Research Journal of Soil and Water Management 1 (1): 10-14, 2010
ISSN: 2075-1095

Plasticity and Compressibility Characteristics of Lateritic Soil from Southwestern Nigeria

B.S. Badmus
Department of Physics, University of Agriculture, Abeokuta, Nigeria

Abstract: Plasticity index and coefficient of compressibility amongst other characteristics were determined for some lateritic soil samples of Southwestern Nigeria with a view to establishing the relationship between their plasticity and compressibility as well as predicting their *in-situ* compressibility and also determine the effect of the parent rocks on the plasticity and compressibility. Two study areas were chosen in Ibadan, Southwestern Nigeria, where ten disturbed and ten undisturbed samples were collected each. The disturbed samples were subjected to a number of geotechnical tests: grain size, distribution analysis, specific grant and consistency test. The undisturbed samples were subjected to consolidation test. The study revealed the rock type in study area A to be quartzschist and area B to be granitic. The specific gravity of the soil grains range between 2.48 and 2.72, while the plasticity index value is between 9.7 and 21.4%. Coefficient of consolidation ranges between 29.39 and 32.56 $mm^2\ mm^{-1}$ with coefficient of volume compressibility raging between 1.08 and $1.94\times10^{-3}\ m^2/KN$. The soil samples are generally well-graded reddish brown, sandy-silt-clay of medium plasticity and compressibility with dominant kaoline clay mineral in the quartzschist derived soil while the dominant clay mineral in the granite derived soil is illite. The most influenced parameter of the parent rocks is the coefficient of compressibility, which revealed the samples of the two study areas to be suitable for construction work as well as landfill site with little compaction.

Key words: Lateritic, compressibility, plasticity, grain size and clay, parent rocks, Nigeria

INTRODUCTION

The most common materials used for construction are lateritic soils because they occur naturally with intense weathering (in the tropics) and there is lack of good quality crushed aggregate as well as economically attractive. Lateritic soils are found in the tropical environment, where there is an intense chemical weathering and leaching of soluble minerals. Laterites are reddish brown well graded and sometimes extend to depth of several tens of metres. They are found almost everywhere in the tropics with wide applications in the construction industries. This makes the study of the characteristic important in the areas of consistency limits, grain size distribution, permeability compaction, consolidation and shear strength.

A lot of research activities have gone on lateritic soils but little emphases have been laid on the relationship between plasticity (consistency limits) and compressibility characteristics. Negligence on the part of construction engineers have led to uncountable road and structural failures within the sub-Sahara Africa. Ashworth (1966) revealed that lateritic soils are gap graded with deficiency in sand and silt-size particles. Gidigasu (1972) worked extensively on lateritic soils of Ghana and concluded that laterite was derived from chemical and mechanical disintegration of the parent materials resulting into concentration of iron and aluminum oxides. Ola (1974) investigated stabilization problems associated with laterite and the modified result is used in production of blocks. Balogun (1982) investigated some physical, geochemical and geotechnical properties of leterite of Shagamu, Southwestern Nigeria; this he found to have significant difference in some index properties.

Adeyemi *et al.* (1990) worked on some index properties and crushing strength of three Southwestern Nigeria lateritic clay deposits with the aim of seeing how the materials could be used for bricks. The result of their findings showed that firing increases the strength tremendously. Adeyemi (2004) investigated the geo-technical properties of lateritic soil developed over quartzschist in Ishara area, Southwestern Nigeria and showed the major mineral clay to be kaolinite with a subordinate amount of illite and montomorillonite.

