The strength and deformation characteristics of an intact and compacted lateritic soil

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ABSTRACT: The shear strength is a key parameter in the design and construction of slopes in both natural material and in embankments. In laterites and lateritic soils, the sesquioxides consisting of the aluminium and iron oxides provide a bonded structure which is destroyed by excavation and re-compaction. The objective of this investigation is to compare the strength and deformation characteristics of an intact and recompacted lateritic soil and to evaluate the influence of stress and of saturation. Intact and re-compacted samples of a lateritic soil at a dry density of 1.64 Mg/m^3 and a water content of 18% were used. Samples of both re-compacted and intact soil were subjected to soaked and unsoaked direct shear tests under a range of normal stresses varying from 20 kPa to 400 kPa. Another set of both re-compacted and intact samples were subjected to consolidated undrained triaxial compression with pore pressure measurement under confining pressure varying from 20 kPa to 150 kPa. The shear strength behaviour of the intact and re-compacted samples under soaked and unsoaked condition over both low and high normal stresses were analysed. The influence of saturation on the results are also discussed.

1 INTRODUCTION

Laterite is a highly weathered material rich in the secondary oxides of iron and aluminium but low in the bases. It usually has a distinctive reddish-brown colour. The secondary oxides of iron and aluminium collectively known as sesquioxides, provide cementation between the clay particles and further binds them into larger aggregates. This creates the so called “ped” structure in laterites with pores at the macro and micro levels (Hobbs et al. 1992, Otalvaro et al. 2015). The nature of the bonding forces and the structure greatly influences the mechanical properties of the lateritic soil.

Lateritic material is abundant in tropical countries. Consequently, they constitute the bulk of the material on which structures are built, and they are also used extensively as a construction material. In road construction in the tropics, deep cuts and embankments are frequently made in laterites and lateritic soils. In most cases the material cut from one section is used as fill for embankments. In its in-situ condition, a lateritic soil is in its natural state and it is said to be intact. Such a soil may be considered structured and undisturbed. However, when the soil is excavated, spread and re-compacted during construction of embankments, the bonds are destroyed and re-compaction may also introduce a new structure. The difference in the shear strength of a lateritic soil in a cut and that in an embankment arises mainly from the different structures in the two. It has also been suggested that the bonds provided by the oxides of iron and aluminium are also destroyed by high confining pressures and by saturation (Ampadu 2007). The overall objective of this study therefore is to compare the shear strength and deformation characteristics of an intact and re-compacted lateritic soil and so derive the effect of soil structure, normal stress and soaking on the shear strength. The shear strengths were measured in drained direct shear tests and in consolidated undrained triaxial compression tests.

2 METHODOLOGY

2.1 *Sampling*

A site on a gentle slope was selected on the KNUST campus. A trial pit was dug manually to a depth of 1m and logged. Bulk disturbed samples were taken from a depth of between 0.4 m to 1 m and put in sacks. Undisturbed block samples were taken at the bottom of the pit using a pickaxe, wrapped in layers of cling film and placed into an insulated container to prevent moisture loss. The samples were then transported carefully to the laboratory for testing. To obtain samples for determination of the in-situ dry density and water content, three core-cutters were gently hammered into the bottom of the trial pit using a wooden plank as a shock absorbed. The core cutters were extruded, wrapped, sealed and placed into an airtight container and also sent to the laboratory. In the laboratory, the soil from the sacks were spread out, lumps were broken down and the soil was air-died for 3 days. The air-dried sample was re-bagged and kept in a temperature-controlled room to prevent moisture changes.

2.2 *Index property tests*

Samples were retrieved from the bags and used for the index property tests. The index properties of the lateritic sample were determined using BS 1377, Part 2:1990. For this study kerosene was used in lieu of water to determine the specific gravity. The cone penetrometer was used to determine the liquid limit of the lateritic sample. The Modified AASHTO compaction characteristics of the lateritic sample were determined according to ASTM D1557-91 using the sample re-use option.

