

Assessment of Index Properties and Bearing Capacities of Soils for Infrastructure Foundations in Maleta, Northcentral Nigeria

*O.O. Owoyemi and M. O. Awojobi

Department of Geology and Mineral Sciences Kwara State University, Maleta, PMB 1530, Ilorin, Kwara State Nigeria

[*Corresponding author: e-mail: lade.owoyemi@gmail.com; ■ :+2347036761892]

ABSTRACT

Maleta is a fast growing suburb with new buildings springing up daily. There has been no documented research on the physical properties and foundation bearing capacity of the soil in this area. This research aimed at determining the suitability of this soil as infrastructure foundation. Bulk samples taken from two selected locations at varying depths of trial pits were tested for their index and shear strength properties using standard methods. Cone penetration resistance, California bearing ratio, compaction, consolidation and permeability characteristics were also assessed. The geotechnical properties determined varied significantly with depth except for specific gravity which did not vary significantly at $\alpha_{0.05}$ with depth. Soil samples from all pits consist mostly of poorly graded gravely sands with little fines. They contain medium to coarse grained sand fraction averagely above 85%. Penetration resistance obtained from cone penetration test ranged from 700 kN/m² to 950 kN/m². The average safe bearing capacity estimated for strip footing using a factor of safety of 3 at depth of 1 m was not less than 473 kN/m² anywhere in the study area. Samples from the two locations generally have good compaction parameters, medium to high permeability and low compressibility. The highest bearing capacities were associated with the lateritized basement top. This implies that the safest depth to place infrastructure foundations in the area is the depth where lateritized basement rock is encountered.

INTRODUCTION

Recently, cases of collapsed buildings in Nigeria has been on the rise. This calls for detailed geotechnical investigations of foundation soils so as to guard against re-occurrence of such ugly incidents. Material study of foundation soils to a large extent, serves as preventive measure for foundation failures. Nwankwoala and Warmate (2014), studied the foundation geotechnical properties of a site in Port Harcourt, Aduoye and Agbede (2014) used Terzaghi's equations to determine the bearing capacity of soil samples from Obafemi Awolowo University Campus. He found correlations between angle of internal friction and bearing capacity of the studied soils. Other researchers such as Ola (1988), Ogunsanwo (2002), Ige and Ogunsanwo (2009), Oyedele et al.(2011), Avwenagha et al. (2014) have worked on the geotechnical properties of foundation soils in Nigeria. However, there is presently no research on the physical properties and foundation bearing capacity of the soil in the study area. Maleta is thereabout the fastest growing village in the vicinity of Ilorin. This is due to the establishment of Kwara State University campus in this rural

area. While many new structures are springing up daily in the area, most of the building owners do not pay attention to the geotechnical properties of foundation soils on which they are building. Hence, some of this building are showing signs of distress while some have suffered partial collapse. This study therefore investigates some geotechnical properties of the foundation soil in Maleta using samples taken at varying depths and chainages from two selected locations.

Location of the area

The study area is located within Kwara State University Campus. Two locations with rapid infrastructural developments were delineated. The two locations are underlain by migmatitic-gneiss of the Migmatite-gneiss complex of Southwestern Nigeria (Geological Survey of Nigeria 1973). The area is on a watershed shedding its storm water into Apodu river as shown by Figure 1. The vegetation of Maleta area is that of woody savannah. The trees include locust bean, "dogonyaro", "iya", sheer butter, and some thorny shrubs. Table 1 shows the location coordinates of trial pits at the sampling locations.

