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Critical state parameters for a saturated lateritic clay

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ABSTRACT: The critical state soil mechanics is the most useful framework for understanding responses of different soil type to mechanical stress. For lateritic soil, widely distributed soil in the tropics and well recognized for its rich composition in oxides of iron and aluminum (sesquioxide), its mechanical behaviour has not been well studied within this framework. Therefore, a good understanding of the compressibility and shear strength properties of the lateritic soil would improve the designs of structures built on it. This paper presents experimental data from isotropic compression and consolidated undrained triaxial tests on saturated samples of lateritic soil from Nigeria. The data from these tests are presented and interpreted within the critical-state framework. Effects of sesquioxide composition on the compressibility and dilatancy is reported and discussed.

1 INTRODUCTION

The critical-state concept is a very important framework for interpreting the behaviour of saturated soils (Schofield & Wroth, 1968). When a soil undergoes shearing, it reaches a state where it continues to deform at constant stress and constant void ratio. This state is the critical state and for most soils, the critical state is reached at large strains above 25%. The conventional understanding is that when a soil is sheared to such a large strain i.e. above 25%, the effects of fabric on the soil behavior should have been erased. Therefore, the strength parameters obtained at this critical state is independent of soil fabric and is unique for each soil type (Atkinson, 1993). The determination of the critical state line for a soil can aid the interpretation of its mechanical behavior especially during shearing. For instance, a soil specimen whose initial state lies above the critical state line in the v-ln p' compression plane is considered wet/loose and according to critical state soil mechanics, a strain-hardening and contraction behavior is expected during drained shearing. For soil specimen whose initial state falls below the critical state line in the same plane, a strain softening and a dilative behavior is expected during drained shearing (Atkinson, 1993).

The critical state theory has been used to interpret the mechanical behavior of many soil type and their unique parameters has been determined (Ng & Chiu, 2001 & 2003; Wheeler & Sivakumar, 1995; Gan & Fredlund, 1996). However, for lateritic soil, a type of soil that is found in many region of the world, there are very few studies investigating its behavior within the critical state framework. Although, there has been several studies on the mechanical behavior of the lateritic soil (Ng et al., 2019; Otalvaro et al., 2015; Futai & Almeida, 2005; Toll, 1990), still there is a wide variation in the shear strength parameters available in the literature. While most authors stop shearing the test at a small strain between 1% and 5%, others adopt the Mohr Coulomb theory for interpretation of the shear result. In Africa, unconfined compression test is one of the most common shear test conducted on lateritic soil and the maximum shear strain is generally less than 10% (Owolabi & Aderinola, 2014; Ola, 1978;). In Brazil, some authors conducted undrained and drained triaxial testing on both compacted and intact lateritic soil specimen (Fagundes & Rodrigues, 2015; Futai et al. 2004). However, majority of these authors studying lateritic soil adopted Mohr-Coulomb theory rather than the critical state framework. As such, the strength parameters in the literature are mostly peak state parameters which are dependent on the initial fabric, (water content and

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sample preparation technique) of the specimen. To understand the behavior of lateritic soil more explicitly, determining unique critical state parameters is therefore important.

Furthermore, the lateritic soil is also well recognized for its rich composition in oxides of iron and aluminum (sesquioxide) (Gidigasu, 1976). The soil, even though a fine material, contains large particles which results from the aggregation influence of its sesquioxide content (Zhang et. al., 2014). Thus, the lateritic soil is a highly structured material depending on the quantity of the sesquioxide content. Previous researchers working on highly structured soil found that the critical state may be impossible to reach even if shearing exceeds 25%. According to previous researchers (Ferreira & Bica, 2006; Martins et al. 2001), the fabric of highly structured soil maybe difficult to erase and therefore no unique parameter may be determined and determining critical state parameters may be unrealistic. There are very few authors who have sheared the lateritic soil to such large strain, therefore, it is still not clear if critical state parameters could be determined for lateritic soil.

