Geotechnical Characteristics of Regolith Derived from Nanka Formation, Southeast Nigeria

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Abstract — Regolith derived from Nanka Formation; Southeast Nigeria was evaluated for their geotechnical characteristics. The methods of investigations include Fieldwork experiment and laboratory analysis of water and soil samples. The result of hydraulic parameters of the soil at 1meter, 2 meters depth and drilled cuttings from boreholes revealed permeability average values of 1.29E-05(cm/s) and 9.15E-06(cm/s), hydraulic conductivity average value of 1.27E-04(cm/s) and 8.93E-05(cm/s). Drilled cuttings from three boreholes revealed permeability average value of 8.15E-06(m/s), 2.68E-06(m/s) and 6.20E-06, hydraulic conductivity average values of 8.90E-03(m/s), 2.92E-03(m/s) and 6.75E-3(m/s). These values indicate permeable soil with high hydraulic conductivity typical of silty-clay and sand. The permeability/hydraulic conductivity accounts for the high infiltration/percolation of water into the soil. Infiltration of water through the soil initiates geochemical reactions and dissolution mineral which leaves the soil loose and unconsolidated. Geotechnical characteristics show low to medium plasticity and a liquid limit average of 42.36 and 35.45, indicating the capacity of the soil to absorb moisture and expand, bulk density average value of 1.90 mg/m³ and compaction test of maximum dry density average value of 1.80 g/cm³ at an optimum water content average of 12.89% indicate low density. Shear strength components of cohesion values range from 0 to 55KN/m² with average value of 25 KN/m² and friction angle values range from 7° to 25° suggesting low cohesion and angle of internal friction. This is attributed to the low clay content and the cohesive force is not enough to sustain the soil. Field experiments of cone penetration test of in-situ results indicate a weak and incompetent soil material that is unstable and vulnerable to erosion. The finding would be relevant in soil mechanics problems.

Index Terms — Regolith, Geotechnical Characteristics, Nanka Formation, Nigeria.

I. INTRODUCTION

Nanka Formation is demarcated as the loose sand facies of the Ameki group of Formations [20]. The formation is extensive and covers an area of about 1,400km² of southeastern Nigeria. The sand facies is best exposed in gullies developed on the Awka-Orlu escarpment, which is the main topographic expression of the formation and covered by lateritic soil development in other places. The soil is rich in oxides of aluminum and iron which gives it its reddish-brown color. Rock weathering and erosion have been recognized as a major factor in soil development [1], [2] and [26]. The weathered soil material exhibits a distinguishing characteristics and properties that are different from the underlain parent sand facies where it is derived.

Rocks which are stable at higher temperature and pressure deep in the crust may be relatively unstable at the earth’s surface [7]. This is because they are subjected to a series of processes such as weathering when it is exposed to the surface [30]. Weathering and pedogenic processes play a major role in the characteristics of the parent material being eliminated [23]. Two main types of weathering include physical disintegration and chemical decomposition [13]. Physical weathering breakdown rocks fragment without changing the composition. It predominates in dry and cool environment where the heating and cooling of the exposed rocks create physical stress and crack without changing the composition. Chemical weathering alters the structure of the parent material by geochemical reactions into constituent minerals when water comes in contact with the rock or infiltrates into the bedrock. The geochemical reactions are accompanied by the release of soluble minerals that are loss in drainage water or by recombination into new minerals. According to [9] and [22], the consistent flow of soil water and groundwater leaves the system unsaturated by fresh influx of reactants and removal of soluble elements. The derived material is usually loose and unconsolidated residues that over lain their bedrock.

Pedogenesis produces magnetic nanoparticles by a set of organic and inorganic crystal-growth and transformation processes [11] and [18]. Soil formation is one of the strongest expressions of the effectiveness of chemical weathering of bedrock and regolith [8]. A special case of how parent material greatly influences soil properties is the formation of soils on parent material rich in clay minerals [28]. The Soil profile shows that the soil is different in physical characteristics from the parent material. In addition, soils also differ from their parent materials in structural and textural characteristics. Parent materials are transformed by processes which produce a spectacular soil materials or regolith that are unique to the environment.

