

# Determination of Resistance Parameters of Contaminated Compacted Tropical Soils in the State of Rio de Janeiro

Fernando B. Mainier<sup>1</sup>, Viktor L. F. S. Ramos<sup>2</sup>, Claudio F. Mahler<sup>3</sup>

Escola de Engenharia, Universidade Federal Fluminense, Niterói, RJ, Brazil.

<sup>1</sup>fmainier@uol.com.br; <sup>2</sup>viktorlabuto@poli.ufjf.br;

Coppe, Universidade Federal do Rio de Janeiro, RJ, Brazil

<sup>3</sup>mahler@coc.ufjf.br

**Abstract** — Direct shear (DS) 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 first soil characterized as Inorganic Clay of high plasticity (CH) was located in a hillside region. The second soil characterized as medium plasticity clay (CL) was located in a central region of the studied area with a slight slope. 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 1.0 m, and undisturbed samples were collected. In addition to the previously mentioned tests, oedometric tests and soil physical classification tests were carried out to evaluate geotechnical parameters. In both tests the shear rate of 0.043 mm / min was adopted. The tests presented coherent results with probabilistic accuracy greater than 95% inreliability compared to three tests with compacted tropical soils.

**Keywords** —direct shear (DS), clay, tension, friction angle, cohesion, liquid limit, plasticity limit, soil.

## I. 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, projects of foundations of structures have been requiring more information about the soils studied.

In particular, tropical coastal regions, such as Rio de Janeiro State, with a large mountainous cluster and steep slopes, are most often exposed to static loads, motivated the study of more critical resistance parameters.

In general, the direct shear (DS) test has been more widely used, and the parameters obtained in the tests have been more frequently used in engineering projects [1].

The research has as main focus the study of tropical soils through particular geotechnical tests, especially the

directshear (DS) test.

The objective is the study of resistance parameters of a tropical compacted soil from the Rio de Janeiro state. Thus, a better understanding of the mechanism of rupture and movement of the slope is sought, as well as the evaluation of the criteria adopted. The experimental results were compared in order to investigate the differences between the parameters of shear strength of authors, DS tests, and others authors tests in different normal effective shear and failure criteria. It is also the basis of the research to describe methods, materials and laboratory tests that could assist in the determination of basic geotechnical parameters and resistance.

## II. CONTEXTUALISATION

### 2.1 Compacted soil structure

According to Seed et al. [2] and Mitchell et al. [3] the soil is the result of rock degradation, transport, mineralogical composition, electric forces between particles and other forces that acted during the history of the soil and interaction between particle arrangements.

It is thus called, by clay soil, the soil with sufficient percentage of clay to govern the behavior of the soil as a whole. According to Marsal et al. [4] clay percentages greater or equal to 30% already influence in a determinant way in the properties of the materials.

The clay fraction is composed of particles smaller than two microns and is called the active component of the soil, while the silt, sand and gravel portion is often referred to as the inert component. This is due to the fact that the clay has physical-chemical phenomena these, not present in the granular fraction.

In clay soils, if the amount of water is increased, allowing the double layers to form, the predominant efforts will be repulsion. Thus, the particles tend to disperse, parallel to each other.

According to Seed et al. [2] 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.

The compaction moisture is thus a factor of great importance in the formation of the soil structure, being dry molded soils of greater capacity of support than those molded with moisture.

However, soil structure is not only defined by compaction moisture, but also by the type of effort employed.

Casagrande and Hirschfeld [5] conducted compaction curve studies for soils compaction. The studied tropical compacted soil presented good parameters of energy levels.

Marsal et al. [4] used the terms open and occluded, to designate the occurrence of air and water, and their interactions with the solid phase.

In the occluded state, characteristic of compacted soils in the moist branch of the compaction curve, the air is present in bubbles, surrounded by the liquid phase, and therefore there is no continuity of the gas phase.

In the open state, common to compressed soils with below-optimal moistures, the gas is fully connected to the atmosphere.

Another item to be approached regarding soil structure is the presence of saprolite soils.

These soils are more easily destroyed by mechanical manipulation or action, due to the high heterogeneity of their characteristics.

Mori [6] 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.

## 2.2 Location

The experimental ground of study is in São João do Meriti, located in the Metropolitan Region of Rio de Janeiro (Brazil).