In this paper, a compacted and saturated lateritic clay from Nigeria undergoes isotropic compression and undrained shearing test at three different confining stresses. The results were interpreted within the critical state framework, a critical state line and its parameters were determined.

2 TEST MATERIAL

Disturbed sample of lateritic clay from Southwest, Nigeria was collected and tested in this study. Large aggregates in the collected samples were broken down and the soil was sieved through 2mm ASTM standard sieve. Particles larger than 2mm were discarded and not used for the determination of the geotechnical index properties and the shearing test. The index properties of the soil were determined following the ASTM standard (ASTM, 2011). Due to possible aggregation as a result of the sesquioxide content in lateritic soil, wet sieving and hydrometer method were used to determine the particle size distribution and a sodium hematophospate was added to the solution as dispersant to separate soil aggregates. The coarse and fine fractions of the tested soil are 42% and 58% respectively. Details of other index properties of the soil are summarized in Table 1. The soil was classified as a sandy lean clay (CL) (ASTM D2487, 2011). The sesquioxide content in the tested soil was determined using X-ray fluorescence test and quantified to be 38%.

3 SPECIMEN PREPARATION AND TEST PROGRAMME

A total number of 6 specimens was prepared for the isotropic compression and undrained shearing tests using static compaction method. Each specimen was compacted in 10 layers at 19% compaction water content to a dry density of 74% of the proctor maximum. Specimen uniformity was ensured by adopting the under-compaction method of preparation proposed by Ladd (1978).

Two series of tests were carried out in a triaxial apparatus, the first series is isotropic loading and unloading tests to obtain the compressibility parameters. The second series is consolidated undrained (CU) shear test to investigate the shear behaviour and obtain the critical state strength parameters. For the shear test, normally consolidated specimen and over-consolidated specimen were tested in order to understand the influence of stress history on the shear behaviour of the tested material. The normally consolidated specimen were sheared at three different confining stresses (50, 100 and 200 kPa). The over-consolidated specimen were sheared at 50kPa confining stress after been loaded and unloaded to achieve over-consolidation ratio of 5 and 7. Details of the test are summarized in Table 2.

4 TEST PROCEDURES

After specimen preparation, a porous stone was placed underneath and on top of the specimen and carefully placed on the base pedestal of the triaxial apparatus. The specimen was flushed with

Table 1. Classification and mineral composition of studied material.

Index test	Lateritic soil
Standard compaction Test	
Maximum dry density: kg/m^3	1696
Optimum water content: %	20
Grain size distribution	
Percentage of sand: %	42
Percentage of silt: %	16
Percentage of clay: %	42
Coefficient of uniformity	35
Coefficient of gradation	1.6
Atterberg limits	
Liquid limit: %	44
Plastic limit: %	24
Plasticity index: %	20
Specific gravity	2.67
Soil classification based on USCS (ASTM,2011)	CL (sandy lean clay)
Chemical composition (%)	
Silicon oxide (SiO_2)	60
Iron oxide (Fe_2O_3)	10
Aluminum oxide (Al_2O_3)	28
Sesquioxide (Fe_2O_3 + Al_2O_3) content	Lateritic soil = 38%

Table 2. Testing conditions for CU shear test on saturated specimens of lateritic clay.

Series	Specimen ID*	Confining stress (kPa)	Initial void ratio	Void ratio after consolidation	Ψ after consolidation*
	L50	50	0.948	0.912	0.06
I	L100	100	0.942	0.872	0.09
	L200	200	0.956	0.803	0.09
II	L50 - OC5	50	0.945	0.821	-0.02
	L50 - OC7		0.962	0.765	-0.07

CO_2 and de-aired water respectively, then back pressure was applied to fully saturate the specimen. The minimum back pressure used in all the test was 250 kPa and minimum B value achieved for all the saturated specimen is 0.97. There was no measurement of volume change during saturation.