Geological settings of the study areas: Two locations within the city of Ibadan are chosen for this research work. The first is within the University of Ibadan, Southwestern Nigeria with

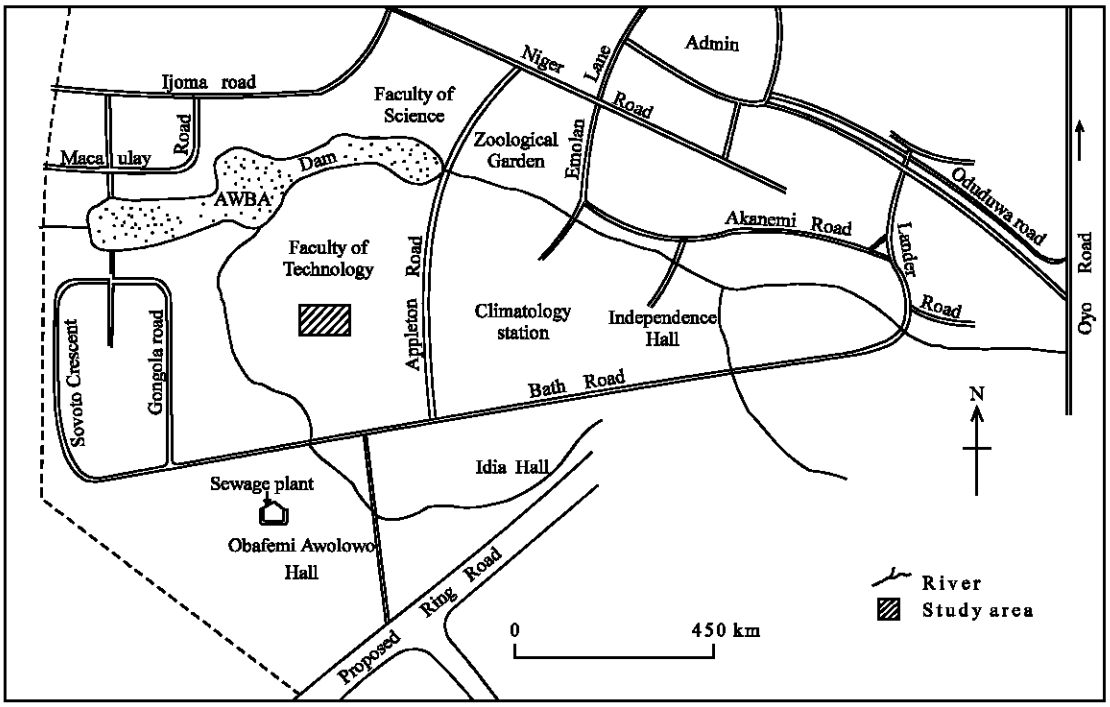

Fig. 1: Location map showing study area A

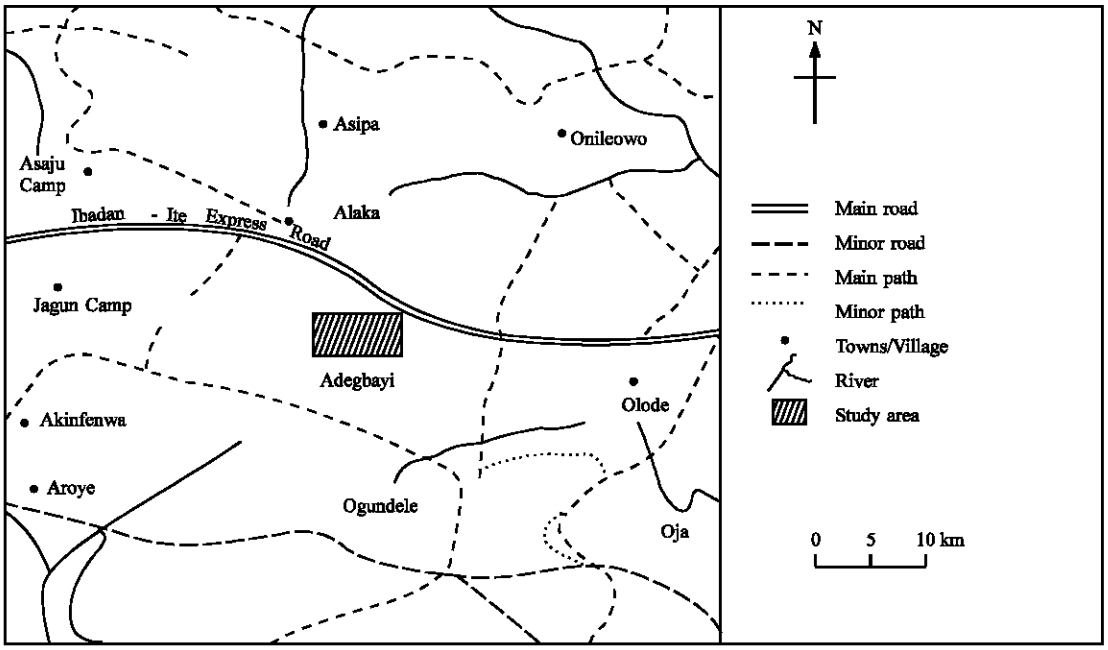

Fig. 2: Location map showing study area B

latitude between 7°27 and 7°29'N, longitude between 4°21 and 4°23'E tagged as study area A (Fig. 1). The second location is around Adegbayi area along Ibadan-Ile-Ife road with latitude between 7°36 and 7°38'N, longitude 4°27 and 4°29'E tagged study area B (Fig. 2).

MATERIALS AND METHODS

Field technique: A total of forty samples were collected with twenty disturbed and twenty undisturbed samples at the two study areas, University of Ibadan and Adegbayi. The sampling was done within an area of 10 m^2 at each location. The undisturbed soil samples were collected through the use of core cutters of about 150 mm in length and 100 mm in diameter. The core cutters were hammered into the borrow pit of about 0.5-1.0 m depending on the topography and the soil profile in the area. The disturbed samples were collected after the collection of the undisturbed samples. Rock samples were also collected from each location to prepare thin section which will give the mineralogical composition of the rocks. The lateritic soil samples collected were soft, cohesive and wet in nature. All samples were reddish brown in colour, collected fresh and not weathered. For sample preparation, the undisturbed samples were collected in a polythen bags to prevent the exchange of moisture content between the soil and the atmosphere. The undisturbed samples were prepared for consolidation tests, while the disturbed samples were air-dried to expel the *in-situ* moisture content, this was done for a period of time, depending on how wet the samples were.

