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EFFECT OF REMOULDED DENSITY ON CREEP OF BLACK COTTON SOIL

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ABSTRACT

A black clay soil collected using the method of disturbed sampling, from Bako village, along Gwagwalada road, Abuja, was characterized and tested for mineral composition. The clay was compacted at four different compaction energy levels (Reduced Standard Proctor, Standard Proctor, West African Standard and Reduced Modified Energy Levels). The resultant compaction characteristics were used to remold the clay for 72 hours loading consolidation test which was used to evaluate the consolidation and creep parameters as well as the magnitude of creep. Results showed that the clay classified as Clay of High Plasticity (CH) according to unified Soil Classification System and composed predominantly of secondary minerals including montmorillonite. The recompression index was observed to reduce from 0.0522 at reduced standard proctor energy level through 0.0472 at standard proctor energy level to 0.0243 at West African standard energy level which ended up with 0.0143 at reduced modified energy level. The compression index however, reduced from 0.489 at reduced standard proctor energy level to 0.398 at standard proctor energy level through 0.305 at West African standard energy level after which the value increased to 0.324 at reduced modified energy level. The trend of creep shows increase from 248.6 mm at reduced standard proctor to 350.6 mm at standard proctor compaction energy level after which the value reduced to 226.5 mm at reduced modified compaction energy level.

Keywords: *Compaction Energy, Creep, Coefficient of secondary consolidation, One-Dimensional consolidation.*

1 INTRODUCTION

Consolidation and creep of clayey soils are aspects of geotechnical engineering that has been and has continued to be under intensive study. Terzaghi (1941) was the first to evolve an organized and classical mathematical model to represent the progress of consolidation for soils which the author defined as a decrease in the water content of a saturated clay soil without replacement of the water by air. Consolidation was believed to be completed when all the water in the soil pore have dissipated. Laboratory experimental result had shown the author that deformation continued after the completion of consolidation.

Buisman (1936) was the first to study in to detail the deformation that follows the Terzaghi's one-dimensional consolidation model. The author evolved a mathematical model:

$$z_t = \alpha_p + \alpha_s \log_{10} t \tag{1}$$

where α_p and α_s are primary and secondary compression index simultaneously. The author took the parameter α_p and α_s to be linearly proportional to the load p. Grey (1936) explain the secondary time effect and observed that it occurs in all soils but more strong in organic clay soils. Taylor and Merchant (1940) are one of the first to evolve a mathematical model combining creep with consolidation:

$$\frac{de}{dt} = \frac{\partial e}{\partial t} + \frac{\partial e}{\partial p}.\frac{dp}{dt} \tag{2}$$

Where e is the void ratio and p is the vertical effective stress. Taylor (1942) termed the first theory as theory A and presented a new theory B. Theory B pointed out that secondary compression in the previous loading steps has significance influence in the next load increment. This theory also considers the resistance against deformation which is assumed to depend on the rate of deformation. The author evolved a mathematical model as:

$$e = e_{Eop} - C_\alpha Log \frac{t}{t_{100}} \tag{3}$$

where e_{Eop} is the void ratio at the beginning of secondary consolidation and C_α is the slope of void ratio versus time in a semi-logarithmic space.

Koppejan (1948) changed Buisman's mathematical model to resemble Terzaghi's logarithmic compression law by making the parameter to depend on logarithm of vertical effective stress. The author developed a model for direct evaluation of magnitude of creep:

$$\varepsilon = \frac{1}{C_p} \ln\left(\frac{\sigma_v^1}{\sigma_{vo}^1}\right) + \frac{1}{C_s} \ln\left(\frac{\sigma_v^i}{\sigma_{vo}^i}\right) \log t \tag{4}$$

where ε is the magnitude of creep, C_p is the coefficient of primary consolidation, C_s is coefficient of secondary consolidation, σ_{v0} is the surcharge pressure and t is time.

