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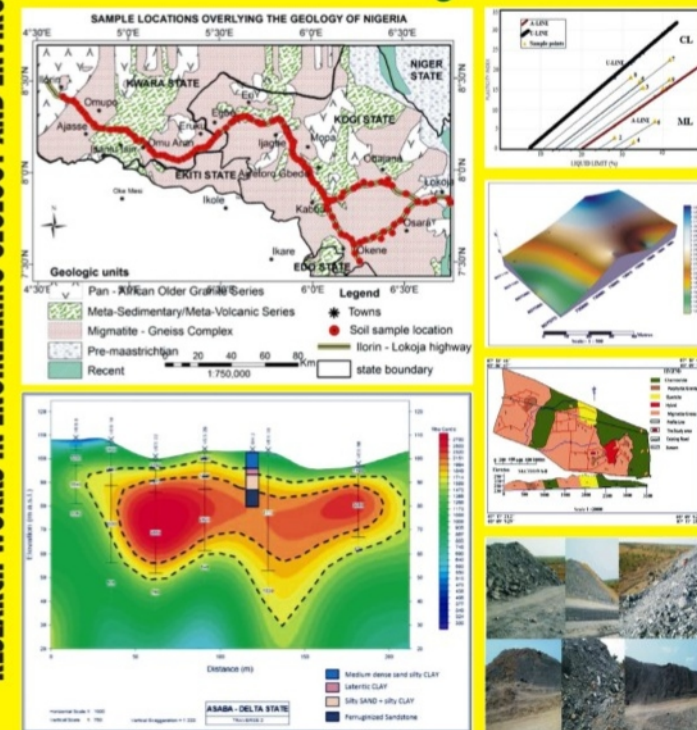
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Multivariate Assessment of California Bearing Ratio with Contrasted Geotechnical Properties of Soils

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Abstract

California Bearing Ratio (CBR) is an important parameter used in designing pavement layers in road construction but testing this parameter requires time, labour and huge cost. The study therefore applies multivariate approach to evaluate CBR based on contrasted geotechnical parameters along Ilorin-Lokoja highway. The results obtained showed that migmatite-gneiss-derived soils constitute more fines (<0.075mm; 7.4-59.6%), more plastic (PI; 1.6-39%), and have low strength (MDD=1.8mg/m³; CBR=29.0%) than the meta-sediments (11-57.7%, 2.0-30%, 1.6mg/m³, 23.6%) and older granite soils (8.2-32.7%, 2.6-13.4%, 1.7mg/m³, 27.8%) respectively. This signals problematic conditions on structures particularly in the migmatite area. Principal Component Analysis (PCA) revealed three major components (eigenvalues>1) which accounted for 83.8% of the total variance at 33.4%, 14.7% and 11.4% rate. Major contributing variables for the components were fines (R=0.87), plasticity index (R=0.7) and coarse sand (R=0.67%). Spatial distribution of these groups established interplay of sediment-gradation and moisture-connection evidenced in hierarchical clustering which revealed patterns of homogeneity and soil relationships. Regression analysis established five models from predictor variables such as fines, activity, free swell, liquid and plastic limits, wPI, OMC and MDD with coefficient of determination (R² = 0.33) and root mean square error (RMSE) of 7.8.

Keywords: *Multivariate, Principal Component Analysis, Hierarchical Analysis, Dendrogram, Cluster Analysis, Geotechnical Properties.*

Introduction

Identification and quantitative characterization of soils are of dire importance in geotechnical assessment despite the difficulties experienced using conventional approach. Index properties are important parameters in evaluating geotechnical conditions, particularly estimating soil strength but to measure compaction and CBR takes 2 – 4 days with attendant cost, severe labour and skill. Consequently, these tests are avoided in many soil investigation projects. Thus, the need to incorporate statistical approach in predicting soil properties becomes inevitable.

Several authors have applied this approach in relating and predicting soil properties. Patel and Desai (2010) considered one to one relationship among soil properties like liquid limit (LL), plastic limit (PL), plasticity index (PI), optimum moisture content (OMC), and maximum dry density (MDD). Furthermore, Carter and Bentley (2016) noted the importance of soil type, density, moisture content in soil relationship and correlated soil expansion index and plasticity index, fine fraction and weighted PI (i.e. product of PI and percentage passing 0.425mm). In addition, Owoseni et al. (2012) and Yildirim and Gunaydin (2011) added that CBR depends on other factors such as type of soils, permeability of soil, MDD and OMC. To correct overlapping problem and uncertainty in prediction,

Yitagesu et al. (2011) applied multiple regressions to improve the ability of soil prediction that better model the extent of their relationship.

This paper attempts to identify geotechnical characteristics of soils developed on different rocks and establish relationships among various properties in order to estimate soil strength capability in three lithological units. Multivariate approach using Principal Component Analysis (PCA) and hierarchical classification methods help to identify patterns, detect and classify parameters into new groups and then propose regression models to determine CBR values in view of its attendant cost and labour.

Materials and Methods

One hundred and thirty one (131) soil samples were collected along the Ilorin – Lokoja highway (>300km length) which spans across Latitude 7°25'N - 8°40'N and Longitude 4°30'E – 6°45'E with coordinate readings using Garmin GPS. The topography is relatively flat to hilly, undulating terrain with elevation ranging between 100m – 700m above sea level. Geologically, the highway is overlying the Precambrian Basement rock of South Western Nigeria (Fig. 1) and cut across three units: the migmatite - gneiss complex (denoted by PCB), the metasediments / volcanic series (PCM) and the older granite series (PCG) (Oluyide et al., 1998).

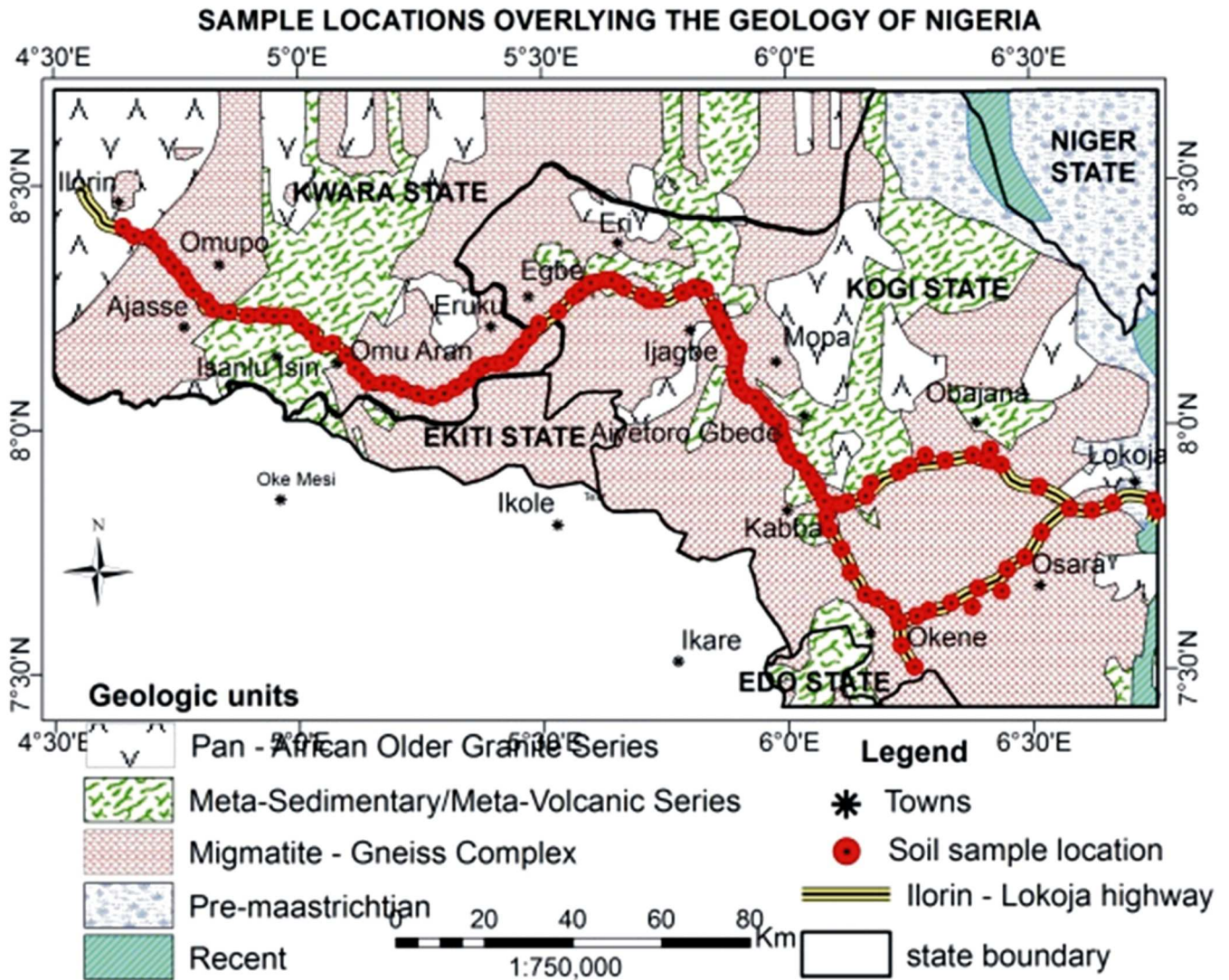


Fig. 1: Geology of Nigeria showing the study highway overlain by soil sample locations.

The area is majorly of migmatite – gneiss origin essentially made up of migmatite and banded gneiss. Others are flaggy quartzite with biotite gneiss, undifferentiated schist, porphyritic granite (porphyroblastic), and medium-coarse grained biotite and hornblende granite. Temperature ranges from 25° – 35°C. Climate is dry to wet, with a mean annual rainfall of 1200mm. Due to heavy rainfall, considerable moisture change occur which dries up at prolonged dry season inducing soils susceptibility.

Laboratory Analysis

Geotechnical tests were carried out on air - dried (35°C - 40°C) soil samples at the Soil Geotechnical Laboratory of Nigerian Building and Road Research Institute following the British Standard (BS 1377:1990), Part 2:

Clause 9.2, 4.5, 5.3, 5.4, and Part 4, Clause 3.3 and 3.4 methods. The soil engineering parameters obtained include natural moisture content, Atterberg limits, particle size distribution, free swell, compaction and California Bearing Ratio (CBR).

Statistical Analysis

In SPSS statistical software, 20 soil parameters were explored and their relationships examined. The strength and relationship trend was examined from Pearson correlation matrix while determining the quantitative measures of linear associations. Principal Component Analysis (PCA) was incorporated to reduce the data with many variables, identify clusters, and transform the soil variables into new uncorrelated variables that preserve most of the information (Jolliffe, 2002).

Components with eigenvalues > 1 were retained and subjected to varimax rotation to maximize correlation between the factors and measured soil variables. Hierarchical cluster analysis was computed to identify analogous behavior among different soil characteristics and soil individuals using Ward's method and squared Euclidean distance as a measure of similarity between soils (Murtaghand Legendre, 2014).

Results and Discussion

Statistically, summary of the laboratory data is shown in Table 1. The soils exhibited wide variations of data clustering around the mean value (1.08 – 88.6) and high coefficients of variation (1.7 - 147%). The median of some parameters was lower than the mean value, indicating low effect of abnormality on sampling values.

Particle Size Characteristics

In the PCB derived soils, gravel and coarse sand varied with coefficient of variation (CV) from 18.3 – 100% (<23.5%), medium to fine sand was between 8.0 – 86% (>32%) while the percentage of silt and clay were 3.2 – 50.8% (52.4%) and 0.9 – 34.6% (58.8%) respectively. However, the percentage of fines (<0.075mm) ranged between 7.4 – 59.6% (48.0%). This proportion of fines

is similar to those reported by (Ige et al., 2018). In the PCM derived soils, content of gravel and coarse sand were higher from 43 – 100% (<20.6%), than medium to fine sand (15.2 – 84.2%) with about 36% CV. Similarly, the percentage of silt and clay ranged between 6.1 – 44.3% (41.2%) and 1.8 – 28.7% (46.3%). Relatively, the proportion of fines (11.7 – 57.7%; 36.6% CV) is as high as those in the PCB area. Similarly, PCG soils exhibited wide range of gradation with gravel and coarse sand between 19.4 – 100% (<33.3%). Medium to fine sand content was lower (14–92%) (CV=28–34.9%) while percent silt and clay varied between 2.4 – 24.5% and 1.2 – 18.8% (CV= 48.6 – 60.8%) respectively. The amount of fines (8.2 – 32.7%) and CV (34.6%) is lowest in the area. This granularity is similar to the work of (Nwaiwu et al., 2006) where the lateritic soils are enriched with gravel and sands between 28.2-40% and 42.2-48% respectively. However, the high percentage passing through No. 200 (0.075mm) suggests the soils are of fine materials and classified according to AASHTO and USCS systems as A-2, A-4, and A-7 soils and clayey sand (SC), silty sand (SM) and silty, clayey sands (SC-SM). Other minor classes include poorly graded sand with silt or clay (SP-SM, SP-SC), poorly graded gravel with clay or silty clayey gravel (GC-GM), silty gravel (GM), sandy lean or fat clay (CL, CH), and sandy silt or elastic silt (ML, MH).

Table 1: Statistical summary of the soil properties

Properties	Units	Migmatite-Gneiss (PCB) = 87					Meta-sediment/Meta-volcanic (PCM) = 33					Older Granite (PCG) = 11					
		Min	Max	Mean	SD	CV	Min	Max	Mean	SD	CV	Min	Max	Mean	SD	CV	
NMC	%	2.6	22.7	11.8	4.5	38	3.1	18.4	12.1	4.1	34	2.4	16.2	7.5	4.3	56.8	
Gravel		38.6	100	89.3	13.2	14.8	65	100	87	10.5	12.1	45	100	85.8	19.6	22.8	
CS		18.3	99.2	80.4	18.9	23.5	43.1	100	74.6	15.3	20.6	19.4	100	77.5	25.8	33.3	
MS		13.6	86	51.5	16.6	32.3	21.5	84.2	53.8	15.3	28.5	18.9	92.1	53.9	18.8	34.9	
FS		8	79.5	28.9	14.3	49.4	15.2	71.1	36.8	13.3	36.1	14	41	28	7.9	28	
Sand		20	87	59.5	14.3	24	25	81	50.1	13.9	27.8	31	81	57.6	17.4	30.2	
Silt		3.2	50.8	17.1	9	52.4	6.1	44.3	20.3	8.4	41.2	2.4	24.5	15.5	7.5	48.6	
Clay		0.9	34.6	12.3	7.3	58.8	1.8	28.7	15.2	7	46.3	1.2	18.8	10.8	6.6	60.8	
Fines		7.4	59.6	23	11	48	11.7	57.7	28.6	10.4	36.5	8.2	32.7	21.8	7.5	34.6	
Ac		0.1	8.5	1.1	1.2	110	0.2	7.9	0.9	1.4	147	0.2	5.1	1.2	1.4	123	
Fsw		1.8	28.4	6.4	5.7	88.3	2.8	19.6	7.1	5.1	72.2	3.2	4.7	4	0.5	11.6	
LL		13.4	69	28.4	11.7	41.1	13.4	48.5	30.3	9.7	32	16.5	32.4	23.2	5.7	24.5	
PL		2.2	50	19	9.6	50.5	10.2	31.7	20.3	6.3	31	8.6	26.5	16.6	5.7	34.5	
PI		1.6	39	9.4	6.5	68.7	2.1	30.1	10	7.3	72.6	2.6	13.4	6.5	3.4	51.9	
wPI		0.3	31.9	5.1	4.6	90.6	0.9	25.3	5.8	5.7	97.8	0.9	7.3	3.5	2.1	60.6	
BD		mg/m ³	1	2.9	2	0.3	15	1.5	2.2	1.9	0.2	8.5	1.7	2.1	1.9	0.1	5.9
DD			0.8	2.4	1.7	0.2	13.9	1.2	1.9	1.5	0.2	10.8	1.5	1.9	1.6	0.1	8
MDD	0.9		2.6	1.8	0.3	14.4	1.3	2.1	1.6	0.2	11.9	1.6	1.9	1.7	0.1	7.6	
MC	%	7.6	27	15.9	4.8	29.8	7.6	23.9	16.5	3.8	23	9.1	19.7	14.1	3.6	25.8	
OMC		5.7	25	13	4.2	32.2	10.1	22.5	15	2.9	19.6	8.9	18.2	14	2.9	20.8	
CBRu		12.5	70	50.8	12.4	24.4	17	75.1	47.7	11.9	25	30.4	58.6	52.5	8	15.3	
CBRs		10	56.4	29	9.7	33.4	11.2	45	23.6	7.1	30	12.1	37.2	27.8	6.5	23.3	
SP		1.5	15.4	5.5	2.8	51.9	2	11.6	6	3	49.8	2.3	7.9	4.3	1.6	38.4	
Dr		0.7	1	0.9	0.1	4.6	0.9	1	0.9	0.2	3.2	0.9	1	1	0.2	1.7	
Wr		0.8	2	1.3	0.2	19.7	0.8	1.6	1.1	0.2	20.9	0.6	1.6	1	0.3	26.8	
LLr	0.7	6.3	2.7	1.1	42.8	1.3	9.1	2.9	1.7	58.3	2	7.8	3.9	1.9	48.3		
PIr	1.1	8.1	1.7	1	56.2	1.1	2.6	1.5	0.4	25.7	1.1	2	1.5	0.3	21.5		

NMC = natural moisture content, CS, MS, FS = coarse, medium, fine sand, LL = liquid limit, PL = plastic limit, wPI = weighted plasticity index, Ac = activity, Fsw = free swell, BD = bulk density, DD = dry density, MDD = maximum dry density, OMC = optimum moisture content, CBRu and CBRs = unsoaked and soaked California Bearing Ratio, CV = coefficient of variation, SD = standard deviation, Dr = density ratio (DD/MDD), Wr = moisture ratio (MC/OMC), LLr = liquid limit ratio (LL/MC), PIR = plasticity ratio (LL/PL) and SP = swelling potential (PI/PIr)

Consistency Limits

A wide range of plasticity (Fig. 2) characterizes the inorganic silty clayey soils in the area. The liquid limit varies between 13.4 - 69% and lower in PCG derived soils (<32.4%), while plastic limit and plasticity index in the PCB area ranges between 2.2 – 50% and 1.62 – 39%, with mean values of 28.4%, 19.0%, and 9.4% respectively (Table 1). The Casagrande plasticity chart reveals majority of the soils from the migmatite – gneiss origin occurs above the A-line, indicating that they are of inorganic clay material and exhibits low to medium plasticity, inferring low to medium swelling and compressibility. The moderate plasticity suggests low to medium dry strength that could easily crumble under load thus leading to pavement failure and possible erosion under climatic threat. The distribution of the soil samples on the chart portrays the variability in soil plasticity characteristics. Also, free swell (Fsw) varies from 1.8 – 28.4% in PCB, 2.8 – 19.6% in PCM and 3.2 – 4.67% in PCG with mean values ranging from 6.4%, 7.1% and 4.04% respectively; while soil activity with mean values oscillate between 0.09 – 8.5 (1.1), 0.17 – 7.85 (0.9) and 0.2 – 5.08 (1.2) accordingly. The weighted PI (wPI) value is between 0.25 – 31.9% (5.1%), 0.92 – 25.3% (5.8%), and 0.87 – 7.3% (3.5%) with mean from the three units. In PCB soils, activity tends to be higher than normal despite the high gravel content (61.4%) and low clay (0.9%), higher wPI, PI_r, and swelling potential (SP) indicating that the soils are active. These observations correspond with Bayamack et al. (2019). The derived plasticity parameters represent the effective contribution of plasticity of fines to the performance of the soil materials.

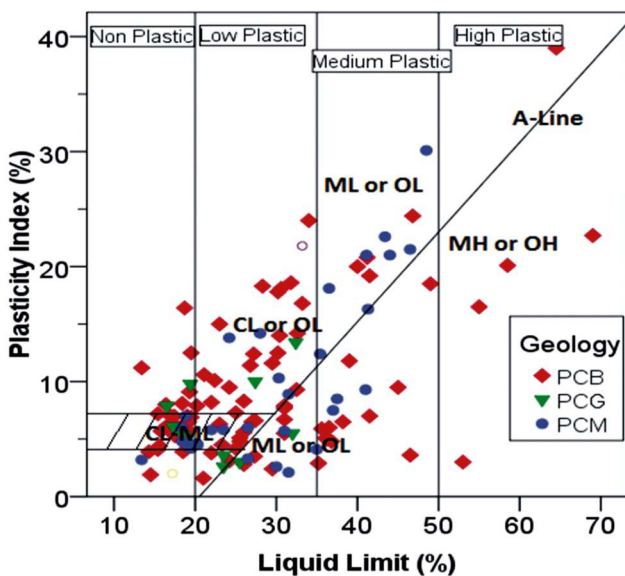


Fig. 2: Casagrande chart of plasticity – liquid limit relationship.

Compaction and California Bearing Ratio

The MDD (2.6mg/m³) of soils from PCB area (Table 1) increases with mean (1.77mg/m³) at OMC (25%) and mean (13%). These values are higher than the PCM and PCG units with 2.1 mg/m³ (1.6mg/m³) MDD and 22.5% (15%)OMC values. The low density - moisture relationship implies low strength instigated by loose soils that are susceptible to erosion. The interaction of the subgrade with water greatly reduces strength and therefore promotes continuous failure of the overlying pavement. Few examples of soil compaction curves shown in Fig.3 illustrate distinct peak of MDD at OMC. The CBR values at 95% OMC after 48hrs immersion are between 10 – 56.4% (PCB), 11 – 45% (PCM) and 12.1 – 37.2% (PCG) soils (Table1). Their mean values also differ between 28.8%, 23.6% and 27.8% respectively.

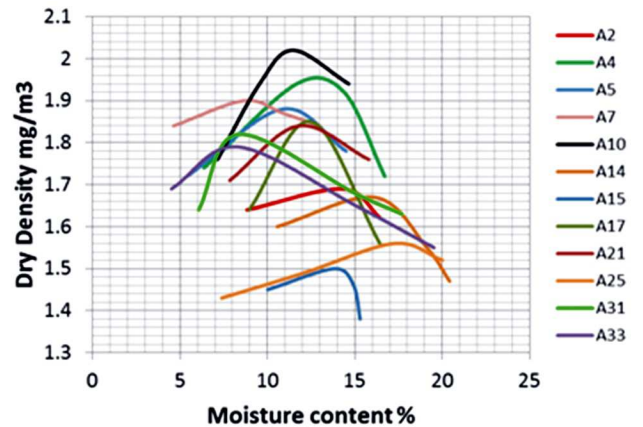


Fig. 3: Compaction curves of selected soil samples

For unsoaked condition, varying CBR occurs between 12.5 – 75.0%. This result shows reduction in strength due to soaking suggesting a probable drastic strength reduction by 50% during wet condition as well as the penetration resistance owing to excessive moisture. These values are similar to those found by Adams and Adetoro (2014) along AdoEkiti – Akure road (27 - 100%). The low mean CBR value (<30%) demonstrates the incapability of the soils to withstand ground vibrations when vehicular load is applied and reinforced its susceptibility to erosion. Soil improvement measures are therefore, envisioned to provide adequate strength and stability for pavement structure.

Principal Component and Hierarchical Analyses of the Components

Seven principal components (PCs) are extracted with eigenvalues >1 accounting for 83.8% of the total

variance of data (Table 2). The first five PCs accounted for >70% of variability with PC1, PC2, and PC3 explaining 33.4%, 14.7%, and 11.4% of the total variance. Major contributing variables for the components are fines ($r = 0.87$), PI ($r = 0.70$) and coarse sand ($r = 0.67$) (Table 2). Based on the communality estimates, the five factors explained more than 70% of the variance in 23 variables and less in 4 variables including CBRs. The values obtained are similar to those obtained by Shukla et al. (2005). Since the factors fairly explained the variance in soaked CBR, it requires a regression technique to predict the property. The parameters are well represented on the correlation circle graph (Fig. 4) indicating they are well explained by the factorial axes. In the graph, three groups of geotechnical parameters exist suggesting the existence of correlation between the variables.

Table 2: Eigen values, proportion of variance and communality estimates of soil variables

Soil variables	PC1	PC2	PC3	PC4	PC5	Communalities
Eigenvalue	9.0	3.9	3.1	2.5	1.6	*
% variance	33.4	14.7	11.4	9.2	5.8	*
Cum %	33.4	48.1	59.5	68.7	74.5	*
SP	0.93	-	-	-	-	0.91
PI	0.87	-	-	-	-0.39	0.95
LL	0.87	-	-	-	0.33	0.94
Fsw	0.86	-	-	-	-	0.85
wPI	0.84	-	-	0.34	-	0.89
CBRu	-0.60	-	-	-	-0.32	0.74
MDD	-	0.96	-	-	-	0.97
DD	-	0.94	-	-	-	0.94
BD	-	0.93	-	-	-	0.91
OMC	0.35	-0.60	-	-	-	0.68
MC	0.43	-0.56	0.32	-	-	0.76
LLr	-	-	-0.78	-	0.39	0.82
Ac	-	-	-0.75	-	-	0.68
Clay	-	-	0.72	0.39	-	0.85
NMC	0.48	-	0.73	-	-	0.80
Silt	-	-	0.67	0.42	-	0.86
Fines	0.32	-	0.61	0.43	0.36	0.96
FS	0.36	-	0.53	0.40	0.42	0.91
Gravel	-	-	-	0.94	-	0.94
CS	-	-	-	0.94	-	0.95
MS	-	-	-	0.78	-	0.79
Sand	-	-	-0.41	0.47	-0.47	0.85
PIr	-	-	-	-	-0.77	0.68
PL	0.43	-	-	-	0.72	0.87
Wr	-	-	-	-	0.89	0.82
Dr	-	-	-	-	0.83	0.74
CBRs	-0.39	0.37	-	-	0.48	0.53

PC1 positively correlates (> 0.84) with SP, PI, LL, Fsw, wPI, and PL, and is termed plasticity parameters. PC2 demonstrates very high correlation with soil densities (MDD, DD, BD) (> 0.90) but negatively with moisture contents (MC, OMC) (0.6) and is termed moisture-density parameters since the variables are important functions of soil moisture density. In contrary, shows

moderate loading in CBRs (0.37) owing to significant correlation of MDD and OMC. Similarly, PC3 and PC4 defined as gradation parameters show positive correlation with clay, silt, fines FS, NMC, gravel, CS, MS and sand (>0.5); and negatively correlated (0.75) with Ac and LLr. These variables are a function of gradation characteristics. PC5 are positively correlated with PL, Wrand Dr (>0.7) but inversely with PIr (0.77) and is referred as soil texture parameter. These groups are confirmed by the hierarchical classification, which reclassifies the parameters into three parts by dendrogram.

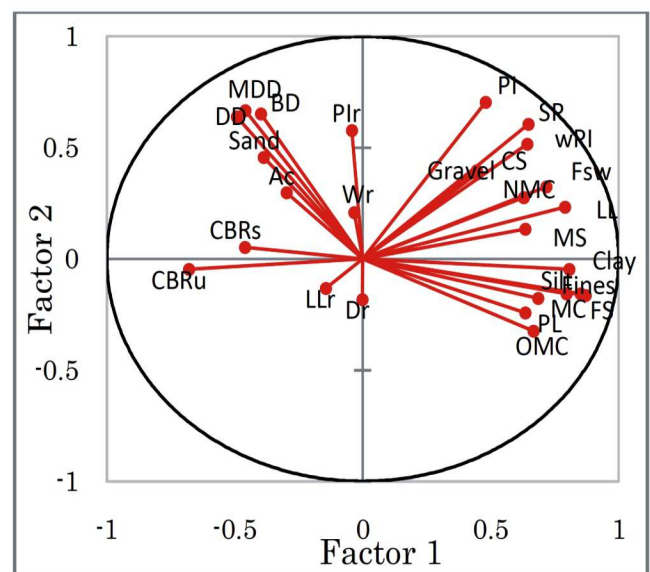


Fig. 4: Correlation circle of PCA

Fines (silt, clay), plasticity, MC and OMC characterize class 1 soils of approximately 47% migmatite-gneiss derived origin (PCB) with a p-value < 0.001 . While class 3 shows significant clustering with density (BD, DD, MDD) of 44% meta-sediment origin (PCM). Class 2 soils performed poorly (9%) with high p-value > 0.05 of PCG origin.

Regression Analysis: Prediction of CBR Value

Since the CBRs parameter is not poorly correlated with any parameter of the components, though it is an essential parameter in road design, a study of its regression through stepwise method results in many models. 70% training dataset accounted for 33% variance with coefficient of determination ($R^2 = 0.33$) and root mean square error of performance (RMSE) of 7.8. Given the p-value < 0.001 of the analysis of variance (ANOVA), the significance level (5%) and the low bias (0.05), the prediction by the explanatory variables is significant.

$$CBRs = 0.31Fines + 1.88Ac + 0.41Fsw - 0.298LL - 0.25PL - 0.73wPI - 0.5OMC + 2.11MDD + 36.03$$

Conclusion

The variability of soil geotechnical parameters used in road construction yielded the following conclusions:

1. The grading characteristics of the soils vary greatly with fine sands dominating the composition.
2. The five PCs accounted for 74.5% of the variability with the first three explaining 59.5% of the total variance.
3. The migmatite – gneiss derived soils are characterised by fines, plasticity, MC, OMC while

meta-sediments are characterised by density parameters.

4. Fines, Ac, Fsw, LL, PL, wPI, OMC, and MDD can explain the CBR parameter.
5. The pavement challenges witnessed on the highway are attributable to the poor soil suitability, the influence of geology and lack of drainage.

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Assessment of Geotechnical Properties of Soils Around Yelwan Tudu and Environs in Bauchi Area, Northeastern Nigeria

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Abstract

The cracking of buildings has been reported in parts of Bauchi area and this phenomenon is usually caused by shrinkage and swelling of soils due to change in moisture content and occurrence of expansive clays in the soils. While most of the existing literatures have focused more on mineralogical properties of the clays in addressing this issue, geotechnical characteristics of the soils which are equally effective in investigating and resolving engineering problems have not received much attention. This study therefore, examines the geotechnical properties of soil supporting buildings and pavement structures in Yelwan Tudu and environs, NE, Nigeria. Nine (9) disturbed representative soil samples were collected at a depth of 1 meter and subjected to geotechnical tests comprising of granulometric analysis, Atterberg limits, compaction and California bearing ratio (CBR), in accordance with the specification of the British Standard Institution. Shrinkage limits of the soils were also estimated from the Casagrande chart. The results of the tests revealed that the area is dominated by uniformly sorted to well sorted clayey sands (SC) and silty sand (SM) with uniformity coefficient between 4.5-20.8%, coefficient of curvature between 0.1-1.0%, liquid limit and plastic limit range of 28.8% to 41.6% and 13.9% to 30.9% respectively. The plasticity index however, is in the order of 1.9-22.6% with an average of 22.8%, while the estimated shrinkage limits ranges between 11.5% to 30.2% (average of 19.0%), suggesting slightly plastic to highly plastic soils with low to moderate degree of expansion. The Maximum Dry Densities (MDD) ranged from 1.70 g/cm³ to 2.50 g/cm³ with a mean value of 1.9 g/cm³ while the Optimum Moisture Contents (OMC) varied from 13.0% to 30.8% with an average value of 19.3%. The CBR range from 5.5% to 92.7% and averages 25.9%, suggestive of fair to very good subgrades. This study shows that the investigated soils generally exhibits moderate to low shrinkage properties and fair to very good subgrade characteristics but poor as sub base materials. This study will serve as base-line information for future foundation design and pavement construction in Bauchi area.

Keywords: Shrinkage, Swelling, Geotechnical test, Plasticity index, California bearing ratio

Introduction

In engineering terms, soil is referred to as regolith's which are derived from weathering of bedrocks. Soils formed under special climates in the tropical settings can be described as lateritic when it is high in hydrated iron oxides or bauxite when it comprises of essentially aluminum oxides. The mineralogy of the soils governs their engineering properties and in most practical applications of soil the fundamental consideration and principles lies in the assessment of the strength and compressibility characteristics of the material (Amadi et al., 2012). Therefore, a good understanding of geotechnical capabilities of soils becomes consequential in addressing geoengineering problems like swelling soil which are mostly responsible for the significant structural damages to houses, pavement foundations and other civil engineering endeavors (Oke and Amadi, 2008).

The problems of expansive clays is normally associated with changes in moisture occasioned by variation in precipitation and hence, water level in most arid and semi-arid regions, leading to shrinkage and swelling with attendant consequences (Christodoulis, 2015). The adverse effects of shrinking or swelling soils

include heaving of buildings, distortion of embankments designed with swelling clays and manifestation of shrinkage cracks on road pavements. In pavement design and construction, the efficacy and durability of the road layers (base course or sub-base course) majorly lies in its ability to transmit the axle-wheel load to the foundation materials (subgrade). The subgrade serve as foundation which supports the entire pavement system. As a result, the performance of the pavement during its design life will depend on the engineering properties of the subgrade. In view of the above therefore, detailed investigation of the subgrade material becomes desirable in order to ascertain the uniformity of support for the pavement slab and realization of the required subgrade shrinkage limit, stiffness and strength (Nwankwoala and Amadi, 2013).

Based on the above premise, there is no gain saying in the fact that the stability of all civil engineering structures such as pavements, building and tunnels, etc, relies on adequate assessment of engineering properties of soils to guarantee the safety and stability of the structure (Cosenza et al. 2006). Different methods have been deployed by different authors to determine the geotechnical characteristics of soil in different parts of the world, these include mineralogical (Orazulike 1986

& 1992; Yılmaz and Karacan, 1996; Christodoulis, 2015), geotechnical techniques (Bello, 2007; Bello et al., 2007; Aghamelu and Okogbue, 2011; Nwankwoala and Amadi, 2013), integrated geophysical and geotechnical approach (Cosenza et al. 2006; Una et al. 2015).

The widespread development of cracks which eventually damage buildings have been reported in parts of Bauchi municipal. These cracks cut through structures, such as buildings, causing the foundations to be distorted. The abovementioned researches were based on chemical data which revealed that the soils in parts of Bauchi area are high in expansive clays like illite and montmorillonite which cause the soils to shrink and swell intermittently with seasonal change in the water table. This study therefore, examines the geotechnical properties of soil supporting buildings and pavement structures in a part of Yelwan Tudu and environs, Bauchi, northeastern Nigeria.

Description of Study Area

The study area is Yelwan Tudu and environs. It is a residential as well as a commercial area and constitute the host community of the Abubakar Tafawa Balewa University and other higher institutions in Bauchi area. The study area extends from longitude $09^{\circ}45'00''\text{E}$ to $09^{\circ}48'13''\text{E}$ to latitude $10^{\circ}15'00''\text{N}$ to $10^{\circ}18'13''\text{N}$ (Fig. 1). The area is characterized by an undulating topography with elevation ranging from 594-618m above sea level and drained by two main river systems which flows from Bauchi – Miri and another from Miri to the north of Miri town, all forming a characteristic dendritic drainage pattern with their numerous tributaries. The vegetation is typically Sudan Savannah type with an annual rainfall of about 700 mm in the north and 1300 mm in the southern part. To distinct seasons characterize the area: the rainy season which runs from May to September, and the dry season from October to April (Bauchi State Atlas, 1983).

The study area is underlain by high grade metamorphic rocks consisting mainly of migmatite and gneisses, which were intruded by granitic material during the preproterozoic episode (Ferre and Caby, 2006). The preproterozoic event was also associated with charnockitization of meta sedimentary units leading to the formation of charnockites which underlie most part of the investigated area. The geological structures in the area comprises approximately NW-SE and NE-SW trending lineaments as revealed by the geological field mapping and spot 5 image analysis.

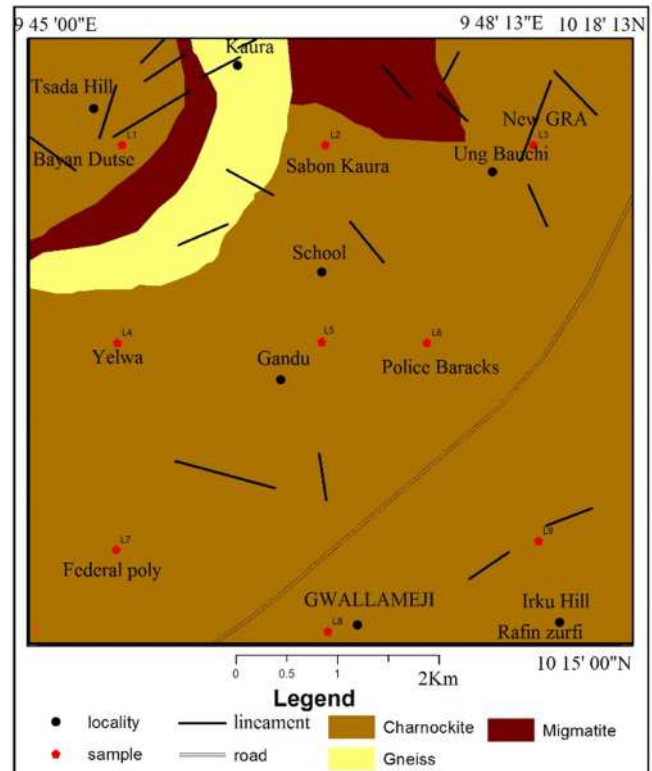


Fig. 1: Geology and sample location map of studied area

Materials and Methods

The sub-soil conditions was investigated from nine trial pits which were manually dug at designated points and samples were collected. The collection of samples was undertaken in line with the specification of British Standard Institution (BSI) 5930 (1981). These samples were collected at a depth not less than 1m for visual inspection. The laboratory geotechnical tests carried out on the soil samples included grain size distribution, Atterberg limits, compaction and California bearing ratio. The sample preparations and testing was undertaken in the civil engineering laboratory of the Abubakar Tafawa Balewa University, Bauchi, and in accordance with standard methods of testing soil for civil engineering (BS 1377: Part 1-9, 1990). For the sieve analysis, approximately 150 g of each sample was used using the BS 200 sieve and the fraction retained on the sieve was air dried and passed through a set of standard sieves. The liquid limit and plastic limit tests were carried out on air-dried samples that passed 0.425 mm (BSI No 36) sieve; both tests then followed standard procedures specified by BSI 1377-2 (1990). Shrinkage limits of the soils were estimated from the Casagrande chart. The compaction tests were limited soil particles that passed through 20 mm BSI sieve and followed procedure specified by BSI 1377-4 (1990). The unsoaked CBR was deployed on compacted soil

samples with load values recorded at penetration of 0.2-7.5m.

Results and Discussion

The result of the laboratory analyses and the unified soil classification system (USCS) notation are presented in Table 1 while the Casagrande's plasticity chart derived from the studied samples are shown in Fig. 2. The detailed analysis of the engineering properties of the soils from the investigated area are elucidated in the following subsections.

Particle Size Distribution

As can be observed from table 1, the percentage of soil passing the No. 200 (0.075 mm) sieve ranges from 3.2 to

34% with an average of 17.8%, indicative of granular materials based on the USCS ($\% < 500$) and the AASHTO ($\% < 35$) which also suggests that these materials are generally good as subgrade and sub base layers. Furthermore, from the grain size distribution curves the values of the coefficient of uniformity and coefficient of curvature ranges between 4.5-20 and 0.1-10 respectively, suggestive of uniformly sorted to well graded soils. The grading and sorting of a soil material gives an idea on the permeability characteristics of the material which have serious influence on the geotechnical properties of the material. Uniformly sorted soil would be permeable would improve drainage conditions and prevent water logging of the soil as compared to the well graded soils with lower permeability.

Table 1: Statistical summary of the soil properties

Samples	% passing	Cu	Cc	LL (%)	PL (%)	PI (%)	MDD (g/cm ³)	OMC (%)	CBR	S.L	USCS
L1	33.5	11.4	0.1	40.0	24.8	15.2	1.9	13.0	12.5	20.0	SC
L2	24.3	12.0	0.1	28.0	25.0	3.0	1.9	17.6	5.5	23.5	SM
L3	34.0	20.8	0.8	35.0	19.5	15.5	2.5	19.0	19.6	15.5	SC
L4	5.8	10.0	0.1	32.7	30.8	1.9	2.1	17.0	24.5	30.0	SM
L5	5.8	4.5	0.4	34.1	17.1	17.0	2.0	20.8	92.7	13.0	SC
L6	3.2	20.0	0.8	38.0	30.9	7.1	1.8	30.8	9.8	27.5	SC
L7	33.6	11.3	1.0	41.6	19.0	22.6	1.9	22.6	34.7	13.0	SM
L8	14.8	12.5	0.5	32.0	13.9	18.1	1.7	13.0	21.9	11.0	SC
L9	4.9	8.6	0.1	41.5	24.0	17.5	1.9	19.5	12.1	18.5	SC
Min	3.2	4.5	0.1	28.0	13.9	1.9	1.7	13.0	5.5	11.0	-
Max	34.0	20.8	1.0	41.6	30.9	22.6	2.5	30.8	92.7	30.0	-
Average	17.8	12.3	0.4	35.9	22.8	13.1	1.9	19.3	25.9	19.1	-

Atterberg Limits

As shown in the table 1, liquid limit (LL) and the plastic limit (PL) ranges from 28-42 (ave. =36) and 14-31(ave. =23) respectively, while the plasticity index (P.I) ranges between 1.9-22.6 (ave. =13). The implication of the atterberg limits on a soil material lies in the fact that the water contents at which the consistency changes from one state to another are found to differ from one clay to another, depending upon the amount and type of clay minerals present. From PI and LL results and as revealed plasticity chart 6 of the samples belong to the CL group (inorganic clays of medium plasticity) while the remaining 3 samples fall within the ML zone (inorganic silt of medium compressibility) of the chart. Based on the grain size data and the atterberg limit results, samples 1, 3,5,7,8 and 9 are clayey sand (SC) and according to the USCS this group is best adapted for embankment construction; the soils are impervious and fairly stable as foundation. Samples 2, 4 and 6 however,

belong to silty sand (SM) category and may not have good compaction characteristics. Furthermore, Federal Ministry of Works and Housing (1997) recommend liquid limits of 50% maximum for sub base and base materials, making all the studied samples suitable as pavement materials. Sowers and Sowers (1970) opined that PI values > 31 suggests hazardous soils with high content of expansive clays.

