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Comparison between DS, DSS and Triaxial Resistance Tests in Compacted Tropical Soils in the State of Rio de Janeiro, Brazil

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Authors' contributions

This work was carried out in collaboration among all authors. All authors read and approved the final manuscript.

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ABSTRACT

Direct shear (DS), Direct simple shear (DSS) and Triaxial tests with controlled shear rates were performed in two soils from the Baixada Fluminense region, in the city of São João do Meriti – Rio de Janeiro, Brazil. The soils in question are deposited on non-compacted soft soil with the addition of Municipal Solid Waste (MSW). Both samples of compacted soil were excavated at a depth of 10.0 m, and undisturbed samples were collected. In both tests, the shear rate of 0.043 mm/min was adopted. The soil at Point 1 was characterized as a clay soil collected in a slope region and the soil at Point 2 is a sand and collected in a central region. The tests presented coherent results with probabilistic accuracy greater than 95% reliability in all three resistance tests.

Keywords: Direct simple shear; triaxial; sand; clay; tension; friction angle; cohesion; soil.

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1. INTRODUCTION

During the last years, tests that determine soil resistance have been extensively studied in order to indicate the best test to be performed on each type of soil. Associated with this, structures foundations projects have been requiring more information about the soils studied [1]. Tropical coastal regions, such as in Rio de Janeiro State, with a large mountainous cluster and steep slopes, are most often exposed to static loads (Dearman et al, 1978 e Matula et al. 1976) motivating the study of more critical resistance parameters. In this sense, the experiments that provide more information and, therefore, used in the Geotechnical area are direct shear (DS), direct simple shear (DSS) and triaxial tests.

In general, DS test has been more widely used, and the parameters obtained in the test have been more frequently used in engineering projects [2]. On the other hand, DSS test are performed only in special situations: when more information related to excess pore pressure are required [3]. The triaxial is the second test more used in geotechnical engineering to obtain resistance parameters. The analysis is carried out in different rupture plans and, therefore, provide more precise resistance parameters results (Casagrande and Hirschfeld, 1960).

The main advantage of the direct simple shear (DSS) test in relation to the direct shear test (DS) is that reproduces more faithfully field conditions and simplicity in relation to triaxial tests . This test has become more common in the most diverse areas of study, especially for the determination of resistance parameters for slope stability studies.

In the studied region (Logística Sendas), the soil is very clayey and sandy, both lateritic, as showed by Ramos [4]. According to Marsal et al. (1976) clay percentages greater or equal to 30% already influence in a determinant way in the properties of the materials. Since the soil has sufficient percentage of clay to govern the its behavior as a whole it is called a clay soil. In the same way, sand percentages greater or equal to 30% have the same behavior. Therefore, both types of experiments are needed to obtain more critical resistance parameters.

Moreover, in this region the soil was already compacted with 20 ton using a Vibrating Single Drum Roller. Further, Mori [5] states that saprolite soils, when excavated and compacted in the field, still maintain much of their structure intact, whereas in laboratory tests, the initial matrix destruction is quite intense. That is, compacted saprolite soils have even more complex structures than those presented in homogeneous compacted soils. According to genealogical origin of tropical soils of the region, the soils studied are a homogeneous lateritic soil Vargas [6] and Vaz [7].

This study has as main focus that comprehends determination of tropical compacted soil resistance parameters, from the Rio de Janeiro state, through geotechnical tests, especially, direct shear (DS) test, direct simple shear (DSS) and the triaxial. These techniques will bring a better understanding of the rupture and movement mechanism of the slope, as well as the evaluation of the criteria adopted. Al tests were done at Geotechnics Laboratory – UFRJ.

2. MATERIALS AND METHODS

2.1 Sample Collection

The experimental ground of study was established in 2010 to study the construction of landfills on layers of non-compacted soft soils with the addition of Municipal Solid Waste (MSW), a common environmental problem throughout the country (Mahler, 2018). Moreover, the region where the samples were collected is in São João do Meriti, located in the Metropolitan Region of Rio de Janeiro (22°47'40.5"S 43°21'05.9"W).

Two undisturbed samples were collected in the field. Point 1 in the slope Region and Point 2 in the Central Region of the development. The samples were 25x25x25 centimeters in size. They were rapidly paraffined and protected, in order to not lose humidity. Therefore, they would not undergo changes during transport to the laboratory, where they were placed in a moisture chamber. For safety factors, Point 1 was chosen for appearing to be more clayey and Point 2 for having higher settlements, as previous mention by Deere e Patton [8].

2.2 Direct Shear

The DS test was performed in accordance with standard ASTM 2974 soil procedures, to determine the shear stress [2]. Six tests were carried out at two different points in saturated state, with initial tensions of 75 kPa, 150 kPa and 300 kPa for both points. The sample cell has 36 cm² in area (6 cm x 6 cm), is horizontally split and secured by two screws. The adopted shear velocity was 0.043 mm.min-1. The DS tests were carried out with flooding for at least one night.