2.3 *Sample preparation for shear strength tests*

2.3.1 *Intact Samples*

To obtain intact samples for the direct shear test, the (100mm x100mm) direct shear cutter was pressed firmly into the undisturbed soil block sample and extruded. The extruded specimen was placed into the direct shear box and trimmed top and bottom to flush with the edges of the box to create the intact samples designated PIU for the unsoaked and PIS for the soaked test series.

For the triaxial test, in order to obtain the intact samples for the PI-TX test series, sharpened tubes were carefully pushed into the block sample and extruded. The extruded sample was then trimmed to the 76mm height. In all cases, trimmings were collected for water content determination

2.3.2 *Compacted samples*

For the compacted specimens, about 500g of the air-dried sample was weighed and the mass of water required to bring the soil sample to the in situ water content was determined and poured evenly over the soil sample in a pan. The soil and water were mixed thoroughly by hand, bagged and kept in a temperature-controlled room for 24 hours to achieve moisture equilibration. Several units of such samples were prepared.

For the direct shear test, the mass of wet soil required to fill the shear box at the in-situ dry density was weighted. Then this mass of soil was compacted directly into the direct shear box with the aid of a mallet and wooden block to create the compacted samples designated PCU for the unsoaked and PCS for the soaked test series. The initial weight of the specimen was taken and trimmings were collected for water content determination.

For the triaxial test, a relationship between the number of blows of the 25kg rammer and the dry density was established in the CBR mould. Then part of the soil was compacted into the CBR mould using the selected number of blows to give the required in-situ dry density. Sample tubes were pressed into the compacted sample to cut out the required samples for the PC-TX test series. The samples were then trimmed for the triaxial test.

2.3.3 *The Direct shear test procedure*

The direct shear test was performed in accordance with ASTM D3080 using a conventional direct shear apparatus. For the unsoaked test series (PU-series) after the specimen was set-up in the direct shear box, the normal stress (consolidation pressure), was applied for about 8 minutes until there was no change in volume of the specimen, measured by the vertical dial gauge reading. The specimen was then subjected to the drained direct shear test at a constant rate of shear displacement of 0.9 mm/min under the specified pressure. The shearing force was measured by the proving ring and the horizontal and vertical displacements were measured by the dial gauges. For the soaked test series after the specimen was set-up in the direct shear box it was flooded for 24 hours. After that the sample was consolidated under the specified consolidation pressure for 30 minutes, before the drained shearing. The test procedure was repeated for all the normal stresses ranging from 20 kPa to 400 kPa

2.3.4 *The Consolidated Undrained Triaxial Test*

The automated GDS triaxial apparatus was used for the test. The sample was set up in the triaxial apparatus, and saturated by back-pressuring. It was noted that high back pressures of the order of 300kPa to 400kPa were required to achieve an acceptable B-value of 0.95. After saturation, the sample was consolidated under isotropic pressures ranging from 20 kPa to 150 kPa. After the consolidation, the samples were sheared undrained to failure with pore pressure measurement.

3 RESULTS AND DISCUSSIONS

3.1 *Properties of lateritic soil*

The trial pit showed a soil profile consisting of 0.1 m of dry loose, dark brown, gravelly sand with plant roots at the top. Below this layer from 0.1 m to 0.4 m was made up of dense, dark brown to reddish, gravelly sand to silty clay. The third layer from which both the disturbed samples and the block samples were taken ranged from 0.4 to 1 m and it comprised a dense, reddish sandy silty clay. The average in-situ dry density obtained using the core-cutters was 1.64 Mg/m^3 while the average in-situ moisture content was 18.5 %. These values were used as target values for the preparation of the compacted specimens.

