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Geotechnical Investigation into the Causes of the Failure of Portions of Pavement Along Army Engineer - Mista - Ali Road, Jos Plateau, North Central

Abalaka I.E., Momoh, A.

Department of Geology, Faculty of Natural Sciences, University of Jos, Nigeria

Abstract

The recurrent failure of pavement sections along the Army Engineer-Mista Ali Road was investigated through a detailed geotechnical assessment aimed at identifying the underlying causes of distress. A total of fifteen (15) representative soil samples were collected and analyzed, comprising nine (9) samples from visibly failed pavement sections and six (6) samples from stable, unfailed sections. Laboratory testing included grain size distribution (sieve analysis), Atterberg limits, standard Proctor compaction, and shear strength tests to evaluate the engineering behavior of the subgrade materials. Results from the Atterberg limits tests reveal significant differences in plasticity characteristics between the failed and unfailed sections. The plasticity index (PI) of soils from the failed sections ranges from 21.5% to 29.4%, indicating high plasticity and substantial swelling potential typical of clay-rich materials. In contrast, soils from the unfailed sections exhibit PI values between 6.2% and 12.7%, reflecting low plasticity and reduced swelling potential, characteristic of sandy or gravelly soils. Grain size analysis further supports this distinction. The failed sections contain 52-73% fines passing through the No. 200 sieve, signifying a predominance of clay and silt fractions. Conversely, only 9-25% fines were recorded in the unfailed sections, indicating granular soils dominated by sand and gravel. Compaction characteristics show that soils from the failed sections possess lower Maximum Dry Density (MDD) values (1.421-1.688 g/cm³) and higher Optimum Moisture Content (OMC) values (21.3-27.4%), compared to MDD values of 1.844-1.948 g/cm³ and OMC values of 10.7-14.8% in the unfailed sections. These lower densities and higher moisture demand reflect weak bearing capacity and poor load-support capability. Shear strength parameters reveal angles of internal friction ranging from 12.4° to 29.3° and cohesion values between 2.2 and 14.3 kN/m², confirming the cohesive nature of the soils. Such soils are susceptible to moisture-induced softening and structural instability under traffic loading. The study concludes that the high clay and silt content of the subgrade soils is the principal cause of pavement failure. Therefore, effective stabilization and proper compaction are strongly recommended during reconstruction and rehabilitation to enhance subgrade performance and long-term pavement durability.

Keywords
Geotechnical,
Failure,
Engineering tests,
Pavement,
Failure,
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Faculty of Science, ATBU Bauchi

INTRODUCTION

Major pavements on Nigerian roads are known to fail shortly after construction and well before their design age "(Olorunfemiet al. 1987)." The inadequate provision of drainage facilities to drain out or remove excess water quickly on or in the road pavement after heavy showers leads to failure of pavements. Rainfall is the most damaging environmental factors on pavements in tropical Africa. The road pavement network in Nigeria is experiencing a systematic deterioration which is equivalent to an asset loss of about N80 billion per annum "(Oguara, 2001)".

Rainfall as well as poor construction materials, bad design, usage factor, poor drainage network are some of the factors considered responsible for these failures. Geological factors are likely /rarely considered as precipitators of road failure even though the high way pavement is founded on the geology "(Olorunfemiet al 1987)". This is due to non-appreciation of the fact that proper design of highway requires adequate knowledge of subsurface conditions beneath the highway route. The non-recognition of this fact has led to the loss of integrity of many highway routes and other engineering structures across the country. It is therefore vital for engineers to carry out pre-design investigations of engineering sites. In view of the efforts of the Federal Road Maintenance Agency (FERMA) to rehabilitate failed segments of the roads across the country, it is imperative that adequate considerations given to the cause of the failures so as to ensure that sufficient safeguards are incorporated in their subsequent rehabilitation.

Road pavement failures could be defined as a discontinuity in road network resulting from

cracks, potholes, bulges and depressions "(Aighedion, 2007)". Failure of road pavement can occur in the form of pilting and rutting, waviness was adjudged the most common form of road failure "(Gidigas, 1974; Adeyemi, 2000)". The majority of road failure in the topics can be attributed to geotechnical factors as reported workers such as "(Gidigas, 1972)". (Poor construction materials, bad design, poor drainage network as some of the factors responsible for road failures. "(Meshida, 1985; Adeyemi, 1990; Momoh et al, 2008)".

