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Subsoil Bearing Capacity from Load Test Results In Two Locations in Rivers State, Nigeria

Akande, O., Akpokodje, E. G. & Ekeocha, N. E*
Department of Geology, Faculty of Science University of Port Harcourt, East-West Road Choba, Rivers State, Nigeria.

Abstract: *Vertical pile load tests using the maintained load test and twice the safe working load (SWL) were used in two different locations (A & B) in Rivers State. Four pre-cast concrete piles of dimension 400mm by 400mm were tested using Kentledge method in location A while three cased piles of 406mm diameter were tested in location B using reaction method. The range of pile head movement (settlement) at maximum load of 1000KN (200% of SWL) was 2.76mm - 5.73mm while the range of 10% of the pile width was 11.04mm - 22.92mm. The elastic rebound varies from 80.49% - 97.65%. In location B, where reaction method was employed, the cumulative settlement at maximum load of 544.4KN was between 1.088mm and 5.70mm while the range of 10% of the pile width was 8.16mm - 23.142mm. The elastic rebound varied from 39.20% - 66.83%. The soil bearing capacity values at depth of 15.0m ranged from 570KN/m^2 - 710KN/m^2 while the pile bearing capacity at depth of 12.0m was 6350KN/m^2 in location A. A pile bearing capacity of 4188KN/m^2 and pile allowable load of between 361KN and 2275KN were respectively recorded at depth of 30m in location B. The pile bearing capacity was greater than soil bearing capacity. Results showed that the piles did not fail the test in both locations since cumulative settlements were much lower than 10% of the pile width. This could be due to factors like skin friction of the piles, elasticity, stiffness and pore water pressure of the soil. Therefore, test piles are capable of withstanding anticipated imposed stress from the super structure without failure. The Kentledge system produced greater influence on the test piles probably because weight used was higher than the safe load capacity of the test pile for safety consideration. The Kentledge weight increased the pile –soil interaction by increasing the unit shaft resistance of the piles. This could probably account for the high values of elastic rebound in location A. Insufficient time interval between driving and testing affected the elastic rebound values of test piles in location B. This is because piles in cohesive soil should be tested after sufficient time has elapsed for excess pore water to dissipate.*

Keywords: *piling, kentledge, reaction method, bearing capacity, shaft resistance, settlement, safe working load*

I. Introduction

Piling activities, which according Ingles and Metcalf (1972) are used to cut off slips or to improve the bearing capacity of weak ground are common in Nigeria, especially the Niger Delta region that has peculiar wet nature and exposure to annual hazards of flooding and river bank erosion. The natural hazard and the general swampiness of the terrain constitute serious constraints to civil engineering construction (Akpokodje, 1986). This makes it imperative to consider deep foundation, which according to Aboutaha et.al. (1993) is one founded deep below ground surface in order for its base bearing capacity is not affected by surface conditions, and occurs at greater than 3m below ground level. Such deep foundations include piling which transmits forces or load through a weak stratum to a lower and stronger stratum with sufficient bearing capacity to support the structure. Piling may be required to support vertical, lateral or uplift loads. In recent years the search for oil has been extended to deeper waters. A structure in deep water needs to be sufficiently strong to resist large lateral forces due to wave and wind loading.

Pile load testing provides an opportunity for continuous improvement in foundation design and construction practices, while at the same time fulfilling its traditional role of design validation and routine quality control of the piling works. In order to achieve this improvement, data from pile tests have to be collected and analysed to ensure the best use of resources. The load bearing capacity of pile is normally estimated by static pile formulae and later confirmed by pile load test. Very often, the pile load test is carried out shortly after the installation of pile. The pile capacity obtained from the load test is often assumed to be the long-term pile capacity, however, during the pile driving process, soils which surround the pile shaft and underneath the pile tip are highly disturbed. The duration for the complete dissipation of excess pore water pressure is dependent on the hydraulic characteristics of the subsoil. The pile capacity could be underestimated if pile load test is carried out while significant excess pore water pressure still remains. The pile capacity increases as the strength of the surrounding soil increases by re-consolidation. This is a common phenomenon for low permeability soils such as silt and clay. For granular soil which has higher permeability, the complete dissipation of excess pore water pressure is normally within few hours to few days (Burland, 1973, Benz, 2010).

This research aims at
i) determining the bearing capacity of the driven piles utilizing the cumulative and residual settlements of the piles,
ii) establish the adequacy of the predicted design capacities in compression of the piles and
iii) determine the effects of variation of different loadings on pile settlement.

This is imperative as the stability and safety of civil engineering structures depend largely on the adequacy of the foundation, which must be ensured that the earth materials supporting the structure are not over stressed. Factors that affect the choice of a piled foundation need to be considered and their relative importance taken into account. These factors include location and type of structure, ground conditions, durability, and cost.