The properties of the lateritic sample are summarized in Table 1. The lateritic sample had a specific gravity (Gs) of 2.83. It had a low gravel content of 2.4 %, but a high clay content of 40 %. According to the Unified Soil Classification System (USCS) it is clay of high plasticity (CH). The compaction characteristics are shown in Figure 1 for compaction using fresh samples and reusing samples. The Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) were 1.72 Mg/m3 and 18.4 % respectively for the Modified AASHTO compaction using the sample reuse option. Figure 2(a) and (b) is a section through an intact and a compacted sample respectively.

Table 1. Summary of characteristics of lateritic sample

Gravel (%)	Sand (%)	Silt (%)	Clay (%)	LL (%)	PI (%)	Gs
2.4	36.4	21.3	39.9	60	33	2.83

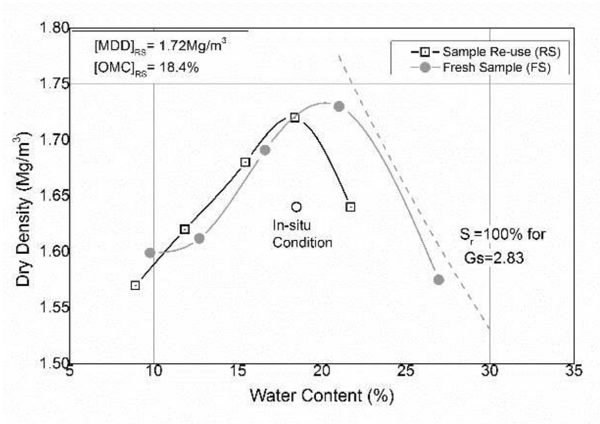

Figure 1. Compaction characteristics of laterite

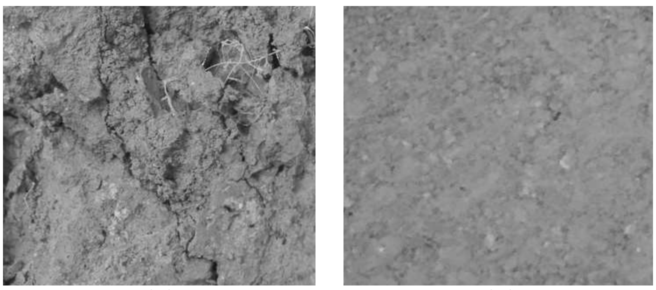

Figure 2. (a) Intact and (b) compacted sample

3.2 *Direct Shear Test Results*

For the compacted samples, the samples were prepared and sheared at an average dry density of 1.65Mg/m^3 and a water content of 17.2 % giving an average degree of saturation of 68 %. The intact samples had an average dry density of 1.65 Mg/m^3 at an average water content of 17.5 % giving a mean degree of saturation of 70 %.

3.2.1 *Stress-displacement characteristics*

The shear stress-displacement characteristics of unsoaked intact samples are compared with the equivalent compacted samples in Figure 3 and 4 for low normal stress (i.e. 20 kPa and 50 kPa) and for high normal stresses (300-400 kPa) respectively. The figures show that for consolidation pressures of 20 kPa and 50 kPa, the intact sample exhibit higher shear strength and slightly more dilation than compacted samples and also tend to be stiffer. The plots also show clear post-peak reduction in shear stress. For the high normal stresses Figure 3 shows that the shear stress-displacement relationship for both intact and compacted sample appears similar and do not show any post-peak reduction in shear stress. The stress-strain curves for the medium range pressures (i.e. 75 kPa-200 kPa) show a transition between these two extreme behaviours. Overall, the stress-displacement relationships show clear post peak reduction in shear stress only at low normal stresses and as the normal stresses increase the post peak reduction in shear stress disappears. This behaviour is similar to that of dense sand