Suklje (1957) criticized Taylor's work pointing out that theory A does not use logarithmic creep law and the stress component in theory B have no clear physical meaning. Both theories cannot be applied to thick clay layers. The author assumed parabolic excess pore pressure distribution and determine the rate of strain which was put equal to the rate of pore water outflow. The stress-strain path was extrapolated to bigger clay thickness by approximation. According to Lambe (1958), the dissipation of pore fluids from the micro voids is the reason for secondary compression. Similarly, Berry and Xu, (1972); De jong, 1968 and Nakaoka *et-al.*, (2004) concluded that secondary compression results from a local mass transfer of water between macro-pores and micro- pores. Gibson et-al (1967) improved Terzaghi's equation by removing the small strain limitation and allowing variable compressibility and permeability of grain skeleton. The effect of the self-weight of the consolidating layer was shown. The resulting equation is giving as:

$$\pm\left(\frac{\rho_s}{\rho_f}-1\right)\frac{d}{de}\left[\frac{k(e)}{1+e}\right]\frac{\partial e}{\partial z}+\frac{\partial}{\partial z}\left[\frac{k(e)}{\rho_f(1+e)}\frac{d\sigma^1}{de}\frac{\partial e}{\partial z}\right]+\frac{\partial e}{\partial t}=0 \qquad (5)$$

where k is coefficient of permeability.

Leroueil *et-al.*, (1985) conducted four different types of one-dimensional odometer tests (constant rate of strain test, controlled gradient test, multiple stage loading test and creep test) on five sites located within the Champlain sea clays. Results showed that the rheological behavior of these clays is controlled by two curves $(\sigma'_p-\varepsilon_v)$ and $(\frac{\sigma'_v}{\sigma'_p}-\varepsilon_v)$. Formula for calculating creep strain is given by Smoltczyk, (2002) as

$$\varepsilon_{cr}=\frac{C_\alpha}{1+e_0}\log_{10}\left(\frac{t}{t_0}\right) \qquad (6)$$

Where e_0 is the initial void ratio, t_0 is initial time at which creep is assumed to start; Cα is the secondary compression which is the slope of e-logt plot of odometer test. The author concluded that Cα changes with change in over-consolidation ratio with its maximum occurring at OCR=1. Meanwhile, Mesri and Godlewski (1977) had proposed that Cα is approximately 0.04 times the compression index C_c for normally consolidated clays.

Alexandre (2006) developed a creep model which was observed to be more general of the earlier models:

$$\sigma_d=\sigma_{df}(\varepsilon)+K(e).\varepsilon^n \qquad (7)$$

Considering specified stress ranges, equation 7 can be rewritten as

$$\sigma_d=E\dot{\varepsilon}+K\dot{\varepsilon}^{-n} \qquad (8)$$

The solution of the equations is given as:

$$\varepsilon=\left(\frac{\sigma_d}{E}\right)-\left(\frac{K}{E}\right).\frac{1}{\left[\left(\frac{K}{\sigma_d}\right)^{\frac{(1-n)}{n}}+\left(\frac{1-n}{n}\right)\frac{E.t}{K}\right]^{\left(\frac{n}{1-n}\right)}} \qquad (9)$$

$$\dot{\varepsilon}=\ .\frac{1}{\left[\left(\frac{K}{\sigma_d}\right)^{\frac{(1-n)}{n}}+\left(\frac{1-n}{n}\right)\frac{E.t}{K}\right]^{\left(\frac{1}{1-n}\right)}} \qquad (10)$$

The effect of loading time on clay soils was studied through oedometer tests considering double and single drainage (Halder *et-al.*, 2017). The author concluded that increased loading time was observed to affect consolidation rates, coefficient of consolidation C_v and secondary compression index Cα. This was attributed to time dependent plastic adjustment of soil fabric. Maria and Maria (2018) presented an experimental evidence which the author believed could contribute to the understanding of the viscous behavior of soft soils. The author presented equation of primary one-dimensional consolidation, including soil viscous resistance and compressibility of water with analytical solution. The general equation for one-dimensional viscous consolidation considering the compressibility of water may be written as:

$$\frac{\partial e}{\partial t}=-C_k\frac{\partial}{\partial t}\left(e-a_v.\sigma'_v\right)+C_v\frac{\partial^2}{\partial z^2}\left(e-a_v.\sigma'_v\right) \qquad (11)$$

