CONSOLIDATION SETTLEMENT OF RESIDUALLY-DERIVED LATERITIC SOILS FROM PART OF SW NIGERIA

Adebisi, N.O., Odutayo, O.D., Kehinde, A. S. and Ogunwo, F. A.

Department of Earth Sciences, Faculty of Science, Olabisi Onabanjo University, Ago Iwoye, Ogun State, Nigeria. e-mail: noadebisi@yahoo.com

ABSTRACT
Consolidation of lateritic soils and related properties had been the subject of few investigations. Previous studies have evaluated the coefficients of compressibility and consolidation of lateritic soils. Initial results indicated that the soils have considerable strength despite the appreciable clay content. Another report revealed probable differential settlement of small magnitude if employed as foundation materials. As a result, there has been a growing interest to investigate settlement effects of lateritic soils. The present investigation focuses on the how total settlement is influenced by complexity and heterogeneity of lateritic soils. The studied soils were developed over the Precambrian, migmatised gneisses of the Basement Complex of Southwestern Nigeria.

Physical model tests out on 12 disturbed and 12 undisturbed soil samples were carried out following the BS Part 2 and Part 8, 1337, IS: 2720 Part 15 and ASTM Designation 4767-11. Results show that the soils contain a variety of particle sizes, with clay and sand being the dominant fractions. They also exhibit very low permeability, low to medium plasticity and cohesive strength range of 20 - 95 kN/m^2. The total settlement values are not in any definite order with depth, but increases with increase in confining pressure. They range between 0.31 and 1.125mm at 800kN/m^2. It is apparent that the tolerable settlements of different structures on the soils vary considerably, which may result to misalignment from foundation settlements causing structural damage to a building frame nuisance.

Key words: ***Consolidation, clay, lateritic, gneisses and plasticity***

INTRODUCTION

Lateritic soils are crucial to sustaining engineered structures as foundation material especially, in the tropical environment of Southwestern Nigeria. One of the fundamental problems in foundation settlement due to building load is response of the soil medium under such a time-dependent loading. Despite its importance to various foundation applications, a categorical statement on consolidation settlement in laterised soils media is yet to be made. This is owing to the complexity of the soils' constituents and heterogeneity in their properties. In order to provide a basis for understanding, comprehensive experimental database need be presented to permit a direct interpretation of total settlement. The available analytical solutions and experimental results from the literature are unable to comprehensively, explain consolidation influences brought about by a combination of particle density, size distribution, plasticity characteristics, and natural moisture content, as well as shear strength parameters.

Malomo and Ogunsanwo, (1983) investigated the pre-consolidation pressure of an amphobolite-derived soil. Values of the pre-consolidation pressures obtained using the Casagrande and the 'Constrained Modulus' are 3.8×10^2 a nd 3.5×10^2 kN/m^2 respectively. The constrained modulus method was however recommended for determination of pre-consolidation pressures of laterite soils in view of its advantages over the other method. Good serviceability by any structure is known to be a function of its foundation satisfying the settlement criterion (Holtz, 1991). Magnitude of probable settlement for foundations soils due to structural loading is therefore necessary to be predicted. Mahalinga-Iyer and Williams (1994) conducted research on shear strength and consolidation of naturally occurring lateritic soils usually employed for foundation purpose. The soils were found to exhibit high shear strength despite their appreciable clay content.

Adebisi (2010) discussed the consolidation parameters of lateritic soils from three different parent rocks in parts of Southwestern Nigeria. The research work focused on predicting the magnitude and the rate of settlement the soils will undergo if employed as foundation material. In a similar development, Adebisi and Adeyemi (2012) and Adebisi *et al.* (2013) reported probable differential settlement of small magnitude for various lateritic soils from part of Southwestern Nigeria based on their coefficient of compressibility values. However, amount of settlement will be the indispensable criterion controlling design as total settlement under safe load may somehow exceed allowable limits.

The current study examines the fact that lateritic soils have the shear strength to be able to support building loads but may also become weak, leading to structural failure. Ascertaining representative values of the total settlement that may occur in the residually-derived lateritic soils when used as foundation material is the core aspect of this research. It generally looks at consolidation influences on laterised foundation soils due to particle density, grain-size distribution, consistency, natural moisture content frictional resistance and cohesive strength. This will better expose physical insight that has been obscured by the complexity of soil properties. In addition, soil deformation property will be employed as an effective tool for the design foundations under static loads.