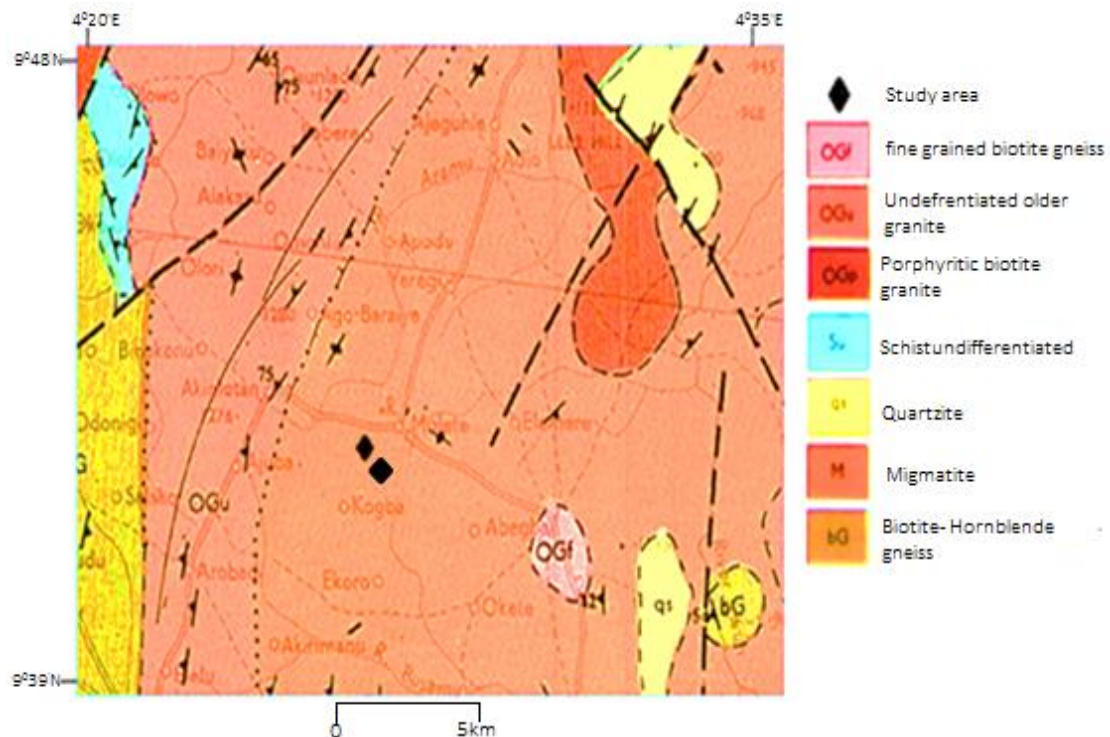


Figure 1: Geological map of Maleta area (Modified from Geological Survey of Nigeria 1973).

Table 1: GPS Coordinates of trial pits.

Pit no. (location 1)	latitude	longitude	Altitude (m)	Pit no. (location 2)	latitude	Longitude	Altitude (m)
Pit1 -1	080 43' 29.1"	0040 29' 002.5"	331	Pit 2 -1	080 43.421'	0040 29.532'	330
Pit 1-2	080 43' 29.0"	0040 29' 002.9"	332	Pit 2-2	080 43.413'	0040 29.541'	317
Pit1-3	080 43' 28.8"	0040 29' 003.1"	335	Pit 2-3	080 43.418'	0040 29.541'	332
Pit 1- 4	080 43' 28.7"	0040 29' 003.4"	333	Pit 2- 4	080 43.414'	0040 29.552'	315
Pit 1 - 5	080 43' 28.6"	0040 29' 003.7"	333	Pit 2 - 5	080 43.411'	0040 29.564'	305
Pit1- 6	080 43' 28.5"	0040 29' 04.0"	332	Pit 2 - 6	080 43.403'	0040 29.562'	316
Pit 1-7	080 43' 29.3"	0040 29' 04.3"	331	Pit 2 -7	080 43.410'	0040 29.577'	309
Pit 1-8	080 43' 28.1"	0040 29' 04.5"	328	Pit 2 -8	080 43.400'	0040 29.575'	316
Pit 1-9	080 43' 29.0"	0040 29' 04.0"	329	Pit 2 -9	080 43.404'	0040 29.583'	314
Pit 1-10	080 43' 29.5"	0040 29' 04.4"	327	Pit 2 -10	080 43.413'	0040 29.541'	317

MATERIALS AND METHODS

Ten trial pits were dug at each location. Disturbed samples were taken at intervals where changes in soil characteristics were observed up to the depths where lateritized basement rock was encountered. The shallowest pit was 4.5 m while the deepest was 1.65 m. Classification tests including sieve analysis, consistency limits, specific gravity and natural moisture content were carried out on samples taken from the trial pits. Shear box test, consolidation, compaction, California bearing ratio and permeability were also carried out on these samples while cone penetrometer test was carried out in the field.