For the isotropic loading test, the following confining pressures were applied in stages: 15, 25, 50, 100, 200, 300, 400 kPa. Unloading was done at 250 and 350 kPa respectively to take the specimen back to 50 kPa given a OCR value of 5 and 7 respectively. After isotropic compression test, consolidated undrained shearing test was carried out. After bringing each specimen to the design effective stresses, drainage valves were closed after the excess pore water pressure has been dissipated, then an undrained shearing test was performed. The strain rate was 0.04 mm/min during the shearing process.

5 DISCUSSION OF RESULTS

5.1 *Isotropic loading and unloading*

Figure 1 shows the response of the lateritic specimen to isotropic loading and unloading. At first, a gradual change in specific volume is observed as the effective mean stress increases. Then, a yield point is reached and the specific volume begins to reduce abruptly. The yield stress corresponding to the yield point is estimated to be 80 kPa using Casangrande's graphical method. At 100 kPa effective stress, the specimen is unloaded in stages to 50 kPa and a very slight increase in the specific volume is observed. The specimen is compressed again from 50 kPa in stages to 300 kPa effective mean stress and then unloaded back to 50 kPa. A post yield isotropic normal compression line (NCL) is determined and its slope (usually denoted as λ) is estimated as 0.07. The slope of the unloading curve is also estimated to be 0.013. The value of λ obtained in this study is compared with the saturated compressibility of some other compacted lateritic soils reported in the literature and summarized in this study are summarized in Table 3. It is clear that the compressibility of lateritic soils varies significantly with particle sizes. The lateritic soil in this study have the least compressibility even though it contains significant amount of clay compared other soils in the Table 3.

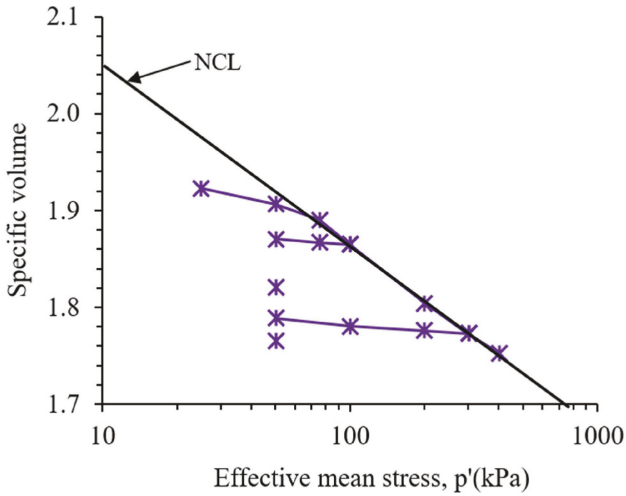


Figure 1. Isotropic compression behavior of the tested specimen.

The reason for this low compressibility of the lateritic soil in this study may be due to the influence of its 38% sesquioxide composition. The sesquioxide content in most lateritic soils is not often quantified, thus, comparing compression behavior may be misleading. The sesquioxide enhances aggregation, hence larger particles are formed and specimen may behave as a coarse grained material (Zhang et al., 2016).

Table 3. Comparison of compression index of compacted lateritic soils.

Lateritic soil types	λ	Parent rock
Sandy lean clay (this study)	0.08	Granitic (Igneous)
Gravel (Toll, 1990)	0.11	Basalt (sedimentary)
Clayey silt (Otalvaro et al., 2015)	0.16	Paranoá (meta-sedimentary)
Clayey sand 1 (Bennatti et al., 2013)	0.15	Colluvium (sedimentary)
Clayey sand 2 (Futai & Almeida, 2005)	0.18	Gneiss (sedimentary)

Figure 2 shows the initial condition of the specimen before shearing. It is noted that all the normally consolidated (NC) specimen lies close to the NCL line determined during isotropic compression. The over-consolidated specimens lies farther away from the NCL compared to the NC specimens.

