Geotechnical assessment of road failure and slope monitoring along Nsukka-Adoru-Idah highway, Southeastern Nigeria

Raphael Iweanya Maduka · Ogbonnaya Igwe ·
Nnadozie Onyekachi Ayogu · Chinero Nneka Ayogu ·
Martin Nwachukwu

Abstract The quality of highway pavement is greatly influenced by the subgrade materials, the general geology of the area, and the materials used for construction. Investigation into the 75-km Nsukka-Adoru-Idah highway revealed that the pavement was underlain by three lithological units—Imo, Nsukka, and Ajali formations. The geotechnical evaluation carried out in the study includes the particle size distribution, Atterberg limit, specific gravity, compaction tests, and California bearing ratio (CBR). The base course has clay/silt (7–14%), fine sand (1–4%), medium sand (6–13%), and coarse sand (65–86%), while the subgrade presented clay/silt (74–82%), fine sand (6–9%), medium sand (10–17%), and coarse sand (1–3%). The average specific gravity results for the studied base course and subgrades are 2.58 and 2.52. Liquid limit (LL) result ranges from 27 to 60%, while plastic limit (PL) ranges between 17 and 24%, and plasticity index (PI) ranges from 5 to 39%. The maximum dry density (MDD) result ranges from 1.70 to 2.10 mg/m^3, while the optimum moisture content (OMC) for the samples ranges between 14.1 and 18.0%. The CBR result for soaked and unsoaked samples ranges from 37 to 74 and 48 to 83%, respectively. The low unsoaked CBR (<80%) and high Atterberg limits (LL > 30% and PI > 12%) failed the stipulated Nigerian standard, signifying the need for stabilization. A geotechnical model of a highway road cut generated a factor of safety of 1.45, indicating possibility of slope failure.

Keywords Subgrades · Base course · California bearing ratio · Stabilization · Highway · Landslide

Introduction

Roads and highways are strategic to any country's development because of the diverse spheres of activities that it supports. Thus, their stability and durability are critical to the success of development projects because easy accessibility and transportation of goods and services creates a veritable ground for a robust economic growth. In Nigeria, land use development such as road network alignment is perturbed by a myriad of problems ranging from surficial geomorphic processes that shapes the terrain, topographic constraints, and collapsing of structures apparently requiring a rational anthropogenic transformation by appropriating geotechnical data which could facilitate choice of construction site and the alignment of transportation networks (Dent and Young 1981; Brink et al. 1982; Ofomata and Umeuduji 1994). Undoubtedly, the negligence of the inherent site conditions during the process of planning, designing, construction, and maintenance of an engineering structure such as network of roads and highways exacerbates the danger of failures. To forestall such geohazard which could be overwhelming because of

R. I. Maduka (✉) · O. Igwe · N. O. Ayogu ·
M. Nwachukwu
Department of Geology, University of Nigeria, Nsukka, Nigeria
e-mail: mycomunity@yahoo.com

C. N. Ayogu
Department of Geography, University of Nigeria, Nsukka, Nigeria

catastrophic morphodynamics, scholars (Vestappen 1983; Ollier 1978) warned against utilization of land without adequate assessment.

In this context, Singh (2008) enunciated that continuing engineering projects on a site which is not ideal more especially where soils are not stabilized have some adverse geological effects such as shear zones and poor strength or expansive soils. In other words, such occurrence obtain where facts and evidence are left to occupy a subaltern position, due to other over-riding geographical and economic considerations. In the light of the above, the present study posits the need to incorporate geotechnical data and results of environmental assessment in road and highway construction.

However, the use of shale as base course and subgrade in highway construction is problematic as reported by many authors, except *it is stabilized* before paving (Arora 2008; Singh 2008; Aghamelu and Okogbue 2011). Stabilization helps to improve the natural soils for the construction purposes. Accordingly, Akpokodje (1986) and Okogbue and Ifedigbo (1995) had advocated that before constructing roads on expansive soils especially where heavy traffic is expected, a detailed geotechnical investigation along the road alignment must be premium. Previously, Chandler (1978) noted that the knowledge of soil geotechnical properties, especially stability index, helps to detect soil strength and assists in the timely checking of irregularities, while Douglas (1978) unequivocally stated that such knowledge is very vital in solving infrastructural-based problems in the humid tropical cities.

Reinforcing the need for a geotechnical evaluation, Okogbue and Ifedigbo (1995) and Okogbue and Ezechi (1988) associated road cut failures along Enugu-Onitsha in Southeastern Nigeria to adverse geologic conditions—expansive soils, high ground water table, poor bearing capacity of the construction aggregates, undulating topography, and high traffic flow. Wolman and Schick (1967) disclosed that change in hydrological regime due to increased peak discharge and sediment flux which impact negatively on the existing culverts and bridges because of its inability to accommodate the run off trigger a lot of unaddressed problems which could have been averted.

Given that our fact-based knowledge is profoundly sophisticated, we conclude that before the construction of a highway, some sort of standard field and laboratory geotechnical tests are carried out in situ and on construction aggregates used as the local materials. On this note, we re-echo that the failure of engineers to meticulously

adhere to the findings of geotechnical investigation would be inviting an unpalatable catastrophic legacies on the study area. In line with Lawler (1995) in pursuance of value, such is described as "intellectual black hole wherein something vital, a praxeological edge or purpose is lost" because the urgent need of averting future hazard is unaddressed. Principally, data generated from this evaluation would assist in making adequate plans for remediation of the road in the event of road failure of the Nsukka-Adoru-Idah highway whose inevitability may be forced largely by human interference emanating from rural-urban connectivity together with the apparent location of the study site under a humid tropical climate and the challenges of susceptible lithological conditions.

Putting it succinctly, we carefully state that the cardinal point of this paper is to obtain the geology of the highway alignment, to evaluate the local materials that were used in the highway construction, to evaluate the geotechnical properties of the subgrade and base course materials along the highway alignment, and importantly to suggest the possible causes of pavement failures along the highway with respect to the numerous potholes that have dotted the highway at Okuje, Okutu, Adoru, and Igabada towns.

Description of the study area

Location and physiography of the study area

The Nsukka-Adoru-Idah highway (study area) is situated near the base of the Enugu escarpment and characterized by undulating terrains. The location is within latitude 6° 52′ N and 7° 7′ N and longitude 6° 44′ E and 7° 28′ E as shown in Fig. 1a. The highest elevation lies on the eastern part of the area with a height of about 490 m above sea level, while the lowest elevation lies on the northwestern and central parts of the area, standing about 29 m above mean sea level. The area is underlain by sandstone, shale, and ironstone as shown in Fig. 2.