This problem of premature failure of our road pavement has been of great concern to most of the geotechnical engineers and geophysicists. It is important to note that in many cases, the materials on which their roads were constructed were not in harmony with the road sub-grade specifications that in some causes may be good enough; making Nigeria road pavements to fail before the life span elapsed. Improper drainage network has led to the occurrence of potholes thus resulting to road failure. "De Graft Johnson (1972)" proposed a criterion for rating a probable field performance of lateritic soil and position in pavement based upon aggregate as well as specific gravity and water absorption tests. Cement stabilization for lateritic soil building or walling and lime/bitumen stabilization for road construction in temperate soils "(Ola, 1983)". "Aguda, (1987)" suggested that inadequate provision of drainage system in Plateau State caused the pavement and road failures.

Method of sample penetration, firing, degree of laterization (sesquioxide coatings) as estimated from index tests and correlated with the crushing strength of fire bricks such

that the degree of laterization of clay controlled by strength of bricks "(Adeyemi, 1990)". Significant difference need not exist between geotechnical properties of soils below stable zones and unstable sections of flexible highway pavement in the tropics. Depreciation in the shear strength and compressibility of soil upon inundations (by carrying out shear strengths and consolidation tests) do reduce their bearing capacity, leading to foundation failures. The reduction in cohesion is as much as between 28kpa to 437kpa while the angle of internal friction decreased from 21 to 34 to 19 and 31. The influence of cement on the compaction characteristic of soil increases with the energy of compaction "(Adeyemi and Oyeyemi, 2000; Ogunsawu, 2000; Adeyemi et al, 2003)". This journal attempts to investigate

the geological factors in terms of the nature of the subsoil, the near surface structures and the bed rock structural disposition as possible causes of a failed section along Army Engineer - Mista Ali, road pavement in Plateau State using the geotechnical methods.

Location of study Area and General Geology

The study area is part of the Naraguta sheet 168NE. It is bounded by latitudes $10^{\circ} 00' 00''$ to $09^{\circ} 56' 24''$ and longitudes $08^{\circ} 48' 00''$ to $08^{\circ} 51' 35''$. The settlements there are easily accessible due to the major road from Army Engineer - Mista Ali. Equally, we have other minor roads that are all surrounded by the younger granite ring complex. Below is a map showing the location of the study area.