Where $C_k=\frac{e}{k.a_v}$ and $C_v=\frac{k(1+e)}{\gamma_w.a_v}$

Using some simplifying assumptions, the equation was mathematically analyzed to give:

$$e=e_0-\sigma.a_v\left\{1-\sum_{m=0}^{n}\left[\frac{\left(Bm_2-c_c\right)}{M\sqrt{A_m}}\ell^{-\frac{Bm_1-t}{2C_nh^2}}-\frac{\left(Bm_1-c_c\right)}{M\sqrt{A_m}}\ell^{\frac{-Bm_2-t}{2(c_n-h^2)}}\right]Sin\left(\frac{m.z}{h}\right)\right\} \qquad (12)$$

$$u=\sigma\left\{\sum_{m=0}^{n}\left[\frac{(Bm_2-c_c).\left(c_k+h^2-\frac{1}{2}\frac{Bm_1c_vt}{c_v}\right)}{M\sqrt{A_m}c_{kt}h^2}\ell^{-\frac{1}{2}\frac{Bm_1-t}{c_{kt}-h^2}}-\frac{(Bm_1-c_c).\left(c_{kt}h^2-\frac{1}{2}\frac{Bm_2c_{vt}}{c_v}\right)}{M\sqrt{A_m}c_{kt}h^2}\ell^{-\frac{1}{2}\frac{Bm_2-t}{c_{kt}-h^2}}\right]Sin\frac{m.z}{h}\right\} \quad (13)$$

These two solutions gave the relationship in terms of void ratio and the pore water pressure.

It has been shown by many authors that variation in compaction energy level affects significantly the geotechnical properties of soils (Daniel and Benson, 1990; Daniel and Xu 1993; Lara *et al.*, 2014; Mada *et al.*, 2013; Singh *et al.*, 2015). But, it is worth to note that all the creep models reviewed above do not consider remolded clay soils specifically; this work is therefore aimed at studying the effect of remolded densities on the black cotton soils of Nigeria.

2 METHODOLOGY

The material used in this study is mainly black cotton soil collected from Bako village along Gwagwalada road, Federal Capital Territory, Abuja, Nigeria. The clay was collected at depth between 0.5 – 1.2m using the method of disturbed sampling. The clay was immediately transferred to the laboratory, air-dried and pulverized using the method highlighted in part 1 of BS 1377 (1992).

The method involves carrying out index properties test (Liquid limit test, plastic limit test, mechanical sieve analysis, hydrometer analysis test and specific gravity tests) on the black cotton soil so as to classify the clay and determine the percentage composition of the silt and clay sized particles contained in the clay soil. X ray diffraction test was carried out on the clay sample. A crushed sample was collected and crushed and small amount of ethyl alcohol was added to stop the hydration reaction. About 10g of the grounded clay was immersed in ethyl alcohol and kept in desiccator at room temperature. The powdered sample was loaded on a sample holder. The powdered samples were scanned with an X ray powder diffractometer using copper and potassium.

Compaction test was then carried out on the clay soil at four different compaction energy levels. The Reduced Standard Proctor energy level (Daniel and Benson, 1990; Daniel and Wu, 1993), Standard Proctor energy level (Proctor 1933), West African Standard energy level (Nigeria General Specification for Road and Bridge Works, 1992), Reduced Modified Standard energy level. This is to allow for the use of four different standard compaction energy levels to obtain wide range of densities which is necessary to achieve the aim of this study.

Reduced Modified Standard proctor is included as a new method which has not been specified in literature. The method involves the use of 4.5kg rammer falling through the height of 46 cm in to a standard compaction mold with volume of 944 cm^3. The soil was placed in the mold at five layers with each layer receiving 15 blows. This is lower than the modified compaction which has 25 blows per layer but higher than West African Standard which has 10 blows per layer. The Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) were evaluated for each of the compaction energy levels. These compaction characteristics were used to mold specimen for consolidation tests.

The consolidation test was conducted using an odometer device and a transparent cell within which a mold was placed. The mold which has a diameter of 50 mm and height of 20 mm, was covered top and bottom with a porous stone (double drainage) was always lubricated with silicon grease to prevent friction between the clay and the mold during consolidation test. Filter papers were placed between the porous stone and the clay contained in the mold to avoid clay soil particles migrating in to the pores of the porous stone to block the pores. Each loading was applied for 72 hours before addition of further loads. This would allow the deformation to enter secondary stage. Five loadings whose present load doubled the previous one were applied before unloading for further 72 hours. The parameters obtained from the consolidation tests were used to evaluate the magnitude of creep from the model developed by (Smoltczyk, 2002).

3.0 RESULTS AND DISCUSSION

The result of the physical properties of the test soil used for this study found to be almost exact, if not exact to that of black cotton soil is shown in Table 1. From the Table, the clay is classified under clay of high plasticity (CH) and is very prone to creep settlement.

