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An Evaluation of the Effects of In-situ and Compaction Induced Structures on the Compression and Strength Characteristics of a Lateritic soil in the Laboratory

Naa Korkoi Ayeh
PhD student, Department of Civil Engineering, Kwame Nkrumah University of Science and Technology, Kumasi, Ghana, nsolomonayeh@gmail.com

Samuel I. K. Ampadu, Frederick Owusu-Nimo, Felix Jojo Fianko Ayeh, Frederick Nii Nai Charkley
Department of Civil Engineering, Kwame Nkrumah University of Science and Technology, Kumasi, Ghana.

ABSTRACT: Laterite and lateritic soils are products of weathering under tropical conditions that is rich in the oxides of iron and aluminium. In-situ, the presence of these oxides leads to the creation of a structure which control most of the engineering properties. When compacted, however, a new compaction-induced structure is formed. In tropical environments, road and railway construction usually involve deep cuts in lateritic soils that is reworked and compacted into embankments. The analysis and modelling of stability and performance of these cuts and embankments require a good understanding of the mechanical behaviour of intact and of compacted lateritic soils. A laboratory experimental study was conducted to determine the differences in the effects of structure on behaviour of intact and of compacted samples. Samples of an intact lateritic soil were obtained and recompacted samples were prepared at the same insitu dry density and water content of 1.654Mg/m^3 and 16.5% respectively. The intact and re-compacted samples were subjected to one dimensional consolidation in the oedometer test and also to the consolidated undrained triaxial tests at stress levels of stress ranging from 20kPa to 400kPa. The consolidation and shear strength behaviour are discussed in terms of the differences between the intact and compacted samples. The material parameters for modelling the behaviour are obtained and discussed.

KEYWORDS: Lateritic soil, intact structure, compacted structure, shear strength, compressibility.

1 INTRODUCTION

Laterites and lateritic soils are formed in tropical climates and are rich in the oxides of iron and aluminium collectively referred to as sesquioxides. These sesquioxides are gels that provide cementation of fine particles into larger aggregates when dehydrated. During road construction in tropical regions, deep cuts are made in laterites and lateritic soils and the resulting cut material is used to build embankments. In the natural state, the bonds provided by the sesquioxides are preserved and the structure is considered intact. In contrast, the mechanical manipulation of the cut material through excavation, hauling, spreading and re-compaction during embankment construction destroys the existing bonds and builds new ones. The influence of soil structure on the mechanical behaviour of lateritic soils has been the focus of many studies. Futai et al. (2004) discussed the results of compression and shear strength tests on intact and compacted specimens from different depths. Fagundes and Rodrigues (2015) reported for a clayey sand lateritic soil that compacted soil showed a significant increase in shear strength, particularly under unsaturated condition. Ampadu et al. (2019) report that apart from soil structure, normal stress and saturation influence the mechanical behaviour of a lateritic soil. Akinniye (2019) compared the microstructure of a lateritic soil with that of a completely decomposed volcanic and granitic soils. The results showed that lateritic soil had a higher clay content but stiffer structure and attributed it to its high sesquioxide content. Okewale (2020) in an experimental study found that the effect of structure was minimal for a lateritic clay soil.

This study compares the compressibility and shear strength properties of a lateritic soil tested in an intact condition and in a compacted state. Samples of an intact and compacted lateritic soil were subjected to one dimensional consolidation and consolidated undrained triaxial compression tests. The results are analysed to compare the one-dimensional compressibility parameters and the undrained shear strength parameters of intact and compacted samples.

2 METHODOLOGY

2.1 *Soil sampling*

The study site was on a gentle slope on the KNUST campus. A 1.5m x 1.5m trial pit was manually excavated first to a depth of 1.5m. Then half of the pit was taken locally down to a final depth of 2.5m to create a soil bench for undisturbed sampling. A number of U-100 steel tubes were placed on the prepared bench, a thick piece of wooden plank was placed on top of the tubes and gently hammered into the soil bench to retrieve undisturbed samples for triaxial testing. Similarly, a number of 75mm diameter tubes were hammered into the bench to retrieve samples for insitu density test. Finally, block samples were recovered using a pickaxe and mattock on the remaining piece of the soil bench. The recovered samples were wrapped with layers of clingfilm and placed in an insulated container to prevent water content change. The remainder of the bench was excavated down to the final depth of 2.5m and bulk disturbed samples were taken in 50kg sacks. The samples were sent to the laboratory for further testing.