Cone penetrometer test was performed using 20 tones capacity hand penetrometer. With this arrangement, it was possible to measure both the point and frictional resistance in the trial pits to the point of stiff resistance encountered at the lateritized basement top encountered at varying depths of either 0.4 m, 1.0 m or 1.65 m. The dial gauges used were calibrated prior to commencement of the tests. All laboratory tests were carried out using the British standard procedure 1377 of 1990. Modifications were however made where necessary. The bearing capacity parameters were obtained from shear box test while bearing capacities were calculated using Terzaghi's bearing capacity equation for shallow strip footing (Equation 1). The safe bearing capacities were calculated using a factor of safety of 3 and equation 2. The depths used in the calculation of the bearing capacities were generally higher than their width.

$$Q_u = c N_c + \gamma D N_q + 0.5 \gamma B N_\gamma \quad (1)$$

$$QS = \frac{Q_u}{3} + \gamma D \quad (2)$$

Where; Q_u ; Ultimate bearing capacity, QS; safe bearing capacity; c = cohesion of soil (kN/m^2); γ = effective unit weight of soil (kN/m^3); D = depth of footing (m). Values of bearing capacity factors: N_c , N_q and N_γ for different

angles of internal friction, ϕ were as proposed by Terzaghi.

RESULTS AND DISCUSSIONS

Classification Tests

This category of tests were carried out on samples taken from 20 trial pits at varying depths from the two locations. The results showed that these samples consist mostly of poorly graded gravely sands with little fines. The amount of medium to coarse grained sand fraction were averagely above 85% in the studied soils. At deeper depths however, the samples contained more gravel. Figure 2 shows some grading curves of the studied soils while Figure 3 presents the bar charts of their grain size distribution. Specific gravity ranged from 2.40 to 2.75 for location one, it ranged from 2.64 to 2.67 for the second location. For location one, natural moisture content ranged from 1.5% and 6.8%, while it ranged from 1.5% to 6.8% at location two. Only few samples with appreciable amount of fines and plasticity were tested for consistency limits because of the general granular nature of the soils in the study area. For location one, liquid limit ranged from 29% to 34%, plastic limit ranged from 18% to 25% while plasticity index averages 10.5%. The liquid limit for location two ranged from 32% to 34%, plastic limit ranged from 22% to 23.5% while plasticity index averages 10%. Casagrande plasticity chart classify soils from the two locations as inorganic to medium plasticity soil.

Most soil samples from both locations classify as A-2 -4 under the AASHTO classification system, rating as good subgrade materials. Bulk density, dry density, specific gravity and natural moisture content increases with depth. These parameters vary significantly $\alpha_{0.05}$ with depth with natural moisture content and dry density having the strongest correlation with increase in sampling depths as shown in Figure 4

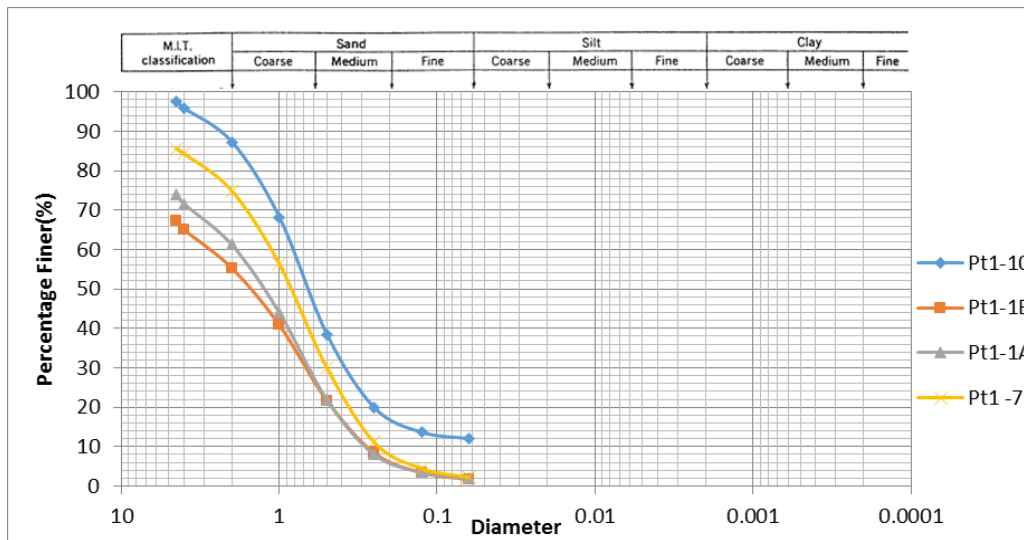


Figure 2: Grading curves of some soil samples from the study area.

