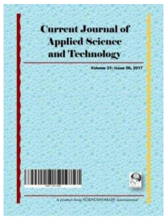

Current Journal of Applied Science and Technology

30(1): 1-13, 2018; Article no.CJAST.43920
ISSN: 2457-1024
(Past name: British Journal of Applied Science & Technology, Past ISSN: 2231-0843, NLM ID: 101664541)

Critical State of Lateritic Soils Taken from Varied Burrow Pit Depth

B. A. Ajayi[1*], H. M. Alhassan[2], H. Y. Gambo[2] and U. Hassan[3]

[1]*Siraj Nigeria Ltd., Plot 1042, Central Business Area, Abuja, Nigeria.*
[2]*Department of Civil Engineering, Bayero University, Kano, Nigeria.*
[3]*Department of Civil Engineering, Ahmadu Bello University, Zaria, Nigeria.*

Authors' contributions

This work was carried out in collaboration between all authors. Author HMA designed the study and managed the literature searches together with author BAA who also wrote the protocol and the first draft of the manuscript. Authors HYG and UH managed the experimentation and the analyses of the study. All authors read and approved the final manuscript.

Article Information

DOI: 10.9734/CJAST/2018/43920
Editor(s):
(1) Dr. Harry E. Ruda, Professor, Stan Meek Chair Professor in Nanotechnology, University of Toronto, Director, Centre for Advanced Nanotechnology, University of Toronto, Canada.
Reviewers:
(1) Rosario García Giménez, Autonomous University of Madrid, Spain.
(2) K. Shyam Prakash, Pvp Sidhartha Institute of Technology, India.
(3) Pavel Kepezhinskas, USA.
Complete Peer review History: http://www.sciencedomain.org/review-history/26829

Original Research Article

Received 17 July 2018
Accepted 02 October 2018
Published 25 October 2018

ABSTRACT

The study aimed to investigate the behaviour of Lateritic soil at a critical state. Four undisturbed samples were taken at 1.5 m and 3.0 m respectively. Laterite materials were sourced from Janguza Burrow Pit. The samples were collected by manual digging to a depth of 1.5m and 3.0 m and inserted in polyethene bags. To classify the material, the tests conducted on the soil samples included natural moisture content, Atterberg limits, specific gravity and particle size distribution. Furthermore, Undrained Triaxial test was conducted on the soil samples with the measurement of pore pressure to determine the critical state condition of the Lateritic soil. The average moisture content value for 1.5 m depth was 3.83%, and that for 3.0 m depth was 5.57%. Atterberg limits at 1.5m depth are as follows: Liquid limit (LL) =22.9%, Plastic limit (PL) =12.6% and Plasticity Index (PI) =10.3% and for 3.0 m depth the values are Liquid limit (LL) =22.1%, Plastic limit (PL) =9.48% and Plasticity Index (PI) =12.7%. The average specific gravity is 2.67 and 2.71 for 1.5m and 3.0m depth respectively. Samples at 1.5m are found to be Well–Graded (GW) gravel-sand mixtures from

**Corresponding author: E-mail: abisco123414@gmail.com;*

sieve analysis result, while samples at 3m depth were found to be Well-Graded except sample A and D which showed poorly graded sand (GP) and gravel-sand mixtures. The samples were found to be overconsolidated and at dry side of critical. The state of stress and void ratio ranges from 30.55 kN/m^2 to 60.47 kN/m^2 and 0.12 to 0.18 respectively. Pore pressure was stable at a range of 0.006%- 0.045% for 1.5m depth and 3.0m depth was stable at 0.005%- 0.004%. The sample reached critical state condition at deviator stress ranges 229.74 kN/m^2 to 274.31 kN/m^2 for 1.5 m and 3.0 m depth. The average values of the critical state soil parameters at 1.5m and 3.0 depth are λ=0.174, Γ=2.07 and M = 0.885 and λ=0.299, Γ=2.839 and M = 0.813, respectively. The results indicated that both samples were heavily overconsolidated. Overconsolidated soils heave when used in road construction, thus presenting maintenance challenges to road authorities. It is important to carry out periodic checks on the state of consolidation of soils to avoid the use of overconsolidated soils in construction. Similarly, to ensure that soils from burrow pits performed satisfactorily an oversight function should be exercised by the appropriate ministry.

Keywords: Lateritic soil; burrow pit; critical state; wet side; dry side; over consolidation.

1. INTRODUCTION

Burrow pit material is an important consideration in the selection of suitable lateritic material for road construction. Identification, selection and testing of suitably identified laterite deposits are used to affect designs to pavement layers. Once suitable burrow pits are found, they are burrowed to bedrock, and no attempts are made to confirm if the material maintains the properties originally attributed to it. Thus laterite materials sampled at different depths may be used for road construction, but that may have been subjected to different overburden pressure. It is unlikely that soils that have been subjected to varied overburden pressure will exhibit similar performance behaviour in service.

Soil failure in engineering projects is commonly attributable to stress changes in service due to the exceedance of design loads, the presence of water or to bulking caused by the breakdown of soil particles to smaller sizes. To understand the failure of these infrastructures while in service, it is necessary to trace the stress levels in soils up to the critical state. This would reveal the behaviour of these soils prior to failure and the extraction of information on how to manage in-service soils successfully.

It is of great importance in geotechnical engineering to make a realistic prediction of the behaviour of soil under various conditions. Most engineering soils are tested for failure at the peak state or maximum deviator stress, and the parameters of the soil at these states are used to affect designs of foundations. While in service, the stress changes are not monitored until distress manifests. Understanding the behaviour of soils under different loading conditions need to be clearly understood as a slight variation in the existing condition could result in enormous changes in the state of the soil. The paper is organised as follows; section 1 presents the introduction while section 2 reviews relevant literature on the subject. Section 3 explains the materials and methods employed in the study. Section 4 presents the results and their implications for the burrowing material. The conclusions and recommendations are given in section 5.