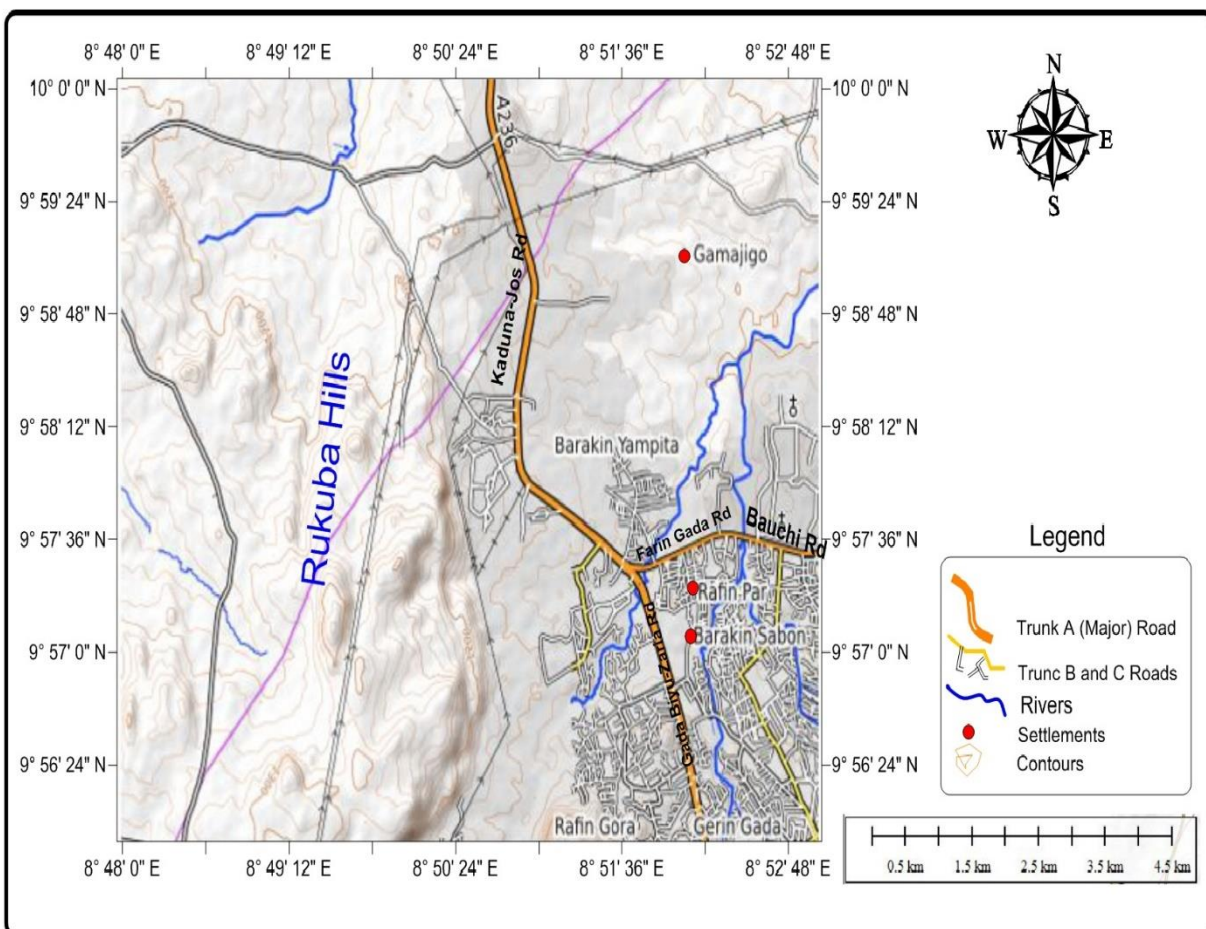


Figure 1: Topographic map showing the relief of the study area.