The major relief in the area is semi-conical residual hills formed by pyroclastic bodies with no established trend (Ofoegbu and Amajor 1987). However, the absence of deep erosion cut valleys in the area results from the low erodibility of the prevailing lithology which is predominantly shale. Topography or landform of a region, lithological characteristics of the rocks or soil, and groundwater conditions constitute important factors that
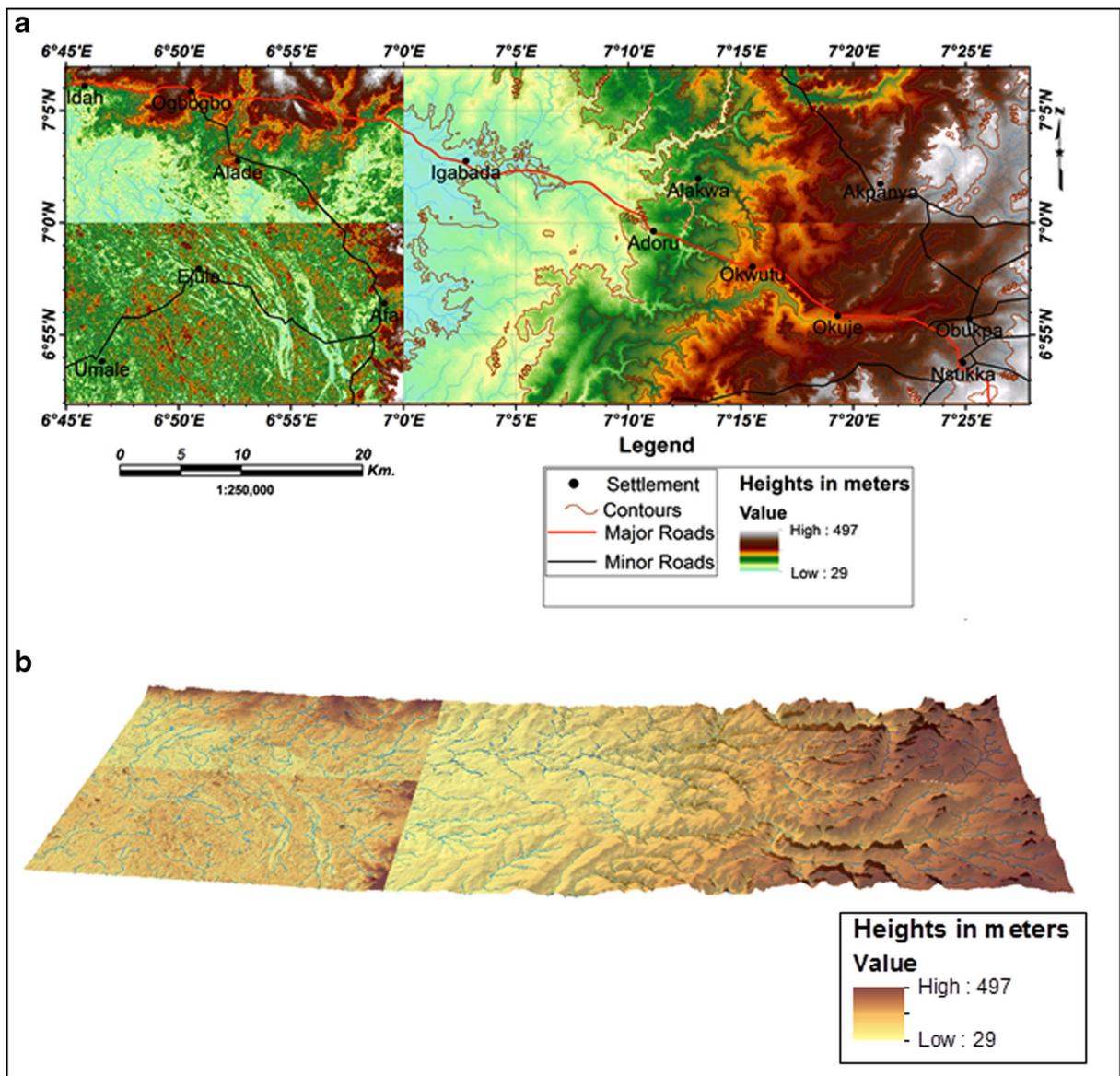
Fig. 1 Physiographic map of the study area

influence and control the selection of alignment of a highway project. The road cuts are generally on the hills, while embankment fills are often needed at the lowlands.

The drainage system in the area is basically dendritic (Fig. 1b). Traversing the area are relatively surface streams attributed to porous sandstones and shale underlay (Anyadike and Phil-Eze 1989), though porous sandstone bodies underlying the study area is a suitable medium through which water bodies are recharged or replenished (Ofomata 2002; Offordile 2002).

Climate

The area is characterized by heavy rainfall typical for the humid tropical climate (Koppen Aw) with an annual rainfall ranges from 1500 to 1800 mm (Fig. 3). Remarkably, two seasons dominate in the study area via wet season which lasts from April to October and most times are accompanied by frequent storms of high intensity. Within the wet periods, minimum temperature values are usually 22 °C (Anyadike and Phil-Eze 1989) with an anomalously short/low August rainfall called
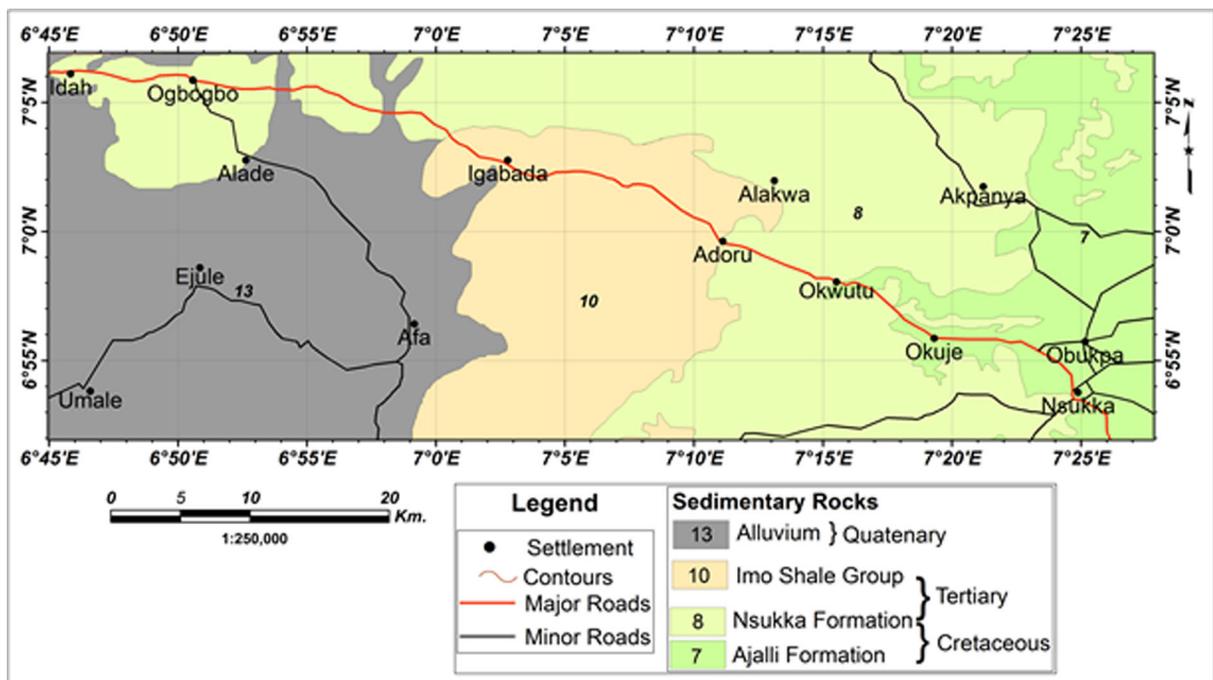
Fig. 2 Geologic map of the study area

"little dry season" or "August break" whose cause is still a subject of debate in climatology (Ireland 1962; Ilesanmi 1972), though authors (Ilesanmi 1981; Adejuwon et al. 1990; Gbuyiro and Orji 2005) described it as integral part of rainfall seasonality in Nigeria. The dry season is experienced from November to March with an average monthly maximum temperature rising to 34 °C recorded and the lowest average monthly minimum temperature of 20 °C (Monanu and Inyang 1975).