5.2 *Shear behavior of the normally consolidated specimen*

Figure 3a shows the stress-strain relationship during undrained shearing of the lateritic specimens at three confining stresses. At 50 kPa confining stress, the deviator stress increases monotonically with the axial strain and no sign of softening is observed till the end of the test. The behavior at

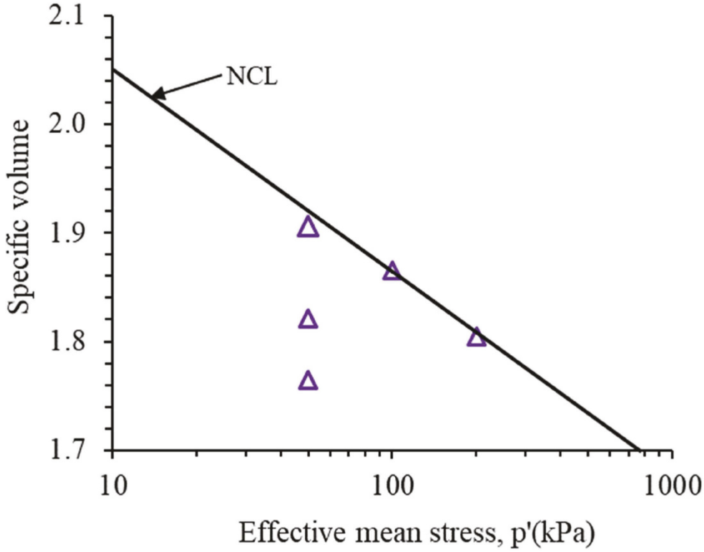


Figure 2. States of all saturated specimens before shearing.

50 kPa is generally referred to as a strain hardening behaviour and this is consistent with the critical state framework for normally consolidated specimen. At 100 kPa and 200 kPa confining stress, a similar strain hardening behaviour with no evidence of softening is observed. As expected, maximum deviator stress increases with confining stress. It can also be observed that the deviator stresses at the three confining stresses are still changing slightly towards the end of the test suggesting a true critical state has not been reached. However, the rate of increase in deviator stress at 50 and 100 kPa towards the end of the test has reduced significantly. The behaviour of the studied soil may be due to the influence of the sesquioxide content which may enhance aggregation of particles (Zhang at al., 2004). Due to aggregation, the fabric of the soil may require larger shear strain to fully reach critical state.

Figure 3b shows the relationship between the excess pore water pressure and the axial strain during the undrained shearing test at all confining stress. At 50 kPa, the specimen shows initial build-up of positive excess pore water pressure, it reaches a peak and then a slight drop from peak is observed till the end of the test. The initial build-up of the pore pressure indicate that specimen exhibit a contractive tendency while the slight reduction indicates a dilative tendency. At other confining stresses, similar behavior observed at 50 kPa is observed in all the specimen. A barreling type of failure was observed in all the specimen after testing. All these observations are consistent with critical state framework for soils sheared at normally consolidated state (Atkinson, 1993).

5.3 *Shear behavior of the over-consolidated specimen*

The stress strain relationship of the over-consolidated specimen sheared at confining stress of 50 kPa is shown in Figure 4a. All the specimen showed a strain hardening behaviour despite having different stress histories. It can be seen that the deviator stress increases monotonically with the axial strain and no peak nor sign of softening is observed. As expected, the maximum deviator stress increases with increasing OCR. Most of the specimen are still showing significant change in deviator stress at the end of the state. This indicates that the specimen have not reached the critical state. Lateritic soil may require shearing to a larger strain (above 25%) in order to reach a true critical state. In Figure 4b, the relationship between the excess pore water pressure and the axial strain is shown. It can be seen that positive excess pore water pressure was initially generated in all the specimen. As shearing continues, a reduction in pore water pressure is observed in all the specimen but only the specimen with OCR of 1 and 5 appears to have reached a plateau at the end of the test.