2. LITERATURE REVIEW

The concept of Critical state is a useful tool for understanding the behaviour of soils. Classical tests on the critical state of soils have been carried out by Schofield and Wroth [1], as well as by Roscoe et al. [2] and Burland [3] in their famous modified cam clay tests. The study of this important concept in soil mechanics gives a clear description of soil under an induced stress, and the interlocking achieved by densification or over consolidation is absent in the case of dense soils. The metastable structure of loose soils also collapses, and the state in which the soil could either compress or dilate is referred to as the critical state strength of the soil [4]. According to Zhao et al. [5] critical state soil mechanics acts as a cornerstone to soil mechanics because it gives the final point of a soil deformation process which is necessary for establishing soil constitutive models. Five different criteria of failure, from which the shear strength of a soil is determined are, the maximum or peak deviator stress criterion, traditionally associated with the testing of soil samples; the maximum stress ratio criterion preferred to the peak stress criterion in some ways because it can provide a better correlation of shear strength with other parameters, or between different types of test, it is particularly useful for clays in which the deviator stress continues to increase at large strains; Limiting strain criteria not often used except in multistage drained triaxial tests. When a soil experience deformation due to stress increase, it moves from peak state to critical state and to residual state.

The critical state of soils has been defined in the literature as in Yimsiri and Soga [4], Zhao et al. [5], Wang et al. [6], Jefferies et al. [7] and Marto et al. [8]. The definitions can be grouped into three categories. Those that consider critical state from the point of view of consolidation, those that view it regarding changes in void ratio and definitions centred on constitutive soil models. All of these use the critical state line as a reference line to evaluate the state parameter which could be used as a model to predict the behaviour of soil materials.

Critical state soil research has been conducted for fine-grain soils mainly and clay soils in particular. Been and Jefferies [9] studied the critical state of loose sandy soils and derived their critical state parameters. Various authors hold the opinion that the critical state of soils does not change under large shear strains, [8] for example. Kayadelen et al. [10] studied the critical state parameters of an unsaturated residual clayey soil from Turkey and demonstrated that matric suction has no influence on the parameters M and λ. Waseem et al. [11] did a comparative study of fly-ash modified cam clay and fly-ash at different consolidation level and confining pressures and evaluated their critical state parameters under undrained triaxial compression tests.

Also, Wang et al. [6] tested unsaturated silty soil under drained conditions in a triaxial cell and found that applying suction to an initially saturated specimen influences the stress-strain behaviour and critical state characteristics. Another work by Zhao et al. [5] studied the critical state parameters of unsaturated soils using steady state thermodynamic process. They developed the necessary conditions for the deformation of the unsaturated soils to reach a critical state based on the theory of local equilibrium thermodynamics. Work by Hamidi et al. [12] considered the effect of temperature on triaxial tests using the saturated cam clay model and the impact temperature has on their mechanical behaviour.

Studies on critical state of soil mixtures with varying content of various additives can be found in Lopera Perez et al. [13] who studied the micromechanical analyses of the effect of rubber size and content in sand-rubber mixtures at the critical state; The critical state line and state parameters obtained by Vu et al. [14] for sand-fines mixtures indicated that there is a unique critical state line with specific fines content; Hsiao et al. [15] worked on the effects of silt contents on the static and dynamic properties of sand-silt mixtures and Mashiri et al. [16] determined the shear strength and dilatancy behaviour of sand-tyre chip mixtures. They found that tyres chips significantly influence the shear strength and dilatancy behaviour of sand-tyre mixtures.

The critical state of dense granular soils have also been studied as in Yan and Zhang [17] who studied the fabric, and the critical state of idealised granular assemblages subject to biaxial shear and the results show an anisotropic distribution of particle arrangement, contact force and void space upon shear. At the critical state, particles are aligned with their long axis parallel to the major principal stress plane. The work in Sitharam et al. [18] focused on the critical state behaviour of granular materials using three-dimensional discrete element modelling. In this study, granular materials, mainly laterites taken from burrow pits was evaluated at critical state to predict their behaviour in service.

3. MATERIALS AND METHODS

This study was carried out for a burrow pit located at Km 8 along Gwarzo road, in the Kano State of Nigeria with coordinates: latitudes 11º 58.25´ N and longitude 8º 22.48´ E. The in-situ state soil was observed to be reddish brown and dark brown from physical examination and had some coarse content. The geology of the area comprises of the Pre Cambrian to Upper Cambrian (i.e. the basement complex), the Jurassic (Younger granite) and Quaternary Chad formation. The mineralogical composition of the soil is Kaolinite, Gibbsite, Goethite, Muscovite and Quartz. The soil was handled as per BS 1337 [19] and taken to the Soil Mechanics Research Laboratory, Civil Engineering Department, Bayero University Kano.

3.1 Method of Research Work

This research has both a field and laboratory component. The fieldwork entail sourcing suitable and sufficient sample without contamination for the tests. The soil sourced was classified according to BS 1337 [18]. Samples were collected at 1.5m and 3.0m depths to establish the effect of overburden pressure on the soil samples and to see the effect of variation in depths with reference to the critical state. For this study samples collected at 1.50 m depth are referred to as S15, while those taken at 3.0 m depth are described as S30. Four spatially different sampling points were used for this research work namely S15A, S15B, S15C and S15D for a depth of 1.5m, while for a depth of 3.0m the sampling points are S30A, S30B, S30C and S30D. For Atterberg tests, samples that passed through 0.425 mm sieve were used. For the Triaxial compression test undisturbed sample were used for determining the critical state of the sample soil.