Geology

The study area is located within the Anambra Basin of the Benue Trough. The Benue Trough provided the main structural control and framework for the subsequent geological evolution of the Southeastern Nigeria. The general stratigraphy of the Anambra-Abakaliki basins is presented in Table 1.

The basin is dominantly filled with clastic sediments constituting several distinct lithostratigraphic units ranging from Upper Campanian to recent in age. The lithostratigraphic units have a thickness of up to 2500 m (Reyment 1965) and consist of Nkporo Shale, Mamu Formation, Ajali Sandstone, Nsukka Formation, Imo Shale, Ameki Formation, Nanka Sands, Ogwashi-

Asaba Formation, Benin Formation, and the Alluvial Plain Sands. The source of the sediments into the basin is principally from the Cameroon massif and the Abakaliki synclinorium (Nfor 2003).

Three geologic units are encountered along the traversed alignment, namely Imo Shale Group, Nsukka Formation, Ajali Sandstone, and Alluvium Sand. The geologic map of the area traversed by the highway is shown in Fig. 2. The Imo Shale consists generally of shale with sandstones, clay stones, calcareous mudstones, and siltstones and usually interbedded. The Nsukka Formation consists of sequence of shale and sandstones with coal seam, while the Ajali Formation consists of mainly false-bedded sandstones (Fig. 2).

Materials and methods

Sample collection

The highway traverses through Idah in Kogi State and terminates at Nsukka in the Nsukka local government area of Enugu State. The portion taken for the study started from Nsukka to Adoru in Kogi State. Samples were collected for laboratory analyses from three selected borrow pits dug in the Nsukka Formation during the
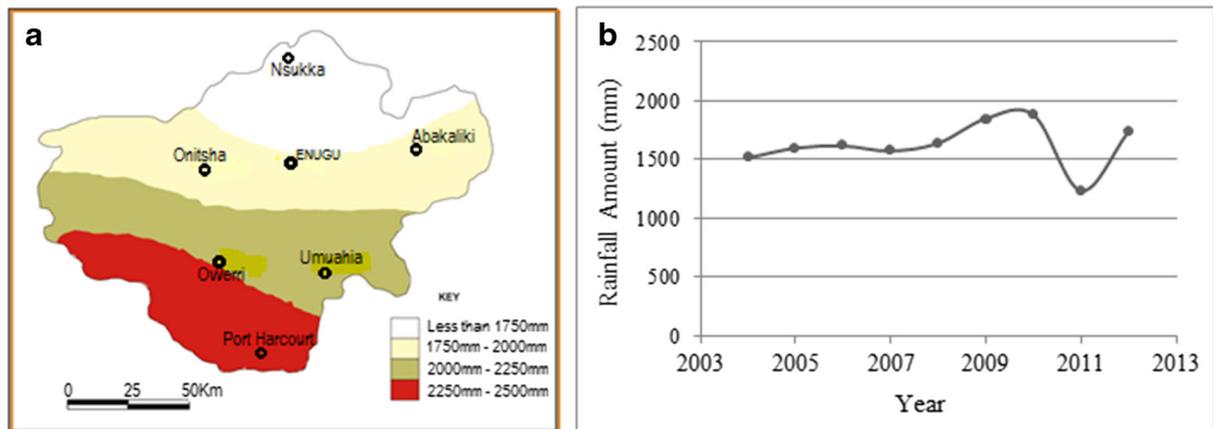
Fig. 3 Rainfall charts: **a** map of Southeastern Nigeria and **b** rainfall trend in recent years

highway construction. The samples from the borrow pits were designated as BP1, BP2, and BP3. Borrow pit BP1 is located at km 3 + 825 on the right hand side (RHS) of the road from Nsukka which is kilometer zero (highway assessment starting point); BP2 is located at km 11 + 825 m left hand side (LHS) of the road, and BP3 is located at km 15 + 825 m at the LHS of the road.

A total of 22 disturbed samples were also collected from the base course at every kilometer (1 km) and carefully labeled. Five additional samples were also collected from the subgrade. The samples were taken to the Soil and Material Laboratory of Setraco Nigeria Limited (an engineering company), Enugu, Nigeria, and the National Steel Raw Materials Exploration Agency, Kaduna, Nigeria, where analyses were done for base course and subgrade evaluation.

Laboratory tests

Sample preparation and laboratory testing to define their geotechnical properties were performed according to the specifications of American Society for Testing and Materials (ASTM). The laboratory tests done on the samples were particle size distribution, specific gravity, compaction tests for maximum dry density and optimum moisture content, California bearing ratio (CBR), and Atterberg limit test. All tests were conducted in accordance to relevant ASTM standards except CBR. The CBR was performed according to British Standard Institute (BSI 1975).

Results and discussions

Particle size distribution

The particle size distribution curve is useful in the design of highway pavements. The result of the grain size distribution analyses are summarized in Tables 2, 3, and 4. All the borrow pits have the following range of particle size distribution: fine sand and gravel as shown in Table 2. BP1 has on average clay/silt 11%, fine sands 8%, medium sands 16%, and coarse sand 65%, while BP2 has clay/silt, fine sands, medium sands, and coarse

Table 1 Stratigraphy of Southeastern Nigeria

Age	Stratigraphic unit
Eocene	Ameki Group (including Nanka Sands, Nsugbe Formation)
Palaeocene	Imo Shale
Maestrichtian	Nsukka Formation, Ajali Sandstone, Mamu Formation
Campanian	Nkporo Group (including Nkporo Shale, Oweli Sandstone, Enugu Shale, Afikpo Sandstone, Otobi Sandstone)
Santonian	Non-deposition
Coniacian	Awgu Group (including Awgu Shale, Agbani Sandstone)
Turonian	Ezeaku Formation (including Amaseri Sandstone)
Cenomanian	Odukpani Formation
Albian	Asu River Group
Precambrian	Basement Complex

Table 2 Tested borrow pit results

BP	Depth	Location	MDD mg/m³	OMC %	CBR % (unsoaked)	CBR % (soaked)	LL %	PL %	PI %	Clay/silt %	Fine sand %	Medium sand %	Coarse sand %	Specific gravity
1	2.5 m	Km 3 + 825 RHS	2.05	14.5	83	69	27	22	5	11	8	16	65	2.58
2	2.5 m	Km 11 + 825 LHS	2.03	15.2	78	67	29	21	8	11	9	13	67	2.58
3	2.5 m	Km 15 + 825 LHS	1.98	14.9	81	68	29	22	7	9	8	14	69	2.57

sand as 11, 9, 13, and 67%, respectively. BP3 has average particle distribution of clay/silt 9%, fine sands 8%, medium sands 14%, and coarse sands 69%. From the result presented in Table 3, the materials used for the base course construction showed like properties to the borrow pits, confirming the source of the base course. However, the subgrades possess clay/silt size range of 74–80%, fine sand 6–9%, medium sand 10–17, and coarse sand <3% (Table 4),

The analysis indicated that sand is the predominant soil particle size in the borrow pits. The particle size curve is situated to the right of the particle distribution chart (Fig. 4a). And according to Arora (2008), if the particle size distribution curve is situated to the right, it indicates a coarse-grained soil. Thus, the base course is well drained and non-plastic, suggesting a good engineering material. The setting of the particle size curve of the subgrade to the left of the particle distribution chart (Fig. 4b) hints a likely poor fine grain subgrade due to plasticity and low permeability.