2.3 Direct Simple Shear

The equipment used was Geocomp's Shear Trac-II-DSS.. A detailed description of the DSS test was given in the classic work by Bjerrum and Landva (1966). Six tests were carried out at two different points in saturated state, with initial tensions of 75 kPa, 150 kPa and 300 kPa for both Points. The sample cell has 31.73 cm² in area (diameter 6.36 cm). The specimen was sheared in drained condition at constant volume and the applied deformation velocity pre-defined as (0.043 mm/min), given the suitability of the velocity for the direct Shear Test (DS). The DSS tests were carried out with flooding for at least one night.

2.4 Triaxial

A detailed description of the triaxial equipment used in this work was given by Head [9]. The triaxial tests were performed in accordance with standard BS 1377:8 [10].

Eighteen tests were carried out at two different points in isotropical drained (CID) and undrained (CIU) consolidated samples. The initial strengths were 25, 50, 100 and 440 kPa for CIU non-percolated tests from Point 2 and 25 and 100 kPa for percolated in the same Point. For the CID analysis the initial strength was 30, 45, 60 and 80 kPa. In the Point 1, the initial strengths were 50, 100, 200 and 300 kPa for CIU and CID. Tensions have been adapted to facilitate comparison with other techniques. The sample cell was molded in the laboratory. The base diameter ranged from 4.78 to 5.10 cm and height from 10.20 to 11.08 cm, with a total sample area ranging from 17.95 to 20.43 cm². The specimen was sheared at a predefined applied strain velocity (0.043 mm/min). Several drainage cycles were performed during the sample saturation period and the sample was considered saturated when it assumed Skempton B values above 95%.

3. RESULTS AND DISCUSSION

As a general principle, it should be taken into account that the shear strength of purely granular soils is basically a phenomenon of friction, and therefore, it predominantly depends on the normal pressure to the shear plane. In the case of cohesive soils, in addition to friction, cohesion plays an important role in resistance.

In resistance tests, the normal pressure σ is varied, measuring the respective shear stress failure [10]. Thus, it is possible to establish the Mohr envelope for a given soil, from points (σ, t) obtained in tests.

3.1 Direct Shear

Fig. 1-a shows the soil behavior at different stresses for horizontal displacement. As can be seen, the behavior had stabilization of the deformations prior to the conclusion of the test with constant growth values. Moreover, Figs. 1-a and 1-b show a standard behavior of clay soil for Point 1 (dotted lines) and sand soil for point 2 (solid lines). The curve stabilization at 75 kPa in Point 2 had a quickly ruptured, unlike all other analyzed samples. Further, Fig. 1-c, shows an elongation behavior in all tests. The friction and cohesion values found from 0 to 10 m deep obtained by the linearization on the Fig. 1-d are showed in Table 1. Consequently, Point 1 is clearly more clayey and Point 2 more sandy Vargas (1953, 1974) and De Mello (1972).

Table 1. Friction and cohesion values found from 0 to 10 m deep in the DS experiments

	Friction(º)	Cohesion (kPa)
Point 1	21.77	24.55
Point 2	31.14	6.47

3.2 Direct Simple Shear

Fig. 2-a shows the soil behavior at different stresses for shear strain. As can be seen, the behavior had stabilization of the deformations prior to the conclusion of the test with constant growth values. The test with 75 kPa in Point 1 shows a small decline after 15% with rapid stabilization afterwards. This didn't affect the measurement since the coefficient of determination (R^2) was 0.9997, as showed in Fig. 2-d. Figs. 2-a and 2-b show a standard behavior. Notwithstanding, only at Point 1 with 75 kPa showing a certain discrepancy, as reported before.

In all trial, Fig. 2-c shows a shortening behavior. As found in the Fig. 2, the straight lines, their friction angle and cohesion coefficients in Fig. 3 showed that Point 1 is clearly more clayey and Point 2 more sandy. Despite this, there is excessive discrepancy between DS and DSS values, as can be observed in Tables 1 and 2, respectively. In this regard, DS test have a defined failure plane, unlike the DSS test. It is possible that this distinction has made all the difference in the results. Moreover, DSS test presented extremely careful safety results, since the failure plane was not horizontal.

Table 2. Friction and cohesion values found from 0 to 10 m deep in the DSS experiments

	Friction(º)	Cohesion (kPa)
Point 1	13.84	19.82
Point 2	25.19	4.90

3.3 Triaxial

The triaxial tests use more stress points than DS and DSS due the technical standard, which requires a minimum of 12 tests [10].