Preliminary tests were carried out on the lateritic soil samples to provide a basis for classification and determination of the preliminary characteristic of the soil and its suitability for engineering purpose using their physical characteristics and appearance. To establish the behavior of the investigated material at a critical state, stress was induced on the lateritic soil using Triaxial Compression Equipment.

4. RESULTS AND DISCUSSION

4.1 Natural Moisture Contents Analysis

The natural moisture contents of the two samples, i.e. S15 and S30 are shown in Table 1. Samples were taken at four different points, A, B, C and D, and their moisture contents were determined. Across the samples, there is no pattern in moisture content results. The result varies across the sampling points. This is true for both samples S15 and S30. Across the depth, samples S30 all have higher values of moisture content over the corresponding S15 values. Consequently, the S30 values have average moisture content 1.74% above the S15 samples.

4.2 Physical and Index Properties of Samples

In the liquid limit results of Table 2, no clear trend in the results can be seen. The values vary across the sampling points as well as across the depth of sampling. The average Liquid limit for sample S15 is higher than sample S30 by 3.62 %. The plastic limit follows similar variability as the liquid limit results across both the sampling points and the depth of sampling. The percentage plastic limit difference between the S15 and S30 samples is 24.8%. The Plasticity index (PI) in Table 2 shows similar characteristics as the liquid limit and plastic limit results; variable plasticity index results across sampling points.

The average PI values for S15 and S30 samples have a percentage difference of 18.89% between them as shown in Table 2. The index parameters were measured in accordance BS1377 (1990) using the Casagrande apparatus. The particle size grading characteristics of all the soil samples are shown in Fig. 1 while the description of the sample soils used is shown in Table 3.

Table 1. Natural moisture content

	A %	B %	C %	D %	Average %
Sample S15	1.20	4.53	3.85	5.74	3.83
Sample S30	6.20	5.49	4.75	5.82	5.57

Table 2. Summary of physical /index properties of soil

Properties	**Results for sample for different depths**							
	S15A	**S15B**	**S15C**	**S15D**	**S30A**	**S30B**	**S30C**	**S30D**
Nat. Moist. Cont. %	1.20	4.53	3.85	5.74	6.20	5.49	4.75	5.82
Liquid Limit %	17.70	29.60	18.60	25.70	18.90	26.00	21.00	22.30
Plastic Limit %	10.80	24.10	6.50	9.00	8.60	18.30	4.30	6.70
Plasticity Index %	6.90	5.50	12.1	16.7	10.30	7.70	16.7	15.6
Specific Gravity	2.68	2.66	2.65	2.67	2.73	2.70	2.72	2.70
% Retained #200mm	0.63	0.04	0.40	0.33	7.03	0.37	0.17	0.23
Coefficient Cu Uniformity	5.56	18.18	9.23	8.13	17.93	3.89	9.00	3.50
Coefficient Cc Curvature	0.89	0.28	0.26	0.43	1.12	0.71	1.00	0.58

Table 3. Classification of soil samples using unified soil classification system

Soil sample	Soil type	Gradation	Symbol
S15 A	Gravelly soil	Well graded (For grain –size distribution)	GW
S30 A	Gravelly soil	Poorly graded (For grain –size distribution)	GP
S15 B	Gravelly soil	Well graded (For grain –size distribution)	GW
S30 B	Gravelly soil	Well graded (For grain –size distribution)	GW
S15 C	Gravelly soil	Well graded (For grain –size distribution)	GW
S30 C	Gravelly soil	Well graded (For grain –size distribution)	GW
S15 D	Gravelly soil	Well graded (For grain –size distribution)	GW
S30 D	Gravelly soil	Poorly graded (For grain –size distribution)	SW

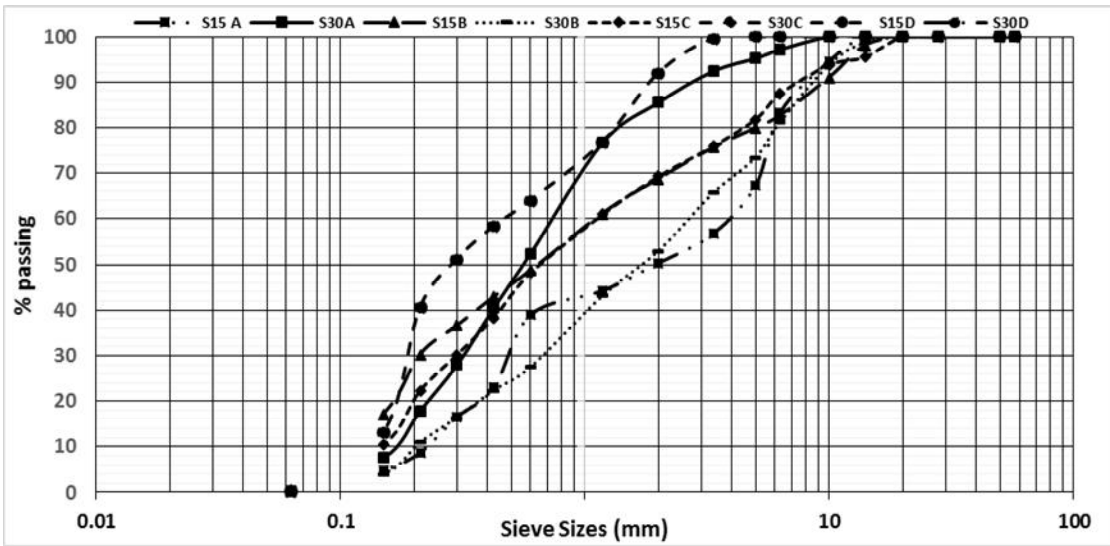

Fig. 1. Particle size distribution

From Table 3, sample S15A shows soil type to be gravelly soil while the index property of the soil is well- graded soil (GW) when compared with S30A where the soil type is gravelly soil and the index property is poorly graded soil (GP).