Geology and Profile Development

The Southwestern Nigerian Precambrian Basement Complex region has been discussed by many researchers including Grant, (1972). The area that lies between latitudes 7°N and 8°N and longitudes 3°E and 6°E right in the equatorial rain forest region of Africa is of interest in this study (Fig.1.1). Foundation soils from such an area would have been derived through actions of various processes of pedogenic factors on the parent rocks (Christopher and Benjamin, 1982). Tropical weathering of these rocks is a prolonged process of chemistry which produces a wide variety of lateritic soils in the thickness (Wright, 1992; Tardy, 1997). These include the topography of the land, climate, *e.t.c.* The main lithologies of the parent rocks in the study area include; the amphibolites, migmatite gneisses, granites, pegmatites and schists (Oyinloye, 2011). Being formed under high pressure and temperatures, the crystals of the minerals in these rocks are somewhat unstable at surface pressure and temperature. Particularly when attacked by water that etch away the soluble components in the minerals, the crystals fall apart, albeit very slowly. It is called spontaneous weathering, but it is accelerated considerably under the influence of vegetation and its acids.

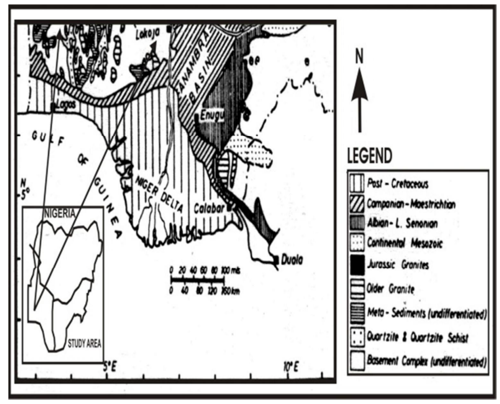

Figure: 1.1. Geological map of showing Southwestern Nigeria

It follows that the rocks weathered under tropical climatic condition which is favourable for the development of lateritic soils. Chemically, lateritic soils are rich in iron and aluminium, formed in hot and wet tropical areas. Nearly all laterites are rusty-red because of the iron oxides. They developed by intensive and long-lasting weathering of the underlying parent rock. A vertical cross section of the soils usually extend from the surface to the parent material and consisting of genetically related horizons and sub horizons created during soil formation. Soil profiles in the area range in thickness from several dozen centimeters to several meters. Soil profiles are divided into natural profiles and those that have been altered by man's activity. The variations in the structure of the natural profile and in the composition and properties of its horizons serve as the basis for consolidation settlement.

Concept of Consolidation Settlement

When a loading is applied on a soil, the incompressible pore water initially supports this loading and no volume change takes place. This leads to a change in the spatial distribution of total head. The pattern of change depends upon the loading configuration and the variability of soil related properties. Furthermore, water from locations with higher heads flows towards locations with lower heads, and the excess pore water pressure dissipates. In this process, loading that was originally supported by the water is transmitted to the soil grains. Terzaghi *et al.,* (1996) noted that this results in the increase of the effective stress of soils. As no holes is to develop in the ground, the volume of the pore water that flows away equals to the volume reduction of soils, and the ground settles.

The permeability of a cohesive soil is rather small, and the process of excess pore water pressure dissipation and the ground settlement takes time to develop. Gopal and Rao (2000) described time-dependent process generally, as consolidation or primary consolidation. Secondary consolidation settlement is another time-dependent settlement which cannot be explained on the basis of excess pore water pressure dissipation. Computationally, it is often considered to take place after primary consolidation is completed (Esu and Ilori, 2002). Our interest in this chapter is limited to one dimensional consolidation. If properly employed, it does meet engineering needs for tackling a wide variety of practical problems.

When stress is removed from a consolidated soil, the soil will rebound, drawing water back into the pores and regaining some of the volume it had lost in the consolidation process. If the stress is reapplied, the soil will re-consolidate again along a recompression curve, defined by the recompression index. Soil that has been consolidated to a large pressure and has been subsequently unloaded is considered to be overconsolidated (Ramamurthy and Sitharam, 2005). The maximum past vertical effective stress is termed the preconsolidation stress.