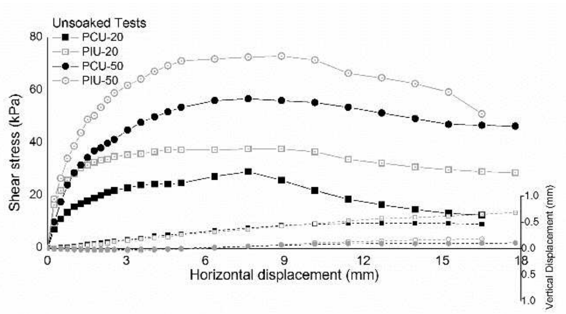

Figure 3. Unsoaked direct shear at low confining stress

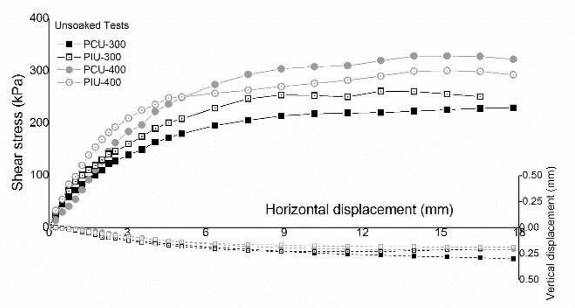

Figure 4. Unsoaked direct shear at high confining stress

For the saturated tests, the shear stress-displacement curves for intact and the compacted samples are shown in Figure 5 and 6 for low and high pressures respectively. The curves appear similar for all normal stress levels, suggesting that the saturation appears to have erased the tendency towards post-peak reduction in shear stress at low confining stresses.

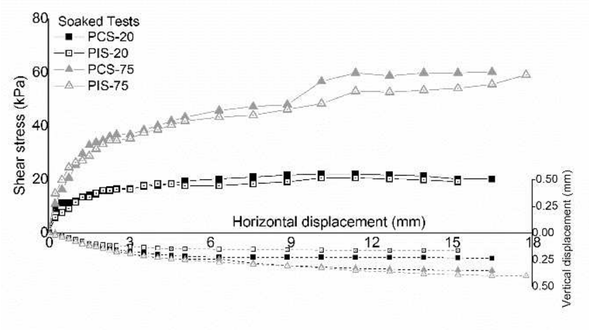

Figure 5. Soaked direct shear at low confining stress

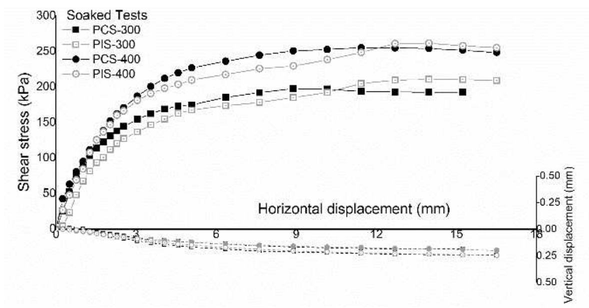

Figure 6. Soaked direct shear at high confining stress

3.2.2 *The failure Condition*

The failure shear stress in drained direct shear is defined as the maximum shear stress or the shear stress at 20mm displacement whichever is attained first. The stiffness is also defined as the secant modulus at 50% of the failure shear stress and the values are shown in Table 2 and Table 3 for unsoaked and soaked samples respectively. The 50% failure shear stress occurred at displacement ranging from 0.45mm to 2.4mm.

Table 2. Unsoaked failure stresses and stiffness values

Normal Stress (kPa)	Failure Shear Stress (kPa)		Stiffness (kPa/mm)	
	Intact	Compacted	Intact	Compacted
20	37.4	29.2	10.9	8.5
50	73.1	56.9	43.8	16.6
75	64.4	87.1	36.6	26.1
100	99.0	98.6	42.4	38.8
150	141.5	155.9	50.0	71.3
200	180.4	177.5	48.7	49.0
300	252.7	217.8	64.2	53.6
400	276.0	307.8	90.0	63.9