For sample S15B from Table 3 the soil type is gravelly soil type, and the index property is well-graded soil (GW), same as the soil sample S30B (GW). Sample S15C and S30C show soil type to be gravelly soil with index property to be well-graded (GW).

Also, for soil sample S15D the soil type is gravelly soil with an index property of well graded (GW) but when compared with soil sample S30D the soil type is sandy soil while the index property is poorly graded (SW).

4.3 Compression Test Results

The compression test results on the soil samples are shown in Fig. 2 and 4 for samples S15A and S30A. Clearly, sample S30A carries more loads at peak state than sample S15A. The peak load for sample S30A is 190% more than the peak load for S150A. This might be due to variation in the grading characteristics of the two samples as shown in Fig. 1. Clearly, S30A sample is poorly graded but have bigger soil particle sizes than S15A. Hence, is more able to resist load than S15A. Besides, the specific gravity is higher than the S15A sample by 1.9%. The deviator stress failure for the two samples is respectively 251.03kN/m^3 and 281.76kN/m^3.

The stress-strain behaviour of the samples is shown in Fig. 3 for S15A and S30A. This indicates that the pore water pressure at critical state is stable at a range of (0.01-0.013 %) strain for sample S15 A and for sample S30 it is (0.01-0.02%). The pore pressure at failure can be seen to be 100 kN/m^2 for sample S30A as compared with 40 kN/m^2 for sample S15A. Another feature of Fig. 3 is that sample S15A can sustain a large axial strain that S30A and can thus be expected to have a residual strength whereas sample S30A may not.

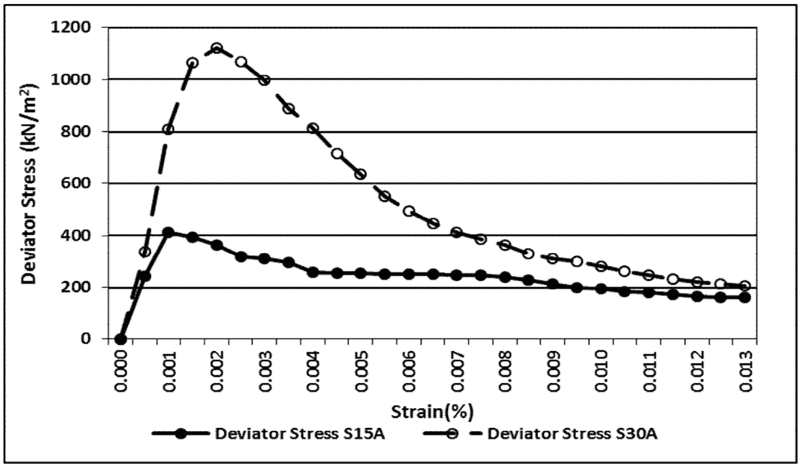

Fig. 2. Deviator stress versus Axial Strain for sample S15A and S30A

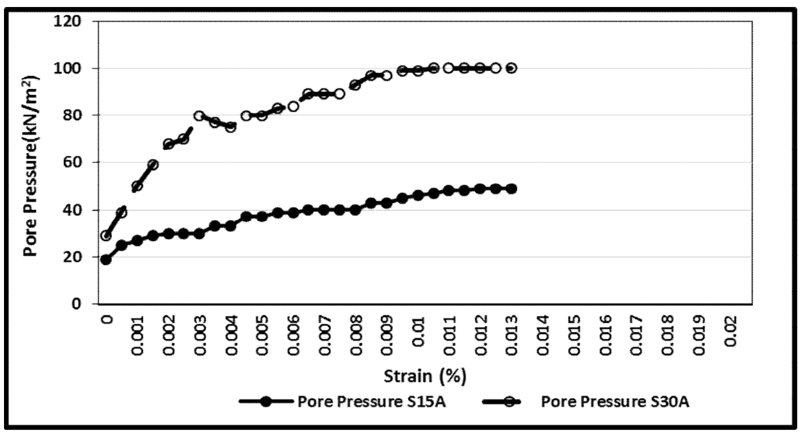

Fig. 3. Pore pressure versus Axial Strain for sample S15 A and S30 A

The results of the compressive stress for sample S15B and S30B are shown in Fig. 4. For sample S15B the deviator stress at failure was 208.5 kN/m^2 while that of sample S30B was 226.2 kN/m^2. As can be clearly seen from the diagram, the peak stress between the two samples varies by about 158%, suggesting that higher overburden pressure of samples support larger loads.

The pore water pressure for sample S15B and sample S30B are indicated in Fig. 5. A strain range of (0.004-.012%) was observed for sample S150B while that of sample S30B also observed for (0.002-0.001%). The pore pressure at failure for the two samples is 50 kN/m^2 for S15B and 250 kN/m^2 for sample S30B.

The behaviour of sample S15C and S30C are consistent with those of the corresponding A and B samples. There are higher levels of deviator stress for sample S30C than S15C Sample S30C had a deviator stress of 256.13kN/m^2 while sample S15C had a deviator stress of 239.50kN/m^2 giving a percentage difference of 6.9%.

The pore water pressure of sample S15C varied continually with a change in strain from 32kN/m^2 to 100kN/m^2 as shown in Fig. 7. For the S30C sample, the pore water pressure remains substantially the same over the range of strain values recorded for the two samples. The range of strain for sample S15C is between (0.01-0.015%), while sample S30C has a range between (0.005-0.015%).