Pile load testing, which is the most definitive method of determining load capacity of a pile, provides valuable information to the foundation engineer and is recommended to be done prior to the foundation design including all kinds of deep foundations that function in a manner similar to piles regardless of their method of installation. These tests involve the application of a load capable of displacing the foundation and determining its capacity from its response. Load test can be routine (when it is done up to 1.5 times the design load), initial (when it is up to 2 – 2.5 times the design load) or cyclic (when it is performed to separate the skin friction resistance from the point bearing resistance at the base) [Garg, 2009].

There are however various uncertainties in representing the real *in-situ* soil conditions by means of a few laboratory tested shear strength parameters. The basic soil parameters are cohesion (c_u), undrained shear strength (Ʈ) and angle of internal friction (φ), which can only be determined by laboratory testing of undisturbed soil samples.

Allowable Bearing Capacity (Q_a)

This is the bearing pressure that will cause acceptable settlement of the structure, i.e. if settlement is excessive the safe bearing capacity value will need to be reduced (by increasing force until settlement is acceptable). Settlement may be either long term consolidation as recorded in clays or immediate as seen in sands and gravels. Allowable bearing capacity is the ultimate bearing capacity divided by a factor of safety (equation 1). Sometimes, on soft soil sites, large settlements may occur under loaded foundations without actual shear failure occurring; in such cases, the allowable bearing capacity is based on the maximum allowable settlement.

$$Q_a = \frac{Qu}{F.S.} \qquad \text{-------------------------} \qquad \text{equation 1}$$

Where:

Q_a = Allowable bearing capacity (kN/m^2)
Q_u = ultimate bearing capacity (kN/m^2)
F.S. = Factor of Safety

Pile Load Test

Static load test is the most basic test and it involves the application of vertical load directly to the pile head. It is considered as the benchmark for pile performance (Fellenius, 1975). Static load tests provide reliable resolution of the problems for installed piles in layered soils with highly diverse strength parameters (Gwizdala and Krasinski, 2013). Loading is generally either by discrete increases of load over a series of intervals of time or alternatively in such a manner that the pile head is pushed downward at a constant rate.

Test procedures have been developed and defined by various codes, including ASTM (1978, 1987) and BS (1986). At least 1% of the total piles installed must be tested depending on the nature of the superimposed structure (BS, 1986). The test may take several forms according to the different reaction systems applied for the loading. Load-settlement curve is obtained simply by plotting the loads applied onto the pile head against the pile head displacement.

Pile and Static Load Test Criterion

Pile failure is identified by a failure load. The failure load is defined as that load at which the load against gross deflection curve reaches a slope of 1/32" of applied load.
In addition, for driven piles only:

Top deflection = B/60 + PL/AE, --------------------- equation 2

where:
B is the pile diameter or width (mm),
P is load (KN),
L is length (mm),
A is cross-sectional area (mm^2), and
E is modulus of elasticity (GPa).

Static Pile Load Test Acceptance Criteria

A Static Pile Load Test will be acceptance if:
The pile was installed in compliance with its specification,
The load test is satisfactory i.e. if the net settlement after rebound does not exceed 10% of the pile diameter.

II. Locations

The study was carried out in two locations A which occurs on the geographical coordinates of 4^o 21’N to 4^o 54’N and 6^o4’E to $6^o56^’$E (Figure 1) and location B that occurs on $4^o21^’$ N to $4^o34^’$ N and 6^o58’ E to $7^o21^’$ E (Figure 2). The locations are both in Rivers state, Nigeria. Figures 3 and 4 show the lithologies encountered at the two locations.