2.2 *Sample Preparation*

In the laboratory, the bulk disturbed samples were emptied into large flat trays and air-dried for at least 7 days to a consistent water content. When the soil was thoroughly dried, it was re-bagged for further testing. The remaining air-dried sample was sieved through the 20mm BS sieve and the soil passing was re-bagged and labelled as “compacted sample” for laboratory compaction tests.

2.3 *Sample Characterization*

2.3.1 *Index properties tests*

A sample of the air-dried soil was retrieved for the index properties tests. The Atterberg limits were determined on the portion passing the 0.425mm sieve in accordance with British Standards procedure, BS1377, Part 2: 1990 using the cone penetrometer method for the liquid limit test. The particle size distribution was evaluated using the wet sieving and hydrometer method with sodium hexametaphosphate (IV) as deflocculating agent.

2.3.2 *Laboratory Compaction Test*

The laboratory compaction characteristics were determined for both the Standard Proctor and Modified Proctor compaction according to ASTM D698 and ASTM D1557 respectively using fresh samples for each trial.

2.3.3 *Chemical and Mineralogical composition tests*

The mineralogical composition of the soil was determined by X-ray diffraction (XRD) method. The equipment used was the D2 PHASER 2nd Generation XRD. The mineralogical analysis was carried out at a temperature of 25^{o}C and test samples were scanned from 5^{o} to 90^{o} at a rate of 0.010s to 64.0s. The X-ray Fluorescence test (XRF) was carried out to determine the chemical composition of the soil according to the procedure proposed by Schramm (2016). The equipment used was the Olympus Vanta^{TM} handheld XRF analyser which uses the energy dispersive method that gives the chemistry of the sample by measuring the fluorescent x-ray emitted when excited by a primary x-ray source.

2.4 *Determination of insitu conditions*

The clingfilm covering on the undisturbed samples from the 75mm tubes were removed and the samples were trimmed. The insitu water content was determined from the trimmings. The tubes were weighed and the volumes computed to determine the in-situ dry density.

2.5 *One dimensional consolidation test*

2.5.1 *Sample Preparation*

The clingfilm was removed from the block sample and a well-oiled one-dimensional consolidation cutting ring was pushed gently and smoothly into the block to cut the intact sample. The top and bottom surfaces were levelled off to produce the intact samples designated "P-1D-I for the one-dimensional consolidation test. For the preparation of the compacted samples, the mass of soil required to fill the one-dimensional consolidation ring to give the in-situ dry density was computed. Then samples of air-dried disturbed soil were brought to the in-situ water content, and compacted into the cutting ring with a mallet. The specimen prepared in this manner was designated as "P-1D-C".

2.5.2 *Testing Procedure*

Consolidation tests were carried according to the procedures outlined in ASTM D2435-03 in a standard fixed ring type oedometer with a cutting ring of dimensions 50mm diameter and 20mm height. After set-up in the consolidation apparatus, the specimen was inundated with water for 24 hours under a small seating stress equal to 1.16kPa, provided by the loading cap and the swell readings were taken during the 24-hour period. After 24 hours, the loading was commenced under vertical stresses increasing from 6kPa to 800kPa. Each stress was sustained for 24 hours. Upon completion of loading, the unloading was also done following the procedures. After the final unloading was complete, the whole set-up was dismantled and the final weight and water content of the specimen was determined. This was done for both the intact and compacted samples.

2.6 *Consolidated Undrained Triaxial Compression Test*

2.6.1 *Specimen preparation*

To obtain intact specimens for triaxial shear strength tests, three sharpened sample tubes were slowly and steadily pushed vertically into the U-100 tubes by hydraulic action to cut samples for the P-TX-I test series.

For the compacted specimens, 3000g of the air-dried soil were sieved through the 19mm sieve and the mass of water required to bring the soil sample to the in-situ water content was added evenly over the soil in the pan. The soil and water were mixed thoroughly by hand and bagged for 24 hours to achieve moisture equilibrium. Based on the compaction curves for the standard and modified Proctor tests, a relationship was established to determine the number of blows required to give the in-situ dry density in the standard Proctor mould. Then, the wet soil was compacted into the Standard Proctor using the required number of blows to give the insitu dry density. Sample tubes were then pressed into the compacted sample to obtain samples for P-TX-C series.

