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ASSESSMENT OF RIVER NIGER BANK STRENGTH FOR ONSHORE STRUCTURAL FOUNDATION WORK

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ABSTRACT

Riverbank erosion is an acute natural hazard all over the world. The same hazard is also frequent in River Niger. It is a generally known truth that the geotechnical features of any foundation construction have a significant impact on its state. This research evaluated the soil subsurface conditions of the River Niger Bank by evaluating the soil strengths and recommending appropriate foundation types for onshore structural foundation works. The deep soil investigations were conducted at the Jamata axis along River Niger. Soil samples were collected at depths ranging from 2.00 to 10.00 meters in four locations 2,500 metres apart to cover the stretch of 10 km at the river bank area of Jamata. Preliminary and engineering tests were performed on the soil samples. The results of preliminary tests revealed that dense sand materials were encountered and were found to be dominance in all four drilled locations at a depth between 6.00 – 10.00m. The laboratory tests showed that the amount percentage of fine sand ranged from 31.70 – 82.50%, natural moisture content from 7.50 – 22.00%, liquid limit ranged of 25.1 – 52.2%, plastic limit ranged from 18.7 – 32,40%, plasticity index ranges from 5.20 to 25.10%, Maximum Dry Density (MDD) ranged from 1387.95 – 1965.42 kg /cm^3, specific gravity ranged from 2.20 – 2.80, California Bearing Ratio (CBR) between 2.96 – 9.81%, and the triaxial shear test ranged from 80.08 – 167.93 kN/m^2. The outcome of the research concluded that the AASHTO classification of the examined subsoils conditions namely A-2-6, A-7, and A-2-7, and in addition to low in both the MDD and percentage CBR, will make the soils unsuitable for onshore structural foundation constructions and hence, there is need for stabilization before and structural foundation work. The outcome of this research will aid Engineers, contractors, designers and construction workers in the appropriate foundation type to be adopted for an onshore structural foundation at the Jamata axis of the River Niger bank.

1. INTRODUCTION

The Niger River is one of Western Africa's biggest rivers. It is about a span of 2,600 miles, it is the third longest river after the Nile and the Congo (4,200 km). The Greeks are said to be responsible for its name, "Niger River". It goes by numerous names along the way which includes the Joliba in the higher reaches. The upper reaches are served by the Mayo Balleo and the Isa Eghirren, while the lower reaches are served by the Kworra.

Civil engineering facilities such as buildings, roadways, canals, jetties, and so on must be

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built on stable and robust soil strata. However, it is not uncommon to come across soils that are unsuitable for the foundation of roadway and waterway infrastructures on new sites, particularly in the development of newly developed environments.

One of the most critical parts of onshore structural foundation construction is an accurate assessment of site geotechnical conditions. Soil examination of the safety of an onshore structural foundation necessitates, among other things, that the foundation has been thoroughly studied, researched, and probed so that it is as well-known as possible. The exploration programme should identify the river bank soil layers that have a crucial impact on the safe performance of onshore structure foundations for the enhancement of water body navigability, rather than developing unnecessary information.

Geotechnical investigations are meant to offer the amount of knowledge relevant to the specific project development stage from project conception to construction and throughout the operation and maintenance phase. In most cases, early geotechnical studies will be vast in scope and span large geographic regions. Geotechnical studies get more comprehensive and cover smaller, more precise regions as project development progresses. Geotechnical investigations for big, complicated projects may include very precise geology mapping, such as a rock surface for a structural foundation.

1.1. Assessment of River Niger Bank Strength

The study region is the Jamata portion of the Niger River in Kogi state, and the research includes both field and laboratory work to assess soil strength for onshore structural construction. The landscape along the bed of a river, creek, or stream between which the flow is limited is referred to as the riverbank. Subsurface investigations using technology to obtain information below the ground surface might be used to determine the banks or shoreline soil structural strength. The equipment is often intrusive and involves varying degrees of ground disruption. Because most of these exploration approaches are quite expensive, they should be properly planned and regulated to give the most information possible. It is important to remember that the quality of the information supplied might vary greatly. If processes are not thoroughly followed and data is not correctly analysed, drastically divergent conclusions might be made. Poor drilling practices, for example, might result in samples with lower strength values. As a result, only qualified geotechnical experts and technicians should be responsible for designing a subsurface study, and only qualified geotechnical professionals and technicians should undertake the drilling and data collection, reduction, analysis, and interpretation.

1.2. Onshore Structural Foundation Work

Foundation soils are integrally tied to development in every country (Achmad *et al.,* 2016). From bottom to top, the platform, subgrade, and sub-base are major elements of many conventional foundations (Adams and Maria, 2013). Poor design and construction operations that did not meet accepted criteria have always been associated with the loss of lives and properties. The prevalence of foundation collapse has also been linked to the specific qualities of the fundamental topography and building materials (Fardin and Rouzbeh, 2017). Soils having poor geotechnical properties, such as low bearing capacity, low maximum dry density, a high liquid limit, and a low plasticity index, commonly fail to support foundations (Ademilua, 2018). The appropriateness of soils as base course, subbase course, and subgrade, on the other hand, is integrally tied to axle load conveyance and bearing, which influences foundation strength, durability, and life (Ajeet *et al.,* 2015).