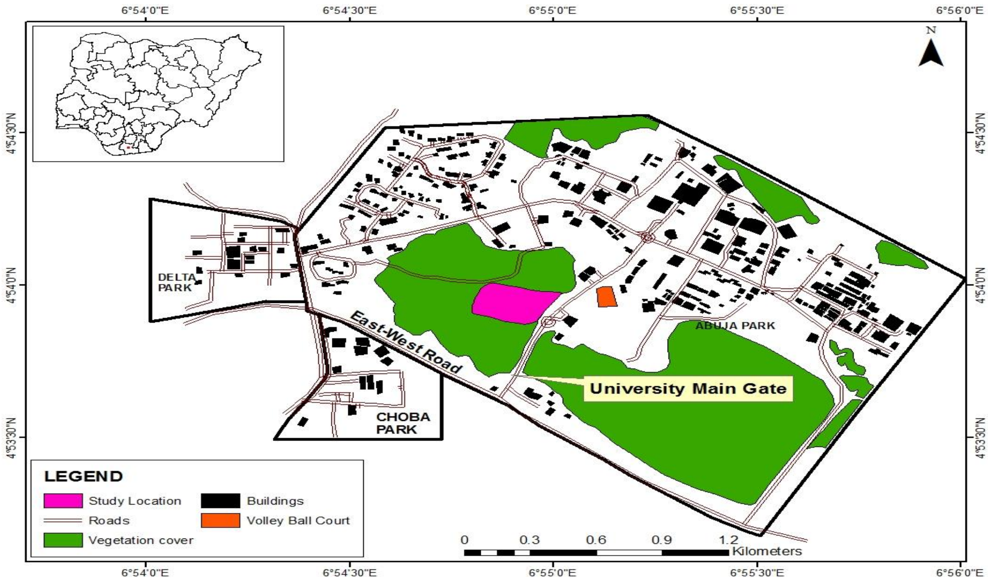

Figure 1: Map of the study area in Location A

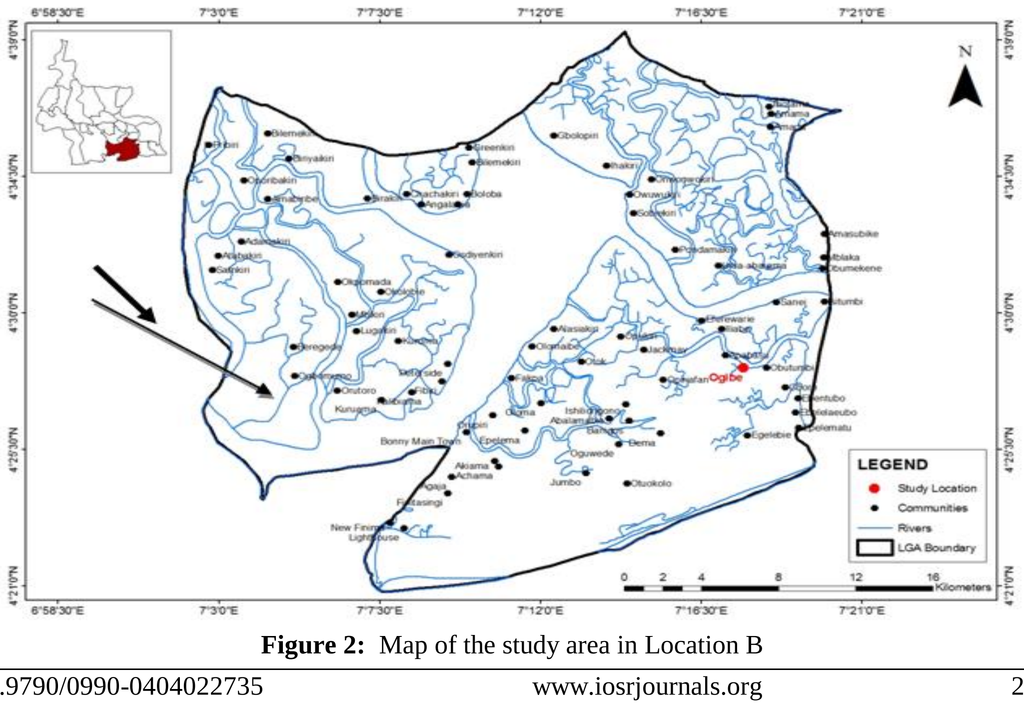

Figure 2: Map of the study area in Location B

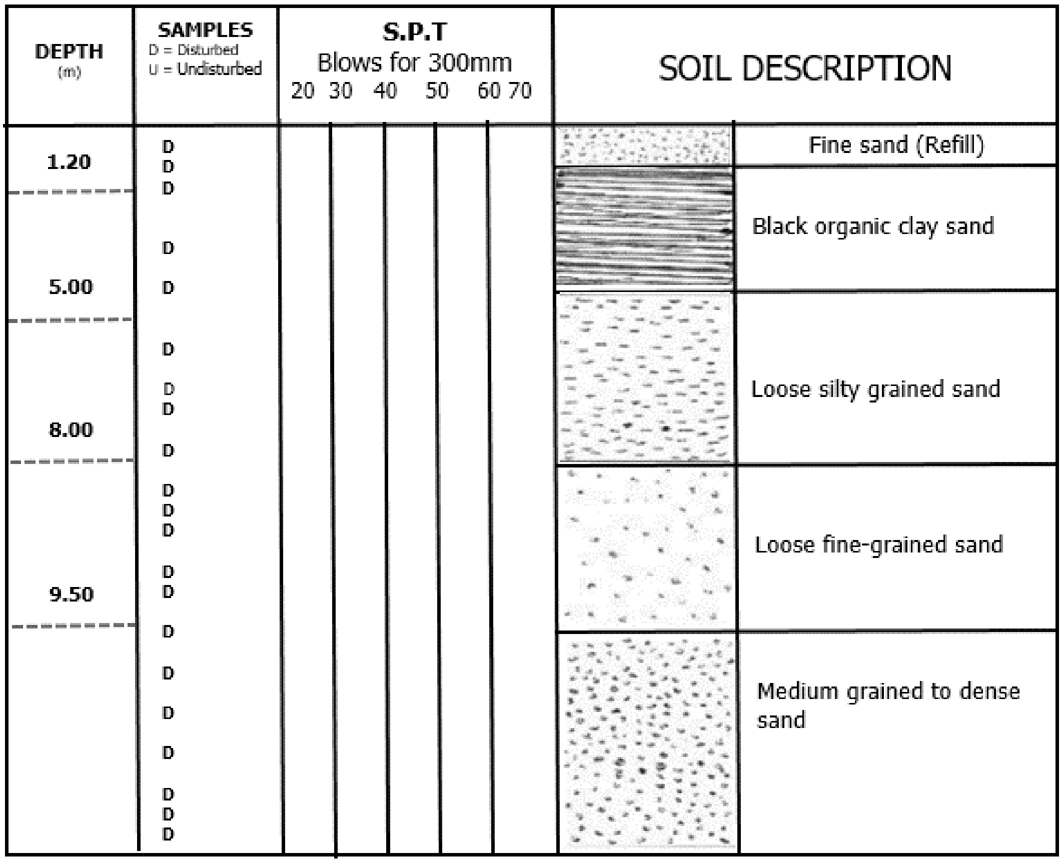

Figure 3: Lithology at Location A

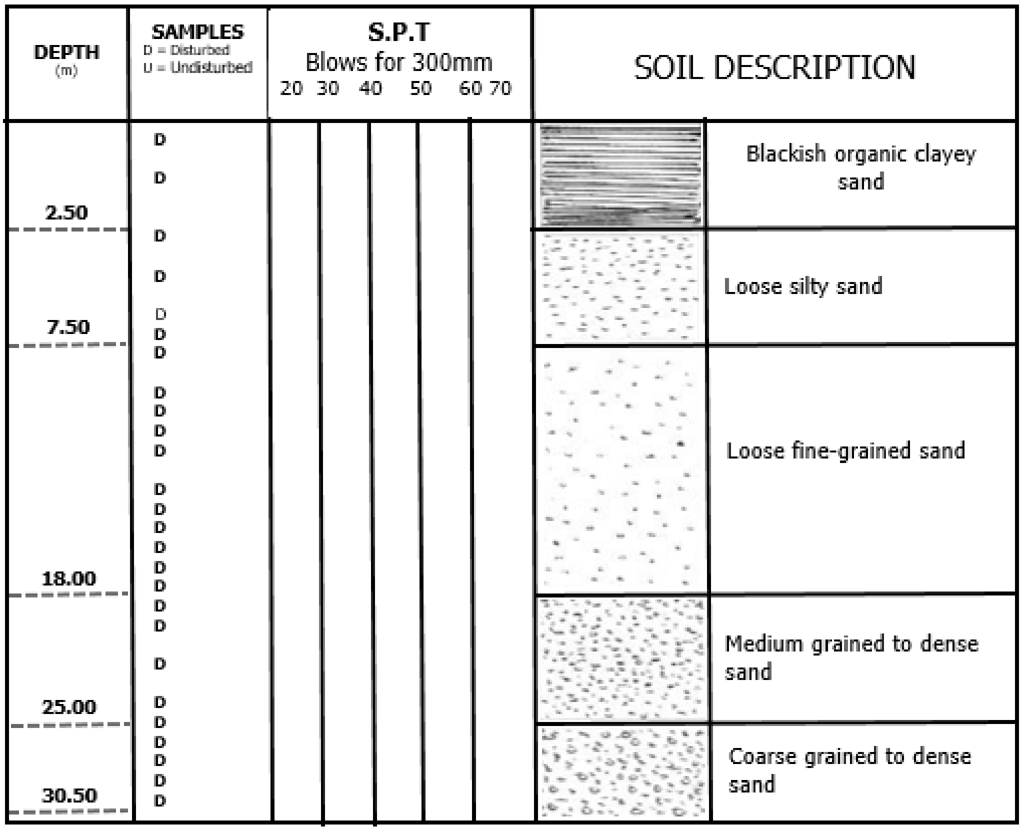

Figure 4: Lithology at Location B

III. Methodology

Piling was carried out using driven reinforced concrete (pre-cast concrete) piles of dimension 400mm × 400mm installed up to 12m depth at location A for the construction of multipurpose structures. At location B, 406mm cased piles were installed to 30m depth for the construction of a bridge. Reinforced concrete piles that are normally available commercially have dimensions of 250 – 400 mm and are 6m, 12m, and 30m deep, with the capability to withstand working loads of 450kN – 3500kN (Coduto, 2001). Precast piles with ordinary reinforcement resist bending stresses arising from picking up and transportation, and bending moments from lateral loads; and provide sufficient resistance to vertical loads and any tension force that may develop while driving (Bowles, 1996).

There were approximately 400 piling points for the entire development at location A while location B had 60 piling points. Four and three piles were tested and analysed at locations A and B respectively. The choices of the piles type for both locations were on the basis of terrain, durability and cost effectiveness. The tested piles in both locations were selected because of their low blow counts.

The maintained load test had the load increased in stages with a factor of safety twice the working-load with the time–settlement curve recorded at each stage of loading and unloading, in accordance with BS (1986).

Loading Cycle

The load was applied to the piles via a factory calibrated hydraulic pump and a hydraulic jack. A plate of diameter 0.61m and thickness of 30mm was used. The load was added gradually by increasing from 0 by 25% up to 200% of the design load. When each load increment was achieved, the next load increment was added every 30 minutes. At each load increment, load, settlement and time were recorded. At the maximum applied load, the load was maintained for a minimum of three hours. The load was then reduced in a reverse order from 175% to 0% of the design load, respectively at interval of 10 minutes. At zero load, rebound movement was recorded at 0, 2, 5, 10, 15, 30, 45 and 60 minutes thereafter until a constant settlement was reached (ASTM, 1995).