2.6.2 *The Test Procedure*

The automated GDS triaxial equipment was used for the test. The specimens were set-up in the triaxial apparatus and saturated by applying a back pressure of between 300kPa and 400 kPa to achieve a minimum B-value of 0.95. After saturation, consolidation of each specimen was done under selected isotropic stresses in the range of 20kPa to 400kPa. After each consolidation, the specimen was sheared undrained to failure with pore pressure measurements at a rate of 0.03mm/min.

3 RESULTS AND DISCUSSION

3.1 *Properties of lateritic sample*

The sample was a medium dense, reddish brown clayey sand containing mica. The properties of the lateritic sample are summarized in Table 1. The lateritic sample had a specific gravity (G_s) of 2.7. The particle size distribution curve for the sample is shown in Figure 1. It had a gravel and sand content of 16.2% and 38.5% respectively and a fines content (percent passing 0.075mm) of 45.3%. According to the Unified Soil Classification System (USCS), it classifies as inorganic silt of high plasticity (MH).

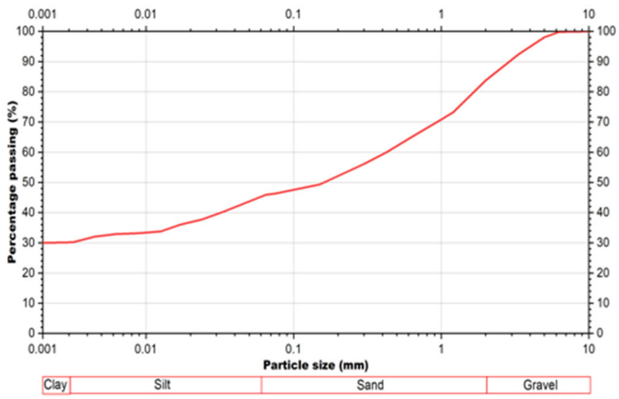

Figure 1. Particle size distribution curve for lateritic sample.

Table 1. Summary of index properties for lateritic sample.

Gravel (%)	Sand (%)	Silt (%)	Clay (%)	LL (%)	PI (%)	Gs
16.2	38.5	15.3	30.0	54.0	21.1	2.7

The results of the mineralogical and chemical composition analysis of the soil are presented in Table 2. Only the major oxides are presented for simplicity. The chemical analysis showed that the soil is composed of silica (SiO_2) as the most abundant oxide with oxides of iron (Fe_2O_3) and aluminium (Al_2O_3) in appreciable amounts, which act as cementing agents. The soil has a silica-to-sesquioxide ratio of 1.25 which makes it a true laterite based on the indices proposed by Morin and Tudor (1975). Mineralogical analysis showed that the soil was dominated by quartz which makes up 50% of the mineralogy of the soil and consistent with the percentage of SiO_2 observed in the chemical composition. The clay mineral found in the soil was identified as kaolinite. With an activity (defined as the PI to clay fraction) of 0.70, the soil may be described as of low activity which is consistent with the high kaolinite clay mineral content.

Table 2. Chemical and mineralogical composition of soil sample.

Chemical composition		Mineralogical composition	
Major oxide	%	Major oxide	%
Aluminium oxide (Al_2O_3)	26.9	Quartz (SiO_2)	50.5
Silica oxide (SiO_2)	48.6	Kaolinite	44.2
Iron oxide (Fe_2O_3)	11.9	Mica	5.3

3.2 *Initial specimen condition*

The in-situ sample uniformity, as well as the uniformity of the intact samples used for the 1-D consolidation (the block samples) and triaxial samples (U-100 samples) are assessed and compared with those of the compacted samples (both static compacted for 1-D consolidation and impact compacted for triaxial testing). The parameters considered are the dry density, ρ_d and the water content, w_c. Table 3 summarizes the average values of these parameters for each sample and test type. The variability in dry density and in the water content was evaluated by the coefficient of variation (COV) defined as the standard deviation divided by the mean value in percentage terms. As a rule of thumb, a COV below 10% is considered "low" variability (Harr, 1987). From Table 3, the highest COV value across sample types for both the dry density and the water content was of the order of 0.9%, indicating that the variation across samples was low. The average in-situ water content and dry density of 16.5% and 1.654Mg/m^3 respectively were used as target values for the preparation of compacted specimens. The mean values of the different sample types are plotted in Figure 2. Again, the very close aggregation of the points indicates the very good uniformity across sample types and compacted sample preparation procedures.