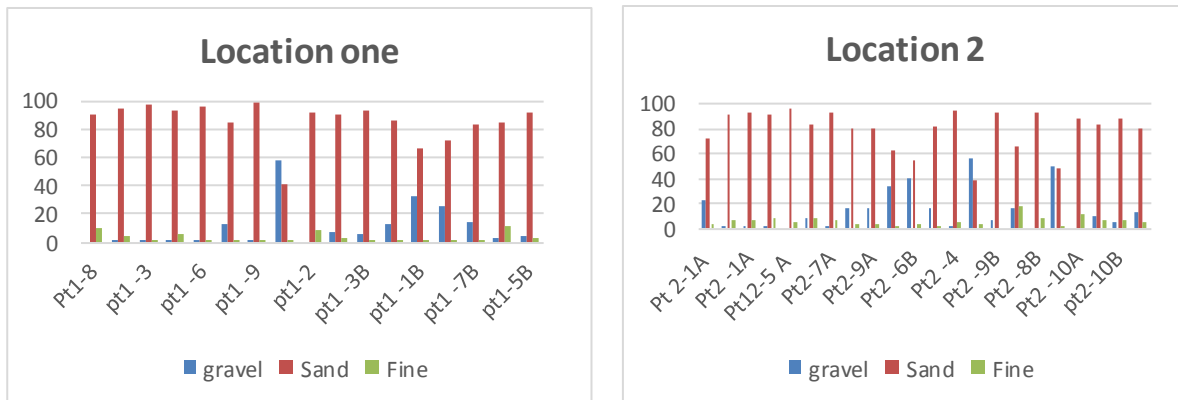


Figure 3: Bar showing the grain size distributions of samples from both locations.

The density of a coarse grained soil is a direct indication of its stiffness and strength. The bulk density characteristic of the sampled soils ranged from 1.44g/cm³ to 1.81g/cm³ in location one and from 1.55g/m³ to 1.88g/m³ in location two.

The dry density of soil samples from location one ranged from 1.36g/m³ to 1.76g/m³ while it ranged between 1.51g/m³ to 1.80g/cm³ for location two soil samples. These values are typical of dense sands.

Figure 5 shows a strong positive correlation between bulk density and dry density of the studied soils.

Cone Penetrometer Test

The CPT values for location one ranged from 750 kN/m² to 950 kN/m² while that of location two ranged from 700 kN/m² to 850 kN/m². The highest CPT values were gotten at the depths where laterised hardpans were encountered at various depths as shown in table 2. The top of the lateritized basement is generally favourable for foundation of structures because of its characteristic high CPT values in all the trial pits.

Shear Strength and Safe Bearing Capacity

The highest bearing capacity values were recorded between 1 m and 1.2 m which is made up of gravelly silty sand at the top of the laterised basement rock in different pits.

Bearing capacity increases with sampling depth with correlation coefficient up to 0.99. Table 3 and 4 show the bearing capacity values for both locations. Fig.6 shows how bearing capacity varies with depth in some trial pits. Using a factor of safety of 3, the safe bearing capacities calculated for strip footing ranged from 395

kN/m² to 2,421 kN/m². Although, these values are high enough for the development of normal functional buildings, it is advisable to place foundations on the lateritized basement for heavier buildings since depth to this weathered bed rock is shallow.