Table 3 Soaked failure stresses and stiffness values

Normal Stress (kPa)	Failure Shear Stress (kPa)		Stiffness (kPa/mm)	
	Intact	Compacted	Intact	Compacted
20	20.5	22.0	11.7	23.4
50	46.1	37.1	27.5	16.2
75	48.2	56.6	31.3	23.7
100	76.7	72.7	31.7	47.5
150	117.0	106.6	35.0	46.0
200	139.7	158.8	57.9	49.6
300	192.2	197.6	58.6	85.6
400	238.3	252.7	83.0	67.2

The failure shear stresses are plotted against the normal stresses in Figure 7. The failure envelope may be modelled as a linear envelope with very good coefficient of regression ranging from 0.96 to 0.99. Such a linear model gives the failure parameters values in Table 4. It can be seen that the compacted sample has a slightly higher angle of internal friction but a lower cohesion intercept than the intact sample. This was found to be true for both soaked and unsoaked condition. Soaking reduces both the angle of internal friction and the cohesion intercept for both intact and compacted samples. However, whereas the reduction in the angle of internal friction is only 3 to 4 degrees, the reduction in the cohesion intercept is substantial (17 kPa) of the order of about 50 %.

Table 4. Failure parameters for linear failure envelope

	Unsoaked		Soaked	
	c_d (kPa)	ϕ_d (°)	c_d (kPa)	ϕ_d (°)
Intact	33	34	16	30
Compacted	29	35	12	32

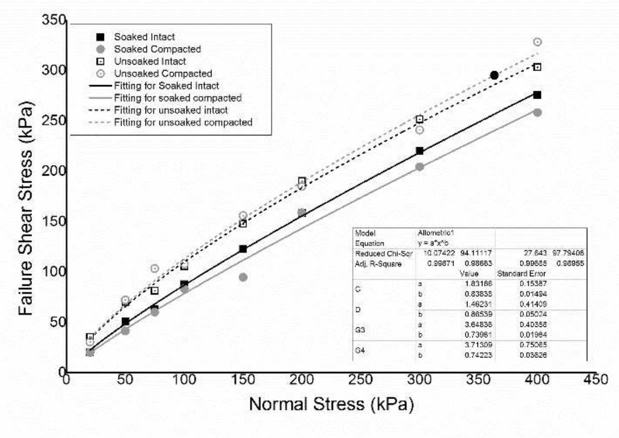

Figure 7. Direct shear failure envelopes

However, a critical observation shows a curved failure surface which may be modelled as a bi-linear failure envelope separately for low and for high confining pressures or as a power function. A bi-linear model gives a breakpoint of between 150 kPa to 200 kPa. Figure 7 shows the fit curves for a power function without any cohesion (Lade 2010), which gives an excellent fit to the data with a coefficient of regression value of not less than 0.98. The figure shows that, overall, there is very little difference in strength between intact and recompacted samples under soaked and unsoaked conditions for stress levels less than about 150 kPa to 200 kPa. At vertical stresses larger than about 200 kPa and about 150 kPa for unsoaked sample and for soaked sample respectively, compacted samples tend to have a slightly higher strength than intact samples.

3.3 *The Triaxial Test results*

Hobbs et al. (1992) pointed to the existence of both micro and macro pores in which different processes take place during consolidation of lateritic soils. They explained that while consolidation progresses in the macro-pores, rehydration simultaneously takes place in the micro-pores resulting in consolidation followed by swelling. Recently, Otalvaro et al. (2015) highlighted the existence of such a double structure in compacted residual soil. This phenomenon was also observed during consolidation at low confining pressure in some of the test samples, where despite achieving a high B-value prior to consolidation, the primary consolidation was followed by a considerable swelling as illustrated with test PC-75-TX of Figure 8. This phenomenon complicates the definition of the specific volume-pressure relationship.