Table 3. Summary of sample uniformity.

Sample and Test Type	Intact		Compacted	
	ρ_d (Mg/m^3)	w_c (%)	ρ_d (Mg/m^3)	w_c (%)
Tube for In-situ condition	1.654	16.5	NA	NA
Block for 1-D Consolidation	1.651	16.2	1.652	16.2
U-100 for Triaxial	1.653	16.5	1.653	16.5
Mean	1.652	16.4	1.653	16.35
COV (%)	0.061	0.862	0.030	0.917

NA: Not Applicable

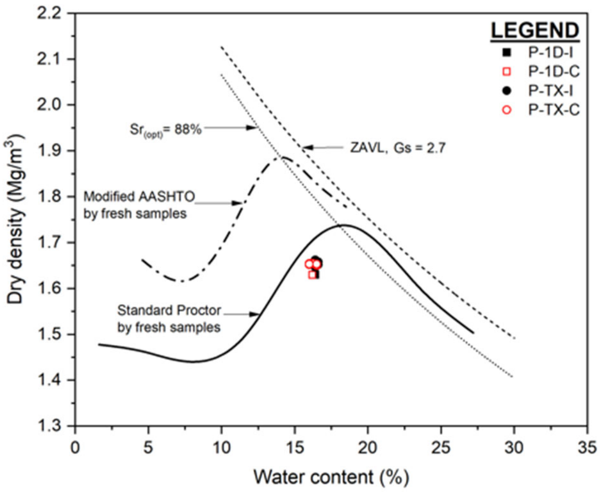

Figure 2. Compaction curves showing initial specimen condition.

3.3 *1D consolidation results*

The results of the 1D consolidation test on intact and compacted samples are shown in Figure 3 as the relationship between vertical stress (σ_v') and void ratio (e). The key indicators are tabulated in Table 4.

Table 4. Summary of 1-D consolidation test results.

Sample	e_o	e_s	Swell (%)	σ_{vp}' (kPa)	Cc	Cr	Cr /Cc
Intact	0.745	0.746	0.07	116	0.173	0.006	0.04
Compacted	0.745	0.747	0.14	91	0.131	0.014	0.11

e_0 = void ratio after sample preparation, e_s = void ratio after swell, C_c = compression index, C_r=recompression index, σ_{vp}'= preconsolidation stress

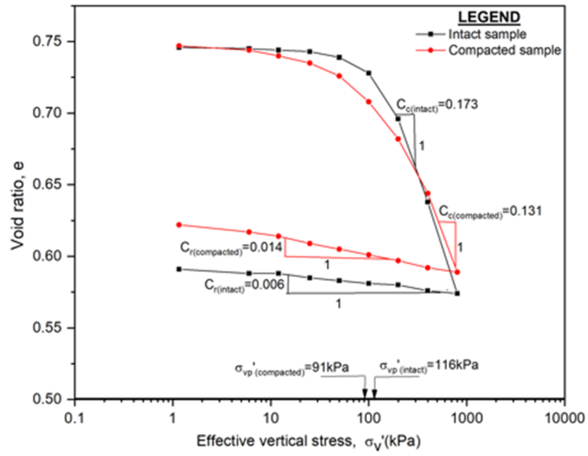

Figure 3. Relationship between consolidation stress and void ratio for intact and compacted sample.

Table 4 shows insignificant swell values associated with kaolinite clay mineral. Studies by Zhang et al. (2016) has also shown that free iron oxide occurring in intact lateritic soils acts as a bridge enhancing cementation and consequently reduces swelling-shrinkage capacity of the soil. The yield behaviour expressed as the pre-consolidation stress

(σ_{vp}') and defined as the onset of structural collapse (Leroueil and Vaughan, 1990) was estimated using the Casagrande (1936) graphical method. Intact sample has a slightly higher yield stress (σ_{vp}') due to the bonded structure which increases its initial stiffness. The slope of the post-yield normal compression line (C_c) is about 32% higher for intact sample compared to compacted sample. In other words, the compacted sample has a stiffer structure post-yield. This behaviour is attributed to progressive disruption of the structure as the samples undergo compressive strains. This is similar to another lateritic clay (Akinniye, 2019) which showed diverging behaviour between intact and compacted samples post-yield even at 800kPa stress. Upon unloading, the slope of the recompression curve (C_r) for compacted sample is 2.3 times greater than intact sample indicating stiffer response in unloading.