Compaction and Strength Characteristics

The result of MDD and OMC (Table 1) ranges from 1.7-2.5 g/cm³ and 13-30.8% respectively. All the studied soil samples with the exception of L9 (Rafin zurfi) revealed MDD values slightly greater than the maximum threshold of 1.7 g/cm³ recommended by the Federal ministry of works and housing guidelines (FMWH, 1997). Consequently, the shear strength of the compacted soils would be below the required minimum, hence, compaction must be closely monitored in the

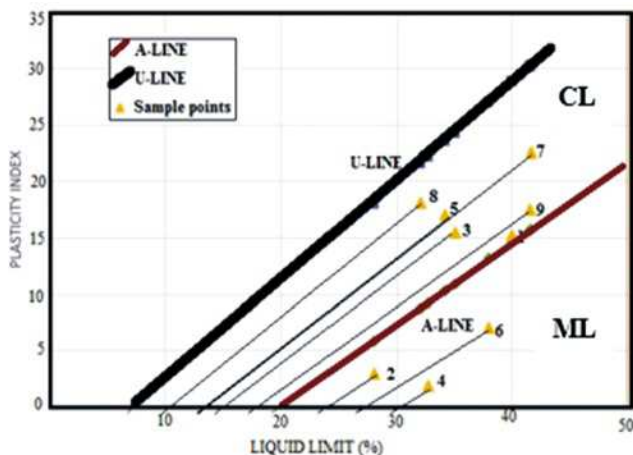


Fig. 2: Casagrade Plasticity Chart showing the Compaction and strength characteristics

field to secure the desired strength. Conversely OMC values provides and the indication of water demand for achieving the desired density in the field (Roy, 2013). As can be seen from table 1, samples L1, L4 and L8 have the lowest OMC values and would require less water to attain the desired density and maximum shear strength.

The CBR test is a measure of the strength of a soil sample in order to determine its suitability as subgrade, sub-base or base course material in pavement system design The CBR of the studied soils ranges between 5.5

in location 2 (Sabon Kaura) to 93 in location 5 (Gandu) and based on FMWH, 2010, samples L1, L3 and L9 are generally fair – good (CBR > 10) while locations 4, 5 & 8 are very good as subgrades (CBR > 20). Samples L2 and L6 however, indicated a poor- fair subgrade materials (CBR < 10). As sub-bases however, the soils are mostly poor, barring few locations (Yelwan Tudu, Birshin gandu and Federal Polytechnic) which revealed excellent- fair ratings.

Summary and Conclusion

The geotechnical assessment of Yelwan Tudu and environs revealed that the soils are uniformly sorted to well-graded clayey and silty sands which are generally good as subgrades and poor as sub- base materials. In terms of their suitability as foundation materials for building however, most of the samples are void of expansive clays except for samples from Gandu, Federal polytechnic and Gwalameji areas which show some tendency of shrinking. The unfavorable soils would generally require improvement by stabilization with cement, mixing with rock aggregates in order to improve their sub-base requirements and adoption of proper construction codes to help ameliorate the problem of cracking buildings and failure of pavement systems in the affected areas.

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Spatial Distribution of Foundation Subsoil Parameters of a Distressing Storey-Building, FUT, Akure, Southwestern Nigeria

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Abstract

Spatial distribution and accurate mapping of soil properties are essential components in understanding geotechnical data from site investigation. To determine and analyze spatial variability data of the foundation subsoil of a distressing storey-building in FUTA, Akure, a combination of conventional, analytical, and geostatistical methods, were employed. A total of 34 samples from 12 trial pits were collected, and soil properties of natural moisture contents (NMC), specific gravity, grain size analysis, Atterberg limits, and compaction were determined using the standard analytical methods. The use of the Inverse Distance Weighting (IDW) interpolation technique was the best for plotting direct visualization of the soil properties (spatial diagrams). Using the overlay technique to superimpose the spatial graphs for NMC, %Fines, PI, and MDD that presented very close similarities, produced a general spatial diagram that classified the area into four (A, B, C, and D) with three strength classes. Two parts (A and D) are of high strength, while B and C are of medium and low intensity, respectively. The regions with low/medium strength occupy more than eighty percent of the studied area. The crack lines are either very close to or fall on the three different strength classes' interfaces. Generally, the subsoil possesses parameters that are unsuitable for foundation purposes. These characteristics might have activated differential settlement, which could be responsible for foundation failure, manifested as cracks on the walls of the building.

Keywords: Spatial, MDD, foundation, subsoil, differential-settlement, IDW.

Introduction

The statistics of failures of buildings throughout the nation have increased in recent times. According to Ademeso et al. (2016), structural failures might be often-times associated with the problems of poor quality of building materials, old age of buildings, the existence of undetected geological structures like fractures, faults, joints, and cavities, the presence of ancient stream channels, shear zones, the position of soil in the profile, degree of weathering and heterogeneity in the near-surface geological sequence as well as unstable foundation. The building in question has shown signs of distress in three main parts displaying cracks that are as wide as between 2 and 7mm (Figure 1). Burland et al. (1977) classification place the cracks between very slight through slight to moderate. In assessing the causes of cracks in the building, it is necessary to determine its' location, pattern, width, length, depth, age, and whether it is active or not, how catastrophic it is, and how to repair it (Supernant and Basham, 1993). The identification of structural cracks on the building makes it mandatory to assess the competence/integrity of the subsoil to continue to bear the load. This paper uses GIS (ArcGIS 10.1's Geostatistical Analyst) to capture, store, integrate, analyze, and display acquired data (which are spatially referenced) in a format that will be useful to planners, architects, and engineers. easily creates a continuous surface or map,

The Study Area

The study area is one of the buildings in the School of Earth and Mineral Science (SEMS) of The Federal University of Technology (FUT), Akure, which falls within the Akure Sheet 264 (Figure 2). FUT, Akure lies between latitude $07^{\circ} 17' 44.6''N$, and longitude $05^{\circ} 08' 6.14''E$, (i.e. 735721.102mE and 806924.66mN) and latitudes $7^{\circ} 17' 59.6''N$, and longitude $05^{\circ} 08' 16.7''E$, (i.e. 736044.983mE and 807388.620mN) covering an area of about 14.977 hectares (0.150 km²). The elevation varies between 372m and 386m above the mean sea level.

Geology of the Study Area

Generally, the Precambrian Basement Complex rocks of South-Western Nigeria underlie the Akure area [Rahaman, (1976; 1988). Olarewaju (2006) and Ademeso (2010)] reported variously on the Precambrian rocks of Akure. The lithologic units of Akure include migmatite-gneiss-quartzite complex, granitic rocks, and the charnockites. The migmatite-gneiss, quartzite, charnockites, the hybrid, and the porphyritic granite underlie the FUTA area, while the migmatite-gneiss underlie the studied site (Figure 3).

Methodology

The systematically sampled soils were stored in

polythene bags and labeled as S1A, S1B, S1C for point No.1 up to S12A, S12B, and S12C for point No.12. The GPS (X, Y, Z coordinates) of each of the points was determined and recorded. The testing of the natural moisture content (NMC), specific gravity, optimum moisture content (OMC), maximum dry density (MDD), and Atterberg limits followed the standard methods. ArcGIS 10.1 was employed in all statistical analyses and spatial diagram construction. All attributes data are stored in the ArcGIS 10.1 environment database through the ArcCatalog window of the software. The research plotted the parameters' spatial distributions, (but the presentation in this report was limited to the base layers' NMC, %fines, PI, and MDD) using the Inverse Distance Weight (IDW) of the Geostatistical Analyst Tools (a geoprocessing tools in ArcToolbox) of ArcMap environment of the software.

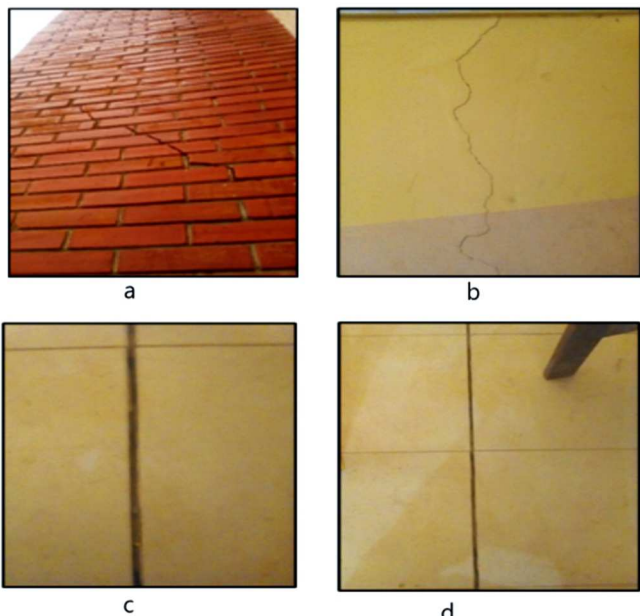


Fig. 1: Pictures of Noticeable Cracks on the walls (a & b) and floors (c & d) of the building.

Presentation and Analysis of Results

Morphological Description of the Weathering Profile

Based on textural and colour observations, the site investigation revealed a top layer, from about 0.2 to 0.5m depth, which is dark brown in colour and rich in organic remains. A layer of dark brown sand underlies this layer to a depth of about 1m. Layers of yellowish/reddish brown clayey sand (to about 1.5m depth), and the freshly weathered soil further underlie this to a depth of about 2m. Besides, the pits revealed profiles varying in number and thickness. Pit 4 had two

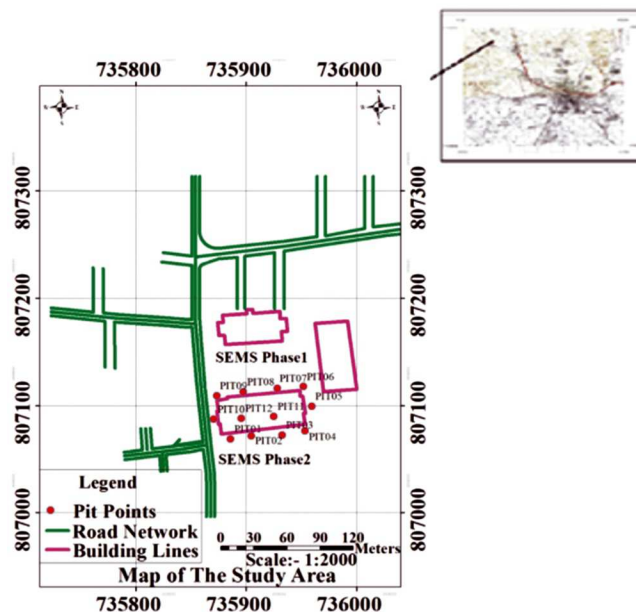


Fig. 2: Topographical Map of the Study Area (Topographical Map of Akure as inset).

horizons; three horizons were in holes 1, 2, 3, 5, 6, 8, 9, and 10; four were in pits 11 and 12, and five were in pit 7 (Figure 4).

Specific Gravity

The range and mean of the specific gravity's results for top layer are 2.54 - 2.65, and 2.59, respectively, for the middle layer, are 2.49 - 2.63, and 2.55, while those of base layer is 2.48 - 2.66, and 2.57. Wright (1986), as cited in Abimbola (2014), opined that the standard range of values of the specific gravity of soils lies between 2.4 - 2.8. The determined specific gravity values are, therefore, reasonable.

Grain Size Analysis

Table 1 reports the grain size analysis results of all the samples. The percentages of gravel, sand, and fines of the top layer range from 2.3 - 38.9%, 28.7 - 54.6%, 32.4 - 51.0% with an average of 16.13%, 38.83%, and 43.36% respectively; the middle layer range from 3.2 - 46.2%, 10.6 - 39.4%, 33.0 - 78.3%, with an average of 20.67%, 23.67%, and 55.65% respectively; while the base layer revealed percentages that range from 1.8 - 55.4%, 11.8 - 29.9%, 29.4 - 81.8% with an average of 23.79%, 17.84% and 58.40% respectively. The spatial distribution of the percentage of fines for base layers is in Figures 5 and 6.

Atterberg Limit Test

The results of the Atterberg limits tests for the top layer revealed that the liquid limit, plastic limit, plasticity index (PI), and linear shrinkage range from 26.28 - 54.06%, 17.17 - 35.71%, 2.60 - 32.06% and 6.9 - 12.5% with averages of 38.86%, 24.44%, 14.28%, and 9.78% respectively.

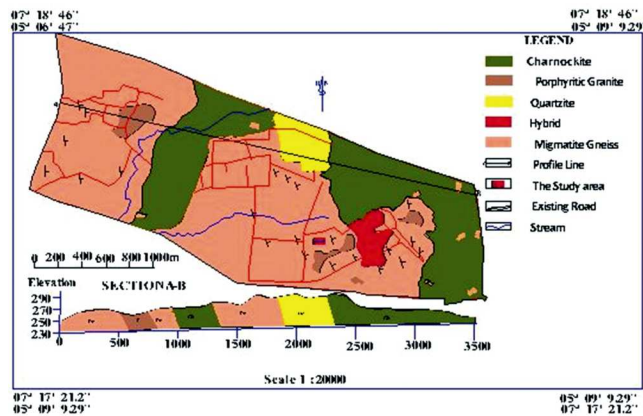


Fig. 3: Geological Map of FUTA (Adapted from Ademeso, 2016)

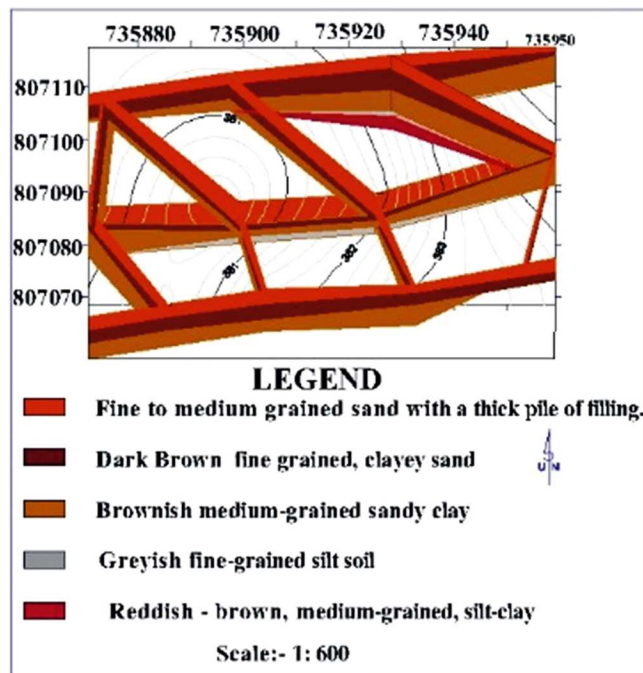


Fig. 4: 3D Soil Profile of Study Area

The middle layer presented a range of 37.17 - 70.72%, 10.57 - 38.21%, 7.52 - 41.58%, and 5.9 - 10.9% with averages of 50.56%, 26.23%, 25.51%, and 11.59% while the base layer revealed 42.49 - 61.39%, 17.10 - 52.76%, 8.64 - 37.5%, and averages of 54.56%, 29.42%, and 18.10% (Tables 2). The spatial distribution for base layers' plasticity index is in Figures 5 & 6.

Compaction Test

The results of proctor compaction tests are as presented in Table 2. The results indicate that for the top layer, MDD ranged from 1668 - 1855 Kg/m³ and OMC ranged from 15.30 - 19.40% with mean values of 1759.75Kg/m³, and 17.29% respectively. Middle layer values range and average are 1493 - 1952Kg/m³, and 12.4 - 24.9%, and 1719.25Kg/m³, and 18.44% respectively while the base layer presented a range, and the average of 1429 - 1738Kg/m³, and 17.1 - 25.3%, and 1570.63Kg/m³, and 21.75%, respectively. The spatial distribution of MDD for base layers are in Figures 5 & 6.

Discussion

The result of the NMC shows a general increase in moisture content from the top to the base layers with an average of 16.25%, 20.41%, and 22.26% for top, middle, and base layers. The values are high, indicating the potential of the soil for water retention. Consequently, high swelling potential, as seen in the spatial diagrams (Figures 5 and 6), where some parts of the studied area had very high NMC, some parts medium values, while some presented low values. The NMC of the base level presented a very high positive correlation with %fines ($r = +0.872006$), moderate positive correlation with liquid limit ($r = +0.517572$), and a high negative with MDD ($r = -0.721752$). The correlation coefficient of MDD/NMC and MDD/%fines revealed that as the value of %fines increases, that of NMC increases, while that of MDD decreases, and vice versa. These confirm that the NMC, and %fines generally combine to impact the MDD. According to Jegede (1997), as cited in Ademeso *et al.*, (2016), the excessive moisture content is detrimental to collapsible, and swelling soils. He opined that swelling soils are known to exhibit substantial volume changes with an increase or decrease in moisture content even under constant external load. All these could lead to differential settlements of the foundation subsoil, which might cause the failure of the foundation. The specific gravity results varied between 2.55 and 2.59. These values are suitable following Wright (1986), which stated that the standard range of values of the specific gravity of soils lies between 2.4 and 2.8. The upper range is higher than the standard specified by FMWH (1997) that suggested a specific gravity of 2.6 for sandy soil, and 2.35 to 2.7 for clay, and silt. The FMWH standard implies that the studied area consists of a high proportion of clay or silt, randomly distributed, as seen in the spatial maps (Figures 5, and 6). The specific

gravity shows a moderate negative correlation with %fines ($r = -0.521979$), and liquid limit ($r = -0.375975$). Inference shows that as %fines increases, specific gravity decreases, and vice versa. The grain size analysis results showed that %fines presented an average of 43.36%, 55.65%, and 58.4% for top, middle, and base layers, respectively, indicating a general increase in the percentage of clay/silt content and consequently high compressibility down the profile. It further shows that the %fines are generally high. According to FMWH (1997) and USCS standards, the samples' particle size gradations do not meet the general specification for sub-grade, sub-base, and base materials as the %fines are more than 35%. Only 17.65% of the samples in this location fall within the above specification.

The results revealed the soils to be clay of high compressibility (Table 1), and therefore, cannot serve as subsoil, sub-grade, sub-base, and base materials. Ademeso *et al.* (2016), and Akpokodje (1986) 's classification put 85% of the samples in this studied area as unsuitable for foundation purposes. The 2D spatial diagram of %fines (Figure 5) showed that a considerable part of the studied area possesses medium/high %fines that are averse to a good foundation. These further strengthened the opinion that differential settlement might have taken place in the studied area.

The soil samples' compaction characteristics showed poor to fair MDD and OMC with an average of 1683.21Kg/m^3 , and 19.16%, respectively. Generally, the average values of the MDD decrease from top to base layer while the OMC increases downward. The samples also revealed that the %fines increased downward. The MDD's relationship with %fines ($r = -0.5872081$), is negative, and moderate while it is negative and strong with NMC ($r = -0.721752$), as well as very strong, and negative with OMC ($r = -0.994570$). %Fines' relationship with OMC ($r = +0.586873$) is moderate, and positive. Following Wood's (1937) classification, 2.94% of the samples rated as "good," 17.65% as "fair," 52.94% as "poor," and 26.47% as "very poor" materials for foundation work. Madedor (1986) opined that such soils could only serve as foundation support, sub-base, and sub-grade materials after stabilization to the desired strength. The 3D spatial distribution (Figure 6) presented the visual highs and lows of the parameters.

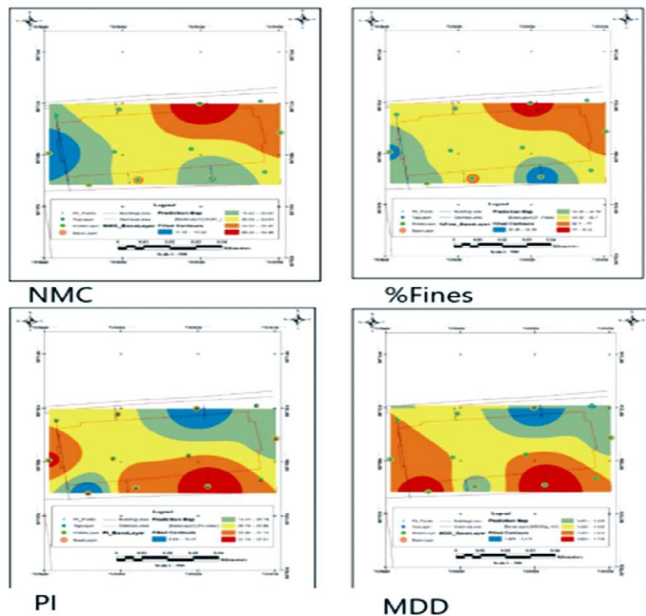


Fig. 5: Spatial Distribution (2D) of Some Base Layer Parameters

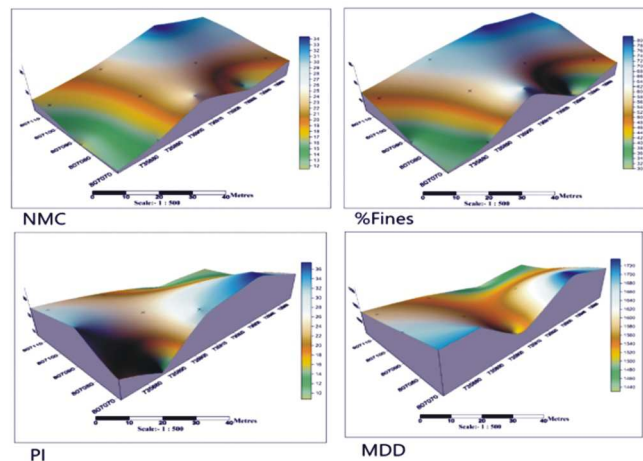


Fig. 6: Spatial Distribution (3D) of Some Base Layer Parameters

Considering the 3D maps of NMC, %fines, PI, and MDD indicated that a substantial part of the studied area exhibited low/medium strength. Based on the superimposition of the 2D spatial diagrams for NMC, %Fines, PI, and MDD, that presented very close similarities, a four-part classification of the area is revealed (A, B, C, and D) (Figure 7). Parts A and D were of high-strength, B is of medium-strength, and C is of low-strength. The area also revealed three strength characteristics depicted as high, medium, and low strength. The parts with low/medium strength occupy more than seventy percent of the studied area. The crack lines are either very close to or fall on the interfaces of the high/medium strength classes and in the medium or low strength areas implying that the strength of the subsoils might have engendered differential settlements with the consequence of cracks manifesting on the walls of the building.

Table 1: Statistical summary of the soil properties

S/No	Sample/No	Location		Datum (m)	Depth (m)	Ht. (m)	NMC (%)	Sp. Gr.	Grain Size		
		Eastings	Northings						Gravel	Sand	Fines
1	S1A	735885.72	807068.64	381.96	1.20	380.76	28.42	19.40	17.70	33.30	49.16
2	S1B	735885.72	807068.64	381.96	1.40	380.56	13.92	18.90	29.40	37.60	32.84
3	S1C	735885.72	807068.64	381.96	2.20	379.76	15.81	18.60	48.00	15.00	37.16
4	S2A	735904.42	807071.58	382.78	0.67	382.11	13.56	17.10	8.80	54.60	36.48
5	S2B	735904.42	807071.58	382.78	1.00	381.78	13.31	16.60	46.20	10.60	43.24
6	S2C	735904.42	807071.58	382.78	2.02	380.76	24.83	23.60	9.80	22.30	67.92
7	S3A	735932.59	807072.39	383.89	0.66	383.23	15.44	17.10	16.60	39.70	43.68
8	S3B	735932.59	807072.39	383.39	0.89	383.00	22.63	20.20	7.80	23.40	68.80
9	S3C	735932.59	807072.39	383.39	1.89	382.00	15.52	17.10	51.30	19.40	29.48
10	S4A	735953.11	807076.25	384.52	0.60	383.92	15.93	18.30	8.00	47.00	45.04
11	S4B	735953.11	807076.25	384.52	1.03	383.49	19.98	16.30	27.90	21.60	50.44
12	S5A	735959.11	807099.08	385.49	0.50	384.99	15.03	17.90	17.60	34.40	48.20
13	S5B	735959.11	807099.08	385.49	0.63	384.86	22.97	19.40	21.80	17.10	61.12
14	S5C	735959.11	807099.08	385.49	2.00	383.49	27.82	23.30	12.40	13.50	74.04
15	S6A	735951.50	807117.45	386.20	0.40	385.80	20.09	18.70	39.80	48.28	54.06
16	S6B	735951.50	807117.45	386.20	0.69	385.51	23.73	21.00	20.40	20.00	59.40
17	S6C	735951.50	807117.45	386.20	1.98	384.22	25.70	23.50	8.00	14.40	77.52
18	S7A	735928.13	807115.83	385.30	0.62	384.68	14.42	16.10	38.90	28.70	32.48
19	S7B	735928.13	807115.83	385.30	1.50	383.80	29.95	19.50	8.60	19.80	71.44
20	S7C	735928.13	807115.83	385.30	2.51	382.79	34.49	25.30	1.80	16.40	81.80
21	S8A	735897.26	807112.55	382.37	0.60	381.77	10.92	15.30	24.90	37.20	37.80
22	S8B	735897.26	807112.55	382.37	1.00	381.37	12.74	18.50	32.80	21.50	45.56
23	S8C	735897.26	807112.55	382.37	1.90	380.46	22.48	22.80	3.50	29.90	66.56
24	S9A	735873.36	807109.08	383.04	0.60	382.44	13.85	17.60	37.20	48.64	29.81
25	S9B	735873.36	807109.08	383.04	1.40	381.64	19.44	19.50	14.00	39.40	46.56
26	S10A	735870.47	807087.05	382.51	0.76	381.75	16.16	16.00	13.80	35.20	50.92
27	S10B	735870.47	807087.05	382.51	1.06	381.45	12.58	14.10	28.60	34.60	36.92
28	S10C	735870.47	807087.05	382.51	2.36	380.15	11.41	19.80	55.40	11.80	32.80
29	S11A	735924.92	807089.67	383.25	1.20	382.05	14.72	15.40	19.20	43.70	37.00
30	S11B	735924.92	807089.67	383.25	2.00	381.25	25.00	12.40	3.20	18.50	79.28
31	S12A	735895.51	807088.01	381.51	1.10	380.41	16.49	18.60	2.30	35.10	41.88
32	S12B	735895.51	807088.01	381.51	2.00	379.51	28.65	24.90	7.30	19.90	72.72

Conclusion

Observations are summarized as follows:

- as the value of %fines increase, that of NMC increase, while that of MDD decreases, and vice versa confirming that the NMC and %fines generally combine to impact the MDD;
- there is an excellent relationship between %fines and specific gravity;
- the generally high %fines makes a substantial part of the area covered by the subsoil less suitable for good foundation; and
- that MDD is typically low and revealed a negative relationship with OMC, which is inimical to a good foundation.

The generalized spatial map (Figure 7) classified the area into four parts (A, B, C, and D). Opposite subsoil underlies parts A and D, and medium subsoil underlies B. At the same time, weak clay underlies C. The classification also identified three strength types (high for parts A and D, medium for B, and low for C). Generally, the subsoil possesses parameters that are

less suitable for foundation purposes. These differences (soil type and strength type) might have activated differential settlement, which could be responsible for foundation failure manifesting as cracks on the walls of the building.

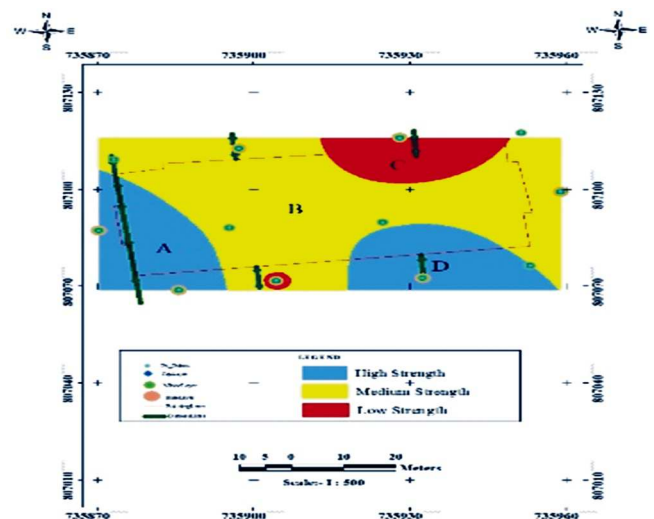


Fig. 7: Classification of the Study Area Based on the Super-Imposition of the Spatial Diagrams of NMC, %Fines, PI, and MDD.

Table 1: Statistical summary of the soil properties

S/No	Sample No	Consolidation				Atterberg Limits			
		OMC (%)	Kg/m ³	MDD	Rating	LL	PL	PI	LS
				Class					
1	S1A	2.52	1668	Poor	2	48.02	35.85	12.12	11.90
2	S1B	2.55	1724	Poor	2	53.09	45.50	7.59	6.70
3	S1C	2.62	1701	Poor	2	55.63	45.43	10.21	10.40
4	S2A	2.63	1762	Fair	3	39.36	17.95	18.70	6.80
5	S2B	2.62	1783	Fair	3	37.17	21.60	15.58	9.80
6	S2C	2.60	1499	V. Poor	1	55.08	23.78	31.30	14.30
7	S3A	2.58	1762	Fair	3	29.94	17.17	12.77	8.60
8	S3B	2.50	1635	Poor	2	48.20	19.09	29.12	13.40
9	S3C	2.66	1738	Poor	2	54.64	17.10	37.51	14.30
10	S4A	2.56	1713	Poor	2	49.94	17.90	32.06	9.00
11	S4B	2.63	1795	Fair	3	46.71	25.50	21.19	12.90
12	S5A	2.60	1751	Poor	2	32.41	22.50	9.93	10.70
13	S5B	2.56	1705	Poor	2	53.03	35.72	17.32	12.70
14	S5C	2.48	1512	V. Poor	1	55.63	44.01	11.62	12.10
15	S6A	2.62	1697	Poor	2	54.06	35.71	18.35	10.40
16	S6B	2.62	1602	Poor	2	49.57	38.21	11.37	12.10
17	S6C	2.50	1503	V. Poor	1	54.06	29.75	24.31	12.86
18	S7A	2.61	1804	Fair	3	32.31	28.19	4.50	8.60
19	S7B	2.44	1664	Poor	2	70.72	29.14	41.58	15.70
20	S7C	2.49	1429	V. Poor	1	61.39	52.76	8.64	10.70
21	S8A	2.61	1855	Fair	3	26.28	23.68	2.60	5.70
22	S8B	2.64	1728	Poor	2	43.74	10.57	33.18	10.40
23	S8C	2.58	1532	V. Poor	1	42.49	22.49	20.57	11.40
24	S9A	2.65	1742	Poor	2	29.81	27.04	2.77	4.30
25	S9B	2.50	1664	Poor	2	37.93	30.42	7.52	10.00
26	S10A	2.62	1808	Fair	3	31.88	23.95	7.94	6.40
27	S10B	2.49	1886	Fair	3	62.47	25.05	37.42	14.20
28	S10C	2.59	1651	Poor	2	57.54	24.90	32.64	8.60
29	S11A	2.55	1832	Fair	3	42.78	21.20	21.63	8.60
30	S11B	2.53	1952	Good	4	49.04	19.80	29.24	9.30
31	S12A	2.54	1723	Poor	2	49.37	22.20	27.11	6.80
32	S12B	2.55	1493	V. Poor	1	55.24	24.92	30.32	11.90

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Assessment of Porphyritic Granite-Derived Soils for Road Construction from Odeda, Southwestern Nigeria

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Abstract

Porphyritic granite-derived soil samples from Odeda, Ogun State were collected and evaluated for road pavement material characteristics. It is necessary to carry out not only detailed soil geotechnical investigation but also geological and geochemical analyses, since highway geotechnical properties of soils also depend on their mineralogical and geochemical characteristics. Soil is heterogeneous thus, every soil must be investigated for optimum use hence the need of this research. Thin section of the parent rock was made and studied under petrologic microscope. Chemical composition of the soil was determined with X-ray fluorescence technique, while X-ray diffractometry was used for mineralogical analysis. Particle size analysis, Proctor compaction, Atterberg limits, California Bearing Ratio (CBR) and permeability tests were employed in evaluating soil for road pavement. The mapped parent rock was porphyritic granite with the percentages of SiO₂, Al₂O₃ and Fe₂O₃ being 74.71, 11.87 and 5.4 respectively. Dominant clay minerals are kaolinite and illite with values of 54% and 10%. Amount of fines and plasticity index is 50-70% and 16.41-20.15 with CBR value ranging between 27.87% and 29.51%. This is a well-graded soil, classified as sandy silty-clay of Group A-5 to A-7 of American Association of State Highway and Transportation Officials Classification System (AASHTO). The soils had low to very low coefficient of permeability (9.64x10⁻⁸-1.07x10⁻⁷ m/sec). From the analysed parameters, the soil can be classified as fair to poor road pavement material.

Keywords: *Geotechnical properties, Petrology, Soil classification and Road construction.*

Introduction

Most Nigerian roads are laden with failed portions, apparently because some of them must have been constructed with extrapolated geotechnical information from other closely related sites. Observations have shown that few and widely spaced samples are usually utilised when geotechnical investigations are carried out on site according to Nigerian Building and Road Research Institute (NBRI, 2013). Thus, some important variations in geotechnical parameters concerning the suitability of soils as construction materials which depend largely on their geotechnical properties would have been omitted. Thus, every soil at a site must be investigated and classified on its own merit.

Most roads show a considerable amount of failure few years after construction which was attributed to poor selection of pavement materials, improper understanding of the geotechnical properties of the soil used as subgrade and sub-base materials for highway (Jegede, 2000). Nigerian roads are underlain by various rock types which are yet to be properly investigated.

Location and Accessibility

Fresh rock samples, undisturbed and disturbed soil samples were taken from Odeda village, along Abeokuta-Ibadan road precisely within Obasanjo quarry, on Latitude N07.25125° and Longitude

E003.53042° (Figure 1). The vegetation here is mainly fresh water swamp and mangrove forest depicted by tropical vegetation of the rain forest type with tall grasses, herbs and small trees (shrub). (Meteorological Station of the University of Ibadan and the International Institute of Tropical Agriculture (IITA)).

Geology of Odeda

Odeda – the study area is underlain by the Basement Complex rock of Precambrian to early Paleozoic age. The rocks here include gneiss, schist, quartz, quartzites, migmatite and amphibolite. These rocks exhibit various folds, faults and foliation trending mainly in the North-South or North-North-East-South-South-West (NNE-SSW) direction. At the sampling location in Obasanjo quarry, the rock here is mainly porphyritic granite. The sampling location can be link through tarred and untarred roads.

Method of Study

Thin section produced from fresh rock sample collected were studied under petrological microscope. Undisturbed soil samples were taken with the core cutter and disturbed samples collected with the help of a stainless scooper. Chemical composition of the soils was determined with X-ray fluorescence technique, while X-ray Diffraction was used for mineralogical analysis (determination of clay

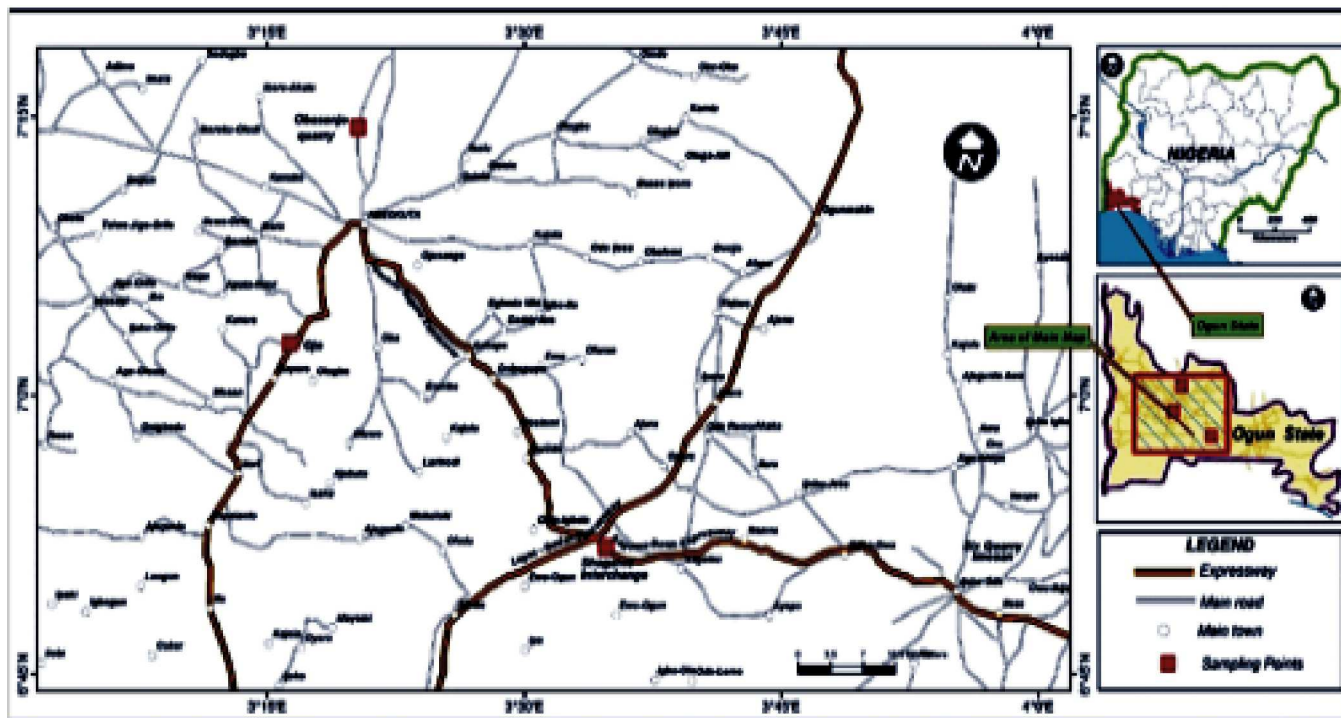


Fig. 1: Location map of the study area (Adapted from Ministry of Works and Housing, 1987)

mineralogy and major oxide geochemistry of soil samples were executed). Geotechnical tests used for evaluation of soils for road construction were also conducted in accordance with British standard BS- 5930 (1981) and ASTM D-402-90, (ASTM, 1985) with some modification brought about by peculiar characteristics of tropical soils. Particle size analysis, Proctor compaction, Atterberg limits, California Bearing Ratio (CBR) and permeability tests were carried out.

Results and Discussion

The diffractometer result indicates the presence of minerals such as quartz, kaolinite, illite/muscovite, K-feldspar and goethite in various proportions with kaolinite as the dominant clay mineral about 54% (Table 1). Kaolinite is a stable clay mineral with tight inexpandible structure which resist the introduction of water into the lattices.

Major Oxides values from X-Ray Fluorescence shows Silica, Aluminium (Al₂O₃) and iron oxides (Fe₂O₃) component of the soil ranging between 49.34 - 52.92%, 20.54 - 23.75%, and 31 - 13.21% respectively with the concentrations of sodium (Na₂O), calcium (CaO) and magnesium (MgO) oxides less than 1% as shown in Table 2.

Values of index and engineering properties of

porphyritic granite derived soil is presented in Table 3. Specific gravity of the grains ranges between 2.55 and 2.65, Natural moisture content (NMC) ranges from 10.92% to 15.06%. The soil has a well graded grain size with the % of fines between 40 to 62% which is more than % of the coarse fractions within the soil thus will have the overriding effect on the geotechnical property of the soil. The soil samples coded SHB has liquid limit (LL) ranging between 35.25 and 46.14% which is an indication that the soil is of moderate (intermediate) compressibility. The plastic limit (PL) ranges between 16.66 and 28.94% and Plasticity index value ranges from 16.05 to 19.35%. This soil is semi-plastic and unstable according their consistency and liquidity indices of 1.13-1.44 and -0.922- (-0.139) respectively. Low to high compressibility with value as 0.094m²/MN -1.01m²/MN was obtained). Bulk Density is high with value of 1893Kg/m³ – 2007Kg/m³ corresponding to bulk density for sand and gravel. Compaction test revealed MDD value range of 1770.77 Kg/m³ - 1872Kg/m³ and OMC of 16.42% -19.20%.

CBR value of porphyritic granite derived soils ranges from 25.49 to 59.51% for unsoaked samples and 9.70 to 16.70% for soaked samples showing a reduction in strength of 45.22% - 76.84% after soaking. Permeability (k) is very low and ranged between 8.05x10⁻⁵cm/sec - 1.01x10⁻⁴cm/sec from falling head test which correspond to that for silty to dirty sand grain.