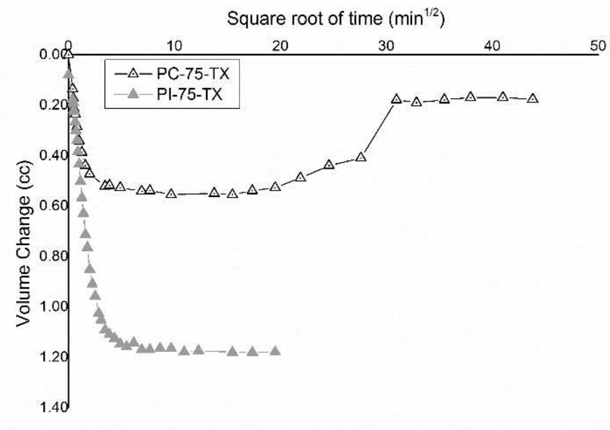

Figure 8. Selected Consolidation characteristics

The stress-strain curves and the corresponding stress-paths for intact samples in the CU-TX tests are shown in Figure 9 and Figure 10 respectively while the equivalent plots for compacted samples are shown in Figure 11 and Figure 12 respectively. The samples did not attain clear peaks at strains less than 15 %, so failure is defined as the maximum deviator stress or the deviator stress at 15 % axial strain whichever occurs first. The failure values are summarized in Table 5 and Table 6 for intact and compacted samples respectively. In these Tables the mean effective pressure values refer to the end of consolidation. However, the stress paths indicate slight deviation from these values at the start of shearing for some tests.

Table 5. Failure condition for CUTX for intact samples

Test ID	Mean Eff. Pressure (kPa)	q_{max} (kPa)	E_{50} (kPa)	c' (kPa)	ϕ' (°)
PI-20-TX	20	179	1583		
PI-50-TX	50	188	3613	13	30
PI-75-TX	72	201	10791		
PI-150-TX	150	275	11385		

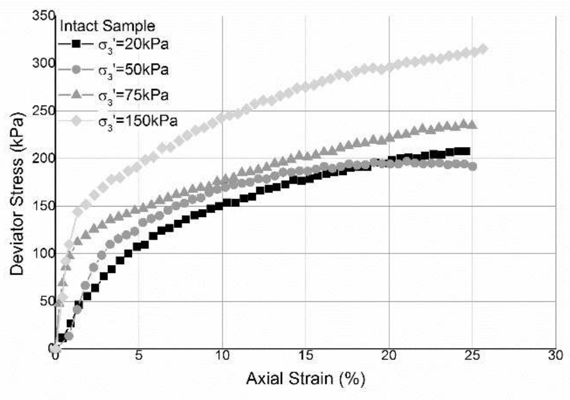

Figure 9. Deviator stress-strain curves for intact samples

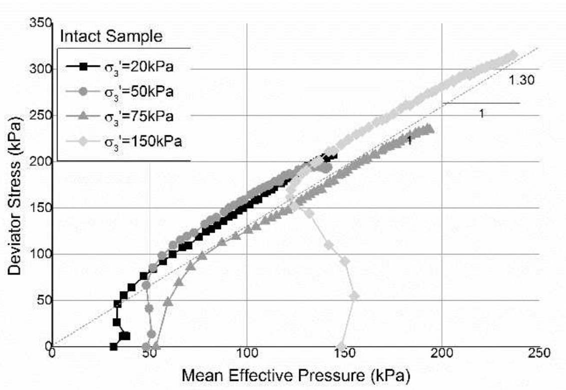

Figure 10. Stress-paths for intact samples

Table 6. Failure condition in CUTX for compacted samples

Test ID	Mean Eff. pressure (kPa)	q_{max} (kPa)	E_{50} (kPa)	c' (kPa)	ϕ' (°)
PC-20-TX	20	327	5255		
PC-50-TX	48	348	7903		
PC-75-TX	75	336	11244	14	34
PC-100-TX	100	434	11472		
PC-150-TX	148	524	20878		

