GEOTECHNICAL STUDIES OF CAUSES OF FAILURE OF SOME SECTIONS OF FARIN GADA – ZABOLO ROAD, JOS- PLATEAU, NORTH CENTRAL NIGERIA.

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ABSTRACT

Damaged sections of Farin Gada – Zabolo road were thoroughly investigated to know the causes by examining the geotechnical properties of the soils found along the failed and the un-failed sections. The engineering tests carried out include sieve analysis, Atterberg limits, natural moisture content, shear strength test and compaction tests. Fifteen (15) samples were investigated in all. Nine (9) samples were gotten from the damaged sections of the road while six (6) samples were gotten from the undamaged sections of the road. The results of the laboratory tests showed that for Atterberg limit tests, the plasticity index for the soils in the damaged section ranged from 20.0 – 29.4% which implies a high swelling potential indicative of clay while that of the undamaged road section ranged from 6.5 –13.7% which implies a low swelling potential indicative of sand/gravel. The sieve analysis results showed that the damaged road section had a range of 54 -73% of the soil material passing through sieve no 200 which are very fine materials indicative of clay while the undamaged road sections had a range of 8 -30% of soil materials passing through sieve no 200 indicatives of the fact that majority of the soil in the area are mainly sand/gravel. For compaction, soils collected at the damaged sections of the road had maximum dry density that ranged from 1.480 – 1.687g/cm^3 and optimum moisture content values that ranged from 21.2 – 27.8%. In the undamaged sections of the road, the maximum dry density values ranged from 1.842- 1.947g/cm^3 while their optimum moisture content ranged from 8.56 – 12.02%. The low MDDs and the OMCs in the damaged sections of the road are indicative of the fact that the soils have low bearing capacities and cannot be used as construction materials except they are well compacted and stabilized appropriately. In terms of shear strength, the angle of internal friction ranged from 12.0 – 29.0^0 while the cohesion ranged from 2.0 -14.0 which is indicative of cohesive soils since none of the shear parameters is zero. Cohesive soils have low shear strength and fails easily under construction. The results show that high clay/ silty nature of the area is responsible for the road failure. This makes the soil unsuitable as road construction materials and hence, the there is need for stabilization during the reconstruction and rehabilitation of the road.

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KEYWORDS – Geotechnical, Failure, Engineering tests and Farin Gada – Zabolo

INTRODUCTION

Most Nigerian roads are known to fail shortly after construction and well before their design age (Olorunfemi *et 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 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 (Olorunfemi *et 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 failures could be defined as a discontinuity in road network resulting in 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 (Gidigasu, 1974; Adeyemi, 2000). The majority of road failure in the topics can be attributed to geotechnical factors as reported workers such as (Gidigasu, 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 roads 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 roads 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 (Adeyerni, 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 research work 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 Farin Gada-Zabolo, road 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^{o} 00' 00'' to 09^{o} 56' 24'' and longitudes 08^{o} 48' 00'' to 08^{o} 51' 35''. The settlements there are easily accessible due to the major road from FarinGada to Zabolo. 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.