A soil which is currently experiencing the maximum past vertical effective stress is said to be normally consolidated (Das, 2006). The overconsolidation ratio, (OCR) is the ratio of the maximum past vertical effective stress to the current vertical effective stress. The OCR is significant for two reasons: firstly, because the compressibility of normally consolidated soil is significantly larger than that for overconsolidated soil, and secondly, the shear behavior and dilatancy of clayey soil are related to the OCR through critical state soil mechanics; highly overconsolidated clayey soils are dilatants, while normally consolidated soils tend to be contractive (Handy and Spangler, 2007).

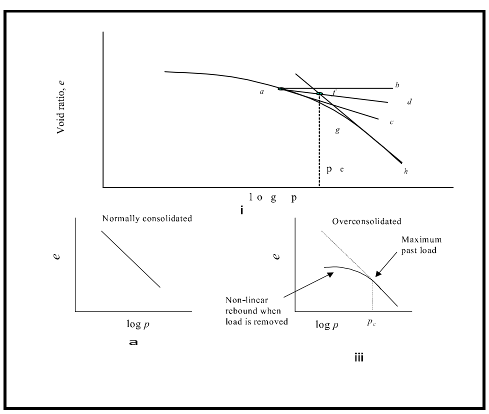

Figure: 3. Typical *e*-log *p* curve (i) for preconsolidated clay, (ii) consolidated and (iii) overconsolidated clay.

Where;

i. at point *a*, *e*-log *p* has minimum radius of curvature,
ii. line ab is the horizontal line from *a,*
iii. line ac is the tangent to curve at *a,*
iv. line *ad* is the bisector of angle *bac*,
v. ah is the projection of the straight-line back to intersect *ad* at *f, and*
vi. abscissa of point *f* is the preconsolidation pressure, p_c

Consolidation is therefore, the process of reduction of bulk soil volume under loading due to flow of pore water (Stanciu and Lungu, 2009). For laterised soils, any surcharge or increment of loading will be initially taken up by the pore pressure and result in consolidation until a new equilibrium is reached where the soil grains takes up the added load.

Lateritic soils have both cohesive and cohesionless components, and so there will be three categories:

i. Immediate settlement which describes elastic deformation of dry soil and moist and saturated soils without change to moisture content due to;
   a. high permeability in sandy fraction, pore pressure in clays support the entire added load and no immediate settlement occurs.
   b. the construction process, immediate settlement is not important.
ii. Primary consolidation settlement is the volume change in lateritic soils because of the expulsion of water from void spaces. However, high permeability of sandy, cohesionless fraction result in near immediate drainage due to the increase in pore water pressure and no primary consolidation settlement occurs.
iii. Secondary compression settlement is a function of plastic adjustment of soil fabric in lateritic soils

According to Coduto (2001), total settlement can be expressed as:

$\Delta h = \Delta hi + U\ \Delta hc + \Delta hs$

Where Δh = total settlement, Δhi = elastic or immediate settlement, Δhc = consolidation settlement, Δhs = secondary compression, U = average degree of consolidation.

Generally, the final settlement of a foundation is of interest, and U is considered equal to 1 (i.e. 100% consolidation)

Immediate settlement Δhi is that part of the total settlement, which suppose to take place during the application of loading. The consolidation settlement Δhc is that part which is due to the expulsion of pore water from the voids and is time-dependent settlement. Secondary settlement Δhs normally starts with the completion of the consolidation. It means, during the stage of this settlement, the pore water pressure is zero and the settlement is only due to the distortion of the soil skeleton.

Methods of Sampling and Testing

Four test pits were established on profiles developed over Migmatite Gneisses through pedological mapping, to enable sampling at relatively uniform depths. Disturbed soil samples were collected with a auger, while undisturbed soil samples were obtained using U-4 narrow cylinder at 0.30, 0.55 and 0.85 m. Furthermore, soil texture, colour, and other characteristics were also observed at four different profiles. The standards employed for the laboratory analyses of the soils are contained in the British Standards, BS 1337: Part 2 and Part 8 (BS, 1990)

Following the IS: 2720 Part 15 (1986) and ASTM (2011) Designation 4767-11, every undisturbed sample of 75 mm diameter and 15-20 mm thick, was enclosed in a circular metal ring and sandwiched between porous stones for the oedometer consolidation test.