The stress-strain curves for samples S150D and S30D are shown in Fig. 8. Like the others, sample S30 has a higher peak stress load than the sample S15D suggesting that peak load increases with depth. The percentage difference in peak load between the two samples is 36.4%. The deviator stress at critical state is 219.94kN/m2 for S15D and 333.05 for S30D. Again, the grading of the two samples, as well as their specific gravity, may be responsible for the observed trend. Quite expectedly, the specific gravity is higher for S30D than S15D. The percentage difference is 1.12%.

The pore water pressure strain plots for the two samples S15D and S30D is shown in Fig. 9. Sample S15D has a strain range of (0.00-0.005%) and sample S30D has a strain range of (0.002-0.015%). Quite clearly sample S15D can sustain a larger strain at constant load than sample S30D which is the critical state of the sample. This cannot be said of sample S30D which exhibits different strain with a change in pore pressure.

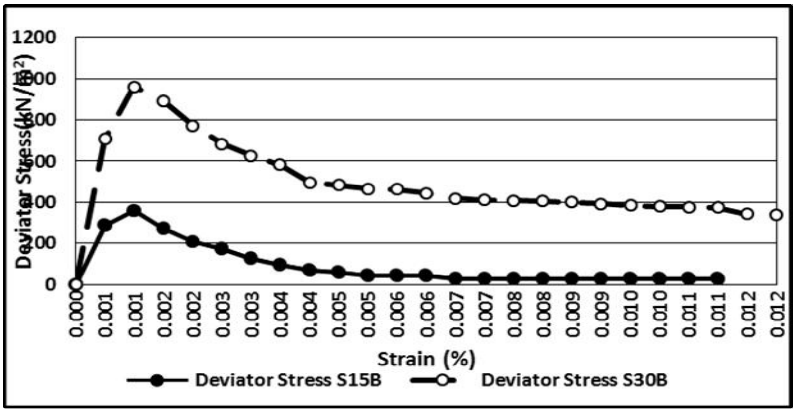

Fig. 4. Deviator stress versus axial strain for sample S15B and S30B

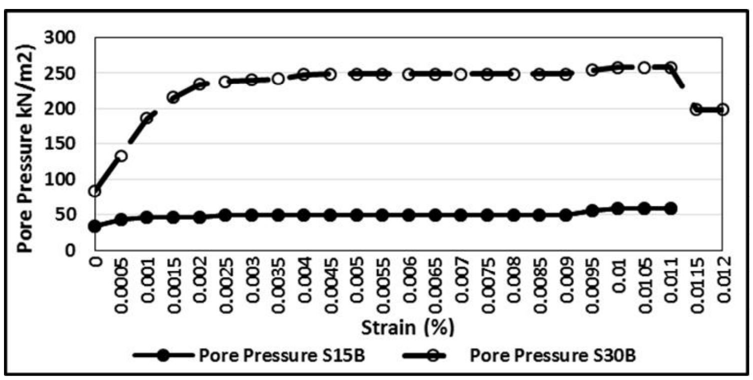

Fig. 5. Pore pressure versus axial strain for sample S15B and S30B

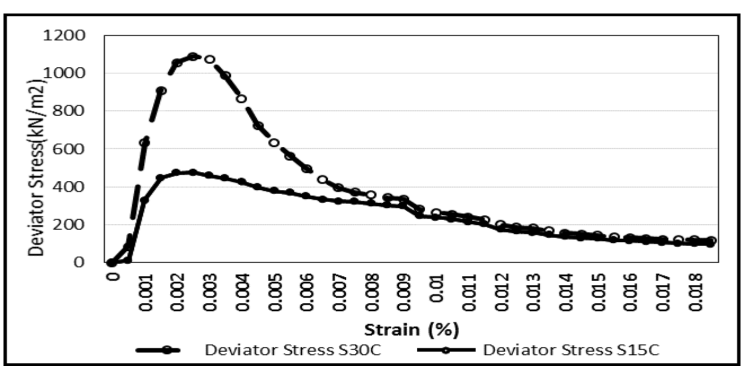

Fig. 6. Deviator stress versus axial strain for sample S15C and S30C

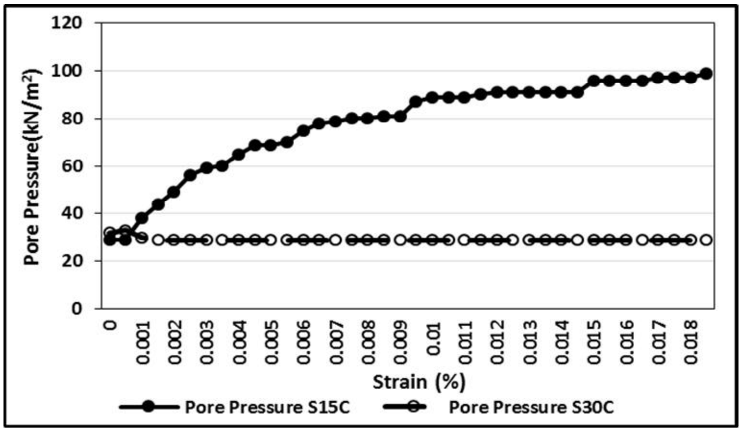

Fig. 7. Pore pressure versus Axial Strain for sample S15C and S30C

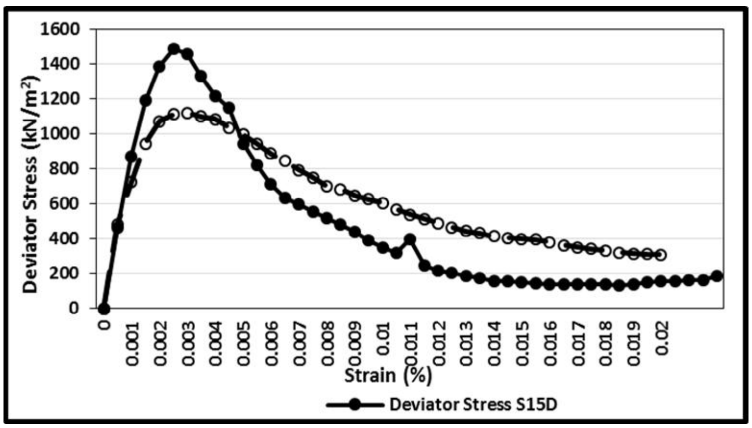

Fig. 8. Deviator Stress –Axial Strain Plot for sample S15D and S30D

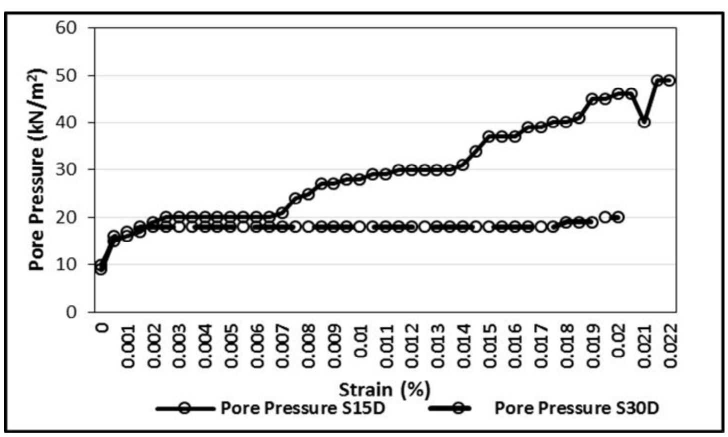

Fig. 9. Pore pressure versus Axial Strain for sample S15D and S30D

4.4 Critical State Parameters

The results of the critical state parameters are shown in Table 4. Quite clearly, the gamma (Γ) values for the two samples are higher for samples taken at 3.0m depth than the sample at 1.5m depth. The gamma value (Γ) of the S15 soil samples have an average value of 2.07 while for

S30 soil samples the average value is 4.25 This may be an indication that the (Γ) value increases with depth. The equivalent of (Γ) in normal soil strength determination is the C-value. Both the (Γ) value and C-value are indicative of soil cohesion. However, in the case of the (Γ) value, the cohesion is counteracted by the $\ln p'$ value, so that the soil loses both densification and internal friction to set it towards fluidity and