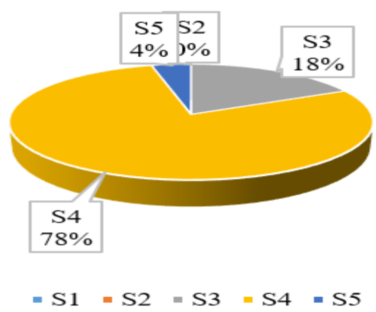
- *Geotechnical Analysis*

The results of complete identification of the materials from the paved and unpaved sections of the road were presented on table subsequent table below.

- *Paved Roads*

From Table 10, the natural water content ranges from 7.8 to 32.2%. The specific gravity of the sample analysed range between 1.595-2.594. From grain size analyses, our samples were fine lateritic soils (A-2–7, A-6 and A-7-5) according to the HRB classification. Figure 4.4 presents the upper and lower limits of the grain size curves for the soil samples. The soils present a liquid limit of 38.5 ≤ LL ≤ 65.5 and a plasticity index of 10.9 ≤ PI ≤ 39.3. The modified proctor results showed a maximum dry density from 1.510 to 1.935 t/m^3 with corresponding optimum waters content from 11.3% and 32.2%. At optimum proctor and 4 days of immersion, the soils samples showed CBR values between 12.9 and 32.2. They fall within the S3 (18%), S4 (78%) and S5 (4%) soil bearing capacity classes (CEBTP, 1984). Table 11 present the general laboratory results of trial pits from pavement material of the paved section of the Bamenda – Bambili road stretch.

Table 11: Summary Table of Laboratory Results of Materials from the Paved Road

Geotechnical parameters	Variation MIN-MAX	Average	Bearing capacity class of the platform		%
% Passing at sieve 80um	59≤%fines≤ 21	42			
Liquid limit	38.5≤LL≤ 65.5	48.6			
Plasticity index	10.9≤PI≤ 39.3	22.6			
Maximum dry density (T/m3)	1.935 ≤γ_d≤ 1.510	1.733			
Natural moisture content (%)	7.8 ≤W_{nat}≤ 33.3	19.4			
Optimal water content (%)	11.3 ≤W_{opt}≤32.2	18.7			
Specific gravity	1.596 ≤γ_s≤1.850	1.734			
CBR at 95% of opt after 4days of immersion	**12.9≤CBR≤ 32.2**	**20.6**	**S1**	**0**	**0%**
			S2	**0**	**0%**
			S3	**10**	**18%**
			S4	**44**	**78%**
			S5	**2**	**4%**

The difference in water content will ease 95% compaction of these grainy soils. Additionally, the local climate and vegetation are favourable for the rapid drying of this soil materials. From the specific gravity test results, this soil material could be used for fills. The plasticity index falls under medium to high plasticity. The bearing capacity (CBR> 36) of the soils was too lower than 80, recommended for the base course, thus we proposed that the said base course be mechanically stabilised with 0/31.5 mm crushed gravel

- *Unpaved Subgrade Material*

The Table below present the general results and Table 4.9 summarises it as; the natural water content of our samples evolved from 16.0 to 51.7%. These values are higher than those of the paved section. Their specific gravity was similar to those of the paved section and ranged between 1.565-2.400. The soils were clayey in nature (A-6, A-7-5, and A-7-6) according to the HRB classification. The upper and lower limits of their grain size curves are presented in Figure 4.5. The liquid limit and plasticity index from 40.8≤LL≤ 88.9 and 12.8≤PI≤ 40.8 respectively. For proctor, the maximum dry density was between 1.314 and 1.82 t/m^3 with corresponding optimum water content between 13.2 to 38.41. CBR values ranged from 8.6 to 28.3. They fall within the S2 (6.5%), S3 (43.5%) and S4 (50%) soil bearing capacity classes (CEBTP, 1984).