Measurement of Settlement

Settlement was measured using 0.01mm dial gauge. The readings on the two dial gauges were recorded and the pressure gauge for the jack at 30 minutes intervals on the Time-Settlement Data Sheet. These gauges were supported on rigid uprights fixed firmly into the ground. After completing the procedure, the load was in decrements of 25% until the final load was achieved. The rebound loads were maintained for 10 minutes and all the primary measuring systems were recorded immediately before removing the next load decrement. The pile's final rebound after it has remained at zero load for 1 hour was measured and recorded to estimate full elastic recovery. Before and after the application of each stage of loading and unloading, readings were taken at two opposite sides of pile cap by means of the dial gauges. The reading was taken at 2, 5, 10, 20, 30, 45, and 60 minutes after each loading and beyond that time every 30 minutes. Where the rate of settlement did not show any appreciable difference in two consecutive readings, the readings were taken at a reasonable interval. For any load, Q, the net pile settlement was calculated using the equation;

$S_{net} = S_t - S_e$ ------------------- equation 3

Where S_{net} is net settlement, S_e is elastic settlement of the piles and S_t is Total settlement. The value of Q was plotted against the corresponding net settlement as shown in Figs. 5 - 11.

IV. Results And Discussion

The readings on the two dial gauges were recorded and the pressure gauge for the jacks after every half hour on the time –settlement data sheets. The elastic rebound of the test piles, which was computed using equation 5 and the 10% of the pile width are presented in Tables 1 and 2 respectively.

Table 1: The Elastic Rebound of the Test Piles

	Pile ID	Pile Depth(m)	Elastic Rebound (%)
	A_1	12	88.80
LOCATION A	A2	12	97.65
	A3	12	90.75
	A4	12	80.49
LOCATION B			
	B_1	30	55.79
	B_2	30	66.83
	B_3	30	39.20

Table 2: The 10% of the Pile Width

LOCATION A	PILE ID	Width mm
	A_1	18.92
	A2	11.04
	A3	22.92
	A4	19.78
LOCATION B		
	B_1	23.14
	B_2	20.99
	B_3	8.16

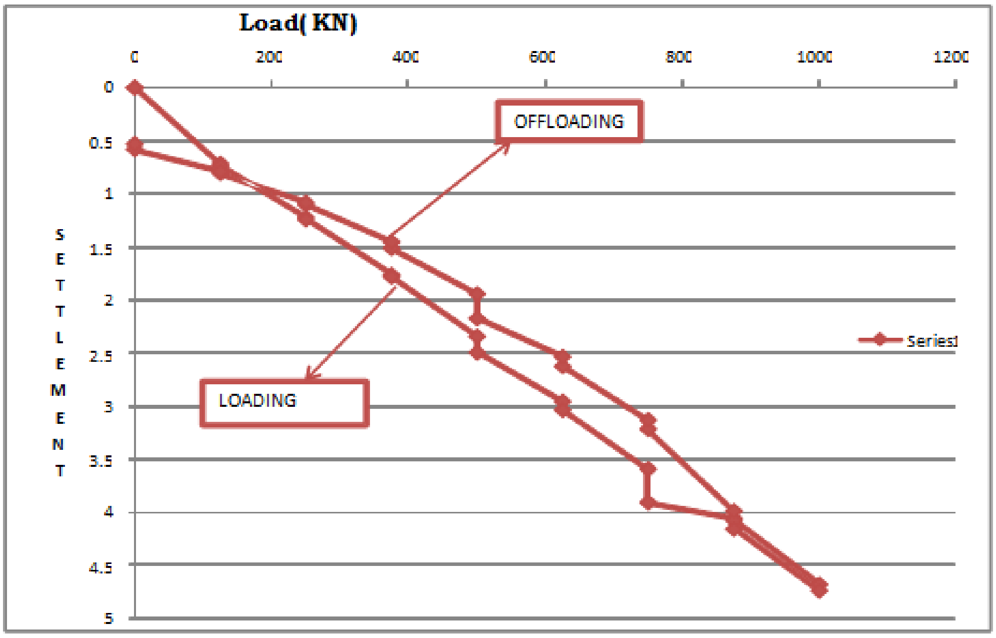

Figure 5: Load- settlement curve for A_1.