Figs. 3-a and 4-a show the soil behavior at different stresses for axial strain. As can be seen, the stabilization behavior of the deformations prior to the conclusion are in constant growth values. Figs. 3-a, 3-b, 4-a and 4-b show a standard behavior with excess pore pressure Hilf (1956) and Simms and Yanful [11]. Figs. 3-c and 4-c show the soil behavior Parameter A for axial strain. As can be seen, all samples reached peak deviator stress (qmax) between 2 and 8% for Point 1 and between 4 and 8% for Point 2. The test with 100 kPa in Point 2 shows a small ascent after 8%, with rapid stabilization afterwards. Probably, there was a harder material like stone or quartz during the test that interfered with the result. Nevertheless, this didn't affect the measurement since the coefficient of determination (R^2) was 0.9974, as showed in Fig. 4-e. Figs. 3-d and 4-d reiterate the situation at Point 2 with 100 kPa, however with a few discrepancies in the other tests.

In all trial, Figs. 3-e e 4-e shows a shortening behavior. In the same way as founded in Figs. 1 and 2, the straight lines, their friction angle and cohesion coefficients in Figs. 3 and 4-e corroborate that Point 1 is more clayey and Point 2 more sandy. The friction and cohesion values found from 0 to 10 m deep obtained by the linearization on the Figs. 3-e and 4-e are showed in Table 3.

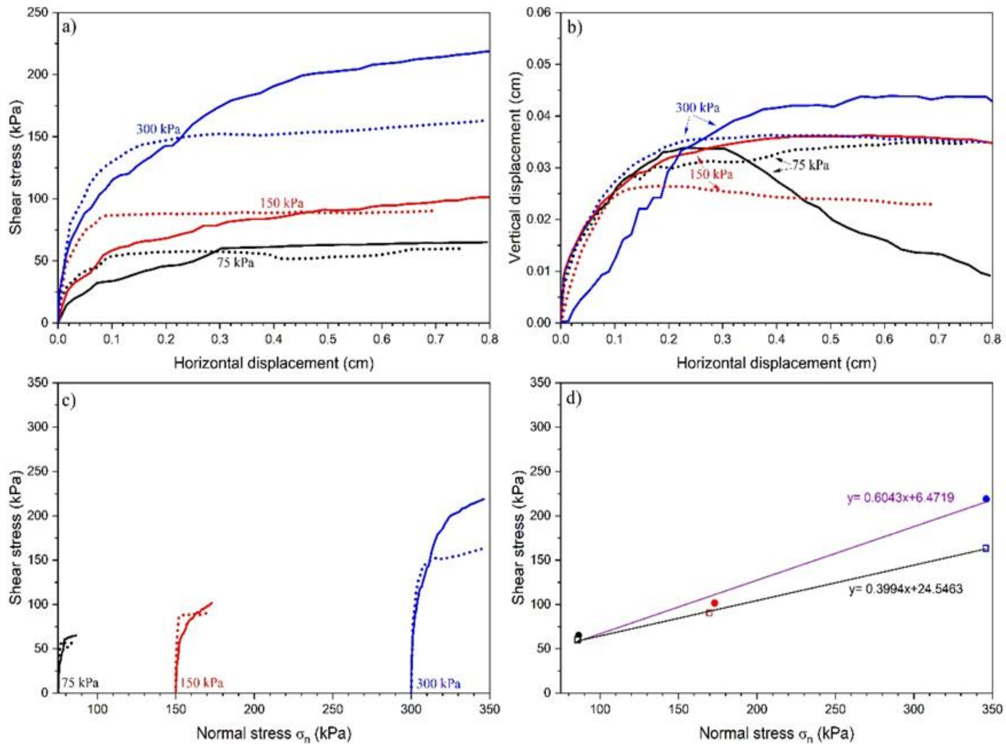

Fig. 1. (a) Shear Stress (kPa) x Horizontal Displacement (cm), (b) Vertical Displacement (cm) x Horizontal Displacement (cm), (c) Shear Stress (kPa) x Normal Stress (kPa) and (d) Mohr´s wrap in the point 1 (dotted line) and 2 (solid line). The applied stresses of 75, 150 and 300 kPa are represented by black, red and blue, respectively