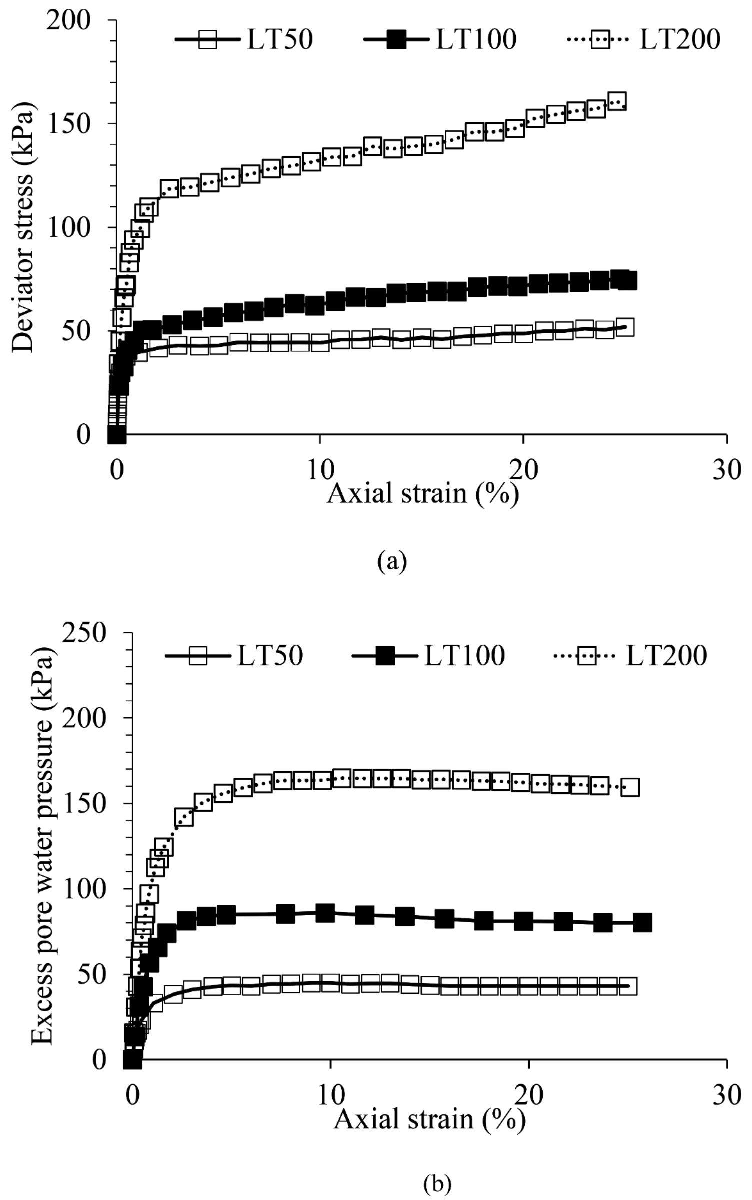
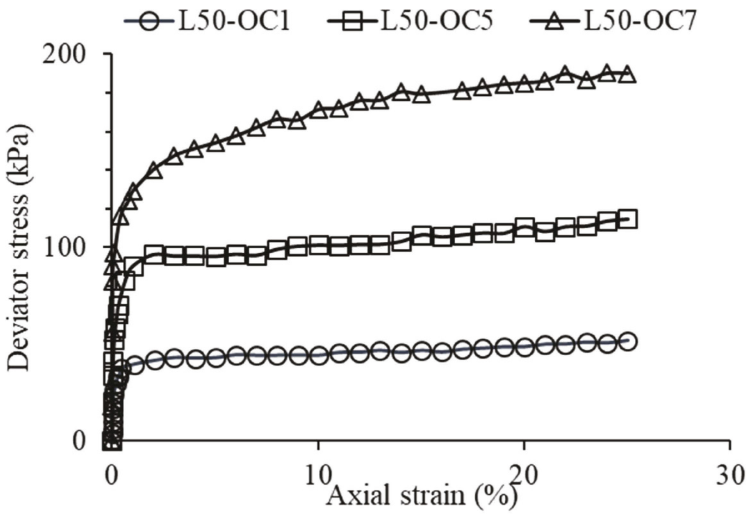


Figure 3. (a) Triaxial stress strain and (b) Pore pressure vs strain responses from undrained tests for the normally consolidated saturated lateritic specimen.