hence large strains and shears. However, λ-values are constant within each soil profile that is S15 soil samples and S30 soil samples but varies again with depth. Samples S15 have an average value of 0.174 while S30 samples average 0.545. The percentage difference is 68.1 %. The M-values of the soils also shown in Table 3 varied with the depth. At 1.5m depth, the average M-value is 0.885 while at 3.0m depth the average value is 0.8125. It may be justifiable to say that increase in depth increases the critical state parameters because of the extra overburden load that stresses the soil. The critical state line equation is given as in equations 1 and 2. Equation 2 is a linear relationship whereas equation 3 is non-linear. In the linear equation, V_{cs} is the specific volume of the sample, Γ is the vertical intercept and $\boldsymbol{\Lambda_{cs}}$ is the slope of the critical state line.

$$V_{cs} = 1 + wG_s \tag{1}$$

$$V_{cs} = \Gamma - \lambda_{cs} \ln \rho'_{cs} \tag{2}$$

$$q_f = M\rho' \tag{3}$$

Equation 3 is the non-linear expression of the critical state line. The parameters of M, Λ and Γ are regarded as constants for a particular soil. Equations 2 and 3 will be used to determine the critical state of the soil for depth S15 and S30.

Table 4. Critical state results for soil samples

Critical state Parameters	Γ	λ	M
S15A	2.03	0.174	0.90
S30A	4.24	0.545	0.88
Average	**3.14**	**0.36**	**0.94**
S15B	2.08	0.174	0.88
S30B	4.17	0.545	0.83
Average	**3.13**	**0.36**	**0.86**
S15C	2.01	0.174	0.96
S30C	4.14	0.545	0.78
Average	**3.08**	**0.36**	**0.87**
S15D	2.14	0.174	0.80
S30D	4.44	0.545	0.76
Average	**3.29**	**0.36**	**0.82**

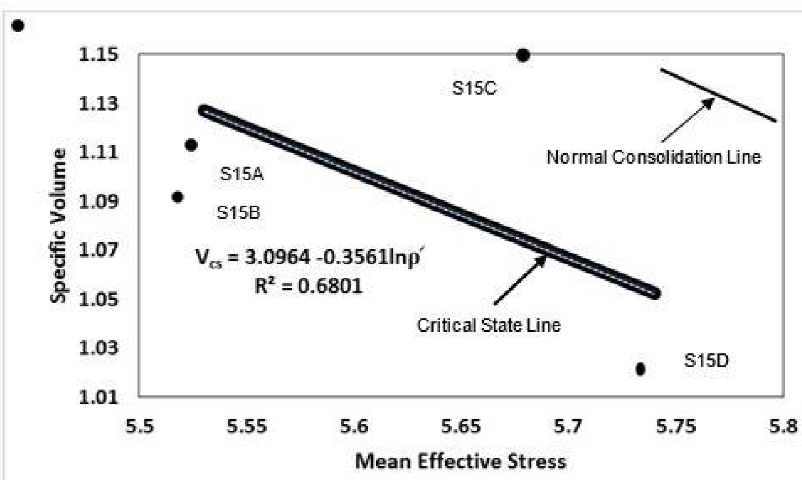

Fig. 10. Critical state line for S15 samples

The data obtained from the triaxial test results were used to generate Fig. 10. Comparing the equation v_{cs} = 3.0964 - 0174lnp’ in the chart to equation 1.2 indicates that Γ = 3.0964 for samples at 1.5m depth and λ= 0.174. The critical state soil parameters for a sample at 1.5m depth are summarised in Table 5. Since the samples are from the same profile, they have the same **λ** value but have different specific volumes. From equation 1.1, it is easy to deduce the water content of the soil at a critical state. Thus, rearranging equation 1.1 into equation 1.4 the moisture contents were determined.

$$w_{cs} = \frac{v-1}{G_s} \tag{4}$$

The results are shown in Table 6.