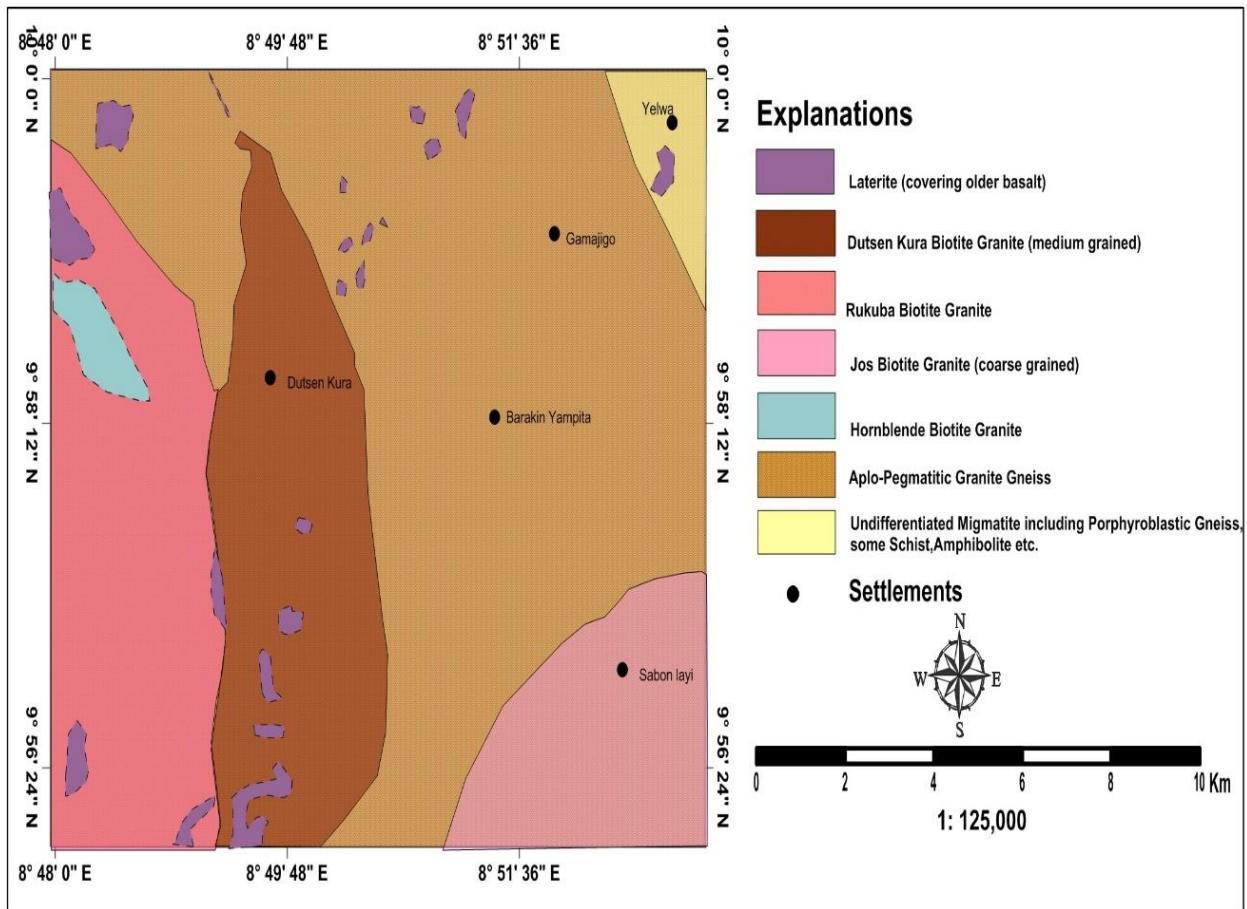


Figure 2: Geological map of the study area

In the study area, in terms of geology, the encountered granite was the major rock type occurring as ring complexes but can be distinguished from another by their locality and grain size. Generally, granites are plutonic rocks composed of alkali feldspars, quartz, mica and small portion of mafic minerals. The biotite granites are the most predominant in the study area. The proportions of mafic minerals are used to name and classify the minerals.

The rocks are classified as follows:

- i. Jos Biotite Granite (coarse grain)
- ii. Jos Biotite Granite (fine to medium grain)
- iii. Jos Biotite Granite (medium to coarse grain)

Figure 3 below is a map showing the sample location points within the study area where the samples were collected for investigation.