Laboratory analyses: The laboratory analyses were grouped into two: classification test for grain size distribution and consistency limits; consolidation test for co-efficient of consolidation and coefficient of volume compressibility.

Grain size analysis involves two methods: Mechanical and Hydrometer Analyses, which require knowledge of the specific gravity of grains. For wet sieving procedure, 500 g of air-dried soil samples was soaked in 2 g of calgon with 1 L of water (sodium hexametaphosphate). This solution was then stirred and left overnight. The soil sample was washed under tap water for a period of 24 h until the water coming out became clean. This is to separate silt and clay fraction from the coarse fractions, using 0.075 mm sieve. These separated coarse fractions were oven-dried for about 24 h at about 110°C; then dry-sieved using a set of sieves, mechanical shaker, weighing balance sensitive to 0.01 g and sieve brush. The sieves were arranged in order of increasing mesh sizes with the smallest at the bottom and the largest at the top. The oven-dried soil was poured into the stack of sieves and transferred into the shaker, which operated for about 10 min.

For hydrometer analysis: This utilizes the relationship between settling velocity of spherical particle, viscosity of the fluid and the specific weight (density) of the particle using Stoke's law:

$V \alpha D (\delta s - \delta w) \bar{\mu}$, for particles with diameters between 0.002 and 0.2 mm. Some substantial qualities of particles are oven-dried and can pass through 0.063 mm at a temperature of 100 and 110°C. Fifty gram of the oven-dried soil was pulverized and poured into one of the measuring cylinder mixed with 2 g of calgon and 1 L of distilled water.