Discussion of Results

Particle Density and Grain-Size Distribution

The values of particle density of twelve samples shown in Table 1 are of narrow range. The common range among the studied soils is 2.64 – 2.77 g cm^{-3}. These values are suitable in accordance with Wright (1986). These are considered appropriate for foundation materials. The particle density of a mass of soil is of interest to the engineer for a variety of reasons including consolidation settlement. Particle density is an important index property of soils that is closely linked with mineralogy. The measured density of a soil is a function of the relative proportions of constituent minerals.

The grain size distribution curves were drawn to determine the percentages of different grain sizes contained within the studied soils. This is important distribution of different grain sizes especially; clay content affects the consolidation settlement of soil. The grading curve shown in Figures 4.a-d give a representation and qualitative picture of the relative proportions of the different grain sizes within the soil mass. All the grading curves for the studied soils cover several log cycles of the semi-log paper, showing that they contain a variety of particle sizes, and are therefore well-graded.

Table: 1. Particle Density Results for the studied soil.

	Depth (m)	Particle Density (g/cm^3)	Range
Pit 1	0.3	2.75	
	0.55	2.75	2.68-2.75
	0.85	2.68	
	0.3	2.77	
Pit 2	0.55	2.72	2.64-2.77
	0.85	2.64	
	0.3	2.69	
Pit 3	0.55	2.66	2.64-2.69
	0.85	2.64	
	0.3	2.75	
	0.55	2.69	2.67-2.75
Pit 4	0.85	2.67	

Table 2 qualitatively summarizes the grain size distribution for the studied soils. Sand and clay fractions dominate the composition of the studied soils with subordinate amount of silt, while gravel varies greatly in composition. Considering the particle density, the classification by Ramamurthy *et al.* (2005) make the studied soils to fall within sandy silty clay class.

The amount of fines in a soil is of paramount importance when considering its consolidation settlement. This is simply the arithmetic sum of clay and silt content of the soil. Based on the Unified Soil Classification System (USCS), the British Standard (1981) for foundation structures, soils with amount of fines between 0% - 5% are generally ascribed to well-graded soils, while soils with amount of fines ranging between 5-15% are said to be well-graded but clayey sand or gravel. The fines composition of the soils ranges between 11.0 and 79.0 %. This clearly reflects the individual fractions present in the general composition of the soils.

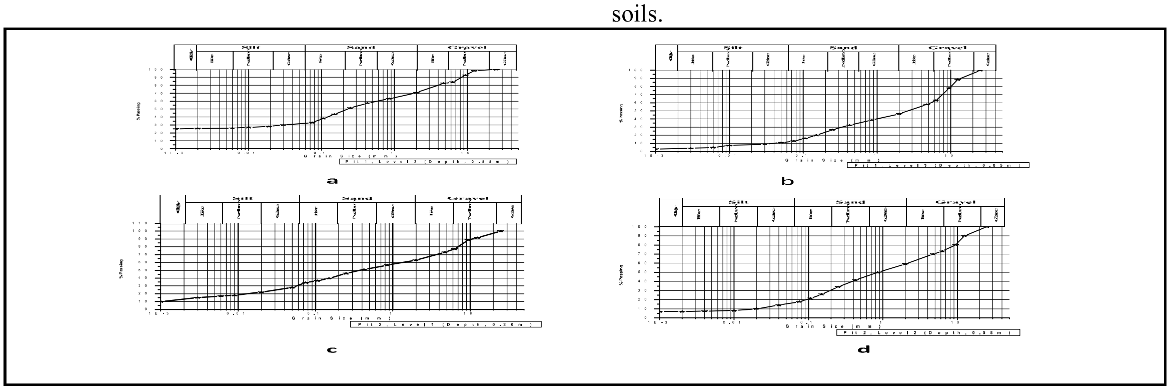

Figure: 4.a-d. Selected gradng curve for the studied soils

Coefficient of Permeability

Permeability values of soils are governed by the makeup of the soils (Carter and Bentley, 1991). The clayey sandy nature of the soils is responsible for low permeability rate. This is an important physical property, which finds application in determining footings in buildings to prevent settling.