Subsoil geotechnical evaluation aids in understanding the nature of soils that may considerably delay foundation building, as well as offering solutions to difficulties affecting both elastic and inelastic soils (Ayininuola and Denloye, 2014). Investigations have previously emphasised the necessity of geophysical investigations and combined geotechnical and geophysical investigations in finding the key causes of foundation collapses in southern Nigeria. Adams (2013) The efficacy and success of any onshore engineering infrastructure design will be heavily influenced by the geotechnical qualities of the soil materials and the in-situ knowledge available before the commencement of the project. As a result, the geotechnical characterisation of the material is critical for cost, design considerations, and overall project planning (Ayininuola and Denloye, 2014).

As previously stated, a complete geotechnical study is required for each land engineering project to acquire information on the soil qualities and the site. Excavation for this purpose is often accomplished by boring or vibrocoring. The vibrocone, on the other hand, is suitable for loose deposits but not for cohesive materials or rock. Simultaneously, field experiments (such as conventional penetration tests, vane shear tests, self-boring pressure meter tests, seismic cone tests, BAT permeability tests, and so on) can be performed to access in-situ soil parameters and retrieve undisturbed samples for laboratory testing, if appropriate. Piston and thin Shelby tubes are commonly used for sampling undisturbed clays.

Structural works include everything strong enough to be utilised in construction. Onshore structures can also be referred to as marine structures or structures visible in the maritime environment. Onshore and offshore are phrases used to describe activities that take place on land parallel to a coastline or river, as well as projects that take place on the water. Onshore structures are frequently located near the sea or a river, and their major function is to prevent coastal land erosion, allow navigation, reduce flooding of nearby properties, and perform other marine operations.

Soil characteristics and attributes have a large impact on the mechanical quality of any soil formation. Onshore structural geotechnical investigations need a combination of sampling, drillings, on-site soil testing, and laboratory soil testing.

Testing to assess subsurface engineering characteristics to develop onshore structures such as jetties, revetment, groynes, and others.

1.3. River Niger Bank Strength and Onshore Structures

River Niger bank is prone to coastal erosion, cliff retreat, and floods, onshore buildings are designed to defend the harbour basin and entrance from waves and to safeguard the navigational network. Inland riverbank erosion occurs all year, with a spike during high or heavy rainfall seasons when rivers convey huge amounts of water that regularly overflow the banks and cause flooding. Bank erosion frequently occurs gradually and unnoticeably. The river waves regularly pound the riverbank, eroding the loose silt layers. Occasionally, a big section of the riverbank collapses into the river and disappears. (Mousa, 2018).

2. METHODOLOGY

2.1. Location and Physiographic Location and Accessibility of the Study Area

Jamata is a settlement that flows southwards along the riverbanks of River Niger at Murtala Mohammed bridge, with coordinates Longitude 6.768026 E and Latitude 8.036843 N. Jamata is in Kogi State, 10.0 - 12.0 kilometres northwest of Lokoja.

The Jamata location is reachable by both road and sea. The area's topography is typically quite flat, with localised sections having a mild incline as seen along the planned road path (Plate 1). The vegetation consists primarily of grasses, bushes, and scattered trees, with predominantly farmed farmlands, and dendritic drainage may be seen along the routes. The studied ground has a modest drop into the river on its topography. Access is through an earth road, which may be difficult to reach during the wet season. The survey location is located along the Niger River's bank.

Plate 1: Jamata area of River Niger bank

2.2. Climate and Vegetation

The climate in the region is Guinea Savannah, with distinctive dry and rainy seasons. Rain falls from May to November, with a maximum temperature of around 37.90. Annual rainfall of about 1000 mm and relative humidity around 60% In December and January, there is significant harmattan. The northeast Trade Wind begins to drift southward into the nation from the Sahara belt around this time. During this season, it is often colder and less humid, although sight is limited at times due to flying dust. The vegetation in the area is characteristic of the transition to Sudan Savannah, with patchy forests separated by plateaus and quite huge trees. Grass cover is more consistent, especially during the wet season.

2.3. Preliminary Studies

To identify the index qualities of the soil and define the soil type, preliminary tests such as natural moisture content, particle size analysis, and Atterberg's limits were performed on soil samples. The Atterberg Limit test was used to calculate the plasticity index (PI) of natural soil. The preliminary tests that were performed on the following soil samples were collected before the engineering testing in the laboratory:

The study's techniques were separated into three sections, namely;

1) Field Investigation
2) Sampling
3) Laboratory testing

The scope of works includes but is not limited to as displayed in Table 1.

2.4. Field Investigations

The field investigation involved the boring of 4 numbers holes up to the depth of 10.00m at 2,500 metres apart to cover the stretch of 10 km at the riverbank area of Jamata. The field work was executed on 20th May, 2019 using the manual bucket auger, soil samples were excavated from the boring locations to a depth of 10.00 meters below the existing ground level. Disturbed samples up to 10.00 meters were preserved and taken for further laboratory test and analysis.

2.5. Pre-Treatment of Soil Sample

Soil samples were collected at four locations at 2500 meters apart along Jamata stretch of River Niger. These were designated as Pit A, B, C and D. These samples were collected at 2.00 m, 4.00 m, 6.00 m, 8.00 m and 10.00 m respectively for all the four location pits. Geotechnical investigation was conducted at all the four locations using soil bucket auger method as shown in Figure 2, to collect disturbed samples for laboratory analysis. The soil samples were collected in a polythene bag to avoid loss of moisture. Insitu experimentation was also conducted with standard penetrometer test. The soil samples collected at each from were examined at various depth from 2 meters to 10 meters at intervals of 2500 meters apart.