Consistency limits

The results of the Atterberg limit test carried out on the selected borrow pit and base course samples are presented in Tables 2 and 3, respectively. The liquid limit (LL), plastic limit (PL), and plasticity index (PI) of BP1 is 27, 22, and 5, respectively. BP2 has its LL, PL, and PI as 29, 21, and 8, respectively, while BP3 has average LL, PL, and PI of 29, 22, and 7, respectively. The selected subgrade consistency limits result showed the ranges of LL, PL, and PI to be 53–60, 17–21, and 35–39%, respectively, as presented in Table 4.

The Atterberg limit results of the tested base course satisfy the specification limit for construction. The values of the PI of the base course are less than the upper limit of 25% recommended for sub-base and subgrade materials in tropical Africa by the French (1963) and therefore are expected to have no swelling and shrinkage abilities, which cause problems during and after construction. But the PI of the subgrade is above the stipulated specification (Table 4). Moreover, Sowers and Sowers (1970) reported that PI > 31 should be considered high as shown in the samples' plasticity chart (Fig. 5) and specifies high content of expansive clay. High PI which is a trait of plastic soils connotes low strength and brings about

Table 3 Base course laboratory results

S/N	Location	MDD mg/m³	OMC %	CBR % (unsoaked)	CBR % (soaked)	LL %	PL %	PI %	Clay/silt %	Fine sand%	Medium sand%	Coarse sand%	Specific gravity
1	Km 0 + 100	1.89	14.7	70	61	34	23	11	11	1	11	77	2.55
2	1 + 100	1.95	14.9	71	60	31	21	10	10	1	10	78	2.57
3	2 + 100	1.91	15.5	79	69	28	21	7	10	2	7	81	2.54
4	3 + 100	1.89	15.7	80	69	27	22	5	10	1	7	82	2.58
5	4 + 100	2.07	14.4	65	56	31	23	8	9	2	6	83	2.55
6	5 + 100	2.06	14.1	63	55	32	21	11	7	1	6	86	2.56
7	6 + 100	2.09	14.5	75	67	31	22	9	7	3	7	83	2.55
8	7 + 100	1.93	15.3	83	63	36	23	13	8	4	12	76	2.60
9	8 + 100	2.05	14.9	76	66	30	20	10	12	1	6	81	2.57
10	9 + 100	2.10	13.8	71	64	31	22	11	15	2	9	76	2.59
11	10 + 100	1.89	15.8	82	71	29	21	8	10	1	9	80	2.56
12	11 + 100	1.98	14.7	72	61	33	22	11	9	2	8	81	2.59
13	12 + 100	1.86	15.6	78	42	34	24	10	11	1	13	75	2.54
14	13 + 100	1.89	15.5	83	71	29	21	8	10	2	9	79	2.56
15	14 + 100	1.87	14.9	66	54	37	24	13	13	1	10	76	2.57
16	15 + 100	1.88	15.6	69	57	32	22	10	8	2	11	79	2.54
17	16 + 100	2.00	14.5	71	63	27	20	7	10	1	8	81	2.56
18	17 + 100	1.97	14.8	73	63	31	22	9	9	4	7	80	2.58
19	18 + 100	1.93	16.0	83	71	29	21	8	12	1	6	81	2.57
20	19 + 100	1.90	14.2	80	68	33	23	10	14	1	10	75	2.58
21	20 + 100	1.89	15.6	82	74	30	21	9	11	3	11	75	2.54
22	21 + 100	1.85	15.0	69	55	34	22	12	10	2	9	79	2.55

Table 4 Subgrade geotechnical results

S/N	Location	MDD mg/m^3	OMC %	CBR % (unsoaked)	CBR % (soaked)	LL %	PL %	PI %	Clay/silt %	Fine sand %	Medium sand %	Coarse sand %	Specific gravity
1	Km 17 + 100	1.70	18.0	51	39	60	21	39	76	9	14	1	2.50
2	18 + 100	1.81	16.9	57	45	54	17	37	79	6	13	2	2.54
3	19 + 100	1.79	17.3	48	37	58	19	39	80	7	12	1	2.53
4	20 + 100	1.71	16.5	49	41	53	18	35	74	6	17	3	2.52
5	21 + 100	1.75	15.8	63	53	56	18	38	82	8	10	0	2.54

decline in load-bearing capacity as moisture increases (Emesiobi 2002).

Soil compaction

Compaction test seeks to simulate the right combination of moisture (optimum moisture content (OMC)) and load (compactive effort) on a soil that would result in increased density. The results of the laboratory compaction test carried out on the samples from the borrow pits and base course are presented in Tables 2 and 3. A maximum dry density (MDD) of 2.05 mg/m^3 was obtained at OMC of 14.5% for BP1, while MDD of 2.03 mg/m^3 was achieved at OMC of 15.2% for BP2. Maximum dry density of 1.98 mg/m^3 was achieved at optimum moisture content of 14.9% for BP3. The subgrade however exhibited MDD range between 1.70 and 181 mg/m^3 and OMC from 15.8 to 18.0% (Table 4).

From the compaction grading shown in Table 5, the MDD for BP1 to BP3 (1.98–2.05 mg/m^3) is rated as satisfactory for base course and foundation material. The MDDs can also be graded as good subgrade materials. Also from the compaction results (Tables 2, 3, and 4), the OMC for all the tested samples is high, though still within safe limit. The relative high MDD could be as a result of high content of clay and calcite which have specific gravity ranges of 2.2–2.6 and 2.6–2.7, respectively (Aghamelu and Okogbue 2011). However, the subgrades of MDDs (1.70–181 mg/m^3) are rated poor to fair (Table 5). The low MDD could be as a result of the presence of water-retaining (swelling clay) minerals in the subgrade. This possible water-retaining ability of the subgrade could be responsible for the higher OMC exhibited in comparison to the base course (Tables 3 and 4).