The loss of structure, which is commonly associated with sample disturbance was assessed using the sample quality framework (C_r/C_c) by DeJong et al. (2018) developed for low plasticity intermediate soils. In this framework, a ratio less than 0.15 is considered "high" quality while higher values indicate increasing level of "disturbance". Though the C_r/C_c ratio for intact and compacted sample indicate high quality rating, the compacted sample showed less structure and by implication relatively higher "disturbance".

3.4 *Triaxial test results*

3.4.1 *Stress-Strain Characteristics*

The stress-strain curves for intact specimens in CU-TX are shown in Figure 4, whiles the equivalent plot for compacted specimens are shown in Figure 5.

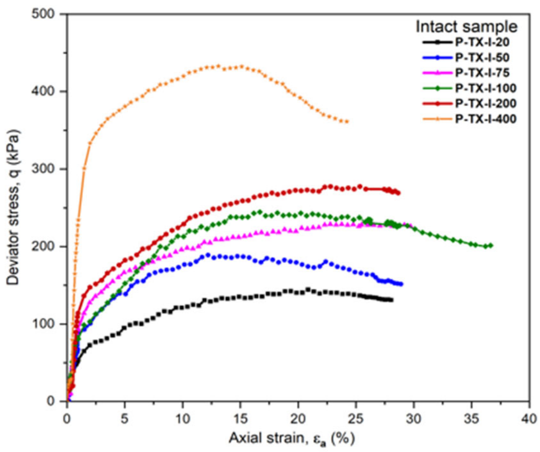

Figure 4. Deviator stress-strain characteristics for intact samples.

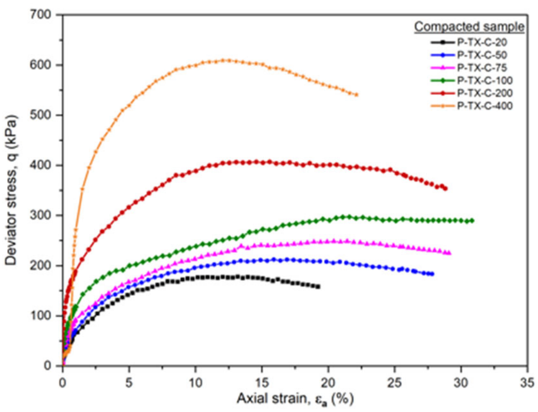

Figure 5. Deviator stress-strain characteristics for compacted sample.

Intact and compacted specimens exhibit overall similar stress-strain behaviour. The specimens show a strain softening behaviour characterised by increasing deviator stress, q, at early strains to a peak value, q_{max}, followed by a decrease with increasing strain till end of shearing. At high confining stress (σ_3'= 400kPa), a prominent peak in q is observed associated with development of shear surfaces (Rahman, 2008). All specimens showed an initial bulge in the early stages of shearing followed by the development of a single distinct shear plane at larger strains. The key strength and compressibility indicators are summarized in Table 7 and Table 8 for intact and compacted specimens respectively.

The stiffness (E_{50}) defined as the secant modulus at 50% of q_{max} is also shown in the Tables for intact and compacted specimens. To examine the influence of structure on the stiffness, the E_{50} values were normalized by the q_{max} and plotted against the mean effective stress (p_o') in Figure 6. The plot shows that for low values of p_0', intact samples had higher stiffnesses than compacted samples. However, as p_0' increases from 50-100kPa intact samples undergo a rapid loss of stiffness as a result of the disruption of the bonds, after which the stiffness increases at a higher rate than that for compacted samples. This means that after disruption of bonds, the rate of increase in stiffness due to consolidation pressure is higher in intact samples than in compacted samples.

Table 5. Failure conditions for intact sample.

Test ID	p_0' (kPa)	q_{max} (kPa)	$\varepsilon_{a(qmax)}$ (%)	E_{50} (kPa)
P-TX-I-20	18.0	144.3	20.9	3592
P-TX-I-50	43.5	178.3	11.8	7313
P-TX-I-75	62.9	228.9	22.8	7548
P-TX-I-100	95.8	244.5	17.9	3793
P-TX-I-200	197.0	277.5	22.8	8466
P-TX-I-400	391.0	432.7	13.1	24840

$\varepsilon_{a(qmax)}$ = Axial strain at maximum deviator stress

Table 6. Failure conditions for compacted sample.