Soil Samples pre-treatment was ensured before the commencement of the laboratory tests, by identifying the soil samples, placing tags on them to describe the depth of collection and the pit it was collected from and the date of collection. Large percentage of soil samples were placed on sacks in the

Table 1: Scope of works

S/No	Test type	Test Description	Quantity
1	Auger Samplings	Carry out auger holes to depth of 10.00m at 2.00 interval within the stipulated riverbank.	4Nos
2	Standard Penetration Test (SPT)	Conduct to ascertain the Bearing Capacities of the immediate overburden materials.	4Nos.
3	Laboratory Analysis	Laboratory test and analysis to evaluate the physical and strength properties of the disturbed soils samples obtained.	

laboratory to air-dry them for a minimum of two weeks, preventing water contamination and direct sunlight contact.

Local drying was prevented by frequently turning of the sacks soil of samples, while little sample was taken for natural moisture content determination.

2.6. Laboratory Testing

The samples were carefully identified, preserved, and taken for laboratory tests and analyses. Laboratory classification tests were carried out on the undisturbed and disturbed samples obtained from the boreholes to improve on field identification and classification tests. The tests carried out include:

- Moisture Content determination;
- Atterberg Limit Tests;
- Particle Size Distribution Tests;
- Bulk density
- Specific Gravity;
- Triaxial Shear Test
- Compaction test
- C B R test

Plate 2: Collection of soil samples and different depth using Bucket Auger

2.7. Determination of Moisture Content

The moisture content of the soil samples was determined to discover the significant change in some properties of the soil. Moisture content can be expressed as a percentage of the dry samples, and it was done in accordance with the provisions of section 3.2 in part 2 of the BS 1377 code.

The procedures applied for the test were as follow;

- The minimum of 30g of soil sample was crumbled and placed in a clean container and weighed to the nearest 0.1g as M_1
- The container with its content was weighted and recorded at M_2 and was then place in an oven for drying at 105°C to 110°C for 24 hours.
- After drying, the container together with its contents were put in desiccator to cool. After cooling, it was weighed and recorded as M_3. Therefore, the moisture contents of soil sample were expressed as a percentage of dry soil samples.

The difference in mass before and after drying was used as the mass of the water in the test material. The mass of material remaining after drying was used as the mass of the solid particles. The ratio of the mass of water to the measured mass of solid particles was the moisture content of the material.

2.8. Determination of the Specific Gravity of Soil Particles

Two methods of specific gravity tests are mostly used which includes gas jar and small pycnometer methods. The gas jar method is suitable for soils containing more than 10% of stones retained, on a 37.5 mm BS test sieve and such stones should be broken down to less than this size. The pycnometer method containing fine, medium and course –grain but not suitable for soil containing more than 10% of stones retained on a 37.5 mm BS test sieve and such stones should be broken to less than this size. The pycnometer method is used for determination of the specific gravity of soil particles of fine – grained soils. The method may also be used for medium and courses grained soils if the coarse particles are ground to pass 2mm BS test sieve before using.

In both test methods, the soil sample is weighed with or without water and oven dried at 105°C.

However, the gas jar method was used in this study according to the provision of section 8.2 in part 2 of the BS 1377 code (Ademilua, 2018). The apparatus for gas jar test is:

- A gas jar, 1 litre in capacity, fitted with a rubber bug.
- A ground glass plate for closing the gas jar
- A mechanical shaking apparatus capable of rotating the gas jar, end over end, at about 50 rev/mins
- A balance readable and accurate to 0.2 g
- A thermometer to cover the temperature range of 0°C to 50°C, readable and accurate to 1°C.

The procedures for the test were as follows;

The gas jar and ground glass plate were cleaned, dried and weighed to the nearest 0.1 g (M_1). Appropriately 200g of air-dried soil passing BS sieve of 4.25 μm opening was placed in the gas jar. The gas jar, ground glass plate and contents were weighed to the nearest 0.1g (M_2).

Approximately 500 ml of water at a temperature within 2°C of the average room temperatures during the test was added to the soil. The rubber stopper was then inserted into the gas jar. The gas jar was shaken by hand until the particles are in suspension and for 20mm to 30mm.

The stopper was then removed carefully and soil was allowed to settle for a few minutes, and the gas jar was filled with distilled water to the brim. The stopper was then placed on top of the jar. The gas jar and the stopper were then carefully dried on the outside and the whole weighed to the nearest 0.1 g (M_3).

The gas jar was emptied, washed thoroughly and filed completely to the brim with water. The gas jar was then dried carefully on the outside and the whole container weighed to the nearest 0.1 g (M_4). These procedures were repeated for each of the soil samples.

2.9. Atterberg's Limits

The Atterberg's limits tests are intended to investigate the clay minerals present in a soil and they were carried on the fraction of the soil passing on 0.425 mm sieve. The tests measure in the water content at which the soil becomes so weak that it is liquid – like in nature and the moisture content at which it strengthens sufficiently to become brittle. Only the liquid and plastic limits were considered in this proposed study. The liquid limit (LL) is the minimum water content at which the soil will flow under a specified disturbing force, also is the moisture content at which material pass from the plastic to liquid state.

The plastic limit (PL) is the minimum water contents at which the soil will deform plastically, also is the minimum water content art which the soil begins to crumble when rolled to a thread of about 3mm in diameter. The liquid limit can be determined by using either the Casagrande liquid limit apparatus or the core penetrometer test.