The formation and occurrence of regolith is widespread and spectacular. Regolith is formed in the temperate and humid tropical regions of the world [4] and [12]. In humid tropical region of south east Nigeria, rainfall is very heavy and precipitated water infiltrates into the soil as well as generates runoff and flood. [27], noted that climate exert a strong influence on weathering. Rocks experience long and intense weathering of destruction and accumulation of minerals with the soluble mineral loss to drainage water and groundwater flow. Weathering produces significant changes in the rock with transformation of the parent material making it vulnerable to erosion. Their characteristics such as
color, texture and properties are different but somehow reflect the original parent material. The transformed rock or regolith occurs in different sizes and depths, their depth ranges from 1 to 2 meters but under suitable conditions, particularly in the humid tropics, weathering can proceed to considerable depths. Shales in Brazil have been altered down to 400ft, and in Georgia granite weathered in-situ to 100ft [5]. The thickness of the weathered soil depends on the weather condition and geomorphic settings.

The effects of weathering on the bulk chemical and mineralogical distribution through weathering profile have been studied [3], [17], [19], [29] and 32. They revealed that weathering is a major factor in the development of soil from bedrock. [24], studied regolith stratigraphy and reported that layering produced by weathering in-situ is different from that produced by depositional processes. The formation of regolith shows distinct property and geotechnical characteristics that are different from their parent materials which may constitute geo-environmental problems. Despite the importance of rocks disaggregation and mineral dissolution processes that turn bedrock into regolith, little is known about the characteristics and the properties of the soil. Hence, the need for assessment of geotechnical characteristics of regolith derived from the Nanka Formation. The method of study will involve fieldwork cone penetration test of in-situ soil, geophysical survey and laboratory analysis of soil samples. The finding would be relevant in soil mechanics problems.

A. Location, Climate and Geology

The study area lies within longitudes 6° 45’ and 7° 19’ East and latitudes 5° 42’ and 6° 18’ North with an area coverage of about 3100 sqkm in Anambra State, southeastern Nigeria.

Two major seasons dominates the study area – rainy season and dry season [15] and [16]. Rainy season occurs from March to October with its peak in July and September, and a short break in August. The dry season is from November to February with the influence of harmattan felt mostly in the months of December and January. These seasonal changes with its attendant variation in temperature, runoff, humidity, atmospheric and pore pressure contribute to weathering of the soil.

Geologically, the study area consists of Nanka Formation underlain by the Imo shale [20] (Fig. 1). The Nanka formation is characterized by friable, loose, and poorly consolidated to unconsolidated deposit of sand, sandstone, shale with clay interactions. The study area is drained by a network of streams and rivers (Fig. 2).

II. MATERIALS AND METHODS

A total of 40 soil samples were collected randomly from boreholes within the study area. Forty test points were carried out using the Dutch Cone Penetration (CPT) machine. The basic principle of the cone penetration test is that a rod is pushed into the ground and the resistance on the tip of the rod measured and recorded. The sample locations and test points were determined using the global positioning system (GPS) and represented on (Fig. 3).

Soil samples were collected at 1m and 2meters depth from different locations using the Hand Auger while drilled cuttings were collected from boreholes as at the time of this study (Fig. 4). Soil samples collected were put in a sample bag sealed, labeled, and transported to the laboratory. Laboratory analysis carried out include particle size distribution, determination of porosity, permeability,
hydraulic conductivity and Atterberg limits test of liquid limit, plastic limit and plasticity index, compaction test (maximum dry density and optimum water content), and triaxial test. For the shear strength test, the Mohr-coulomb failure envelopes were constructed to determine the cohesion and angle of internal friction. Field experiment using cone penetration test (machine) was conducted to test the strength of the soil.

Fig. 3. Location of sample point.

A. Analytical Processes

Soil samples were analyzed using the mechanical sieve shaker and the resulted tabulated using the format for grain size analysis. The percentage passing was plotted against the particle diameter on a semi-log graph paper and from the graph; the particle sizes at D₆₀ and D₁₀ were obtained for calculation of uniformity coefficient.

Uniformity coefficient (U) was computed using the formula:

\[ U = \frac{D₆₀}{D₁₀} \]  (1)

where: D₆₀ is the largest size of the smallest 10 percent fine and D₁₀ is the largest size of the smallest 60 percent fine.

Phi values were obtained from the cumulative curve graph and used to computed for the mean and sorting using the formula below, Geometrical mean (Gme).

\[ (\phi₃₁₆ + \phi₅₀ + \phi₈₄)/3 \]  (2)

where, \( \phi_{316} \) is phi value at 16 percentile, \( \phi_{50} \) is phi value at 50 percentile and \( \phi_{84} \) is phi value at 84 percentiles.