Table 1: Result of quantitative phase analysis of the studied soil (wt%)

MINERAL	FORMULA	ODEDA (n=3)AV.
Quartz	SiO ₂	18
Kaolinite	Al ₂ Si ₂ O ₅ (OH) ₄	54
Anatase	TiO ₂	1
Illite/ Muscovite 2M1	K _{0.65} Al _{2.0} Al _{0.65} Si _{3.35} O ₁₀ (OH) ₂ / KAl ₂ AlSi ₃ O ₁₀ (OH) ₂	10.5
K-feldspar (Orthoclase, Microcline)	KAlSi ₃ O ₈	7
Hematite	α-Fe ₂ O ₃	2
Goethite	α-Fe ³⁺ O(OH)	7
Total		100

Table 2: Summary Results of Major oxides of the studied soils

Major oxides	Range (n=3)	Av.
SiO ₂	49.34 - 52.92	50.9
Al ₂ O ₃	20.54 - 23.75	21.74
Fe ₂ O ₃	10.31 - 13.21	11.54
CaO	0.14 - 0.29	0.22
MgO	0.27 - 0.32	0.29
Na ₂ O	0.06 - 0.09	0.07
K ₂ O	1.39 - 1.49	1.43
MnO	0.03 - 0.07	0.05
TiO ₂	1.03 - 1.51	1.32
P ₂ O ₅	0.08 - 0.13	0.11

Geotechnical Evaluation of the Studied Soil

High bulk density, CBR and MDD with low OMC means soil of high strength which is good for engineering construction while opposite of these implies soil of low geotechnical qualities. (Shodeko, 2004). High value of MDD and low OMC are typical of good sub-base soil. This soil shows high MDD value and high OMC meaning the soil is presumed a good sub-base relying on MDD value but fails to meet the criteria stipulated for OMC. There is a high reduction in California Bearing Ratio of 42.-77% due to soaking with water ingress of 12-24%. This means that little variation in water content will have great detrimental reduction on the strength of the soil, this kind of soil will be susceptible to soaking. Federal Ministry of Works

Table 3: Values of the geotechnical parameters of porphyritic granite derived soil

code	Gs	% C	% S	% F	% Si	% S	% G	LL (%)	PL (%)	PI	BD Kg/m3	MDD Kg/m3	OMC %	CBR (%)
SHB1	2.55	1.9	43	62	33	5	41.02	21.00	19.10	1904	1790	18.60	59.51	
SHB 3	2.60	15	40	55	30	15	36.01	16.66	19.35	1981	1830	18.81	57.68	
SHB 4	2.60	15	40	55	31	14	35.25	18.99	16.26	2007	1825	17.6	49.07	
SHB 9	2.60	20	35	55	36	9	42.12	23.67	19.05	2007	1871	16.42	36.52	
SHB 11	2.65	18	37	55	30	15	41.00	24.95	16.05	1968	1810	17.2	27.89	
SHB 12	2.60	15	40	55	30	15	41.00	24.94	18.16	1953	1831	16.81	30.25	
SHB 13	2.65	10	30	40	32	28	39.50	22.73	16.77	1893	1850	17.00	56.97	
SHB 14	2.65	10	30	40	34	26	36.00	19.67	16.33	1997	1870	16.80	31.94	
SHB 15	2.60	13	37	50	31	19	46.14	28.94	17.20	1936	1770	19.20	25.49	
SHB 16	2.60	12	38	50	32	18	45.01	26.89	18.12	1952	1812	16.42	30.66	

Gs Gravity of grain; C, Clay; F, fines; Si, Silt; S, Sand; G, Gravel; LL, Liquid Limit; PL, Plastic Limit; PI, Plasticity Index, BD, Bulk Density; MDD, Maximum Dry Density; OMC, Optimum Moisture Content; CBR, California Bearing Ratio

and Housing (FMWH) 1997 in Nigeria recommended minimum CBR value of 30% for sub-base and minimum CBR value of 80% for base material for soil to be use in road construction. From the CBR value

obtained, it was observed that all the CBR value of the unsoaked and soaked values obtained are below the 80% recommended for base materials by FMWH(1997). This soil using CBR value can be used as

subbase material except sample SHB11 and 15 (Table 1) but none as base material for road construction. Jegede (2000) stated that reduction in strength estimated from CBR due to soaking of soils must be small, that any reduction more than 20% in CBR, the soil is not good for road construction. In the studied soil, reduction in

strength (CBR) due to soaking was very high (about 44.25% - 72.28%) more than 20% stipulated by Jegede (2000). High CBR reduction value is an indication of high-water retaining capacity, implying, these soils would be prone to failure if exposed to water ingress.

Table 4: Ranges of index and engineering parameters with FMWH (1997) specifications

Parameter/ location	Range	FMWH (1997) Road Specification
(%) LL	35.25–46.35	not greater than 35%
% PI	16.05–22.22	not greater than 12
% Fines	40.00–66.00	not more than 35%
% CBR	25.49–59.51	Min. 30% for subbase, min. 80% for base
CBR% reduction by soaking	45.22–76.84	not more than 20% reduction (Jegede 2000)

Conclusions

It is obvious the studied soil did not meet the specification as a road subgrade material in Nigeria. This soil samples can be appraised as poor to fair road construction material in their natural state according to AASHTO stipulation. Most of the engineering indices obtained were short of the specification by the Federal Ministry of Works and Housing (1997). The index and engineering parameters (high reduction in CBR due to

soaking by water ingress, excess in amount of fines, high liquid limit, high plastic limit and high plasticity index indicate this soil is of sub-standard paving qualities. Finding show that highway geotechnical parameters of the studied soil is typical of fair to poor subbase-subgrade materials according to AASHTO and need to be upgraded through stabilisation before being use. The implication is that if this soil is used as sub-base and sub-grade materials for road pavement, such road would be expected to have a short live span.

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Preliminary Geological Assessment of Dugbe-Osin Area for Earth Dam Construction, Irepodun, Kwara State, North-Central Nigeria

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Abstract

Dugbe-Osin Dam site was evaluated to reveal the feasibility of constructing a reservoir that will serve the community. Investigated area of the proposed dam axis lies between Latitude North 8.38899, Longitude East 4.81503 and Latitude North 8.38889, Longitude East 4.81162. Mapping of the lithological facies, evaluation of salient Engineering Geological parameters and geophysical assessment of the area to reveal the thickness of the layers. Geotechnical field work entailed boring trial/test pits in the area along the proposed dam axis using the posthole hand auger boring equipment. Six trial pits and two deep boring points at right and left banks of the river complemented with Standard Penetration Test (SPT) were carried out. Geophysical investigation of the area at maximum spread of 30m with 6 VES points conducted along the proposed dam axis. The findings of the study revealed brownish and dark fine silty sand, reddish and brown lateritic soil with lateritic concretion. SPT showed light and dark brownish medium to fine silty clayey sand and Brownish weathered coarse sand. Soil bearing capacity varied from 75-220 KN/m². Natural moisture content was 11.2-14.0%, bulk density varied from 1,600-1940 kg/m³. Specific gravity varied from 2.57-2.67 (Gs). Grain size gradational analysis conducted showed wide variation at D10, D15, D60 and D85 that varied from 0.08 to 18.8 mm. Six VES data collected revealed values ranging in thickness from 0.5-8.1 m. The resistivity values revealed a range of 25.5-26861.5 Ω m. Proposed Dugbe-Osin Dam is feasible for construction as reservoir water will be tight. Seepage is expected to be very minimal within the tolerant level if sealing of few fracture areas at weathered zones of layers 2, 3 and 4 is done using bentonite or cement slurry. Collapse not envisaged and pore spaces for infiltration of underground water will not lead to excessive settlement to warrant immediate desilting.

Keywords: Lithological facies, Trial pits, Atterberg's limit, Plasticity index and Settlement.

Introduction

Delivery of an earth dam for water reservoir storage in Dugbe-Osin will go a long way to solve the problem of acute shortage of irrigable water for the inhabitants. During the dry season, it is commonly unrealistic to rely upon stream flow at a time when temperature and evaporation are often at a peak. It may become essential for a dam to be constructed on the river or stream bed to allow for off-season storage of vital water supplies. Logs may be based either on visual inspection of ditch-cutting samples brought to the surface (geological logs) of drilling operation or on physical measurements made by instruments lowered into the hole (geophysical logs). (Ibrahim et al, 2019). Geological well logs were thus utilized for the Dugbe-Osin dam site investigation as the collected ditch cuttings from trial/test pits were visually studied and subjected to geotechnical laboratory analyses to determine salient load carrying capacity of the proposed dam.

Location and Geology of the study area

The proposed dam axis geographical location falls between Lat N8.38899, Long E4.81503 and Lat N8.38889, Long E4.81162. The study area lies within the undifferentiated basement complex of Nigeria. The study area is underlain by Precambrian basement rock

which consist mainly of medium to coarse grained granite – gneiss with foliation, some aplitic, fine to medium grained granite quartz. Oyawoye (1972). The proposed earth dam is to be constructed over River Oshin in Dugbe Village of Irepodun, Kwara State.

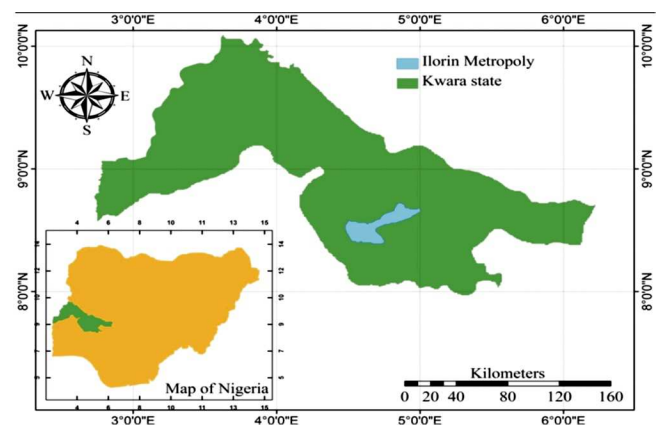


Fig. 1: Map of Kwara state showing Dugbe-Osin in Irepodun area marked red portion

Methodology and Field Work

Geotechnical field work entailed boring trial/test pits in the area along the proposed dam axis using the posthole hand auger boring equipment. Six trial pits and two deep boring points at right and left banks of the river complemented with Standard Penetration Test (SPT)

were carried out. The soil encountered were logged, visually studied and taken to the laboratory for further tests. Geophysical investigation of the area at maximum

spread of 30m with 6 VES points was conducted along the proposed dam axis, full result of which shall be published soon.



Fig. 2: Conduction of VES and Trial pit for soil sampling at Dugbe-Osin area

Good quality laterite, and other construction materials sourcing are available in the area in the upstream direction of the proposed dam axis as mapped out during the field work and preceded by reconnaissance survey. Soil variation recorded at six (6) trial pits (Figs. 3 and 4) and two (2) deep boring points complemented with Standard Penetration Test (SPT). The borings were conducted at both left and right bank of the river. Dugbe-Osin dam site consist of Dark/light brownish medium–fine grained silty/clayey sand at depth of 0 – 0.5m. Dark brownish lateritic soil at depths of 0.5 to 4.0m (Figs 3 and 4).

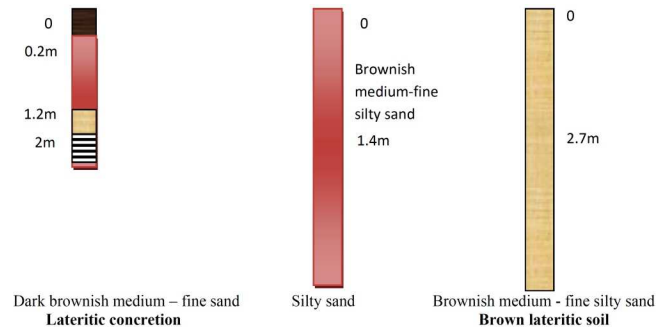


Fig 4: Lithological facies of recorded trial pit DB4, DB5 and DB6 samples of Dugbe-Osin

Data analysis and Discussion

Lithological facies of Dugbe-Osin area

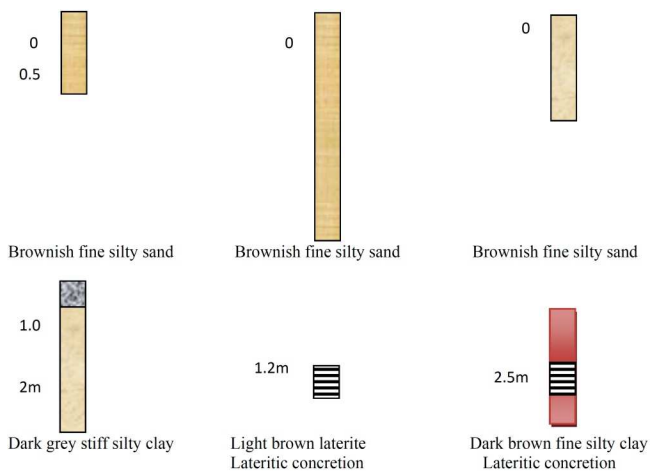


Fig 3: Lithological facies of recorded trial pit DB1, DB2 and DB3 samples of Dugbe-Osin

Soil Bearing Pressure

The shear strength data obtained from the laboratory test at different depths were utilized to compute the corresponding soil bearing capacities. The Terzaghi's modified theory for square/rectangular footings for a uniform vertical loading assuming a general shear failure for square/rectangular footings were adopted in the computation of the bearing capacity.

$$q_{ult} = 0.667CN'c + \gamma DfN'q + 0.5BN \dots \dots \dots (1)$$

$$q_{ult} = 0.867CN'c + \gamma DfN'q + 0.4\gamma BN\gamma \dots \dots \dots (2)$$

$$q_a = q_{ult} / F \dots \dots \dots (3)$$

Where;

- q_a** = Safe bearing capacity (kN/m²)
- q_{ult}** = Ultimate bearing capacity (kN/m²)
- C** = Cohesion (kN/m²)

- D_f = Depth of footing
- γ = Density
- B = Breadth of footing
- $N'c, N'q, N'\gamma$ = Bearing capacity factors is a function of frictional angle
- F = Factor of safety (Taken as 3).

DB6 sample at a maximum depth of 2.4m has the highest safe bearing pressure of 360qa, while DB1 sample at a maximum depth of 0.5m has the smallest safe bearing pressure of 70qa. The bulk density of the Dugbe-Osin area using the representative samples that ranges from DB1-DB6 also showed a value that ranges from 1600-1780 Kg/m³ (Table 1).

Table 1: Showing Soil bearing capacity of studied samples at various depth intervals

Boring Location Distance (m)	Depths (m)	Bulk Density Kg/m ³	Strength Data Cohesion		Safe Bearing Pressure (qa)
			C (kN/m ²)	ϕ (Degree)	KN/m ²
DB1 (0+000)	0 – 0.5	1600	31	10	75
	0.5 – 1.0	1730	30	30	220
	1.0 – 2.0	1770	30	31	265
DB2 (0+ 050)	0-12	1770	20	29	160
DB3 (0+100)	0-0.45	1740	15	33	145
	0.45-2.5	1790	15	33	270
DB4 (0+200)	0 – 0.2	1710	20	29	135
	0.2 – 1.2	1750	20	29	160
	1.2 – 2.0	1720	20	29	200
DB5 (0 + 250)	0–1.7	1730	18	34	240
DB6 (0 +375)	0 – 2.4	1780	30	34	360

Grain Size Gradational Analysis

The grain size gradational analysis of selected samples in Dugbe-Osin was done. Samples displayed more or less equally distributed representatives of many grain sizes (Figs. 5-6). This can be interpreted to be uniformly graded sediments because they comprise of more or less equally distributed representative fractions of many grain sizes. Most studied samples exhibited well graded sediment fraction with unique fine to coarse sandstone grain particles with greater porosity and permeability that tends to allow underground water infiltration with good compaction facies. The samples exhibited 0.1-1.0 mm diameter of grain sizes in most of the curves of the graphs (Wentworth, 1922) (Figs. 5-6). Studied samples from these pits revealed a moderate percentage of coarse materials like silty and clayey sand fractions of about 67% in totality.

DB1S1, DB1S2, DB1S3 and DB1S4 soil sample

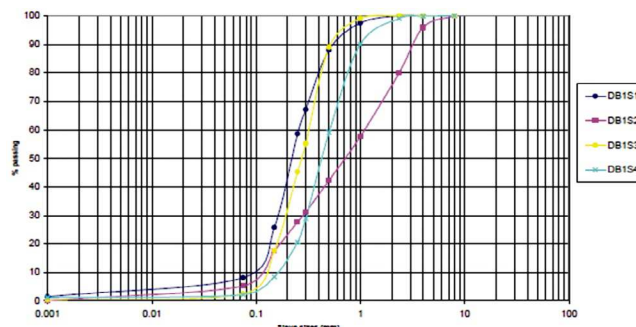


Fig. 5: Grain size gradational graph of DB1S1, DB1S2, DB1S3 and DB1S4 studied samples

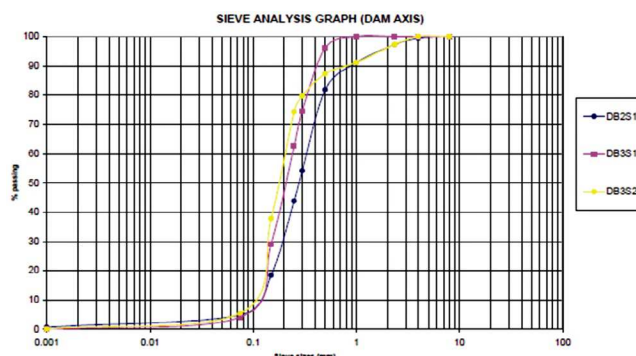


Fig. 6: Grain size gradational graph of DB2S1, DB3S1 and DB3S2 studied samples

composition from the gradational curve lie together with almost same engineering property of very low fine material with high proportion of coarse grained sand in its matrix. The fine and medium grained sand components of the samples from these pits revealed it is moderate for the construction of a dam with no envisaged collapse.

Studied samples DB1S1-DB1S4, DB2S1, DB2S1, DB3S1, DB3S2 and DB1S1-S3 are all well graded coarse to fine grained sand and little amount of clay/silt in the matrix with sandstone facie property (Figs. 5-6). The interlocking property of the grains due to the sub-angularity of the particles will make them suitable for dam construction because they have revealed a higher shear strength with good transportation and erosion histories.

Conclusions

Dugbe-Osin dam construction in Irepodun area is feasible with varieties of lithological facies, good graded gradational grain sizes recorded and other measured geotechnical properties have all revealed it will be water tight to serve the inhabitants better for irrigation purpose.

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Evaluating the MDD and OMC of Remolded Cohesive Materials from some North-eastern Nigerian Lithoseismic Layers

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Abstract

Dugbe-Osin Dam site was evaluated to reveal the feasibility of constructing a reservoir that will serve the community. This study is aimed at assessing the relationship of lithoseismic layers with the maximum dry density (MDD) and optimum moisture content (OMC) of the layers remolded materials. Five (5) locations, located North-eastern Nigeria, were each drilled to depths of 33m to create seismic refraction survey wellbore and to recover geological materials to be used for geotechnical tests. At each location, the seismic surveys were conducted at 0m, 1m, 2m, 3m, 4m, 5m, 7m, 10m, 15m, 20m, 25m and 30m depths to ascertain the lithoseismic layers and their corresponding velocities. The seismic survey revealed that each of the 5 locations have 2 lithoseismic layers within depths range of 0 to 30m; the 2nd layer has higher velocity than the 1st layer. Based on the number of layers, the remolded materials recovered from each location were divided into 2 batches and each batch subjected to sieve analysis to determine their gradation and further subjected to compaction to determine their MDD and OMC. The sieve analysis revealed that all the remolded materials are cohesive while the compaction tests revealed that the 1st layer materials have higher OMC and lower MDD than the 2nd layer materials.

Keywords: *Evaluation, MDD, Remolded Cohesive Materials, North-eastern Nigeria, Lithoseismic Layers*

Introduction

Lithoseismic layers are litho-images got from seismic refraction or reflection surveys. Seismic refraction surveys are used in determining weathered zones prior to civil engineering projects and/or oil exploration data processing. The physical principle guiding seismic refraction presumes that seismic velocity generally increases with depth and that lithostrata of similar lithologic characteristics make up alithoseismic layer, which is different from the underlying layer. Conventionally, the initial/shallowest layer(s) is the weathered lithoseismic zone and has lower velocity than the underlying layers. Seismic refraction is achievable through either the surface seismic method or hole (uphole, downhole or crosshole) seismic method. In either case, the physical principle guiding the survey is same and the interest is in ascertaining the seismic velocity of the various lithoseismic layers from which the required information is inferred.

Seismic wave velocity, basically, is dependent on the elastic moduli and densities of the geological material through which the wave propagates. There is direct relation between seismic wave velocity and density of geological material. In the case of undisturbed sedimentary materials, density is equivalent to the extent of compaction; the more the compaction, the more the seismic velocity of the lithoseismic layer. Works by Han et al (1986), Kahraman (2002), Ojha and Sain (2014) and Salah et al (2018) using some undisturbed sedimentary rock core samples (claystone, limestone, sandstone and mudstone) revealed that

seismic velocity of the lithoseismic layers increases with both the depth of burial and extent of compaction. The above discussed velocity of lithoseismic layers have been chiefly based on undisturbed sample with little or no interest on the variations of seismic velocity with compaction of remolded materials.

The present research focuses on how seismic wave velocity changes with laboratory generated compaction parameters of the remolded lithoseismic layer cohesive materials. The cohesive materials used for the research were collected from 5 downhole seismic locations in Gongola Arm, North-eastern Nigeria.

Regional Geology of Research Area

The research area occurs within latitude 9°57¹¹N and 10°11¹¹N and longitude 10°58¹¹E and 11°00¹¹E in Gombe state, North-Eastern part of Nigeria. The area is underlain by the Post-Santonian Kerri-Kerri and Gombe Formations of the Gongola Arm, Upper Benue Trough. The tectonic evolution of Benue Trough as well the Gongola Arm can be found in literatures like Wright (1976), Ofoegbu (1983) and Tukur *et al.* (2015). The Maastrichtian Gombe Formation is composed of Sands, silts, shales and coal while the overlying Paleocene Kerri-Kerri Formation is composed of claystones, coarse-grained sandstones, gritty clay and siltstones (Popoff *et al.* 1986; Zaborski *et al.* 1997).

Materials and Methods

Each sampled location was drilled following rotary

drilling with flush method in which water-based drilling fluid was used for flushing following ASTM D5783-18 standard. The wellbore was drilled to depths 33m and geological materials collected at intervals of 1.5m. Significant amount of the material is collected from each interval, labeled and preserved for the relevant geotechnical analyses.

In accordance with ASTM D7400D7400M-19 standard, a downhole seismic refraction surveys were conducted at depths of 0, 1, 2, 3, 4, 5, 7, 10, 15, 20, 25 and 30m. The source energy was taken at 0m depth at an offset of 3m from wellbore while the receiver sensor was lowered inside the wellbore at the depths stated above. The target is to generate seismogram of each depth. Using a spreadsheet based Interpretation System Software, the time of first break indicated in the seismogram was interpreted into thickness of lithoseismic layers and corresponding velocity of wave across the various layers. Based on the lithoseismic layers for each location, the recovered geological materials were divided into batches.

Each batch of geological materials were subjected to wet sieving following ASTM D1140-17 standard to ascertain their gradation and subsequently to light compaction test using 2.5kg rammer according to BS1377-4 standard. The compaction test was to determine their maximum dry density (MDD) and optimum moisture content (OMC).

Results and Discussion

Interpretation of the seismogram showing the lithoseismic layers and their corresponding velocities are shown in Figures 1a-e.

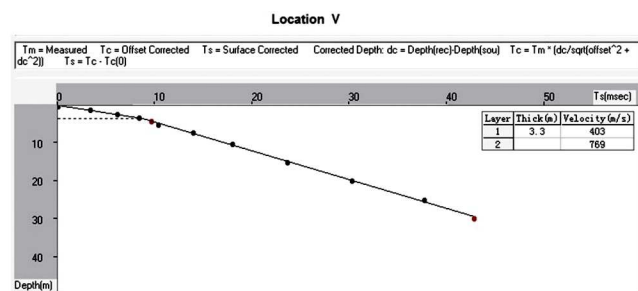


Fig. 1a: Downhole Data Interpretation of Locations V

From Figures 1a-e it can be seen that within the survey depth range (0m to 30m), there are two lithoseismic layers in the 5 locations. The velocities of the 1st lithoseismic layers ranges from 318 to 403m/s while velocities of the 2nd lithoseismic layers ranges from 738

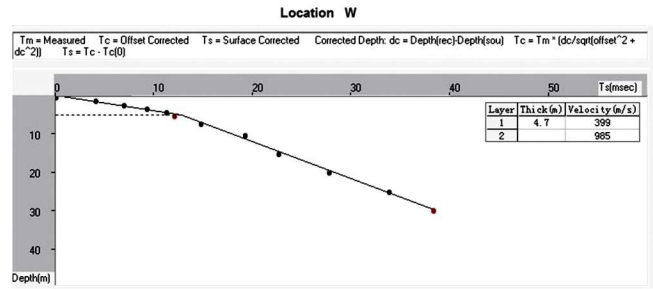


Fig. 1b: Downhole Data Interpretation of Locations W

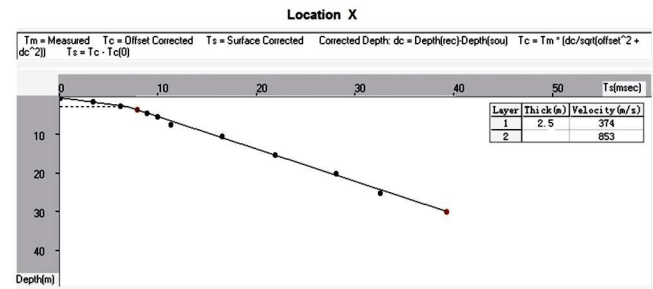


Fig. 1c: Downhole Data Interpretation of Locations X

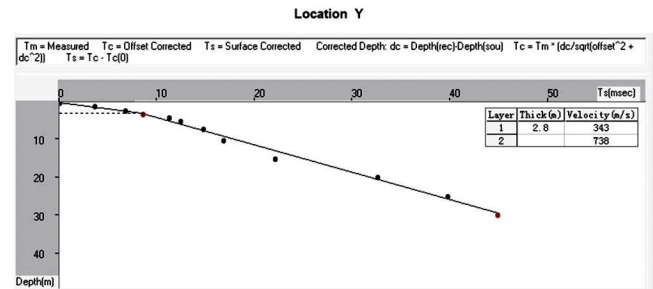


Fig. 1d: Downhole Data Interpretation of Locations Y

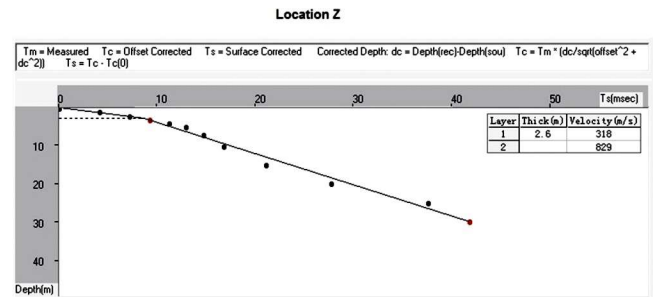


Fig. 1e: Downhole Data Interpretation of Location Z

to 985m/s. As expected, velocity of 1st (weathered) layer is lower than that of the underlying 2nd layer in all cases. This is in concordance with the general geophysical principle that seismic velocity increases with depth which is generally commensurate with the overburden pressure.

The particle distribution curves of remoulded materials collected from the lithoseismic layers are shown as Figure 2a and b with a chart showing summary of the

materials gradation is shown in Figure 3.

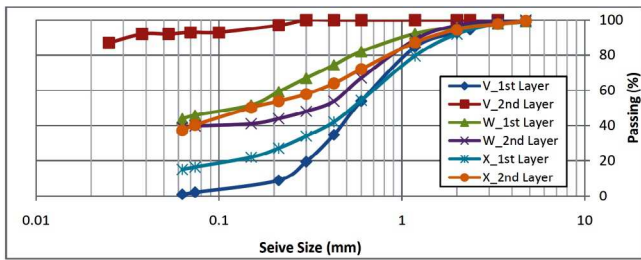


Fig. 2a: Particle size distribution curves for materials from locations V, W and X

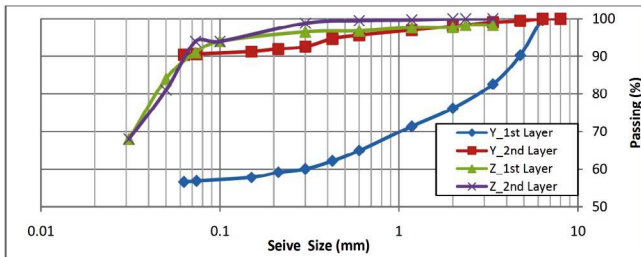


Fig. 2b: Particle size distribution curves for materials from locations Y and Z

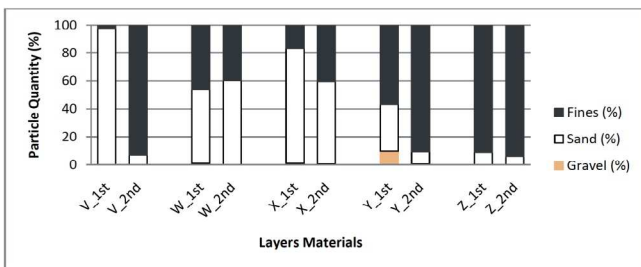


Fig. 3: Summary of particle size distribution of the materials

Figure 3 shows that all the 2nd layers contain more fine than their corresponding 1st layer except in the case of location W. Amongst all the locations materials, only materials from W have coarse-fine admixtures of near equal proportions both in the 1st and 2nd layer materials. Comparing Figures 3 and Figures 1b, it can be seen that 2nd layer of W has the highest velocity and the 1st layer is the thickest in relation to others. This indicates that lithologic characteristics of the layers materials have effect on relative velocities of the lithoseismic layers of a location but the effect of depth takes precedence over the effect of lithology.

The MDD and OMC for the 1st and 2nd lithoseismic layers are shown as Figure 4a and b.

Figure 4a shows that in all the 5 locations, MDD of the 2nd lithoseismic layers is more than the MDD of 1st lithoseismic layers. Conversely, Figure 4b shows that in all the 5 locations, OMC of the 1st layers is more than the OMC of the 2nd layer. Recall that in undisturbed state,

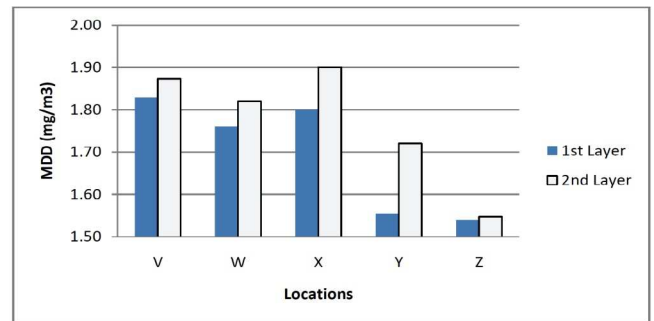


Fig 4a: MDD of materials from the 5 locations

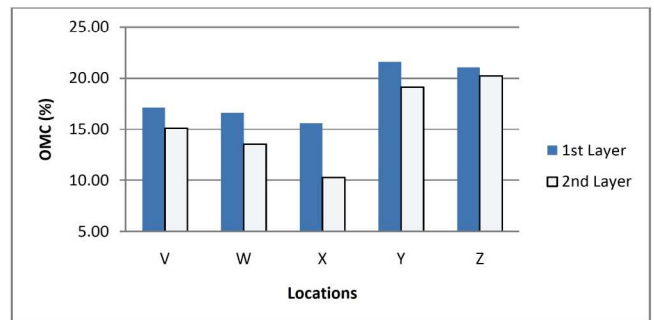


Fig. 4b: OMC of materials from the 5 locations

materials of the 2nd layers are more compacted (higher density) than materials of the 1st layers while materials of the 1st layers have higher permeability than the materials of 2nd layers. Remolded materials of 2nd lithoseismic layers, which had higher compaction, showed higher maximum dry density (MDD) than remolded materials of the 1st lithoseismic layers, which had lower compaction. Remolded materials of 1st lithoseismic layers, which had higher permeability, showed higher OMC than remolded materials of 2nd lithoseismic layers, which had lower permeability. Therefore, the compaction history of lithoseismic layer cohesive materials also manifest in their remolded state. This may be evident in only the geological materials of 1st and 2nd lithoseismic layers.

Conclusions

1. Within the depth range of 0m to 30m in the study area (North-eastern Nigeria), there are 2 lithoseismic layers.
2. The undisturbed relative compaction imprints of 1st and 2nd lithoseismic layers cohesive materials manifest in their remolded state. Materials of 2nd layers have higher MDD and lower OMC than the materials of 1st layers.
3. The lithologic characteristics the lithoseismic layers have secondary effect on seismic velocities of the lithologic layers. The overburden pressure has primary effect on the seismic velocities.

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Evaluation of Potential Radiological Hazards Associated with Rocks in and around Mika Uranium Mineralization in Taraba State, Northeastern Nigeria

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Abstract

The radioactivity of rocks contributes to the external alpha/gamma dose rates that humans receive from the environment. Therefore, it is very important to measure the radioactivity of rocks and understand the dynamics of the radioisotopes in the natural environment as it affects building or construction practices. Previous studies of natural radioactivity within the northeastern part of Nigeria revealed high anomalous uranium concentration within Mika and environs. Hence, the need to further study the radiological hazards of the radionuclides associated with uranium and other singly occurring radionuclides which contributes significantly to the alpha and gamma radiation exposure in the environment. For this purpose, fifteen (15) rock samples were collected within Mika town and environs, mainly dominated by Medium-grained granite and Porphyritic granite. Gamma spectrometric analysis was carried-out using the 76x76 mm NaI(Tl) detector crystal, optically coupled to a photomultiplier tube (PMT) for the determination of ²²⁶Ra, ²³²Th and ⁴⁰K activity concentrations, while the Durrige RAD7 electronic radon detector was used to measure radon emission from the rock samples. The results show that the concentration of ²²⁶Ra ranged from 15.33 to 63.38 Bqkg⁻¹, ²³²Th ranged from 41.51 to 333.64 Bqkg⁻¹ and ⁴⁰K concentration ranged from 161.73 to 2166.56 Bqkg⁻¹. It further revealed that ²³²Th and ⁴⁰K are the radionuclides contributing the most of the gamma radiation dose within the study area with a dose rate range of 79.50 to 270.46 nGyhr⁻¹ and an annual effective dose range of between 0.24 to 0.83 mSvyr⁻¹. Radon (²²²Rn) emanation coefficients of the rocks ranged between 0.7 to 5.1 with an arithmetic mean value of 2.17 across the area, while, the exhalation rates ranged from 0.73 to 4.83 mBqkg⁻¹hr⁻¹. Radon-222 was also found to be contributing significantly to the radiation exposure within the study area, with effective alpha dose equivalent ranging between 0.20 – 1.19 mSvyr⁻¹. Most of the results show values higher than WHO and ICRP recommended levels for human radiation exposure, hence, it is highly recommended that radiological hazards related to the rocks within the Mika Uranium Mineralization be evaluated before it is used for any construction purpose.

Keywords: Radioactivity, gamma spectrometry, Radon-222, Radiation dose rate.

Introduction

Radioactivity in the environment consists of the three well-known radioactive series: the uranium series originates with ²³⁸U, the thorium series originates with ²³²Th and the actinium series originates with ²³⁵U. There are also several singly occurring radionuclides, the most important one is ⁴⁰K because it is a gamma-ray emitter in addition to beta decays and therefore contributes significantly to the gamma radiation exposure (El-Dine, *et. al.*, 2001). The gamma radiation doses as a result of the combined effect of ²²⁶Ra, ²³²Th and ⁴⁰K radionuclide exposure, alpha radiation due to radon (²²²Rn), radon emanation coefficient and exhalation/exhalation rates from rock samples collected within the Mika uranium mineralization in the northeastern part of Nigeria, is presented in this report.

Study Area

This study was conducted within the basement (granitoids) of the area in the Northern part of the Adamawa Massif. The area can be located on the topographic map of parts Monkin sheet 216 and Dong sheet 195. It lies between latitudes 08°48' and 09°4' and

longitudes 11°30' and 11°48'. The study area is dominated by porphyritic granite and medium-grained granite. The climate of the area is typically a tropical climate, marked by dry and rainy seasons, with a mean annual rainfall ranging between 819 – 1761mm, spanning about seven months, i.e. from April to October (Turner, 1983).

Materials and Methods

The analysis was carried-out using a 76x76mm NaI (Tl) detector crystal optically coupled to a photomultiplier tube (PMT) and a DURRIDGE RAD7 electronic radon detector.

Radon Emanation Coefficient Measurement

The emanation coefficient is defined as the fraction of radon atoms generated that escape the solid phase in which they are formed and become free to migrate through the bulk medium. This term has also been referred to as the emanation fraction or the emanation power (Ishimori, *et. al.*, 2013). The total activity of radon released into the air from the samples were evaluated using the DURRIDGE RAD7 electronic

radon detector, to measure radon concentration of each rock sample, whereas, the total activity of radium in the rock samples were determined using gamma spectrometry.

The emanation coefficient was calculated using the formula;

$$E = \frac{VC}{MR} \dots\dots\dots(1)$$

Where; E = the emanation coefficient, V = the effective volume of the sampling device (m³), C = the radon concentration (Bqm⁻³), M = the total mass of the sample (kg), R = the radium activity concentration (Bqkg⁻¹).

Radon Exhalation Rate Measurement

Radon exhalation rate is defined as the radon activity released into the air per unit time from the mass of the matrix and is measured by enclosing the sample in a closed chamber and monitoring the buildup of radon concentration (using DURRIDGE RAD7 electronic radon detector) in the chamber at regular time intervals (Petropoulos, *et. al.* 2001; Chen, *et. al.*, 2010).

The exhalation rate;

$$E_m = \frac{CV\lambda}{M(T + \lambda^{-1}[e^{-\lambda T} - 1])} \dots\dots\dots(2)$$

Where; E_m = is the mass exhalation rate (Bqkg⁻¹h⁻¹), M = is the total dry mass of the sample (kg), V = is the effective volume of chamber (m³), C = is the ²²²Rn concentration present in the chamber volume (Bqm⁻³), λ = is the radioactive decay constant of ²²²Rn (h⁻¹), T = is the measurement time (h).

Dose Rates Calculations

The absorbed dose rate (D) in nGyhr⁻¹ from ⁴⁰K isotope and the radionuclides from ²²⁶Ra and ²³²Th was calculated using the formula below;

$$\text{Dose Rate (nGyhr}^{-1}\text{)} = 0.460A_{Ra} + 0.614A_{Th} + 0.0418A_K \dots\dots\dots(3)$$

Where; 0.460, 0.614 and 0.0418 are the conversion factor (nGyhr⁻¹/Bqkg⁻¹) for ²²⁶Ra, ²³²Th and ⁴⁰K respectively, while A_{Ra}, A_{Th} and A_K are the radiological

concentration (Bqkg⁻¹) for ²²⁶Ra, ²³²Th and ⁴⁰K respectively.

Alpha Dose Equivalent (H_E)

The level of alpha dose due to the presence of radon (²²²Rn) and its short-lived daughters in air play a significant role in radiation exposure [Pankaj, *et. al.* 2017]. CEC in 1990, recommends the conversion factor of 1 Bqm⁻³ from ²²²Rn corresponds to an effective dose equivalent of 0.05 mSvyr⁻¹.

The effective alpha dose equivalent, H_E (mSvyr⁻¹) can be expressed as:

$$\text{Alpha Dose Equivalent, } H_E = 0.18fA_{Ra} + 0.45 \dots\dots\dots(4)$$

Where; f is the emanation coefficient and A_{Ra} is the radium specific radioactivity in materials.

Annual Effective Dose Rate due to Gamma Radiation (DE_{out})

The annual outdoor effective dose rate (DE_{out}) to a member of the population within the study area was calculated from the absorbed dose rate (D), taking into account the conversion factor (CF) of the absorbed dose in air to the corresponding effective dose, and the occupancy factor (OF_{out}). This is equal to;

$$DE_{out} = D \times CF \times OF_{out} \dots\dots\dots(5)$$

Where; DE_{out} units are in mSvyr⁻¹, D units are in nGyhr⁻¹, CF = 0.7x10⁻⁶ Sv/Gy [8], and OF = f_{out} x 24hrs x 365.25days. According to UNSCEAR (2000), humans are expected to spend 20% of their time outdoors and 80% indoors, this means that f_{out} = 0.2 and f_{in} = 0.8, for the annual outdoor and indoor effective dose rates respectively. The f_{out} and f_{in} proposed by UNSCEAR (2000) is not realistic for the settlers in the study area because they are mostly farmers, cattle/sheep rearers, hunters etc., so they spend much more time outdoors and less time indoors compared to the time proposed by UNSCEAR (2000), therefore, for this particular work, an average of 12 hours was taken (i.e. f_{out} = 0.5 which means f_{in} = 0.5) to estimate annual outdoor effective dose rates, assuming that 50% of their time is spent outdoors and 50% is spent indoors.

Radium equivalent (Ra_{eq}) Determination

Assessment of radiological hazards was made by

calculating the radium equivalent activities index. The radium equivalent activity (Ra_{eq}) is a weighted sum of activities of the ^{226}Ra , ^{232}Th and ^{40}K based on the assumption that 370 Bq/kg of Ra, 259 Bq/kg of Th and 4810 Bq/kg of K produce the same gamma-ray dose rates UNSCEAR (2000) as given by the following equation;

$$Ra_{eq} = A_{Ra} + 1.43A_{Th} + 0.077A_{K} \dots\dots\dots(6)$$

Where; A_{Ra} , A_{Th} , and A_{K} are the activity concentration (in Bq/kg) of ^{226}Ra , ^{232}Th and ^{40}K , respectively.

External Hazard Index (H_{ex})

The model of the external hazard index (H_{ex}) places an upper limit to the external gamma radiation dose from materials to unity, which corresponds to a radium equivalent activity of 370 Bq/kg. It is defined;

$$H_{ex} = \frac{A_{Ra}}{370 \text{ (Bqkg}^{-1})} + \frac{A_{Th}}{259 \text{ (Bqkg}^{-1})} + \frac{A_{K}}{4810 \text{ (Bqkg}^{-1})} \leq 1 \dots\dots\dots(7)$$

Where; A_{Ra} , A_{Th} and A_{K} are the mean activity concentrations of ^{226}Ra , ^{232}Th and ^{40}K in Bq/kg, respectively. The value of this index should be less than unity to keep the radiation hazard negligible (Beretka and Mathew, 1985).

Results and Discussion

Dose Rates due to Alpha/Gamma Radiations

In order to evaluate radiation hazards, the radioactivity results were converted in four dose related quantities: effective alpha dose equivalent due to radon gas, the absorbed gamma radiation dose rate in the air (and also, the annual effective dose rate) (IAEA, 2003), the radium equivalent (Ra_{eq}) and External hazard index (H_{ex}) (Beretka and Mathew, 1985). Ra_{eq} and H_{ex} are parameters employed to check radioactivity hazards associated with naturally occurring radioactive rocks or any material used for construction purposes, e.g. granite chips used for borehole gravel packing, building constructions, etc. Table 1 shows analysis results and resultant doses due to the radionuclides of interest.