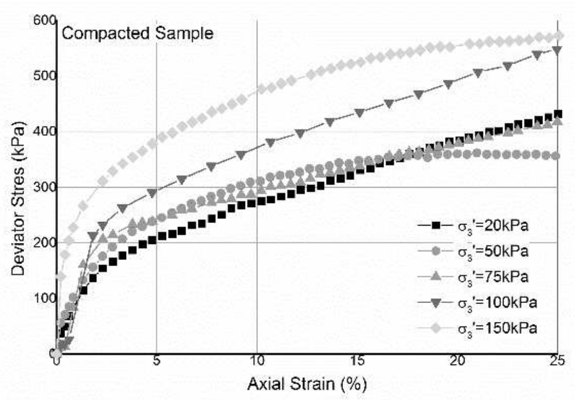

Figure 11. Deviator stress-strain curves for compacted samples

Based on the maximum deviator stress the effective angle of internal friction of compacted samples was 34° while that for the intact samples was 30°. The cohesion intercept values of 13 kPa and 14 kPa were similar. Thus, in triaxial compression both intact and compacted samples have similar cohesion, but the compacted sample has higher effective angle of internal friction than the compacted sample. This high angle of internal friction is characteristic of cohesionless soils like silts or fine sand rather than a soil of clay content of 40%. This may be attributed to the bonded structure.

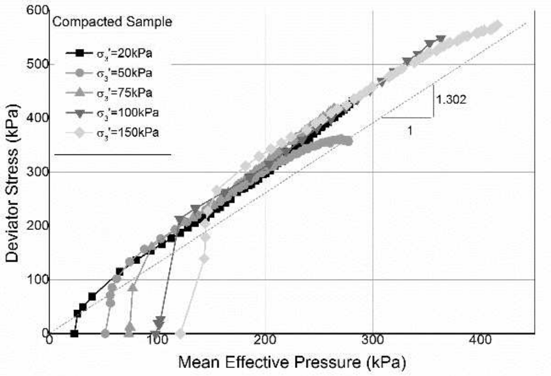

Figure 12. Stress paths for compacted lateritic soil

The stress paths all resemble heavily overconsolidated soils characterized by an initial increase in the stress ratio (q/p') to a maximum followed by a reduction as the sample moves towards the critical state. This behavior characteristic of laterites has been referred to as pseudo-overconsolidation which is said to be due to bonding rather than stress relieve (Vaughan 1988, Barksdale & Blight 2012). In this testing programme the samples did not attain clearly defined critical states as both the deviator stress and the mean effective pressure continued to increase with shearing even in the cases where the stress ratio q/p' had attained a constant value. However, from the results of PI-50-TX for the intact sample and those of PC-50-TX and PC-150-TX and the trends, the critical state stress ratios were deduced. It was observed that both the intact and compacted samples attained a similar value of the critical state stress ratio of about 1.30 which is equivalent to a critical state angle of 32°.

4 CONCLUSIONS

Drained direct shear and consolidated undrained triaxial compression tests were performed on intact and 156recompacted samples at an average dry density of 1.65 Mg/m^3 and average water content of 17.5 %, the following conclusions may be made:

In direct shear, intact and compacted samples have similar shear strengths under both saturated and unsaturated conditions.

Saturation reduces both the angle of internal friction and the cohesion intercept for both intact and compacted samples. However, whereas the reduction in the angle of internal friction is only 3 to 4 degrees, the reduction in the cohesion intercept is substantial (17 kPa) of the order of about 50 %.

Both intact and compacted samples exhibit post-peak reduction in shear stress at low confining stresses, but saturation and high stress erase this tendency.

In triaxial compression compacted samples are up to three times stiffer than intact samples and the difference in stiffness reduces with increasing confining pressure. In terms of strength in triaxial compression, both intact and compacted samples have similar cohesion of about 13 kPa but the compacted sample has higher effective angle of internal friction than the compacted sample.

Both the intact and compacted samples attained a similar critical state stress ratio of 1.30 which is equivalent to a critical state angle of 32°.

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