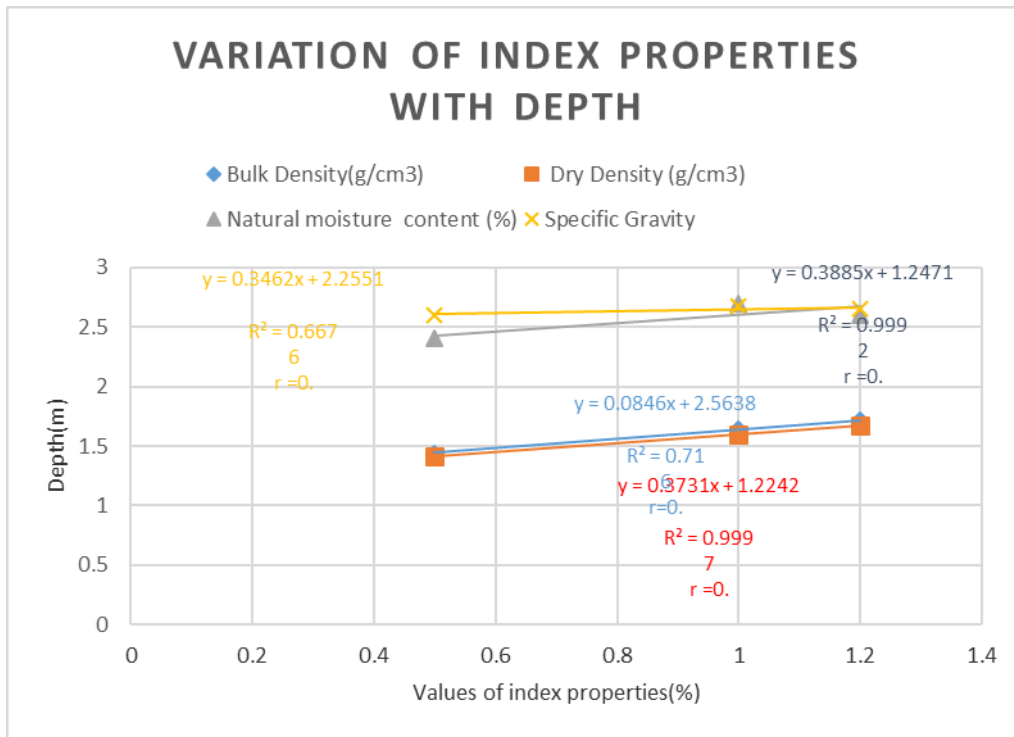


Figure 4: Variation of some index properties with depth.

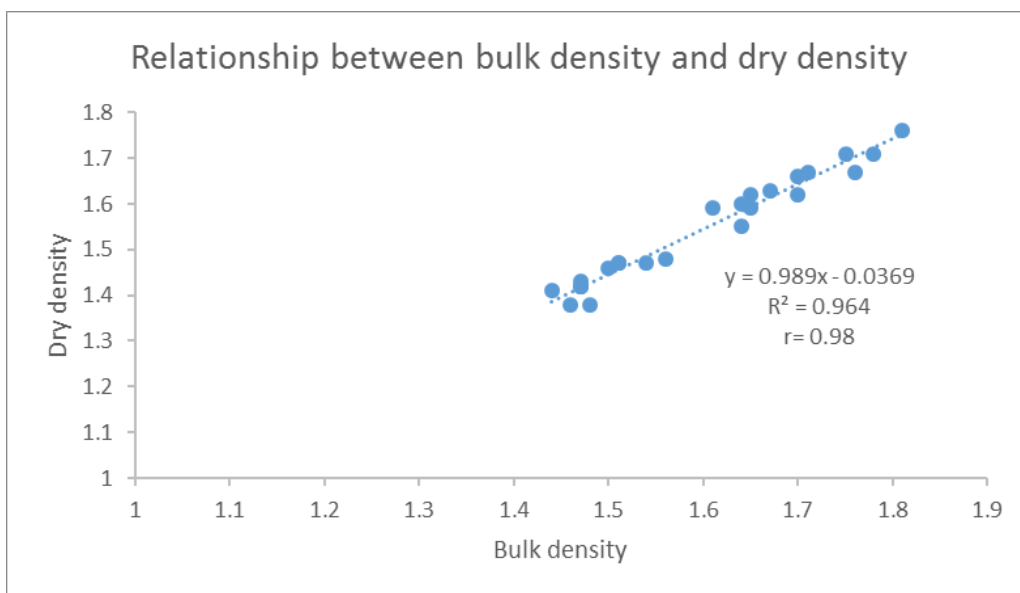


Figure 5: Relationship between bulk density and dry density.