Table 1: Analysis results and Calculated Radiation doses due to some Naturally Occurring Radionuclides in Rocks

Sample ID	Coordinates	Sample Location	^{222}Rn (Bq/m ³)	K-40 (Bq/kg)	Ra-226 (Bq/kg)	Th-232 (Bq/kg)	Em (mBq/kg/hr)	E (%)	H_E (mSvyr ⁻¹)	D (nGyhr ⁻¹)	DE_{out} (mSvyr ⁻¹)	Ra_{eq} (Bq/kg)
1	08°58.121'N 11°43.988'E	ZING	211	1172.46	36.07	333.64	3.89	2.9	0.96	270.46	0.83	603.46
2	08°54.431'N 11°42.419'E	YAKOKO	43.6	1343.24	17.50	203.18	0.80	1.3	0.23	188.95	0.58	411.48
3	08°50.481'N 11°42.168'E	MONKIN	138	161.73	53.42	171.95	2.24	1.3	0.65	136.91	0.42	311.76
4	08°59.076'N 11°43.121'E	KAKULU	155	947.12	15.33	70.58	2.86	5.1	0.73	89.98	0.28	189.19
5	08°59.354'N 11°37.941'E	MIKA	173	1533.13	63.38	279.95	3.19	1.4	0.82	265.13	0.81	581.76
6	09°00.520'N 11°39.505'E	WURO-YAYA	119	2166.56	17.53	137.39	2.19	3.4	0.56	182.98	0.56	380.82
7	09°03.945'N 11°36.312'E	MANZALANG	39.7	1052.40	28.48	195.44	0.73	0.7	0.20	177.09	0.54	388.99
8	09°03.845'N 11°32.148'E	MARARABAN YORRO	114	923.64	33.49	41.51	2.10	1.7	0.53	79.50	0.24	163.97
9	09°00.518'N 11°33.132'E	BAKINYA	262	1419.60	30.20	193.25	4.83	4.3	1.19	191.89	0.59	415.86
10	08°59.429'N 11°32.180'E	DILA	123	864.99	44.50	175.71	2.27	1.4	0.58	164.51	0.51	362.37
11	08°56.551'N 11°31.061'E	KPANTISAWA	216	1231.56	55.39	228.62	3.98	2.0	1.02	217.33	0.67	477.15
12	08°53.384'N 11°34.999'E	NYAJA	123	1025.51	59.51	46.64	2.27	1.0	0.55	98.69	0.30	204.76
13	08°50.809'N 11°37.987'E	TAPENLA	134	1048.68	30.01	84.27	2.47	2.2	0.62	109.38	0.34	231.27
14	08°49.059'N 11°38.582'E	KASSA	76	1375.58	19.58	62.71	1.40	1.9	0.36	105.01	0.32	215.18
15	08°51.614'N 11°41.291'E	BODUGA	122	1032.50	31.87	203.08	2.25	1.9	0.57	182.51	0.56	401.78

The alpha dose equivalent due to radon-222 within the study area ranges from 0.20 – 1.19 mSvyr⁻¹. The result shows that the rock samples from Bakinya town has the highest H_E (1.19 mSvyr⁻¹) which is far above the WHO

recommended level of 0.1 mSvyr⁻¹ (WHO, 2000) and slightly above ICRP recommended level of 1 mSvyr⁻¹ (ICRP, 1991). Also the rock samples at Kpantisawa town has an alpha dose equivalent value of 1.02 mSvyr⁻¹,

which is also slightly above the ICRP recommended level. All the other rock samples show values less than ICRP recommended level but above WHO recommended level.

Calculated dose rate as a result of gamma radiations from the study area ranged from 79.50 nGyhr^{-1} (at Mararaban Yorro community) to $270.46 \text{ nGyhr}^{-1}$ (at Zing town). Mika town recorded the second highest dose rate with $265.13 \text{ nGyhr}^{-1}$ and the second lowest was recorded at Kakulu town with dose rate of 89.98 nGyhr^{-1} . The mean dose rate across the study area is $164.02 \text{ nGyhr}^{-1}$. All the dose rates calculated for the study area (Table 1) are higher than the world median value of 60 nGyhr^{-1} (UNSCEAR, 2000).

Annual Effective Dose Rate due to Gamma Radiations

The calculated annual effective dose due to gamma radiation in the study area ranged from 0.24 to 0.83 mSvyr^{-1} , with an average value of 0.50 mSvyr^{-1} . Zing and Mika towns recorded the highest and the second highest annual effective dose rates, respectively, while Mararaban Yorro and Kakulu recorded the lowest and the second lowest annual effective dose rates, respectively. WHO recommended a level of 0.1 mSvyr^{-1} (WHO, 2000) for annual effective dose. While, according to the ICRP recommendations; the public should not be exposed to more than an average of 1 mSvyr^{-1} (ICRP, 1991). Based on the parameters used to calculate the annual effective dose rates, all the samples have values higher than WHO recommended limit but lower than ICRP recommended tolerable limit.

Radium Equivalent Index (Ra_{eq})

The calculated radium equivalent index of the rock samples collected from the study area varies between 163.97 to 603.46 Bqkg^{-1} with an arithmetic mean value of 355.99 Bqkg^{-1} . A comparison of the calculated radium equivalent index from the measured samples with the corresponding world accepted upper limit ($Ra_{eq} = 370 \text{ Bq/kg}$, (UNSCEAR, 1982), reveals that 53% of the rock samples have Ra_{eq} values higher than the world accepted limit with the highest Ra_{eq} value recorded on the very

coarse-grained granite of Zing town while 47% of the rock samples have Ra_{eq} values within the world accepted value, the least value was recorded on the rock sample collected at Mararaban Yorro community. The calculated average value shows a much higher value than the average global value of 35 Bq/kg (UNSCEAR, 1982).

External Hazard Index (H_{ex})

The calculated external hazard index (H_{ex}) ranged from 0.44 to 1.63 with an average value of 0.96 . A comparison of the calculated H_{ex} from the measured samples with the corresponding world accepted upper limit ($H_{ex} = 1$) shows that there is a correlation between Ra_{eq} and H_{ex} because the samples having Ra_{eq} values higher than 370 Bq/kg also have H_{ex} values higher than unity and samples having Ra_{eq} values lower than 370 Bq/kg , have H_{ex} values lower than unity but overall, the calculated H_{ex} average is lower than the world accepted upper limit (1).

Conclusions

Gamma spectrometry, revealed the background radioactivity of rocks, with respect to ^{226}Ra , ^{232}Th and ^{40}K radionuclides within the study area, and the result showed that ^{226}Ra has its average value (35.72 Bq/kg) very close to the world median value (35 Bq/kg). ^{232}Th and ^{40}K are the radionuclides contributing the most of the gamma radiation within the study area, because they both have values far above their respective world median values. The combine effects of these radionuclides gave rise to radiation dose rate values higher than the world median value (60 nGy/hr). The radium equivalent index (Ra_{eq}) and external hazard index (H_{ex}) of the granite rock samples collected within the study area (These rocks are commonly used for construction purposes in the area) has an average value for Ra_{eq} much higher than the average global value of 35 Bq/kg . Hence, there is need to evaluate the radiological hazards related to the rocks within the Mika Uranium Mineralization before it is used for any construction purposes.

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Geological Evaluation of a Site for Sanitary Landfill in Asaba, Southern Nigeria

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Abstract

Asaba, a city in southern Nigeria, is associated with huge waste generation and indiscriminate disposal. The city is underlain by sedimentary rock units which permits easy interaction between surface contaminants and the groundwater systems. This necessitates that wastes in such environment are properly disposed in a geologically designed sanitary landfill. This study was designed to undertake an engineering geological evaluation of a designated site in Asaba to ascertain the suitability of the subsurface materials. Sixty-four vertical electrical sounding points with electrode spacing of 133m and 10m were probed. One hundred and forty-four soil samples comprising ninety-six disturbed samples at 0.5m interval and forty-eight undisturbed soil samples at 1.0m interval were obtained from sixteen test pits (3m depth and 70m apart) and from two geotechnical boreholes at 17.25m and 20m depth. Index and engineering properties determined were Natural Moisture Content (NMC), Specific Gravity (SG), grain size distribution, consistency limits, permeability and coefficient of consolidation. The geo-electric layers were: top soil, lateritic soil, shale, ferruginised sandstone and saturated sandstone. Depth to groundwater was about 40m. The range of NMC for the soil samples was 8.62 - 22.72%. The soil samples had SG with a range of 2.59 - 2.97. Plasticity index ranged from 3.70 - 32.00% suggesting low to high swelling potential. The soils were all well-graded and plotted above the A-line. Permeability coefficient (3.97×10^{-8} - 7.6×10^{-6} m/s) was within the optimum range required for attenuation as it revealed a moderately slow to very slow flow of leachate. Coefficient of consolidation range was 9.46×10^{-2} - $4.66 \text{m}^2/\text{year}$ under varying pressure of 50.00 to 400.00 kN/m². The subsurface materials at Okpanam junction, Asaba with minimal soil enhancement are suitable for the construction of a geologically designed landfill.

Keywords: Geological, Evaluation, Site, Sanitary, Landfill, Asaba, Southern Nigeria

Introduction

The rapid increase in population and industrialization in Asaba city has resulted in a huge volume of waste, hence the need of a landfill owing to the fact that such is yet to be constructed in the area. Ogunsanwo (1996) investigated the geotechnical properties of three soils from south-western Nigeria as regards their suitability for use as mineral seals in waste disposal landfills. Adeyemi and Oyedirin (2007) employed integrated geophysical and geotechnical methods to confirm that a location in the outskirts of Ibadan is suitable or otherwise for landfill. Ige (2007) assessed five soils from south western Nigeria for use as mineral seals in sanitary landfills.

The aim of the study is to undertake an engineering geological evaluation a site to ascertain the suitability of the subsurface materials in supporting the provision of a sanitary landfill in the city, while the objectives include evaluating the geologic units beneath the area, characterizing the geotechnical properties of the subsurface materials and to ultimately determine the suitability of the area for a sanitary landfill construction.

The study area is located at the outskirts of Asaba the capital city of Delta State, Nigeria. It falls within Longitudes N6°12'35.4"-N6°12'45.8" and Latitudes E6°39'7.3"-E6°39'14.0" (Fig. 1.1). It is easily accessible

via the Benin - Onitsha highway.

The major geological formation comprises the Bende-Ameki Formation. It is capped by the Ogwashi-Asaba formation and underlying it is the Imo Formation (Fig. 1.2).

Methods of Study

It involved geological field mapping, geophysical survey and geotechnical investigation. The geophysical survey was carried resulting in 64 Vertical Electrical Soundings with electrode spacing of 133m and 10m were probed. Sixteen pits (3m depths) at 70m apart were excavated. Index and engineering properties determined were natural moisture content (NMC), specific gravity (GS), grain size distribution, consistency limits, permeability and Coefficient of consolidation parameters.

Results and Discussions

The soil lithology ranges from medium dense reddish brown silty clay to hard mottled ferruginised sandstone. The Electrical Resistivity Survey reveals a layered geology of Top Soil/Lateritic clay/ Shale/Ferruginised Sandstone. The resistivity of clay layers ranges from 654 - 1576Ω-m (Fig. 1.3). The range of Natural Moisture Content for the soil samples was 8.62 -

22.72%. With low to high swelling potential, the soils exhibit low hydraulic conductivity and are suitable for liner purpose. The soil samples had specific gravity with a range of 2.59 to 2.97. Plasticity index ranged from 3.70 – 32.00%.

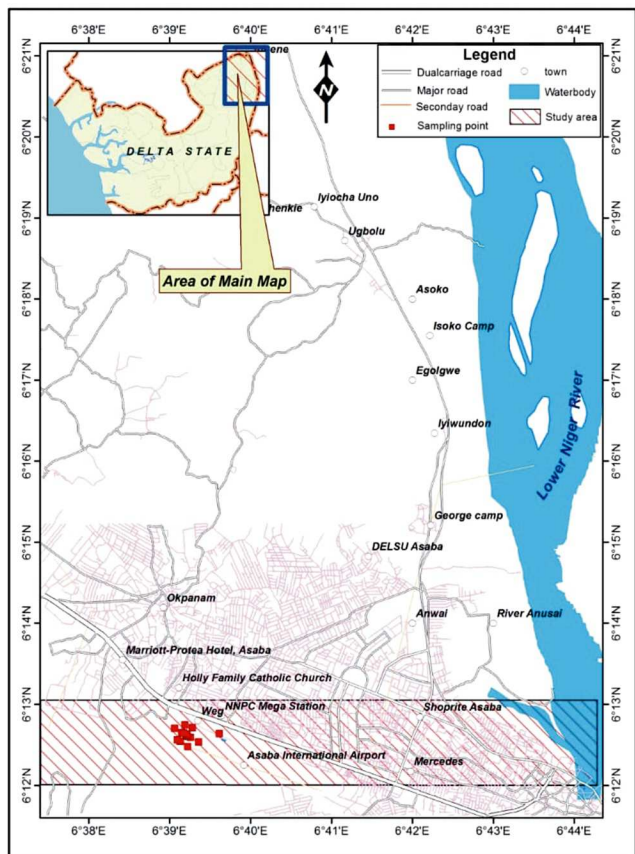


Fig. 1.1: The Study area.

The soils were all well-graded and plotted above the A-line. Permeability coefficient ($3.97 \times 10^{-8} - 7.6 \times 10^{-6}$ m/s) was within the optimum range required for attenuation as it revealed a moderately slow to very slow flow of leachate. Coefficient of consolidation range was $9.46 \times 10^{-2} - 4.66 \text{ m}^2/\text{year}$ under varying pressure of 50.00 to 400.00 kN/m².

Conclusions and Recommendations

The fairly thick, near-surface lateritic clay (15m) will prevent pollution by leachates from any waste in the vicinity. This project has presented significant preliminary results which suggest that the shales can be utilized as a component of barrier design in landfill application. The study area is suitable for a landfill location because it meets most of the requirements for siting a landfill as recommended by the World Bank (2004) in siting a landfill. Based on the generated profile

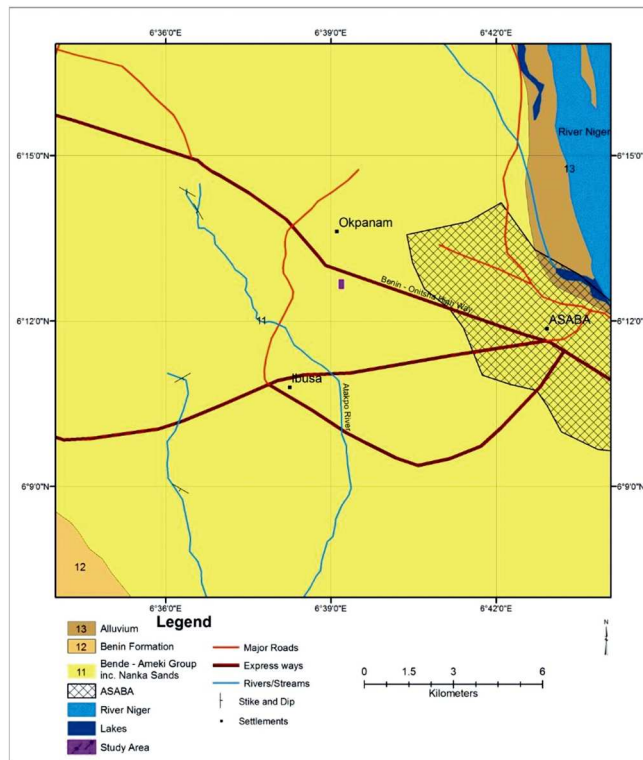


Fig 1.2: The geological map of the study area (NGSA, 2006)

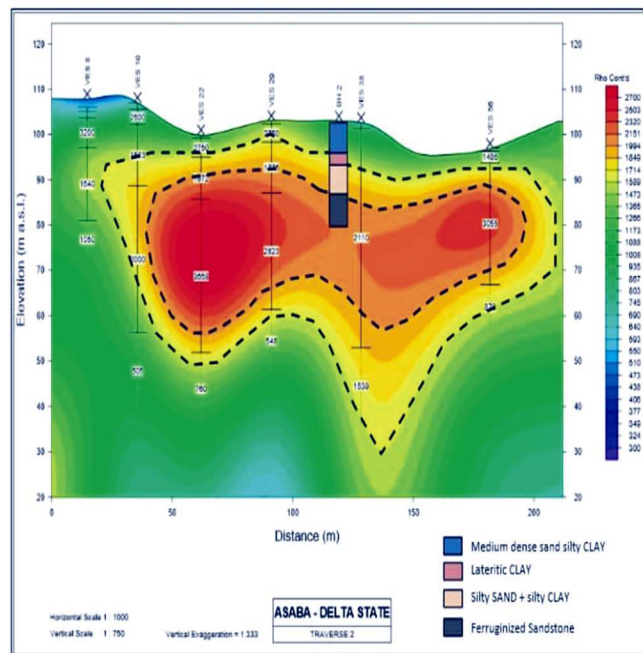


Fig 1.3: The Resistivity of a geophysical profile

and properties of soils, the investigated location is suitable for a sanitary landfill, as well as the clay as a landfill liner.

However more comprehensive mineralogy and geochemical studies is recommended. Seismic

refraction method is sensitive to fractures and is also recommended alongside other geophysical methods. A

comprehensive hydrogeological study is also recommended.

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The Relationship Between Slope Angle and Landslide Dimensions Derived from Limit Equilibrium Simulations and Statistical Analysis

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Abstract

Two-dimensional limit equilibrium simulations and statistical analysis were used to study the relationship between slope angle and landslide dimensions in South-East Nigeria. A total of sixteen (16) simulations were performed, four for each of the geologic settings. The slope height and shear strength parameters were fixed for each geologic setting, while the slope angle was varied. The representative slope angles were 30°, 40°, 50° and 60°. The results of the simulations revealed that the potential slip mass volume increased with increase in slope angle. Similar results were obtained in all the different geologic settings which are Ameri/Enyigba, Nanka, Iva Valley and Obudu. It also showed that the geologic material does not necessarily determine the volume of the potential sliding mass in any geologic setting. The results of The Pearson correlations for Obudu revealed that Slope angle had high significant relationship with elevation, length and width. This implies that the landslide parameters in this area are significantly related and this agrees with the results of the limit equilibrium simulations obtained in the area. In Nanka, The Pearson correlations showed that Slope angle had high significant relationship with elevation, length and width. The correlation analysis in Nanka area infers that the landslide parameters in this area are significantly related and this agrees with the results of the limit equilibrium simulations obtained in the area. In Iva Valley, The Pearson correlations revealed that the landslide measurements are significantly related and supports the results of the limit equilibrium simulations. Ameri/Enyigba had minimal significant linear relationship among the landslide measurements due to the nature of the terrain.

Keywords: Simulations, Limit equilibrium, slope angle, statistical analysis.

Introduction

Landslides represent a serious problem almost in all parts of the world, because they cause economic or social losses on private and public properties. In Nigeria, landslides constitute a serious environmental concern not only because some are catastrophic, destroying lives and damaging properties but also because the variations in their mode of occurrence, frequency, and spatial distribution are wide and often unpredictable. Annually, precipitation during rainy season induces fewer but long-travel landslides on the basement terrains and more but short-travel mass movements in the sedimentary basins (Igwe, 2015a; Igwe et al., 2015, 2013). When landslides occur in different geologically and geomorphologically localities, they vary in modes and magnitude, which has important practical implications since the risk associated with a landslide depends on both mode and magnitude (Igwe, 2014). This implies that landslide risk reduction in any locality or region is not possible without first understanding the mode and magnitude of landslides in that area. Recently, many researchers have made contributions to the size of landslide occurrences globally. Korup et al. (2007) stated that occurrence of giant landslides is promoted by either available mean local high relief or large-scale discontinuities with low rock strength prevalence.

The controls of landslide size differ for homogenous and heterogeneous environments, landslide size appears to be constrained by the heterogeneities like fractures and weak layers in a heterogeneous environment (Katz and Aharonov, 2006). Liu and Koyi (2012) studied the internal deformation of flow-like downslope mass movement and noted that the size of landslide decreases with increasing material strength. Katz et al. (2014) used two-dimensional discrete element numerical simulation to carry out a study of controls on the size and geometry of landslides. They concluded that in addition to the peak strength of the slope material, the initial slope angle is another factor that controls the amount of material available for landslides, thus the size of the resultant landslide. This suggests that in steeper slopes, more material disintegrates for a given material strength, and consequently the resultant landslide is larger. Igwe (2015a) suggested that in addition to the slope material, steeper slopes produce larger landslides. Though various authors have made contributions on the relation of slope angle to landslide dimensions, the probable size and geometry of an individual landslide resulting from a given geological and geomorphological setting have not been addressed. This paper will use limit equilibrium simulations and statistical analysis to establish a relationship between landslide sizes and slope angle in four different geological and geomorphological settings (Ameri and Enyigba, Nanka, Iva Valley, and Obudu) in Southeastern Nigeria.

The Study Area

Geology and Geomorphology

Southeastern Nigeria is generally regarded as having fairly uniform landform of low relief. Though the low lands and plains dominate the entire area, they are still areas of highlands and prominent cuesta landscapes. The sedimentary terrains have lower elevations while the metamorphic terrains have higher elevations (Fig. 1). Ameri/Enyigba is dominated by the Abakaliki shale and have low relief with elevations ranging from 47-105m above sea level. Nanka is part of a regional system of escarpments. Nanka lies near the minor escarpment of the Awka-Orlu uplands formed by the Nanka sands in the Nanka area (Okagbue, 1992). Iva valley is dominated by the friable Ajali sandstone and is part of the Enugu-Okigwe Cuesta. The scarp slope of most of the cuesta is dissected by a complex of gullies created by head ward erosion of east flowing rivers sourced from the rock formations. Mass wasting processes are progressively causing the Cuesta face to retreat to the west (Obi et al., 2001).

The Oban Hills and Bamenda Massif (Obudu) are metamorphic terrains and have the highest elevations of between 450 -1600m above sea level. The topography is rugged and comprises north-east trending ridges separated by lowlands, which form valleys and passes (Igwe et al., 2015). The dominant rock unit in the Plateau is the migmatite-gneiss complex, while schist, quartzite, dioritic and peridotitic rocks are comparatively less in abundance (Ekwueme, 1990a, 1994a). Igneous bodies like granites, dolerites, charnockites and diorites intruded into this dominantly gneissic and schistose complex (Ekwueme, 1990a, 1994a).

Methods of Study

Field surveys covering the actual locations of landslides and neighbouring areas were undertaken to study the geologic, geomorphologic and structural features related to the mechanisms of landslides on the different geologic formations found in the study area. Each landslide was described in the field to allow classification according to Cruden and Varnes (1996). Landslide bodies were mapped from crown to toe of rupture, details of the nature and types of materials associated with slope movement, slope angles, depths, width and lengths of landslide, elevation of landslide source and toe were recorded. Samples were collected at different landslide locations and subjected to laboratory analysis. Results from the laboratory analysis were used

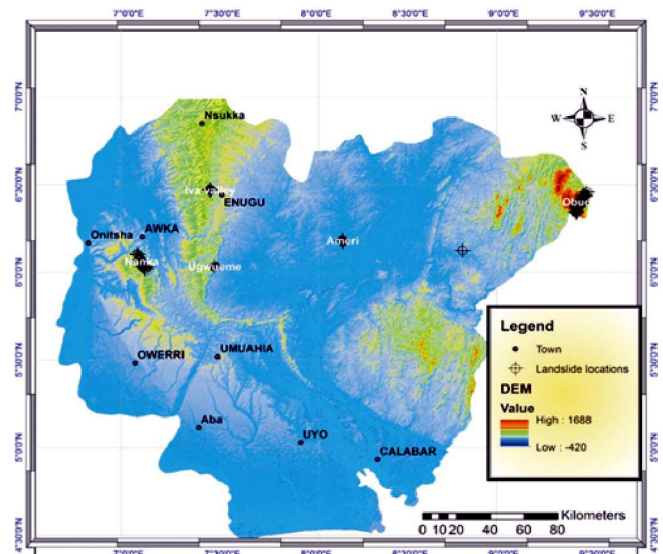


Fig. 1: Digital elevation map of the studied area

to perform two-dimensional slope stability analysis using the general limit equilibrium (GLE) method integrated in the software SLOPE/W of GeoStudio for stability analysis of slopes following the method from Chen et al. (2015). To study the relationship between the slope angle and the sliding mass volume, a two-dimensional homogeneous slope model from Slope/W GeoStudio version (2016) software based on the analysis type of the Morgenstern-Price limit equilibrium method was constructed. The slope material parameters needed for this simulation are material angle of internal friction, cohesion and unit weight (Table 1). In this model, the slope height, slope base length and the slope angle were the parameters used to determine the geometry of the slope. The slope height was fixed at 55m, which was a constant value for all the sixteen (16) simulations. However, the slope angle was adjusted from 60° to 30° by changing the horizontal distance.

Statgraphics_Centurion_16.1.11 version was used to perform the correlation analysis by correlating the measured slope angle in the field to the corresponding width, length, elevation and depth obtained in the field.

Results and Discussion

Relationship Between Landslide Size and Slope Angle

A total of sixteen (16) simulations were performed, four for each of the geologic settings. Four simulations were done for Ameri/Enyigba, the slope height, shear strength parameters were fixed while the slope angle was varied. The four representative results for slope angle were 30°, 40°, 50° and 60°. The simulations showed

that the potential slip mass volume increases with increase in slope angle (Figures 2 and 3). It seems that such a relationship is associated with the changes of geometry of the critical slope surface because when the slope angle is 60°, the critical slip surface was bigger and more curved, but as the slope angle decreases, the curvature became smaller (planar) and leading to reduction in the volume of probable sliding material. Similar results were obtained from Nanka, Iva Valley and Obudu following the same procedure but the shear strength parameters differed from each other because the geological settings were different. Table 1 shows the shear strength parameters for each geologic setting. Because of limited space, we will only show the results of Ameri/Enyigba (Figure 2) and Iva Valley (Figure 3). This result demonstrated that the geologic material does not necessarily determine the volume of the potential sliding mass since different geologic settings gave similar results. The result of our simulations is in agreement with two-dimensional discrete element numerical simulations by Katz et al. (2014) and suggestion by Igwe (2015a).

Table 1: Slope material strength parameters for simulation

Location	Angle of internal friction (°)	Cohesion (kPa)	Unit weight (kN/m³)
Ameri/Enyigba	32	14	30
Iva Valley	30	9	30
Nanka	35	8	30
Obudu	15	22	30

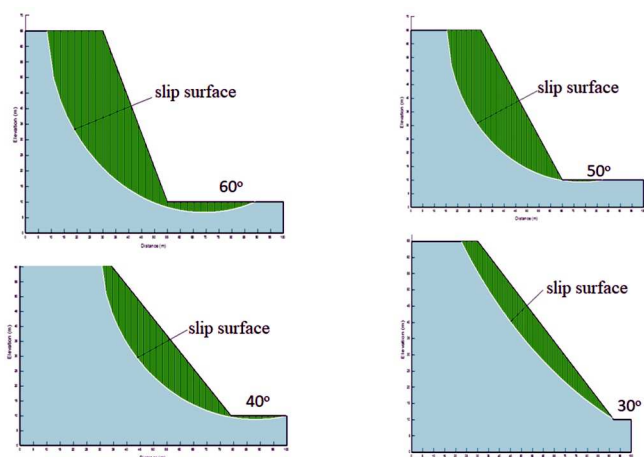


Fig. 2: Ameri/Enyigba simulations

Relationship among Landslide Dimensions and Slope Angle from Scattered Plots

Scattered plots were prepared among landslide dimensions and slope angle in different locations with a linear line drawn along the slope angle axis to show the

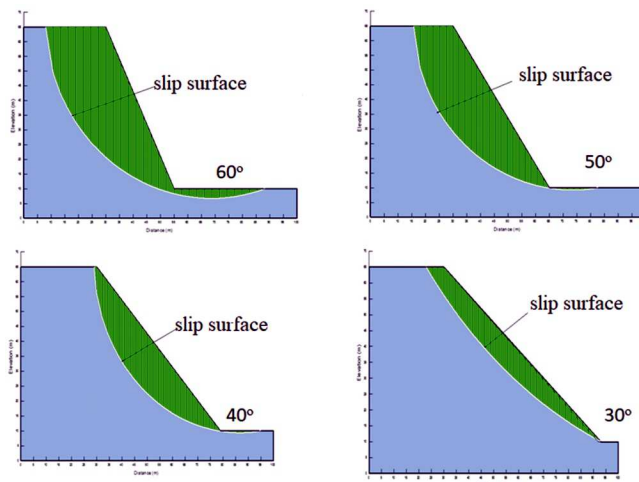


Fig. 3: Iva Valley simulations

dominant critical slope angles in each area. Ameri/Enyigba plot revealed that most of the landslides occurred between slope angles of 40–50°, which represents 73.33% of the total landslides recorded in the area. Nanka plot revealed that most of the landslides occurred between slope angles of 40–50°, which represents 81.67% of the total landslides recorded in the area. Iva valley plot showed that most of the landslides occurred between slope angles of 25–35°, which represents 58.73% of the total landslides recorded in the area. Obudu plot revealed that most of the landslides occurred between slope angles of 40–50°, which represents 86.25% of the total landslides recorded in the area. Correlation between frequency of landslides and slopes angle was also plotted (Fig. 4).

Sidele and Ochiai (2006) reported that slopes above 25° are potentially subject to landslides especially where the soil mantle is not well bound to the underlying rocks while above 35°, most slopes are subject to any type of landslide.

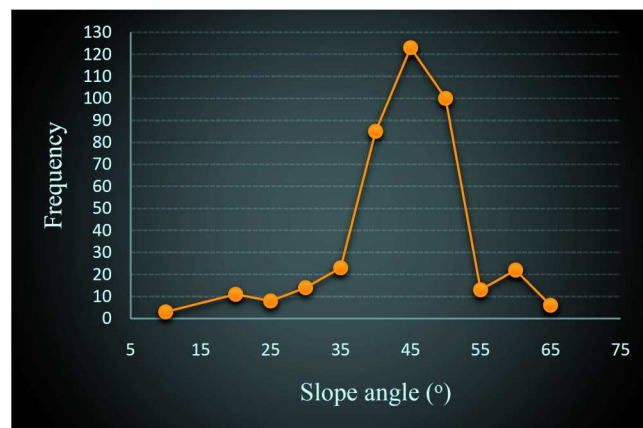


Fig. 4: Correlation between frequency of landslide and slope angles in southeast Nigeria

Statistical Analysis of Landslide Dimensions and Slope Angle

Results of the Pearson correlations for Obudu revealed that elevation had high significant relationship with slope angle and length. Slope angle had high significant relationship with elevation, length and width. Length had high significant relationship with elevation, slope angle, width and depth. Width had high significant relationship with slope angle, length and depth while depth had high significant relationship with length and width. This implies that the landslide parameters in this area are significantly related and this agrees with the results of the limit equilibrium simulations obtained in the area.

In Nanka (Table 2), the Pearson correlations showed that elevation had high significant relationship with slope angle, length and width. Slope angle had high significant relationship with elevation, length and width. Length and width had high significant relationship with all the parameters, while depth had high significant relationship with length and width. The correlation analysis in Nanka area infers that the landslide parameters in this area are significantly related and this agrees with the results of the limit equilibrium simulations obtained in the area.

In Iva Valley, the Pearson correlations revealed that the landslide measurements are significantly related and supports the results of the limit equilibrium simulations. Ameri and Enyigba had minimal significant linear relationship among the landslide measurements due to the nature of the terrain.

Table 2: Nanka Correlations

	Elevation (m)	Slope angle (°)	Length (m)	Width (m)	Depth (m)
Elevation (m)		0.3008 (120)	0.3843 (120)	0.2129 (120)	0.0534 (120)
		0.0008	0.0000	0.0196	0.5623
Slope angle (°)	0.3008 (120)		0.2732 (120)	0.2890 (120)	0.1252 (120)
		0.0008	0.0025	0.0014	0.1730
Length (m)	0.3843 (120)	0.2732 (120)		0.3232 (120)	0.2370 (120)
		0.0000	0.0025	0.0003	0.0091
Width (m)	0.2129 (120)	0.2890 (120)	0.3232 (120)		0.3568 (120)
		0.0196	0.0014	0.0003	0.0001
Depth (m)	0.0534 (120)	0.1252 (120)	0.2370 (120)	0.3568 (120)	
		0.5623	0.1730	0.0091	0.0001

Correlation
(Sample Size)
P-Value

Conclusions

Two-dimensional limit equilibrium simulations and statistical analysis were used to study the relationship between landslide dimensions and slope angle in four different geologic settings (Ameri/Enyigba, Nanka, Iva Valley and Obudu) in Southeastern Nigeria. The simulations revealed that the potential slip mass volume increases with increase in slope angle in all the different geologic settings. Results demonstrate that the geologic material does not necessarily determine the volume of the potential sliding mass since different geologic settings gave similar results. The results from this research will help in predicting landslides and can form basis for land-use planning in Southeastern Nigeria.

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Variation of Maturity Across a Lateritic Profile: An Example from Ilorin

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Abstract

This study characterizes the three distinct horizons of the lateritic profile of the study location with a view to determining their relative degrees of laterization. The three distinct horizons (the upper crustal, middle gravelly and basal less/non-gravelly) were subjected to series of geotechnical tests. The results of the tests elucidated a useful connection between the parameters determined and the degree of laterization. They suggest that the gravelly middle horizon has attained the highest degree of laterization and maturity relative to the other horizons.

Keywords: Maturity, Laterization, Engineering Performance, Lateritic Profile.

Introduction

Laterite has arguably become the most important construction material of the tropical world. It is widely used as foundation material and for other construction purposes in the subtropical and tropical regions where it is ubiquitous. Notwithstanding its ubiquity, its characteristics vary from location to location, depending on individual and cumulative effects of factors such as parent material, topography, climate, drainage among others. Consequently, the maturity or degree of laterization is also variable, but indicated by properties such as specific gravity, sesquioxide content, dry density, mineralogy and moisture content (Tuncer and Lohnes (1977) cited in Adeyemi and Ogundero (2001)). The location of this study is Ilorin, North central Nigeria, where the typical lateritic profile above the basement complex presents a peculiar distinct three-horizon situation. The uppermost horizon is the crust or hard pan, followed by a middle gravelly horizon and a basal horizon with lesser gravel than the middle horizon or no gravel at all.

Research Methods

The study location is situated along Asa Dam Road, Ilorin and it is essentially defined by Longitude E 004° 32' 52.6" and Latitude 08° 27' 02.4" (Figure 2). It is a remnant of a road cut with well-defined horizons of the lateritic profile. Samples were collected across the profile from the three horizons: uppermost horizon (A), the middle gravelly (B) and the basal horizon with fewer or no gravel (C). The samples were appropriately prepared and subjected standard geotechnical tests. The following tests were carried out on the samples: grain size analyses, Atterberg limits, compaction, density, specific gravity, CBR test and shear strength tests,

according to procedures specified in the British Standard 1377 (BS, 1990).

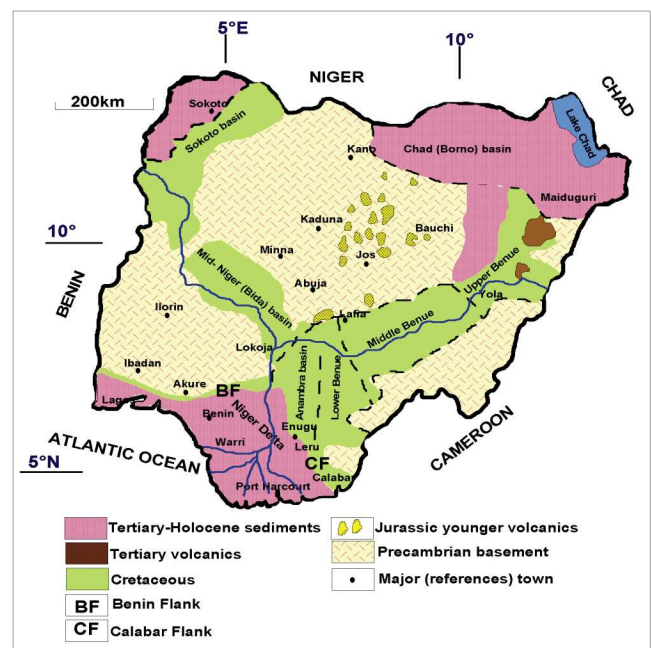


Fig. 1: Geological Map of Nigeria showing the location of the study area Olasehinde *et al.*, 1998).

Results and Discussion

The results of the geotechnical tests carried out on samples collected from three sampling points from the study location are presented in Tables 1 and 2. The samples were collected across the three distinct horizons designated as A (crust), B (Gravelly) and C (Non-/Less-Gravelly).

Moisture Content

As presented in table 1, Horizon C has the highest

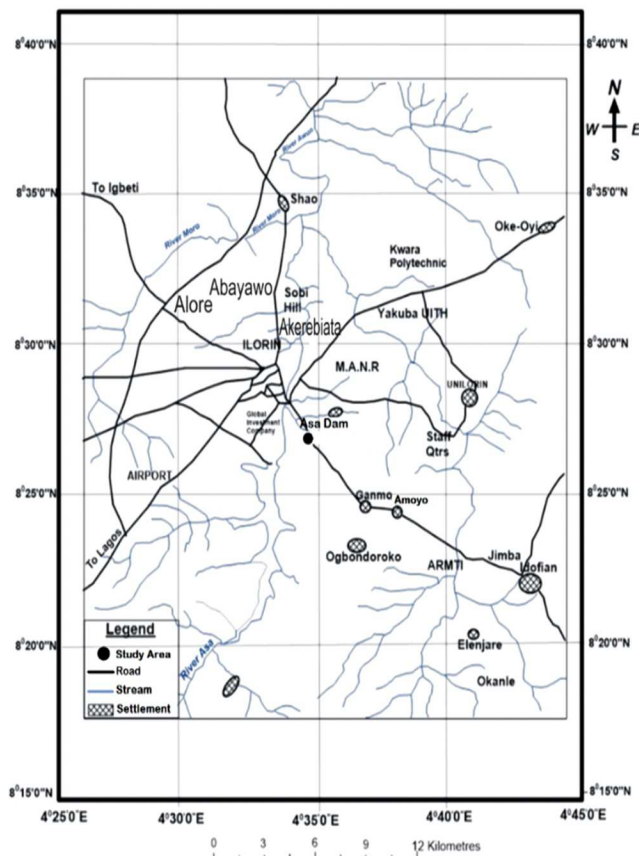


Fig. 2: Map of Ilorin and Environs showing the study area (Modified after (Obaje, 2009).

moisture content of 5.26%, Horizon B with 4.5% is intermediate, while Horizon A with 3.43% has the least moisture content. Moisture content essentially describes the natural propensity of soil for water (Adeyemi and Akinseli,1995), and it also indicates the potential for self-stabilization of a soil (Gidigasu, 1976).Excluding Horizon A that is crustal, the moisture contents of both Horizon B and C can be related to the amount of their clay size particles. This resorts from the high affinity for water by clay. Horizon B with a lower moisture content than Horizon C equally has a lower amount of clay size particles than C. Since the amount of clay is indicative of a degree of laterization, the moisture content also suggests that horizon B is more laterised and more matured than Horizon C.

Specific Gravity

Table 1 presents the results of the specific gravity test. From the table, the mean specific gravity for horizon B (the gravelly horizon) is highest at 2.64, that of horizon A (the crust) is intermediate at 2.603, while that of Horizon (non-/less-gravelly) is the least at 2.523. According to Akroyd (1963), the maturity of lateritic soil can be rated by the specific gravity. Tuncer and Lohnes (1977) similarly discovered a positive linear

Table 1: Results of Moisture Content, Specific Gravity, Grain Size Analysis and Atterberg Limits Tests

Horizon	Sampling Point	Grain Size						Atterberg			
		Moisture content (%)	Specific Gravity	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	SL (%)	PI
A (Cust)	1	3.0	2.60								
	2	3.5	2.61								
	3	3.8	2.60								
	Mean	3.43	2.603								
B (Gravelly)	1	4.1	2.66	17	51	9	23	27.5	22.0	5.2	7.5
	2	4.6	2.64	12	45	14	29	31	18.6	5.3	12.4
	3	4.8	2.62	11	46	16	27	34	20.5	5.7	13.5
	Mean	4.5	2.640	13.33	47.33	13	26.33	30.8	20.36	5.4	11.13
C (Non-/Less-Gravelly)	1	4.9	2.55	0	31	23	46	37	22	6.1	19.5
	2	5.2	2.52	6	30	16	48	42.5	15.5	6.5	27
	3	5.7	2.50	4	29	16	49	46.5	16.6	6.8	29
	Mean	5.26	2.523	3.33	30	18.33	47.66	42	16.53	6.46	25.16

correlation between the degree of laterization and specific gravity. Therefore, Horizon B (gravelly) has attained the highest degree of laterization cum maturity, followed by Horizon A (crust) while Horizon C (less/non gravelly) is least laterised.

Textural Analysis

As deduced from Table 1, Horizon B with a mean gravel content of 13.33% and sand content of 47.33% is more granular than Horizon C, while Horizon C with a clay

Table 2: Results of CBR, Compaction and Shear Strength Tests

Horizon	Sampling Point	CBR				Compaction				Shear Strength Parameters			
		Standard Proctor		Modified Proctor		Standard Proctor		Modified Proctor		Standard proctor		Modified Proctor	
		Unsoaked	Soaked	Unsoaked	Soaked	OMC (%)	MDD (g/cm ³)	OMC (%)	MDD g/cm ³	C (KPa)	Ø (°)	C (KPa)	Ø (°)
A (Cust)	1												
	2												
	3												
	Mean												
B (Gravelly)	1	5	3	6	5	9.0	1.90	8.0	1.97	35	31	40	33
	2	4	3	5	4	13.5	1.73	12.5	1.80	30	28	43	32
	3	4	4	5	4	10.2	1.84	9.5	1.91	55	37	120	43
	Mean	4.33	3.33	5.33	4.33	10.9	1.82	10	1.89	40	32	67.66	36
C Non- /Less- Gravelly)	1	2	2	3	5	14.9	1.62	13.8	1.68	80	23	110	25
	2	2	1	3	1	15.0	1.69	13.0	1.78	60	20	100	30
	3	3	2	3	1	15.2	1.59	14.1	1.61	90	25	125	35
	Mean	2.33	1.66	3.66	1.66	15.03	1.63	13.63	1.69	76.66	22.66	111.66	30

content of 46.66%, conversely contains higher content of clay than Horizon B. According to Gidigas (1976), the amount of clay size particles of laterite is related to its degree of laterization, which is also an indication of the level sesquioxide coating. He stated that, the lower the clay content, the higher the degree of laterization. Consequently, Horizon B, with lesser clay content, is dimmed to have attained a higher degree of laterization than Horizon C.

Consistency Limits

The results of consistency limit tests are shown in table 1 while the plasticity classification is shown in figure 5. Soils of horizon B have a lower mean plasticity (11.13%) than that of Horizon C (25.16%). Relative plasticity of a soil is related to relative amount of clay size particles, which is in turn a reflection of the effectiveness of sesquioxide coating. Consequently, the lower plasticity of the samples from Horizon B further suggests its higher degree of laterization (Madu, 1977). Both the Horizon B and C however, plot has inorganic clay of medium plasticity.