Specific gravity (G_s) of solids is an important parameter used for the determination of void ratio. According to Arora (2008), G_s reflects the density of the materials in each sample exclusive of the voids they may contain. The results for the specific gravity (G_s) of the borrow pits BPI, BP2, and BP3 have G_s values of 2.57, 2.58, and 2.58, respectively. The G_s of the base course is a little higher than the subgrade, which recorded G_s range between 2.49 and 2.54. However, Reidenouer (1970) stated that rocks of specific gravity as low as 2.65 are usually weak and non-durable and could deteriorate at the foundation sites, especially on the influx of moisture.
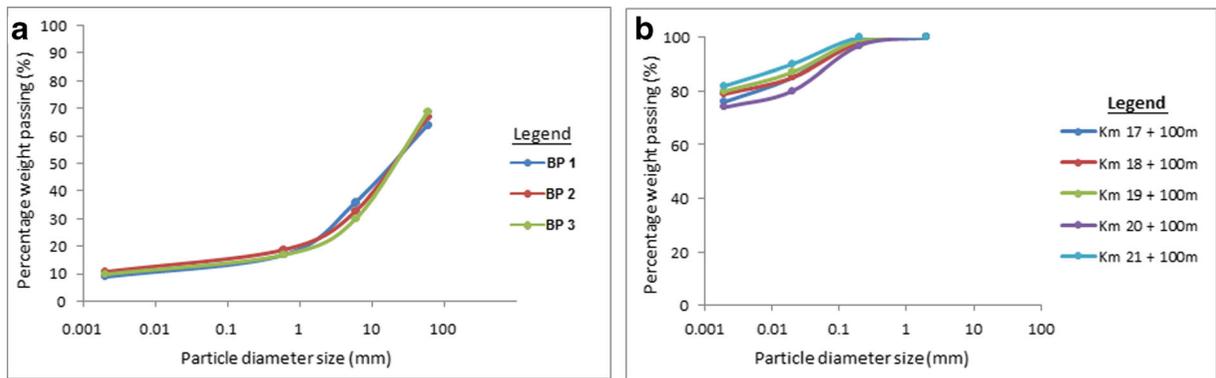
Fig. 4 Borrow pits and subgrade particle size curves

California bearing ratio

The CBR results of the unsoaked and soaked samples of the studied borrow pit samples and tested base course material are summarized in Tables 2 and 3. The unsoaked CBR values for the studied borrow pits BP1, BP2, and BP3 are 83, 78, and 81%, respectively, while the soaked CBR values obtained are 69, 67, and 68%, respectively. The BP1 sample has the highest CBR value for unsoaked and soaked conditions. The subgrade has lower CBR than the base course, with unsoaked and soaked values ranging from 48 to 63 and 37 to 53%, respectively.

CBR has been correlated with pavement performance and has often been used to determine a benchmark in the assessment of material strength, employed in establishing design curves for pavement thickness (Yoder 1959; Sowers and Sowers 1970; Arora 2008).

A comparison of the result of the study and the Nigerian specification for road materials presented in Table 6 revealed that the CBR values of the base course (both soaked and unsoaked) are moderately satisfactory for construction purpose and might require minor stabilization. The subgrade however is unsatisfactory and can be classified as poor construction materials unless major stabilization is performed before use.

Classification and suitability of the subgrades and base course materials

The vital geotechnical property information on suitability of the road construction materials are available from the borrow pit results (Table 2). However, some sections of the highway subgrade (km 17 + 100 to km 21 + 100) depicted a different property from the base course (Tables 3 and 4). This section of the highway has shown

Fig. 5 Samples' plasticity chart
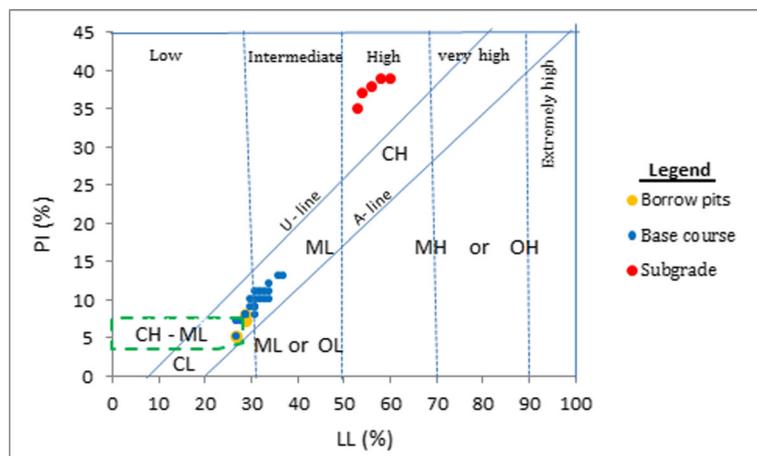
Table 5 Compaction grading (Emesiobi 2002)

Maximum dry density (MDD) kg/m³	General value as a subgrade foundation material
Over 2121	Excellent
1958–2121	Good
1795–1958	Fair
1632–1795	Poor
1142–1632	Very poor

continuous severe pavement failure at Okpuje, Okutu, Adoru, and Igabada towns as shown in Fig. 6.

According to the Unified Soil Classification System (USCS), the base course can be classified as SM-SC (silty/clayey sand), with 5–12% passing sieve no. 200 and PI > 7. The subgrade can be classified to be CH (high plastic clay), with LL > 50% and PI > 30% (Bell 2007; Arora 2008). Following the American Association of State Highway and Transport Official (AASHTO) classification, the base course can either be graded as A-2-4, with less than 35% passing sieve no. 200 and LL and PI less than 40 and 10%, respectively, or as A-2-6, with less than 35% passing sieve no. 200, LL < 40%, and PI > 11%. In general, the base course can be classified as excellent to good construction materials. However, the subgrade within the failed pavements (around Adoru and Okuje) can be grouped as A-7-6, with more than 36% passing sieve no. 200, LL > 41%, PL < 30%, and PI > 11% (Table 4). Thus, the subgrade can be grouped as poor to fair construction material. Nonetheless, the subgrade can be improved with the application of the base course, which conducts water away from the pavement due to apparently high porosity, permeability, and hydraulic conductivity.

It is worth mentioning that the recommended MDD, OMC, and CBR standard depends on specific use. For example, the Nigerian Specification for Road and Bridge Materials (Nigeria Federal Ministry of Works and Housing 1970) recommends that for a material to be used generally as fill, it should possess MDD > 0.0047 mg/m³, OMC < 18%, and soaked CBR > 5% (Aghamelu and Okogbue 2011). A comparison between Nigeria specification for construction and the studied borrow pits shown in Table 6 showed that all

Table 6 Comparison of subgrade results with the Nigerian specification for construction (adapted from Okagbue and Ochulor (2007)

Properties of materials	Nigerian specification	Borrow pit 1	Borrow pit 2 (base course)	Borrow pit 3	Subgrade
General filling and embankment					
MDD (mg/m³)	>0.047	1.90	2.20	1.96	1.70–1.81
Optimum moisture content (OMC %)	<18	15.8	14.0	15.0	16.5–18
Liquid limit (LL %)	<40	28	32	29	53–60
Plastic index (PI %)	<20	5	8	7	35–39
Passing no. 200 (%)	≤35				
Soaked CBR at British standard	>5	68	67	69	37–45
Remarks		Strongly satisfactory			Poorly satisfactory
Sub-base course					
LL (%)	<30	28	32	29	53–60
Plastic index (%)	<12	5	8	7	35–39
CBR at west African standard (%) and OMC (%)	≥30				
Remarks		Strongly satisfactory			Unsatisfactory
Base course					
LL (%)	≤30	28	32	29	53–60
PI (%)	≤12	5	8	7	35–39
Unsoaked CBR at modified AAHTO and OMC	≥80	83	78	81	48–57
Remarks		Moderately satisfactory			Unsatisfactory

Fig. 6 Bad spots (washouts and portholes) along the highway alignment

the test parameters satisfied the requirement for the base course material with the exception of the unsoaked CBR. The CBR result shows that the entire base course from the three borrow pits is moderately satisfactory either as base course or a subgrade. The materials could fail under immersed moisture influx and thus need minor improvement or stabilization to reach the basic requirement. Comparing the base course LL with compressibility grading available in Table 7, the low LL (<35%) of all the tested base courses indicates low compressibility, which is good for highway construction.