The [31] sorting,

\[ \delta₁ = (d₁₀ − d₈₄)/4.0 + (d₅ + d₉₅)/6.6 \]  (3)

where, \( \delta₁ \) is sorting and d₅, d₁₀, d₉₅, d₈₄ are particle diameter (mm). From the grain-size analysis, hydraulic properties were estimated viz; porosity, permeability (k) and hydraulic conductivity (K).

B. Porosity (n)

Porosity defines the percentage of pore spaces in the soil. Its values were derived from the empirical relationship between standard values and the uniformity coefficient of grain (Cu) according to [31] as follows:

\[ n = 0.255(1 + 0.83u) \]  (4)

where n is porosity, u is the uniformity coefficient.

C. Hydraulic Conductivity (K)

Hydraulic conductivity was calculated by substituting the mean and sorting values into the [14] equation to calculate for permeability (k) which was substituted into the hydraulic conductivity (K) Equation.

\[ k = 760(M)^2 e^{(-1.31Qb)} \]  (5)

where k is permeability in Darcy (d), M is geometric mean (mm) and Qb is the sorting.

Hence,

\[ K = k\delta g/\mu \]  (6)

K is hydraulic conductivity (cm/s), \( \delta \) is density of water = 0.9982 (g/cm³), k is permeability (cm²) conversion factor 9.87x10⁻⁵ (cm²/d), \( \mu \) is viscosity of water 0.01 (g/cm²), g is acceleration due to gravity =980 (cm/s²).

D. Atterberg Limits Test

Atterberg Limit test includes the Liquid Limit, Plastic Limit, and the Plasticity index.

E. Liquid Limit Test (L.L)

The liquid limit is the moisture content at which the soil will flow under its own weight. A paste of a given quantity of the soil was made and placed in a Casagrande cup and a groove made in the center with a spatula. It was determined by ascertaining the moisture content at which two halves of the soil paste will come together for a distance of 13mm along the bottom of the groove separating the two halves of the soil paste on the Casagrande cup. The Casagrande equipment with the soil sample was subjected to blow count when the cup is lifted up and dropped down to close the groove at a rate of 2 drops per second. The values obtained were plotted on a graph of moisture content against the number of blows. From the graph, the moisture content at 25 blows represents the liquid limit (L.L).

F. Plastic Limit Test (P.L)

The plastic limit test was carried out by making a paste of the soil with distilled water. The paste was formed into a ball shape and rolled between the hand and the glazed plate with enough pressure to form a thread of uniform diameter. If the diameter of the thread decreases to 3mm without crumbling, the soil is re-kneaded together and rolled out again. This process of re-kneading reduces the moisture content by evaporation and the test is repeated at least three times with the same soil sample.

G. Plasticity Index (P.I)

Plasticity index is the numerical difference between the liquid limits and the plastic limit.

\[ L.L − P.L = P.I \]  (7)

H. Moisture Content Test (Mw)

The moisture content (Mw) is defined as the ratio of the weight of water to the weight of the solids in the same soil sample. The wet soil sample was weighed and then oven-dried to a constant weight at a temperature of about 105 °C.

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After oven drying the sample was reweighed to get the weight of the solid. The change in weight is equivalent to the weight of the water, which is calculation with the formula below:

\[ M_w = W_{(\text{wet})} - W_{(\text{dry})} - W_{(\text{pan})} = \frac{W_{W}}{W_{S}} \]  

where \( W_{(\text{wet})} \) is weight of wet Sample, \( W_{(\text{dry})} \) is weight of dry sample, \( W_{(\text{pan})} \) is weight of empty pan, \( W_{W} \) is the moisture content (weight of water), \( W_{S} \) is weight of solids.

I. Compaction Test

Compaction test was used to determine the dry density and optimum moisture by compaction of the soil sample in a test mould of known volume using a specified compaction weight. The water content and the weight of the soil sample required to fill the mould were determined. Results obtained were plotted as dry density against water content on a graph. Bulk density and dry density were calculated using the following formula:

Bulk density \( = \frac{(M1 - M2)}{1000 \text{ mg} / \text{m}^3} \)  

Dry density \( = \frac{(M2 - M1)}{(1000 (1 + m))} \)

\( M1 \) is mass of mould (g), \( M2 \) is mass of mould and compacted sample (g), 1000 is the volume of mould (cm\(^3\)), and \( m \) is moisture content.