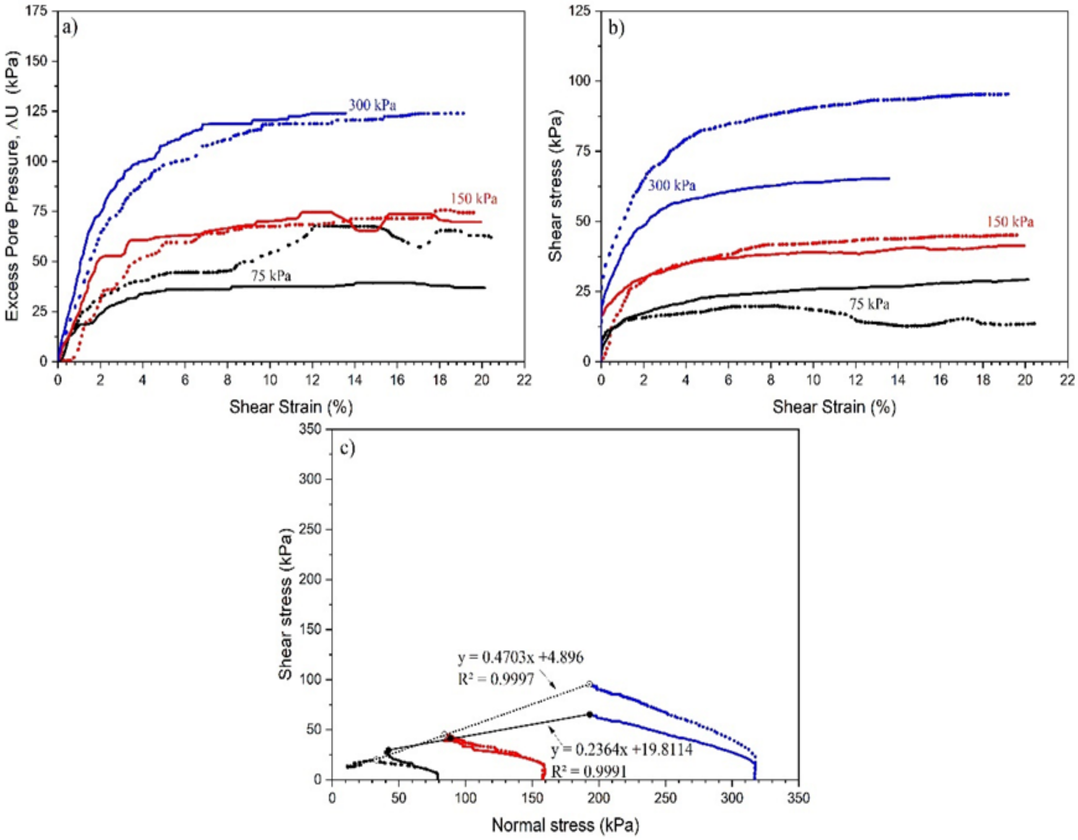

Fig. 2. (a) Excess Pore Pressure (kPa) x Shear strain (%), (b) Shear Stress (kPa) x Shear Strain (%), (c) Shear Stress (kPa) x Normal Stress (kPa) and Mohr´s wrap in the point 1 (dotted line) and 2 (solid line). The applied stresses of 75, 150 and 300 kPa are represented by black, red and blue, respectively

Table 3. And cohesion values found from 0 to 10 m deep in the Triaxial-CIU experiments.

	Friction Triaxial CID (°)	Friction Triaxial CIU (°)	Cohesion Triaxial CID (kPa)	Cohesion Triaxial CIU (kPa)
Point 1	27,02	22,46	9,11	15.92
Point 2	32.75	26.97	0.15	0

Figs. 5-a and 6-a shows the soil behavior at different stresses for axial strain. As can be seen, the stabilization behavior of the deformations prior to the conclusion of the test was constant with growth values. Figs. 5-a, 5-b, 6-a and 6-b show a standard behavior with volumetric strain. In all trial, Figs. 6-c e 7-c show a shortening behavior. As found in the Figs. 1, 2, 3 and 4 the straight lines, their friction angle and cohesion coefficients in Figs. 5-c and 6-c showed that Point 1 is more clayey and soil 2 more sandy.

3.4 Comparison of Results

The samples used in the different cutting tests show reduced variability in terms in physical state and identification characteristics. Therefore, it is possible to joint analysis of the results obtained.

Table 4 presents the results obtained in this work, together with results found in the literature to facilitate the comparison between the data. According to Seed et al. [12] and Casagrande and Hirschfeld (1960), another way of obtaining dispersion is to print large shear stresses on the soil mass. According to the authors, the boundary region of two types of structure above is for clayey soils around the optimal moisture, for energy levels compatible with the Normal Proctor. Therefore, information related to compaction, equipment used, soil type and adopted shear speed in the resistance test are summarized in this table.