Table 2: Result of Cone Penetrometer Test

Location 1			Location 2		
Test	Depth(m)	CPT Value (kN/m ²)	Test location	Depth(m)	CPT Value (kN/m ²)
PT1-2C	1.65	800	Pt2-1A	0.8	850
PT1-3C	1.2	850	Pt2-2A	0.4	800
PT1-4B	0.9	850	Pt2 -3A	0.4	750
PT1-5B	0.7	850	Pt2-4A	0.2	800
PT1-6B	0.8	850	Pt2-5A	0.2	850
PT1-7B	1.0	900	Pt2-6A	0.3	750
PT1-8C	1.3	750	Pt2-7A	0.5	900
PT1-9B	0.45	850	Pt2-8A	0.5	800
PT1-10B	0.55	950	Pt2-8B	1.0	850
			Pt2-9B	1.0	700
			Pt2-10A	0.5	750
			Pt2-10B	1.0	850

Table 3: Shear Strength and Safe bearing capacity values for locaion 1

Test location	Depth(m)	Angle of internal Friction (°)	Cohesion (kN/m ²)	Shear Strength (kN/m ²)	Safe Bearing Capacity (kN/m ²)
PT2 -1A	0.5	40	0	272	419
PT2- 1B	0.8	37	2	246	473
PT2-2A	0.4	38	4	257	448
PT2-3A	0.4	41	0	282	230
PT24A	0.3	41	5	287	404
PT2-5A	0.2	39	8	271	413
PT2-6A	0.3	38	0	257	188
PT2-7A	0.5	37	5	249	414
PT2-8A	0.5	38	0	257	435
PT2-8B	1.0	37	0	244	473
PT2-9A	0.5	37	5	249	429
PT2-9B	1.0	37	5	249	550
PT2-10A	9.5	40	5	277	551
PT2-10B	1.0	37	10	254	716

Compaction and Californian Bearing Ratio

Samples from the two locations have good compaction parameters when compacted using the modified AASHTO (American Association of State Highway Transportation Officials) method. For location one, the maximum dry density ranged from 2000 kg/m³ to 2006 kg/m³, while optimum moisture content ranged from 9.6 % to 11.6%. For location two, the maximum dry density ranged from 2000 kg/m³ to 2040 kg/m³,

while optimum moisture content ranged from 7.5% to 8.5%. This indicates that the studied soils require only little moisture increase to achieve good compaction.

Table 4: Shear Strength and Safe bearing capacity values for locaion 2

Test location	Depth (m)	Angle of internal Friction (o)	Cohesion (kN/m ²)	Shear Strength (kN/m ²)	Safe Bearing Capacity (kN/m ²)
PT1-1A	0.5	37	0	224	355
PT1-1B	1.0	42	0	292	992
PT1-1C	1.2	45	8	332	2,421
PT1-2A	0.6	30	0	187	108
PT1-2B	1.3	33	0	211	305
PT1-2C	1.65	33	0	211	413
PT1-3A	0.6	33	0	211	176
PT1-3B	1.0	33	0	211	257
PT1-3C	1.2	38	0	253	643
PT1-4A	0.5	34	0	219	165
PT1-4B	0.9	36	0	236	351
PT1-5A	0.5	38	0	253	312
PT1-5B	0.7	46	0	336	1,268
PT1-6A	0.5	36	0	236	302
PT1-6B	0.8	37	0	244	408
PT1-7A	0.5	32	4	236	226
PT1-7B	1.0	33	0	244	276
PT1-8A	0.5	37	5	239	382
PT1-8B	1.0	37	5	239	523
PT1-8C	1.3	37	5	239	688
PT1-9A	0.3	37	5	239	387
PT1-9B	0.45	38	5	258	483
PT1-10A	0.5	37	0	239	407
PT1-10B	0.55	38	0	253	395