This experimental ground 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. A comprehensive research project was started in 2017, under the supervision of Professor Claudio Fernando Mahler, from COPPE /UFRJ.

Figure 1 below shows the location:

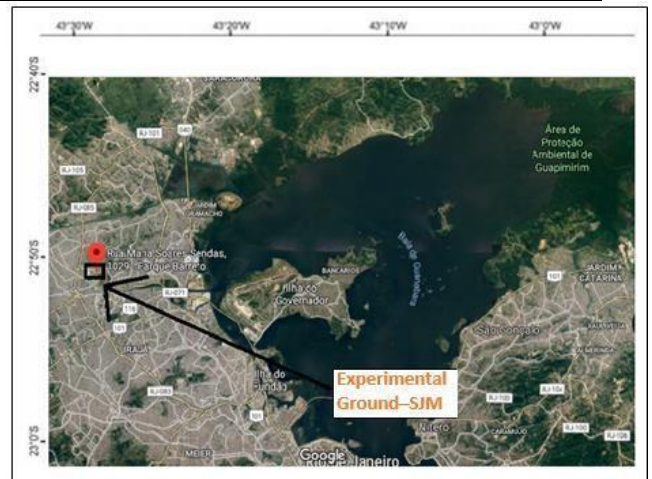


Fig. 1: Location of ground of study (Google Maps)

## III. MATERIALS AND METHODS

### 3.1 Undisturbed block of soil

For the field removal of the undisturbed block 5 campaigns were carried out to collect the samples in block. First, it was necessary to excavate 1.0 m depth from an area of 1.0 m x 1.0 m. The excavation was performed manually. Subsequently it was molded to the selected size (30 cm x 30 cm x 30 cm) so that it could be packed in the vehicle used. For the correct mold were used string, knife, wire saw and steel cables. Although it was not always possible to cut the block at the desired location due to the presence of pebbles in the soil, it was generally sought to adopt the central plane as the base axis. During the molding, the ruler was used for the correct dimensioning of the block.

The block was packed with the smallest changes possible. For this purpose a layer of PVC film paper was used, followed by two layers of aluminum foil and one more of a canvas fabric.

Immediately after the procedure, step 3 was started, resin melting.

Step 4 was then followed: filling the external surface area of the fabric with a resin in the liquid state with the aid of a shell and then naming the block using label and pilot, which will be glued to the resin. Immediately after its solidification in ambient state the block is then laid in a wooden box with the desired dimensions and filled in the gaps with sawdust. This same block is then stored in a polypropylene box for better transportation, and then stored in the vehicle.

Figure 2 below shows the collect of undisturbed block of soil:



Fig. 2: Undisturbed block of soil

### 3.2 Soil Physical Classification

The test is initiated with a visual tact analysis in order to pre-identify the soil to be worked on.

In this work a careful analysis of the grains was made, in order to define how the soil sample would be chosen and, thus, from the current norms, to obtain data that allows its characterization and use by means of curves.

The materials and equipment used in this phase are: sieves, metal trays, mortar, clock, scales, greenhouse, beaker, dispersing apparatus, glass beakers, hydrometer, thermometer, mechanical stirrers, metal brushes, glass stick, replaceable metal propellers and cup with metal baffles, vacuum pump with registers, vacuum gauge and connections, capable of applying a vacuum of 8 kPa, funnel, flexible blade spatula, check height dropping shell, steel ball with 8 mm diameter, suitable containers, glass plates and porcelain capsule with 120 mm diameter.

Figure 3 below shows soil sifting:



Fig.3: Soil Sifting

### 3.3 Oedometric Test

The oedometric test followed the following steps:

- Molding of the specimen with the aid of a bevelled

- ring to reduce disturbances in the sample during cutting;
- Placement of the ring with the ground on the rigid cell which should contain a porous stone at its base to allow drainage of the water from the specimen;
- Assembly of top plate, which shall also contain a porous stone;
- Adjustment of the displacement meter for vertical displacement measurements;
- Application of vertical loads;