Table 6 is very revealing. It shows that in all cases the natural moisture content of the sample S15 is higher than the critical state water content. This is very important in determining the state of consolidation of the samples. When the natural moisture content is higher than the moisture content at the critical state the soil is on the dry side of critical state (i.e. heavily overconsolidated clay or dense sand). Hence it can be inferred that the S15 samples are heavily overconsolidated soils.

From the triaxial test results for S30 samples, a plot was made of specific volume and mean effective stress (lnp’) and it is shown in Fig. 11.

A similar comparison was made between equation 1.1 and the equation in Fig. 11. It can, therefore, be deduced that Γ is equal to 1.656 and $\boldsymbol{\lambda}_{cs}$ is 0.299. The sign of the slope is again indicative of the direction of the linear curve. A summary of the critical state parameters for samples at 3.0 m depth is given in Table 7.

To have an idea of the critical state water content, the study again employ the use of equation 1.1 and re-arrange the parameter as in equation 4 and shown in Table 8. When the natural moisture content of the sample is higher than the critical state moisture content, the sample is in the wet side of the critical. When the natural moisture content is less the critical state moisture content, then the soil is in the dry side of critical.

Table 5. For critical state line parameter for sample S15

Sample	V	Γ	λ	Inp´ kN/m^2	M	q_f kN/m^2	P´$_f$ kN/m^2
S15A	1.032	2.03		5.74	0.80	251.03	312.02
S15B	1.121	2.08	0.174	5.53	0.83	208.50	251.50
S15C	1.103	2.01		5.52	0.96	239.50	250.50
S15D	1.153	2.14		5.67	0.76	219.94	289.95

Table 6. Critical state water content and parameters (S15)

Sample	Γ	λ_{cs}	In p’ kN/m^2	w_{cs}	N.M.C %
S15A	2.03		5.74	0.01	1.20
S15B	2.08	0.174	5.53	0.06	4.53
S15C	2.01		5.52	0.04	3.85
S15D	2.14		5.67	0.06	5.74

Table 7. For critical state parameter for sample S30

	v	Γ	λ_{cs}	In p´ kN/m^2	M	qf kN/m^2	P´f kN/m^2
S30A	1.168	1.681		5.57	0.99	281.76	282.76
S30B	1.140	1.707	0.299	5.67	0.88	226.29	257.29
S30C	1.128	1.654		5.80	0.78	256.13	327.13
S30D	1.050	1.583		5.94	0.80	333.05	414.04

Table 8. Critical state water content and parameters for sample S30

	Γ	λ_{cs}	In p` kN/m^2	w_{cs}	N.M.C %
S30A	1.681		5.57	0.06	6.20
S30B	1.707	0.299	5.67	0.05	5.49
S30C	1.654		5.80	0.05	4.75
S30D	1.583		5.94	0.06	5.82

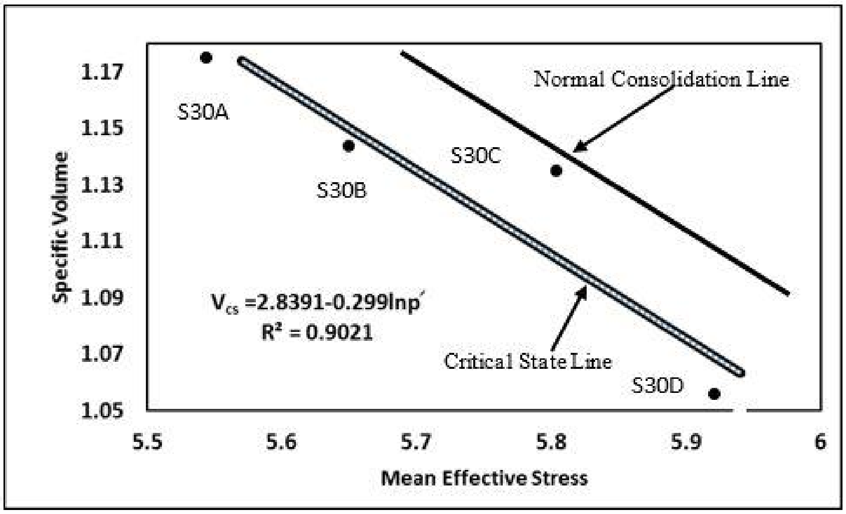

Fig. 11. Critical state line for S30 samples

4.5 Consolidation State of Sample

The state of consolidation of the sample can be established by plotting the coordinates of the samples on the specific volume – effective stress plot. The points should lie above the critical state line to indicate that the samples are lightly overconsolidated. If on the hand the points lie below the critical state line, the samples are heavily overconsolidated. In the case of samples S15, three samples lie below the critical state line shown in Fig. 10. Sample S15D lies above the critical state line and could be said to be lightly overconsolidated. Similarly, sample S30D also lies above the critical state line for samples taken at 3.0m depth, the remaining lying below the critical state line as shown in Fig. 11. The critical state parameters in this study are for S15 samples: Γ = 3.0964, λ = 0.174 and = 0.838; and for S30 samples are: Γ = 2.8391, λ = 0.299 and M =0.863. When compared to Marto et al. [8] the critical state parameters of this study fall within the ranges established by him [8] for Sand Matrix soils given as M = (0.803-0.998), λ = (0.144-0.248) and Γ = (1.727-2.279) respectively. Similar work by Jefferies et al. [7] also gives comparable values to this study thus giving credence to the study values.