Test ID	p_0' (kPa)	q_{max} (kPa)	$\varepsilon_{a(qmax)}$ (%)	E_{50} (kPa)
P-TX-C-20	18.7	177.9	17.4	4516
P-TX-C-50	48.8	212.2	15.9	5108
P-TX-C-75	73.8	248.7	21.4	5002
P-TX-C-100	87.2	244.5	21.6	8556
P-TX-C-200	195.9	407.0	15.6	15940
P-TX-C-400	391.9	609.1	12.1	25129

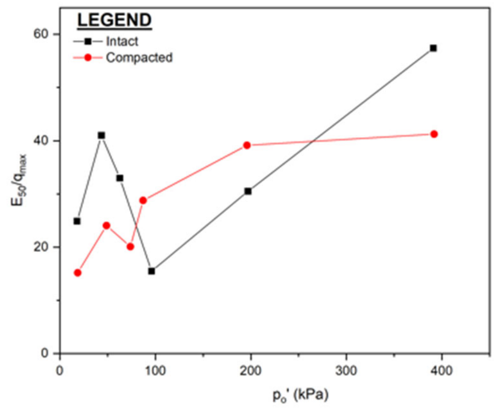

Figure 6. Relationship between stiffness and effective mean stress.

3.4.2 *Effective stress path*

The stress paths in terms of deviator stresses against mean effective pressure are plotted in Figure 7 and Figure 8 respectively for intact and for compacted samples. For p_0' less than 100kPa, the stress paths resemble heavily overconsolidated soil characterised an increase in mean effective stress (dilation) followed by a reduction as critical state is approached. This behaviour, characteristic of lateritic soils has been referred to as pseudo-overconsolidation and it is due to bonding rather than stress relieve (Barksdale and Blight, 2012). For p_0' increasing from 100kPa to 400kPa, the stress paths initially bend to the left (contraction) and then to the right (dilation), particularly after reaching the peak q/p' ratio. Asaoka et al (2000) have associated this behaviour to the effects of the initial structure and the decay of this structure as shearing progresses to generate an overconsolidated state.

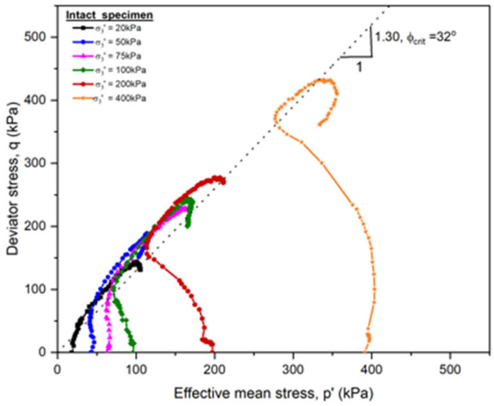

Figure 7. Stress paths for intact sample.

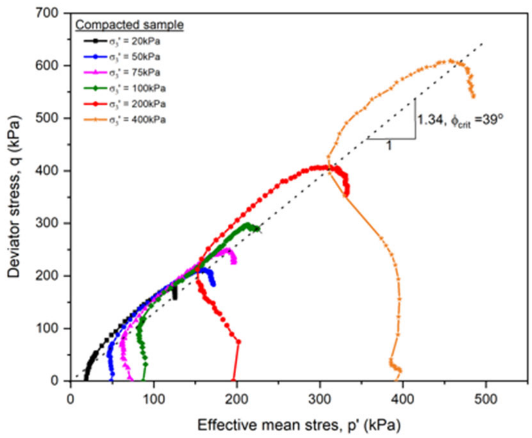

Figure 8. Stress paths for compacted sample.

3.4.3 *Failure Condition*

The maximum deviator stress values (q_{max}) tabulated in Table 5 and Table 6 for intact and compacted sample respectively are plotted against the consolidation mean effective pressure (p_0') in Figure 9.

For similar confining pressure, compacted specimens had a higher q_{max} values than intact specimens over the whole range of p_0' studied. Also, the behaviours at low consolidation pressures (p_0'< 100kPa), are obviously different from that at higher consolidation pressures.