For this test the first loading stage was 0.031 (kg/cm<sup>2</sup>), with 8 loading stages which were applied to the sample. The loading variation was 0.031; 0.062; 0.125; 0.250; 0.500; 1,000; 2,000 and 4,000 (kg/cm<sup>2</sup>), and three stages of unloading 2,000; 1,000; 0.500 (kg/cm<sup>2</sup>). Each charging stage should last for 24 hours. During the execution of each loading stage, vertical compression measurements of the sample are made as a function of time, for the times of 0,0.1,0.25,0.50,1,2,4,8,15,30,60,120 and 240 minutes. With these data the density curves are constructed, that is, displacement versus time. With the aid of these curves, the coefficient of soil densification is determined by a process found in the literature, the Casagrande method (log t scale)

With the development of the consolidation process, the pore-pressures dissipate in the sample. With the values of deformation, at the end of each loading stage, a curve of the effective tension versus the deformation produced by the increase of this tension is constructed. This curve can be presented in several ways, such as vertical effective tension versus pre-consolidation coefficient.

Figure 4 below shows the test, while Figure 5 shows the sample from point 1 in a saturated state.



Fig. 4: Oedometric Test

Figure 7 below shows the granulometric (particle size distribution) curves of point 1 and point 2 with the respective material composition present in the soil. Figure 8 shows the Atterberg Limits of point 1 and point 2.



Fig 5: Sample of point 1 - Saturated state

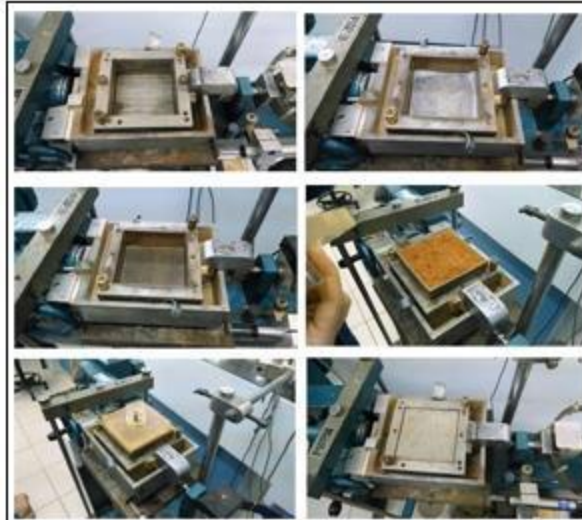


Fig. 6: Direct Shear molding

### 3.4 Direct Shear(DS)

The direct shear test is used to determine the shear strength parameters of the soils. This test allows the study of the resistance in a predetermined plane of rupture. The specimen is formed from an undisturbed specimen. Initially, the top of the undisturbed sample is set. Placing the nozzle (metal mold) on it, pressing it lightly and forcing it to penetrate the sample. As the nozzle is introduced, the soil around it is roughly trimmed with a small knife. This operation must continue until the soil appears just above the metal mold. Then the top of the spout is scraped off and the base is highlighted, scraping also on the other side. Once this is done, the sample will be ready for the direct shear (DS) test. Figure 6, below, shows a Direct Shear (DS) molding.

## IV. RESULTS AND DISCUSSION

### 4.1 Soil Physical Classification

#### 4.1.1 Point 1 and Point 2

The soil of point 1 can be characterized as an inorganic clay soil of high plasticity (CH). On the other hand, the soil of point 2 can be characterized as a medium plasticity (CL) inorganic clay soil.