California Bearing Ratio (CB)

Adeyemi (2003) opined that CBR is enhanced by increasing degree of laterization. Expectedly as shown in Table 2, all the values for the modified proctor energy level are higher than those of the standard energy level. Similarly, the values for the unsoaked specimens are also higher than the equivalent values for soaked specimens. Mean CBR values for Horizon B are 4.33 and 3.33 respectively for unsoaked and soaked specimens at the energy of standard proctor, while they are 5.33 and 4.33 for unsoaked and soaked at the energy

of modified proctor. For horizon C, 2.33 and 1.66 for unsoaked and soaked at energy of standard proctor and 3.66 and 1.66 respectively for unsoaked and soaked at the energy of modified proctor. Consequently, at both the standard and the modified proctor energy levels, whether soaked or unsoaked, Horizon B has higher CBR values than the corresponding values for Horizon C. This suggests a higher bearing capacity, higher engineering performance and a higher degree of laterization cum maturity.

Compaction

The results of the compaction tests are shown in table 2 and figures 3 and 4. Horizon B which is the gravelly horizon, has a mean maximum dry density (MDD) of 1.82 g/cm³ and a corresponding optimum moisture content (OMC) of 10.9% when compacted at the energy of standard proctor. It also has a maximum dry density (MDD) of 1.89 g/cm³ and optimum moisture content (OMC) of 10% when compacted at the energy of modified proctor. For the horizon with no or less gravel, Horizon C, the maximum dry densities (MDD) are 1.63 g/cm³ and 1.69 g/cm³ while the optimum moisture contents (OMC) are 15.03% and 14.1% when compacted at the energies of the standard and modified proctors respectively. Be it at the energy of the standard or modified proctor, Horizon B has better compaction characteristics compared to Horizon C. According to the findings of G/Medhin (2008), who worked on the compaction properties of lateritic soils of Assossa, Ethiopia, maximum dry density (MDD) increases while the optimum moisture content (OMC) reduces with increasing degree of laterization across a profile where the layers are developed over the same parent material. This implies that, Horizon B has attained a higher

degree of laterization and will exhibit a better engineering performance than Horizon C.

Shear Strength

For both the standard and modified proctors, Horizon B shows better mean shear strength parameters than Horizon C (Table 2). The mean cohesions for Horizon B are 40 KPa and 66.67 KPa while that of Horizon C are 76.66 KPa and 111.66KPa for the standard and modified proctors respectively. For the angle of internal friction, it is 32° and 36° for Horizon B and 22.66° and 30° for Horizon C at the energies of standard and modified proctors respectively. The two shear strength parameters relate directly to the relative abundance of clay size and granular particles. The higher the amount of clay size particles, the higher the value of cohesion, while an enhanced quantity of granular materials will result in higher angle of internal friction. According to Baldovin (1976), the higher the degree of laterization, the more favourable the shear strength parameters. Horizon B is thus adjudged to be more mature, having attained a higher degree of laterization.

Conclusion

Lateritic soil from the three distinct horizons, but developed over the same parent rock, have been investigated to determine their relative degrees of laterization. Due to the crustal nature of Horizon A, only its moisture content and specific gravity were determined. However, for Horizon B (gravelly) and C (non/less – gravelly), properties such as shear strength parameters, CBR, compaction parameters, Atterberg limits, grain size distribution as well as the specific gravity and moisture content were determined.

The relatively lower quantity of clay size particle Horizon B suggests an enhanced sesquioxide coating, thus a higher degree of maturity and laterization. This is further affirmed by its relative specific gravity and the other investigated parameters, all of which have been positively enhanced compared to the other horizons. Of the three horizons, Horizon B, based on its characteristic features, will therefore perform better as an engineering material.

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Hydro-Geochemical and Bacteriological Characteristics of Surface and Groundwater in Kakwagom-Irruan, Boki, Cross River State, Nigeria.

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Abstract

Hydro-geochemical and bacteriological characteristics of surface and groundwater in KakwagomIrruan was conducted to compare the hydro-geochemical and bacteriological parameters of both surface and groundwater in the study area, access their quality and potability and finally to ascertain the causes of both surface and groundwater quality variation in the study area. The surface water (streams) and groundwater (wells) were collected (five samples each) in the study area and analyzed for various physiochemical, heavy metals and bacteriological parameters. The methods used for laboratory analysis were 'American Society for Testing and Material (ASTM) 2010 and American Public Health Association (APHA) 2005. The study indicated that the mean values of physiochemical parameters and heavy metals, except pH of both surface and groundwater were within the World Health Organization (WHO) acceptable value. The mean pH of groundwater was 8.85 while that of surface water was 6.20. The result also shows higher mean value of TDS (58.0mg/l), Cl (16.62mg/l), EC (59.66 μ S/cm and Na⁺(25.0mg/l) for groundwater compared to surface water; 16.60mg/l, 9.94mg/l, 34.9 μ S/cm and 24.74 mg/l respectively. However, the mean value of sulphate (5.92mg/l), CO₃(4.32mg) Ca²⁺(4.14mg/l) were higher than that of groundwater which were 1.26mg/l, 2.88mg/l and 1.16mg/l respectively. The mean concentration of heavy metals (Lead, Zinc, Manganese and Cadmium) for surface water was slightly higher than the groundwater; however, all were within the WHO maximum acceptable level. There was also occurrence of bacteriological elements for both surface and groundwater in the study area but surface water in the area had more concentration which is attributed to anthropogenic (human) contributions. Plots of Na⁺ versus Cl⁻ for all the data indicate that most of the samples for both groundwater and surface water show higher concentration of Na⁺ compared to Cl⁻. A plot of TDS versus the weight ratio of Na⁺+K to Na⁺+K+Ca²⁺ (Gibbs Diagram) in the study area indicate contribution from precipitation and rock weathering.

Keywords: Hydro-geochemistry, Bacteriology, Quality variation, Gibbs Diagram.

Introduction

Water is the basic necessity of life, a wonderful gift of nature to mankind and other organisms living on the earth. Potable water is fast becoming a scarce commodity in most part of the world including most part of Nigeria. Water quality is a major prerequisite governing the health of human beings, animals as well as plants (Edet, et al., 2013). Water quality is the physical, chemical and biological characteristics of water. Groundwater contamination is a function of the geologic settings of an area as this determines the quantity and time taken for water that fell as rain to infiltrate through the soil and reach the water table. It is through this infiltration processes that the groundwater gets contaminated as the infiltrated water carries with it some contaminants. The geologic setting of an area also affects the rate at which any contamination will degrade or breakdown. The earth materials may acts as natural filters to screen out some contaminants. The geochemical characteristic of water and bacteriological properties of the geosphere evolve with time due to rock-water interactions. Replacement of dissolve Ca²⁺ by dissolve N^{a+} also occurs due to ion exchange reaction between clay minerals and water.

Hydro-geochemical and bacteriological examination of water samples reveal the quality of water that is suitable for various uses. They also help tounderstand the changes in water quality due to rock-water interaction as well as anthropogenic influence. The quality of surface water in the study area had been affected more; this could be attributed to human activities over the years in the study area and weathering of rocks in addition to atmospheric contribution.

The Study Area

The study area (KakwagomIrruan) is a village located in Irruan in Western part of Boki, CrossRiver State. It lies between Latitudes 6^o27'.184''N and 6^o.27'.918''N and Longitudes 8^o56'.203''E and 8^o56'.823''N (Fig. 2). It is located within the subequatorial climate region of Nigeria with a total annual rainfall of more than 300 – 400cm (Iloje, 1991). The temperature ranges from 23-33^oC. the study area experience two seasons, these are the raining season which last from April to September with a peak in June and July while the dry season lasts from October to March. The elevations of the area range from 90m to 160m above the sea level. The relief is characterized by undulating surfaces running at undefined direction and demarcating the very lowlands

areas from moderate relief landmark. The area is drain by major streams like Ajuju, Owoetuent, Jacob stream etc. The streams and a few existing wells and boreholes form the major water sources for domestic, agricultural and other important uses in the study area.

Kakwgom-Irruan which is part of the Mukuru sheet 305 is located at the foot of Bamenda Massif of the Cameroon Volcanic Line (CVL), in it extension to Boki L.G.A. of Cross River State. The study area is underlain by the Precambrian rocks of the Basement complex of the Southern-Eastern Nigeria and concealed in most places by a variably thick overburden. The crystalline basement of the Cameroon Volcanic Line (CVL) forms part of the mobile belt between the West Africa and Congo creators and comprises Pan Africa granitoids (Toteu et al. 1994; Van Schmus et al 2018). They are mostly of metamorphic rocks which have undergone polyphase deformation (Dokubo et al, 2007).

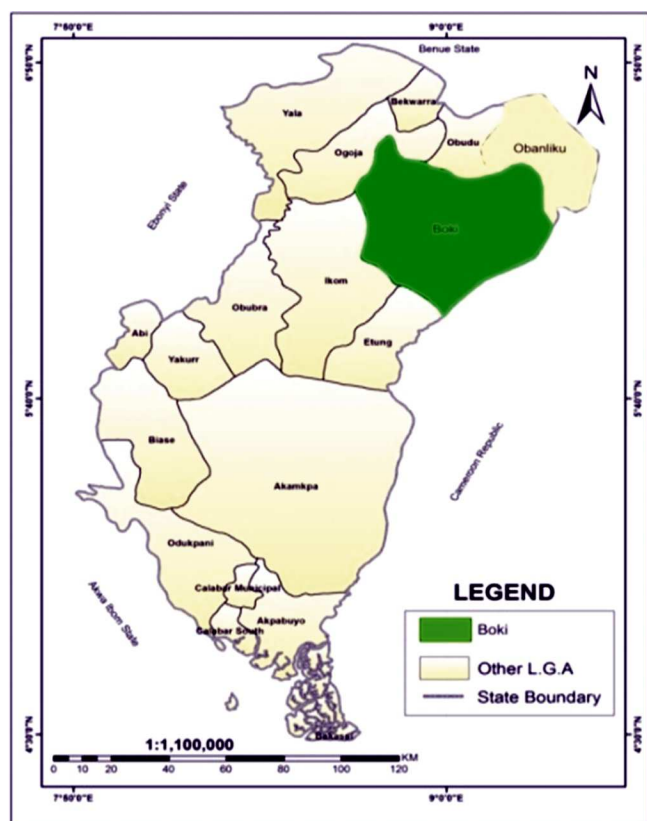


Fig. 1: Map of Cross River State showing Boki LGA.

Materials and Methods

Ten (10) representatives water samples; five (5) each from both sources (stream and wells) were collected from the study area using 1.5L plastic containers. The plastic containers used for the collection of the water

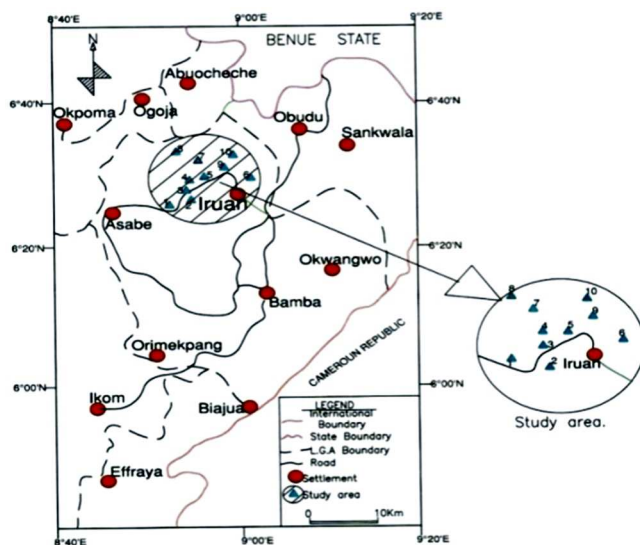


Fig. 2: Map of Boki LGA showing the study area

samples were rinsed with the sample to be collected prior to use. The electrical conductivity, temperature and pH were measured in situ. Field measurements of pH and electrical conductivity (EC) were made using a handheld pH and EC meter while temperature was measured by a thermometer.

The total analysis of the water samples (physicochemical, heavy metals and bacteriological) were carried out in the laboratory using different analytical methods.

The methods used for the analysis were 'American Society for Testing and Material (ASTM) 2010 and American Public Health Association (APHA) 20th Edition 2005.

Results and Discussion

The quality of both surface and groundwater in the study area was investigated and the result shows variation in the quality of water of both sources. The Summary statistics for physiochemical, heavy metals and bacteriological parameters for surface and groundwater in the area is shown on the table below.

pH: Groundwater locations 3, 5 and 7 were within the permissible limit (6.5 – 8.5) of drinking water standards of WHO, while locations 4 and 10 were below the standard, with 5.25 and 5.64 respectively. The pH value for surface water range from 7.30 to 10.13. The mean pH value of 6.20 for groundwater and 8.5 for surface water were within the WHO standard for drinking water.

Table 1: Summary statistics for physiochemical, heavy metals and bacteriological parameters for surface and groundwater in the area

	PARAMETER	GROUNDWATER					SURFACE WATER				
		Min	Max	Mean	Mid	SD	Min	Max	Mean	Mid	SD
1	PH	5.25	6.78	6.20	6.62	1.73	7.3	10.13	8.52	8.51	1.46
2	Temperature, °C	25	25	25	25	0	25	25	25	25	0
3	TDS, mg/l	25	98	55.5	52	28.63	32	95	58	57	23.76
4	Turbidity, NTU	0.4	1.2	0.64	0.5	2.16	2.1	6.2	3.48	3.4	2.12
5	Oil and Grease, mg/l	0.001	0.001	0.001	0.001	0	0.001	0.001	0.001	0.001	0.001
6	Total Hardness	4	10	7.2	8	3.98	12	16	13	12	2.45
7	Chloride (Cl), mg/l	7.1	27.4	16.62	16.2	6.12	8.1	12.1	11.3	10.2	3.63
8	Conductivity, µs/cm	25.4	98.5	59.66	58.3	22.24	27.4	41.7	39.87	36.8	13.28
9	Sulphate, mg/l	1	1.8	1.26	1.2	2.50	4.8	7.1	5.18	5.65	1.83
10	Phosphate, mg/l	0.1	0.1	0.1	0.1	0.04	0.1	0.21	0.13	0.11	0.05
11	Nitrate, Mg/l	0.11	0.3	0.17	0.14	0.31	0.15	0.92	0.53	0.56	0.34
12	CO ₃	2.4	4.8	2.88	2.4	1.26	2.4	4.8	4.0	4.8	1.24
13	TSS	6	14	9.2	8	7.23	8	28	14.33	10	8.52
14	ALKALINITY	4	8	4.8	4	2.11	4	8	6.67	8	2.07
15	Potassium (mg/l)	2.3	8.9	4.48	2.7	2.55	0.7	6.4	3.63	3	2.25
16	Calcium (mg/l)	1	1.4	1.16	1.1	2.30	2.4	8.4	3.67	2.7	2.54
17	Magnesium, (mg/l)	2.65	3.4	3.08	3.11	0.78	4.33	4.65	4.26	4.445	0.58
18	Sodium (mg/l)	3.1	73.1	24.88	17.3	25.06	5.2	67	21.62	14.2	23.38
19	H ₂ S	0	0	0	0	0	0	0	0	0	0
20	Lead (mg/l)	0.01	0.01	0.01	0.01	0.005	0.01	0.02	0.014	0.01	0.01
21	Zinc (mg/l)	0.09	0.85	0.64	0.78	0.45	0.91	1.82	1.18	1.095	0.37
22	Manganese (mg/l)	0	0.013	0.009	0.01	0.14	0.018	0.35	0.15	0.125	0.16
23	Cadmium (mg/l)	0	0.01	0.002	0	0.004	0.001	0.001	0.001	0.001	0.004
24	E-Coli	0	1	0.4	0	0.548	2	4	2.8	3	0.837
25	Total coliform (MPN/100ml)	1.0	1.0	1.0	1.0	0	1.0	20	12.6	12	7.403
26	Feacal coliform (cfu/100ml)	<1.8	4.8	4.8	<1.8	0	1.8	10	6.36	6	3.04

Total Dissolved Solids (TDS): The mean value of TDS in the study area for groundwater is 55.5mg/l while the range is between 25mg/l and 98mg/l while it varied from 32 to 95mg/l with a mean value of 58.0mg/ for surface water of 500mg/l.

Turbidity: The turbidity for groundwater in the study area was below the maximum limit of WHO standard for drinking water. The mean turbidity was 0.64 NTU while the maximum value was 1.2 NTU. The values for surface water range from 2.1 to 6.2 NTU with a mean value of 4.08NTU which is within the WHO limit but the value obtained in location 8 is 6.2 NTU is above the WHO limit of <5.0 NTU. This can be attributed to the addition of surface runoff due to rain fall.

Oil and Grease: The oil content for both water sources in the area were below the maximum acceptable standard of WHO. The content in all the locations were less than 0.001mg/l.

Total Hardness (TH): groundwater value range from 4 to 10 and surface water range from 12 to 16 with a mean

value of 13.60 which are below the WHO maximum limit of 150

Chlorine: It range for groundwater is from 7.10mg/l to 27.4mg/l whereas it is 8.10 to 12.10mg/l for surface water. These values of chloride were all below the WHO acceptable limit of 250mg/l.

Electrical Conductivity: It has a mean value of 59.66 µs/cm for groundwater with a maximum value of 41.70mg/cm in location 9 for surface water. Both sources were below the maximum permissible limit of 250 for WHO standard of drinking water.

Sulphate: The mean content for groundwater was 1.26mg/l and 5.92mg/l for surface water. Both sources locations values were below WHO permissible values of 250mg/l

Phosphate: The mean content for groundwater was 0.1mg/l while surface water have a mean value of 0.13mg/l and a range from 0.10 – 0.21mg/l which were all within the WHO permissible limit.

Nitrate: Its mean value for groundwater was 0.17mg/l and a range of 0.11mg/l to 0.030mg/l but 0.15 – 0.92mg/l with a mean value of 0.61mg for surface water. The value for surface water locations 8 and 9 which were 0.85 and 0.92mg/l respectively were above the WHO maximum permissible value of 0.68mg/l. This could be attributed to surface runoff that washes down excess fertilizer and organic compound from nearby farms.

Carbonate: The range for both water sources for carbonate was from 2.4 to 4.8mg/l with a mean value of 2.88mg/l and 4.0mg/l for groundwater and surface water respectively which are below the maximum permissible WHO limit.

Total Soluble Solute (TSS): The mean value of TSS for groundwater was 9.2mg/l while it was 14.3mg/l for surface water. Both were within the acceptable limit.

Alkalinity: Its values for both sources range from 4 to 8mg/l with a mean value of 4.8mg/l for groundwater and 6.67mg/l for surface water.

Potassium: Its range for groundwater was from 2.3 to 8.9mg/l while it was 0.7 to 6.4mg/l for groundwater all of which were within the acceptable limit

Calcium: Its range for groundwater was 1.0mg to 1.4mg/l with a mean value of 1.16mg/l. The values for surface water were from 8 to 28mg/l with an average of 15.60mg/l. Both sources were below the maximum permissible limit of 200mg/l for WHO standard of drinking water

Magnesium: Its range was 2.65 to 3.40 mg/l for groundwater and 4.33 to 4.65mg/l for surface water. Both sources were below the maximum permissible limit of 150mg/l for WHO standard of drinking water

Sodium: Its range for groundwater was from 3.7 to 73.1mg/l while it was 5.2 to 67.0mg/l for surface water all of which were found to be below the WHO permissible limits of 200mg/l for drinking water.

Hydrogen Sulphide (H₂S): It did not occur in any of the locations as shown in the result.

Lead: Its level range from 0-0.01mg/l for groundwater and 0.01 to 0.02mg/l for surface water these were below the permissible limit of 0.03mg/l.

Zinc: It had a mean value of 0.638mg/l and a range of

0.09 – 0.85mg/l for groundwater and from 0.91 to 1.82mg/l and a mean of 1.25mg/l for surface water which were below the WHO permissible standard for drinking water.

Manganese: The range for groundwater was from 0.01 to 0.13mg/l and 0.018 to 0.35mg/l for surface water which were below the WHO acceptable limit.

Cadmium: Its range from 0 – 0.001mg/l for groundwater and from 0.001 to 0.002mg/l for surface water which were lower than the maximum permissible limit of 0.3mg/l for WHO standard of drinking water.

Bacteriological Quality of Water Samples

E-coli can be used as bio-indicators of aquatic ecosystem dynamics and determinations of their occurrence may help to assess the water quality; (Usharani et al 2010). Presence of E-coli in all the surface water locations makes it unfit for drinking. The range was from 2 to 4 and a mean value of 2.8. For groundwater, locations 3, 4, 5, had 0 while locations 7 and 10 had 1. This makes the groundwater more suitable for drinking. Table 1 shows the summary of bacteriological counts such as total coliform, E-coli and fecal coliform. From the table it was evident that the surface water apart from location 9, were contaminated. It also denotes potential health hazards. The maximum permissible value of total coliform for drinking water is 10/100ml (WHO). The total coliform had a mean value of 12.6 MPN/100ml for surface water and a range of 1 to 20 MPN/100ml with all the location having the same value (1MPN/100ml). Total plate count can indicate the total count of bacteria in water. Too high total bacterial count means that the water is totally infected, and the water has already been polluted by microbes (Aydin, 2007). Drinking water with a total plate count of 100-500 CFU/ml (2-5 CFU/100ml) will harm the health of human beings (Mudryket al., 2003). The fecal coliform for surface water range from 1.8 to 10 CFU/100ml with a mean value of 6.3CFU/100ml. This also indicates that the surface water was unfit for drinking apart from location 9 with a value of 1.8CFU/100ml. Presence of coliforms organisms in water regarded as evidence of fecal contamination as their origin in the intestinal tract of human and other warm blooded animals. This clearly indicates that the bacterial contamination in the surface water is majorly caused by human excreta and domestic waste, which is objectionable for drinking purposes.

Key: The serial numbers 1 to 10 in the two diagrams represent the different locations.

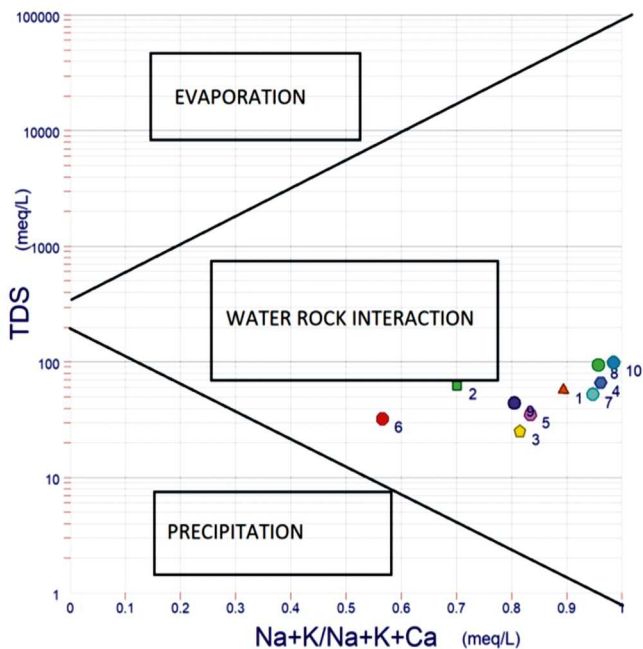


Fig. 3: Plot of TDS vs Na+K/(Na+K+Ca)

Location 1, 2, 6, 8 and 9 are surface water whereas 3, 4, 5, 7 and 10 are groundwater

A plot of TDS versus the weight ratio of $Na^+ + K^+$ to $Na^+ + K^+ + Ca^{2+}$ (Gibbs, 1970) indicates contributions from precipitation and rock weathering (Fig. 3).

Plots of Na^+ versus Cl^- (Scatter diagram) for all the data indicates most of the samples for both sources show higher Na^+ concentrations than Cl^- . When Na^+ is higher compared to Cl^- , it indicates silicate weathering is significant whereas when Cl^- is higher than Na^+ it indicates that some other series contributed Cl^- to the water in the area.

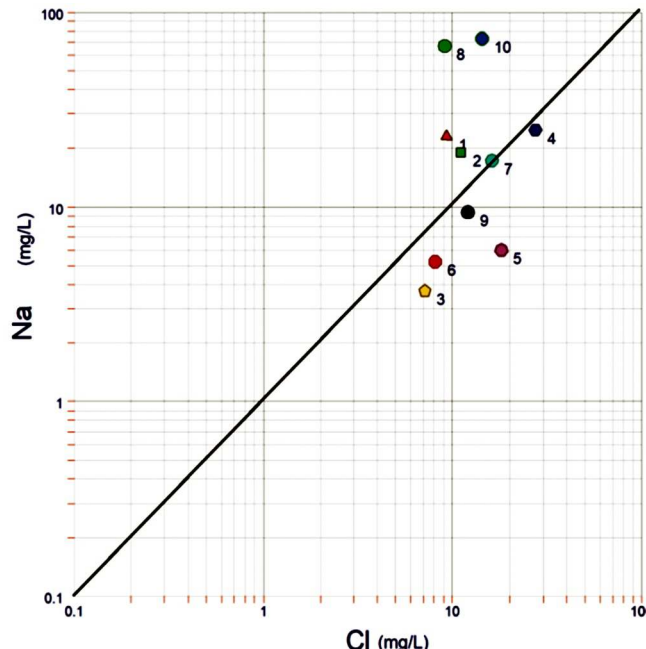


Fig. 4: Plot of Na vs Cl

Summary and Conclusion

From the analysis of samples collected in the study area, it is ascertained that the surface water bodies in the area has been more contaminated than the groundwater. About 80% of the surface water in the study area is not potable. This is because they contain E-coli and fecal coliform. However, both sources (surface and groundwater) could still be used for other purposes (irrigation, washing, bathing, industrial machine cooling etc). The over contamination of the surface water compared to groundwater in the area could be attributed to its exposure to both natural occurrences (weathering, surface runoff, atmosphere etc) and anthropogenic sources of contamination in the area.

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Environmental Impacts Assessment for the Proposed Lafia/Obi In-Seam Exploration Project in Nasarawa State, Nigeria

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Abstract

The Lafia/Obi coal is the only known coking coal deposit in Nigeria and has been extensively explored by geological mapping and geophysical surveys, sinking of two vertical shafts and drilling 138 boreholes, involving the study of over 36,000 metres of drilled core length. Due to the complex geological character of the deposit, an in-seam exploration by two dip roads with section of 12m² and length of 800m each, tagged "south-west dip road" and "west dip road" is suggested as a definite method of upgrading the geological knowledge of the deposit which drilling alone cannot give. Mining, be it surface or underground, by its very nature inevitably has tremendous environmental effect. This paper therefore, assesses the likely impacts of proposed in-seam exploration project on the environment using the matrix method developed by Dr. Luna Leopold and others of United States Geological Survey. Results show that there are positive and negative impacts of proposed in-seam exploration project on the environment. Appropriate mitigation measures to reduce or remediate significant adverse effects on the environment were advanced.

Keywords: Coal deposit, Dip roads, Environment aspects, Environmental impact, Mitigation measures.

Introduction

The Lafia/Obi coal deposit is the only known coking coal deposit in Nigeria. The deposit has been extensively explored, which consisted of accumulating available information, performing geological mapping and geophysical surveys, sinking of two vertical shafts and drilling 138 boreholes, involving the study of over 36,000 metres of drilled core length (Okeudo O. E., 1979). Due to the complex geological character of the deposit, an in-seam exploration by two dip roads tagged "south-west dip road" and "west dip road" is suggested as a definite method of upgrading the geological knowledge of the deposit which drilling alone cannot give (POLSERVICE, 1981).

Mining, be it surface or underground, by its very nature inevitably has tremendous environmental effect. With the increasing global awareness and rapidly expanding environmental demands, the need for mining companies to discharge their duties as responsible corporate citizens has become increasingly necessary (Mallo S. J. *et al*, 2012). Also, the host communities of any mining projects are now demanding that mining operations should, in addition to contributing to wealth, be environmentally friendly and sustainable for the use of future generations. In view of this and other reasons, the need to assess the likely impacts of proposed in-seam exploration project on the environment becomes imperative. This paper, therefore, assesses the likely both negative or positive environmental impact of the proposed dip roads driving project and proffers appropriate mitigation measures to reduce or remediate significant adverse effects of the project.

Project and Environmental Setting

The project would involve driving two dip roads with a section of 12m² and a length of 800m each in the southern part of the deposit. The first dip road runs towards south-west and traverses the overburden with a dip 12^o, followed by a dip of 5 -6^o in line with the dip of the coal seam (POLSERVICE, 1981). Both dip roads would be driven in claystone coal-bearing measures. The South-west dip road will traverse at least 4 major faults which may prove to be water-bearing. The general topography of the project area is flat and lies between Obi and Agwatastu villages about 40km southeast of Lafia, in Nasarawa State. The lowest point rises some +200m above sea level. The deposit area is covered by bushes and dense high grass and is partially wooded. The concavities of the area are partly filled with shallow and small water reservoirs, part of the water running off beyond the deposit area. At a distance of 20km north-east of the deposit area, the Dip river is flowing. The climate of the region is so characterized by high humidity, temperature and high rainfall in the rainy season.

Ground Conditions

The deposit is folded and the folds axes run from NE to SW. The coal-bearing measure is the lower coal measures of the Agwu Formation, belonging to lower Turonian (Upper Cretaceous). The lower coal measures of about 230m of total thickness form a complex of claystone sediments, mudstones and limestones. The three economic seams 12, 13 and 20 constitute the 80m. The commercial seam thickness is 0.5 – 1.70m, whereas

the mineable thickness of the seam including the intercalations is 0.7 – 2.5m. The average compressive strength for the formation generally ranges from 50kg/m² to 360kg/m². No presence of methane is expected to a depth of 50m below the ground surface. Below the depth of 50m the presence of small quantities of methane is to be expected. At the depth of about 300m the presence of methane, amounting to 2m³ per one tonne of pure coal is expected. It is noted that seam 12 will be highly liable to spontaneous combustion. The geothermal gradient for the deposit is 33m. Down to the depth 200m, the air temperature in the dip roads will not exceed 35°C. The estimated quantities of inflows to the dip roads are as follows: For South-West dip road – ($Q_{\min} = 1.2 \text{ m}^3/\text{min}$ and $Q_{\max} = 3.6 \text{ m}^3/\text{min}$). The water occurring down to the depth of 50m below the surface is slightly alkaline pH (7.2 – 8.6) and contains some H₂S. At the depth 50 – 200m the water is characterized by a complex chemical composition, with prevalence of Na⁺, HCO₃⁻ and the presence of soluble gases – mainly nitrogen and methane.

Environmental Aspects

The environmental aspects of the proposed project are access road construction, soil clearing and land leveling, workers' temporary accommodation and offices, provision of storage yards for dumping of coal, excavation for starting the drifting of the dip-roads, drilling and blasting, removal of coal and rock, disposal of rock, and transportation of coal, underground and surface water disposal, restoration of topsoil on rebuild areas; and reclamation activities-vegetation, maintenance. The anticipated environmental impacts due to proposed project would include impact on the air quality, water resources, water quality, noise levels, land use, forests, vegetation and wildlife, human settlements, health, infrastructure, support services, and employment. The general characteristics of mechanization for driving the proposed dip roads are shown in Table 1.

Table 1: General characteristics of mechanization for driving the proposed dip roads

S/N	OPERATION	UNIT	EQUIPMENT/MATERIALS
1.	Wining		Blasting materials + Pick hammers
2.	Face Loading		L.H.D Equipment
3.	ROM Transport from face to surface		L.H.D Equipment
4.	ROM Transport to dumping yard		L.H.D equipment
5.	Materials transportation		L.H.D equipment
6.	Support		Steel arches
7.	Electric power installed (KW)	450	
8.	Ventilation		Forced by booster fans
9.	Dewatering		Intermediate by a few stages
10.	Expected monthly face advance (m)	2 x 50	
11.	Rough estimation of employment	100	
12.	Total advance of the drifts m/18month	1600	

Assessment of the Environmental Impact

The environmental impact of the proposed project is assessed using the Leopold matrix method. The Leopold system is an open-cell containing project actions along the horizontal axis and environmental 'characteristics' and 'conditions' along the vertical axis (Anderson J., 1998). The result of environmental impact without control measures of the project during preparation and drifting phases is shown in Table 2. The criteria adopted for quantitative evaluation of impact is presented in Table 3 and the total impact score shown in Table 4.

The impact score without control measures is -2300 indicating that the mitigation measures are crucial to reduce the adverse impact. However, a number of the impacts for this project such as human settlement, infrastructure and support services, and employment are actually positive.

Environmental Protection Measures

The significant adverse impacts requiring mitigation measures are: land use, air pollution control, water and noise pollution.

Mitigation Measures for Air Pollution Control

The dust formations would be eliminated by water spraying at loading, unloading and dumping sites. The emissions of construction equipment should be controlled and tarred haul roads provided. The overburden dumps must be stabilized.

Mitigation Measures for Water Resources

The water requirements of the project would be met by borehole. The mine water pumping is estimated to be 1.7 – 5.2 million litres per day. It would be let off into

Table 2: Environmental impact matrix without control measures

S/N	Environmental Parameters	Importance Value	Impacting actions									Impact score		
			Preparation phase			Drifting phase								
			Land Transformation	Civil Work Construction	Erection of Mechanical & Drifting Equipment	Drifting Operations	Disposal of Liquid Effluents	Dumping of Coal and Rock	Provision of Water Supply, Electricity,	Housing Provision	Transportation		Medical Facilities	
1.	Air quality	100	-1	-1	-1	-2		-1				-1		-700
2.	Water resources	75		-1				-1		-1				-225
3.	Water quality	100				-1	-2	-1	-1					-500
4.	Noise and vibration	75	-1	-1	-1	-2		-1				-1		-525
5.	Land use	150	-2	-1		-1		-1						-750
6.	Forests and vegetation	150	-1											-150
7.	Wildlife	50	-1											-50
8.	Human settlements	75		+1							+1			+150
9.	Health	100				-2							+1	-100
10.	Infrastructure and support services	50		+1					+2		+1	+2		+300
11.	Employment	50	+1	+1		+2			+1					+250
12.	Places of tourist/archaeological importance	25												0
	Total	1000	-625	-225	-175	-700	-275	-425	-25	+75	-125	+200		-2300

+ Sign shows beneficial impact; - Sign shows adverse impact

* The importance value attached to each environmental parameter is based on the priority assigned to various environmental problems.

Table 3: Criteria for Quantitative Evaluation of Impact

Severity criteria	Impact Score
No impact	0
No appreciable impact	1
Significant impact-slight or short-term effect	2
Major impact- occasional irreversible effect	3
High impact- irreversible or long-term effect	4
Permanent impact	5

Table 4: Total Impact Score

Up to -1000	No appreciable impact on environment.
-1000 to -2000	Appreciable but reversible impact, mitigation measure important.
-2000 to -3000	Significant impact mostly reversible after a short period, mitigation measures crucial.
-3000 to -4000	Major impact which is mostly irreversible, site selection to be reconsidered.
-4000 and above	Permanent irreversible impact, alternative site to be considered.

drains.

Mitigation Measures for Water Quality Control

All used water would be purified before return to natural sources. The mine water would be discharged into the drains after settling to reduce suspended solids. All liquids used in machine operation and maintenance must be controlled. The service water must be treated before letting it into the drains.

Mitigation Measures for Land Use

The civil work construction and erection of equipment would be concentrated in small areas away from housing and recreational areas. The top soil and stock piling should be carefully kept in such a manner that it may be reused when re-establishing the original state of the environment after end of construction. All areas around engineering constructions should be replanted.

The former pastures should be re-established and where forest has been removed or damaged should be reforested, and agricultural areas should be re-cultivated.

Mitigation Measures for Noise, Methane, and Water Influx Control

The major impact would be on the health of mine worker who will be exposed to noise, methane and dust pollution. Continuous control of the noise level should be targeted. Noise absorbent paddings should be used in fixed plant installations and silencers/mufflers in LHD equipment. The mine workers should ensure that they make use of ear-muffs and reduce their time of noise exposure during the shift. Since there is possibility of unexpectedly encountering significant quantities of water in the road face, a probe hole should be drilled ahead of the face. In case of appearance of water inflow under pressure from the probe hole, additional holes should be made in the face for maximum drawdown of

the pressurized water level in the strata and further face advancing calls for caution. Immediately, after blasting all fans should be switched on to purge the face of gas and to dilute any outburst quickly. Automatic and manual monitoring of both combustible and poisonous gases must be carried out. Provisions for limiting ignition sources and for automatic shutdown of drifting machines must be ensured. All machines in use at the face should be flame-proofed to coal mine standards. All diesel engines must have catalytic, cooled exhausts and all electrical switches must be in flameproof enclosures.

Environmental Impact after Mitigation Measures

The environmental impact after implementation of mitigation measures is presented in Table 5. The impact score reduces to -875 after mitigation compared to -2300 without implementation of control measures. The project can be implemented without significant adverse impact.

Table 1: Summary stati

S/N	Environmental Parameters	Importance value	Impacting Actions										Impact Score			
			Preparation Phase					Drifting Phase								
			Land Transformation	Civil work construction	Erection of mechanical & drifting equipment	Drifting Operations	Disposal of liquid effluents	Dumping of coal and rock	Provision of water supply, Electricity etc.	Housing provision	Transportation	Medical facilities				
1.	Air quality	100	-1	-1				-1								-300
2.	Water resources	75					-1		-1							-150
3.	Water quality	100						-1	-1							-200
4.	Noise and vibration	75	-1	-1	-1			-1				-1				-375
5.	Land use	150	-1	-1				-1								-450
6.	Forests and vegetation	150	-1													-150
7.	Wildlife	50	-1													-50
8.	Human settlements	75		+1							+1					+150
9.	Health	100											+1			+100
10.	Infrastructure and support services	50		+1						+2		+1	+2			+300
11.	Employment	50	+1	+1			+2			+1						+250
12.	Places of tourist/archaeological importance	25														0
	Total	1000	-475	-150	-75	+100	-75	-425	-25	+75	-25	+200	-875			

+ Sign shows beneficial impact; -Sign shows adverse impact

* The importance value attached to each environmental parameter is based on the priority assigned to various environmental problems.

Conclusion

Mining, be it surface or underground, by its very nature inevitably has tremendous environmental effect (Mallo S. J. *et al*, 2012). Environmental issues in mining operations should be given due consideration because it makes sense for employers, and because of ever-expanding environmental law and regulation with the threat of severe penalties those who, wittingly or

otherwise, break the rules. In view of this, an environmental impact assessment for the proposed Lafia/Obi in-seam exploration project was carried out. Results show that there are positive and negative impacts of proposed in-seam exploration project on the environment. And appropriate mitigation measures to reduce or remediate significant adverse effects on the environment were advanced.

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Stability of Waste Rock Dumps and the Proposed Earth Dams in Enyigba Mining Province, Southeastern Nigeria

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Abstract

An author had earlier in 2014 suggested the reuse of waste rocks from Enyigba mines as fills and embankments. This paper evaluates the suitability of these waste rocks as embankment materials in terms of structure stability. To meet the set objective, six disturbed waste rock samples were collected from various locations within the mining province and tested for their geotechnical properties according to the American Society for Testing and Materials (ASTM) relevant standards. Laboratory results were analyzed and interpreted using numerical simulations. Results revealed that the rocks are competent plastic materials with moderate shear strength, which agrees with the findings of earlier researchers. The simulation results exposed the unsafe conditions of the indiscriminate waste rock dumps present in the fields, with factors of safety ranging between 1.25 – 1.73. These values were possibly influenced by high slopes which ranged from 11 – 15 m. Evidence has shown that low to moderately high embankments of 10m constructed with these materials may possess factor of safety as high as 3.8 and 1.76, signifying stable conditions at both the upstream and downstream respectively. Nevertheless, it was observed that the factor of safety may drop to as low as 1.0 at the upstream during fast drawdown scenarios, indicating critical stability to instability. This condition suggests that modification is required on the proposed embankments. This study recommends the stabilization of proposed earth embankments by implanting drains or impervious cores, possibly high plastic clay in order to improve the stability of the earth engineering structures.

Keywords: Earth embankment; Factor of safety; Slope failure; Stabilization; Waste rock dumps.

Introduction

A drive around the Enyigba mining province reveals several tons of indiscriminately dumped waste rocks in numerous locations within the ecosystem (Fig. 1). The negative impacts of mine waste such as dump slope failure, acid mine drainage and pollution have been reported by previous authors (Igwe and Chukwu 2019; Shettima et al. 2016). Igwe and Chukwu (2018) studied the stability of dumps within the mining province and affirmed the critical state of stability in the fields.



Fig. 1: Indiscriminate waste dumps in Enyigba

Failure of these mine waste dumps may cause unquantifiable environmental, social, health and economic problems in the region due to poor quality of the wastes, which presumably may contain harmful heavy and radioactive elements. Following the conviction of Iwuanyanwu (2014), the present authors wish to suggest the recycling and reuse of these mine wastes as embankment materials for tailing storage facilities. Therefore, this study aims to determine the suitability of the waste rocks as fill-materials in terms of the stability of structures constructed with them.

Enyigba mining province is located 14km from Abakiliki, the Ebonyi State capital, and it is bounded by Latitudes 008° 05' E-008° 10' E and Longitudes 006° 05' N-006° 15' N. Geologically, the territory is part of the Abakiliki anticlinorium, a major geological structure that marks the southeast of the Benue Trough. The area is underlain by the Abakiliki Shale of the Asu-River Group, deposited during the Albian age as one of the oldest Cretaceous sediments in the basin. The metamorphosed and highly fractured shale played host to the Pb-Zn mineralization and made up a large part of the studied mine waste rocks (Table 1).

The Abakiliki Shale, with an estimated thickness of 1.5km is often made up of black shale and poorly-bedded sandy limestone, which sometimes are bleached to pale grey with mottles of red, yellow, pink and blue through weathering and ferruginization (Igwe and

Table 1: General characteristics of mechanization for driving the proposed dip roads

Mine	S/N	Location		Rock type	Soil lithology	Slope geometry			FOS
		Latitude	Longitude			Height	Width	Angle	
Enyigba	ENY 01	06°11'37.1"	008°08'22.4"	Shale	Clay	15	16	49	1.38
	ENY 02	06°11'23.0"	008°08'24.5"		Clayey silt	14	16	46	1.35
	ENY 03	06°08'01.1"	008°09'45.7"		Clay	15	20	43	1.41
	ENY 04	06°11'03.1"	008°08'30.6"		Clay	15	15	44	1.25
	ENY 05	06°11'04.7"	008°08'31.0"		Silty clay	13	18	40	1.21
	ENY 06	06°11'23.5"	008°08'24.7"		Clay	9	13	38	1.73

Chukwu, 2019) as shown in Fig. 1.