The values of measured permeabilities for the studied soils are presented in table 3. Wide range of permeabilities exists among the studied soils as they vary from depth-to-depth without following any trend. The highest permeability value is recorded in pit 2 at depth 0.55m as 6.30×10^{-7}. The lowest permeability value recorded is from pit 4 at depth 0.85m. The average permeability values were recorded thus: 1.36×10^{-7}, 2.31×10^{-7}, $1.50.\times10^{-7}$ and 1.56×10^{-6} for pits 1, 2, 3, and 4 respectively. These values are indicative of very low permeable soils, which qualify them as good foundation materials.

Table: 2. Grain size distribution of the studied soils

Sample No.	Depth(m)	Amount of Clay (%)	Amount of Silt (%)			Amount of Fines (%)	Amount of Sand (%)			Amount of Gravel (%)		
			Fine	Medium	Coarse	Clay + Silt (%)	Fine	Medium	Coarse	Fine	Medium	Coarse
	0.30	21	1	1	1	24	10	21	32	7	6	0
Pit1	**0.55**	25	1	2	4	32	16	12	12	11	17	0
	0.85	25	1	2	4	32	16	12	11	12	17	0
Average		23.6	1	1.2	3	29.3	14	15	18.3	10	13.3	0
	0.30	13	4	5	9	31	12	11	9	13	24	0
Pit2	**0.55**	7	1	2	7	17	13	16	14	13	27	0
	0.85	6	1	1	5	13	12	12	10	19	34	0
Average		8.6	2	2.6	7	20.3	12.3	13	11	15	28	0
	0.30	6	2	5	5	18	15	20	29	18	0	0
Pit3	**0.55**	42	2	1	2	47	13	15	8	17	0	0
	0.85	64	4	8	3	79	10	5	5	1	0	0
Average		37.3	2.6	4.6	3.3	48.0	12.6	13.3	14	12	0	0
	0.3	28	2	3	4	37	10	18	14	11	10	0
Pit4	**0.55**	2	2	2	5	11	14	19	30	22	4	0
	0.85	18	1	2	7	28	21	34	17	0	0	0
Average		16	1.6	2.3	5.3	25.3	15	23.6	20.3	11	4.6	0

Table: 3. Coefficient of permeability of the soils

Depth	Coefficient of Permeability (m/sec) Pit1	Pit2	Pit3	Pit4
0.3m	3.49×10^{-8}	4.749×10^{-8}	3.78×10^{-8}	2.105×10^{-8}
0.55m	3.305×10^{-7}	6.30×10^{-7}	2.83×10^{-8}	4.56×10^{-6}
0.85m	4.47×10^{-8}	1.74×10^{-8}	3.85×10^{-7}	1.213×10^{-7}
Average	1.36×10^{-7}	2.31×10^{-5}	$1.50.\times10^{-5}$	1.56×10^{-6}

Natural Moisture Content and Consistency Limits

In almost all soil tests natural moisture content of the soil is essential to determine. The knowledge of the natural moisture content is essential in all studies of soil mechanics. Natural moisture content finds application in problems which relate to the bearing capacity and settlement of foundation soils. The natural moisture content is a reflection of the state of soil in the field. The average natural moisture contents of soils from pits 1, 2, 3 and 4 are 15, 14.3, 14.4 and 15.2 respectively. It is apparent that values of moisture contents of all the pits are overlapping.

The physical properties of lateritic soils vary much at different water contents due to clay content. Clay may exist almost in liquid state or show plastic behaviour or be very stiff depending on the moisture content (Casagrande, 1948). Plasticity data shown in table 4 is a property of outstanding importance for lateritic soils, which defines changes in shape without rupture to be undergone by the soils.

The liquid limit of the studied soils has been observed to vary throughout the test pits. The highest liquid limit value recorded for the studied soils is 49 % at 0.58 m in pit 2, while the lowest liquid limit value is 29 % at pit 1. The plastic limit values have short ranges throughout the pits, and they mostly lie between 13 – 16 %. Average Plastic limit for pits 1, 2, 3 and 4 are 14.3, 13.6,14 and 14 % respectively.