The liquid limit in the study was determined by using Casagrande apparatus as shown in Plate 3 in accordance with the provisions of section 4.5 in part 2 of the BS 1377 code. The plastic limit was determined by the procedures' in section 5 of BS 1377: 1990 (Ademilua, 2018).

The tests were firstly carried out on soil sample in its natural state without any additives and additives were added later. The total weight of cement, wood ash and soil were equal to 200g.

The procedures for the liquid limit test are as follow.

A soil sample weighed 200g was taken from the material passing through BS sieve 0.425 mm. This was poured onto the glass or steel plate and mixed with distilled water using spatula until the soil mass became thick paste. The tip of the spatula was used to take a portion of the soil paste and then placed in the cup of the liquid limit device (Casagrande) and leveled off parallel to the base and then divided by drawing the grooving tool along the diameter through the centre of the hinges holding it normal to the surface of the cup.

The hander of the liquid limit device was then related at a uniform speed of two revolutions per second (2 rpm), causing the cup to be lifted and dropped until the two halves of the soil paste close the groove for distance of 13 mm. Therefore, the number of blows that causes this closure were recorded. At this point, part of the paste in the cup up to about 10 g was taken from the two sides of groove divided line and its moisture content was determined.

The remaining paste in the cup was then removed and more water was added to the paste and thoroughly mixed for the determination of other number of blows that causes a closure at more moisture content.

The liquid limit of the soil was graphically determined and this was done by plotting moisture content of the paste against the number of blows required to close the groove. The liquid limit was the moisture content at corresponds to 25 blows. The test was repeated for the remaining soil samples.

The procedures applied for the plastic limit test were as follows:

Soil paste of about 50 g was taken from the remaining soil sample used for the liquid limit test and thoroughly mixed with distilled water on a glass plate.

The ball of soil sample was molded between the fingers and rolled between the palms of the hands until the heat of the hands has dried the soil sufficient for slight cracks to appear on its surface.

The ball sample was divided into two sub samples of about 10 g each. Each sub sample was then divided into four approximately equal parts and each part was rolled between the fingertips and a clean flat glass plate with sufficient pressure to reduce the thread to 3 mm diameter.

The procedure was repeated until longitudinal and traverse cracks appear along the rolled 3 mm diameter thread, crumbled threads were weighed, and oven dried to determine its moisture content.

Plate 3: Determination of Liquid Limit using Casagrande apparatus.

2.10. Particle Size Analysis

These were performed by means of sieving and/or hydrometer readings. Sieving was carried out for particles that would be retained on a 0.075 mm sieve, while additional hydrometer readings were carried out when a significant fraction of the material passes a 0.075 mm sieve. Dry sieving was carried out by passing the soil sample over a set of standard sieve sizes and then shakes the entire units for few minutes with sieve shaker (machine). Particle size is presented on a logarithmic scale so that two soils having the same degree of uniformity are represented by curves of the same shape regardless of their positions on the particle size distribution plot. The general slope of the distribution curve may be described by the coefficient of uniformity Cu, where Cu = D60/D10, and the coefficient of curvature Cc, where Cc = (D30)2/D10 x D60.

The Unit Weight

The Unit weights were determined from measurements of mass and volume of the soil. The unit weight (kN/m^3) refers to the unit weight of the soil at the sampled water content. The dry unit was determined from the mass of oven-dried soil and the initial volume.

2.11. Main Engineering Tests

These tests involve determination of strength properties of the soil for onshore structural foundation work purposes. Engineering tests conducted were compaction test, California bearing ratio test, Triaxial Shear test.

2.11.1 Compaction test

. Practically, every earth fill or back fill constructed under contract today is subjected to some degree of compaction and the procedures are becoming increasable familiar to all concerned. Compaction is the term used to designate any of the many procedures, usually by mechanical means used to increase the density of soil by some form of rolling on tamping as distinct from static, dead with loading.

The methods used for compaction tests can be classified as AASHTO or proctor test (standard test), the modified ASSHTO test, and the West African level method, but only the West African level method will be used in this proposed study and will be carried out in accordance with the provision of section 3.5 in part 4 of the BS code (Ademilua, 2018). The test was firstly carried on soil sample to determine maximum dry density (MDD) and optimum moisture content (OMC) of the soil.

The procedures applied for the compaction test were as follows:

A 3kg sample of air-dried soil passing through 5mm BS sieve was obtained, the mould, with the base plate attached was weighed to the nearest 1 g and recorded as M_1,

The sample was mixed thoroughly with suitable amount of water of approximately 6% by sample and divided into three equal parts. Each of the parts was poured into the mould. Each part or layer was giving 27 blows from the rammer dropped from a height of 450 mm above the soil as shown in Plate 4. The blows were distributed uniformly over the surface of each layer, ensuring that the tube of the rammer is kept clear of soil so that the rammer always falls freely.

Compacted soil was carefully leveled off to the top of the mould by means of the straight edge knife. The mould and soil was then be weighed to the nearest 1g and recorded as M_2.

A small quantity of compacted soil specimen was taken from the mould and its moisture content was determined.

The remainder of the soil specimen was broken up rubbed through the 5 mm BS test sieve, and then mixed with the remainder of the original sample. Suitable increments of water were added successively and mixed into the sample (i.e. 3% by weight of the soil sample). The procedures were repeated for each increment of water added until the weight of compacted soil has dropped. The procedures were repeated all over again for stabilized sample of stable optimum percentage of cement and varying percentage of wood ash combination.