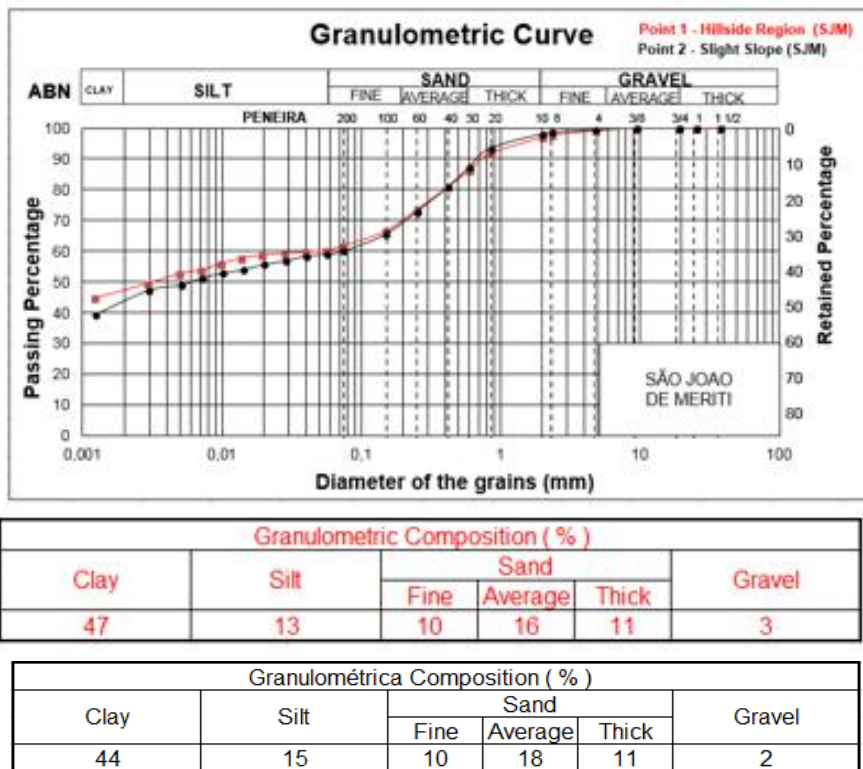


Fig. 7: Particle-Size Distribution Curves

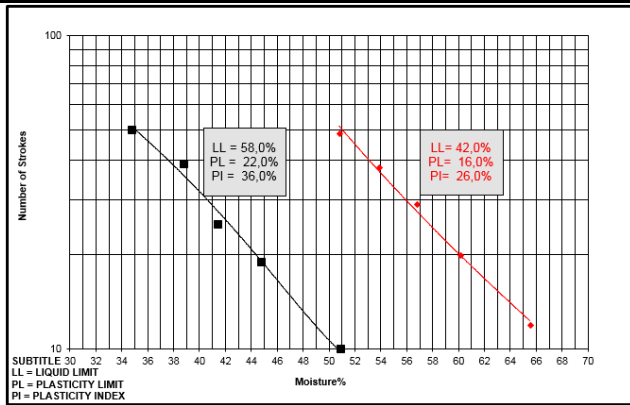
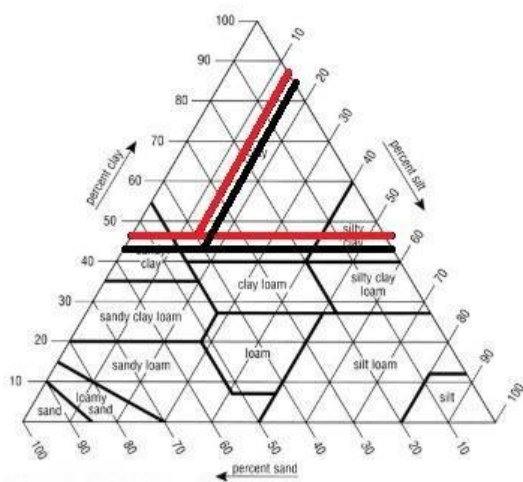


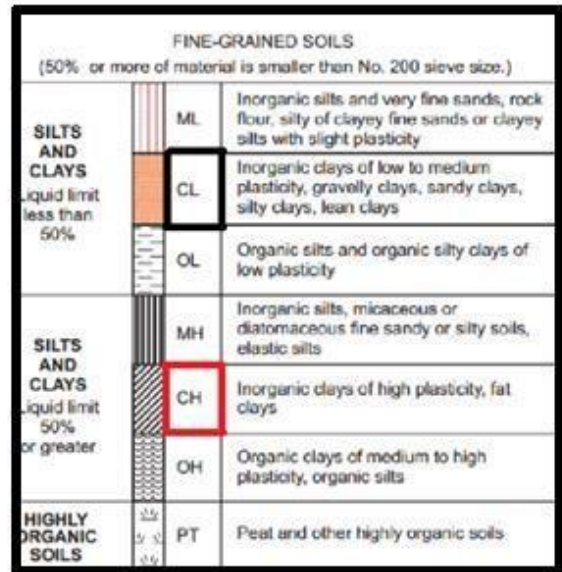
Fig. 8: Atterberg Limits

Figure 8 below shows the Feret Triangle used by the U.S. Department of Agriculture Soil Texture that was used to determine the soil type of point 1 and point 2. Figure 9 shows the Unified Soil Classification System (USCS) created by Civil Engineering Arthur Casagrande that was used to determine the soil characteristics.