The mixture is shaken vigorously until a uniform suspension was formed. The hydrometer was immediately inserted into the cylinder and timed immediately at 15 and 30 sec, 1, 2, 4, 8, 15 and 30 min, 1, 2, 4 and 24 h. Fine particle sizes were determined using the Stoke's equation.

Specific gravity: This is the measure of the density of a soil relative to that of water. It is a means of identification and evaluation of lateritic soils as it relates to mechanical strength classification (De Graft-Johnson, 1972). Fifty gram of the soil sample that can pass through sieve 0.425 mm was added to the pycnometer, weighed and recorded as M_2. Sufficient air-free distilled water was added to the soil sample in the bottle and shaken to eliminate air indirection. The bottle and its content was weighed and recorded as M_3. The pycnometer was later filled with distilled water and weighed as M_4. M_1 is the weight of the pycnoneter. Mathematically, the specific gravity was calculated using M_2-M_1/(M_4-M_1)-(M_3-M_2).

Plastic limit: This is the moisture content at which the soil can no longer behave like a plastic material; the soil can be rolled into a thin thread without breaking up. The soil samples were air-dried, pulverized and passed through a sieve slot 0.425 mm, mixed with water to form a homogenous paste. This paste was rolled into balls forming thread of about 3 mm in diameter. The weight of the thread was determined and transferred into an oven of 100-110°C for 24 h. The thread was re-weighed after removing it from the oven. Plastic limit was calculated from the expression:

$$\frac{a-b}{a} \times 100$$

Where:
a = Wet weight of thread
b = Dry weight of thread

Consolidation test: This test was carried out to establish the co-efficient of consolidation and co-efficient of volume compressibility. Coefficient of consolidation can be used to estimate the rate of settlement of any structure built on a compressible soil deposit while coefficient of volume compressibility is used to estimate the amount of settlement of the structure. Consolidation test is usually

done in the laboratory using it Oedometer. Coefficient of consolidation C_v was calculated using the equation:

$$C_v = \pi d^2/4t_i$$

Where:
d = Thickness of the original sample/2
t_i = Time/settlement

For coefficient of volume compressibility

$$Mv = -\frac{dh}{dp} \times \frac{1}{h}$$

Where:
dh = Change in thickness
dp = Change in pressure
h = Average of h_1 and h_2 (thickness of samples)

Knowing the C_v and M_v we can now estimate the coefficient of permeability K for the sample:

$$K = C_v M_v \delta w$$

Where:
δw = Unit of weight of water

The amount of settlement, S, of structures in a compressible soil deposit is estimated from M_v as:

$$S = M_v \delta p\, h$$

Where:
h = Thickness of compressible soil layer
δp = Expected stress from the structure

RESULTS AND DISCUSSION

Grain size distribution: The samples from the quartz schist based soil in study area A showed greater amount of fines ranging from 40-55% with an average value of 46.8%, while sample from granite based soil in study area B range between 30 and 47% with an average value of 38.6%.

These samples show greater amount of coarse fraction between 53 and 70% with an average value of 61.4% (Table 1). This showed that the soils from both study areas are generally well-graded, reddish-brown, sandy-silt-clay soils. The lower the clay size fraction, the higher the coarse fraction and the better the parent rock. The soils from study area B, derived from granite showed that granite is a better construction material when compared with quartzschist.

Specific gravity: The values for the quartzschist derived soil range between 2.6 and 2.72 with an average value of 2.66, while those of the granite derived soil is between 2.48 and 2.70 with an average value of 2.61 (Table 1). The average specific gravity for both study areas fall within the range specified by De Graft-Johnson *et al.* (1969) for lateritic soil. And the higher the specific gravity, the higher the degree of laterization, provided the soils are from the same parent material.