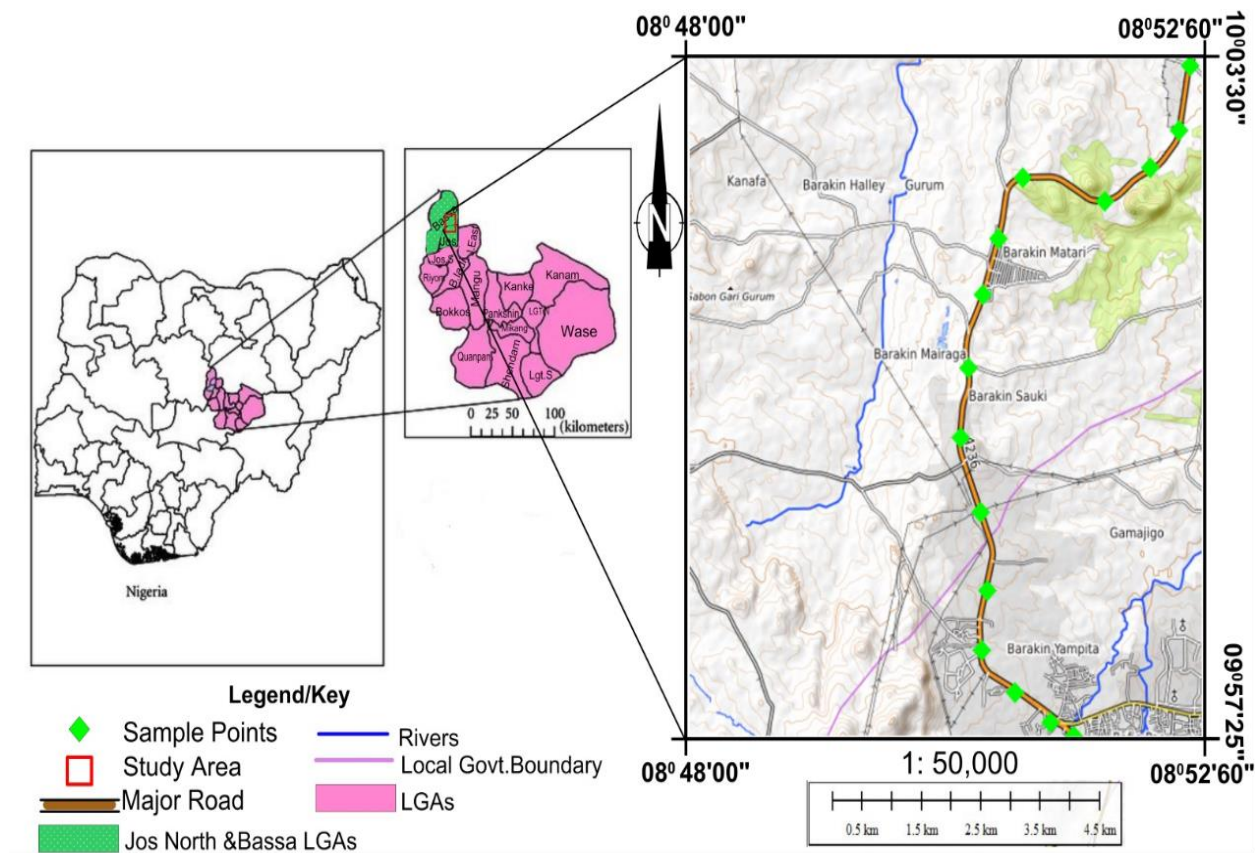


Figure 3: Map showing the sample location points within the study

MATERIALS AND METHODS

Here, samples were collected from road sites along Army Engineer - Mista Ali Road pavement which involved digging 15 pits up to 1.5m, the soil samples were collected from the damaged and the undamaged sections of the road to serve as control. The samples were labelled and packaged with their coordinates. There were taken to the laboratory for sieve analysis, grain size analysis, Atterberg limit tests, shear strength test and compaction test. 15 samples were analyzed in all.

RESULTS AND DISCUSSIONS

The geotechnical investigation involved both field and experimental work. The field work aspect involved extensive trekking in order to

determine lithological units and sampling for laboratory tests.

Selection of Sampling Sites

Sample sites were selected using the systematic sampling method. Samples were collected at every 200 m both for the damaged and the undamaged road pavement sections.

Sampling Procedure

Samples were collected at a depth of 1.5 meters by the method of digging using the digger both at damaged and the undamaged road sections within the study area. Fifteen soil samples are collected in all. Diggers, hoe, cutlass were used for digging. Samples were immediately placed in sample bags to avoid loss of moisture.

Sample Preparation and Analysis

The samples were dried and analyzed in the laboratory. The procedures are as discussed below.

GRAIN SIZE ANALYSIS

Sieve analysis is also known as gradation test. It is carried out using the British Standard Test Sieves (BS 1377), (1975) of classification. It involves the use of 500 grams of natural soil sample which was dried in the oven by passing them through series of sieves with different aperture sizes. Sieve number 200 is the last one placed at the bottom. The sieves are powered electrically to shake for some minutes. Soils samples which are retained are weighed and recorded. The percentage passing and the percentage retained in each of the sieves are calculated. The percentage passing is plotted against the grain size using the semi log graph sheet. Grain size analysis are carried out to quantitatively determine the particle / grain size distribution for soil particles of size 4.5 mm or above and + 75 micron.

ATTERBERG LIMITS TEST

Atterberg limits are boundaries that define the several states of consistency for plastic soils. This boundary can be determined by the gradual addition of water the soil needs to move from the plastic to the liquid state boundaries. The boundaries are called the plastic limit and the liquid limit, and the difference between them is called the plasticity index. The shrinkage limit is also very important in the determination of the Atterberg limits of soils. When cohesive soils are wet, they change into any shape without the appearance of any cracks on them (i.e. they can be molded). The results of this test can be

used to infer other engineering and geotechnical properties of the soils. Only the air-dried samples that passed through the British standard sieve number 36 (0.425 mm) can be used for the tests. The samples were them mixed with ordinary tap water until it reached plastic limit for a duration of five minutes and left-over night for 24 hours.

Liquid Limit Test

The soaked soil samples are mixed thoroughly with ordinary tap water to a uniform paste mixture. This mixture was placed in the liquid limit cup and levelled off parallel to the base. A standard Casagrande grooving tool was used for grooving the soil paste through the center. The handle (crank) of the liquid limit device was turned and the number of blows necessary to close the groove in the soil was noted. It was necessary to close the groove by flow of the material and not by slippage. At intervals samples were also taken from the center of the closed groove for moisture content determination. This is repeated four times for each of the properly mixed paste with addition of dry powdered samples. The number of blows that closed each groove and corresponding moisture content for the soil is also noted and recorded. As a result, their moisture content values were plotted (on an ordinary scale) against the number of blows (on logarithmic scale). The best straight line between these points was drawn, and the moisture content corresponding to 25 blows on this line was taken as the liquid limit of the sample.