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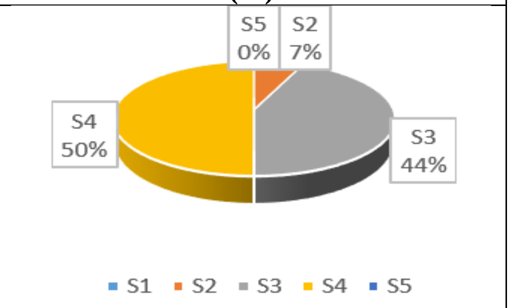
Table 12: Summary Table of Laboratory Results of Materials from the Unpaved Road

<table>
<tr><th>Geotechnical parameters</th><th>Variation
MIN-MAX</th><th>Average</th><th colspan="3">Bearing capacity class of the platform (%)</th></tr>
<tr><td>% Passing at sieve 80um</td><td>97≤%fines≤ 28</td><td>70</td><td colspan="3" rowspan="7">
</td></tr>
<tr><td>Liquid limit</td><td>88.9 ≤LL≤ 40.8</td><td>64.5</td></tr>
<tr><td>Plasticity index</td><td>40.8 ≤PI≤ 12.8</td><td>26.7</td></tr>
<tr><td>Optimal dry density (T/m3)</td><td>1.820 $\leq \gamma_s \leq$ 1.314</td><td>1.597</td></tr>
<tr><td>Natural moisture content (%)</td><td>51.7$\leq W_{nat} \leq$ 16.0</td><td>34.4</td></tr>
<tr><td>Optimal water content (%)</td><td>38.1 $\leq W_{opt} \leq$13.2</td><td>23.7</td></tr>
<tr><td>Specific gravity</td><td>2.02 $\leq \gamma_d \leq$1.565</td><td>1.774</td></tr>
<tr><td rowspan="5">CBR after 4days of immersion</td><td rowspan="5">28.3≤CBR≤ 8.6</td><td rowspan="5">15.7</td><td>S1</td><td>0</td><td>0%</td></tr>
<tr><td>S2</td><td>3</td><td>6.5%</td></tr>
<tr><td>S3</td><td>20</td><td>43.5%</td></tr>
<tr><td>S4</td><td>23</td><td>50%</td></tr>
<tr><td>S5</td><td>0</td><td>0%</td></tr>
</table>

The difference in water content will ease 95% compaction of these grainy soils. Additionally, the local climate and vegetation are favourable to the rapid drying of this soil materials. From the specific gravity test, this soil could be used for fills. The plasticity index falls under medium to high plasticity. The bearing capacity (CBR> 36) of the soils was too lower than that recommended for the base course (>80) thus we proposed the base course be ameliorated with 0/31.5 mm crushed gravel.

➤ *Borrow Pit at GTHS NKWEN*

From grain size analyses, 28 to 31% of the material was above 2 mm. The percentage of fines (≤ 0.08 mm) varies from 26 to 29%. Liquid limit and plasticity index ranged from 44 to 49 and 19 to 21 respectively. The MDD ranged from 1.785 to 1.842 t/m^3 with CBR from 30 and 34. Table 4.10 Present the results of the complete identification and Figure 1, the maximum and minimum ranges of grain size analyses of the borrow pit (GTHS Nkwen) materials. Plates 1 shows some of the field processes and technique applied in carrying out the geotechnical investigations and soil analysis.

Plate 1: Photos of Field Investigations and Techniques Applied in Sample Testing
Source: Fieldwork (2023)

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Table 13: Results of the Complete Identification, Borrow PIT (GTHS NKWEN)