Plastic limit: The plastic limit values obtained for the quartzschist derived soil range between 21.10 and 28.92%

Table 1: Summary of grain size analyses, plastic limit values, plasticity index values, specific gravity, coefficient of consolidation and coefficient of volume compressibility

Samples	Gravel	Sand	Silt	Clay	Amount of fines	Plastic limit values (%) X	Y	Z	Average	Plasticity index values (%) Liquid limit	Plastic limit	Plasticity index	Specific gravity	Coefficient of consolidation C_v	Coefficient of volume compressibility M_v (M^2/KV)
	----------%----------														
A1	3	42	24	31	55	29.01	28.67	29.09	28.92	46.00	28.92	17.08	2.60	32.56	1.67×10^{-3}
A2	4	50	29	17	46	28.00	28.67	28.36	28.34	45.00	28.34	16.66	2.62	30.68	1.19×10^{-3}
A3	6	41	21	32	53	27.82	26.83	26.51	27.05	46.00	27.05	18.95	2.72	32.56	1.45×10^{-3}
A4	5	48	24	23	47	24.29	25.12	24.42	24.61	44.00	24.61	19.39	2.62	30.68	1.08×10^{-3}
A5	6	49	24	21	45	22.66	23.13	23.53	23.11	37.00	23.11	13.89	2.68	29.39	1.53×10^{-3}
A6	6	48	24	22	46	28.19	27.78	28.42	28.13	42.00	28.13	13.87	2.70	30.68	1.53×10^{-3}
A7	8	52	15	25	40	22.95	22.31	22.73	22.66	38.00	22.66	15.34	2.66	29.39	1.51×10^{-3}
A8	14	41	20	25	45	21.05	20.69	21.56	21.10	38.00	21.10	16.70	2.62	32.56	1.48×10^{-3}
A9	6	48	24	22	46	23.77	25.00	24.82	24.53	36.00	24.53	11.47	2.64	29.39	1.43×10^{-3}
A10	8	47	27	18	45	25.38	25.70	24.98	25.35	35.00	25.53	9.47	2.70	30.68	1.15×10^{-3}
B1	4	49	22	25	47	23.36	23.16	21.17	22.56	44.00	22.56	21.40	2.65	30.68	1.43×10^{-3}
B2	2	57	26	15	41	25.33	25.58	24.54	25.15	42.00	25.15	16.85	2.69	32.56	1.71×10^{-3}
B3	4	51	25	20	45	21.43	21.01	20.57	21.00	42.00	21.00	21.00	2.65	32.56	1.71×10^{-3}
B4	7	51	20	22	42	26.06	25.77	26.00	25.94	44.00	25.94	18.06	2.70	30.68	1.77×10^{-3}
B5	10	57	17	16	33	19.57	19.72	20.14	19.81	34.00	19.81	14.19	2.53	31.53	1.76×10^{-3}
B6	10	50	23	17	40	21.21	20.86	21.33	21.13	36.00	21.13	14.87	2.48	32.56	1.52×10^{-3}
B7	3	60	22	15	37	23.57	22.48	22.96	23.00	42.00	23.00	19.00	2.50	30.68	1.81×10^{-3}
B8	2	62	23	13	36	20.00	20.53	20.00	20.18	38.00	20.18	17.82	2.52	30.68	1.94×10^{-3}
B9	3	67	16	14	30	23.84	23.78	24.36	23.99	42.00	23.99	18.01	2.64	32.56	1.84×10^{-3}
B10	8	57	16	19	35	27.14	26.71	26.67	26.84	47.00	26.84	20.16	2.70	30.68	1.81×10^{-3}

with an average value of 25.38%, while those of the granite derived soil range between 19.81 and 26.84% with an average value of 22.96% (Table 1).

Plastic index: The plastic index values of the soil samples from the study area A range between 9.4 and 19.38% with an average value of 15.3%, while in area B, the range is between 14.19 and 21.44% with an average value of 18.14%. Soil samples from study are B derived from granite has higher average plasticity index value than those derived from quart schist in the study area A (Table 1).