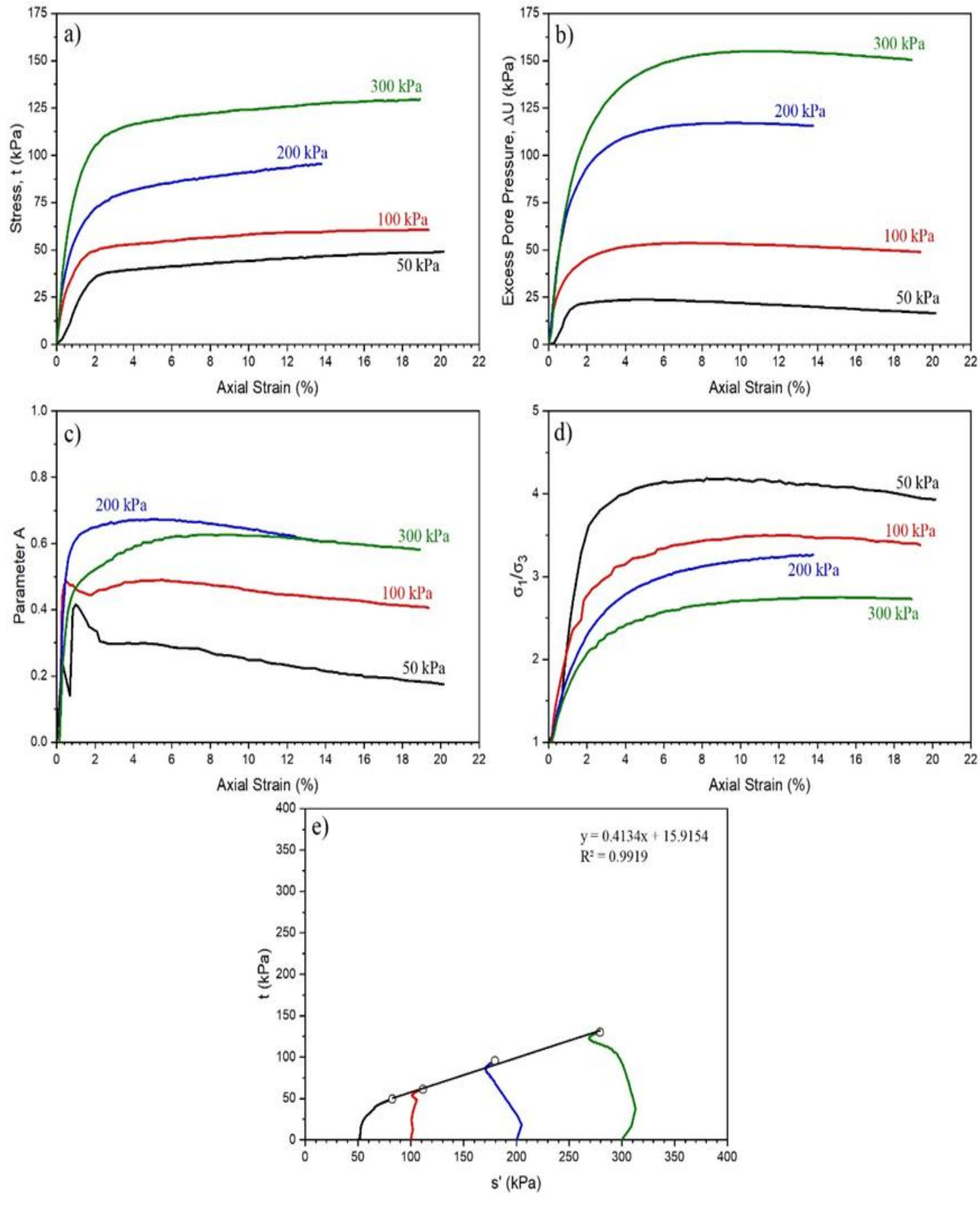

Fig. 3. CIU Point 1 (a) Stress, t (kPa) x Axial Strain (%), (b) Excess Pore Pressure (kPa) x Axial strain (%), (c) Parameter A x Axial Strain (%), (d) Analyse σ1/ σ3 (kPa) x Axial Strain (%) and (e) Mohr´s wrap in the point 1. The applied stresses of 50, 100, 200 and 300 kPa are represented by black, red, blue and green respectively