Pit N°	Vegetable cover	Depth of sampling (m)	Nat w (%)	Sieve analysis - Cumulated % passing at mm									Att. Limit		Classif (H.B.R)	Specific gravity g/cm³	O.M.P		C.B.R at 95% after 4 days immersion	Bearing capacity class
				31.5	25	20	16	10	5	2	0.4	0.08	LL	IP			Yd KN/m³	W (%)		
1	0.15	1.9	7.0	100	97	91	84	65	42	31	27	24	36.8	15.5	A-2-7(2)	2.215	2.125	14.1	36.6	S5
2	0.1	2.0	7.8	100	100	99	97	78	49	39	32	23	37.1	18.3	A-2-7(1)	2.3	1.8	14.3	31.3	S5
3	0.1	2.1	9.4	95	94	93	90	78	56	38	29	23	43.0	24.0	A-2-7(1)	2.7	1.8	16.0	27.4	S4
4	0.1	1.8	8.5	100	98	97	95	85	66	38	29	25	51.0	22.2	A-2-7(1)	2.245	1.805	14.5	37.1	S5
MAX			9.4	100	100	99	97	85	66	39	32	25	51.0	24.0		2.704	2.125	16.0	37.1	S5
AVERAGE			8.2	99	97	95	91	77	53	36	29	24	42.0	20.0		2.370	1.871	14.7	33.1	
MIN			7.0	95	94	91	84	65	42	31	27	23	36.8	15.5		2.215	1.770	14.1	27.4	
Difference			1.2	3	3	4	8	12	12	6	3	1	5.2	4.5		0.155	0.101	0.6	1.8	
Dispersion %			15%	0	0	0	0	0	0	0	0	0	12%	22%		6.5%	5.4%	4.2%	5.5%	

- *Borrow PIT (Mankon)*

From grain size analyses, 28 to 31% of the material is above 2 mm. The percentage of fines (≤ 0.08 mm) varies from 26 to 29%. Liquid limit and plastic index range from 44 to 49 and 19 to 21 respectively. Maximum dry density varies from 1.785 to 1.842 t/m^3 with CBR values between 30 and 34. Present the results of the complete identification of borrow Pit materials (Mankon) and Figure 4.8: present the maximum and minimum ranges of grain size analyses of materials from the borrow pit (Mankon).

Table 14: Results of the Complete Identification, Borrow PIT (MANKON)

Pit N°	Vegcover	Depth of sampling (m)	Nat w (%)	Sieve analysis - Cumulated % passing at mm									Att. Limit		Classif (H.B.R)	Specific gravity g/cm³	O.M.P		C.B.R at 95% after 4 days immersion	Bearing capacity class
				31.5	25	20	16	10	5	2	0.4	0.08	LL	IP			Yd KN/m³	W (%)		
1	0.15	2.1	22.5	100	98	97	95	85	66	38	29	25	48.7	19.9	A-2-7(1)	2.185	1.840	17.1	38.1	S5
2	0.1	2.0	24.3	97	92	91	86	71	52	34	28	26	45.9	20.9	A-2-7(1)	2.384	1.860	15.1	34.6	S5
3	0.1	2.1	24.3	97	92	92	86	72	53	35	29	28	46.7	15.9	A-2-7(1)	2.275	1.838	14.7	32.3	S5
4	0.1	2.1	24.3	100	98	97	95	86	69	46	30	27	51.1	22.1	A-2-7(1)	2.345	1.730	14.4	30.3	S5
MAX			24.3	100	98	97	95	86	69	46	30	28	51.1	22.1		2.384	1.860	17.1	38.1	S5
AVERAGE			23.9	98	95	94	90	79	60	38	29	26	48.1	19.7		2.297	1.817	15.3	33.8	
MIN			22.5	97	92	91	86	71	52	34	28	25	45.9	15.9		2.185	1.730	14.4	30.3	
Difference			1.4	76	3	3	5	7	8	4	1	1	2.2	3.8		0.112	0.021	0.6	0.8	
Dispersion %			6%	0	0	0	0	0	0	0	0	0	5%	19%		4.9%	1.2%	4.1%	2.2%	

The thickness of the borrow pit (GTHS Nkwen) has an estimated volume of 16,700 m^3 and that of Mankon is 34,188 m^3, thereby giving a total volume of 50,888 m^3. Their distances from the Bamenda-Bambili road stretch were 1.5 km and 15 km respectively.

VII. DISCUSSIONS

The results of the CBR tests prepared at 95% Modified Proctor compaction on borrow materials gave indices between 30 and 34, while for the sub-grade soil, the CBR was between 12.9 and 38.41. These CBR values were slightly lower (borrow pit) and higher (subgrade) than those of Katte *et al*, 2019 (borrow pits), CBR between 32 and 49 and sub-grade soil, between 6.5 and 19. These low CBR values (borrow pits) could be due to the geomorphology,

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characterised by steep slopes within the study area and its richness in clay content. These values made it possible to classify the soils within the S5 bearing capacity class (borrow pits) and S4 (sub-grade) materials using the Liautaud criterion. Since the CBR of sub-grade soils are all greater than 5, it follows that the sub-grade soils do not need improvement. However, to meet certain constructional exigencies, the base course soils can be improved by mechanical stabilisation using granitic gravels as suggested by Hyoumbi *et al* (2017).