The subgrade requires major stabilization before use due to low unsoaked CBR (<80%) and high LL (>30) and PI (>13%) as shown in Table 6. High plasticity materials are generally susceptible to high compressibility (Sowers and Sowers 1970). These values are suggestive that the severe pavement failure at Okuje, Adoru, and other towns along the highway alignment could be as a result of poor subgrades. Any good stabilization method such as cement and lime stabilization could be applied to help upgrade the subgrade material to standard. It is noteworthy that the base course from the three borrow pits is satisfactory to be used for general filling and embankment, according to the Nigerian specification (Table 6).

Possibility of road cut slope failure

Road cuts are commonly anthropogenic features, typically consisting of sloping cut surfaces flanking a road

bed on one or both sides that remains after local topography is minimized by cutting an elongated depression through higher ground during road construction (Fig. 7). These road cuts could pose a threat to the highway pavement if the slopes are unstable.

The Slope/W Geostudio 2012 software developed by Geo-Slope International Limited was used to produce a modeled diagram of a slope morphology and landslide prediction (in terms of safety factor) along the road alignment. The analysis method adopted was the Morgenstern-Price limit equilibrium method. Parameters from the geotechnical tests conducted on the slope lithologies were imputed into the Geostudio software. The parameters keyed in included the unit weight, cohesion, internal friction angle as well as the piezometric water level and the slope geometry which are mainly the slope height and angles as shown in Fig. 8. The piezometric level of three localized boreholes around the highway alignment between Okutu and Okuje towns was measured using Solinst Model 101B Water Level Meter developed by Solinst Canada

Table 7 Relationship between liquid limit and compressibility (Bell 2007)

Liquid limit	Compressibility
<35%	Low compressibility
35–50%	Medium compressibility
>50%	High compressibility

Fig. 7 High shaley road cuts with possibility of sliding

Limited, and the average piezometric level of 4 m was recorded for the model.

The shear strength parameters such as cohesion and internal friction angle computed from the result of the undrained triaxial compression test done on consolidated road cut soil units by plotting the shear strength against the values of shear stress are shown in Fig. 9. The cohesion and angle of internal friction of the clayey sands (for the stress range of interest) range between 32 and 34 kPa and 20°–22°, respectively, while the cohesion of the sandy shale was 35 kPa, and the internal friction angle was 15°. The unit weight of both lithologies averages 19.2 and 20.3 kN/m^3, respectively.

The factor of safety (FS) calculated from the Slope/W Geostudio model is presented in Fig. 8. A FS of 1.45 was computed for the analyzed slope (Fig. 8a). A factor of safety of 1 indicates incipient failure (Coduto 2007). Thus, any slope with FS greater than 1.0 will purportedly be stable. However, because of uncertainties in analysis that must be accounted for and the presence of triggers in the field, there is always a need to consider larger values of FS. Hence, most common design criterion requires a FS of at least 1.50 for stability.

In this paper, the extent of stability and instability is expressed as the closeness of FS values to 1.5 and 1.0, respectively. Therefore, the FS of 1.45 indicated that the slope is moderately stable but not at safe level required. Though the FS was apparently far from 1, which is the value required for an incipient failure, it was not up to 1.5 which is the value for satisfactory stability. Thus, landslides cannot be ruled out since the triggers could still be able to cause large soil movements. In the event of a possible failure, the model showed that the failure would be a rotational slip surface passing just below the toe of the slope, having a depth of about 10 m and a sliding mass of about 148 m^3 as shown in Fig. 8b. A landslide with this geometry can be considered to be a medium to high magnitude soil movement.

With the increase in rainfall over the last few years as seen in Fig. 3b, rainfall-triggered landslides within high road cuts could be anticipated. Recently, there have been reports of landslide occurrences in various locations in Southern Nigeria such as the Obudu, Ikwette, Iva Valley, and Imande-UkusuItulygh landslides documented by Brooks et al. (2014a, b), Igwe (2015a, b), Igwe et al. (2011), and Igwe et al. (2013).
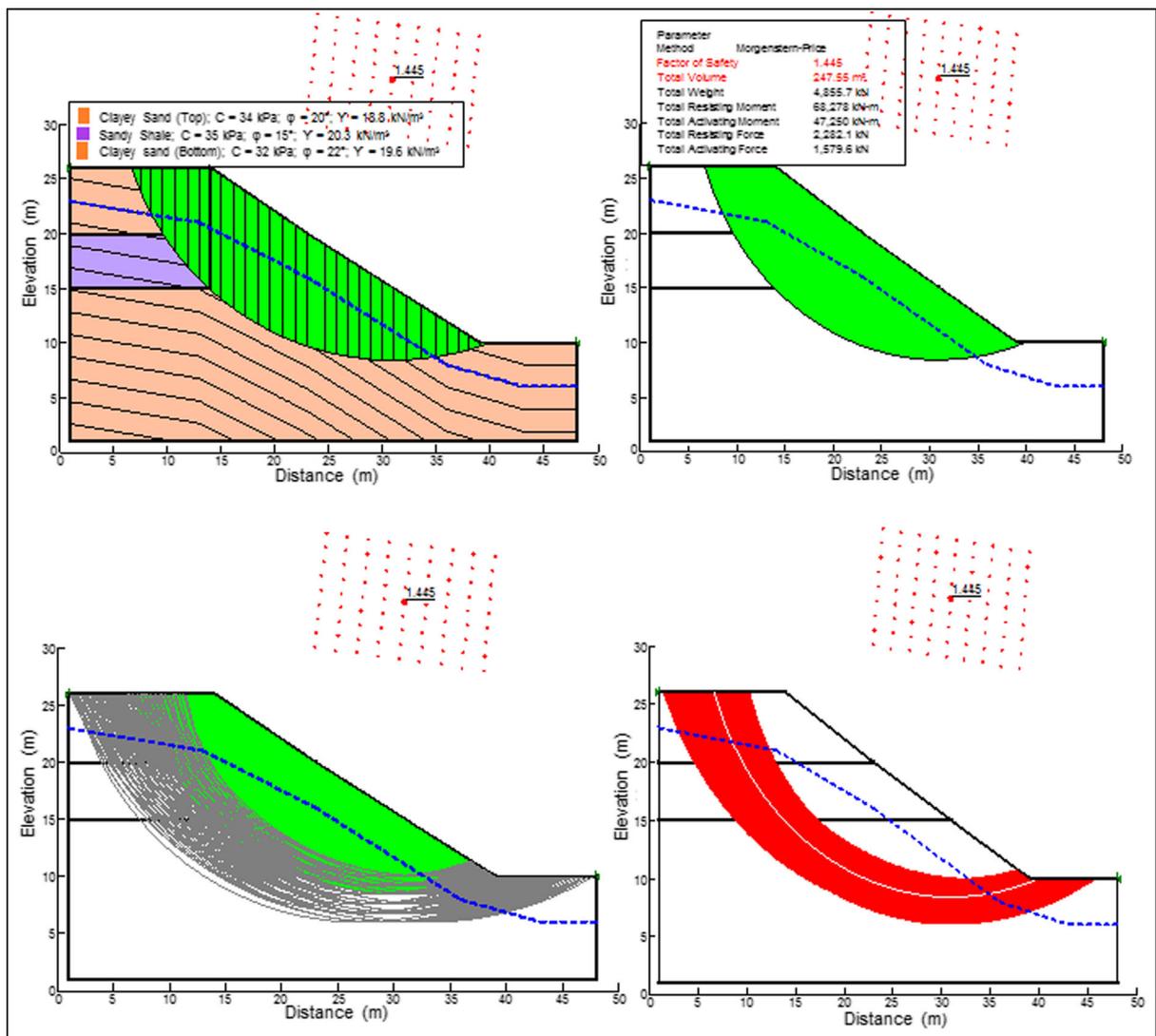
Fig. 8 Slope/W Geostudio model of a road cut along the highway