Coefficient of consolidation C_v: These values range between 29.39 and 32.56 $mm^2\ min^{-1}$ for the soil analysis in study area A with an average value of 30.87 $mm^2\ min^{-1}$, while in study area B, coefficient of consolidation ranges between 30.68 and 32.56 $mm^2\ min^{-1}$ with an average value of 31.52 $mm^2\ min^{-1}$. Soil samples from study area B have higher average rate of settlement and due to the low values of consolidation co-efficient observed in both study areas, the soils are suspected to be good foundation materials (Table 1).

Coefficient of volume compressibility M_v: The values range from 1.08 and 1.67×10^{-3} m^2/KN for soil samples collected from the study area A, with an average value of 1.42×10^{-3} m^2/KN, while those in the study area B range between 1.43 and 1.94×10^{-3} m^2/KN with an average value of 1.74×10^{-3} m^2/KN (Table 1). Soil samples derived from granite in the study area B revealed higher average co-efficient of compressibility than those of quartzschist derived soil of study area A. The moderate compressibility values make those samples suitable for construction purposes.

Parent rocks influence on plasticity and compressibility characteristics: This was determined by considering the mineralogy and wreathing processes of quartzschist and granite. High percentage feldspartic minerals (plagioclase) and micaceous mineral (biotite and muscovite) coupled with the presence of foliations in the rocks resulted to low resistance in weathering. The soil obtained from the rock contains high amount of fines and little amount of coarse fraction. The dominant clay mineral in the quartzschist derived soil at the study area A is kaolinite, while in study area B for granite derived soil the dominant clay mineral isillite. The space within the three layered structure of illite is prone to penetration of water and result in high plasticity index.

CONCLUSION

From the various tests carried out both laboratory and geotechnical, the study revealed that quartzschist and granite derived lateritic soils are generally well-graded reddish brown, sandy-silt-clay of medium plasticity and compressibility with some little contents of clay of inorganic origin and higher plasticity index. The dominant clay mineral in the quartzschist derived soil of study area A is kaolinite, while illite dominates the granite derived soil in study area B. However, soils in the study area B have higher values of co-efficient of consolidation, C_v and co-efficient of volume compressibility, M_v than those in quartzschist derived soil. The study also showed that the most influenced parameter by the parent rock is the co-efficient of compressibility followed by amount of fines, plasticity index and specific gravity, while the least influenced is the co-efficient of consolidation. From geological and engineering perspectives, quartzschist and granite derived soils are good construction materials and with little compaction, the soils are suitable materials for landfill sites.

REFERENCES

Adeyemi, G.O., 2004. Geotechnical properties of lateritic soils developed over *Quartzschist* in Ishara Area Southwestern Nigeria. J. Mining Geol., 38: 65-69.

Adeyemi, G.O., A.O. Ojo and M.O. Omidiran, 1990. Strength of three southwestern Nigerian lateritic clay deposits. Nig. J. Tech. Res., 2: 33-38.

Ashworth, R., 1966. Highway Engineering. Heinmann Education Book Ltd., London, pp: 62-79.

Balogun, L.A., 1982. The physiochemical and geotechnical properties of lateritic soils from southwestern Nigeria. Proceedings of first International Conference, Nigeria Geotechnical (NGA'82), Lagos, pp: 42-59.

De Graft-Johnson, J.W.S., 1972. Laterite gravel evaluation of road construction. J. Soil. Mech. Div. Amst. Soc. Civil Eng., 98: 1245-1265.

De Graft-Johnson, J.W.S., H.S. Bhatia and D.M. Gidigasu, 1969. The engineering characteristics of the lateritic gravels of ghana. Proceedings of 7th Inernational Conference on Soil Mechanics and Foundation Engineering, Mexico, Vol. 2, Aug. 28-29, Asian Institute of Technology, Bangkok, pp: 13-43.

Gidigasu, M.D., 1972. Mode of formation and geotechnical characteristics of laterite materials of Ghana in relations to soil forming factors. Eng. Geol., 6: 79-150.

Ola, S.A., 1974. The potentials of lime stabilization of lateritic soils. Eng. Geol., 11: 305-317.