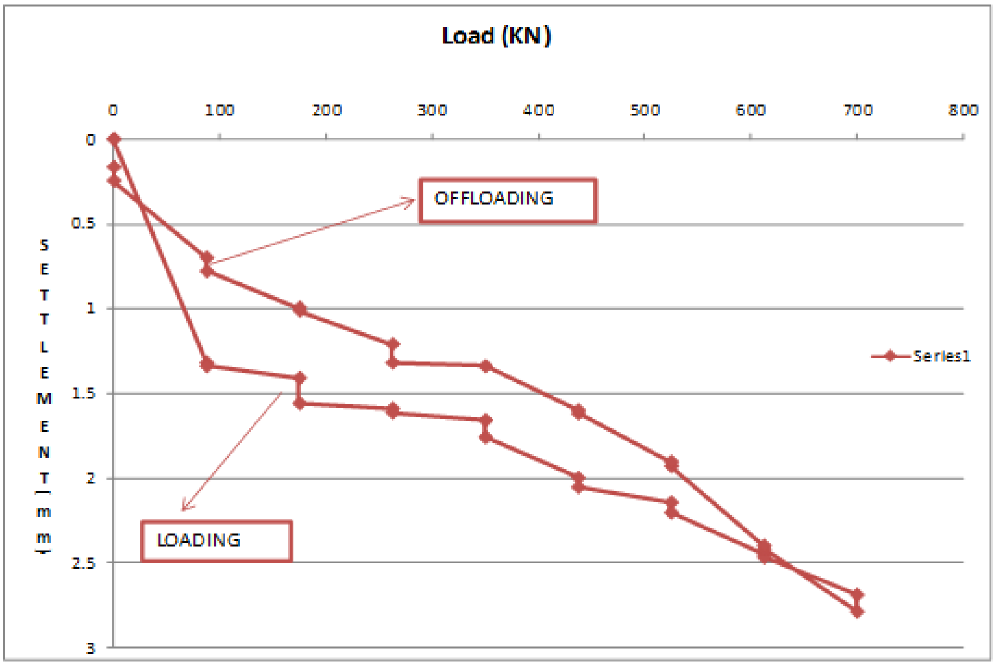

Figure 6: Load- settlement curve for A_2.