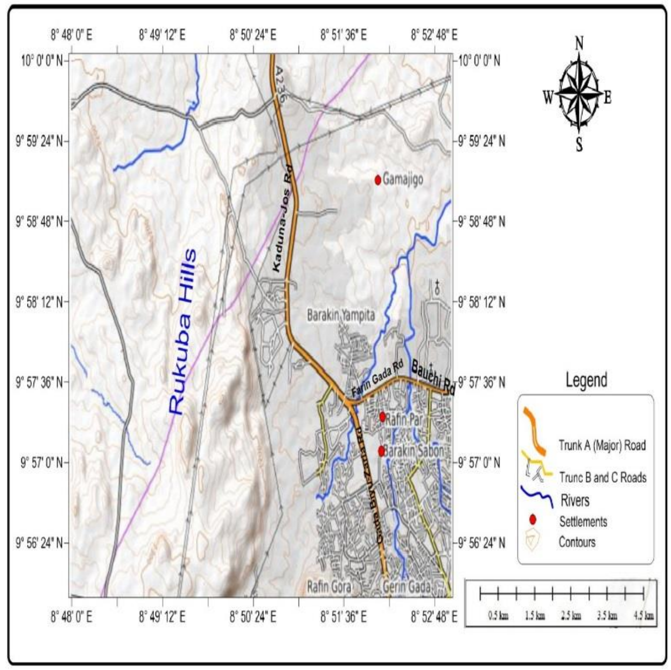

Figure 1: Topographic map showing the relief of the study area**.**

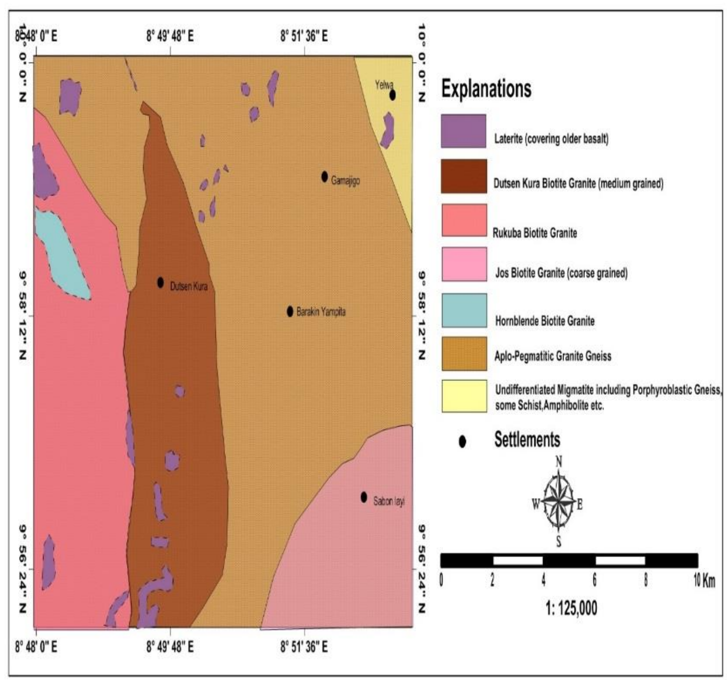

Figure 2: Geological map of the study area

In 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)

MATERIALS AND METHODS

Samples were collected from road sites along FarinGada –Zabalo road 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 compactiontest. 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 200m both at the damaged and the undamaged road 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

The above analysis was carried out in accordance with the BS 1377. (1975) using the British Standard test sieves. In this analysis, 500 grams of the natural soil sample was oven dried and passed through a series of sieves of known aperture with a number 3/8 inches on top a number 200 at the bottom. Sieving consisted of electrically shaking the sieves for some minutes. Fractions of the sample retained were weighted and recorded. From the weights recorded, the percentage retained and the percent passing in each of the sieves were calculated. The percentage passing was then plotted against grain size on the semi –log paper.

Atterberg Limits

The Atterberg, limits define the boundaries of several states of consistency for plastic soils. The boundaries are defined by the amount of water a soil needs to be at one of those 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 a part of the Atterberg limits. 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 help predict other engineering properties. The portion of air-dried sample passing the British standard sieve number 36 (0.425mm) was used for tests. The sample was mixed with ordinary tap water up to plastic limit five minutes and left-over night for 24hours.

Liquid Limit Test

The soaked sample was mixed thoroughly with ordinary tap water to a uniform paste. A portion of the paste was placed in the liquid limit cup and leveled off parallel to the base. A standard casagrande grooving tool was used for grooving the soil paste through the Centre. 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. Small samples were taken from the Centre of the closed groove for moisture content determination. This test was repeated for four more times on each properly mixed paste with addition of dry powdered samples and the number of blows that closed each groove and corresponding moisture content for the soil is recorded. The moisture content values were plotted (on an arithmetic 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 utilized for the plastic limit test. In this test, a sample paste of about 20 to 30g was broken unto several smaller portions and rolled back and forward under the palm on 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 (1/8m) was determined. This test was repeated thrice and average moisture content calculated as the plastic limit of the soil.

Plasticity Index

The plasticity index was calculated from the liquid limit using the following formula:
Plasticity index = Liquid limit – plastic limit
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 Casagrande plasticity index

Compaction Test

Compaction is the application of mechanical energy to the soil, to rearrange the particles and to reduce the void ratio usually by driving out air. 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 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 unit weight and 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 5mm (3/16) inches British Standard sieve. The retained portions were discarded. The soil was air dried to -2% moisture 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 in three layers in a Standard proctor mould using a 25 blows/layer of a 2.5kg rammer falling from a height of about 0.3005m (1ft). The sample was leveled to the top of the empty mould gave the weight of the compacted material. The experiment was repeated at higher water content for each of the percentage mixtures. Density of the material was calculated using the relationship.

$$Yd = \frac{\text{Bulk density}}{1 + Mc}$$

Where yd = dry density of the material and Mc = moisture content. The maximum dry density values and the corresponding optimum moisture contents were read off from the plots of dry density versus moisture content.