Table 1: Physical Properties of Test Soil

Description	Quantity
Sand (%)	18.4
Silt (%)	28.9
Clay (%)	53.0
Liquid Limit (%)	64.3
Plasticity Index (%)	35.9
Specific Gravity	2.66
Unified Soil Classification	CH

The result of the mineral composition of the clay soil is shown in Table 2 while the graphical representation of the XRD result is also shown in Figure 1.

Table 2: Mineralogical Composition of Clay

Description	**Quantity**
Quartz (%)	50.00
Ankerite (%)	6.25
Calcium Silicide (%)	6.25
Montmorillonite (%)	6.28
Anorthite (%)	8.33
Sodium Aluminium Silicate Hydrate	6.25
Anothoclase	8.33
Orthoclase	10.07

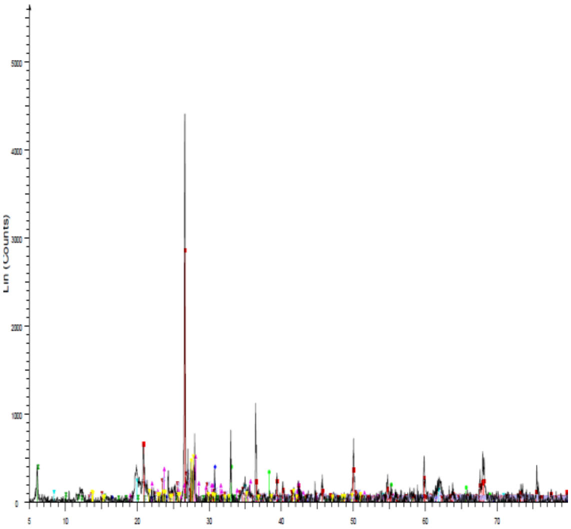
Figure 1: XRD Result of Test Soil

From Table 2 and Figure 1, the soil contained substantial composition of secondary minerals including troublesome montmorillonite mineral. This will increase the activity of the clay and consequently, the creep response of the clay.

2.1 COMPACTION CHARACTERISTICS

The result of the compaction characteristics of the clay soil at varied compaction energy level is shown in figure 2 and 3. Figure two show the variation of maximum dry density (MDD) with compaction energy level while figure 3 shows the variation of optimum moisture content (OMC) with varied compaction energy level.

Each compaction test result shows increase in dry densities with increase in moisture contents down to optimum moisture content after which the dry densities decreases. These trends are in agreement with Lambe (1958) and Yusoff *et al*. (2016) who attributed the variation to soil structure and electrical double layer theory.

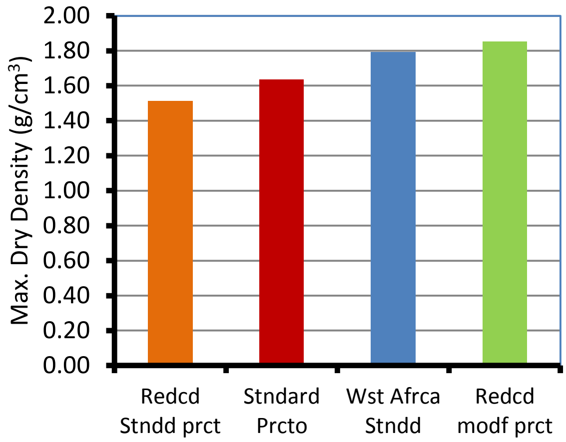

Figure 2: Variation of MDD with Compaction Energy Levels

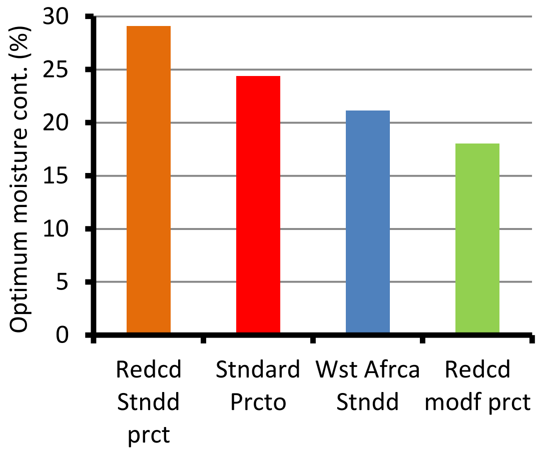

Figure 3: Variation of OMC with Compaction Energy Levels

Clay soils usually possess attractive Van der Waals' forces between two clay particles and a repulsive force which is due to double layers of adsorbed water trying to come in to contact with each other. The attractive force tends to remain the same while the repulsive force increases with increase in water. If the net force between the particles is attractive, flocculated structure will result and if the net force is repulsive, dispersed structure will result. Addition of small amount of water will reduce the activity of the double layer which will encourage the clay particles to be in a flocculated form thus resulting in to soft soil mass. Increase in water will reduce the action of the double layer around clay particles thus resulting in particles of dispersed form which are dense in structure.