Methodology

Detailed fieldwork was carried out in December 2019 using appropriate field mapping instruments such as hand-held Global Positioning System (GPS) device, compass, geologic hammer, sample bags, measuring and masking tapes. The fieldwork entailed description of waste lithology, waste dump geometry measurements, waste rock sampling and GPS location logging as seen in Table 1. Waste rocks collected for geotechnical analysis were transported within 48 hours to the National Steel Raw Materials Exploration Agency laboratory, Kaduna for analysis. All tests were performed according to the applicable American

Society for Testing and Materials (ASTM) procedures. The laboratory results were analyzed using numerical simulations carried out with the GeoStudio® 2012 software suite developed by Geo-Slope International Limited.

Results and discussions

Waste rock geotechnical measurement results showed that the waste rocks were predominantly shale-cum-clays, with fines, sand and gravel contents ranging between 35 – 81, 16 – 34 and 3 – 31% respectively (Table 2). High clay content had been reported to have severe implication on the stability of slopes depending on the amount and type of clay (Igwe and Chukwu, 2019).

Table 2: Waste rocks' index and geo-mechanical characteristics

Sample No.	Gravel (%)	Sand (%)	Fines (%)	LL (%)	PL (%)	PI (%)	γ (KN/m ³)	C (kPa)	ϕ (°)	MDD (g/cm ³)	K (m/s)
ENY 01	17	25	58	35	15	20	16.25	30	17	1.88	1.59×10^{-6}
ENY 02	31	34	35	43	28	15	16.88	18	24	2.07	5.28×10^{-7}
ENY 03	24	16	60	59	34	25	15.65	20	22	2.13	2.63×10^{-6}
ENY 04	3	25	72	38	24	14	14.80	21	20	2.16	9.59×10^{-8}
ENY 05	21	27	52	60	33	27	15.30	20	13	2.14	2.16×10^{-8}
ENY 06	-	19	81	43	32	11	14.95	19	20	1.93	3.07×10^{-7}

Plastic inorganic soils like these have been noted to cause landslides even under moderate rainfall due to reduction in shear resistance with increase in pore water pressure, which is often orchestrated by poor material permeability (Igwe and Chukwu, 2019). The consistency limits suggested the waste rocks as problematic plastic materials with liquid limit and plasticity index ranging between 35 – 60 and 11 – 27% respectively (Table 2). The waste permeability ranged from 2.16×10^{-8} to 2.63×10^{-6} m/s, suggesting poor permeability of the materials. However, the material's maximum dry density range of 1.88 – 2.16 g/cm³ qualifies the wastes as competent construction materials (Igwe and Chukwu, 2019). This agrees with the shear strength results which registered cohesion and angles of

internal friction of 18 – 30 kPa and 13 - 24° respectively, signifying moderate shear strength of the materials (Table 2).

Waste Dump Present Field Condition

The numerical simulations of the field and laboratory results suggested that unsafe conditions presently exist in most of the mine waste dumps in the fields (Fig. 2). The factor of safety (FOS) computed for the waste rock dumps ranged from 1.21 to 1.73 (Table 1). These values ranged from poor to good stability. Slope height may be the major factor influencing the safety of the dumps (Igwe and Chukwu, 2019). Consequently, most of the high dumps need slope reduction to attain good stability.

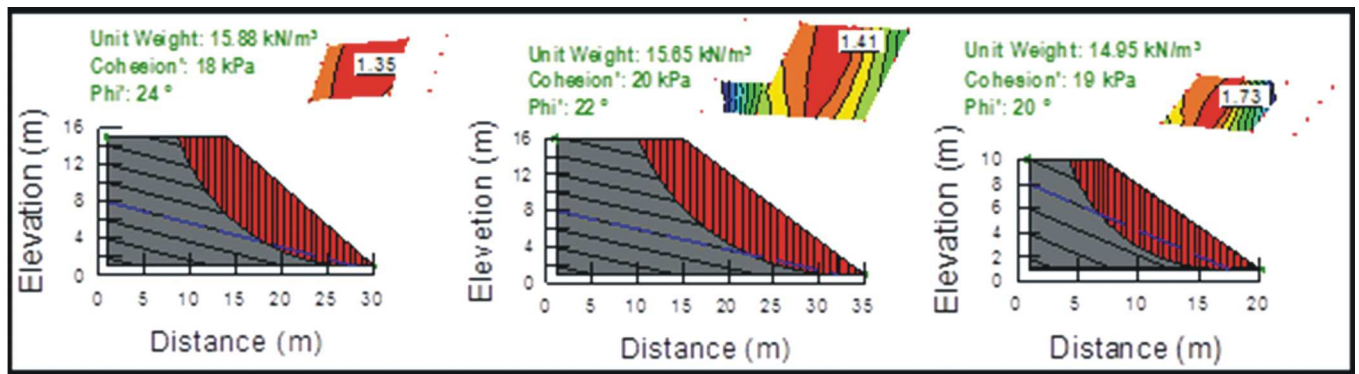


Fig. 2: Stability models of waste dumps' present field conditions

Stability of the Proposed Earth Embankments

Seepage through the proposed earth structures, which is expected to be highest at the center of the embankments, could be in the range of 0.0123 m/day at full operational capacity (Fig. 3a). This value may only have a little impact on the stability of a moderately low embankment of 10m. Stability analysis on the structure revealed that the upstream and downstream FOS could be above 3.80 and 1.75 respectively (Fig. 3b and c), signifying good stability above the recommended value of 1.5. However, there could be a drop in the upstream FOS during a drawdown of the pond (Fig. 4a-d) while the downstream FOS continues to increase (Fig. 4e-h). The downstream situation remains the same with continuous increase in FOS as excess pore water pressure dissipates from the embankment during both fast and slow drawdowns (Fig. 5a).

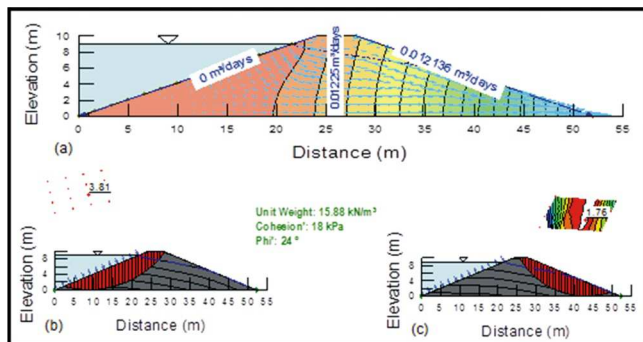


Fig. 3: Seepage coupled stability analysis of proposed earth embankment

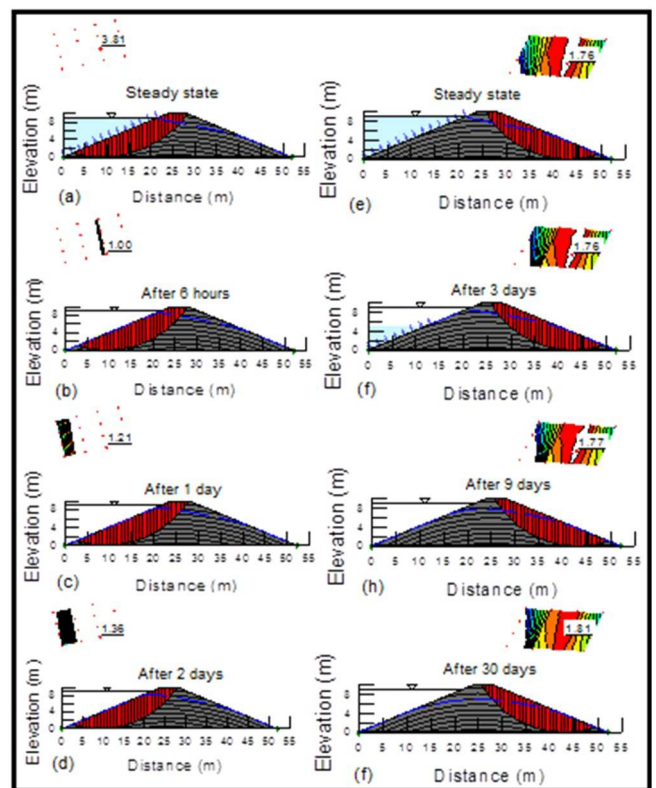


Fig. 4: Change in factors of safety during conditions

However, the drop in FOS at the upstream is likely to be more evident during a fast drawdown, and may drop to as low as 1.0, symptomatic of instability to critical stability of the proposed embankments during sudden

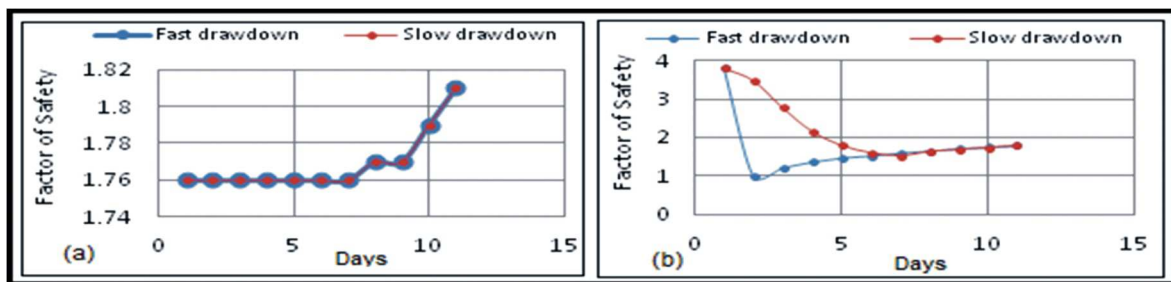


Fig. 5: Effects of drawdowns on the proposed earthdams

draw downs of the reservoirs (Fig. 5b). This drop in upstream face FOS during a sudden drawdown may be the cause of many reported river bank and Levee failures in Nigeria. Therefore, the earth structures may require modification, possibly an implant of impermeable cores or drains to improve the stability of the dams.

Conclusion

This study analyzed the suitability of waste rocks in a Nigerian mine as earth dam materials. First, it was observed that the waste dumps' present field conditions

are unsafe and liable to slope failures. Laboratory results showed that the waste rocks are competent rocks, with moderate shear strength for engineering works. Numerical simulation revealed good stability for low to moderately high earth structures constructed with the waste rocks. However, result also proved that slope failure of the proposed structures is a possibility, particularly at the upstream during rapid drawdown scenarios. Therefore, it is recommended that impermeable cores, probably of highly plastic clay materials, be implanted to improve the stability of the proposed earth embankments.

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Effect of Geochemical Composition of Lateritic Soils on their Geotechnical Properties

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Abstract

Although most laterites are considered suitable for road and pavement constructions, recent investigations have attributed the frequent failure of these engineering structures to the nature of the lateritic soils that was used. The variation in the engineering properties of the lateritic soils could be as a result of their sources, since the laterites are derived from different parent rocks. To verify this assertion, it becomes imperative to assess the contribution of geochemical composition of lateritic soils on their geotechnical properties. Therefore, soil samples collected from freshly exposed lateritic deposits were subjected to X-ray diffraction and X-ray fluorescence analysis in order to determine their geochemical compositions. The soil samples were further subjected to geotechnical tests following standard procedures in order to determine their particle size distribution, consistency limits, compaction, and shear strength properties. The results were then analysed using statistical correlation analysis in order to find the relationships that exists between the soils geotechnical properties and their geochemical compositions. It was observed that silica-aluminium-iron oxide ratio (SAIR), Al₂O₃, Fe₂O₃, and SiO₂ content of the lateritic soils contributed significantly to their geotechnical properties. Thus, there is need for geochemical investigations prior to sourcing of laterites as road and pavement construction aggregates.

Keywords: Laterites; iron oxide; aluminium oxide; geotechnical properties; geochemical compositions

Introduction

Laterites are extensively used as road and pavement construction materials in most tropical regions such as southeastern Nigeria. The geotechnical properties of this soil type in most cases have proved to be a good construction material both for sub-grade and sub-base purposes (Alabo and Pandey, 1987; Northmore et al., 1992). However, investigations have revealed that most of the roads constructed within this region are easily deteriorated over a short period of time; hence increasing the cost of maintenance of these infrastructures. The reason for this road failures have been attributed to the geotechnical properties of the laterites used for the construction (Aginam et al., 2015); hence control measures have been proposed based on such findings. But these control measures which are solely based on geotechnical parameters have not been totally effective in curbing the rate of road and pavement failures within this region. The reason for this observed partial effectiveness of the control measures is because most of the construction companies only carry out quality check on the geotechnical properties of the lateritic soils, while ignoring the contributions of the geochemical composition of the soils (Adeyemi and Wahab, 2008; Bayewu, 2012). One of the major reasons while the geochemical assessment of lateritic soils is ignored prior to use in engineering construction purposes could be because laterites have been loosely

attributed to all reddish coloured soils found within the tropical region; hence the negligence in acknowledging the variations that exists in this soil type.

Variations which exist in residual soils are dependent on the chemical composition of the parent materials and the prevailing climatic conditions which gave rise to them. Geological materials (rocks) which are rich in mafic minerals usually give rise to laterites with high iron and aluminium oxide contents under intensive tropical weathering, while silicic rich rocks under the same weathering conditions gives rise to laterites with relatively high percentage of quartz compared to their iron and aluminium oxides content (Northmore et al., 1992). Schellmann, (1986) has revealed that as weathering of the parent rocks intensifies, the ratio of aluminium to iron oxide content in the resultant lateritic soils decreases, thus weathering is one of the major factors contributing significantly to the nature of residual soils.

Evidence from existing research has shown that the chemical and mineralogical variations in these residual soils have significant effect on their index geotechnical properties such as their particle size distribution (PSD) and consistency limits (Schellmann, 2018). While iron oxide content appears to have a significant effect on the PSD of residual soils, Ismaiel (2006) revealed that the plasticity of their fines is dependent on the type and

amount of clay minerals in them. Consequently, PSD and plasticity of residual soils have significant effect on their engineering properties such as compaction and shear strength. High plastic soils are also associated with high volume change (expansion and shrinkage) which often results in cracking of roads and pavements (Emeh and Igwe, 2016).

Nonetheless, because of the availability of laterites in South-eastern Nigeria, and its cost effectiveness in roads and pavements construction projects, it will continue to serve as the major aggregate material for base and sub-grade. Therefore, in order to ensure that quality and durable laterites are being sourced for as an aggregate material in road and pavement construction purposes, it becomes imperative to characterize these lateritic deposits using the correlation that exists between their geochemical compositions and geotechnical properties. The result will help in quick appraisal of lateritic aggregates before utilizing them as a road and pavement construction material.

Methods of Study

The study area lies within Longitudes 6° 33' 4.0" - 7° 59' 24.4"E and Latitudes 7° 0' 34.3"-5° 30' 49.5"N in southeastern part of Nigeria (Fig. 1). It is underlain by cretaceous to tertiary sediments within the Southern Benue Trough such as Agwu, Nkporo, Mamu, Ajali, Nsukka, Imo, and Ameki Formations. These

sedimentary rock Formations which underlie this area are composed of geological materials with varying textural properties and geochemical compositions that have been described by several authors like Rayment, (1965), Nwajide, (1979, 1990), Arua and Rao, (1987), and Ibe and Akaolisa,(2010). However, intensive tropical weathering has affected most of the rock Formations within the study area, thus resulting in formation of dark-red coloured soils which are generally referred to as laterites or oxisols that underlies most of the studied sites.

Field Investigations and Sampling

Field investigations were carried out to determine the degree of deterioration of roads within the study area. This was done by noting the amount of potholes and cracked section of the road within a given distance. The soil type was physically ascertained by noting the colour and texture of the underlying soil, while the underlying geology was determined by examining the outcropping sediments in each area. 16 different areas were investigated, and disturbed soil samples were collected in duplicate in each of these areas. The sampling locations were in areas which are currently serving as borrow pits for construction companies. About 5kg of freshly exposed soils by excavators were collected using a hand trowel, and was bagged in properly labelled cellophane bags, pending further laboratory tests and analysis.

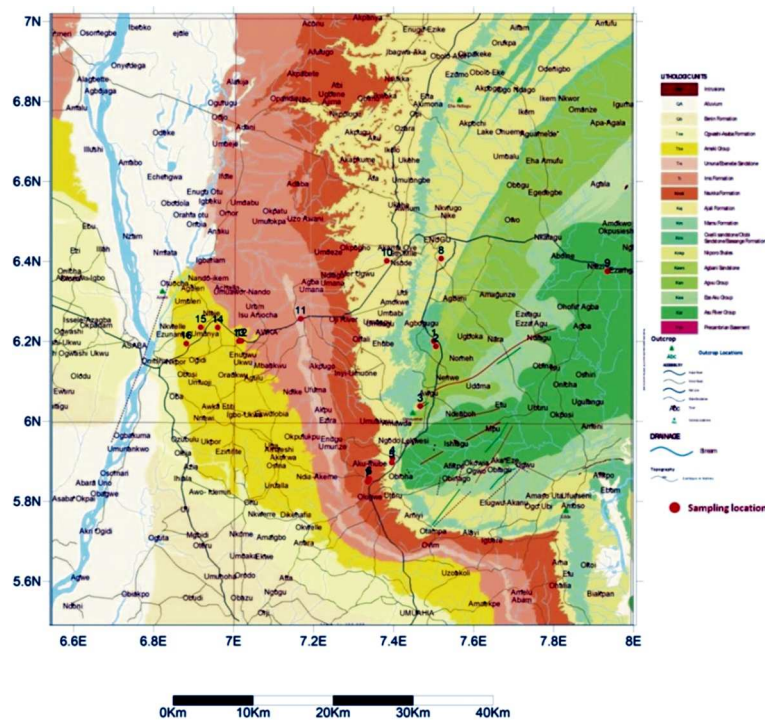


Fig. 1: Geologic map of the study area

Soils Geotechnical Analysis

The air-dried natural soil samples were subjected to sieve analysis using the American society for testing and materials (ASTM) sieve mesh numbers 5, 10, 18, 40, 60, 100 and 230 stacked on an electrically controlled sieve shaker, which was allowed to vibrate for about 25-30 minutes. Particles of air-dried natural soil samples passing ASTM sieve number 40 were used to determine the soils liquid and plastic limit; hence its plasticity index following, ASTM D2487 (2011) standard procedures. The natural soil samples were further subjected to proctor compaction tests following BS 1377 (1975) standard in order to determine their compaction properties. To determine the shear strength properties of the soils, the soil samples were subjected to direct shear test following ASTM D3080-04 (2004) standard test method.

Soils Geochemical Analysis

In order to determine the mineralogical and chemical composition of the soil samples, air-dried natural soil samples which was pulverized to pass the 45 μ m sieve size, were subjected to X-ray diffraction (XRD) and X-ray fluorescence (XRF) analysis, respectively. The XRD and XRF analysis were achieved with the aid of x-ray diffractometer (Schimadzu-6000 model) and Thermo-scientific Advant'x (1200 model), which was set to run at recommended standard operating conditions. Thereafter, the identification and percentage composition of the minerals and oxides were automatically done by the mineral cards of the software preinstalled in the individual machines (X-ray diffractometer and Spectrometer).

Statistical Analysis

All tests were carried out in triplicate, and the average results were used for the statistical correlation analysis of the geochemical and geotechnical data obtained, with the aid of GenStat discovery (Edition 3), a statistics analytical package.

Results and Discussion

Mineralogy and Chemical Composition of Lateritic Soils and the Relationship Between Them

The results of the mineralogical and chemical composition of the lateritic soil samples were tabulated in Table 1. The diffraction pattern of most of the soil samples revealed that the soils were predominantly

composed of quartz, kaolinite, and haematite. However, gibbsite, goethite, and montmorillonite were occasionally found in some of the lateritic soils, especially in soils derived from shale Formations or Formations that were relatively rich in Al_2O_3 , such as Asu-river, Nkporo, Awgu, and Nsukka Formations. The chemical composition of these lateritic soils was predominantly SiO_2 , Al_2O_3 , and Fe_2O_3 . Other oxides such as TiO, CaO, MgO, Na_2O , K_2O , and MnO were also found in very small quantities.

It was observed that quartz and kaolinite content of the soils were strongly correlated with their SiO_2 and Al_2O_3 composition, with correlation coefficient (R) of 0.92 and 0.93, respectively, while haematite was moderately correlated with Fe_2O_3 composition with R value of 0.6.

The silicate-aluminium-iron oxide ratio (SAIR), which is the ratio between SiO_2 content of the soil and the sum of its Al_2O_3 and Fe_2O_3 content, reflected the mineralogical composition of the underlying geology where the soils were derived from. SAIR values of lateritic soils derived from Asu-river, Mamu, Nkporo, Agwu, and Nsukka Formations which are mainly characterized by shales, mudstones, and silt stones were found to be relatively low, with an average value of 1.3. Soils derived from Ameki group has a relatively moderate SAIR with an average value of 2.4, while those derived from Ajali and Ebenebe sandstones has a relatively high SAIR with an average value of 5.1 (Table 1).

Relationship Between Some Index Properties and Geochemical Composition of Lateritic Soils

The lateritic soils were classified according to the universal soil classification system (USCS) using their particle size distribution (PSD), coefficient of curvature (C_c), and plasticity index (PI). They ranged from silty gravels (GM) to inorganic clays of high plasticity (CH) (Table 2). The Lateritic soils were grouped into six according to the similarity in their PSD (Fig. 2). All the lateritic soils derived from Formations which are predominantly composed of shales and mudstones (Agwu, Nkporo, Asu-River, Mamu, and Nsukka) were poorly graded with coefficient of curvature ranging from 0.34-0.87. This poor gradation was as a result of high gravel content and /or high fines content, thereby resulting in little amount of sand sized fraction within the soil matrix. In general, they can be classified to be gap-graded. Most of the lateritic soils that were derived from Formations that are predominantly composed of

sandstones (Ameki, Ajali, and Ebenebe) were well graded with coefficient of curvature that ranges from 1.01-2.81 (Table 2).

There were no significant direct correlation between the gravel (G) or coarse-medium grained sand (CS-MS) content of the soils and their chemical or mineral composition. However, a moderate correlation ($R = 0.56$) was observed between their fines content and their aluminium-iron oxide ratio (AIR) (Table 3). This implies that increase in iron oxide content of the soils which will result to decrease in their AIR values will subsequently lead to decrease in their amount of fines. Kaolinite and quartz content of the lateritic soils were also weakly correlated with their fine content, with correlation coefficients of 0.48 and -0.48 respectively. This implies that as the kaolinite composition of lateritic soils increases, their amount of fine content are likely to increase as well, while those that are primarily composed of quartz will likely have low amounts of fines.

The plasticity index of the fines was strongly correlated with their Al_2O_3 and SiO_2 content with R values of 0.87 and -0.88 respectively. This implies that the plasticity of lateritic soils increases with increase in Al_2O_3 content, and decreases with increase in SiO_2 content. It was also observed that the plasticity of the soils was strongly correlated with their SAIR with R value of -0.78, implying that increase in SAIR will subsequently lead to decrease in the soil's plasticity. As weathering intensifies, Al_2O_3 content of lateritic soils becomes relatively lesser than their Fe_2O_3 content due to variations in their degree of resistant to weathering

(Railsback, 1993). This will subsequently lead to decrease in the AIR value of the resultant lateritic soil (Fitzpatrick and Schwertmann, 1982), thus resulting in increase in SAIR. Since SAIR is negatively correlated with PI, it implies that the plasticity of lateritic soils derived from freshly weathered rocks will be higher than those derived from severely weathered rocks. Therefore, the degree of weathering of the parent material will be a major determinant of the engineering properties of the resultant residual soils; hence a major factor to be considered prior to sourcing of laterites as aggregates for roads and pavements constructions, since their durability depends significantly on the plasticity of the sub-base and subgrade material.

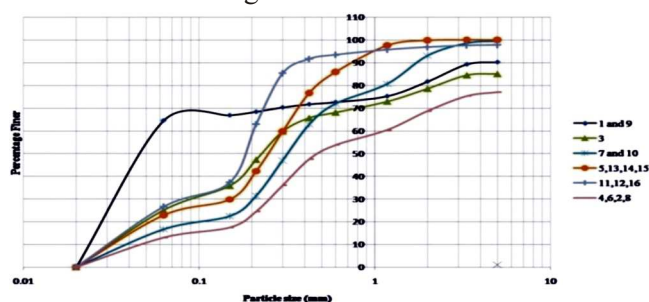


Fig. 2: Particle size distribution of the lateritic soils

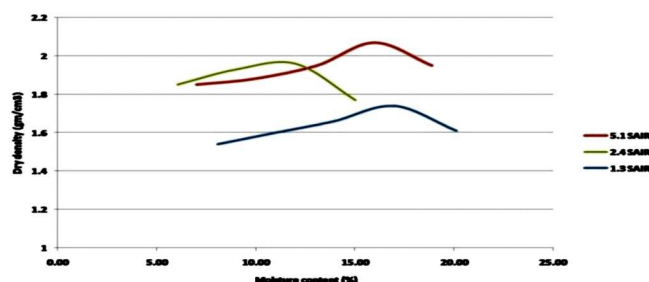


Fig. 3: Compaction properties of 3 typical lateritic soils with varying SAIR

Table 1: Geology, mineral and chemical composition of the lateritic soil samples

Location	Coordinate		Geology	Mineral composition (%)			Chemical composition (%)			Geochemical ratios	
	Lat. (N)	Long. (E)		QTZ	Kao	Hae	SiO_2	Al_2O_3	Fe_2O_3	SAIR	AIR
1	6.200667	7.500667	Mamu	56.4	27.5	13.2	55.4	35.5	7.6	1.29	4.67
2	6.187722	7.505917	Ajali Fm	78.3	9.0	11.0	85.6	8.0	4.7	6.74	1.70
3	6.037833	7.466583	Awgu Grp	52.3	31.3	14.2	53.7	36.2	8.3	1.21	4.36
4	5.907389	7.397167	Ajali Fm	76.2	11.8	8.0	78.7	16.5	3.4	3.95	4.85
5	5.896778	7.395389	Ajali Fm	74.6	7.0	13.1	81.6	11.5	5.3	4.86	2.17
6	5.855833	7.338833	Nsukka Fm	63.7	21.8	11.3	62.3	29.6	6.3	1.74	4.70
7	5.849278	7.335833	Nsukka Fm	54.8	29.4	13.0	53.6	37.4	7.0	1.21	5.34
8	6.407028	7.519083	Nkporo shl	51.2	35.2	9.4	55.1	36.7	6.2	1.28	5.92
9	6.282889	8.006861	Asu-river Grp.	49.1	38.2	8.0	55.5	38.2	3.2	1.34	11.94
10	6.401444	7.383083	Ajali	76.0	12.6	10.0	76.9	16.6	5.1	3.54	3.25
11	6.256806	7.167667	Ebenebe sst	71.3	8.9	13.1	83.3	8.7	6.3	5.55	1.38
12	6.20125	7.01975	Ameki Grp	61.8	21.4	12.6	66.5	26.2	5.2	2.12	5.04
13	6.200833	7.0135	Ameki Grp	60.4	23.8	13.2	76.0	18.7	3.0	3.50	6.23
14	6.234694	6.960417	Ameki Grp	61.7	25.4	10.4	60.0	32.7	5.1	1.59	6.41
15	6.235278	6.917611	Ameki Grp	69.4	19.3	9.0	70.8	21.8	4.3	2.71	5.07
16	6.194417	6.881611	Ameki Grp	58.3	24.9	11.8	67.0	24.6	6.2	2.18	3.97

QTZ= quartz, Kao = kaolinite, Hae = Haematite, Fm = Formation, Grp = Group, sst = sandstone, shl = shale, SAIR = silicate-Aluminium+ iron oxide ratio, AIR = Aluminium-iron oxide ratio

Table 2: USCS soil classification, index and engineering properties of the lateritic soils

Location	Consistency limits (%)			Particle size distribution (%)				USCS			Compaction properties		Shear strength properties	
	LL	PL	PI	G (>2mm)	CS-MS (<2-0.25mm)	FS (<0.25-0.063mm)	Fines (<0.063mm)	C _c	Soil class	Grading	OMC (%)	MDD (g/cm ³)	c (KPa)	Φ (°)
1	54	27.4	27	23.71	8.27	5.15	62.87	0.87	CH	PG	19.04	1.79	120.00	13.9
2	32	NP	0	49.38	13.53	12.13	24.96	0.34	GM	PG	12	2.02	35.40	26.7
3	60	31	29	21.50	18.81	34.65	25.04	0.87	SC	PG	19.8	1.68	48.04	28.6
4	33	30	3	57.44	13.99	15.95	12.62	0.31	SM	PG	14.6	2.13	12.17	38.1
5	28	26	2	0.30	35.38	47.98	16.34	2.62	SM	WG	12.5	2.07	12.59	27.6
6	31	22	9	53.68	24.89	15.32	6.11	0.46	GW-GC	PG	13.45	1.92	1.68	39.9
7	34	20	14	8.35	43.83	26.96	20.87	2.62	SM-SC	WG	15.02	2.02	27.80	37.2
8	49	21	28	46.42	14.69	19.96	18.93	0.36	SC	PG	19.5	1.69	38.86	35.4
9	47	23	24	13.05	14.44	6.33	66.17	0.78	CL	PG	16.85	1.83	71.17	27.9
10	28	NP	0	5.72	48.17	33.73	12.38	2.81	SM	WG	12.2	2.17	37.40	32.2
11	28	NP	0	1.73	16.83	56.04	25.40	1.01	SM	WG	13	2.06	67.76	25.0
12	27	18	9	0.46	9.95	67.47	22.11	1.13	SM-SC	WG	15.35	1.87	53.50	23.9
13	26	20	6	0.05	31.64	39.20	29.10	2.76	SM-SC	WG	14.75	1.96	56.64	26.9
14	32	17	15	0.53	35.00	35.04	29.42	2.67	SC	WG	11.2	1.82	56.96	28.8
15	28	22	6	0.10	58.24	25.17	16.49	2.58	SM-SC	WG	12	2.11	32.20	28.1
16	30	12	18	7.50	7.83	52.56	32.10	1.06	SC	WG	13.5	1.91	53.02	26.0

LL = liquid limit, PL = Plastic limit, PI = Plasticity index, C_c = Coefficient of curvature, G = Gravel, CS-MS = Coarse-medium grain sands, FS = Fine sand, USCS = Universal soil classification system, PG = Poorly graded, WG = Well graded,

Table 3: Correlation matrix of the relationships among some chemical and mineralogical compositions, and geotechnical properties of lateritic soils

	AIR	Al ₂ O ₃	c	CS	MS	FINES	FS	Fe ₂ O ₃	G	Hae	Kao	LL	MDD	OMC	Φ	PI	PL	QTZ	SAIR	SiO ₂
AIR	1.00																			
Al ₂ O ₃	0.66	1.00																		
C	0.23	0.30	1.00																	
CS_MS	-0.07	-0.11	-0.38	1.00																
FINES	0.56	0.42	0.84	-0.42	1.00															
FS	-0.41	-0.33	-0.05	0.03	-0.32	1.00														
Fe ₂ O ₃	-0.35	0.38	0.18	-0.12	-0.03	0.14	1.00													
G	-0.03	0.05	-0.33	-0.42	-0.19	-0.63	-0.01	1.00												
Hae	-0.47	0.01	0.24	-0.12	0.02	0.46	0.61	-0.32	1.00											
Kao	0.74	0.93	0.36	-0.15	0.48	-0.33	0.27	0.03	-0.06	1.00										
LL	0.37	0.67	0.46	-0.40	0.53	-0.48	0.45	0.30	0.12	0.70	1.00									
MDD	-0.40	-0.75	-0.44	0.51	-0.45	0.13	-0.45	-0.14	-0.26	-0.78	-0.77	1.00								
OMC	0.33	0.66	0.23	-0.52	0.28	-0.35	0.20	0.46	-0.02	0.52	0.76	-0.74	1.00							
Φ	0.08	0.07	-0.83	0.31	-0.67	-0.19	-0.09	0.45	-0.40	0.04	-0.20	0.22	0.03	1.00						
PI	0.52	0.87	0.45	-0.36	0.55	-0.28	0.49	0.09	0.13	0.88	0.87	-0.89	0.62	-0.18	1.00					
PL	0.49	0.57	0.00	-0.08	0.18	-0.28	0.06	0.16	0.08	0.52	0.51	-0.43	0.65	0.09	0.57	1.00				
QTZ	-0.63	-0.93	-0.40	0.18	-0.48	0.23	-0.39	0.04	-0.15	-0.98	-0.72	0.83	-0.51	0.04	-0.89	-0.53	1.00			
SAIR	-0.62	-0.94	-0.23	-0.01	-0.28	0.15	-0.40	0.09	-0.01	-0.85	-0.51	0.63	-0.42	-0.09	-0.78	-0.61	0.85	1.00		
SiO ₂	-0.60	-1.00	-0.31	0.11	-0.40	0.29	-0.51	-0.02	-0.07	-0.92	-0.68	0.76	-0.47	-0.05	-0.88	-0.55	0.92	0.94	1.00	

Φ = angle of internal friction, c = cohesion, S = shear strength, AIR = Aluminium/iron oxide ratio, SAIR = Silica/ Aluminium + Iron oxide ratio

Relationship Between the Compaction Properties and Geochemical Composition of Lateritic Soils

The MDD of the lateritic soils was weakly correlated with their amount of fines content, with R value of -0.44. This implies that increase in their amount of fines content may likely result to reduction in the MDD of the soils. However, the correlation analysis also reveals that there was a moderate-strong correlations between the MDD of the soils and their geochemical compositions such as, SiO₂, Al₂O₃, and Fe₂O₃ with R values of 0.76, -0.75, and -0.45, respectively. This implies that the fines content of lateritic soils may not be the major determinant of their MDD, but rather the chemical

compositions of such fines. This assertion was further buttressed by the fact that there were strong correlations between the OMC of the soils and their liquid limits (LL) (R= 0.77); thus soils with relatively high LL also have high OMC and corresponding lower MDD. Looking at the correlations that exists among the LL, OMC, or PI and their SiO₂, Al₂O₃ and Fe₂O₃ (Table 3), it could be ascertained that the moisture content of lateritic soils depends more on the mineralogical composition of the prevailing fines than on the quantity of fines. Thus, increase in the amount of Al₂O₃ and Fe₂O₃ will lead to increase in the OMC and subsequent reduction in MDD, while increase in the amount of SiO₂ composition will lead to reduction in the OMC required

to achieve high degree of MDD. Therefore lateritic soils derived from Formations with relatively high SAIR may be more desirable for construction purposes than those derived from Formations with relatively very low SAIR. However results from this research revealed that lateritic soils derived from geological Formations with relatively moderate SAIR values of about 2.4 especially those from Ameki Formation has the best compaction properties (MDD of about 1.95 gm/cm^3 at 11% moisture content) in terms of road and pavement constructions (Fig. 3).

Relationship Between the Shear Strength Properties and Geochemical Composition of Lateritic Soils

The consolidated drained shear strength of the lateritic soils obtained using the direct shear test method revealed that all the soil samples were cohesive-frictional soils (Table 2); however, their degree of cohesion and friction varies. There was no strong direct correlation between the shear strength properties and the geochemical compositions of the soils. However, it was observed that cohesion of the lateritic soils was strongly correlated with their amount of fines content, and weakly correlated with their LL, PI, and quartz content, with R values of 0.84, 0.46, 0.45, and -0.40, respectively (Table 3). This implies that lateritic soils with high content of fines are likely to be cohesive, and that their degree of cohesion is also dependent on their plasticity index which has a strong positive correlation with the amount of Al_2O_3 content. But the negative correlation observed between the cohesion and the quartz content implies that if the fines are primarily composed of quartz, their increase may not necessarily lead to increase in cohesion of the soils. Therefore increase in cohesion is not solely dependent on the amount of fines, but also on the plasticity index of the soils which strongly reflects the geochemical composition of the soil particles.

Similarly, the angle of internal friction (Φ) of the lateritic soils does not have any significant direct correlation with their geochemical compositions. The frictional force was observed to be moderately correlated with the amount of fines and weakly correlated with the gravel content of the soils, with R values of -0.67 and 0.45 respectively (Table 3). These values implies that as the amount of fines is increasing, the frictional force acting within the soil matrix reduces, while increase in the gravel content is likely to increase the frictional forces. However, as revealed by Igwe et al., (2009), the maximum friction and cohesion required for the ideal shear strength will depend on the optimum particle size mix.

Conclusions

From the result analysis and interpretations, the following conclusion could be drawn;

1. That the geochemical composition of lateritic soils has a significant influence on their geotechnical properties; hence reflecting the contribution of the underlying geology.
2. That SAIR and AIR could serve as an index geochemical properties of lateritic soils which could be used for quick appraisal of aggregates for engineering construction purposes.
3. That the index properties of lateritic soils such as particle size distribution, plasticity index, and consistency limits are directly related to their geochemical composition unlike their engineering properties such as compaction and shear strength that are indirectly related.

These interpretations were based on the statistical correlations that exist among the different geotechnical properties and geochemical compositions of the lateritic soils, and it is therefore worthy to note that all R values which are less than 0.497 are not statistically significant at 5% level for a non-directional two-tailed test.

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Estimation of the Strength Characteristics of Residual Soils in a Nigerian State: Achieving Soil-Foundation Relationship Comfort

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Abstract

This study is aimed at providing insight to the strength characteristics of residual soils in Benin City, Southwestern Nigeria. In order to achieve this, a total of 18 soil samples were obtained from the study area. Seven trial pit excavations and 1 Standard Penetration Test (Soil Boring) were carried out. These SPT involved taking 11 samples at some intervals between 0.05m and 7.5m. The following tests were carried out on these samples: Natural Moisture Content (NMC), particle size analysis, Atterberg limits, specific gravity and linear shrinkage, while angle of internal friction, cohesion and permeability tests were carried out on samples taken from trial pits only. NMC values range from 14.4% to 25.6%. The values for liquid limit, plastic limit, plasticity index, and linear shrinkage range from 30.8% to 36.2%, 19.4% to 23.2%, 9.6% to 13.65% and 10.6% to 11.5% respectively. All the sampled soils are within the acceptable standard of liquid limit, plastic limit and plasticity index for foundation requirements. For linear shrinkage test, values were higher than the required which makes them poor. The particle size analysis showed that all the soils had more coarse contents than fine contents and are well graded but they all fall below 35% of maximum permissible use for a good foundation material. The specific gravity of the tested soil samples range between 2.60 and 2.66 which classified these soils as either sand or silty sand. The values of the shear strength parameters are the angle of internal friction ranged from 22.4° to 30.0°, and the cohesion ranges from 100.0 to 106.1kPa. All the seven samples from trail pits fell within the hard category. This implies that they had high bearing capacity and considerable strength to withstand shear stresses, thereby serving as good foundation materials.

Keywords: *Strength, Characteristics, Residual, SPT, Foundation, Benin City, Nigeria.*

Introduction

In recent times, foundation material (soil) has been considered as fundamental and integral item to consider in engineering construction. Increasing population, industrialization and the need to conform to global best practices in Nigeria has resulted in the significant increase in the erection of more buildings and urbanization. According to Coduto, 2001, foundations are structural elements that connect buildings, bridges and other structures to the ground. Structure of different types such as buildings, dams, bridges, highways, and canals are held firmly to the soil with the help of the foundation (Bowles, 2008). More recently, the increasing trend (in figures) of building failures throughout Nigeria and other developing countries of the world has been attributed to the neglect of that important aspect of building construction. The design of a structure which is safe, durable and with low maintenance costs is a factor of well calculated estimates with proper knowledge of the soil on which the structure will stand (Joint Departments of the Army and Air Force, USA, 1983). A proper and detailed site investigation will help to mitigate the difficulties that may arise during construction because of ground (British Standards Institution, (1999) and geological conditions (fault and groundwater table). Engineering properties which include consolidation, permeability, compaction and shear strength are influenced by the

mode of formation, stress history, groundwater conditions and physiochemical characteristics of the surficial deposits deduced from parent material (Craig, 2004). Hence, the need to prepare numerous samples and the use of expensive laboratory equipments in the estimation of some of the basic engineering properties of soils, such as the cohesion and friction angle (Ersoyl, *et al.*, 2013). The permeability of soils determines the stability of foundations, seepage loss through embankments of reservoirs, drainage of subgrades, and excavation of open cuts in water bearing sand as well as the rate at which water flow into well (Murthy, 2012). The understanding of the shear strength of a soil is important in the assessment of bearing capacities of foundations (Too, 2012), and slope stability (Liu, *et al.*, 2004). Standard Penetration Test data have been used in correlations for unit weight, relative density, angle of internal friction and unconfined compressive strength (Kulhawy and Mayne, 1990). The engineering approach of this investigation is to estimate the strength properties of soils in Benin area when subjected to loads and also recommend the best foundation type.

Study Area

The area of investigation is Ebvolekpen Community, very close to Benin City, Edo State. It lies within Longitudes 5°30'30"E to 5°32'00"E and Latitudes 6°17'30"N to 6°19'00"N with an average elevation of

68m (Fig. 1). The study area lies within the tropical rain forest and has two distinct seasons (wet; April to October, and dry; November to March). The study area lies within the Benin Formation (Fig. 2) which extends from the west across the whole of the Niger Delta area and southward beyond the present coastline.