The vertical deformation for the paved section gave admissible stress/strain (172.8µdef) and calculated stress/strain (155.6) µdef and admissible stress/strain (848.7 µdef) and calculated stress/strain (804.7 µdef) for wearing and foundation courses respectively. Additionally, the unpaved section gave admissible (172.8 µdef) and calculated (151.5 µdef) and admissible (848.7 µdef) and calculated (587.7 µdef) for wearing and foundation courses respectively. From these results, it is evident that the designed pavement structure presents good characteristics since all the calculated values are lower than the permissible values. In other words, the pavement structure will not undergo any structural distress within the design period (15 years). These values are greater than those obtained by NOUROU-Dine IMAM (2003, 10.8 µdef and 226.8 µdef) and Katte et al (2019, 4.5 µdef and 103.8 µdef). The difference in these values could be because they consider T3 traffic, CBR class of S3 and Poison coefficient of 0.25.

The average liquid limit (64%) obtained from the subgrade materials (paved, unpaved section), was slightly higher than 63% obtained by Djuickouo (2012), and higher than 48.6% obtained by Ananfouet Djeufack (2012). However, the average plasticity index (26) obtained was lower than those of Kamtchueng *et al*, 2015 (30 and 31%) and similar to those obtained by Hyoumbi *et al*, 2017. The Casagrande plasticity chart, therefore classifies them as inorganic clays of medium to high plasticity. These materials were described using the HRB criterion as A 2-7 and A-7-5 soils based on their plasticity indices and its fine contents. The A 2-7 and A-7-5 materials are classified as fair to poor materials for roadworks. This result is consistent with the study of Katte *et al*, (2019) who obtained similar results (A-7-5).

VIII. CONCLUSION

To contribute in curbing the problems of insufficient geotechnical investigations in Cameroon, geotechnical studies were carried out along the Bamenda-Bambili road, in view of rehabilitation and widening. The subgrade soils gave a bearing capacity class of S4, which was used for the designed of this road stretch. The heavy penetrometer test carried out on the Mile 4 Nkwen Bridge gave an average depth of 5 m with corresponding resistance of 0.5 bars which will be used for the structural analysis of this bridge. Based on the results from the geotechnical characterisation of the borrow materials for fills and the pavement design, we concluded that these borrow materials could be used for fills (subgrade layer) and the base course be ameliorated with 0/31.5 mm aggregate. The aggregates (sand and crushed stones), could be used for the formulation of hydraulic and bituminous concretes. The pavement design using CBR method and ALIZE software, retained two structures, 5 BB + 20 GNT 0/ 31.5 (paved section) and 5 BB + 20 GNT 0/31.5 + 30 laterite (unpaved section).

RECOMMENDATIONS

As for the government, when a project is launched, members of the technical review commission or the Project Engineer should visit the projects site with the Enterprise and after, hold a meeting to review the term of reference, thereby having a clear understanding of the projects. Furthermore, the term of reference (TOR) should be developed after site visit; while the contract duration should be respected by avoiding delays in approving the different phases of the project. In other dimension, prior to the approval of the geotechnical program, LABOGENIE, MINTP and the Enterprise should carry out a site visit, to evaluate the quantities with minutes elaborated and sign contradictorily thereby minimising delays. Also, the technical review commission (at least inception, preliminary studies and APS) should be held in the Region where the project is being executed and quarries should be owned by the state.

As to the Consulting firms, adequate sampling technique and procedure be followed while adequate sample management be implemented. More so, assign skilled personnel to conduct geotechnical investigation and efficient presentation of investigation result be done.

The Bamenda University (NAHPI) should do an acquisition of a category A, Geotechnical laboratory to ease research.

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