Plastic Limit Test

The same sample used for the liquid limit test was the same as that for the plastic limit test. In this test, sample paste of about 20 to 30 g was broken into several smaller portions and rolled back and forth under the palm on the

surface of a glass plate with sufficient pressure to form a thread of uniform diameter. The sample was weighed wet and then weighed dry. The moisture content at which the soil crumbled as its diameter approached 3mm (1/8 m) was determined. This test was repeated thrice and average moisture content was calculated as the plastic limit of the soil investigated.

Plasticity Index

The plasticity index was calculated from the liquid limits and the plastic limits of the soil. Plasticity index gives an index of how much water the soil is capable of absorbing. In classifying the soil, the plasticity index was plotted against the liquid limit on the semi log paper. Plasticity Index is the Liquid limit minus the Plastic limit.

Shrinkage Limits

This is the lowest water content at which soil can still be saturated. It is the water content at which soil changes from the semi solid state to the solid state. It is the boundary between the semi solid and solid states.

COMPACTION TEST

Compaction can be seen as a mechanical process whereby mechanical energy is applied on the soil to rearrange the particles and to reduce the void ratio usually by driving out air particles. It is a process whereby mechanical means is used to constrain soil particles together by reducing the air void of the soil, the compaction of a soil is a measure of the dry density of the compacted soil which is the weight of the wet soil divided by the volume of the soil. This test is used to determine the maximum dry density and the optimal water content a soil can achieve for a given compaction effort.

One standard was used in the compaction test, the British standard (BS). The material used in each case was that passing through the 5 mm (3/16 inches). British Standard sieve. The retained portions were discarded. The soil was air dried before the test. In the British Standard test, a certain percentage of water was added to the soil and mixed very well until uniform moisture distribution is attained. The mixture was compacted into three layers of a Standard proctor mould by the application of 25 blows / layer of a 2.5 kg rammer falling from a height of about 0.3005 m. The sample which was levelled to the top of the empty mould gave the weight of the compacted material. The experiment was repeated by gradual addition of water for each of the percentage mixtures. The dry density of the soil samples is calculated using the relationship.

$$\gamma_d = \frac{\text{Bulk Density}}{1+M_c}$$
 where γ_d is the dry density of the material and M_c is the moisture content.

SHEAR STRENGTH TEST

This is the test that is used to determine the shear strength properties of soils. It simulates the confining pressure a soil will see through the ground. Usually, the soil is subjected to three compressive stresses at right angles to each other. One of the three stresses is continually increased until the soil fails in shear.

Undisturbed soil samples were collected in three (3) tubes for each location and taken to the laboratory for analysis. The samples were extruded from the tubes into a mould where it is being removed and weighed (wet weight). The sample is then covered with a rubber membrane to prevent loss or gain of moisture. The extremities of the samples were covered with flat porous plates and the prepared

sample is set up in the triaxial chamber. Cell pressures of 15, 30 and 45 KN/m² were used as the confining pressures for the test while axial loading was applied electronically at a constant rate through a proven ring of a known constant. The dial reading was nulled and the machine set in motion. The deviator load is read from the load dial gauge at predetermined strain dial values corresponding to known percentages of strain. The following calculations were made to determine the compressive strength of the sample.

The vertical deformation is then read from a chat. The values of the major ϕ_1 and minor ϕ_3 principal stresses were used to construct the Mohr's circle of stress while the envelope was used to read off the angle of shearing

resistance (slope of the envelope) and the cohesion, c (intercept of the failure envelope) of the sample.

RESULTS AND DISCUSSIONS

Grain Size Analysis

It is done with use of sieves. According to the "American Association of State Highway and Transport Officials (AASHTO), in 2012", soils which passes through sieve 200 mesh are generally high in silt and clay and are generally referred to as poor construction materials which are incompetent. According to the AASHTO recommendations, if the soil material passing through the sieve no 200 is greater than 35%, it shows that the soil is incompetent.