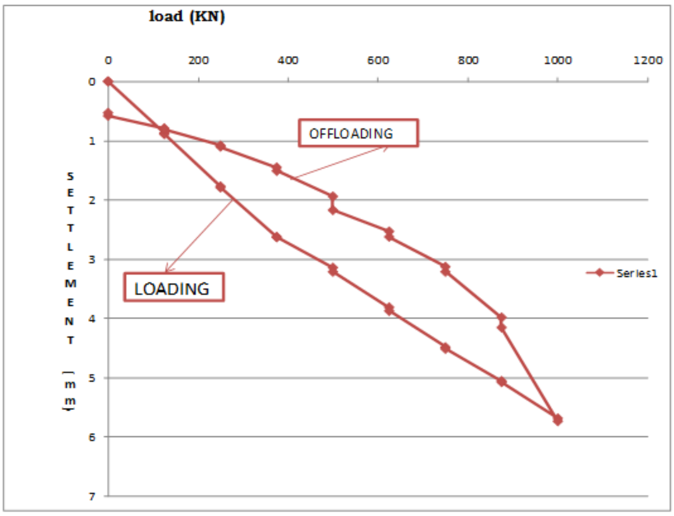

Figure 7: Load- settlement curve for A_3

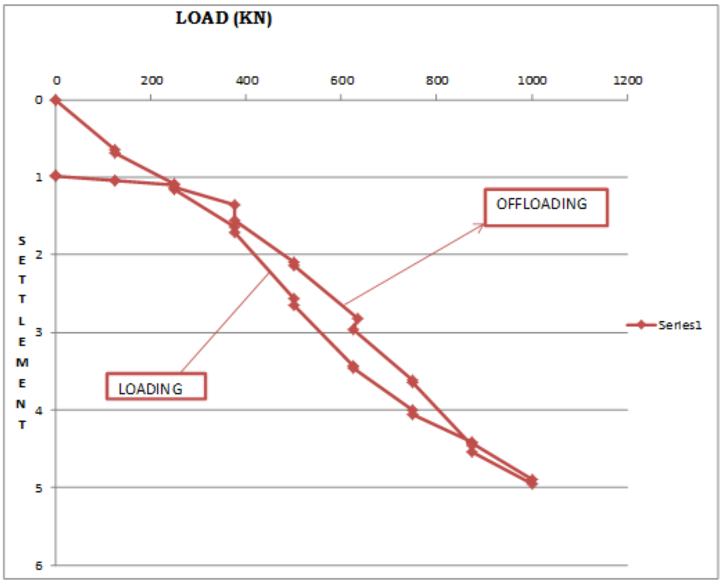

Figure 8: Load- settlement curve for A_4

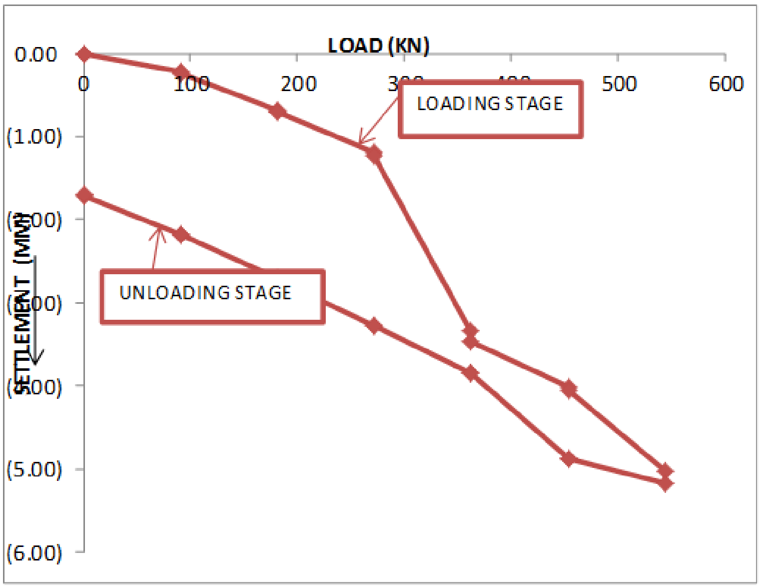

Figure 9: Load- Settlement Curve for B_1

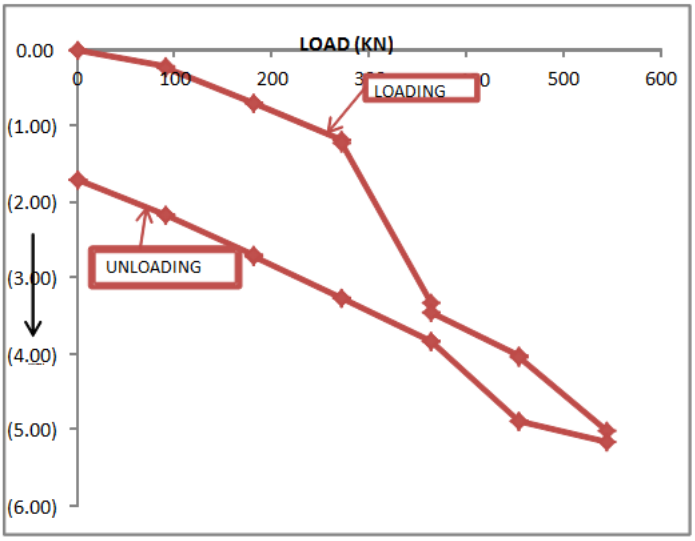

Figure 10: Load- Settlement Curve for B_2

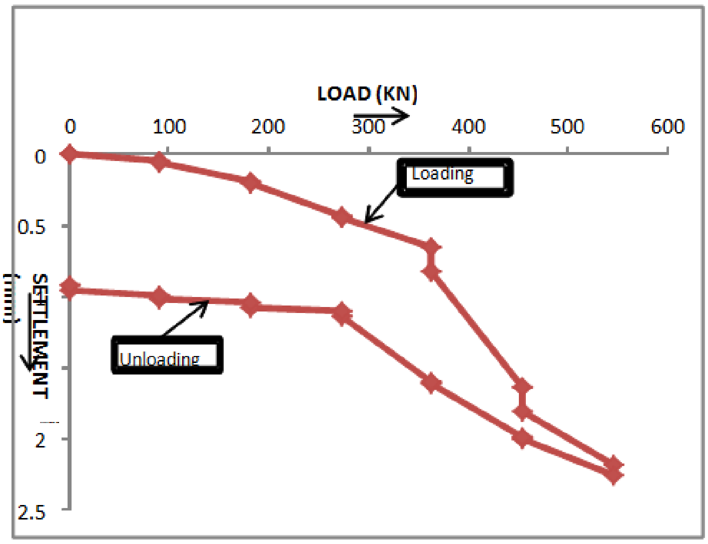

Figure 11: Load- Settle Curve for B_3

Calculations of Bearing Capacity of the Test Piles

$$\text{Pressure} = \frac{force}{area} \quad \text{---} \quad \text{equation 4}$$

Calculation of Elastic Rebound of Test Piles

$$\text{Elastic rebound} = \frac{\text{Residual settlement}}{\text{Cumulative settlement}} \times 100\% \quad \text{--------------} \quad \text{equation 5}$$

The Figures 5, 6 and 8 followed the same trend i.e. the settlement rate was steady indicating a firm and dense sandy formation. The settlement rate in Figure 7 was steady and the loading almost forming a straight line curve which indicates the uniformity of the strata encountered.

Figures 9 & 10 present a settlement rate that was steady up to 200KN. There was a sharp drop between 250 - 350KN which indicate a loose sandy formation. The settlement was steady from 400 – 544KN, indicating that the formation at this interval was sandy, coarse and dense.

In the case of figure 11, the settlement rate was steady up to 100KN. However, there was a sharp drop in the graph between 350 - 450KN which indicates loose fine sandy formation.

The piles A_1, A_2, A_3 and A_4 recorded the elastic rebound of 88.80%, 97.65%, 90% and 80.49% respectively. This implies that the piles terminated in good and firm strata. However, Kentledge system may probably have larger influence on the test piles due to the additional weight higher than load capacity of the test pile needed for safety consideration. The *kentledge* weight increases the pile-soil interaction by increasing the unit shaft resistance of the pile. This could probably account for high values of elastic rebound in location A. The piles with identification B_1, B_2 and B_3 have elastic rebound of 55.79%, 66.83% and 39.20% respectively. Insufficient time interval between driving and testing could probably affect the elastic rebound values of test piles in location B. This is because piles in cohesive soil should be tested after sufficient lapse for excess pore water pressure to be dissipated (Benz, 2010).