2.11.2. California Bearing Ratio (CBR)

The California bearing ratio is used to determine the capacity bearing of a soil sample. It is used to determine the strength of subgrade, sub-base, and base course materials in road building. The CBR test (California Bearing Ratio Test) is an in-situ penetration test designed by the California Highway Department to assess sub-grade strength. The CBR mould was assembled with its base plate and weighed. The collar fitted and a filter paper was placed at the bottom. 5 kg of the soil sample was thoroughly mixed at the optimum moisture content as determined from the compaction test. The mixture was divided five equal parts by mass. The parts were then successively poured into the mould and compacted with the mould collar attached using the 4.5 kg rammer and each part given 27 blows. After the five layers have been compacted, the collar was removed and the edge trimmed off to flush with the top of the mould. The mould containing the specimen with the base plate in position was placed on the CBR machine with the top face facing the plunger as shown in Plate 5. The plunger was then made to penetrate the specimen at a uniform rate of 1mm/min, and readings were taken at intervals of 0.25mm penetration. The base plate was removed from mould and the bottom face of the specimen was placed under the plunger and the procedure is repeated.

Plate 4: Compaction test on the soil samples

Plate 5: CBR Machine.

2.11.3. Triaxial Shear Strength

Depending on the consistency of the cohesive material, the test specimen was prepared by trimming the sample or by pushing a mould into the sample. A latex membrane with thickness of approximately 0.2 mm was placed around the specimen. A lateral confining pressure of 25 kPa to 150 kPa is maintained during axial compression loading of the specimen. Consolidation and drainage of pore water during testing is not allowed. The test is deformation controlled (strain rate of 60%/h), single stage, and stopped when an axial strain of 15% is achieved. The deviator stress is calculated from the measured load assuming that the specimen deforms as a right cylinder. The presentation of test results includes a plot of deviator stress versus axial strain. The Undrained shear strength, Cu, is taken as half the maximum deviator stress. When a maximum stress has not been reached at strains of less than 15%, the stress at 15% strain is used to calculate undrained shear strength.

3. RESULTS AND DISCUSSION

3.1. Results of the Preliminary Tests

The detailed results from the preliminary tests of the soil samples are shown in Table 2.

3.1.1. Natural Moisture Content

The natural moisture content (NMC), which changes depending on the depth of the soil, plays a vital role in increasing or decreasing the density indices of soils. One of the factors that influence the dry density of soils is their moisture content. The clayey soils will experience considerable volume variations as a result of frequent rainfall fluctuations in the moisture content of the soil (Adetayo et al., 2019). The natural moisture content of the soil samples studied ranged from 9.4% to 22%. Six of the sample locations had values that were higher than the suggested value of (5–15 %) for construction by the (FMWH 1997). This indicates that the soil materials have a high-water adsorption potential. The findings revealed that the obtained values will result in a reduction in shear strength. As a result, they are unsuitable for use as onshore structural foundation work. In clayey soils, substantial volume variations are frequently caused by high fluctuations in moisture content. In terms of moisture content, the findings range from favourable to marginally acceptable to bad (FMWH 1997).

3.1.2. Specific Gravity results

The examined soil sample's specific gravity ranged from 2.20 – 2.60 for pit A, 2.50 – 2.80 for pit B, 2.40 – 2.80 for pit C and 2.30 – 2.70 for pit D as indicated earlier in Table 4.1 above. The specific gravity of soil is known to be linked to its strength, and it is used as a criterion for choosing a suitable material for structural foundation construction materials, particularly when used in conjunction with other materials (Olufowobi, 2014). Low specific gravity is linked to the weathering of feldspar, which resulted in the formation of clay. When compared to the residual soils in Nigeria's basement complex, the value is low. Specific gravity has been found to have a substantial relationship with a soil's chemical composition and mineralogy. According to (Gidigasu, 1983), the larger the specific gravity, the greater the degree of lateralization. The lower the specific gravity, the higher the amount of clay fraction and the higher the alumina content.

3.1.3. Particle size distribution test result

The grain size analysis is crucial in assessing the soil's strength as well as the particle size distribution of the soils under study. From Table 1 mentioned above, the percentage number of clays, sand, and gravel ranged from 31.7 –82.5 %, 12–48.9%, and 3.20 – 36.8%. According to (FMWH 1997), subgrade soils should have less than 35% sand (fine). The finding indicates that the soils are prone to frequent shrinkage and swelling potentials due to seasonal variations, which are typical of the research location's environmental conditions. The high fines content has been connected to the predominance of clay, which may have a dominant control on the mass behaviour of soil, rendering it mechanically unsuitable (Amu, 2010). When compared to (Underwood 1967), which specifies that subgrade soils have fewer than 35% fines, the study's findings demonstrate that 55% do not meet the standard for subgrade soil while 45% do. The particle size chart for soil samples is displayed in Figure 1 below.

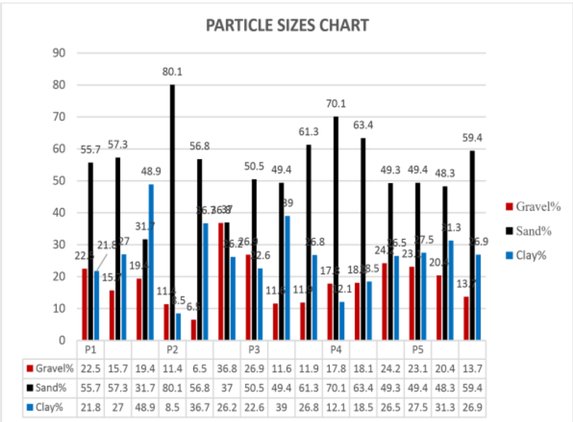

Figure 1: Particle size chart

3.1.4. Atterberg's Limits Tests

The results of the Atterberg's limits test of the soil samples collected from different depths at the four pits (Pits A, B, C and D) are shown in Table 3.