Point 1 - Slight Slope (SJM) Point 2 - Hillside Region (SJM)

Fig. 9: Feret Triangle



Point 1 - Slight Slope (SJM) Point 2 - Hillside Region (SJM)

Fig. 10: Unified Soil Classification System (USCS)

#### 4.2 Oedometric Test

The consolidation coefficients presented to the soil varied from  $34.61 \times 10^{-3}$  to  $44.24 \times 10^{-3}$  cm<sup>2</sup> / s, according to table 1 with average results (Point 1 and Point 2) and relative standard deviation.

Soil moisture ranged from 16 to 19% on days of field trials before to soil consolidation.

The initial void indices ranged from 0.59 to 0.86 depending on the soil removal local, also varying according to the effective stresses applied according to the graphs.

Figure 11 and Figure 12 below shows the variation of the void indices by the applied vertical tension for Point 1 and for Point 2. Table 1 and Table 2 shows the initial soil moistures in the field tests (different days) and after the oedometric tests. Table 3 shows the values of the consolidation coefficients.

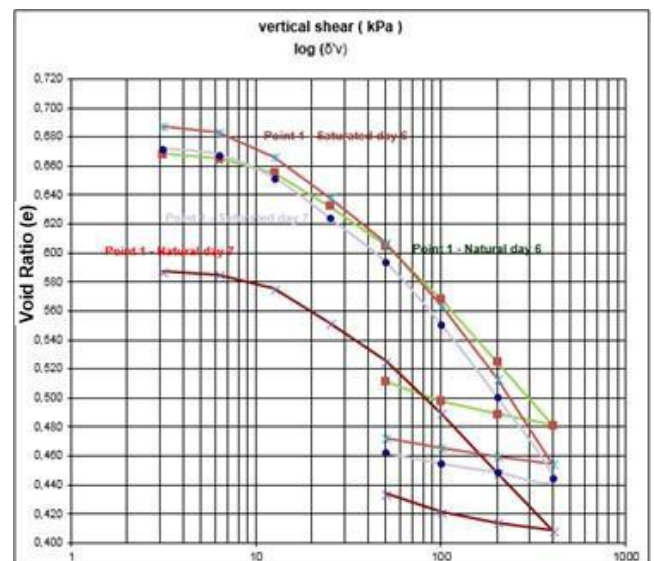


Fig. 11: Void Ratio (e) x Vertical Shear (kPa) –

Point1

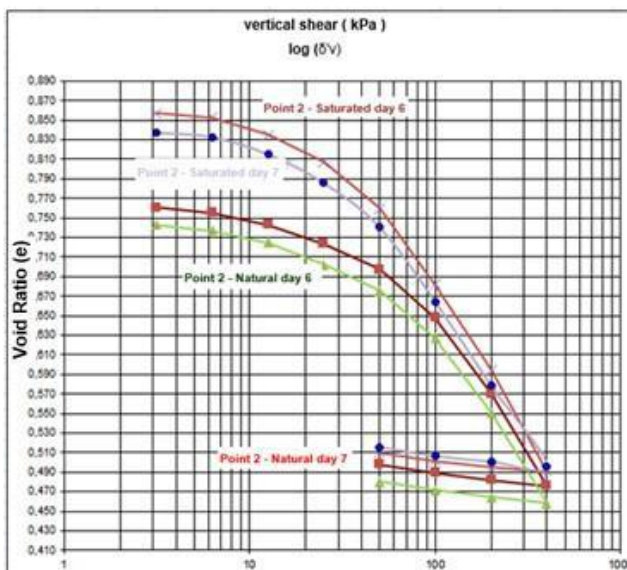


Fig. 12: Void Ratio (e) x Vertical Shear (kPa) – Point2

Table 1: Moisture initial and final of the samples –Point 1

N°	w <sub>initial</sub> (%)	w <sub>final</sub> (%)
Sample Point 1 (Natural)	16,83	12,62
Sample Point 1 (Natural)	16,82	12,62
Sample Point 1 (Saturated)	16,68	27,98
Sample Point 1 (Saturated)	16,68	27,50