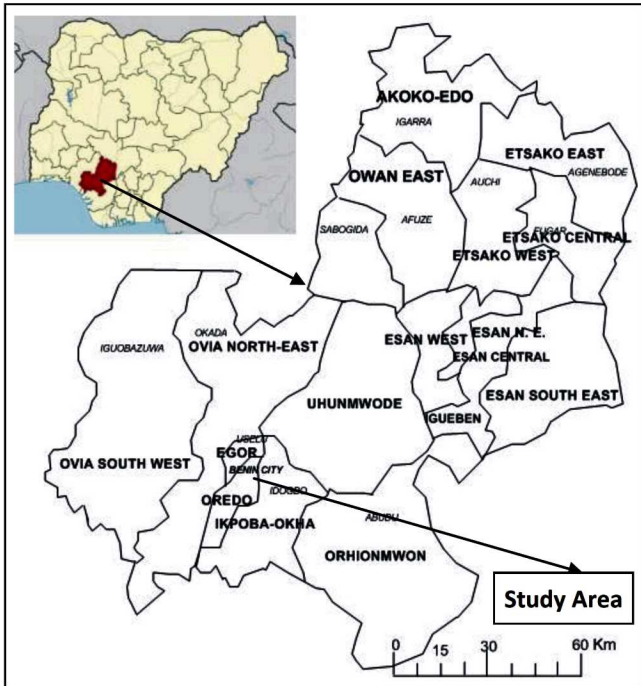


Fig. 1: Map of Edo State showing the study area

Materials and Methods

A reconnaissance survey was embarked upon to determine the points where samples would be collected and also to have an overview of the geology of the study area. A total number of 18 samples were collected for the analysis. This SPT involves taking 11 samples at the borehole at 0.05m, 0.75m, 1.5m, 2.25m, 3.0m, 3.75, 4.50m, 5.25m, 6.0m, 6.75m, and 7.5m respectively. Samples taken from trail pits are represented by TP1, TP2, TP3, TP4, TP5, TP6, and TP7, in accordance with (British Standards Institution, 1981). The colour and the texture of the samples taken from the borehole were observed and recorded (Table 1). The laboratory analysis carried out include Atterberg limit, grain size analysis, natural moisture content, specific gravity and linear shrinkage tests. While cohesion, angle of internal friction and permeability were carried out on samples taken from trial pits excavation only. The analysis was carried out at the Engineering Geology Laboratory of the Federal University of Technology Akure (FUTA) in accordance with (British Standards Institution, 1990).

Results and Discussion

The values of natural moisture content range from 14.4% to 26.4% (Table 2). According to the average range (5–15%) specification of FMWH (2010), for

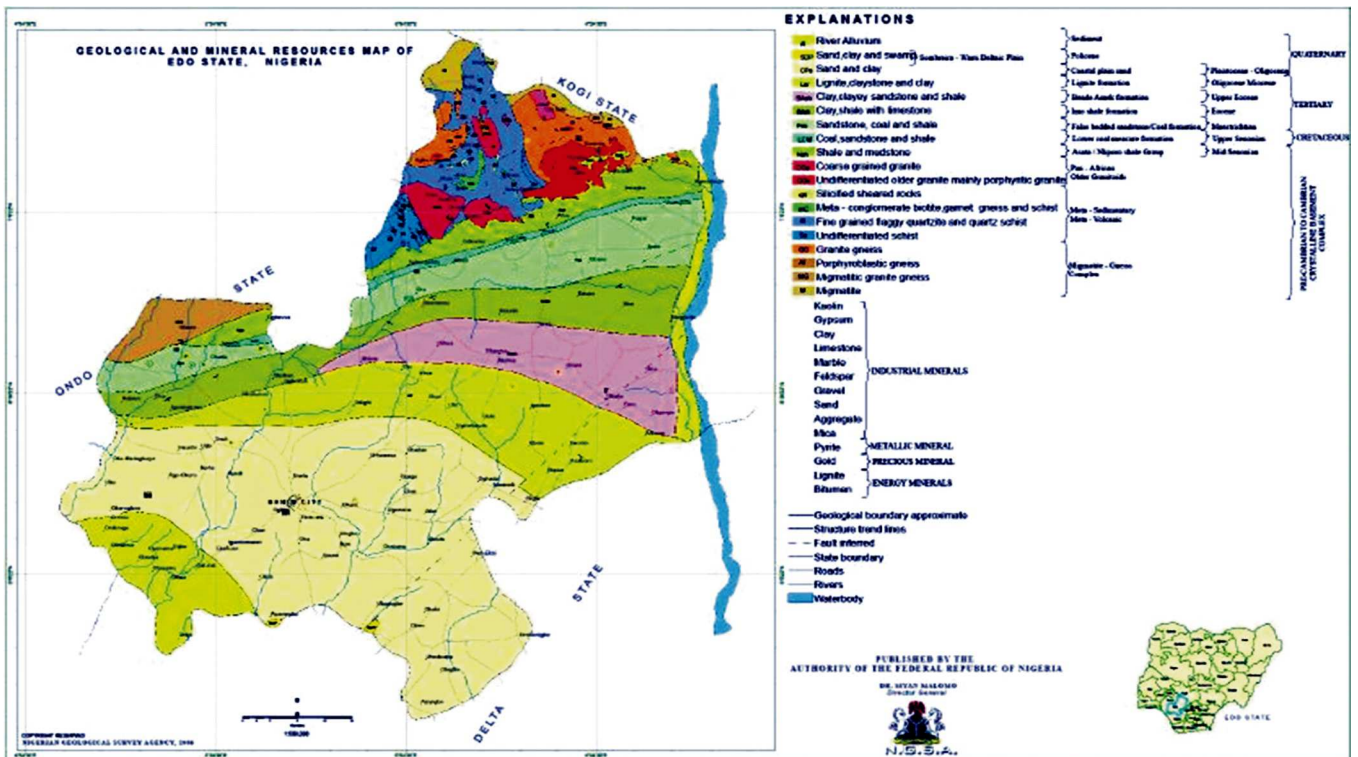


Fig. 2: Geological Map of Edo State Showing Benin City and other Locations (Nigerian Geological Survey Agency, 2006)

Table 1: Borehole log and soil description

DEPTH (M)	SOIL DESCRIPTION	NO. OF BLOWS
0.05	Topmost soil	
0.75	Yellowish brown lateritic soils	
1.50	Yellowish brown lateritic soils	2/3/3
2.25	Reddish brown lateritic soils	
3.00	Lateritic soils	4/7/9
3.75	Lateritic soils	
4.50	Lateritics soils and gravels	
5.25	Lateritic soil and gravel	
6.00	Lateritic soils and gravel	9/18/20
6.75	Lateritic soil and gravel	
7.50	Refusal depth	

Source: Researcher's adaptation 2020

engineering construction, all the sampled soils except one fall above the required recommendation. This is an indication of swelling potential. High variation in the moisture content causes large volume changes in the clayey soils (Daramola, *et al.*, 2018). The coarse contents of the sampled soil range from 59.6% and 64.6% while the fine contents range from 35.4% and 40.4% (Table 2). All the sampled soils have more coarse contents than fine contents but they all fall below the FMWH (1997) of 35% maximum permissible use for a good foundation. From the grain size characteristics

curve, all of the samples are well graded which is a reflection of decreasing degree of weathering and leaching in the area. Soils with higher coarse contents are an indication of low plasticity and vice versa. For the consistency test, the liquid limit ranges from 31.4% to 36.2%, plastic limit ranges from 19.4% to 23.2% while the plasticity index ranges from 9.6% to 13.65% (Table 2). All the sampled soils fall within the acceptable limit for foundation material. The Federal Ministry of Works and Housing (1997) recommended 50%, 30% and 20% maximum for the liquid limit, plastic limit and plasticity index respectively. All of the sampled soils fall within the acceptable limit values for foundation materials of FMWH (1997) for liquid limit, plastic limit and plasticity index. The results of linear shrinkage range from 10.6% to 11.5% (Table 2). All of the soil samples have values higher than 8% which make them not suitable as foundation material. Higher specific gravity gives more strength for roads and foundations (Ola, 1983). The results of the specific gravity of the tested soils ranged from 2.60 to 2.66 (Table 2). Correlation of the results with (Bowles, 2012) classification shows that these sampled soils are either sand or silty sand. The coefficient of permeability for the soils sampled from trial pits range from 2.144×10^{-5} to 1.337×10^{-4} (Table 2). According to U. S. Bureau of Reclamation, soils are classified as (i) Impervious: k (coefficient of

Table 2: Showing the summary of results of geotechnical laboratory analysis

Sample	LL (%)	PL (%)	PI (%)	MC (%)	% Fines	% Coarse	SG	Permeability (cm/s)	Permeability	Traxial °/KPa
TP1	31.4	19.4	12.05	17.2	40.2	59.8	2.64	0.00002144	LOW	26°/101
TP2	32.7	19.7	13.0	18.1	36.8	63.2	2.64	0.00003216	LOW	25°/106
TP3	33.0	20.1	12.9	16.2	36.0	64.0	2.64	0.00013374	LOW	26°/102
TP4	32.0	20.3	11.75	16.3	36.7	63.3	2.64	0.00012146	LOW	30°/102
TP5	32.3	19.9	12.40	16.2	37.1	62.9	2.64	0.00003129	LOW	25°/100
TP6	34.2	21.9	12.30	14.4	37.2	62.8	2.64	0.00003047	LOW	22°/100
TP7	33.8	20.9	12.95	15.6	38.7	61.3	2.65	0.00002573	LOW	27°/100
BH@0.05	32.2	21.3	10.95	24.1	35.7	64.3	2.64			
BH@ 0.75	34.1	23.1	11.05	26.3	36.9	63.1	2.63			
BH @ 1.5	35.6	23.2	12.45	26.2	36.9	63.1	2.66			
BH@ 2.25	36.2	23.2	13.05	26.4	40.4	59.6	2.64			
BH@ 3.00	34.0	21.6	12.40	25.6	38.4	61.6	2.63			
BH@ 3.75	35.7	22.1	13.65	25.3	37.1	62.9	2.60			
BH @4.50	32.6	21.1	11.50	22.1	38.4	61.6	2.63			
BH @5.25	30.8	21.2	9.65	20.9	36.1	63.9	2.62			
BH @6.00	33.3	21.2	12.15	20.1	37.2	62.8	2.61			
BH @6.75	34.1	21.3	12.85	17.9	35.4	64.6	2.64			
BH @7.5	32.2	22.6	9.6	16.2	36.2	63.8	2.65			

LL= Liquid Limit, PL= Plastic Limit, PI= Plasticity Index, MC= Moisture Content, SG= Specific Gravity, Traxial (Angle of Friction/Cohesion)

Source: Researcher's adaptation 2020.

permeability) less than 10^{-6} cm/sec, (ii) Semi-pervious: k between 10^{-6} to 10^{-4} cm/sec (iii) Pervious: k greater than 10^{-4} cm/sec; this implies that all the soils are semi-

pervious and have low degree of permeability. All of the sampled soils from trial pits have low degree of permeability. When these soils come in contact with

water, the soils will not allow the easy passage of water. This shows that the soils are good as foundation materials. The values of the shear strength parameters are; the angle of internal friction ranged from 22.4° to 30.0° , the cohesion ranges from 100.0 to 106.1kPa (Table 2). Applying the USCS classification, all the seven samples obtained from trial pits fall within the hard category. Soils that fall within the hard and stiff categories will have considerable strength to withstand shear stresses, have high bearing capacity, and thereby serve as good materials for foundation construction. These types of soils lack minerals that can bring about drastic change in their properties.

Conclusion

Results from consistency limit tests revealed that the

soils are good as foundation materials. On the other hand, results of linear shrinkage and NMC revealed the soils as poor foundation materials. All of the sampled soils have more coarse contents than fine contents but they all fall below the FMWH (1972) of 35% maximum permissible use for a good foundation. Coefficient of permeability revealed that all of the sampled soils from trial pits have low degree of permeability, indicating that the soils are good foundation materials. The values of the shear strength parameters show the soils to fall within the hard category. Soils that fall within the hard and stiff categories will have considerable strength to withstand shear stresses, have high bearing capacity, and thereby serve as good foundation material.

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Evaluation of Geotechnical and Mineralogical Characterization of Soils Derived From Basement Complex in Southwestern Nigeria.

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Abstract

There is a widespread study on soils as typical construction materials in the tropics. Yet, oftentimes few sampling locations are considered in assessing the impact of geology on the performance of soils for engineering purposes. This paper attempts to characterize soils based on their engineering and mineralogical properties to depict engineering performance in different geological environments. Fifty soil samples were mineralogically characterized using X-ray powder diffractometry, and soil index and compaction tests were carried out following BSI standard. XRD analysis detected the presence of five clay minerals with dominance of average kaolinite (22.9%), pyrophyllite (18.7%) and illite (14.3%) in the migmatite-gneiss complex (PCB) more than other environments. The soil index properties showed mixed gradation with high amount of fines (59.6%) dominating the entire soils, low to slightly high plasticity (14.3-53% LL & 2.6-22.7% NMC) and moderate swelling (3-20.4%) particularly in the migmatite-derived soils. The soil strength characteristics showed average maximum dry density and optimum moisture content of 1.8mg/m³ and 16.9% respectively. Average unsoaked and soaked California Bearing Ratio varied between 40-54% and 18-25% respectively. Intrinsic result revealed the presence of smectite minerals clearly omitted in previous studies which elucidates the reason for the perpetual failure of some sections of the highway. This information could be useful in remedial works on the rehabilitation of the failed sections of the road and guide future pavement designs in similar terrain.

Keywords: X-ray Diffraction, index properties, strength properties, mineralogy, basement complex

Introduction

All structures and infrastructures are built on soil, a geological material that is composed of properties controlled by the minerals present in them. The knowledge of these minerals present in a particular soil formation is therefore important to the safety and success of the infrastructure built on/from them. Failure of road pavement is very common in Nigeria. Adeyemi et al. (2003) reported the presence of abundant kaolinitic mineral with quartz, illite, chlorite and vermiculite along Ibadan–Ago Iwoye highway. They equally established a high correlation between kaolinite and uniaxial compressive strength of the soil and reported that though the soil were of A-5 AASHTO grouping which indicated poor to fair subgrade soil, their plasticity and strength characteristics were of good subgrade. Similarly, Ogundalu and Oyekan (2014) observed the presence of quartz, calcite, larnite, kaolinite and high swelling montmorillonite minerals in the black cotton soils along the Maiduguri–Gamboru highway, which negatively influenced the stability of the road. They underscored the importance of mineralogy in the mechanism of swelling and shrinkage behavior of soils and its effect on the soil strength. This was particularly essential in the application of chemical stabilizer in order to improve the soil properties as it depends on chemical reaction and bonding between the stabilizer, soil mineral and chemical composition (Ogundalu et al., 2013; Oyediran & Fadamoro, 2015).

The aim of this study is to evaluate geotechnical properties and mineralogical composition of soils along the study area with a view to determine the factors responsible for the failure of the highway.

Location and Geology

The study area is located in Southwestern Nigeria, between latitudes 6°15'N and 8°30'N and longitudes 4°30'E and 6°50'E. The topography is made of undulating terrain with a mean elevation of 400m above sea level (Fig. 1). Underlying the highway is the Precambrian rocks of the Basement Complex composed of the migmatite-gneiss, metasediments and Older granite rocks (Oyinloye, 2011). Two main widespread types of gneisses common in the area are the fine-grained biotite gneiss with strong foliation and the banded gneiss (Adekeye & Akande, 2006). The older granites that vary extensively in composition occur as large circular masses within the schists and the older migmatite-gneiss complex. These rocks were disintegrated and transformed into soils, which have variable thickness, geochemical, mineralogical and geotechnical characteristics (Boul et al., 2003).

Method of Investigation

Fifty disturbed soil samples were collected along Ilorin – Kabba highway at the depth of 0.8 - 1.5m (Fig. 1) at both stable & unstable road portions while taking

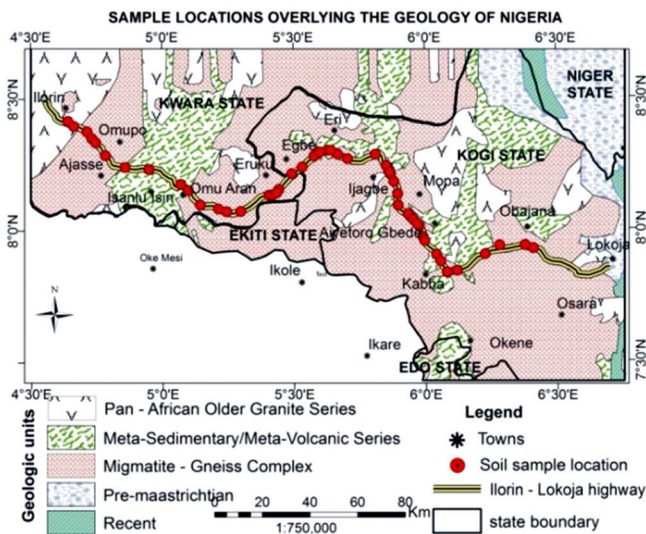


Fig. 1: Geology of Nigeria showing the study highway overlain by soil sample locations.

coordinates and elevation using global positioning system (GPS). Using Surfer 8 software, the elevation points were analyzed and contour lines generated into a 3D vector-based digital image of the area. Drainage patterns were extracted from ASTER imagery and overlain on a 3D surface model of the area.

Laboratory soil tests were conducted on air-dried (35°C) samples of 0.425mm diameter following the British standard (BS 1377: 1990) protocols reported by Ige et al. (2018) and summarized in Table 1. These tests include particle size distribution, consistency limits, compaction and CBR. X-ray diffraction was conducted on a 10g portion of each soil sample using Bruker D8 Advance X-ray diffractometer with monochromatic Cu-K α_1 radiation ($\lambda = 1.54\text{\AA}$) in the range of 10° and 90°, at the Department of Chemical Sciences Laboratory, University of Johannesburg, Doornfontein Campus, South Africa. XRD is a rapid analytical technique used extensively for phase identification of crystalline material based on constructive inference of X-ray and crystalline sample.

Table 1: Laboratory tests and standards

S/N	Test	Method/Standard
1	Particle size analysis	BS 1377: 1990 Part 2 Clause 9.2
2	Liquid limit (LL)	BS 1377: 1990 Part 2 Clause 4.5
3	Plastic limit (PL)	BS 1377: 1990 Part 2 Clause 5.3
4	Plasticity index (PI)	BS 1377: 1990 Part 2 Clause 5.4
5	Free swell (Fsw)	IS 2720: 1985 Part 40
6	Natural Moisture Content (NMC)	BS 1377: 1990 Part 2 Clause 3
7	Optimum Moisture Content	BS 1377: 1990 Part 4 Clause 3.3
8	Bulk Density (BD)	BS 1377: 1990 Part 2 Clause 6.7
9	Maximum Dry Density (MDD)	BS 1377: 1990 Part 2 Clause
10	California Bearing Ratio (CBR)	BS 1377: 1990 Part 9
11	X-ray Diffraction analysis	Bragg's law
Derived Measurements		
12	Weighted Plasticity index (wPI)	Look 2019
13	Activity (Ac)	Skempton 1984

The diffractograms with the corresponding intensity and 2 θ were analyzed in Origin 8 & Match 3 software packages for pattern fitting with established Mineral Power Diffraction File Data Book (ICDD, 2001). Other information obtained were crystalline size, % crystallinity, phase quantification and lattice parameter determination.

Results and Discussion

Mineral Characterization

The mineralogical analysis is summarized in Table 2 based on the geology of the area. Five clay minerals and eight non-clay minerals were identified with prevalence in kaolinite and quartz. This agrees with findings of many researchers including Ige (2015). XRD patterns of some soil samples are shown (Fig.2). Illite, pyrophyllite, chlorites and smectite clay minerals are also found in the soils in varying amount ranging from major (>30%), moderate (10-30%), minor (2-10%), and trace (<2%). One-fifth of the soils contain 2:1 clay minerals particularly around Yagba, Ijagbe and Mopa areas. Apart from the clay minerals which significantly influence the engineering behavior of subgrade soils, other non-clay minerals such as quartz, plagioclase, k-feldspar, berlinite, alunite, goethite are also identified with traces of calcite and zeolites in the mineralogical assemblage. These minerals however, do not contribute to soil shrinkage due to lack of activity in them. The proportions of kaolinite (ka), chlorite (ch), pyrophyllite (pr) and illite (il) ranges with mean from 0.9 - 57% (22.9%), 1 - 55% (18.4%), 0.1 - 51% (18.7%) and 0.8 - 40% (14.3%) respectively in migmatite gneiss (PCB) region. The percent occurrence is highest, followed by meta-sediments (PCM) 4.3 - 50.5% (31.5%), 1.4 - 22.1% (10.8%), 5.7 - 17.9% (11.8%) & 2.8 - 28.8% (15.7%) and older granite (PCG) from 5.9 - 29.3% (17.4%), 10.6 - 26.1% (17.3%), 5.4 - 28.9% (17.2%) & 0.2 - 14.8 (9.2%) respectively. The percentage of quartz is more in PCB and PCG environments than the PCM derived soils, which compares to a slightly high variation (24.4%) in plasticity index of the same area.

K-feldspar consists of orthoclase. Plagioclase is mostly albite mineral. Smectite is mainly montmorillonite and nontronite.

Kaolinite is a low latitude 1:1 mineral formed during chemical weathering in a hot and dry, humid climatic conditions and soil forming processes (Chamley, 1989). Its abundance and co-occurrence with illite in the studied samples suggests the predominance of warm

Table 2: Semi-quantitative result of mineral contents (per percent of the crystalline phases)

Geologic environment	Statistical Parameters	CLAY MINERALS (%)					NON-CLAY MINERALS (%)							
		Kaolinite (Al ₂ (Si ₂ O ₅)(OH) ₄)	Pyrophyllite Al ₂ Si ₄ O ₁₀ (OH) ₂	Chlorite Al ₂ (Mg,Fe) ₅ Si ₁₁ (OH) ₂	Illite K _{1.5} Al ₄ (Si _{6.5} Al _{1.5})O ₂₀ (OH) ₄	Smectite (Na,Ca)([Mg,Fe]Al ₃ [Si ₈ O ₂₀](OH) ₄)	Quartz SiO ₂	Plagioclase (Ca ₁ Na)Al _{1.2} Si _{3.2} O ₈	K-Feldspar KAlSi ₃ O ₈	Berlinite AlPO ₄	Alunite Al ₃ H ₆ K _{0.8} Na _{0.5} O ₁₄ S ₂	Goethite FeOOH	Calcite CCa _{0.9} Mg _{0.1} O ₃	Zeolite Mg ₂ Na ₇ (Si,Al) ₁₂ O ₄₈
MIGMATITE -GNEISS COMPLEX (PCB)	Min	0.9	0.1	1.0	0.8	0.1	3.4	0.7	0.9	1.5	0.2	3.1	0.8	11.9
	Max	57.3	51.0	55.3	40.3	23.2	87.3	78.7	21.6	69.3	12.3	9.1	17.2	18.9
	Mean	22.9	18.7	18.4	14.3	8.7	28.9	20.9	7.4	22.8	4.7	6.1	9.0	15.4
	SD	14.1	15.4	20.2	12.7	7.8	23.4	20.4	8.4	18.4	5.0	3.0	11.6	4.9
META-SEDIMENT / META-VOLCANIC (PCM)	Min	4.3	5.7	1.4	2.8	4.4	3.6	2.5	0.8	1.3	4.4	1.3	4.0	6.1
	Max	50.5	17.9	22.1	28.8	22.0	58.3	32.2	63.5	34.6	16.4	10.9	7.9	17.5
	Mean	31.5	11.8	10.8	15.7	9.7	26.2	10.7	20	14.2	9.7	6.7	6.4	11.8
	SD	13.4	8.6	10.5	10.7	7.0	16.8	14.3	25.7	11.1	6.1	4.3	2.1	8.1
PAN AFRICAN OLDER GRANITE (PCG)	Min	5.9	5.4	10.6	0.2	0.2	13.6	23.7	1.9	2.1	13.6	1.1	8.1	-
	Max	29.3	28.9	26.1	17.8	14.8	82	27.2	1.9	13	13.6	18.7	20.7	-
	Mean	17.4	17.2	17.3	9.0	7.2	47.5	25.5	1.9	8.5	13.6	9.9	14.4	-
	SD	10.1	16.6	8.0	12.4	7.9	23.6	2.5	-	4.8	-	12.4	8.9	-

climate with rainfall (Branski, 2014) which characterizes the study area.

Based on the lithology, the presence of the major minerals also varies (Fig. 3). In the banded-gneiss derived soils, high kaolinite, pyrophyllite and quartz occur, with clear absence of illite in the porphyritic granite soils but subtly appear in banded-gneiss derived soils. Similarly, pyrophyllite, chlorite and smectite are in trace in the porphyritic soils but moderately high in the migmatite-gneiss. These soils with reasonable amount of smectite (1-25%) and illite (1-40%) would have low strength, high swelling potential & other undesirable properties than the kaolinite dominated soils with mixed clay assemblage (Kariuki et al., 2004; Yitagesu et al., 2011).

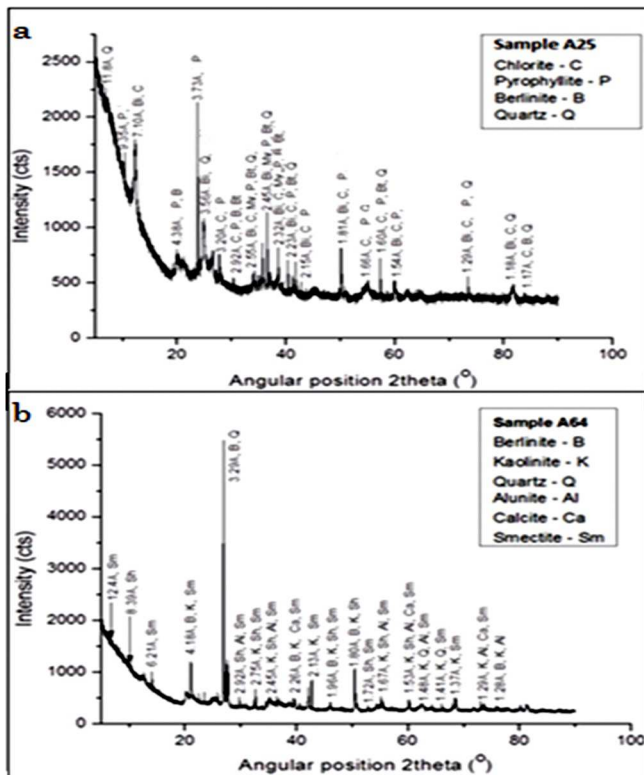


Fig. 2: X-ray diffractograms patterns of two soil samples around (a) Eruku and (b) Ijagbe town showing abundant mineral compositions

On one hand, the proportion of the mineral composition in the migmatite-gneiss environment is similar to the findings reported by Ige (2015) and Adebisi & Osammor (2017). On the other hand, the presence of smectite mineral was clearly missing in their reports. This might be due to lack of adequate sampling, sampling distance and extent of coverage in their works.

Index Properties

Summary of the geotechnical properties of the soils shows wide variations (Table 3). It is evident that the fines dominates the particle size distribution of all soils

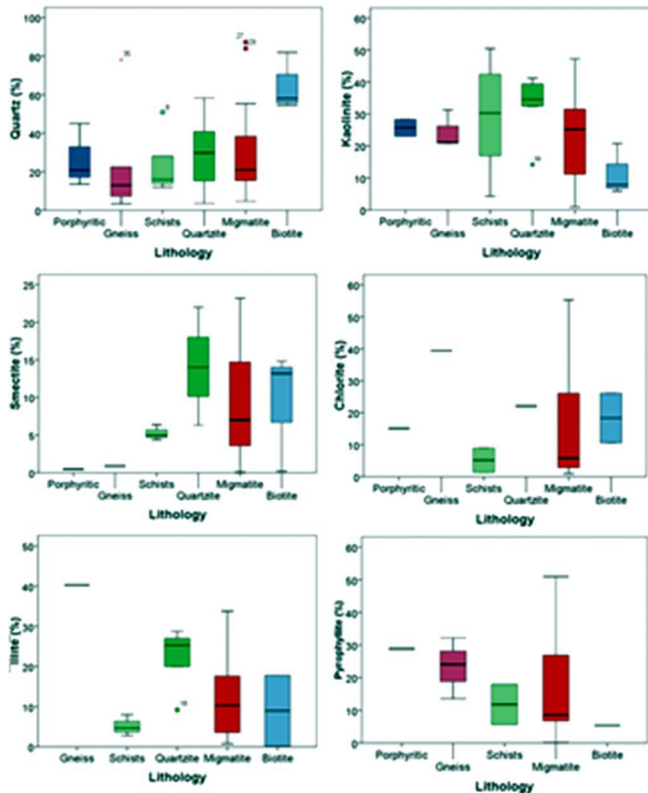


Fig. 3: Boxplots of major minerals showing variation in occurrence based on lithology. Porphyritic (blue), banded gneiss (purple), undifferentiated schists (light green), quartzite (green), migmatite (red), and biotite (light blue).

with the highest (59.6%) in migmatite (M) followed by (57.7%) in flaggy quartzite (Qs) while the biotite granite (OGe) exhibits the lowest amount of fines (27.5%). The grading characteristics of the soils vary greatly with relative amount of gravels across the lithology. Migmatite soils have slightly higher liquid limit (LL) (53%) and corresponding natural moisture content (NMC) (22.7%) compared to other environments of moderate plasticity (<25%). This high LL may be attributable to the presence of smectite mineral. The high NMC values are higher than the average range of 5-15% specified by FMWH (2013) for engineering purpose. The high fines, LL and NMC suggest a high water retention capability. Casagrande classification indicates inorganic clay soils of low to high plasticity, compressibility, hence low to high activity (0.1 – 8.5) and swelling potential (SP) (1.5 – 11.7%) which is suggestive of the presence of swelling clay minerals in the soils as revealed by the X-ray diffractions.

According to the AASHTO system, materials underlying the road are characterized by A-2 and A-7 soils except the banded gneiss (bG) soil which falls under A-1 and A-6 soils. Based on FMWH (2013) specification, A-1 and A-2 soils are excellent to good subgrade materials while A-6 and A-7 soils are poor subgrades. Similarly, USCS indicates the soils are predominantly silty, clayey sands (SC, SM, SC-SM)

Table 3: Summary of index properties and soil classification

Index Properties		Migmatite (M)	Banded Gneiss (bG)	Quartzite (Qs)	Undiff. Schist (Su)	Biotite (OGe)	Porphyritic (OGp)
Particle size distribution	Gravel (%) (>4.25mm)	38.6 - 100 (91.7)	57.4 - 100 (85.5)	70.6 - 100 (93.4)	65 - 100 (87.4)	62.4 - 100 (86.4)	45 - 100 (69.9)
	Sand (%) (2mm)	27 - 85 (61.4)	47 - 82 (60.8)	25 - 81 (51.7)	35 - 77 (58.3)	33 - 78 (57.5)	31 - 65 (45.0)
	Silt (%)	4.9 - 50.8 (20.3)	3.2 - 24.6 (12.1)	12.7 - 44.3 (24.2)	6.1 - 29.4 (16.8)	10.2 - 22.5 (14.9)	2.4 - 24.2 (14.8)
	Clay (%)	0.9 - 34.6 (14.6)	1.2 - 22.4 (9.52)	8.6 - 28.7 (17.3)	1.8 - 24.2 (12.2)	4.1 - 18.4 (10.02)	2.2 - 18.8 (11.1)
	Fines (%) (<0.075mm)	9.6 - 59.6 (24.5)	7.4 - 33.9 (18.4)	17.3 - 57.7 (32.9)	11.7 - 32.6 (22.8)	15.6 - 27.5 (21.7)	8.2 - 29.5 (19.9)
Atterberg Limits	LL (%)	14.3 - 53 (29.6)	17.5 - 32.5 (25.1)	24.2 - 46.5 (35.4)	19.5 - 44 (33.6)	16.5 - 27.4 (22.7)	18.5 - 32 (23.2)
	PL (%)	8.4 - 50 (20.3)	12.2 - 23.2 (18.4)	10.4 - 25 (20.9)	13.8 - 31.7 (23.7)	8.6 - 21 (16.7)	13.2 - 19 (15.5)
	PI (%)	1.9 - 24 (9.29)	1.6 - 14 (6.68)	1.0 - 22.6 (14.5)	2.1 - 21.0 (9.88)	2.6 - 10 (6.0)	4.6 - 13.4 (7.77)
	wPI (%)	0.5 - 12.8 (4.6)	0.24 - 7.9 (3.68)	1.12 - 17.5 (8.78)	0.9 - 14.8 (4.94)	1.6 - 7.02 (3.85)	0.87 - 7.3 (3.51)
	NMC (%)	2.6 - 22.7 (12.0)	3.0 - 14.8 (7.76)	7.3 - 16.2 (11.8)	3.1 - 18.4 (9.75)	3.2 - 10.6 (7.18)	2.4 - 12.2 (7.15)
	Activity	0.1 - 8.5 (1.31)	0.4 - 4.3 (1.44)	0.4 - 1.4 (0.88)	0.2 - 7.8 (1.85)	0.3 - 1.9 (0.77)	0.4 - 2.1 (1.07)
	Free Swell (%)	3.0 - 20.4 (7.35)	3.4 - 4.7 (4.06)	3.97 - 19.2 (9.75)	3.8 - 17.6 (8.17)	3.2 - 4.2 (3.83)	3.6 - 4.7 (4.04)
	SP (%)	1.6 - 11.7 (5.41)	1.5 - 7.6 (4.43)	2.89 - 11.6 (7.99)	1.95 - 10.9 (6.17)	2.3 - 6.4 (3.94)	3.5 - 7.9 (5.04)
Soil Classification	USCS	GP-GC, SM	SC, SW-SM	CL, SM	SC, SM	GC-GM, SM	GC-GM, SC-SM
	AASHTO	A-2-4, A-7-6	A-1-a, A-6	A-2-6, A-7-6	A-2-4, A-7-6	A-2-4	A-2-4, A-2-6

mixtures. Other minor classes are poorly graded gravel with clay or silty clay (GP-GC), lean clay (CL), well graded sand with silt (SW-SM), silty, clayey gravel (GC-GM).

Strength Properties

At standard proctor compaction, the respective average maximum dry density (MDD) and corresponding optimum moisture content (OMC) are 1.7 mg/m³ and 14.8% (Table 4) which is considerably lower for the A-1 and A-2 soils. The high moisture content points to a high water absorption capability, an indicator for the low soil strength, as OMC is inversely proportional to MDD. Unsoaked CBR (CBRu) ranges with mean from 18.8 to 68% (54.8%) while soaked CBR (CBRs) varies between 11.2 to 33.4% (25.0%). The low strength is thus attributable to the presence of active clay minerals in the soils.

Table 4: Meanmoisture–density relationship

Lithology type	Moisture dry density relationship					California Bearing Ratio	
	Bulk Density (mg/m ³)	Dry Density (mg/m ³)	Maximum Dry Density (mg/m ³)	Moisture Content (%)	Optimum Moisture Content (%)	unsoaked California Bearing Ratio (%)	soaked California Bearing Ratio (%)
Migmatite (M)	1.9	1.6	1.7	16.5	14.8	52.0	25.0
Banded Gneiss (bG)	2.0	1.6	1.8	14.1	11.7	51.2	25.1
Quartzite (Qs)	1.9	1.5	1.6	18.7	16.9	40.5	18.6
undifferentiated Schist (Su)	1.9	1.6	1.7	14.4	14.2	46.6	22.3
Biotite (OGe)	1.9	1.6	1.7	16.2	13.6	48.5	23.2
Porphyritic (OGp)	2.0	1.7	1.8	11.5	13.6	54.8	24.5

The result also reveals that the migmatite soils have slightly higher CBR than other areas. This implies that at a given geological environment, the strength of the residual soils varies in relation to volume change due to moisture influx. This result also agrees with the outcome of Adebisi and Osammor (2017) which

underscores the unsuitability of the soils to transmit axial loads for a long period without failure. In addition, lack of drainage results in collection of water from run-off on the shoulder of the road which promotes failure by lowering the soil strength.

Conclusions

The studied soils are predominantly composed of quartz and kaolinite, a 1:1 clay mineral as reflected in the mineralogical study. The grading characteristics of the soils vary greatly with fine particles dominating the composition of the soils. Based on the consistency limits, the residual soils revealed inorganic silty, clayey sand of low to slightly high plasticity with associated swelling potential due to the presence of active 2:1 clay mineral. Compaction and CBR characteristics indicate the subgrade soils have average strength, which might not be enough to withstand heavy vehicular load. The mineralogical characteristics support the poor geotechnical properties of the soils with the presence of smectite. Based on the FMWH standard, thick pavement is thus recommended for the highway that will enable a wide spread of loads over the sections of the weak subgrade and prevent soil deformation and subsequent pavement failure along the road. This information could be useful in remedial works on the rehabilitation of the failed sections of the road as well as guide future pavement design in similar terrain.

Acknowledgements

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Hydrogeology and Groundwater Potential of Dutsen Waikaduna, North Central Nigeria

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Abstract

A total number of 10 boreholes were considered and their pumping test data were analyzed and aquifer characteristics determined using Jacob's Straight line method. Data obtained from the measurement were plotted on semi-log and the changes in drawdown period of borehole pumping were found to obtain steady and unsteady flows in other instances. Vertical Electrical Sounding was carried out and four different layers with varying geological characteristics showing different degrees of weathering were seen; the top soil has a resistivity of 12.93 ohm m to 2783.2 ohm m, the weathered layer has a resistivity of 28.48 ohm m to 216.3 ohm m, the fractured layer was found to have a resistivity of 26.13 ohm m to 9779.3 ohm m while the bedrock layer has very high resistivity with infinite thickness. Rock samples were taken to produce geological map of the area with strikes and dips measured for lineament analysis which indicate a predominantly NE-SW trend of lineament density being the feasible zone of groundwater prospecting.

Keywords: Hydrogeology, Groundwater, Pumping Test Data, Aquifer, Drawdown, Nigeria.

Introduction

Due to the complexity of the geology, poor groundwater recharge, and the associated problems, groundwater tends to be highly limited in extent in crystalline basement rocks. The finding of suitable target in complex terrain requires the use of local knowledge; geophysics, (magnetic surveys, vertical electrical sounding VES, and electromagnetic methods, etc); detailed structural and geological mapping (e.g. field mapping, aerial photography and satellite imagery analyses); exploratory drilling etc.

Fracture-trace, or lineament analysis, is a powerful ground-water exploration tool. It depends on identifying faults and lineaments on aerial photographs or Landsat images and then locating them on the ground. They may be evident on high resolution satellite imagery and through geomorphic expression on topographic maps (Fetter, 2001).

Study Area Location

The study area is bounded by Latitudes 8° 15'00"E to 8° 30' 00"E and Longitudes 10°45' 00"N to 11°00' 00"N and covers an area of about 770km². It is located in the north-east part of Dutsen Wai Sheet 125 NE.

Materials and Methods

Determination of Aquifer Characteristics

Pump test data of ten (10) boreholes were used for the determination of aquifer characteristics. The ability of

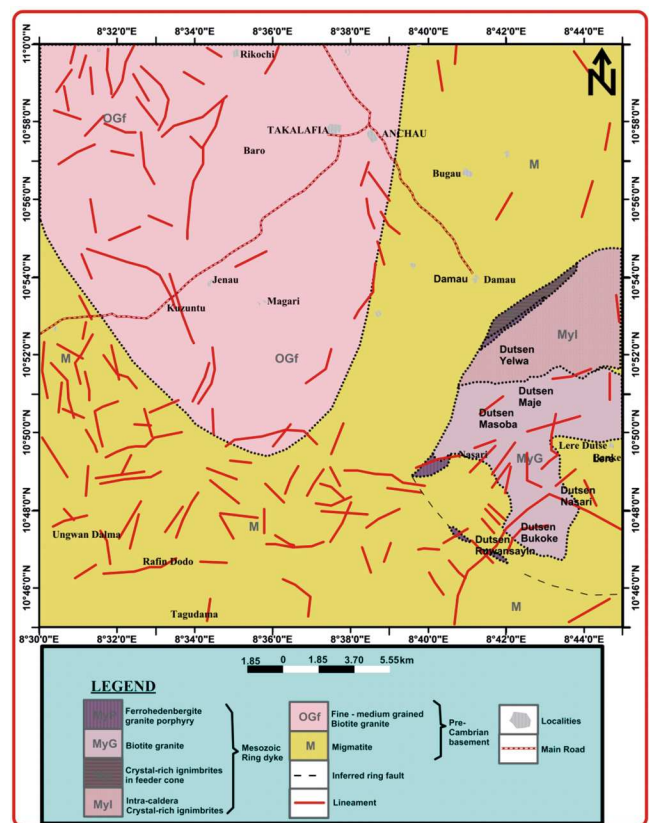


Fig. 1: Geological map of the study area showing lineaments

an aquifer to transmit water in economic quantities in wells depends on the aquifer characteristics and they include hydraulic conductivity (K), transmissivity (T) and storativity (S). The properties were determined using Jacob's straight line method (Jacob, 1947). Parts of the data obtained from the measurements were plotted on semi-log graph in order to observe the

changes in drawdown for a period of pumping of the borehole(Fig 2). The drawdown(s) in meters on the ordinate is plotted against time (t) in minutes. When there is significant change in drawdown during the period of pumping, the flow is considered unsteady, but if the change in drawdown is not significant (nearly horizontal line on the graph), the flow is regarded as steady flow. However, all the pump test data used for this study are of unsteady flow. Data used were obtained from Rural Water Supply and Sanitation Agency, Kaduna.

Transmissivity was determined from the relation:

$$T = \left[\frac{(2.30Q)}{(4\pi\Delta s)} \right] \times \left[\log_{10} \left(\frac{t_2}{t_1} \right) \right] \dots\dots\dots(1)$$

Where: T = transmissivity (m²/day),Q= discharge rate (rate of pumping, m³/day), Δs = change in drawdown; and t₂,t₁ = time (days) since pumping started.

Geophysical Investigation

A vertical electrical sounding (VES) of electrical

resistivity method was carried out using Ohmega resistivity meter with Schlumberger configuration. A total of ten (10) VES points were collected.

Results and Discussions

Hydraulic Properties of the Aquifers

The pump test results were obtained from the Rural Water Supply and Sanitation (RUWASA) Kaduna State. The aquifer properties of the boreholes were derived using the following procedure. A drawdown time graph was plotted using the data obtained from the pumping test as shown below. Based on the values obtained (VES plots) and (Pumping test data), Transmissivity, Hudraulic Conductivity and Specific Capacity of the various boreholes in the 10 locations were determined.