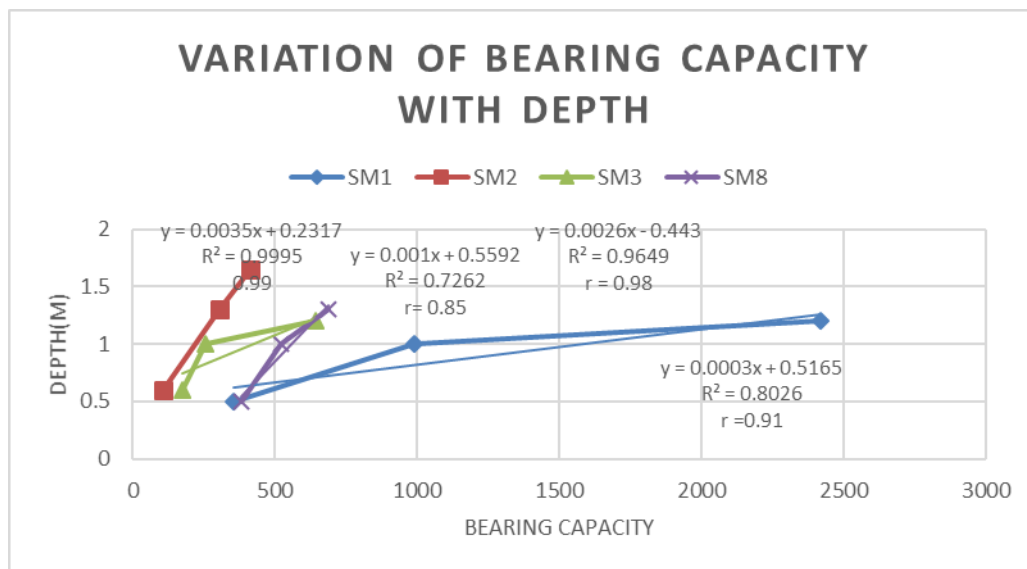


Figure 6: Variation of Bearing capacity with Depth

Consolidation and Permeability

The consolidation test shows that, coefficient of compressibility and coefficient of volume change are as low as $2 \times 10^{-3} \text{m}^2/\text{kN}$ and $1.2 \times 10^{-3} \text{m}^2/\text{kN}$ respectively for location one while estimated amount of settlement is 12 mm. The coefficient of compressibility and coefficient of volume change are as low as $2.5 \times 10^{-3} \text{m}^2/\text{kN}$ and $1.7 \times 10^{-3} \text{m}^2/\text{kN}$ respectively for location two while the estimated amount of settlement above the lateritized basement is 19 mm. Skempton and McDonald (1956) proposed a limiting amount of settlement up to 35 mm for shallow foundations on sandy soil. Although, the estimated amount of settlement for both locations are within acceptable limits, in order to avoid differential settlement due to spatial variation in top soil compressibility, it is safer to put infrastructure foundations on the lateritized basement rock. The top soil on the lateritized basement generally has high to medium permeability.

REFERENCES

- Aduoye, G.O and Agbede, O.A. (2014) Bearing Capacity Determination by Multiple Regression. *Journal of Multidisciplinary Engineering Science and Technology*, **1**(5): 285 – 286.
- Avwenagha, E., Akpokodje, E. G. and Tse, A. C (2014) Geotechnical Properties of Subsurface Soils in Warri, Western Niger Delta, Nigeria. *Journal of Earth Sciences and Geotechnical Engineering*, **4**(1): 89 – 102.
- British, Standard Institute (1990) Methods for Tests for soils for civil Engineering Purposes. B.S 1377.
- Ige, O.O and Ogunsanwo, O. (2009) Environmental geological assessment of a Solid Waste Disposal site: a case in Ilorin, Southwestern, Nigeria. *Nature and Science*, **1**(6): 53-62.
- Nwankwoala, H.O and Warmate, T. (2014) Subsurface Soil Characterization of a Site for Infrastructural Development Purposes in D/Line, Port Harcourt, Nigeria. *American International Journal of Contemporary Research*, **4**(6): 139-148.
- Ogunsanwo, .O. (2002) Effect of inundation on the geotechnical properties of some soils from parts of southwestern Nigeria. *Journal of Mining and Geology*, **38**(1): 57 -63.
- Ola, S.A. (1988) Geotechnical properties of Nigerian soils and ten years collaborative research with Nigerian building and road research institute, (NBRRI). 189-218.
- Oyedele, K.F., Oladele S. and Adedoyin O., (2011) Application of Geophysical and Geotechnical Methods to Site Characterization for Construction Purposes at Ikoyi, Lagos, Nigeria. *Journal of Earth Sciences and Geotechnical Engineering*, **1**(1): 139 - 148.
- Skempton, A. W. and MacDonald, D. H., (1956) Allowable settlement of buildings. *Proc. Inst. Civ. Engrs.* **5**, Pt. III, 727-768.

CONCLUSION

The importance of detailed geotechnical investigation for building foundations cannot be over emphasized. The soil located in the study area exhibit adequate bearing capacities. Soil samples from both locations in the study area classify as poorly graded gravely sands with little fines. These soils belong to group A-2-4 of the AASHTO classification system. The soil from the two locations exhibit good compaction parameters when compacted with high energy. They have low compressibility and medium to high permeability. Apart from specific gravity, other classification parameters vary significantly with depth. Safe bearing capacity also increases with depth, therefore a deeper depth especially the lateritized basement top is the most favourable for infrastructure foundations in the area.