5. CONCLUSIONS

The purpose of this study was to determine the effects of varied depth on the consolidation state of burrow pit samples. Two depths at 1.5m and at 3.0m were tested and the following are the conclusions drawn.

1. Classification tests conducted on the burrow pit indicated that soils samples taken at 1.5m depth are Well- Graded (GW) by the Standard of Unified Soil Classification System.
2. The state of the soil samples taken at 1.5 m depth is (30.55 kN/m^2, 0.12), representing the stress and Void ratio respectively. The state of the soil samples at 3.0m depth is (60.47 kN/m^2, 0.18).
3. The deviator stress at failure for soils taken at 1.5m depth average 233.9 kN/m^2. While the failure for soils at 3.0m depth is 270.15 kN/m^2 .
4. The pore pressure increased with strain for soils taken at 1.5 m depth and were

observed to be stabled at a range of (0.006%-0.045%) while for soils at 3.0m depth it were at a range of (0.005%-0.004%).

5. The state of consolidation of the samples at 1.5 m depth is on the dry side of the critical state line. Indicating that all the samples taken at 1.5 m are heavily overconsolidated. Heavily overconsolidated soil cannot attain further compression or densification rather it dilates.
6. The state of consolidation of the samples at depth of 3.0m is also on the dry side of critical indicating the soils are heavily overconsolidated.
7. The Critical state Parameters for soils at 1.5 m depth are: Γ= 2.07, λ = 0.174 & M =0.885 and the critical state parameters for soils at 3.0m depth are: Γ= 4.25, λ = 0.545 & M =0.813.
8. Depth of sampling has important effect on the behaviour of samples used in this research work. The samples tested in this study are heavily overconsolidated.
9. Overconsolidated soils do not offer any benefits in construction as they tend to dilate in service. They present maintenance challenges when roads are built with them.
10. For burrow pit management, it is important to carry out periodic checks on the state of consolidation of soils in order to avoid the use of overconsolidation soils in construction.
11. To ensure that soils from burrow pits performed satisfactorily an oversight function should be exercise by the appropriate ministry.
12. Since the specific volume (v_s) and the water content of a soil sample are relevant at both normal and critical states, one set of parameters could be used to predict the other.

COMPETING INTERESTS

Authors have declared that no competing interests exist.

REFERENCES

1. Schofield AN, Wroth CP. Critical state soil mechanics. Geotechnique, McGrawHill, London, UK; 1968.
2. Roscoe KH, Schofield AN, Wroth CP. On the yielding of soils. Geotechnique. 1958;35(2):22-53.
3. Burland JB. On the compressibility and shear strength of natural clays. Geotechnique. 1990;40(3):329-78.
4. Yimsiri S, Soga K. DEM analysis of soil fabric effects on behaviour of sand. Geotechnique. 2010;60 (6):483-95.
5. Zhao CG, Li J, Cai Y. Critical state for unsaturated soils and steady state of thermodynamic process. 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris; 2013.
6. Wang Q, Pufahi D, Fredlund DG. A study of critical state on an unsaturated silty soil. Canadian Geotechnical Journal. 2002; 15(9):213-218.
7. Jefferies M, Been K, Hachery J. The critical state of sand. International Journal of Rock Mechanics and Mining Sciences and Geomechanics Abstracts. 1995;22(6): 234-246.
8. Marto A, Choy S, Ahmad M, Twing L. Critical state of sand matrix soil. The Scientific World Journal. 2014;2:1-6.
9. Been K, Jefferies M. Stress- Dilatancy in very loose sandy. Canadian Geotechnical Journal. 2004;41(5):972-989.
10. Kayadelen C, Sirikaya T, Taskiran, Guneyli H. Critical state parameters of an unsaturated Residual Clayey Soil from Turkey. Engineering Geology. 2007;94(1-2) 1-9.
11. Wasseem M, Prashant A, Chandra S. (2006), "Predication for Stress –Strain Behaviour of Panki Fly- Ash Using Modified Cam Clay Model". Indian Institute of Technology Kampur, India-208016
12. Hamidi A, Touchi S, Kardooni F. A critical state based thermo-elasto-plastic constitutive model for saturated clays. Journal of Rock Mechanics and Geotechnical Engineering. Accepted manuscript; 2017.
    DOI: 10.1016/j.jrmge.2017.09.002.
13. Lopera Perez JC, Kwok CY, Senetakis K. Micromechanical analysis of the effect of rubber-size and content on sand-rubber mixtures at critical state. Geotextiles and Geomembranes. 2017;45(2):81-97.
14. Vu TP, Hsiao D, Nguyen PT. Critical state line and state parameter of sand–fines mixtures. Sustainable Development of Civil, Urban and Transportation Engineering Conference. Procedia Engineering. 2016;142:299-306.
15. Hsiao DH, Phan TAV. Effects of silt contents on the static and dynamic

properties of sand-silt mixtures. Geomechnic and Engineering; 2014.
16. Mashiri MS, Vinod JS, Neaz Sheikh M, Tsang H. Shear strength and dilatancy bahaviour of sand-tyre chip mixtures. Soils and Foundations. 2015;55:517-528.
17. Yan WM, Lin Zhang. Fabric and the critical state of idealised granular assemblages subject to biaxial shear. Computers and Geotechnics. 2013;49:43-52.
18. Sitharam TG, Dinesh SV, Srinivasa Murphy BR. Critical state behaviour of granular materials using three dimensional discrete element modelling. Granular Materials: Fundamentals and Applications. London: RSC. 2004;135-56.
19. BS 1337. Soils for Engineering Purposes Part 8 Shear Strength Tests (Effective Stress) British Standard Institution; 1990.

Peer-review history:
The peer review history for this paper can be accessed here:
http://www.sciencedomain.org/review-history/26829