Table 1: Sieve Analysis Results of % Particle Passing Through Sieve No 200

Soil Sample ID	% Passing Through Sieve No 200	Type of Soil	Remarks
Sample 1	72	Clay	Incompetent
Sample 2	52	Clay	Incompetent
Sample 3	65	Clay	Incompetent
Sample 4	71	Clay	Incompetent
Sample 5	73	Clay	Incompetent
Sample 6	54	Clay	Incompetent
Sample 7	65	Clay	Incompetent
Sample 8	68	Clay	Incompetent
Sample 9	09	Sand / Gravel	Competent
Sample 10	55	Clay	Incompetent
Sample 11	20	Sand / Gravel	Competent
Sample 12	67	Clay	Incompetent
Sample 13	14	Sand / Gravel	Competent
Sample 14	25	Sand / Gravel	Competent
Sample 15	20	Sand / Gravel	Competent

From the results above, soil samples collected at locations 9, 11, 13, 14 and 15 are competent since they were basically sand and gravels and collected at the undamaged sections of the

pavement while the rest are incompetent because there are basically clays. It can be inferred from the result that samples collected from the damaged sections of the

pavement had a range of 52 - 73 % passing through the sieve no 200 which are very fine materials indicative of clay. The undamaged section of the pavement had a range of 9 - 25 % of materials passing through the sieve which indicates that majority of the soils in the undamaged section of the pavement is sand / gravel. The clayey nature of the soils in the damaged sections makes it to be susceptible to frequent shrinkage and swelling as a result of variations in climatic conditions.

Atterberg Limit

This is the degree of consistency of the soil with gradual increase or addition of water. This gives rise to the liquid limit (LL) and the plastic limit (PL).

$LL - PL = PI$. PI is the plasticity index which has a direct bearing with swelling potentials of soils. According to "Specifications for roads and bridges, Ministry of works, Benue Plateau in (2017)", there is a direct link between plasticity index and swelling potential of soils as seen in the table below:

Table 2: Swelling Potentials of Soils

Plasticity index %	Swelling potential	Type of material
0-15	Low	Sand/gravel
15-25	Medium	Silty clay
25-35	High	Clay
>35	Very high	Clay

The results of the plasticity indices of soils examined and interpreted compared to Table 1 as shown below:

Table 3: Showing Plasticity Indices Results

Sample location	Plasticity index %	Swelling potential	Type of soil sampled	Remarks
Sample 1	26.0	High	Clay	Incompetent
Sample 2	24.9	High	Clay	Incompetent
Sample 3	21.5	High	Clay	Incompetent
Sample 4	23.0	Medium	Silty clay	Slightly incompetent
Sample 5	30.4	High	Clay	Incompetent
Sample 6	27.0	High	Clay	Incompetent
Sample 7	27.4	High	Clay	Incompetent
Sample 9	7.4	Low	Sand/gravel	Competent
Sample 10	28.2	High	Clay	Incompetent
Sample 11	9.0	Low	Sand/gravel	Competent
Sample 12	29.4	High	Clay	Incompetent
Sample 13	6.2	Low	Sand/gravel	Competent
Sample 14	10.0	Low	Sand/gravel	Competent
Sample 15	12.7	Low	Sand/gravel	Competent

From the results above, soils collected at sample locations of 9,11,13,14 and 15 are considered competent because they are basically made up of sand and gravel. There were collected in the undamaged parts of the road pavement. The rest are incompetent because they are made up of clay, there were collected in the failed parts of the road pavement. Soils in the competent areas were taken from the portions where the road pavement is undamaged.

Atterberg limit is used to determine the consistency limits which is useful in determining the settlement and strength characteristics of the soil for road construction. In the damaged sections of the road, their plasticity index ranged from 21.5 - 29.4 %. As a result of their high plasticity index, they have a potential of failure due to their high plastic nature (clay as a result of weathering of feldspars in the rocks). The other samples have a range of 6.2 - 12.7 %.

There are competent because they had low swelling potentials and can be recommended for high way subgrade materials especially in the construction of pavements because of their low plastic nature.

Compaction Test

In trying to analyze the competency of soils using their compaction parameters, one looks out for their Maximum Dry Densities (MDD) as well as their Optimum Moisture Contents (OMC) as excellent parameters when a soil is been compacted. According to the specifications for Roads and Bridges by Attimeyer and in collaboration with the Department of Scientific and Industrial Research in 2017, a table was put forward to establish the relationship between the type of material, their maximum dry densities and their optimum moisture contents as seen below:

Table 4: Relationship Between The MDD, OMC and the Soil Type

Type of material	Maximum dry density	Optimum moisture content
Clay	1.440-1.680	20-30
Silty clay	1.680-1.840	15-25
Sand/gravel	1.840-2.160	8-15

The results of the compaction test results are examined and interpreted compared with the table above as shown below:

Table 5: Showing The Compaction Tests Results

Sample location	Maximum dry density	Optimum moisture content	Results
Sample 1	1.481	26.3	Silty clay - incompetent
Sample 2	1.484	28.4	Silty clay_ incompetent
Sample 3	1.522	24.4	Clay_ incompetent
Sample 4	1.671	21.3	Clay_ incompetent
Sample 5	1.543	27.6	Clay_ incompetent

Sample 6	1.624	21.7	Clay_ incompetent
Sample 7	1.586	21.7	Clay_ incompetent
Sample 8	1.421	22.7	Clay_ incompetent
Sample 9	1.852	14.8	Sand/gravel_ competent
Sample 10	1.528	25.7	Clay_ incompetent
Sample 11	1.844	13.3	Sand/gravel_ competent
Sample 12	1.688	26.8	Clay_ incompetent
Sample 13	1.927	10.7	Sand/gravel_ competent
Sample 14	1.846	11.1	Sand/gravel_ competent
Sample 15	1.948	12.5	Sand/gravel_ competent

From the interpretation above, soil samples collected at locations 9,11,13,14 and 15 are referred to as the competent soils since there are made up of sand and were collected from the undamaged portions of the road pavement. The remaining nine (9) are referred to as incompetent soils because they were collected from the damaged sections of the road pavement. In terms of the optimum moisture content, the damaged sections had a range of 21.3 - 27.4 % while the undamaged road section had a range of 10.7 - 14.8 %. In terms of maximum dry densities, soils in the damaged sections had a range of 1.421 - 1.688 g/cm³ while soils in the undamaged road sections had a range of 1.844 - 1.948 g/cm³. According to Specifications for Road and Bridges in 2017 which stated that the maximum dry density values should be above 1.70g/cm³ and optimum moisture content of 8.56 - 12.02%, only the six samples gotten from the undamaged sections of the road pavement met the requirements. Soil samples gotten from the damaged sections

of the road pavement have low bearing capacities due to their low OMC and MDD. They are basically clayey in nature due to the weathering of the feldspars in the rocks within the study area.

Shear Strength Test

The essence of the shear strength test is to determine the bearing capacity or strength of the soil. Here, the angle of friction (ϕ) and the cohesion (c_u) of the soils are determined from the Mohs scale diagram. According to the Specifications for Roads and Bridges of the Federal Ministry of Survey, if any of the shear parameters is zero it means that the soil is non cohesive but from the results, both the frictional angles and the cohesion value intercepts have values which are not zero implying that all the soils are cohesive. Cohesiveness is associated with clay. The result of the shear strength tests is displayed as shown below:

Table 6: Shear Strength Test Result

Sample location	Angle of internal friction ($^{\circ}$)	Cohesion
Sample 1	17.1	6.0
Sample 2	18.2	4.1
Sample 3	15.4	12.2

Sample 4	29.3	13.0
Sample 5	12.3	8.4
Sample 6	19.1	9.3
Sample 7	12.4	3.2
Sample 8	23.0	7.0
Sample 9	20.3	5.1
Sample 10	14.2	4.0
Sample 11	28.1	10.4
Sample 12	16.0	2.2
Sample 13	28.4	9.1
Sample 14	25.2	11.1
Sample 15	22.3	14.3

From the results, both the frictional angles and the intercepts have values which are not zero which implies that there are all cohesive soils in the study area. The fact is that the degree of cohesion in the undamaged road section is lesser than that of the damaged road section. This is due to the high amount of sand and gravel compared to the damaged pavement section that has high amount of silt and clay. In terms of geology, the Biotite Granites in the damaged sections of the road are highly weathered and have low shear strength. As a result of these portions of the pavement around this area will continue to fail under imposed loads.

CONCLUSION

The results of the analysis show a very high content of clay / silt in the damaged sections of the road pavements. Clay soils are cohesive in nature and also expands when wet and contracts when dry. This frequent expansion and contraction lead to cracks, openings, potholes and failure. Abrupt geological variations of the soils where the road is constructed between clay and other rock types also contributed to the failure. The Granitic rocks in the study area have a lot of Feldspars

which is continually weathering into clay there by damaging the road pavements constantly. Samples 9,11, 13, 14 and 15 are competent and were collected in the undamaged sections of the road pavement. There are basically more of sand and gravel. Samples 1, 2, 3, 4, 5, 6, 7, 8, 10 and 12 are incompetent and were collected from the damaged sections of the road pavement and are clayey in nature. The results of this research will be very useful for the repairs and rehabilitation of the damaged sections of the road pavement within the study area.

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