Figures 2 shows increase in MDD with increase in compaction energy level from 1.512g/cm^3 at reduced standard proctor compaction energy level through 1.635g/cm^3 at standard proctor energy level to 1.793g/cm^3 at West African Standard energy level which ended with 1.851g/cm^3 at reduced modified standard proctor energy level. The OMC however, reduces with increase in compaction energy level from 29.1% at reduced standard proctor compaction energy level through 24.4% at standard proctor energy level to 21.1% at West African Standard energy level which ended in 18.0% at reduced modified standard proctor energy level. This trend is in agreement with Daniel and Benson, 1990; Daniel and Xu (1993), Lara *et al.*, (2014), Mada *et al.*., (2013), and Singh *et al.*, (2015).

2.3 CONSOLIDATION CHARACTERISTICS

The result of consolidation tests carried out on the clay soil at four different compaction energy levels are shown in figure 4. It was observed that recompression index reduced from 0.0522 at reduced standard proctor energy level through 0.0472 at standard proctor energy level to 0.0243 at West African standard energy level which ended up with 0.0143 at reduced modified energy level.

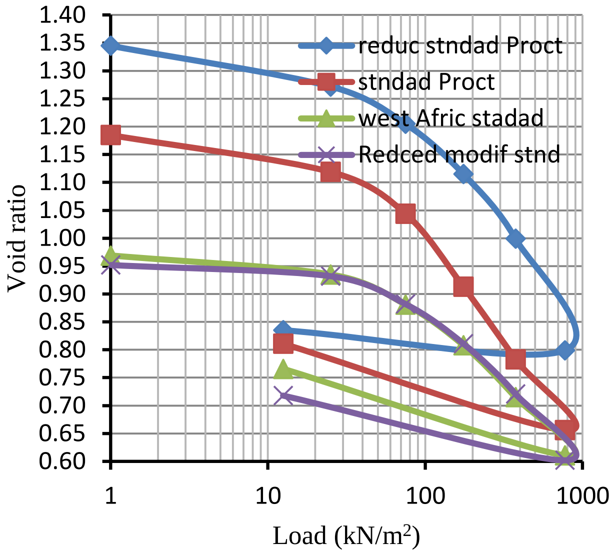

Figure 4: Void Ratio-Effective Stress Relation for Varied Compaction Energies

The compression index however, reduced from 0.489 at reduced standard proctor energy level to 0.398 at standard proctor energy level to 0.305 at West African standard energy level after which the value increased to 0.324 at reduced modified energy level. These trends can be attributed to the rate of water absorption of the clay during the 24 hours soaking with a seating load, before the application of the subsequent loads and the rate at which water dissipate out of the clay during consolidation.

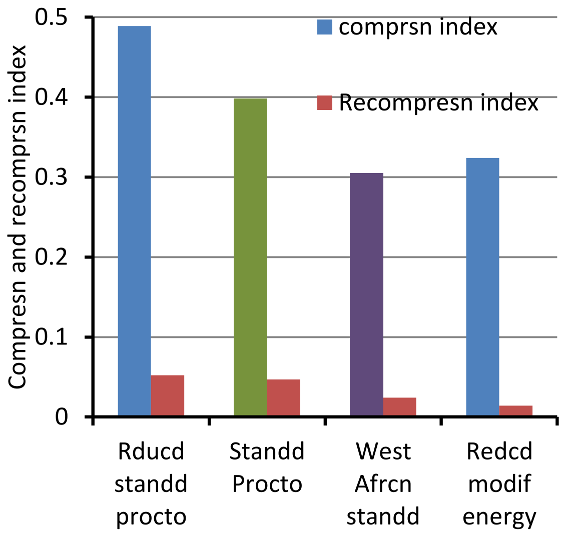

Figure 5: Compression and Recompression Index with Varied Compaction Energies

At lower energy level (reduced standard proctor), the molding moisture was higher while the dry density was lower. This allowed only little more water to be absorbed in to the clay mineral structure and the void spaces during the initial 24 hours soaking. During the consolidation process, only outward dissipation of water will occur and will continue until all water contained in the voids are dissipated. This process will be free and fast since there is no further absorption of water by the clay particles thus, resulting in higher compression index. At higher compaction energy level (reduced modified energy level), there was still more affinity for water absorption by the clay mineral structure which will reduce outward dissipation of water during consolidation. This is probably responsible for the increase in compression index at reduced modified energy level.