Shear Strength Test

This is a type of test that is used to determine the shear strength properties of a soil. It simulates the confining pressure a soil would see deep into the ground. It can also simulate drained and untrained conditions. As the name implies, this test subjects the soil specimen to three compressive stresses at right angles to each other. One of the three stresses being increased until the sample fails in shear. Undisturbed samples were collected in three tubes for each location and taken to the laboratory. The samples were extruded from the tubes into a mould where it is being removed and weighed (wet weight). The sample is covered with a rubber membrane to prevent loss or gain of moisture. The extremities of the sample were covered with flat porous plates and the prepared sample set up in the triaxial chamber. Cell pressures of 15,30 and 45 KN/m2 respectively were used as confining pressures for the tests while axial loading was applied electrically at a constant rate through a proving ring of known constant. The dial readings were nulled and the machine set in motion. The deviator load 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 was read from a chart. The values of the major Φ_1 and minor Φ_3 principal stresses were used to construct the Mohr's circle of stresses 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.

The results of the tests carried out and their various interpretations are shown below:

DISCUSSION OF RESULTS

Atterberg Limit

This has to do with 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 1: Swelling Potentials of Soils

Plasticity index %	**Swelling potencial**	**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 2: Showing Plasticity Indices Results

Sample location	**Plasticity index %**	**Swelling potential**	**Type of soil sampled**	**Results**
Sample 1	25.8	High	Clay	Incompetent
Sample 2	20.9	High	Clay	Incompetent
Sample 3	21.9	High	Clay	Incompetent
Sample 4	20.0	Medium	Silty clay	Slightly incompetent
Sample 5	29.4	High	Clay	Incompetent
Sample 6	26.7	High	Clay	Incompetent
Sample 7	28.4	High	Clay	Incompetent
Sample 9	8.4	Low	Sand/gravel	Competent
Sample 10	26.2	High	Clay	Incompetent
Sample 11	9.2	Low	Sand/gravel	Competent
Sample 12	27.4	High	Clay	Incompetent
Sample 13	6.5	Low	Sand/gravel	Competent
Sample 14	10.2	Low	Sand/gravel	Competent
Sample 15	13.7	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 while the rest are incompetent because they are made up of clay. Soils in the competent areas were taken from the portions where the road 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 (Adewoye *et al*., 2004). In the damaged sections of the road, their plasticity index ranged from 20 – 29.4%. As a result of their high plasticity index, they have a potencial of causing a major under load due to their high plastic nature (clay as a result of weathering of feldspars in the rocks). Only six samples having a range of 6.5 -13.7% were competent because they had low swelling potentials and can be recommended for high way subgrade materials because of their low plastic nature (Attimeyer, 2016).

Sieve Analysis

This is done with the use of sieves. According to the American Association of State

Highway and Transport Officials (AASHTO), in 2012, soils which passes through sieve number 200 mesh are generally high in silt and clay and are grouped as poor materials which are generally referred to as incompetent soils. By implication, if the percentage of the soil material passing through the sieve number 200 is greater than 35%, it shows that the soil is incompetent.

Below is a table showing the sieve analysis results and their interpretation.

Table 3: Showing the Sieve Analysis Results of ٪ Particles Passing Through no Sieve 200

Soil sample location	% passing through no 200 sieves	Type of soil	Results
Sample 1	73	Clay	Incompetent
Sample 2	55	Clay	Incompetent
Sample 3	67	Clay	Incompetent
Sample 4	70	Clay	Incompetent
Sample 5	74	Clay	Incompetent
Sample 6	54	Clay	Incompetent
Sample 7	66	Clay	Incompetent
Sample 8	67	Clay	Incompetent
Sample 9	8	Sand/gravel	Competent
Sample 10	56	Clay	Incompetent
Sample 11	21	Sand/gravel	Competent
Sample 12	68	Clay	Incompetent
Sample 13	16	Sand/gravel	Competent
Sample 14	30	Sand/gravel	Competent
Sample 15	26	Sand/gravel	Competent

From the results above, soils samples collected at locations 9,11,13,14 and 15 are competent which are made up of sand and gravels and were collected in the undamaged parts of the roads while the rests are incompetent because they are made up of clays. It can be inferred from the result that the samples collected from the damaged sections of the road had a range of 54 -73% passing through the sieve no 200 which are very fine materials indicative of clay while the undamaged road section had a range of 8 -30% of materials passing through the sieve which indicates that majority of the soils in the undamaged section of the road is sand/gravel. The clayey nature of soils in thedamaged section of the make the soil to be susceptible to frequent shrinkage and swelling during the variation in climatic conditions. This makes the soil to be mechanically unfit. The frequent weathering of the Biotite Granite rocks (fine to medium grain) which has a lot of feldspars in them always give birth to clay (Jegede, 2000; 2004).

Compaction

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) when a soil is undergoing compaction. 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: Showing the 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:

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 while the remaining nine (9) are referred to as incompetent soils because they were collected from the damaged sections of the road.In terms of the optimum moisture content, the damaged sections had a range of 21.2 – 27.8%while the undamaged road section had a range of 10.7 – 14.6%. In terms of maximum dry densities, soils in the damaged sections had a range of 1.480 – 1.687g/cm^3 while soils in the undamaged road sections had a range of 1.842 – 1.947g/cm^3

According to SpecificationsFor Road and Bridges in 2017 which stated that the maximum dry density values should be above 1.70g/cm^3 and optimum moisture content of 8.56 – 12.02%, only the six samples gotten from the undamaged sections of the road met the requirements. Soil samples gotten from the damaged sections of the road 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. This is the major cause of the road failure in the study area.