When determining the settling and strength characteristics of soils for structural foundation construction, consistency limits are applied. The liquid and plastic limits, results for the studied soil samples ranged from 25.10% to 52.20%, and 18.70% to 32.40%. The results of the plasticity index for the soil samples were 11.90 – 21.60%, 12.00 – 19.90%, 5.20 – 25.10 % and 8.00 – 24.60%. Due to their plastic natures, all of the examined soils have the potential to cause significant deformation under load. It is worth noting that 95% of the soils investigated have a liquid limit of less than 50%, making them suitable for use as subgrade, sub-base, and base materials in structural foundation construction (FMWH 1997). Furthermore, only eight of the investigated soil samples fall within the specified maximum plasticity index of 20% for subgrade materials (Adams and Maria, 2015), whilst the remaining soil samples are above the stated suggested threshold, making them unsuitable for subgrade construction materials. Soils with a plasticity score of less than 25% have a low to moderate swelling potential. The plasticity index is a major consideration when choosing materials for subgrade and subbase. In contrast to grain size data, it provides far more detailed information on the characteristics and behaviour of clays (Ademilua, 2018). Excessive plasticity frequently causes fluctuations as a result of plastic flow when axle load is applied. Low bearing capacity is common in soils with exceptionally high liquid limits. According to (Casagrande, 1947), soil samples with low, medium, and high plasticity exhibit low, medium, and high compressibility.

The group index is a metric that indicates the load-bearing capacity of a soil group according to AASHTO standards. As a result of the increased group index, the load-bearing capacity will be reduced. According to AASHTO, the soils are classed as A-2, A-2-4, A-2-6, and A-7.

3.2. Soil clay activity

Soil clay activity is the ratio of the plasticity index (PI) and clay content of the soil in a single metric. The activity was calculated using the % clay-sized fraction and plasticity index values to assess the measure and degree of possibility of showing colloidal behaviour. This has something to do with the geologic history and mineralogy of clays found in the soils, the physicochemical characteristics of the individual constituent minerals, as well as the relative proportions of the minerals present, are used to determine the features of clay soils (Bandna, 2016). Table 4 shows that the activity values in the examined soil range from 0.19 to 2.17, indicating active to inactive clays. As a result of the activity levels, the tested soils had low – medium and negligible high expansion ability.

Table 2: Particle size distribution for the soil samples

Soil Sample	Interval (m)	Particle Size Distribution			Specific Grav-itySG	Natural Moisture Content NMC (%)	Optimum moisture Content, OMC (%)	Maximum Dry Density, MDD (k/gm^3)
		Gravel (%)	Sand (%)	Clay (%)				
Pit A	2.00	22.50	55.70	21.80	2.60	11.60	3.00	1912.82
	4.00	15.70	57.300	27.00	2.20	20.00	17.00	1819.67
	6.00	4.76	57.8	22.000	2.20	18.90	18.00	1789.50
	8.00	4.55	57.30	22.000	2.50	21.10	18.00	1899.71
	10.00	19.40	31.70	48.9	2.30	15.40	18.00	1784.86
Pit B	2.00	11.40	80.10	8.50	2.50	12.50	9.00	1826.60
	4.00	6.50	56.80	36.7	2.80	16.40	21.00	1759.34
	6.00	6.20	79.40	32.40	2.70	17.45	18.00	1743.80
	8.00	3.20	76.90	29.00	2.70	19.65	17.00	1722.93
	10.00	36.80	37.00	26.20	2.80	15.00	16.00	1965.42
Pit C	2.00	26.90	50.50	22.60	2.60	22.00	6.00	1497.45
	4.00	11.60	49.40	39.00	2.60	20.60	21.00	1448.85
	6.00	11.90	61.30	26.80	2.80	7.50	11.00	1432.04
	8.00	17.80	70.10	12.10	2.60	13.20	4.00	1407.71
	10.00	16.50	82.50	13.50	2.40	15.10	17.00	1387.95
Pit D	2.00	18.10	63.40	18.50	2.30	12.60	15.00	1486.76
	4.00	24.20	49.30	26.50	2.40	14.90	12.00	1535.14
	6.00	23.10	49.40	27.50	2.70	9.40	5.00	1747.33
	8.00	20.40	48.30	31.30	2.60	14.00	18.00	1922.64
	10.00	13.70	59.40	26.90	2.60	12.20	12.30	1790.25

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Table 3: Atterberg's Limits test results

Soil Sample	Interval (m)	Atterberg Limits			AASHTO
		Liquid Limit (%)	Plastic Limit (%)	Plasticity Index PI (%)	
Pit A	2.00	45.40	23.80	21.60	A-2-7
	4.00	47.90	27.50	20.40	A-7
	6.00	46.20	25.80	20.40	A-7
	8.00	46.10	24.90	21.20	A-7
	10.00	42.40	30.50	11.90	A-6
Pit B	2.00	40.80	22.30	18.50	A-2
	4.00	46.90	29.90	17.00	A-7
	6.00	44.70	24.80	19.90	A-7
	8.00	45.90	27.90	18.00	A-7
	10.00	41.70	29.70	12.00	A-7
Pit C	2.00	31.30	20.50	10.80	A-2-4
	4.00	49.70	26.20	23.50	A-7
	6.00	25.10	19.90	5.20	A-4
	8.00	36.60	18.70	17.90	A-2
	10.00	44.20	19.10	25.10	A-7
Pit D	2.00	30.40	22.40	8.00	A-2-4
	4.00	52.20	29.20	23.30	A-7
	6.00	46.20	21.60	24.60	A-7
	8.00	46.40	32.40	14.00	A-7
	10.00	33.60	22.60	11.00	A-2-6