J. Triaxial Shear Test

Triaxial compression test was carried out using a defined cylindrical specimen of the soil sample placed in the triaxial machine and subjected to a constant all round fluid pressure (\( \sigma_3 \)) and the two ends subjected to axial stress (major principal stress \( \sigma_1 \)). The stress condition in the soil goes on changing with the increase of the major principal stress. The surrounding fluid was pressurized and the stress on the plates increased until the material in the cylinder fails and forms sliding regions within it, known as shear planes. The test result obtained was used to plot the Mohr circle diagram, from which the values of the cohesion and angle of internal friction were obtained as shear strength parameters.

K. Fieldwork

A total of 40 test points was carried out using the Dutch Cone Penetration (CPT) machine. The basic principle of the cone penetration test is that a rod is pushed into the ground and the resistance on the tip of the rod measured and recorded.

L. Dutch Cone Penetration Test (CPT)

The cone has a cross sectional area of 1000 \( \text{cm}^2 \) and an apex angle of 60\(^\circ\) which was pushed to the required depth at a rate of 20 \( \text{mm/s} \) in the close position by exerting pressure on the outer conning tube. The force on the cone was read from the gauge meter as the cone was push downward by means of the inner pressure rod independent of the outer sounding tube. This procedure was repeated at regular interval of 0.25 \( \text{m} \) depth. The data obtained from the in-situ cone penetration test were used to plot the cone resistance against depth (fs/qc). From the graph the soil type was identified, and their bearing capacity computed using the equation,

\[ Q_a = 2.7 \times Q_c \text{ KN/m}^2 \]  

where \( Q_a \) is allowable bearing capacity and \( Q_c \) is Cone resistance.

III. RESULTS

The sedimentary sequence as observed from on-site boreholes logs of different locations show a succession of lateritic topsoil, clay, shale, silt, sand, sandstone, and gravel (Fig. 4 and Fig. 5).

Fig. 4. Lithologic sections of drilled borehole.

Fig. 5. Lithologic sections of drilled borehole.
Fig. 6 and Fig. 7 shows a section of the regolith exposed from the Nanka Formation that is highly weathered. The topsoil is lateritic in nature and often derived from the underlying geologic unit. Some of the soils are cemented and indurated to form thin to thick ironstone bands [20]. The iron oxide is responsible for reddish-brown colour of the soil.

A. Hydraulic Parameters

Table I results of hydraulic parameters of the soil at different locations from one meter, two meters depth reveal permeability values range from 9.95E-07 (cm/s) to 4.12E-05 (cm/s) with average value of 1.29E-05 (cm/s) and 1.32E-06 (cm/s) to 3.76E-06 (cm/s) with average value of 9.15E-06 (cm/s) for 1 meter and 2 meters depth, hydraulic conductivity values range from 9.77E-06 (cm/s) to 4.04E-05 (cm/s) with average value of 1.27E-04 (cm/s) and 1.30E-06 (cm/s) to 3.69E-04 (cm/s) with average value of 8.93E-05 (cm/s).

Table II results of range of values for hydraulic properties of drilled cuttings from borehole one, borehole two and borehole three reveal permeability values range from 2.70E-06 (m/s) to 1.97E-05 (m/s) with average value of 8.15E-06 (m/s), 1.33E-06 (m/s) to 5.22E-06 (m/s) with average value of 2.68E-06 (m/s) and 3.46E-06 (m/s) to 1.10E-05 with average value of 6.20E-06 (m/s) respectively, hydraulic conductivity values range from 2.94E-03 (m/s) to 2.15E-02 (m/s) with average value of 8.90E-03 (m/s), 1.45E-05 (m/s) to 5.69E-03 (m/s) with average value of 2.92E-03 (m/s) and 3.77E-03 (m/s) to 1.20E-02 (m/s) with average value of 6.75E-03 (m/s) respectively. The result indicates that the soil material is permeable with high hydraulic conductivity typical of silty-clay and sand horizon within the unconsolidated deposit. The permeability/hydraulic conductivity accounts for the high infiltration/percolation of water into the soil. Infiltration of water through the soil initiates geochemical reactions and the release of soluble minerals which are loss to water draining downward leaving the material loose and unconsolidated.