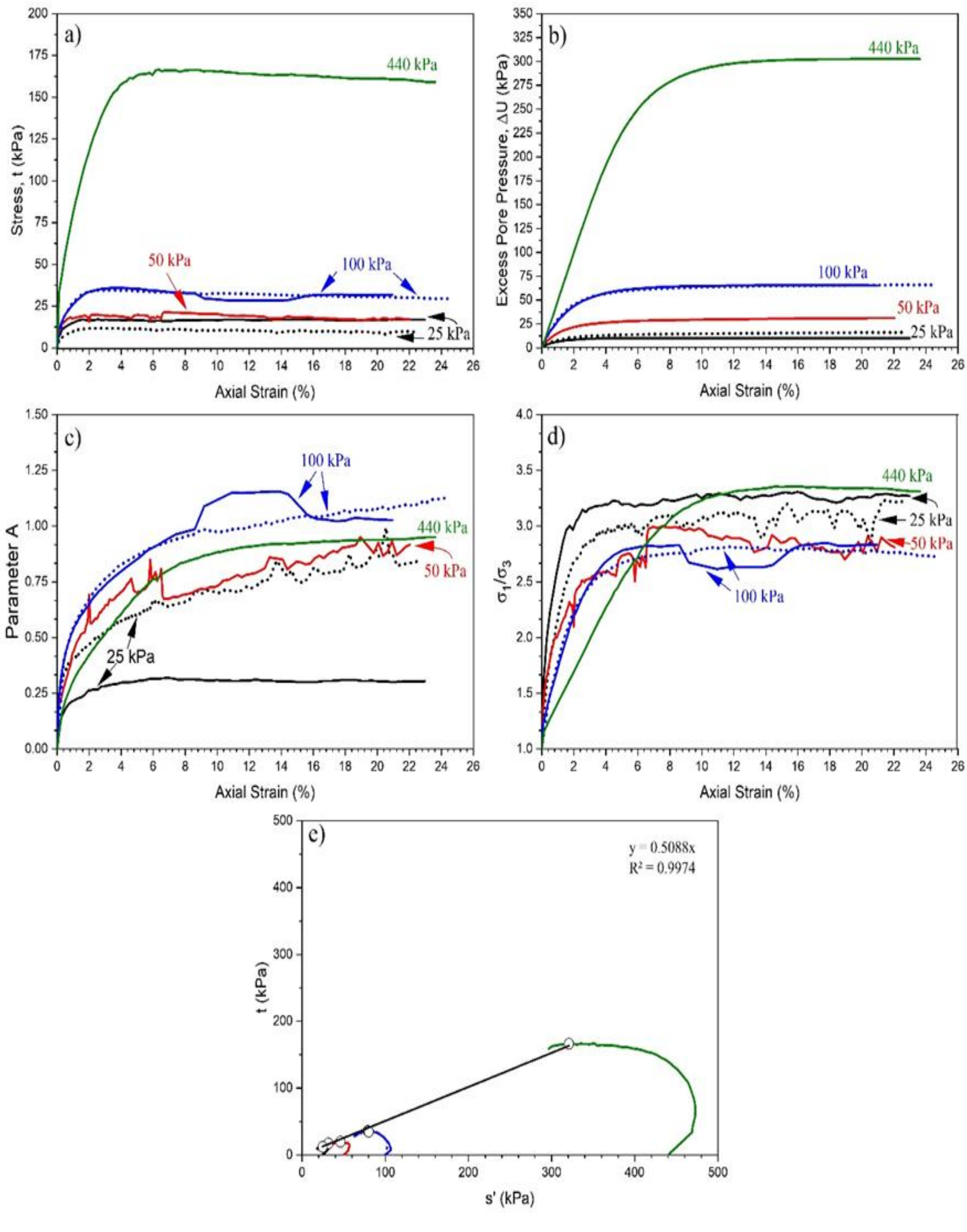

Fig. 4. CIU Point 2 (a) Stress, t (kPa) x Axial Strain (%), (b) Excess Pore Pressure (kPa) x Axial strain (%), (c) Parameter A x Axial Strain (%), (d) Analyse σ1/ σ3 (kPa) x Axial Strain (%) and (e) Mohr´s wrap in the point 2. The applied stresses of 25, 50, 100 and 440 kPa are represented by black, red, blue and green, respectively. The solid line represents non-percolated samples and the dotted line percolated samples