3.3. Results of the compaction test

The outcome of the compaction test results is shown in Figure 2. The maximum dry densities (MDD) of the soil samples ranged from 1784.86 -1912.82 kg/m^3 for Pit A, 1722.93 – 1965.42 kg/m^3 for pit B, 1387.95 – 1497.42 kg/m^3 for pit C and 1486.76 – 1922.64 kg/m^3. The maximum dry density MDD is less than the prescribed value of 2000 kg/m^3. Five soil samples were found to meet the (FMWH 1997) criteria, which says that the MDD values must be greater than 1700 kg/m^3. According to FMWH (1997), soils should have a maximum dry density between 1500–1780 kg/m^3 and optimum moisture content between 8.56 – 12.02%. As a result of the low optimal moisture content and maximum dry densities, the residual soils in the examined area have limited bearing capacities. The soils cannot be utilized as construction materials unless they are adequately compacted and stabilized to remove void spaces in the soil, which will reduce permeability and improve the strength of the soil materials occasionally. It's worth noting that for any soil to be appropriate as foundation materials, it must be compacted above the MDD and OMC values to provide the needed strength to bear load impact and prevent water infiltration. In comparison to Underwood (1967), the results demonstrated that the soil samples analysed have a fair to poor foundation attribute.

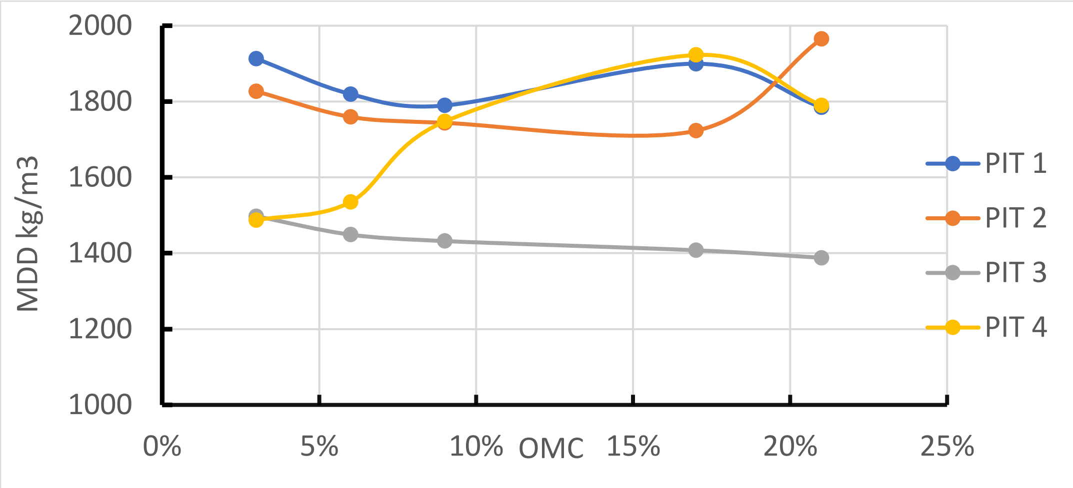

Figure 2: Compaction for the soil samples at the various Pits

Table 4: Soil Clay Activity

Soil Sample	Interval (m)	Plasticity Index PI (%)	Clay (%)	Activity
Pit A	2.00	21.60	21.80	0.99
	4.00	20.40	27.00	0.75
	6.00	20.40	22.000	0.93
	8.00	21.20	22.000	0.96
	10.00	11.90	48.9	0.24
Pit B	2.00	18.50	8.50	2.71
	4.00	17.00	36.7	0.46
	6.00	19.90	32.40	0.61
	8.00	18.00	29.00	0.62
	10.00	12.00	26.20	0.45
Pit C	2.00	10.80	22.60	0.47
	4.00	23.50	39.00	0.60
	6.00	5.20	26.80	0.19
	8.00	17.90	12.10	1.47
	10.00	25.10	13.50	1.85
Pit D	2.00	8.00	18.50	0.43
	4.00	23.30	26.50	0.86
	6.00	24.60	27.50	0.89
	8.00	14.00	31.30	0.44
	10.00	11.00	26.90	0.40

Table 5: Result of the CBR

Soil Sample	Interval (m)	Plasticity Index PI (%)	CBR (%)
Pit A	2.00	21.60	8.02
	4.00	20.40	5.95
	6.00	20.40	4.81
	8.00	21.20	5.13
	10.00	11.90	6.00
Pit B	2.00	18.50	2.96
	4.00	17.00	9.00
	6.00	19.90	4.98
	8.00	18.00	5.89
	10.00	12.00	5.25
Pit C	2.00	10.80	6.00
	4.00	23.50	3.81
	6.00	5.20	5.00
	8.00	17.90	9.81
	10.00	25.10	7.74
Pit D	2.00	8.00	6.54
	4.00	23.30	7.93
	6.00	24.60	9.00
	8.00	14.00	4.98
	10.00	11.00	9.81

*3.3.1. **California Bearing Ratio Test***

The bearing ratio in California Table 5 shows the CBR values for the soils investigated. The California Bearing Ratio is commonly used to assess the strength of subgrade, subbase, and base course soils used in road building. The CBR values for the sample locations ranged from 2.96 – 9.81 for all the soil samples. The findings were less than the desired maximum of 80% (FMWH 1997). These findings imply that the soils are weak, and that soil stabilisation measures would be required to raise the soil strength before it could be used for structural foundation construction. The California Bearing Ratio (CBR) for subgrade materials, according to (Ayininuola and Sogunro, 2013), should be greater than 10%. The CBR values of several soils were found to be lower than the required value. The failure of highway pavement in the research area is partially due to the low CBR values.