5.4 *Phase transformation behaviour*

Figure 5 shows the stress paths of the normally consolidated lateritic specimen during undrained shearing in the q – p′ plane. For all the specimen, the effective mean stress initially reduces (showing a tendency of contraction), then at a later stage, a turning point is reached and phase transformation occurs. Under subsequent shearing, the effective mean stress increases (showing a tendency of dilation, accompanied by an increase in deviator stress) and soil state finally reaches the CSL. The initial contraction is likely attributed to the collapse of large inter-aggregate pores. The subsequent dilative behaviour is due to re-arrangement and interlocking of the large-sized aggregates. Futai & Almeida (2004) also observed a phase



(a)

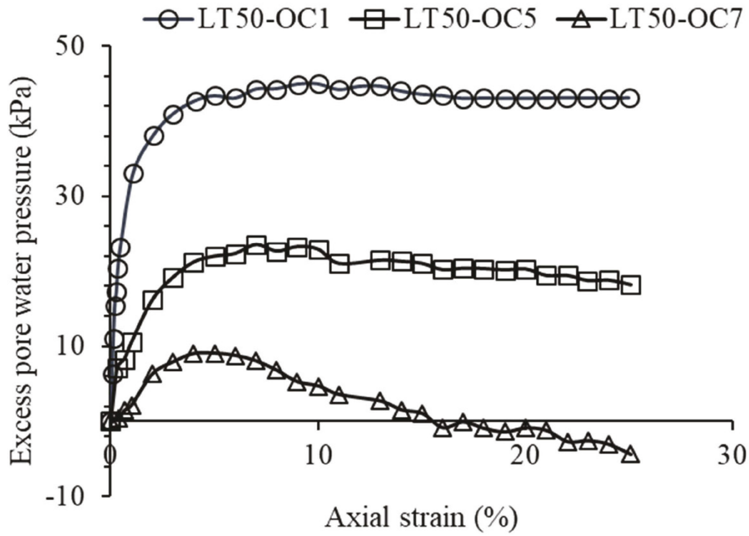


Figure 4. Influence of stress history on the shearing response of the studied lateritic soil (a) stress strain and (b) pore pressure vs strain responses from undrained tests.

transformation behaviour on a saturated and compacted lateritic clay from Brazil. This is likely due to the effect of the aggregates giving the specimen a behaviour typical of coarse-grained material.

Figure 6 shows the stress paths of the specimen at OCR of 1, 5 and 7 during undrained shearing in the $q - p'$ plane. At 50 kPa confining stress, the effective mean stress of the specimen at OCR of 1 initially reduces indicating a tendency of contract. Under subsequent shearing, the effective mean stress increases (showing a tendency of dilation, accompanied by an increase in deviator stress) and soil state finally reaches the CSL. For the specimens at OCR of 5 and 7, the effective mean stress initially increases indicating a tendency of dilate. Under subsequent shearing, the effective mean stress decreases (showing a tendency of contract, accompanied by an increase in deviator stress).

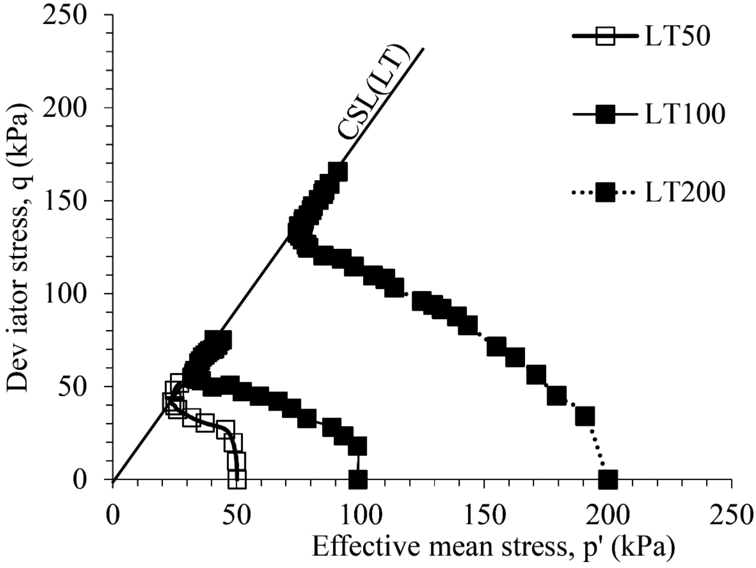


Figure 5. Stress paths of normally consolidated saturated lateritic specimen in q - *p*' space.