Geophysical Results

The aim of Vertical electrical Sounding is to determine

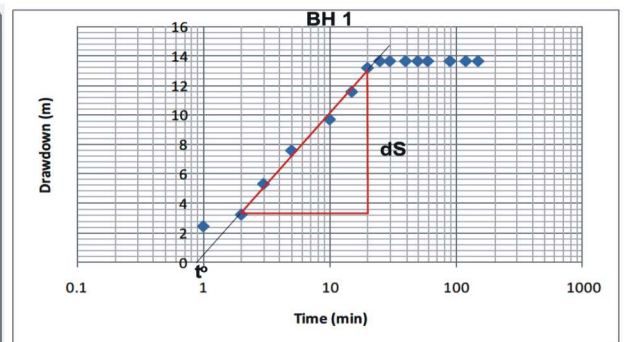
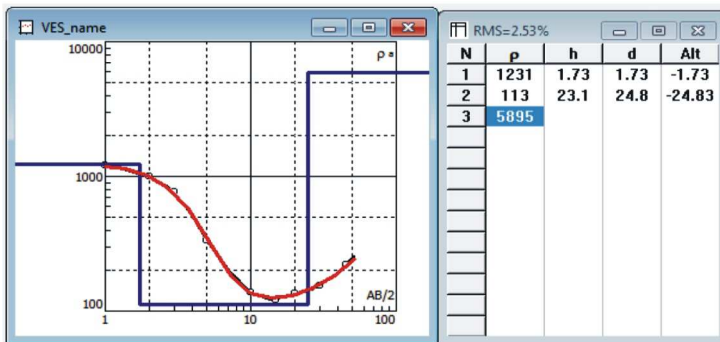


Fig. 2: VES and drawdown VS Time Plots

Table 1: Locations and Hydraulic Properties of Various boreholes

S/N	Location	Transmissivity (m ² /day)	H.C. (m ³ /day)	S. Conductivity (m ² /day)
1	U.Goboruwa	0.314	2.934	9.14X10 ²
2	Hayin Ade	0.216	2.72	9.12X10 ⁻²
3	Ungwanku da kudumi	0.418	2.66	9.62X10 ⁻³
4	Sabo gari wudu	0.717	3.10	9.11x10 ⁻²
5	Ashema	0.611	3.72	9.24X10 ⁻²
6	Kawankari	0.224	3.44	8.16X10 ⁻²
7	Rafinbusa	0.819	4.26	9.16X10 ⁻²
8	Ungwarkaji	0.518	3.16	9.16X10 ⁻²
9	SabongariKuzuntu	0.618	2.88	9.21X10 ⁻²
10	UngwarWorobusa	0.336	2.981	9.24X10 ⁻³

1. Transmissivity
T=2.3Q/4πΔs; T= transmissivity, Q=discharge, s=drawdown
2. Hydraulic Conductivity
k = T/b; where b =aquifer thickness
3. Specific Capacity of a well is the ratio of the yield to its drawdown usually expressed in m³/day/m.
Specific capacity =Q/s

the different geoelectric layers in the subsurface, the aquifer units and their characteristics, as well as general hydrogeological condition. Four different layers were obtained from the VES points in the study area. They have varying geologic characteristics, showing different degree of weathering and other secondary porosity in form of fracturing of the bedrock. Plots of calculated apparent resistivity in ohm-m against electrode spacing with geoelectric sections showing resistivity and depth of the subsurface layer are as presented above (VES and Drawdown vs Time Plots) and the pumping test data. These layers could be grouped as follows:-

Top Soil

This is a surface dry layer of high resistivity that ranges from 12.93 ohm m to 2783.2 ohm m. Its thickness varies from 1.12m to 11.5m. The low resistivity end is diagnostic of sandy clay and very high resistivity could be a fresh basement.

Weathered Layer

This layer is thought to be highly decomposed crystalline rock. The resistivity value ranges from 28.48ohm m to 216.3 ohm m. The thickness varies from 5.29m to 54.34m. According to Dan-Hassan and Olurenfemi (1994), it consists of clayey sand/ sandy clay layer. The layer is highly decomposed by weathering to form sand and clayey sand depending on the local variation of the mineralogy. However, this layer is believed to be the regolith. Hazell *et. al.*, (1992) made it clear in the geophysical assessment of crystalline aquifers that, resistivity values reflects the preferred lithological range, high value indicates a granular regolith, and intermediate values indicating silty clayey regolith. Dan-Hassan and Olurenfemi (1994) identified this layer to be the major aquifer unit. If the depth of weathering is sufficiently thick as exhibited by most of the VES points in the study area; the weathered mantle could contain water in storage large enough to produce a successful borehole.

Fractured Layer

The resistivity value ranges from 26.13 ohm m to 9779.3 ohm m. The thickness varies from 23.18 m to 68.60 m. The fractured zones are difficult to detect geophysically, unless it is of greater thickness. Where the fractured zone is saturated, a high groundwater yield can be obtained from borehole penetrating such a sequence.

Bedrock

This has a very high resistivity with infinite thickness. But where it is fractured and saturated; the resistivity reduces. It is not a source of groundwater unless fractured. However, when the shape of the VES curves approaches a very steep gradient of about 45° (steady increase in resistivity), it may indicate a fresh basement rock without fractures.

Therefore the geophysical result shows that, generally the area has a good potential for groundwater development, especially in places underlain by porphyritic granite because of its thick regolith and extensive fractures.

Orientation/Trend Analysis

The trend of the lineaments was computed directly from the endpoints earlier obtained using the linears - rose diagram - endpoint feature of Rockworks 15 software.

Lineament Analysis and Groundwater Potential

Structural trends such as discontinuity can be detected in many forms, such as faults, joints, bedding planes or foliation (Mogaji *et al.*, 2011), and may be detected in the form of a lineament using remotely sensed data such as conventional aerial photographs and satellite imagery. According to Anudu *et al.*, (2011) the commonest method used to calculate lineament density is based on the number of lineaments per unit area (number/km²), or the total length of lineaments per unit area (km/km²) or combining both. The NE-SW is the major trend of lineaments in the area. The zones of high lineaments density could probably be feasible zones for groundwater prospecting in the study area.

Conclusion

This study reveals that the rocks in the area have undergone relatively deep weathering and fracturing producing various thicknesses of the soft overburden and fractured crystalline aquifers. The depth of the water table is highly variable in the area, and from the elevations mapped and studied in detail, the water level in wells coincides with the water table. Its pattern is usually a subdued replica of the surface topography, reaching its highest elevations beneath the watersheds and then flows towards the valleys into streams.

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Soil Variation Characteristics and Foundational Footing Assessment: Case Study of Okeho Bridge, Oyo State, South-Western Nigeria

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Abstract

Site investigation to re-construct Okehobridge was done due to large vehicular traffic that navigates the area associated with past sand mining and dredging activities. Moreso, the existing dilapidated bridge can no longer serve the teaming vehicular population. Geotechnical field work entailed boring trial/test pits in the area using the posthole hand auger. Standard Penetration Test was carried out for great insight into the lithological facies of the area. Geotechnical laboratory analysis was done to characterize the salient features that determined the load carrying capacity of the bridge. The findings have revealed a lithological profile at Okeho abutment of reddish-brown lateritic soil at 0-6.55 m with SPT of 11, brownish micaceousilty fine sand at 6.56-7.15 m with a corresponding SPT of 12 and Greyish micaceousilty sand at depth interval of 7.15-7.75m with an SPT of 24. Moreover, at mid-stream, 2 main lithological facies recorded were Greyish micaceous weathered silty sand and Greyish fresh basement rocks. At Iseyin abutment, Reddish brown lateritic soil at 0-2.4 m with SPT of 13, Brownish/mottled micaceousilty fine sand at 2.4-3.6 m with a corresponding SPT of 14, Greyish micaceous sandy clay at depth interval of 3.6-4.2m with an SPT of 40 and Brownish micaceous fine sand at 4.2-6.0 m. Bulk density varied from 1680-2060 Kg/m³, while specific gravity varied from 2.57 to 2.69 Ga. Safe bearing capacity for pad and strip footings were measured and ranges from 55-195 KN and 60-205 KN respectively. Allowable pile load capacity recorded across Okeho abutment was 400-900 mm with laboratory and SPT safe working pile loads of 240-1056 KN and 1998-4646 KN respectively. Iseyin abutment measured pile diameter was 400-900 mm with laboratory and SPT safe working pile loads of 197-863 KN and 1938-4360 KN respectively. Construction of modern concrete reinforced Okehobridge is feasible to ease the vehicular movement across the area with recommended safe working pile load of between 197 to 4360 KN.

Keywords: Lithological facies, Abutment, Atterberg's limit, Specific gravity and Footing

Introduction

Site investigation for all superstructural like dams, roads, bridges, skyscrapers, railroads, tunnels etc should include a vivid address of salient information of site suitability, design, construction, contamination and materials for the load carrying capacities. (Ibrahim et al, 2020). It is as such that the load carrying capacity of Okehobridge in Oyo state was conducted. Bridges are categorized based on their characteristics including structural type, structural length, number of spans, maximum span length, skew angle, construction materials and bearing type (Wael et al, 2016). Okeho existing bridge was thus analysed for its superstructure, bearing, substructure and other critical aspects to facilitate a better site evaluation for the construction of a modern bridge. It was observed during this investigation and among others that existing bridge does not contain box girders, there is slight lateral movement under traffic loading, little earth tremor can collapse the existing bridge, as the wall is skewed, there is offset ie gaps at expansion joints, bearing seats under abutment end-diaphragm is not continuous and much more importantly, extensive erosion of soil close to the superstructure.

Location of the Study Area

The site of the proposed modern bridge is located at CH 12+230 and CH 14 +675 along Iseyin-Okeho road, Oyo state. The coordinate lies between Latitude North 8.02696, Longitude East 3.48473, elevation of 300.8 above the sea level and Latitude North 8.02427, Longitude East 3.48450. An elevation of 302.5 was recorded. Site investigation was carried along the Okeho-Iseyin abutments to facilitate the construction of the new modern concrete bridge in the area that will allow the safe passage of goods and services.

Materials and Methodology

The index properties were determined for preliminary soil assessment according to ASTM test methods. Shallow and deep borings were carried out along the Okeho and Iseyin abutments to maximum of 7.5 metres depth, with collection of geologic samples ie BH1S1, BH1S2, BH1S3, BH2S1, BH3S1, BH3S2, BH3S3 and BH3S4 for laboratory analyses and inferences for their load carrying capacity. Standard Penetration Test (SPT) was carried out to have great insight into the lithological facies of the area. This dynamic test as described in BS1377 is a measure of the density of the soil with

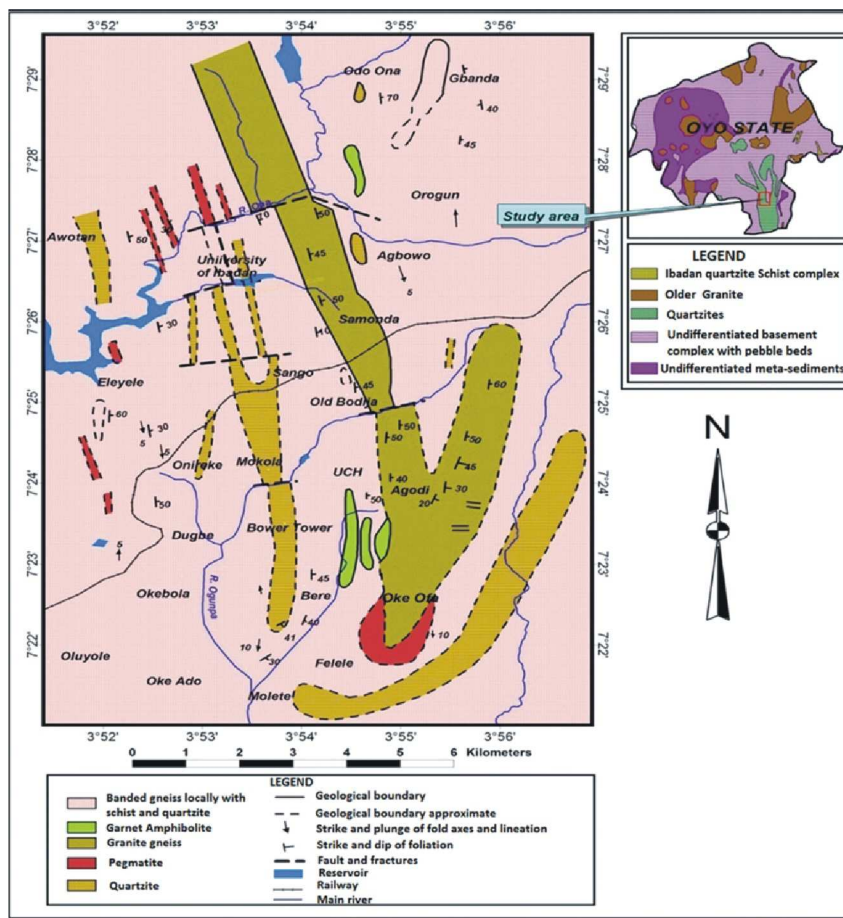


Fig. 1: Geological map of the study area showing metasediments and deeply seated rocks

recorded values. The test incorporates a small diameter tube with a cutting shoe known as the 'split barrel sampler' of about 650mm length, 50mm external diameter and 35mm internal diameter. The sampler was forced into the soil dynamically using blows from a 63.5kg hammer dropped through 760mm. The sampler was forced 150mm into the soil then the number of blows required to lower the sampler each 75mm up to a depth of 300mm recorded. This is known as the "N" value. For coarse gravels the split barrel was replaced by a 60 degree cone.

Field Work:

Field work was carried out at the location to get great insight of the nature of the area (Figs. 2 and 3). More importantly, collection of samples and location of borrow materials that may be useful for construction materials were mapped out. During the field work, it was established that the existing bridge is a composite one with H-steel beam, width of 4.5m and maximum depth of foundational footing to be 3.75-6.3 m.

The existing Okeho bridge (Fig. 3) has no box girders, quite narrow for vehicular traffic as such cannot withstand large traffic and can collapse in no distant time, especially with very little tremor and currently its wall is largely skewed with offset ie gaps at its expansion joints. Large water erosion has eaten deep into the existing foundation footing. Surrounding bushes already encroaching the bridge passage way with an impending threat to vehicular traffic (Fig. 3).

Data Analysis and Discussion

The lithological facies encountered along the Okeho abutment include reddish brown lateritic soil, brownish and grayish micaceousilty fine sand and grayish fresh basement rock of the area (Fig. 4 and Table 1). This abutment has undergone intense weathering activity as such need for excavation to the fresh basement rock to form a slope that the foundation of the bridge will sit on. This stratum of fresh basement rock is capable of sustaining the proposed foundation pressure without exceeding the allowable settlement. With a depth interval of 7.75 m and above for the fresh basement and



Fig. 2: Field investigation equipment set-up at Okeho and Iseyin abutments



Fig. 3: Skewed and dilapidated existing Okeho bridge

a refusal SPT after 24 at the preceding layer, then the lithological facie of the dam slope will be adequate for the Okeho bridge stability.

Furthermore, at the Iseyin abutment, the lithological facies recorded were reddish brown lateritic soil at 2.4m, grayish/mottled micaceous sandy clay at 3.6m-4.2m and brownish micaceous fine sand at 4.2-6.0m that served as the point of refusal for the SPT (Fig. 5 and Table 1). This clearly showed a competent layer of micaceous sand before the basement rock. Midstream, Greyish micaceous weathered silty sand was recorded (Table 1). With an interval of Standard Penetration Test of 13-40 N values (Table 1) before the refusal, then it's a valid point that has shown that the foundational slope on which the footing will rest on is good and competent to withstand the strength of the structural bridge upon construction.

Bulk Density and Specific Gravity

Eight samples from the Okeho and Iseyin abutments were subjected to density and specific gravity tests of

which the result showed BH1S1, BH1S2 and BH1S3 of the Okeho abutment at various depth intervals of 0-6.15, 6.15-7.15 and 7.15-7.75 meters (Table 2) revealed a gradual reduction in bulk density and varied 2050, 1800 and 1770 Kg/m³ respectively. This is an indication of gradual reduction in this property from shallow to deeper part of the abutment that signifies a good load carrying capacity. A similar trend was established at Iseyin abutment where there was a gradual decrease from 2030-1830 Kg/m³ across studied samples of BHSS1-BH3S4 (Fig. 5).

Having made the necessary calculations and determined the safe bearing capacity of the ground using the collected samples of the Okeho area with regard to the proposed foundation it is concluded that the ground strength is adequate relative to the foundation pressure.

Computed Safe Bearing Capacity of Bridge Footing

Pad and strip foundational footings were estimated for the encountered soil samples at different depth intervals to transmit the load from the bridge to the ground.

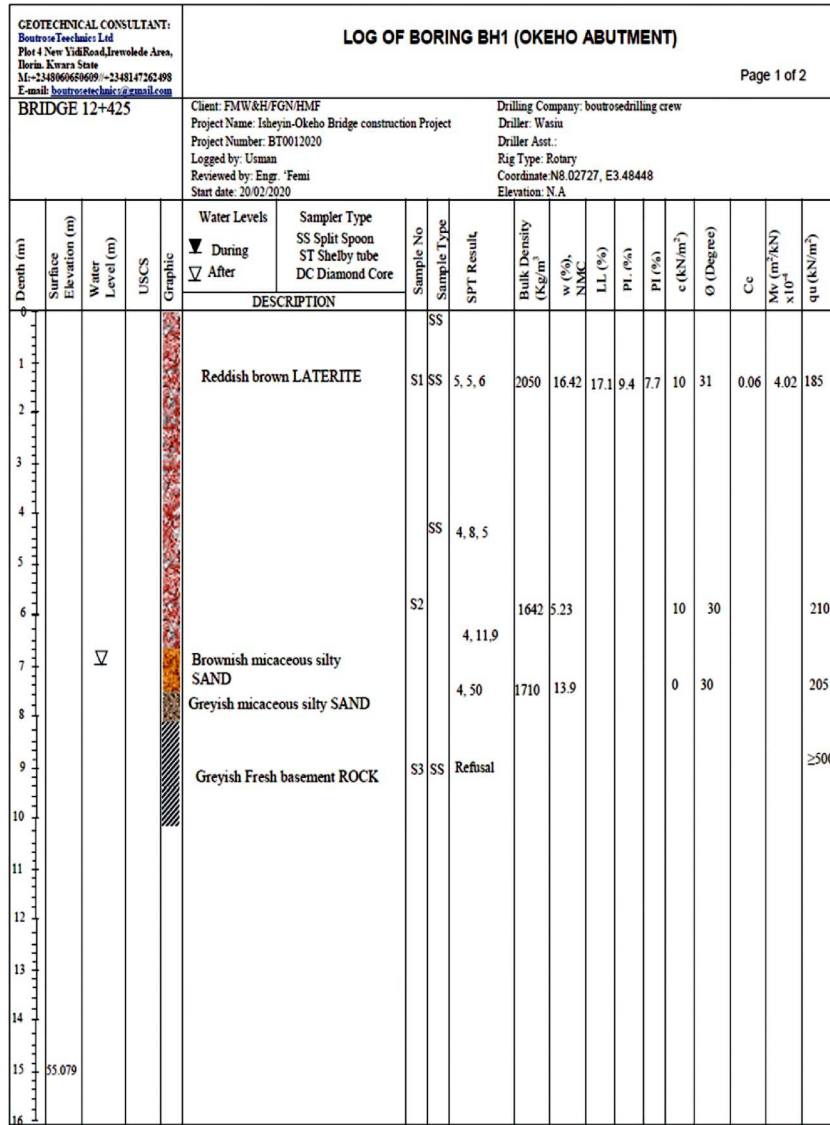


Fig. 4: Lithological facies of log boring at Okeho abutment

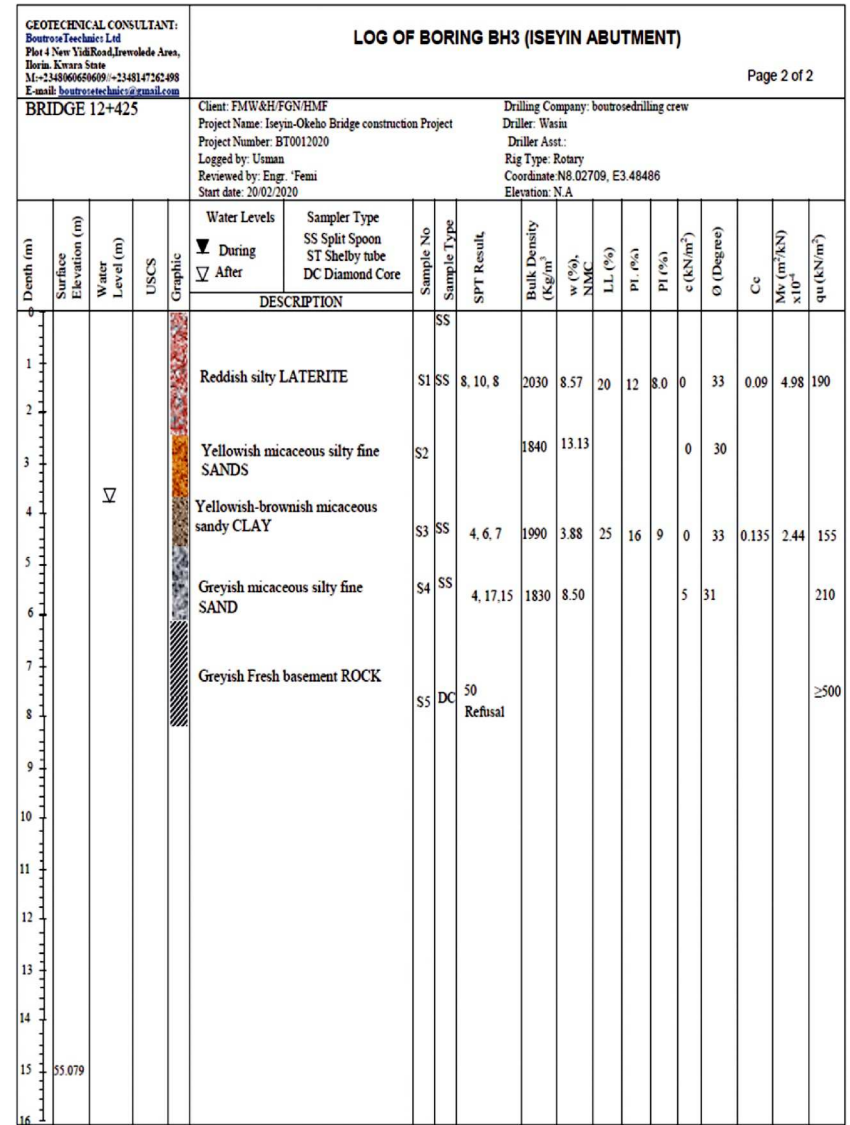


Fig. 5: Lithological facies of log boring at Iseyin abutment

Table 1: Summary of the subsoil encountered at the Bridge site Km 12+425

Bridge	Boring Locations	Geologic Materials Encountered	Depth (m)	Corresponding SPT-N Values
KM 12+425	OKEHO Abutment end (12BH1)	Reddish brown lateritic soil	0-6.55	11
		Brownish micaceoussilty fine sand	6.55-7.15	12
		Greyish micaceoussilty sand	7.15-7.75	24
		Greyish fresh basement	7.75-∞	Refusal
	MID STREAM	Greyish micaceous weathered silty sand	2-2.5	
		Greyish fresh basement	2.5-∞	18
	ISEYIN Abutment end (12BH3)	Reddish brown lateritic soil	0-2.4	13
		Greyish/mottled micaceous sandy clay	2.4-3.6	14
		Greyish micaceous sandy clay	3.6-4.2	30
		Brownish micaceous fine sand	4.2-6.0	Refusal
Greyish fresh basement rock		6.0-∞		

Table 2: Estimated bulk density and specific gravity across the abutments

Bridge site	Boring and sample location	Depth (m)	Bulk Density (Kg/m ³)	Specific Gravity (Ga)
Okeho abutment	BH1S1	0-6.5	2050	2.67
	BH1S2	6.5-7.15	1800	2.60
	BH1S3	7.15-775	1770	2.69
Mid-Stream Iseyin Area	BH2S1	0-1.6	1680	2.66
Iseyin abutment	BH3S1	1.6-2.4	2030	2.57
	BH3S2	2.4-3.6	1840	2.68
	BH3S3	3.6-4.2	1990	2.63
	BH3S4	4.2-6.0	1830	2.67

Table 3: Estimated safe bearing capacity of the Dugbe-Osin from selected samples

Safe bearing capacity												
Depth (m) / Samples	Square (Pad) footing						Strip footing					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
BH1	100	130	165	195	225	215	90	120	155	185	215	205
BH2	N.A	N.A	N.A	N.A	N.A	N.A	-	-	-	-	-	-
BH3	55	90	125	125	155	210	75	110	140	160	185	210

Conclusion

Greyish micaceoussilty sand, reddish brown lateritic soil and others are main lithological facies recorded.

The area is adequate relative to the foundation pressure of the ground area and will not be overstressed by the proposed bridge load to warrant pile foundational.

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Evaluating Erodibility Potential and Gully Intensity of Some Gully Slopes in Ezimo and Onuiyi Areas of Enugu State Using Limit Equilibrium Simulations

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Abstract

Erodibility and gully intensity of a soil is a function of the stability of gully slopes which is highly dependent on its geotechnical properties. This work comprises of detailed mapping of 50 gully slopes, geotechnical assessment of 14 samples, and stability analysis of the studied slopes. All geotechnical analysis followed the American Society for Testing and Materials (ASTM) applicable procedures. The laboratory results were analyzed using stability models carried out with the GeoStudio[®] software suite. Evidence has shown that the study areas experience scouring and slumping of gully walls. The geotechnical result revealed that the gully slopes are predominated by sand. Ezimo and Onuiyi erodible soils comprise of non-plastic silt and low to medium plastic clays respectively. Although soils in Onuiyi displayed better geotechnical properties than Ezimo, both soils are poorly compacted and have low shear strength. This suggests that the materials are susceptible to shear failure under stress. Stability analyses revealed critically stable gully slopes during the dry season, which is predisposed to instability (failure) during the rainy season, especially with an increase in slope height and pore-water pressure. Simulation results also demonstrated that potential sliding mass could significantly increase with an upsurge in pore-water pressure. Poor soil properties and fierce surface runoffs are the major causes of gully advancement in the study areas. Hence, the authors proposed gully walls' stabilization through proper runoff evaluation and appropriate drainage control methods, and if possible, runoff harvesting and treatment using catchment pits should be encouraged for clean water supply within the area.

Keywords: Erodibility; Slope stability; Gully slope; Runoff; Pore-water pressure; Slope failure

Introduction

Gully erosion is the removal of soil particles by surface runoff cutting and creating downstream channels and subsequently the collapse of borders. It is the final, most destructive, devastating, and difficult stage of erosion to manage (Fig 1). Authors like Tang (2004); Zheng *et al.* (2004); Jing *et al.* (2005); Rahman *et al.* (2009) noted that it is a major environmental, economic, and societal problem which causes intense land degradation, soil productivity loss, and posing serious threats to slope stability, health of society, natural resources and the environment at large (Fig. 1). It can be caused by natural and anthropogenic factors; climate change, tectonic forces and base level drop (Istanbulluoglu *et al.*, 2005). Gully intensity is greatly determined by the erodibility potential of the soil, whereas the latter is controlled by the soil properties. Soil properties such as soil structure, stability, infiltration rate, and morphology can be greatly affected by changes in climatic conditions (Karmakar *et al.*, 2016), and this plays a great role in the formation of gullies.

The stability of gully slopes determines the intensity of a gully site. Slope stability is the state of inclined soil or rock slopes to be able to withstand or undergo movement. Slope movement occurs for an unstable slope. Failure of a gully slope occurs when shear stress exceeds shear resistance leading to a decrease in its

factor of safety. With increase in rainfall infiltration, surface-subsurface hydrology of gully slopes changes through unsaturated-saturated interactions that lead to gully slope failure. Crosta and Frattini (2008) and Igwe and Fukuoka (2014) noted that such infiltration weakens slopes by reducing suction and strength. Prolonged rainfall causes saturation of soil pore spaces leading to runoff, which could be either surface runoff or subsurface flow. The later can cause saturation of gully walls leading to slumping of both gully heads and sides (Rose *et al.*, 2014). Therefore, slope stability analysis of gullies will aid in evaluating the erodibility of soils and how intense the gully can grow.

Materials and Methods

The Study Area

The study area is located in Nsukka and Udenu Local Government Areas of Enugu State, southeastern Nigeria. It is located within latitude 6°48'N and 6°56'N and longitude 7°23'E and 7°34'E (Fig. 2) and lies within the Anambra Basin of the Lower Benue Trough. It is situated on the Udi-Nsukka cuesta. The area falls within the Southeastern Guinea Savannah (Rainforest-Savannah) vegetation type, with typical equatorial climate (Nwajide, 2013; Emeh and Igwe, 2017; Igwe and Egbueri, 2018). Erosion sites were observed to mostly occur within the Ajali and Nsukka Formations.



Fig. 1: Gully erosion ravaging the area

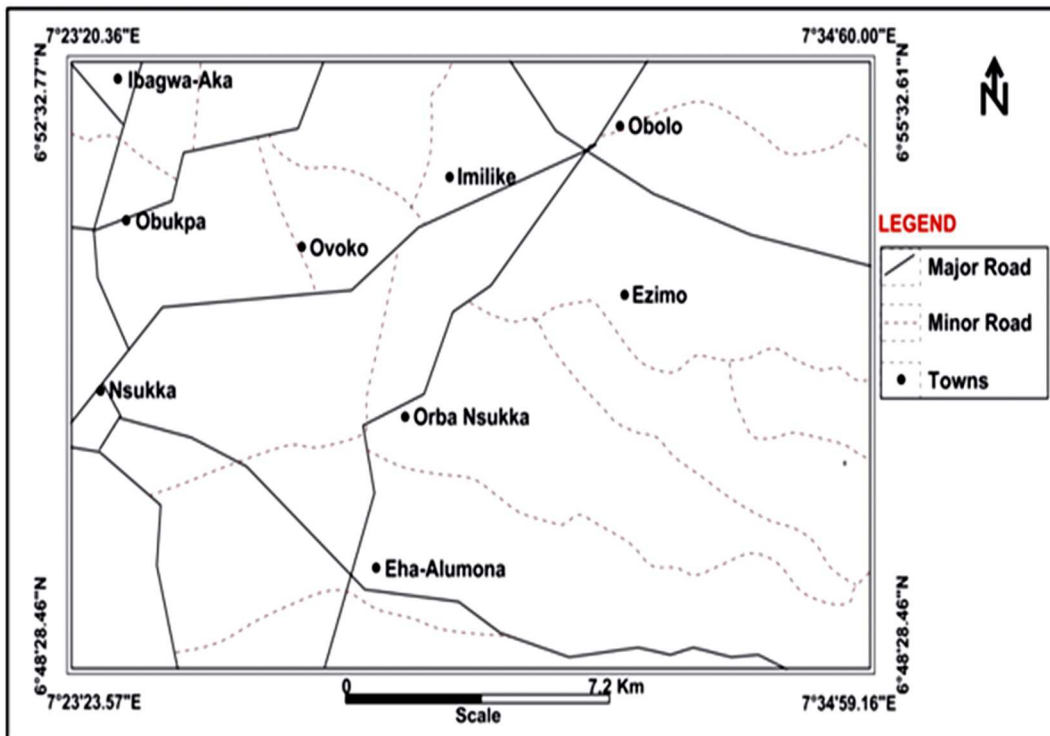


Fig. 2: Location and accessibility map

Methodology

Fifty (50) gully slopes were studied in the Ezimo and Onuiyi area of Enugu State. A total of 14 disturbed samples were collected from gully slopes (7 for each section) into an air-tight polythene bags. These samples were analyzed for their geotechnical properties in the Geology and Civil Engineering Departments of the University of Nigeria, Nsukka. All tests followed the American Society for Testing and Materials (ASTM) applicable procedures. Stability simulations using the limit equilibrium method (LEM) were employed in laboratory data analysis. The models were produced with the GeoStudio® software suite using Morgenstern-Price simulation approach.

Results and Discussion

Index and geotechnical characteristics

Results from gradation analysis (Table 1) show that percentage of gravel, sand, and fines range from 0, 69.8 – 95.17, and 4.83 – 30.2 for Ezimo and 0 – 0.6, 87.04 – 92.8, and 7.2 – 12.96 for Onuiyi respectively (Fig. 3). These values imply that the gully slopes are technically devoid of gravels which cause a reduction in the edibility of soil material. The slopes are predominated by sand, and the low proportion of fines acting as soil binder implies that the soil materials are susceptible to gully processes and occurrence (Dandi and Ray, 2014; Nebeokike *et al.* 2020).

Table 1: Index and geotechnical properties of soil

Parameter	Onu 1	Onu 2	Onu 3	Onu 4	Onu 5	Onu 6
GRAVEL (%)	0.6	0	0	0	0	0
SAND (%)	91.34	90.34	90.23	92.8	90.76	87.67
FINES (%)	8.06	9.66	9.09	7.2	9.24	12.32
Cc	1.13	0.97	1.07	1.1	1.31	2.01
Cu	3.13	3.41	3.12	3.06	3.54	5.25
USCS soil classification	SP-SC	SP-SC	SP-SC	SP-SC	SP-SC	SC
LL (%)	38	34	30	30	34	33
P L (%)	21.02	22.84	21.5	20.09	18.71	17.66
PI (%)	16.98	11.16	8.5	9.91	15.29	15.34

C. stable means critically Stable; F. stable means fairly stable

The results from natural moisture content unveil a range of 8.3 – 29.1% and 9.5 – 32.4% for Ezimo and Onuiyi respectively, suggesting a moderate water retention ability of the slope materials. The Atterberg's limits revealed that the analyzed slope samples have non-plastic fines, with samples Ezi 1 and 5 having a liquid limit of 19% and 28%. The liquid limit (LL), plastic

limit (PL), and plasticity index (PI) ranged from 30 – 38%, 14.83 – 22.84%, and 8.5 – 16.98% respectively for Onuiyi, implying low to medium plasticity (Fig. 4). These low values of Atterberg limits imply a possible loss of bottom soil and slumping of upper soil layers and subsequently, the collapse of the gully walls (Deng *et al.*, 2016).

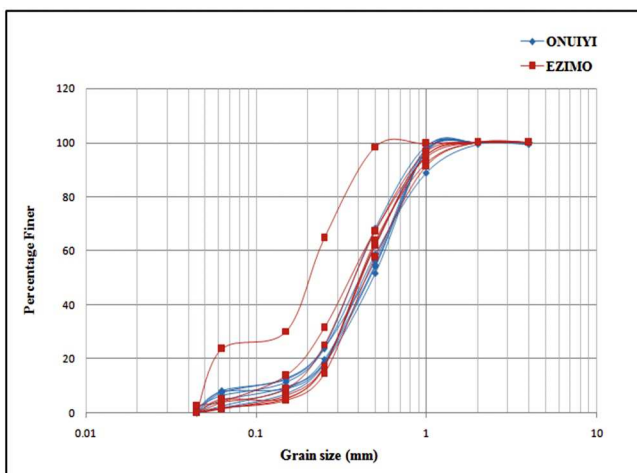


Fig. 3: Particle size distribution curve

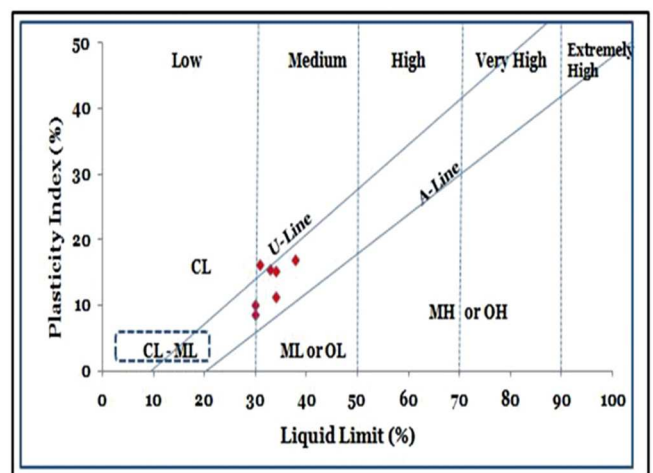


Fig. 4: Soil plasticity chart

Compaction test revealed that optimum moisture content (OMC) of the soils from Ezimo and Onuiyi ranged from 16.5 – 19% and 17 – 19.6% whereas the maximum dry density (MDD) ranged from 1.6 – 1.71 g/cm³ and 1.71 – 1.89 g/cm³ respectively (Fig. 5). The compaction result implies that the slope materials have high OMC and moderate MDD, signifying loose soils capable of easy detachment and transportation when acted upon by erosive agents. The loose nature of the soils has a greater influence on their water retention ability and hydraulic conductivity, therefore, will require very little force to detach and transport the particle (Igwe and Egbueri, 2018; Nebeokike *et al.*, 2020).

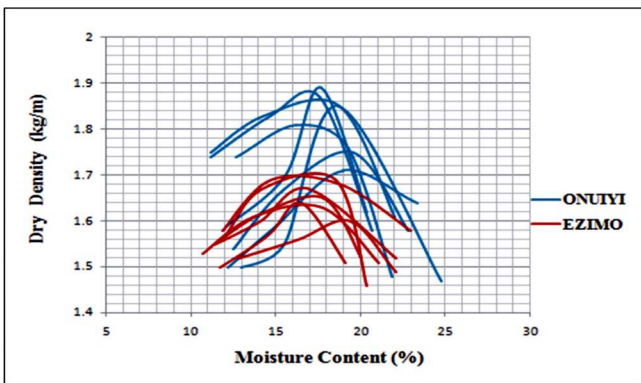


Fig. 5: Soil compaction curves

The shear strength characteristics revealed that cohesion ranged between 4-18 kPa and 11-18 kPa in Ezimo and Onuiyi soils while angle of internal friction ranged from 13-31° and 19-31° respectively (Fig. 6). These values suggest that the materials have very low cohesive strength and are susceptible to shear failure under stress.

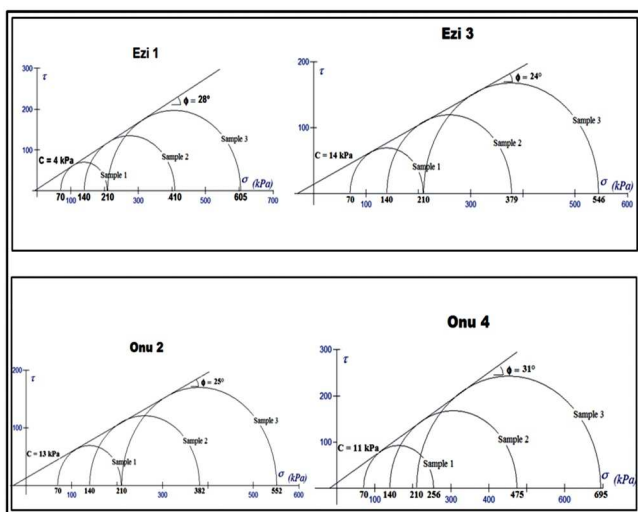


Fig. 6: samples' representative failure envelopes

Slope Stability Simulations

Results obtained from the slope simulations showed that pore-water pressure and slope height are major factors affecting the stability of the studied slopes. The range of factor of safety (FoS) decreased from 3.92-0.80 during the dry season (with little or no pore-water pressure) to 2.35-0.57 when pore-water pressure is applicable during the rainy season (Table 1). Moreover, the potential slide mass increased from 56.92 m³, 102.3 m³, and 62.71 m³ to 140.34 m³, 110.11 m³, and 64.85 m³ respectively with the addition of pore-water pressure (Fig. 7). This implies that the presence of pore-water pressure undermines the stability of gully slopes. Also, slopes of higher height were observed to be unstable stable (FoS = 0.80) without the influence of pore-water pressure during the dry season, and thus suggestive of high erodible soils and gully intensity.

The decrease in factor of safety with a corresponding increase in the potential slide mass with pore-water pressure applicable suggests the undermining of the gully slope with moisture influx. Therefore, at the peak of the rainy season, the gully slopes become unstable, experiencing minor landslide in Ezimo and scouring of gully walls in Onuiyi. Gully slopes in Ezimo are less stable to those in Onuiyi due to poorer engineering characteristics, hence are more erodible with higher gully intensity. However, stabilization the gully walls and finding appropriate mitigation measures for both erosion sites are needed to curb the gulying process and maintain the earth's balance.

Conclusions

The present study evaluated the erodibility and stability of gully walls in some parts of Enugu state. Soil index properties showed that both areas are dominated by sand. Soils in Ezimo were observed to be non-plastic whereas Onuiyi soils showed low to medium plasticity. The geotechnical analysis revealed that poor geotechnical properties of soils in Ezimo contributed to high erodibility of materials and intense gully erosion in the area. Although soils in Onuiyi exhibited good geotechnical properties to withstand erosive agents, the area experiences advancing gully erosion orchestrated by fierce surface runoff in the area. Gully simulation models revealed that except for slopes higher than 20 m, gully slopes in the areas are fairly stable to very stable during the dry season. But, the stability range of the gully slopes drop to critical stability to instability during the rainy season. Although, gullies located within the Ajali Sandstone (Ezimo) are likely to be more

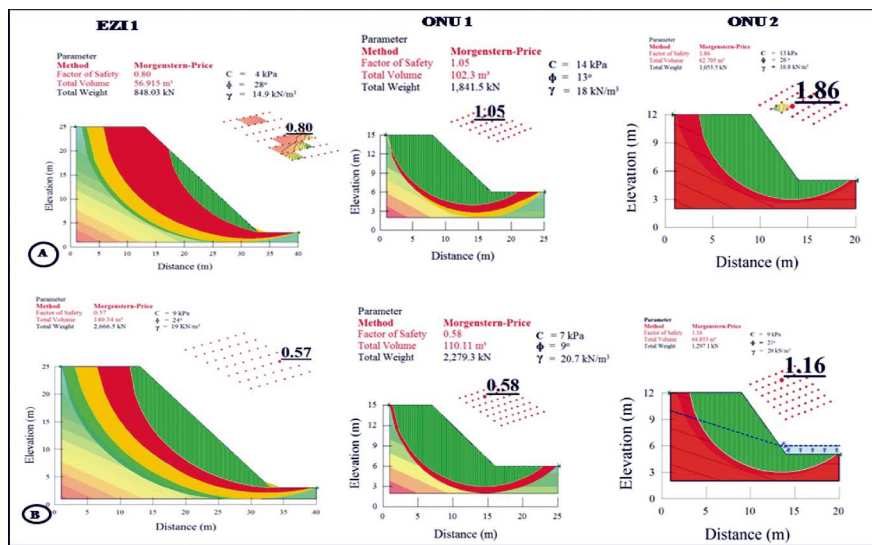


Fig. 7: Stability model of gully slopes (a) without pore-water pressure (b) with pore-water pressure

susceptible to erosion during both seasons, authors suggest stabilization of gully walls, proper runoff

evaluation and adequate drainage control measures as erosion mitigation plan for both areas.

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