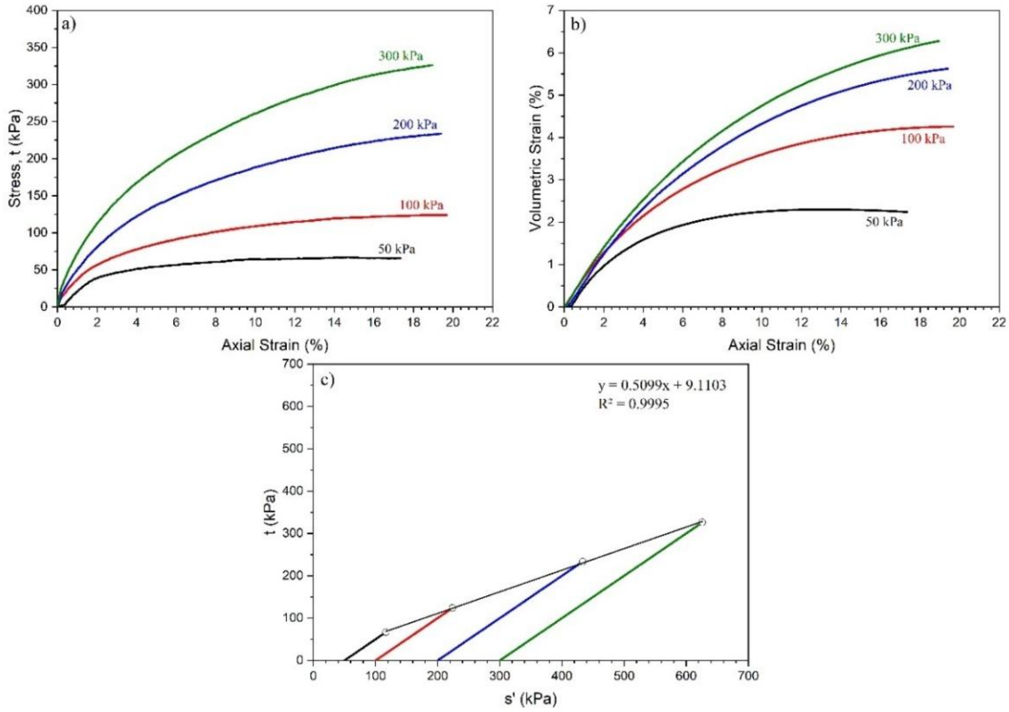

Fig. 5. CID Point 1 (a) Stress, t (kPa) x Axial Strain (%), (b) Volumetric Strain (%) x Axial strain (%), (c) t (kPa) x s´ (kPa), The applied stresses of 50, 100, 200 and 300 are represented by black, red, blue and green, respectively

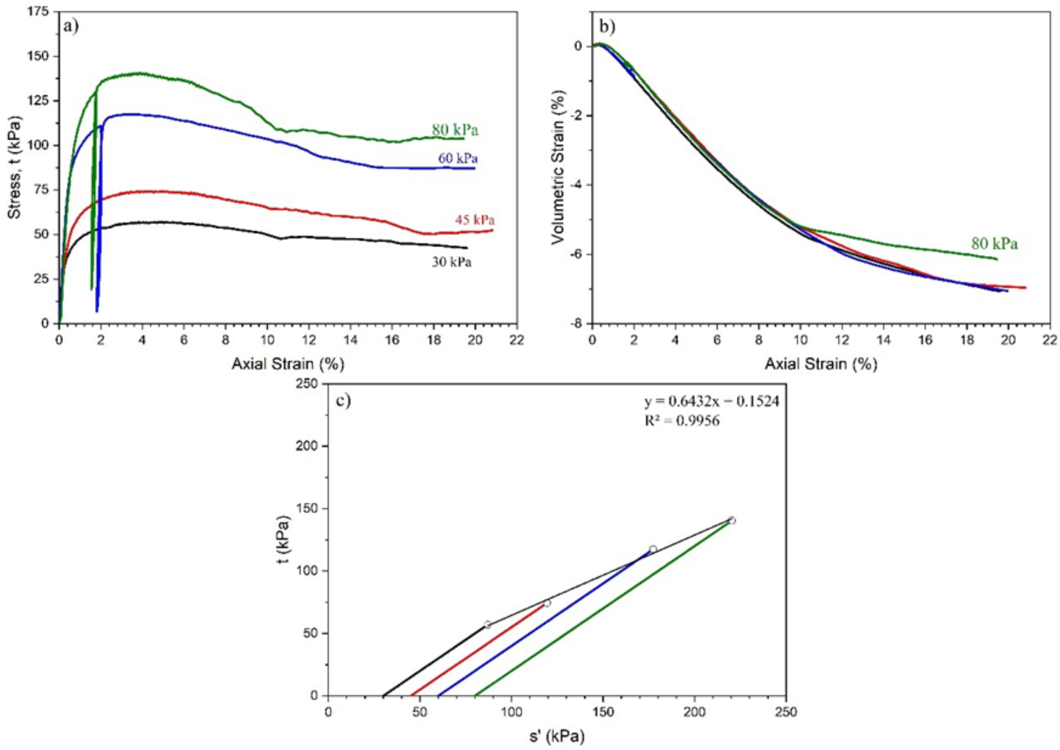

Fig. 6. CID Point 2 (a) Stress, t (kPa) x Axial Strain (%), (b) Volumetric Strain (%) x Axial strain (%), (c) t (kPa) x s´ (kPa), The applied stresses of 30, 45, 60 and 80 kPa are represented by black, red, blue and green, respectively

Table 4. Comparison between results found in DSS, Triaxial and DS experiments of this work as well as those found in the literature