Table 5: Showing the Compaction Tests Results

Sample location	Maximum dry density	Optimum moisture content	Results
Sample 1	1.480	16.3	Silty clay – incompetent
Sample 2	1.485	16.2	Silty clay_ incompetent
Sample 3	1.520	24.2	Clay_ incompetent
Sample 4	1.670	21.2	Clay_ incompetent
Sample 5	1.540	27.8	Clay_ incompetent
Sample 6	1.621	21.7	Clay_ incompetent
Sample 7	1.585	21.7	Clay_ incompetent
Sample 8	1.420	22.7	Clay_ incompetent
Sample 9	1.850	14.6	Sand/gravel_ competent
Sample 10	1.529	25.5	Clay_ incompetent
Sample 11	1.842	13.4	Sand/gravel_ competent
Sample 12	1.687	26.7	Clay_ incompetent
Sample 13	1.925	10.7	Sand/gravel_ competent
Sample 14	1.848	11.2	Sand/gravel_ competent
Sample 15	1.947	12.6	Sand/gravel_ competent

Shear Strength

The essence of the shear strength test is to determine the bearing capacity or strength of the soil. Here, the angle of friction (₵) and the cohesion (cu) 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 has values which are not zero implying that all the soils are cohesive. Cohesiveness is associated with clayThe result of the shear strength tests is displayed as shown below:

Table 6: Showing The Shear Strength Test Result

Sample location	Angle of internal friction	Cohesion
Sample 1	17.0	6.0
Sample 2	18.0	4.0
Sample 3	15.0	12.0
Sample 4	29.0	13.0
Sample 5	12.0	8.0
Sample 6	19.0	9.0
Sample 7	12.0	3.0
Sample 8	23.0	7.0
Sample 9	20.0	5.0
Sample 10	14.0	4.0
Sample 11	28.0	10.0
Sample 12	16.0	2.0
Sample 13	28.0	9.0
Sample 14	25.0	11.0
Sample 15	22.0	14.0

From the results, both the frictional angles and the intercepts has 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 because of the high amount of sand and gravel compared to the damaged road 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 this road sections around this area will continue to fail under imposed loads.Below is a map showing the sample location points along the road both at the failed and unfailed sections of the roads.

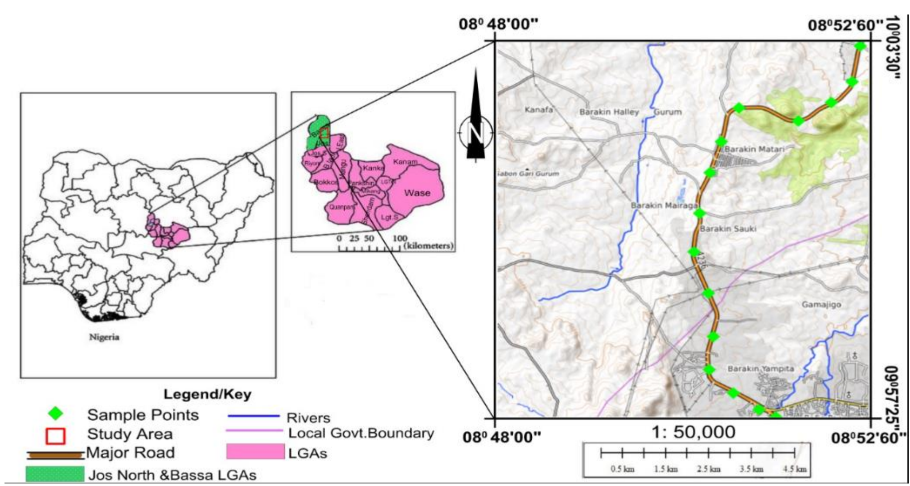

Figure 3: Map showing study areaSample Location

CONCLUSION

High content of clay (kaolin) in the damaged sections of the road is responsible for the failure. Clay (kaolin), generally has the ability of expansion when wet and contraction when dry. This frequent expansion and contraction leads 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 (kaolin), there by damaging the road constantly. Samples 9,11, 13, 14 and 15 are competent. They were collected in the undamaged sections of the road and there are more of sand and gravel while samples 1, 2, 3, 4, 5, 6, 7, 8, 10 and 12 are incompetent and were collected from the damaged sections of the road and are clayey (kaolin), in nature. The results of this research will be very useful for the repairs and rehabilitation of the damaged sections of the road within the study area if appropriate measures are carried out.

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