Table: 4. Consistency limits of the studied soils

Sample No	Depth	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Natural Moisture Content
	0.3	29	14	15	15.5
Pit1	0.55	29	13	16	12.7
	0.85	47	16	25	16.9
Average		35	14.3	18.6	15.03
	0.3	41	13	28	13.3
Pit2	0.55	39	14	25	13.8
	0.85	49	14	35	15.9
Average		43	13.6	29.3	14.33
	0.3	31	13	18	15.8
Pit3	0.55	35	13	22	15.3
	0.85	41	16	25	12.3
Average		35.6	14	21.6	14.47
Pit4	0.3	31	14	17	
	0.55	37	13	24	17.2
	0.85	40	15	25	14.0
					14.5
Average		36	14	22	15.23

The plastic limit values increase with depth in pits 2 and 3. The plasticity index values, of the studied soils range between 15 and 28 %. These are the ranges of moisture content over which the soils would be in a plastic condition. The plasticity index values of the soils generally, increase with depth in all the test pits. The Casagrande plasticity charts for classification of the studied soils are shown in figures 5:a-d. All the studied soils plot above the A-Line within the liquid limit range of 29 and 49 %. This implies that the soils contain clays of low to medium plasticity

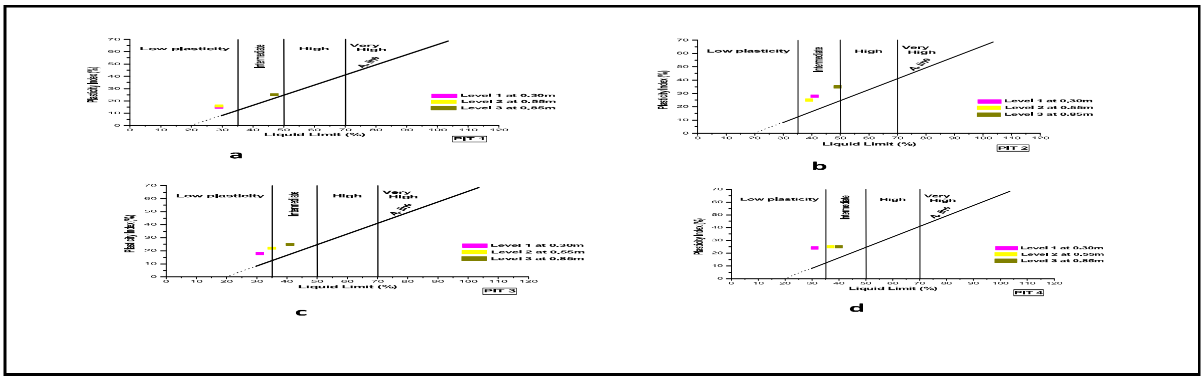

Figure: 5.a-d. Casagrande plasticity chart for the soils

Shear Strength Parameters

Lateritic soils possess shear strength and are capable of resisting applied shear and normal stresses. The shear strength of soil depends on the effective stress, the drainage conditions, and the density of the particles, the rate of strain, and the direction of the strain. These furnish information on the development of shear strength in the soils and the stresses may be applied under different loading conditions. It also enables analysis and design of some important soil related structures, such as foundations. The shear strength of the studied soils is expressed in terms of two parameters; cohesion and angle of internal friction (Table5). These define the shear resistance of the studied soils as a result of friction, and cementation or bonding at particle contacts.

Table: 5. Cohesive Strength and Angle of Internal Friction

Sample No	Depth	Cohesion (C_u) kN/m^2	Range	Angle of Internal Friction ($\emptyset_u$) Degrees	Range
	0.3	20		3	
Pit1	0.55	55	20-65	5	3-5
	0.85	65		3	
Average		46.7		3.7	
	0.3	70		6	
Pit2	0.55	25	25-95	5	4-6
	0.85	95		4	
Average		63.3		5	
	0.3	60		7	
Pit3	0.55	75	55-75	3	3-7
	0.85	55		6	
Average		63.3		5.3	
	0.3	35		10	
Pit4	0.55	55	20-55	6	6-10
	0.85	20		6	
Average		36.7		7.3	

The cohesive strength (C_u) of the soils neither decreases nor increases with depth in all the pits. However, its values range from 20 to 95 kN/m^2 with average of 63.3 kN/m^2 for soils from pits 2 and 3. The values of the angle of internal friction (φ) obtained vary in a fairly strong interval from 3 to 10 degrees. Values of shear strength parameters reflect the relative heterogeneity of grain size distribution and the particle density of the soil under study.