Using the Mohr-Coulomb failure criteria, the effective cohesion intercept (c') and the effective angle of internal fiction (ϕ') were computed assuming a linear envelope. The cohesion value was 22kPa for intact sample while that of compacted sample was 5°. Thus, in triaxial compression, cohesion is more than four times higher than compacted sample and may be attributed to the bonded structure of intact sample

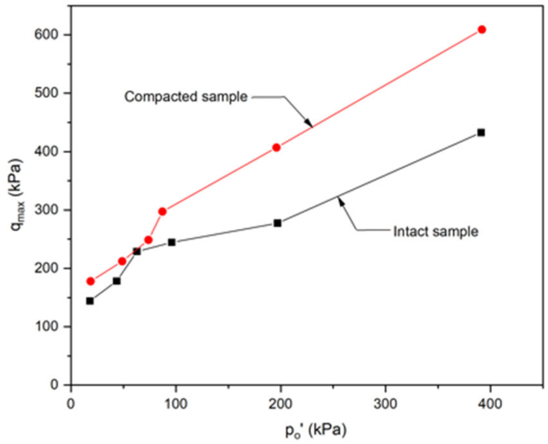

Figure 9. Maximum deviator stress vs. mean effective stress for samples.

At the end of shearing, most of the specimen had still not attained the critical state as the stress ratios (q/p') were changing even at large strains in excess of 25%. However, the results of P-TX-I-50, P-TX-I-75, P-TX-I-100 and P-TX-I-200 for intact sample and P-TX-C-50 and P-TX-C-100 for compacted sample, which were approaching the critical state, were used to estimate the critical state stress ratio. Using these points, if the critical state envelope is modelled as linear, instead of curved as has been reported for lateritic soils by Futai et al. (2004) and Akinniye and Bagheri, (2023), then the linear critical state line may be superimposed in Figures 7 and 8 giving a critical state stress ratio of 1.30 and 1.34 for intact sample and for compacted sample respectively. Table 9 summarizes the shear strength and critical state parameters. Typical unique critical state stress ratios reported for some lateritic soils are 1.73 (Akinniye and Bagheri, 2023), 1.30 (Ampadu et al. 2019) and 1.03 to 1.20 (Futai et al. 2004).

Table 7. Shear strength and critical state parameters.

Parameter	Intact	Compacted
c' (kPa)	21.6	4.7
ϕ' (°)	28.9	32.3
ϕ'crit (°)	32	39
M	1.30	1.34
Γ	2.00	1.92
λ	0.08	0.06

M and λ= slope of projection of CSL in q-p' space and v-In p' space respectively, Γ = intercept of CSL at p'=1kPa in v-In p' space

4 CONCLUSIONS

One dimensional consolidation and consolidated undrained triaxial compression tests were performed on intact and on compacted samples of a true laterite that had a liquid limit of 54 and a PI of 21 at an average insitu dry density of 1.654M/m^3 and water content of 16.5%. The following are the conclusions from the study:

1. Under 1-D consolidation tests:

- the intact samples had a higher pre-consolidation stress of about 120kPa compared with the compacted sample value of about 90kPa. Also, at a coefficient of consolidation of

0.173, intact samples were about 32% more compressible post-yield than the compacted sample.

- Upon unloading, the recompression index for compacted sample of 0.014 was 2.3 times greater than that for intact sample indicating stiffer response in unloading.

2. Under triaxial compression, the key observations are:

- At the same consolidation pressure, compacted specimens had a higher q_{max} values than intact specimens over the whole range of p_0' studied, but the strength characteristics at low consolidation pressures (p_0'< 100kPa) was different from that at higher consolidation pressures.
- In terms of stiffness, however, at low values of p_0', intact samples had higher stiffness than compacted sample but the disruption of bonds as p_0' increases from 50 to100kPa leads to a rapid loss of stiffness.
- The stress paths of both intact and compacted show structure pseudo-overconsolidation behaviour below 100kPa but a clear compression-dilation behaviour at higher stress.
- Intact and compacted samples had similar effective friction angles but intact sample had effective cohesion value 4 times higher than the compacted sample which is attributed to the sesquioxide bonds.
- The estimated critical state friction angle of 39^o for intact samples were significantly higher than the 32^o for compacted samples.

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