These landslides were noted to be rainfall-triggered slope failures.

Many slip surfaces were analyzed (Fig. 8c). The 150 most critical trial slip surfaces were considered to enable us to produce a safety map of the analyzed slope. The FS for these trial slip surfaces range from 1.45 to 6.55. However, the safe FS zone is area under the red band as shown in Fig. 8d. This area is considered to be a zone of potential failure in which the factor of safety of the trial slip surfaces is similar. In the present analysis, all the factors of safety within the red band are between 1.45 and 1.50, indicating a fairly good or moderate stability.

A good method to improve the stability of slopes is by lowering or completely removing the groundwater from slopes. The removal of groundwater from the slope in this analysis increased the factor of safety from 1.45 to 1.68 and also reduced the sliding mass from 248 to 199 m³ (Figs. 8 and 10a). However, methods of groundwater removal from a slope are expensive and can require a long time.

The difficulty of water removal from slope might be the reason for the use of the benching method in the stabilization (in terms of lowering the slope angle and slope length, and possible slip surface) of some of the road cut slopes along the highway alignment (Fig. 7).
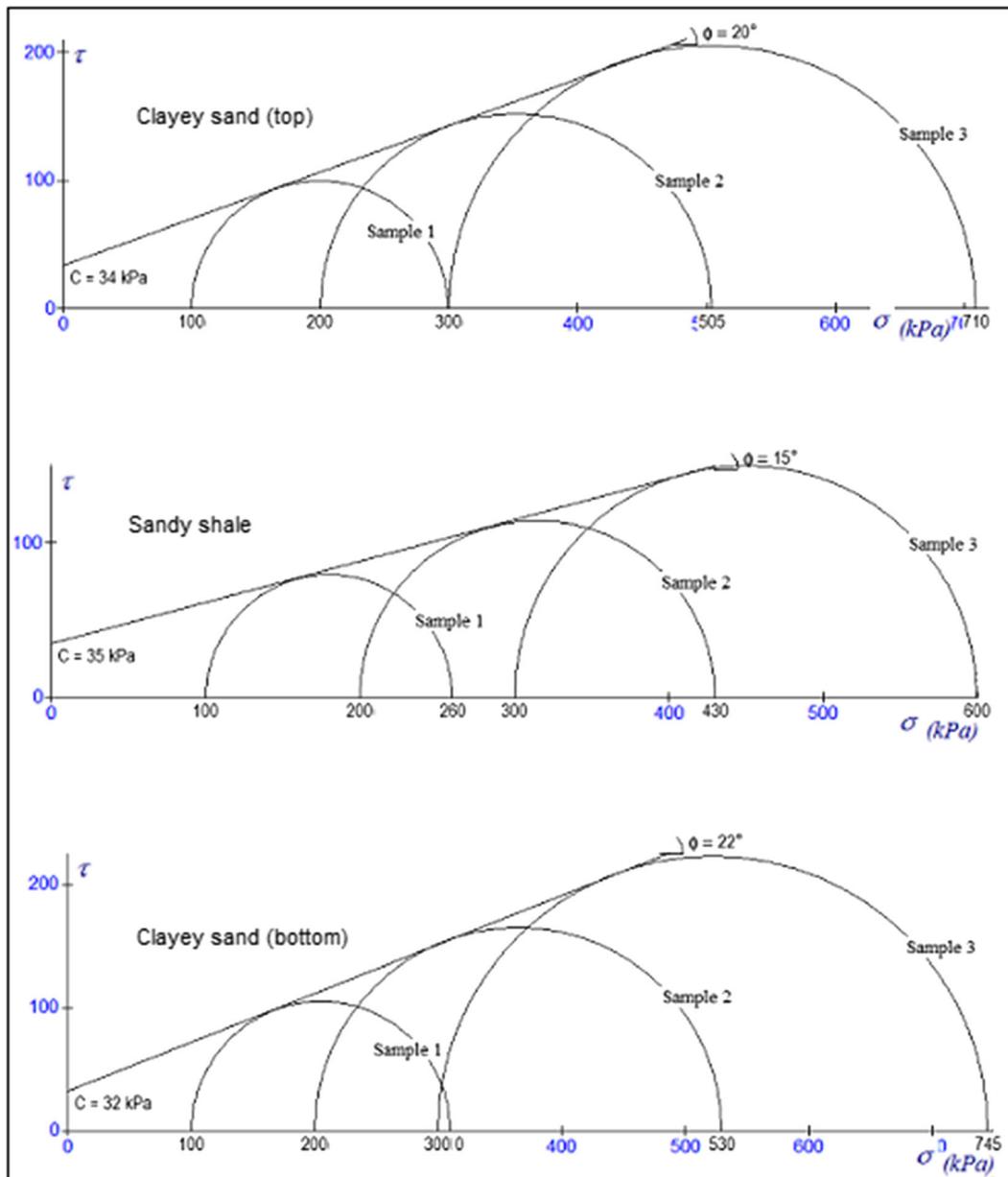
Fig. 9 Mohr envelops and shear strength parameters of the various slope lithologies

The benching method improved the factor of safety from 1.45 to 1.67 and reduced the sliding mass from 248 to 228 m³ as seen in Figs. 8 and 10b, respectively. The values of this method were close to the results obtained from the drained slope model (Fig. 10a, b). Hence, the benching method was also efficient but less demanding to achieve and maintain.

The authors are suggesting that the method of benching be applied to other road cuts along the

highway to facilitate suitability, safety, and durability of the slopes and, by extension, increase effectiveness of the road alignment and security of lives and property.