2.4 CREEP CHARACTERISTICS OF CLAY

Each of the loadings during consolidation test was allowed for 72 hours before increment of next load. This is to allow for evaluation of secondary consolidation properties which can be used to evaluate the secondary compression index and subsequently the magnitude of creep.

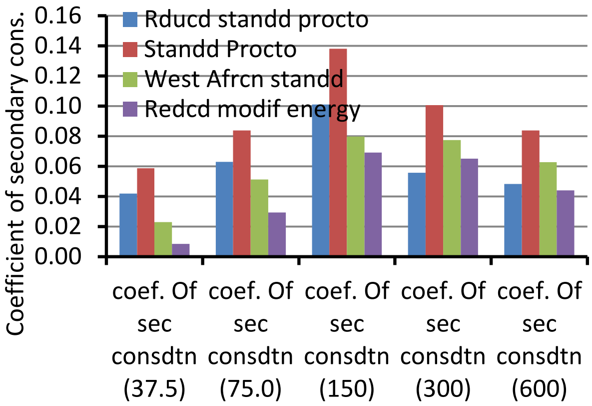

Figure 6: Coefficient of Secondary Consolidation for Varied Compaction Energy and at Varied Loading

The result of coefficient of secondary consolidation for the varied load increments and at varied compaction energy levels is shown in Figure 6. The trend for all the loading increments showed increase from the first loading (25.0 kN/m^2) to its maximum at the loading of 175 kN/m^2 after which the values reduced. Variation of coefficient of secondary consolidation with change in loading has been observed by Larsson (1986), but the work did not give any specific trend in their relationship. The trend observed in Figure 6 probably resulted from the time resistance concept (R) as observed by Janbu (1969). Since excess pore pressure is zero during creep, time can be taken as an action and creep strain as a reaction.
The magnitude of creep for three years was evaluated using Smoltczyk (2002) model for each compaction energy level at the loading of 375 kN/m^2 and the results are shown in Figure 7.

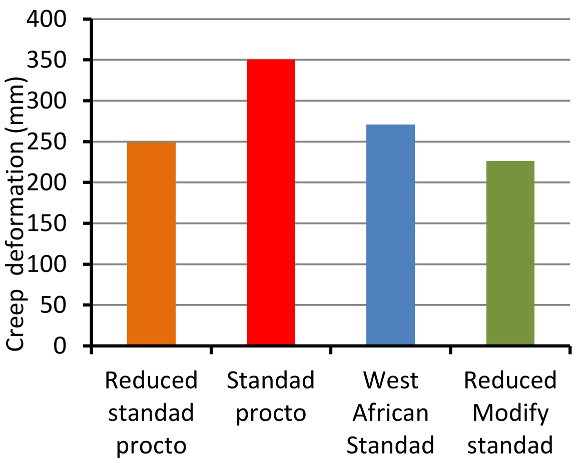

Figure 7: Magnitude of Creep in 3 Years for Varied Compaction Energies

The trend shows increase from 248.6 mm at reduced standard proctor to 350.6 mm at standard proctor compaction energy level after which the value reduced to 226.5 mm at reduced modified compaction energy level. The low value observed in the reduced standard proctor results from the excessive initial void ratio.

3 CONCLUSION

1. The soil used is classified as clay of high plasticity according to Unified Soil Classification system and A-7-6 according to AASHTO soil classification system.
2. The compression index decreased from 0.489 at reduced standard proctor down to 0.305 at West African compaction energy level after which the value increased to 0.324 at reduced modified compaction energy level.
3. The coefficient of secondary consolidation for all the compaction energy levels was observed to increase from loading of 25 kN/m^2 to its maximum at loading of 175 kN/m^2 after which the values dropped.
4. The magnitude of creep was observed to increase from 248.6 mm at reduced standard proctor compaction energy level to maximum of 350.6 mm at standard proctor compaction energy level after which the values dropped to 226.5 mm at reduced modified compaction energy levels.

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