According to the stress path shown in Figure 7, the final states of all specimens fall on a single line. The slope of the critical state line, M (commonly referred to as the stress ratio of the critical state) line, is estimated to be 1.73. The corresponding critical state angle of internal friction φ′ is 42° for the studied specimen. The high friction angle is due to the influence of large particles and aggregates in their microstructure.

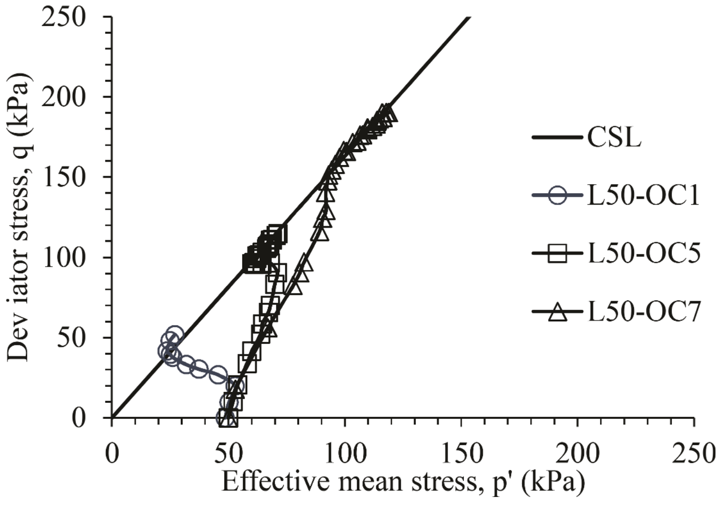


Figure 6. Stress paths of the saturated lateritic specimen at different stress history in q - *p*' space.

Figure 8 presents the stress path of the specimen in the v-ln p' compression plane. The effective mean stress of the normally consolidated specimens shift to the left at the end of shearing indicating a reduction in effective mean stress. On the other hand, the over consolidated specimen shifts to the right indicating an increase in the effective mean stress after shearing. For all the specimen tested in this study, a unique line is determined which is parallel to the slope of the NCL. This line is the critical state line in the compression plane. The slope of the CSL (λ) in this plane, the specific volume (Γ) at p'=1kPa were determined. These two parameters are

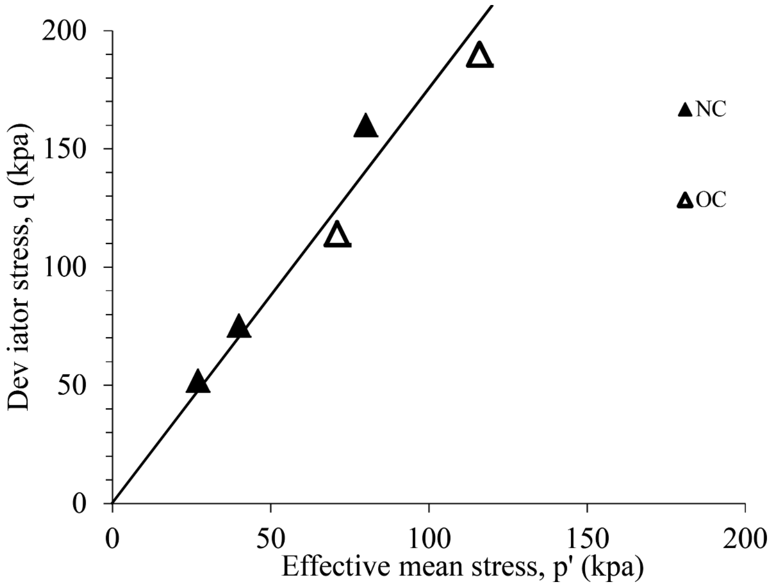


Figure 7. The critical state line of the studied lateritic soil in q – p’ space.

critical state parameters and are unique to the soil tested in this study. The critical state parameters obtained for the saturated lateritic specimen tested in this study are summarized in Table 4. These parameters are useful for interpreting the behaviour of lateritic soils.