B. Geotechnical Characteristics

The geotechnical characteristics were evaluated viz; grain size analysis, porosity, tests of Atterberg limits, compaction, triaxial and cone penetration in situ test of soil. Table II shows grain size analysis for the soil samples at one meter and two meters depths. Uniformity coefficient values range from 2.14 to 10 with average values of 3.72 and 2.13 to 8.24 with average value of 4.18. Sorting values range from 0.72 to 1.77 with average value of 1.32 and 0.47 to 2.73 with average value of 1.48. The porosity values range from 29% to 43% with average value of 39% and 31% to 43% with average value of 38%.

| Parameters          | Borehole 1 | Ave | Borehole 2 | Ave | Borehole 3 | Ave |
|---------------------|------------|-----|------------|-----|------------|-----|
|                     | Range      | Ave | Range      | Ave | Range      | Ave |
| 1 Permeability (K/m/s) | 2.70E-06 -1.97E-05 | 3.15E-06 | 1.33E-06 -5.22E-06 | 1.10E-05 | 2.68E-06 | 3.46E-06 -1.10E-05 | 6.20E-06 |
| 2 Hydraulic conductivity (K/m/s) | 2.94E-03 - .2.15E-02 | 8.90E-03 | 1.45E-05 -5.69E-03 | 2.92E-03 | 3.77E-03 -1.20E-02 | 6.75E-03 |
Table IV show results of drilled cutting from boreholes one, two and three. The uniformity coefficient values range from 1.41 to 3.94 with average value of 2.22, 2.27 to 4.62 with average value of 3.32 and 1.70 to 5.29 with average value of 3.06 respectively. These values indicate uniform soil that is poorly graded and easy to erode. Analysis of grain sizes from the particle size distribution curves show that fine grained values range from 0% to 5%, medium grained 75% to 80% and coarse grained 5% to 10%. All soil samples analyzed revealed that the percentage of fines decreases with increase in the percentage of sand. The porosity values for the drilled cuttings range from 38% to 45% with average value of 43%, 36% to 42% with average value of 39% and 35 to 44 % with average value of 40% respectively These percentages indicate porous sediments.

Table V results of Atterberg limits test of soil samples from one meter and two meters depth shows liquid limit values range from 24.00 to 63.00 with average values of 42.36 and 0.16 to 64.00 with average value of 35.45, plastic limit values range from 8.59 to 42.00 with average value of 25.44 and values of 0.30 to 48.87 with average value of 23.39, plasticity index values range from 1.00 to 42.08 with average value of 17.10 and values range of 0.54 to 40.33 with average value of 15.07. All samples tested show low to medium plasticity when compared with the reference limits [6] and [26]. The grain size analysis result combine with the Atterberg limit test results aided the classification of the soil as silty/clayey sand.

Table VI result of soil samples from one meter and two meters depth shows bulk density values range from 1.72 mg/m³ to 2.10 mg/m³ with average value of 1.90 mg/m³, indicate low density soil. The compaction test of maximum dry density values range from 1.67 g/cm³ to 2.38 g/cm³ with average value of 1.80 g/cm³ at an optimum water content values range from 8.90% to 18.30% with average water content of 12.89% Comparing these values with [21], it can be deduced that all samples satisfied the compaction standard and can be classified as silty/sandy clay. The low compacted and bulk density indicates poorly consolidated soil which is prone to erosion. This is in line with the observation of [10]. In addition, Fig: 8 graph of compaction curves and their corresponding optimum water content shows a smooth concave curves exhibited by uniform soil. The upward and right shift shows increasing water content and the optimum water it can retain at the maximum (peak) dryness.

### Table III: Range of Values for Geotechnical Parameters of Soil at One Meter and Two Meters Depth

| s/n | Parameter              | Range | Ave | Range | Ave |
|-----|------------------------|-------|-----|-------|-----|
| 1   | Coefficient uniformity (Cu) | 2.14 – 10.00 | 3.72 | 2.13 – 8.24 | 4.18 |
| 2   | Porosity (n) %          | 29 – 43% | 39% | 31 – 43% | 38% |
| 3   | Sorting                 | 0.72 – 1.77 | 1.32 | 0.47 – 2.73 | 1.48 |