	Friction Angle (°)	Cohesion Intercept (kPa)	Soil type	Compaction and equipment used	Adopted shear speed (mm/min)
Authors 1 (CID) – Triaxial	27.0º	9.1	Lateritic inorganic compact, high plasticity sandy clay	20 ton Vibrating Single Drum Roller (normal proctor)	0.043
Authors 1 (CIU) – Triaxial	22.5	15.9	Lateritic inorganic compact, high plasticity sandy clay	20 ton Vibrating Single Drum Roller (normal proctor)	0.043
Authors 2 (CID) - Triaxial	32.8º	0.15	Lateritic inorganic compact, sandy clay of medium plasticity	20 ton Vibrating Single Drum Roller (normal proctor)	0.043
Authors 2 (CIU) – Triaxial	26.97	0	Lateritic inorganic compact, sandy clay of medium plasticity	20 ton Vibrating Single Drum Roller (normal proctor)	0.043
Schroder ([15] – (CID) - DS	37.7º	145	Lateritic inorganic compact, high plasticity sandy clay	H Hydraulic jack 12 ton (normal proctor)	0.2
Schroder [15] – (CID) - Triaxial	31º	180	Lateritic inorganic compact, high plasticity sandy clay	H Hydraulic jack 12 ton (normal proctor)	0.2
LG ' (Normal Proctor) – Strong (2011) – (CID) - DS	24.4º	95.6	Lateritic organic compact, high plasticity clay	Bulldozer 14 ton (normal proctor)	0.025
LG' (Normal Proctor) – Strong (2011) – (CID) - Triaxial	37 º	65	Lateritic organic compact, high plasticity clay	Bulldozer 14 ton (normal proctor)	0.025
Mello (1946) – CID - DS	29.8º	254	Compressed organic lateritic blue clay high plasticity	Bulldozer 14 ton (normal proctor)	0.025
Authors 1 – DS	21.8 º	24.6	Lateritic inorganic compact, high plasticity sandy clay	20 ton Vibrating Single Drum Roller (normal proctor)	0.043
Authors 2 (2018)– DS	31.1º	6.5	Lateritic inorganic compact, sandy clay of medium plasticity	20 ton Vibrating Single Drum Roller (normal proctor)	0.043
Authors 1 (2018) (CIU) – DSS	13.8º	19.8	Lateritic inorganic compact, high plasticity sandy clay	20 ton Vibrating Single Drum Roller (normal proctor)	0.043
Authors 2 (CIU) - DSS	25.2º	4,9	Lateritic inorganic compact, sandy clay of medium plasticity	20 ton Vibrating Single Drum Roller (normal proctor)	0.043
Schroder [15] – (CIU) – DS	27.7º	20	Lateritic inorganic compact, high plasticity sandy clay	H Hydraulic jack 12 ton (normal proctor)	0.2
Quental (1986) – (CIU) – Triaxial – Normally Dense	33.1 º	6.1	Lateritic organic compact, high plasticity clay	Bulldozer 14 ton (normal proctor)	0.025
Mello (1946) – CIU - DS	28.4 º	130	Compressed organic lateritic blue clay high plasticity	Bulldozer 14 ton (normal proctor)	0.025
Dib (1985b) – CIU - Triaxial	26 º	25	Organic compacted clayey silt	Bulldozer 14 ton (normal proctor)	0.025

It can be observed based on resistance parameters obtained in this work (Table 4) that the DS test is reasonably equal to the values presented in the Triaxial CID and greater than the Triaxial CIU. The values in DSS are the smaller one because of the predefined orientation plane as previously discussed. Table 4 also includes other tests carried out in the academic literature. It was expected that the geotechnical parameters of friction angle and cohesion intercept were much lower in direct shear tests (DSS) than those obtained in direct shear (DS) due to the predefined orientation plane of the Direct Shear test. According to HANZAWA et al. (2007), the results obtained through the direct simple shear (DSS) tests represent more faithfully the conditions in the field than the direct shear (DS) results.

The tests obtained in this work had results very consistent with the literature. The Triaxial-CIU results obtained by Coutinho e Bello [13], the soil was especially similar in the characteristics with the soil of Point 2.

For the Triaxial-CID, the results were closer to Fortes [14] in the Point 1. The soil worked by Fortes [14] is the closest geologically with Point 1, since they are both tropical soils. Probably, the difference between the tests by Mello (1946) and Schroder [15] is due to the geological formation of the studied soil, since the last two are temperate soils [16-25].

4. CONCLUSION

The study carried out throughout this article allows us to conclude that the soil presents resistance, in terms of effective stresses, ranging from 21 to 33º. This result is much higher than what would be expected given its plasticity characteristics. However, it is perfectly justified by the predominantly silty granulometric composition with sand in the Central Part of the Landfill and more clay in the part of the slope of the Landfill up to 10m deep.

It is visible that for the specific situation of this landfill, the DS and Triaxial tests fit better, with the DSS being very cautious.

It is understood that the adoption of soaked Direct Shear tests, at different points of the landfill, is the best solution, as they present satisfactory parameters regarding the error requirement and qualify the enterprise data.

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COMPETING INTERESTS

Authors have declared that no competing interests exist.

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