Table 6: Variation of Shear strength of the soil samples

PIT	**Interval (m)**	**Cohesion, Friction C (kN/m^2)**	**Angle of Internal Friction (ϕ)**	**Average Deviator Stress (σ_1-σ_3) (kN/m^2)**	**Shear stress = C+(σ_1-σ_3) tanϕ (kN/m^2)**
A	2	35.56	21.94	116.67	82.55
	4	18.28	26.81	161.62	99.95
	6	19.47	30.79	151.28	109.62
	8	18.55	26.92	181.55	110.74
	10	72.70	27.86	180.17	167.93
B	2	99.62	17.20	135.56	141.58
	4	65.37	21.68	156.39	127.53
	6	35.79	33.01	130.38	120.49
	8	28.96	24.40	190.72	115.47
	10	45.77	16.49	220.03	110.90
C	2	32.81	12.04	221.75	80.09
	4	23.19	27.49	182.70	118.24
	6	35.56	21.94	116.67	82.55
	8	45.77	16.49	220.03	110.90
	10	19.47	30.79	151.28	109.62
D	2	28.96	24.40	190.72	115.47
	4	65.37	21.68	156.39	127.53
	6	18.28	26.81	161.62	99.95
	8	99.62	17.20	135.56	141.58
	10	32.81	12.04	221.75	80.09

3.3.2. Triaxial Shear Test

The triaxial test is one of the most dependable techniques accessible for the assurance of shear parameters, an expansion in the shear strength of soil demonstrates an improvement in the strength of the soil and an improvement in construction functionality (Asreazad, 2014). The synopsis of the outcomes for the triaxial test on unstabilized soil specimens is shown in Table 6. The test was performed from soil samples got from the remoulded soil with the OMC obtained from the compaction test. Cell pressure of 20 kN/m^2, 40 kN/m^2 and 80 kN/m^2 was applied. The shear strength of the soil samples ranged from 82,55 – 167.93 kN/m^2, 110.90 –141.58 kN/m^2, 80.09 – 118.24 kN/m^2 and 80.09 - 141.58 kN/m^2 respectively for unstabilized from pits A, B, C and D. The relatively low shear strength further affirms that an improvement prior to the soil before structural foundation construction would be desired.

4. CONCLUSION

The following conclusions were drawn from the tests carried out on the soil samples. The preliminary tests include the moisture content determination, Atterberg Limits, particle size distribution, bulk density, and specific gravity, and the main engineering tests were a triaxial shear test, compaction test and California Bearing Ratio.

The conclusions are explicitly stated below as follows; From preliminary test results, it could be seen that:

i. The soil samples are mostly granular and clayey and compacted easily. Because of the climatic and hydrological conditions in the area, the soil samples show a variable moisture content. The AASHTO classification of the studied soil revealed that 20% of the soils are classified as A-2-4, while the remaining outstanding 80% are classified as A-2-6, A-5, and A-7, all of which reflect fair to poor foundation attributes. The low specific gravity values obtained from the investigation ranging from 2.20 – 2.80 may affect the onshore structural foundation works as an increase in specific gravity values reflects a reduction in the void ratio.

ii. The natural moisture content of the soil samples ranged from 7.50 % to 22%.

iii. The Atterberg's limit s tests revealed that all the samples collected from all depths in pit A have intermediate plasticity index ranging from 11.90 – 21.60 %, 12.00 – 19.90 % for pit B, 5.20 – 25.10 for pit C, and 8.00 – 24.60 for pit D.

iv. The results of the compaction test showed that the maximum dry density for All the soil samples ranged from 1387.95 to 1965.42 kg/m^3.

v. From the CBR test results, it could be inferred that because of the low CBR values (2.96 – 9.81) achieved at all the four locations, the soil materials are unsuitable for onshore structural foundation construction.

Recommendations

The results of the geotechnical investigation showed that the soil materials are of low shear strength, but the strength can be improved when subjected to stabilization measures. This should be put into consideration during the planning and construction of the onshore structural foundation works.

Sections of the locations with sandy and clayey materials should be scooped out from the subsurface to a depth of 3 m – 6 m from the topsoil of the riverbank and put back with competent fill materials. The materials around the proposed onshore structural foundation should be backfilled and well compacted to a stable dry density.

Considering the layer of material at between 2.00 – 6.00 m, the foundation of the proposed onshore structural construction could be padding footing at the depth of 4.00m having an allowable bearing capacity of 180 – 430 kN/m^2.

The foundation of the proposed jetty could be bored and cast in place piles at depth of 6.00 – 10.00 m below the existing ground level.

Future investigations are to be carried out by extending the drilling depth of the soil locations beyond 10 meters to at least 20 meters to find out more about the geological formation of the soil strata for onshore structural foundation works.

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