Total settlement

Results shown in Table 6 indicate that the total settlement (Δh) did not follow any trend with depth as they vary for all the soils. It ranges from 0.0258 to 0.32mm at $100kN/m^2$, 0.080 to 0.52mm at $400kN/m^2$ and 0.31 to 1.125mm at $800kN/m^2$. However, it increases with increase in confining pressure.Guidelines to limiting values are suggested by a number of sources following routine limits, but Skempton and Mac Donald, (1956) appeared to be conventionally acceptable. For sandy soils, maximum total settlement should be equal to 40 mm. In case of isolated footings total settlement may range from 40 to 65 mm for rafts. For clayey soils, maximum total settlement need be equals 65 mm, while for isolated footings settlement range between 65 and 100 mm is adequate for rafts. However, Holtz (1991) favoured about 40 mm as maximum differential settlement that could be between adjacent columns. For this reason, soils do require treatment by addition of stabilizer such as ash (Eberemu, 2011). It is apparent that structure found on the soils will not settle uniformly as a whole regardless of how small the settlement may be, which may lead to damage of the structure. In this case it is logical to resolve that the tolerable settlements of different structures will vary considerably.

Table: 6. Amount of settlement of the studied soils

Sample No	Depth	Total settlement Δh(mm)			
		@ 100 kN	@ 200 kN	@ 400 kN	@ 800 kN
	0.3	0.32	0.52	0.79	1.125
Pit1	0.55	0.28	0.44	0.661	1.09
	0.85	0.0258	0.080	0.218	0.781
Average		0.2086	0.34667	0.55633	0.99867
	0.3	0.054	0.152	0.278	0.482
Pit2	0.55	0.084	0.169	0.490	0.674
	0.85	0.09	0.155	0.265	0.475
Average		0.076	0.15867	0.34433	0.54367
	0.3	0.03	0.09	0.14	0.31
Pit3	0.55	0.086	0.142	0.346	0.565
	0.85	0.10	0.204	0.780	1.31
Average		0.072	0.14533	0.422	0.72833
	0.3	0.056	0.107	0.214	0.39
Pit4	0.55	0.17	0.265	0.48	0.79
	0.85	0.025	0.134	0.210	0.233
Average		0.08367	0.16867	0.30133	0.471

CONCLUSIONS

The settlement estimation of a shallow foundation at early design stages of building construction project becomes necessary in order to select the most safe foundation type such that settlements exceeding allowable limits will be avoided. The key factor required for foundation types in this case, is the consolidation settlement due to clay content. Generally, soil consolidation involves decrease in water content of a saturated soil without replacement of water by air.

The studied soils in natural state are heterogeneous and exist above migmatite gneisses at mostly shallow depth, with variable different properties. The soils are well-graded with particle density typical of clay and sand, which make them not very poor for engineering materials. The average permeability values for the soils relate to size distribution and fall within this range can be said to have low permeability. Hence based on their permeability coefficient, they are good soils for engineering Based on the Atterberg limits, the average consistency of the soils under consideration shows that they are suitable for use as foundation materials.

Inspite of the complexity and heterogeneity in their properties, the soils' shear failure will rarely result in excessive building distortion or collapse. However, this amount of settlements can result in structural damage to a building frame nuisance such as sticking doors and windows, cracks in tile and plaster, and excessive wear or equipment failure from misalignment resulting from foundation settlements.

REFERENCES

Adebisi, N. O., Adeyemi, G. O., Oluwafemi, O. S., and Songca, S. P. 2013. Important properties of clay content of lateritic soils for engineering project. Journal of *Geography and Geology.* Vol. 5, No. 2, pp.99-115.

Adebisi, N. O. and Adeyemi, G. O. 2012. Assessment of compressibility characteristics of Residual laterised soils in Southwestern Nigeria. *Science Focus.* Vol. 17, No. 2, pp.198-208.

Adebisi, N. O. 2010. *Geotechnical investigation of foundation soils in parts of the Basement Complex Terrain of Southwestern Nigeria*. Ph.D. Thesis, University of Ibadan.

Adeyemi, G. O., Ojo, A. A. and Omidiran, M. O. 1990. Relationship between some index properties and crushing strengths of three southwestern Nigerian lateritic clay deposits. *Nigerian Journal of Technical Resouces.* Vol.2, pp.33-38.