Table 2: Moisture initial and final of the samples – Point 2

	w <sub>initial</sub> (%)	w
Sample Point 2 (Natural)	18,24	
Sample Point 2 (Natural)	18,23	
Sample Point 2 (Saturated)	18,57	
Sample Point 2 (Saturated)	18,47	

Table.3: Consolidation Coefficient

	Consolidation Coefficient (average) x 10 <sup>-3</sup>	Standard Deviation x 10 <sup>-3</sup>
Point 1 (Natural)	32,12 cm <sup>2</sup> /s	13,45 cm <sup>2</sup> /s
Point 1 (Saturated)	37,09 cm <sup>2</sup> /s	10,14 cm <sup>2</sup> /s
Average Point 1	34,61 cm <sup>2</sup> /s	11,91 cm <sup>2</sup> /s
Point 2 (Natural)	47,82 cm <sup>2</sup> /s	25,69 cm <sup>2</sup> /s
Point 2 (Saturated)	40,66 cm <sup>2</sup> /s	7,37 cm <sup>2</sup> /s
Average Point 2	44,24 cm <sup>2</sup> /s	18,90 cm <sup>2</sup> /s

4.3 Direct Shear(DS)

4.3.1 Point 1 and Point 2(Natural)

With the values of normal tension and shear stress tension, referring to the peak points of each test, the

Mohr-Coulomb failure criterion is constructed. From the straight line developed between the normal stress and the shear stress the values of the cohesion intercept and the friction angle of the soil are drawn.

The Mohr-Coulomb failure criterion,  $\tau$  (kPa) =  $c + tg(\Phi) * \sigma$ , is adjusted with the points below (Table 4 and Table 5):

Table 4: Normal Stress and Shear Stress – Point1

$\tau$ (Kpa)	$\sigma$ (Kpa)
154	77
182	160
253	326

Table 5: Normal Stress and Shear Stress – Point2

$\tau$ (kPa)	$\sigma$ (kPa)
118	85
201	173
381	317

From the previous points they are obtained the lines that best fit the points, providing an equation. Figure 13 below shows the lines provided.

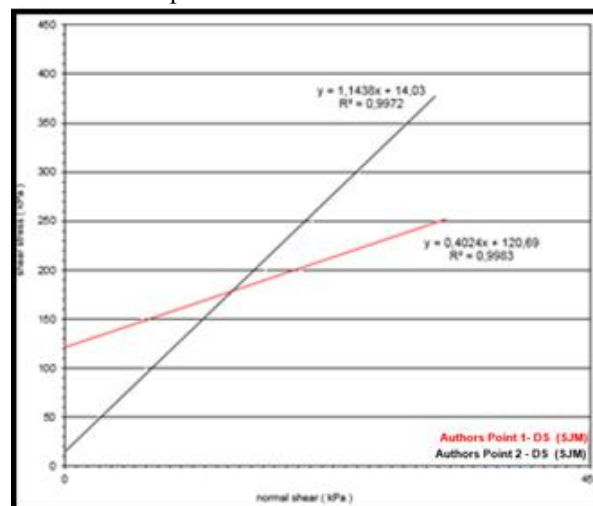


Fig. 13: Shear Stress (kPa) x Normal Shear (kPa)

From this equation the cohesion intercept and friction angle of the material are obtained. Values follow below:

$\Phi_1 = 24.33^\circ; c_1 = 120.69\text{kPa}$

$\Phi_2 = 54.16^\circ; c_2 = 14.03 \text{ kPa}$

4.3.2 Point 1 and Point 2(Saturated)

With the values of normal tension and shear stress tension, referring to the peak points of each test, the Mohr-Coulomb failure criterion is constructed. From the straight line developed between the normal stress and the shear stress the values of the cohesion intercept and the friction angle of the soil are drawn.