V. Conclusion

The piles terminated at very firm dense and coarse sand formation in both locations (Figures 3 and 4). Using the test loads of 700KN and 1000KN, the bearing capacities of the test piles at depth 12.0m were 4375KN/m^2 and 6250KN/m^2 respectively in location A, while at location B, the test load of 544.4KN resulted in a corresponding test pile bearing capacity of 4188KN/m^2. The pile bearing capacity was greater than soil bearing capacity, and the results showed that the piles did not fail the tests in both locations since cumulative settlements were much lower than 10% of the pile width. This could be due to factors like skin friction of the piles, elasticity, stiffness and pore water pressure of the soil. The results obtained in this study are in good agreement with the studies in centrifuge test with displacement piles (Fioravante *et al.*, 1995). It is therefore concluded that the test piles are capable of withstanding the anticipated imposed stress from the super structure without failure.

Kentledge system produced greater influence on the test piles due to the additional weight higher than load capacity of the test pile needed for safety consideration. The *kentledge* weight increases the pile-soil interaction by increasing the unit shaft resistance of the pile. This could probably account for high values of elastic rebound in location A.

Insufficient time interval between driving and testing affected the elastic rebound values of test piles in location B. This is because piles in cohesive soil should be tested after sufficient time has elapsed for excess pore water pressure to dissipate (Benz, 2010).

References

[1]. Aboutaha, M., De Roeck, G., Van Impe, W. F. (1993). Bored versus displacement piles in sand-experimental study. Deep Foundation on Bored and Auger Piles, 157-162.

[2]. Akpokodje, E. G. (1986). The geotechnical properties of lateritic and non-lateritic soil of south-eastern Nigeria and their evaluation for road construction" Bulletin of International Association of Engineering Geology No. 33, 115-121.

[3]. American Society for Testing and Material (ASTM), 1978. Standard Method of Testing Piles Under Static Axial Compressive Load (D1143), Annual Book of Standards, Philadelphia Vol. 408.

[4]. Benz, T. (2010). Small Strain Stiffness of Soils and its Numerical Consequences. Ph.D Thesis, Institute for Geotechnical Engineering, University of Stuttgart.

[5]. Bowles, Z. T. (1996). "The point load test in geotechnical practice "Engineering Geology, 9, 1-11.

[6]. Burland, J. B. (1973). "Shaft Friction of Piles in Clay", Ground Engineering, Vol. 6, No.3, 30 – 42.

[7]. BS8004 (1986). Ninth Laurites Bjerrum Memorial Lecture: "Small is beautiful" - the stiffness of soils at small strains in Canadian Geotechnical Journal, 26: 499–516.

[8]. Coduto, N. G. (2001). "From Strain Measurements to Load in an Instrumented Pile", Geotechnical News Magazine, Vol.19, No.1,

[9]. Das, B. M. (1998). "Principles of Foundation Engineering. PWS Publishing Brooks/Cole Publishing company U.S.A. 4th Ed., 34.

[10]. Fellenius, B. H. (1975). Test loading of piles. Methods, Interpretation, and New Proof Testing Procedure. ASCE, Vol. 101, GT9, 855 - 869.

[11]. Fellenius, B. H. (2001). "The O-Cell–An Innovative Engineering tool", Geotechnical News Magazine, Vol. 19, No. 2 , 32- 33

[12]. Fellenius, B. H. (2002). "Determining the Resistance Distribution in Piles. Part 2: Method for Determining the Residual Load", Geotechnical News Magazine, Vol. 20, No. 3, 25 – 29.

[13]. Fioravante, A. Y, Carol, T. A., & Collin M. V. (1995). Centrifuge test with displacement piles, Geotechnical engineering department, Cambridge University.

[14]. Garg, S. K. (2009). Soil Mechanics and Foundation Engineering, Khanna Publishers, New Delhi, 7th revised ed., 596.

[15]. Gwizdala, K. and Krasinski, A. (2013). Bearing capacity of displacement piles in layered soils with highly diverse strength parameters. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris, 2739 – 2742.

[16]. Ingles, O. G. and Metcalf, J. B. (1972). Soil stabilization principles and practice, Butterworths, Brisbane, 321

[17]. Meyerhof, G. G. (1956). "Bearing Capacity and Settlement of Pile Foundations". Proceedings of the A.S.C.E, GT3, 197-228, March.

[18]. New York State Department of Transportation (NYSDOT, 1977)., "Static pile load test manual", Official Insurance No.741-3, Soil Mechanics Bureau, Soil Control Procedure, SCP-4/77, New York State Department of Transportation, Albany, 41.

[19]. Rollins, M. K., Reissner, D. J., and Gerber, M. T. (2005). "Measured and computed lateral response of a pile group in sand. *Journal of Geotechnical and Geoenvironmental Engineering.,* Vol. 121: 103-114.