Potential triggers and effects of road cut failures

Many landslides have occurred along highways because of rock cuts (Ayalew and Yamagishi 2005). Road construction can cause slope disturbance, which increases
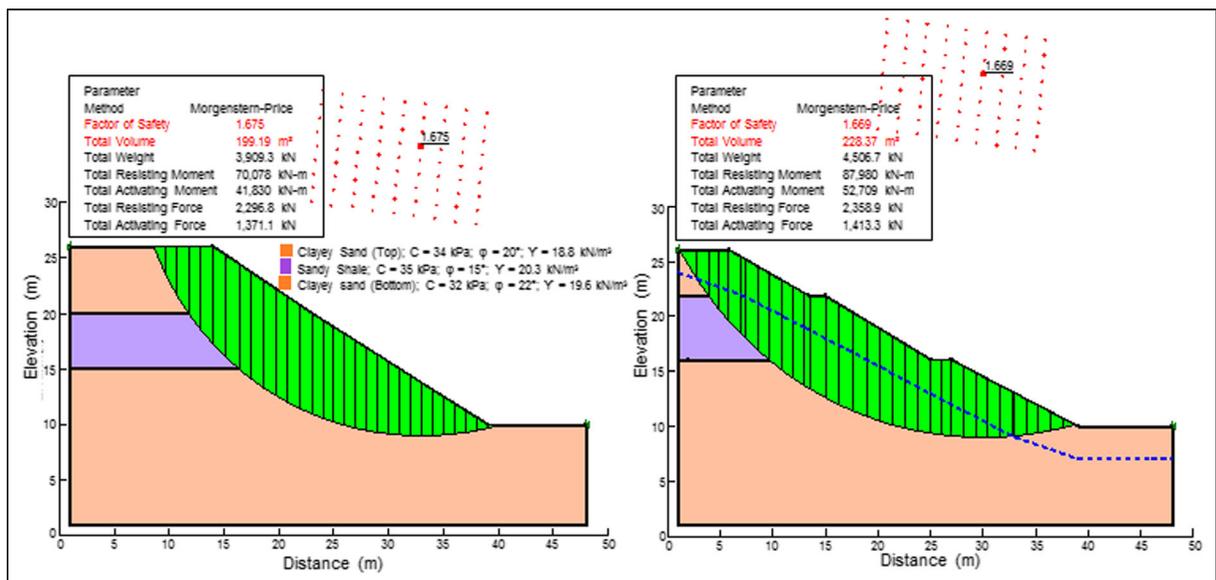
Fig. 10 Slope/W Geostudio model showing the effects of slope stabilization

stress on the back of slopes, loss of lateral support, and increase strain behind slopes. An environmental impact assessment was carried out on the road to establish the impact of the road on the environment in terms of the physical, social, and economic conditions of the people. Severe flooding and soil erosion are constant occurrences within the extent of the road alignment, especially in those sections of the road where road cutting and in filling of materials were dominant during the construction process (Fig. 11).

Some of the major factors that can predispose the slopes to failure are nature of slope material, discontinuities, and weathering (Iverson 2000; Msilimba and Holmes 2010; Wang et al. 2002; Sassa et al. 2004; Guzzetti et al. 1999) as seen in Fig. 11b. Prolonged rainfall, floods, and secondary floods (stagnancy of rain on low permeability surfaces) around the slopes could trigger landslides (Petrucci and Polemio 2009), coupled with the fact that there are many rivers within the shale formations (Fig. 1b). Inclined shale is known to slide when involved with water. The streams and runoff of water from towns might negatively affect the slope stability (Wolman and Schick 1967) by saturating the lower part of material unit, resulting in increased water content (Gokceoglu and Aksoy 1996; Youssef et al. 2014).

The rivers which are mostly near the toe of the slopes could trigger toe failures, in which the failure occurs along the surface that passes the toe of the slope. Erosion removes a slope lateral support and thus increases the slope angle (Fig. 11b), thereby increasing the chance of slope failure. Changes in topography and decrease in load on toe may lead to the development of some tension cracks (Youssef et al. 2014). The slopes could fail by rotation along a slip surface when water causes a reduction in effective stress by increasing pore pressures within its bedding planes (Figs. 8 and 10). These slopes could also undergo translational failure if a thin shale bed (weak layer) within a road cut provides a sliding surface when lubricated with water. Excessive water in the sand portends unduly high soil water content and thus increases the pore water pressure within the soil pores, which is one of the initiators of landslides.

Landslide could worsen the negative effects of erosion and flooding already being experienced within the area such as pollution of surface waters which serve as drinking water sources for the local communities. The landslides could wash away arable and productive agricultural lands or bury communities. The communities could also be deprived accessibility by deposited debris.

Table 8 showed the highway construction procedures as at the time of the study, which could cause problem to the highway. Often, cost considerations lead to economical construction rather than optimum construction. However, lack of drainage structures and back filling, inadequate

Fig. 11 Flood and erosion-ravaged spots on the highway around Okuje and Okutu communities

clearing out, and weak road bed stabilization could cause erosion and water logging along the highway alignment and surrounding bushes (Fig. 11) and thereby triggering creeping of the clays through clay mineral activity and increased pore pressures within clay layers. The provision of surface drainage systems and the completion of the other mentioned construction services would go a long way in the maintenance of the highway and thus increase

the life span of the pavement. If preventive measures are not done in the near future, pavement washouts and potholes (Fig. 6) and landslides can develop on the road which might lead to total failure of the highway pavement.

Conclusion

The result of the geotechnical investigations relative to the Nigerian specification for construction showed that the parameters examined satisfied the required specifications of a good base course for highway pavement except CBR.

The particle size distribution was above the stipulated standards for highway construction. Higher percentages of coarse sand particles than fine particles of the base course signify good material for highway construction. Also, the Atterberg limit analysis results (LL, PL, and PI) of the base course are satisfactory for materials used for base course highway construction.

The recorded MDD and OMC for the entire tested base course are above specification for highway construction. Result of the CBR recorded portrayed that both unsoaked

Table 8 Studied construction procedures along the highway

Procedure	Percentage completion
General survey	100%
Traffic studies	100%
Clearing out	75%
Setting out	100%
Cut and fill	75%
Road bed construction	75%
Final surface	100%
Drainage structure and back filling	0%

and soaked test results reached the recommended limit for base course. This situation suggested very likely that the tested samples are fairly suitable as base course, though it can be improved upon. Despite the fact that the highway under investigation shows good base course quality for construction, the CBR, LL, and PI values of the subgrade suggested that potholes which are prevalent in some parts of the road alignment around Ukuje, Okutu, Adoru, and Igabada cannot be ruled out along the highway pavement because of the underlying shale, which has the tendency to slide over another when involved with water, especially on sloppy ground as shown in Fig. 6a (Arora 2008; Singh 2008; Aghamelu et al. 2011).

The pavement failure due to clay activities could have been activated by high rainfall pattern experienced these days due to global climate change, especially in the humid tropical region wherein the study site is located, with its effects worsening at places with high road cuts and embankments. The stability of the analyzed slope was improved by the benching method of slope stabilization, which increased the factor of safety from 1.45 to 1.67 and thus reduces the possibility of landslide occurrence on the slope.

The authors therefore recommended that thorough geotechnical and geophysical investigations be carried out and results evaluated before the construction of any key community facility. More importantly, the investigation output should be taken into cognizance in the practical execution of the constructions to avoid or minimize environmental risks and engineering failures.

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