### Table IV: Range of Values for Geotechnical Parameters of Drilled Cuttings from Borehole (Sandy Horizon)

| Parameters          | Borehole 1 | Borehole 2 | Borehole 3 |
|---------------------|------------|------------|------------|
|                     | Range | Ave | Range | Ave | Range | Ave |
| Uniformity coefficient (Cu) | 1.41–3.94 | 2.22 | 2.27–4.62 | 3.32 | 1.70–5.29 | 3.06 |
| Porosity (n) %      | 0.38–0.45 | 0.43 | 0.36–0.42 | 0.39 | 0.35–0.44 | 0.40 |

### Table V: Range of Values for Atterberg Limit Test

| s/n | Parameter | Range | Ave | Range | Ave |
|-----|-----------|-------|-----|-------|-----|
| 1   | Liquid Limit (LL) | 24.00 – 63.00 | 42.36 | 0.16 – 64.00 | 35.45 |
| 2   | Plastic Limit (PL) | 8.59 – 25.44 | 24.00 | 0.30 – 48.87 | 23.39 |
| 3   | Plasticity index (PI) | 1.00 – 42.08 | 17.10 | 0.54 – 40.33 | 15.07 |

Table VI Range of Values for Geotechnical Parameters

| s/n | Parameters | Range | Ave | Average |
|-----|------------|-------|-----|---------|
| 1   | Bulk density (mg/m³) | 1.72 – 2.10 | 2.60 |
| 2   | Maximum density (MDD)(g/cm³) | 1.67 – 2.38 | 2.00 |
| 3   | Optimum moisture content(%) (OMC) | 8.90 – 18.30 | 12.89 |
| 4   | Cohesion (KN/m) | 0 – 55 | 25.00 |
| 5   | Friction angle | 7° – 25° | - |
| 6   | Angle of inclination | 49° – 58° | - |
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**TABLE VII: RANGE OF VALUES FOR CONE RESISTANCE (qc) kN/m² AND ALLOWABLE BEARING CAPACITY (qa) KN/m²**

| S/ N | Locations          | Cone resistance (qc) kN/m² | Allowable bearing capacity (qa) KN/m² |
|------|--------------------|----------------------------|--------------------------------------|
|      | Range              | Ave                        | Range                                | Ave                        |
| 1    | Otolo Nnewi        | 0 – 30                     | 13.57                                | 0 – 81                     | 36.64                     |
| 2    | Ozubulu            | 0-220                      | 68.81                                | 0 – 594                    | 185.79                    |
| 3    | Otc Cable Nnewi    | 0-75                       | 22.62                                | 0 – 202.5                  | 61.07                     |
| 4    | Nnewichi           | 0-135                      | 47.14                                | 0 – 364.5                  | 122.79                    |
| 5    | Ebenator Ubaru     | 0-25                       | 10.71                                | 0 – 67.5                   | 28.93                     |
| 6    | Unuogho Nnewi     | 0-150                      | 54.29                                | 0 – 405                    | 146.57                    |
| 7    | Isu-Umuedem Oba    | 0-60                       | 18.10                                | 0 – 162                    | 48.86                     |
| 8    | Amanoba Obosi      | 0-90                       | 31.19                                | 0 – 243                    | 84.21                     |
| 9    | Osumentyi Akabo    | 0-50                       | 23.57                                | 0 – 135                    | 63.64                     |
| 10   | Umuzu Uke          | 0-35                       | 31.43                                | 0 – 202.5                  | 84.86                     |
| 11   | Ire Oboi 1         | 0-25                       | 10.00                                | 0 – 67.5                   | 24.42                     |
| 12   | Ire Oboi 11        | 5-45                       | 15.95                                | 13.5-121.5                 | 43.07                     |
| 13   | Ire Oboi Odaasius  | 10-60                      | 27.86                                | 27-162                    | 75.21                     |
| 14   | Ezinifite          | 0-20                       | 7.86                                 | 0 – 54                    | 21.21                     |
| 15   | Omagba Phase 2     | 0-20                       | 7.38                                 | 0 – 54                    | 19.93                     |
| 16   | Nkpors New Parts   | 0-50                       | 16.19                                | 0 – 135                   | 43.71                     |
| 17   | Abba Gully         | 0-230                      | 92.62                                | 0 – 621                    | 250.07                    |
| 18   | Enugu Ukwu         | 10-55                      | 36.54                                | 27-175.5                  | 91.04                     |