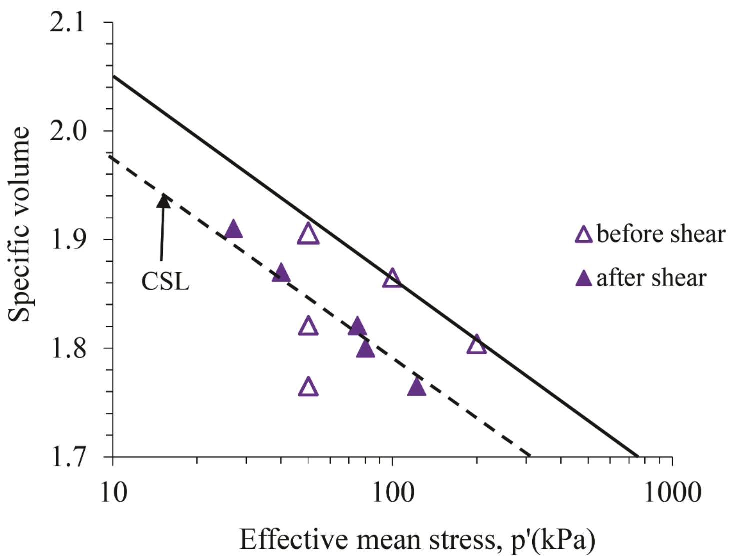


Figure 8. States of all saturated specimens before and after shearing in v – ln p’ space.

6 CONCLUSION

In this study, the critical-state parameters for a saturated sample of a lateritic clay were experimentally determined using the conventional triaxial test apparatus. Isotropic compression and consolidated undrained shearing test were carried out on both normally consolidated and

Table 4. Summary of the critical state parameters obtained in this study for the tested lateritic specimen.

*CSL parameters	Lateritic Soil
λ	0.07
Γ	1.98
N	2.05
M	1.733

* where M is the slope of the projection of the critical-state line in q– p' space, Γ and N are the intercept of the CSL and NCL respectively (at p' = 1 kPa in the v – ln p' space), λ and M are the slope of the projection of the critical-state line in v – ln p' space and q – p' space respectively.

over consolidated specimen. Specimen were fully saturated with a back pressure not less than 200 kPa. At the end of shearing, most of the specimen did not reach the true critical state as the deviator stress is still increasing at the end of the test. Unlike most clays where the critical state is often reached at about 25% axial strain, the lateritic soil needs to be sheared to a larger strain in order to erase the effects of fabric on the strength parameters. The behaviour of lateritic soil is attributed to the influence of its highly aggregated microstructure which is due to the influence of its sesquioxide content. Even though a true critical state was not reached by some of the specimen, critical state parameters were still derived at the end of the test. The test results have shown that the slope of the NCL and CSL lines for the saturated lateritic specimens are parallel to each other. The parameters of M and λ have been determined and it was found that the lateritic clay has a larger friction angle which is not typical for a sandy clay material. The large friction angle is therefore attributed to the influence of its sesquioxide content which enhances formation of large grain sizes through aggregation of soil particles. The intercept of the critical state line in the compression plane was also determined which is unique to the studied soil. Considering the variability in the microstructure of different lateritic soil owing to the presence of sesquioxide, it is impossible to generalize the critical state parameters in this study for all lateritic soil. Therefore, the result of this study could be useful for interpreting lateritic specimen with similar grain size distribution and sesquioxide content.

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