ASTM 2011 Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils. America Society for Testing and Materials (ASTM) D4767-11

Carter, M. and Bentley S.P. 1991. *Correlations of soil properties*. Pentech. Press Publishers. London.

Casagrande, A. (1948). "Classification and identification of soils," Transactions, ASCE, Vol. 113, pp. 901-930.

Coduto, D. 2001. *Foundation Design*, Prentice-Hall, ISBN 0-13-589706-8

Christopher C. M. and Benjamin M. D. 1982. The influence of geology on the expansive potential of soil profiles. *Geological Society of America Bulletin*, Vol. 93, pp. 565-571

Das, B. M. 2006. *Principles of geotechnical engineering*. Stamford,CT: Thomson Learning College. Lecture note.

Eberemu, A. O. 2011. *Consolidation Properties of Compacted Lateritic Soil Treated with RHA*. Geomaterals, 1, 70-78.

Esu, E.O. and Ilori, O. 2002. Deformation characteristics of Coastal Plain Sands at a water treatment site, Calabar, Southeastern Nigeria. *Electronic Journal of Geotechnical Engineering* (EJGE). Retrieved 13th September 2004 from file://A:\EJGE%20Paper.htm pp. 1-17

Gopal, R. and Rao, A.S.R. 2000. *Basic and Applied Soil Mechanics*. New Age International (P) Ltd., New Delhi.

Grant, N.K. 1972. Geochronology of Precambrian Basement rocks from lbadan, Southwestern Nigeria Earth planetary Science Letter. Vol.10, pp. 29-38.

Handy, R.L. and Spangler, M.G. 2007.*Geotechnical Engineering: Soil and Foundation Principles and Practice*. McGraw-Hill, Fifth Edition.

Holtz, R. D. 1991. Stress distribution and settlement of shallow foundations. In *Foundation Engineering Handbook,* 2nd ed., ed. H.-Y. Fang, pp. 168–222, Van Nostrand Reinhold, New York.

IS: 2720 (Part 15) 1986. *Indian Standard Methods of Test for Soils: Determination of Consolidation Properties*, Bureau of Indian Standards.

Krebs, R.D. and Walker, E. D., 1971. *Highway materials.* Mc.Grill-Hall. Federal Ministry of Works and Housing Nigeria, ―Specifications for roads and bridges of 1974, 26p.

Malomo, S. and Ogunsanwo, O. 1983. The pre-consolidation pressure of a laterite soil. *Bulletin of the International Association of Engineering Geology*. Vol. 28, Issue 1, pp 261-265

Mahalinga-Iyer, U. and Williams, D. J. (1994) Consolidation and shear strength properties of a lateritic soil. *Engineering Geology*, Vol.38, 1-2, pp. 53-63.

Oyinloye, A.O. 2011. *Geology and geotectonic setting of the Basement Complex rocks in Southwestern Nigeria: Implications on provenance and evolution*. 'Earth and Environmental sciences' Imran Ahmad Dar and Mithas Ahmad Dar, edited pp. 97-118, ISBN 978-953-307-468-9

Ramamurthy, T. N. and Sitharam, T. G. 2005. *Geotechnical Engineering.* S. Chand, New Delhi. 289p.

Seed, H.B. 1967. Fundamental Aspects of the Atterberg Limits. *Journal of Soil Mechanics and Foundations Div., 92(SM4)*, Retrieved from http://trid.trb.org/view.aspx?id=38900

Stanciu, A.and Lungu, I. 2009. A new methodology of determination of soil compressibility. *Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering (ICSMGE)*, Alexandria, Egypt. **DOI:10.3233/978-1-60750-031-5-40**

Skempton, A. and MacDonald, D. 1956. The allowable settlements of buildings. In *Proceeding of Institution Civil Engineers*. Volume 5, pp. 727-784.

Tardy, Y. 1997. *Petrology of Laterites and Tropical Soils*. ISBN 90-5410-678-6.

Terzaghi, k., Peck,R.B. and Mesri G. 1996. *Soil Mechanics in Engineering Practice, Third Edition.* Wiley-Interscience.

Wright, E.P. 1992. *The hydrogeology of crystalline basement aquifers in Africa*. Geological Society, London, special publications, 66, pp.1-27.