**Fig. 9 a, b. Triaxial graphs.**

**Figures:**
- Nanka Site
- Amaeze Orakwukwu
- Umuru Ide Alor
- Nnobi Community School

**Note:**
- Table values are approximate and based on specific measurements.
Table VII shows the range and average values of the penetrative resistance (qc) and the calculated allowable bearing capacity (qa). Field experiments of cone penetration test of in-situ soil results were used to determine the allowable bearing capacity of the soil material which was compared with the average value for penetrative resistance of competent subsurface soil material given as 100 kg/m². The result indicates a weak and incompetent soil material. Most of the values fall below the standard with the exception of few samples which implies that the soil is prone to erosion and hence gully development.

IV. DISCUSSION

Geologically, the soil consists of layered sequence of lateritic top soil, sand, sandstone, shale with intercalations of clay and siltstone. These geologic units enhance the groundwater resources potential of the study area. However, their good hydrogeological properties influence the geotechnical characteristics/behavior of the soil. The soil properties determined were permeability (k), hydraulic conductivity (K), grain size analysis from which the uniformity coefficient and porosity values were evaluated.

Almost all soil samples show uniform grain and poorly graded soil, porous and permeable with high hydraulic conductivity that favors the movement of water through it. Infiltrating water may initiate geochemical reactions and subsequent leaching of soluble minerals. This will leave the soil material loose and unconsolidated. The bulk density and compaction density values show that the density of the soil is relatively low, poorly consolidated and in line with the observation of [10]. The low densities might be a reflection of the poor bonding condition that exist between the soil particles. The moisture content shows that the soil is not saturated and will allow infiltration of water into it which is probably a reflection of the vadose zone.

The results of Atterberg limit test show some degree of plasticity despite the sandy nature of the soil. The low to medium plasticity accounts for the drying of the soil and the formation of tension and desiccation cracks on the land surface. These cracks maybe transferred into the sand material where they serve as pathways for water penetration to deeper depths. Furthermore, the liquid limit and the plastic limit test indicate that the clay has the capacity to absorb moisture which may cause it to expand and contract. This hydrophilic tendency of the soil is detrimental to slope stability.

The shear strength results indicate low cohesion and angle of friction. The low cohesion is attributed to the low clay content of the soil. The friction angle can be drastically reduce due to infiltration of rainfall where cohesion and friction force is not enough to sustain the soil particles together. Any change by increase in water content of the soil will reduce the grain contact and shear strength which may trigger-off erosion and gully development. It is also possible for the soil to fail as observed by applying the Mohr-coulomb criterion for rapture of material. Failure is eminent because the failure envelop which touch the flow line at a point indicate the critical point representing the shear strength. This implies that there exists a critical slope where seepage is erosive and any increase above the shear point/threshold may cause the soil to fail or flow.

Field experiments of cone penetration test of in-situ soil were used to determine the bearing capacity of the soil material. The result indicates low allowable bearing capacity and an incompetent soil material. This observation agrees with the shear strength that the soil material is incompetent and unstable and therefore vulnerable to erosion.

V. CONCLUSION

The following conclusions were drawn from this study:

Regolith is a highly weathered soil material is a loose and poorly consolidated residues held by a weak bond and overlain their parent material. The characteristics of the soil expresses uniformly grained to poorly graded, porous, and permeable with high hydraulic conductivity. These characteristics compares favorably with the loose and unconsolidated deposit of the Nanka Formation. The Formation exhibits typical Geotechnical characteristics of loose and poorly consolidated sediment. The soils show some degree of plasticity and the capacity to absorbed moisture. This would lead to expansion and contraction of the soil and consequently loss in shear strength which is detrimental to slope stability and hence gully development. The values of the bulk density and compaction density with its optimum water content shows that the density of the soil is relatively low and poorly consolidated. The low density is as a result of the poor bonding condition that exists between the soil particles. The upward and right shift of the density graph shows increasing water content and the optimum water it can retain at the maximum (peak) dryness. Therefore, the chances of detachment are possible because the soil material cannot withstand much stress.

Field experiment of cone penetration test of in-situ soil indicates low allowable bearing capacity and an incompetent soil material. This observation agrees with the shear strength components of low cohesion and angle of internal friction. The soil material is incompetent and unstable and therefore vulnerable to erosion. These characteristics compares favorably with the loose and unconsolidated deposit of the Nanka Formation.

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