The Mohr-Coulomb failure criterion,  $\tau$  (kPa) =  $c + tg(\Phi) * \sigma$ , is adjusted with the points below (Table 6 and Table7):

Table.6: Normal Stress and Shear Stress – Point1

$\tau$ (kPa)	$\sigma$ (kPa)
60	86
90	170
163	346

Table 7: Normal Stress and Shear Stress – Point2

$\tau$ (kPa)	$\sigma$ (kPa)
65	86
102	173
219	346

From the previous points they are obtained the lines that best fit the points, providing an equation. Figure 14 below shows the lines provided.

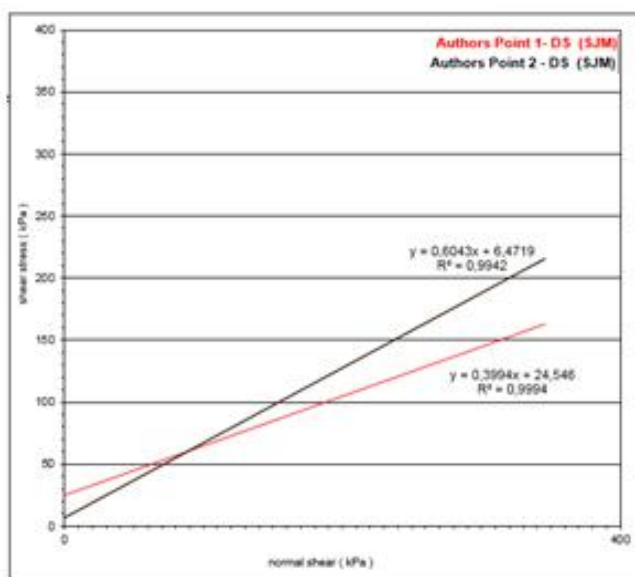


Fig. 14: Shear Stress (kPa) x Normal Shear (kPa)

From this equation the cohesion intercept and friction angle of the material are obtained. Values follow below:

$$\Phi_1 = 24,22^\circ \quad c_1 = 24,55 \text{ kPa}$$

$$\Phi_2 = 34,59^\circ \quad c_2 = 6,47 \text{ kPa}$$

### 4.3.3 Comparative Results

It is observed in Table 8 and Table 9 below, which based on the results, the curves performed by authors present an average value among the analyzes present in the literature. This assertion further supports the degree of reliability of the data, already calculated probabilistically. The only exception is the point 2 of the consolidated drained direct shear test (CD) where it is possible to visualize (black line) that the soil behaves more like a sand than a clay. This is due to the fact that the soil of point 2 has a greater percentage of sand in relation to point1.

Table.8: Friction Angle and Cohesion Intercept – Consolidated Undrained (CU)

Nº	Friction Angle (°)	Cohesion Intercept (kPa)
Authors Point 1 – DS	24,22	24,55
Authors Point 2 – DS	34,59	6,47
Stein et al. (2018) - DS	27,7	20
Coutinho et al. (2011) –Triaxial	40	0

Figure 15 and Figure 16, below, plots the different Mohr-Coulomb failure criterion for the compacted clays studied.

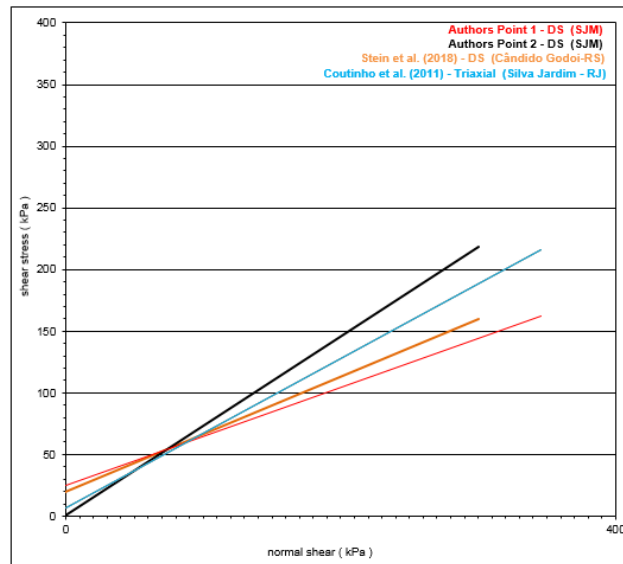


Fig. 15: Different Mohr-Coulomb failure envelope (CU)

Table 9: Friction Angle and Cohesion Intercept – Consolidated Drained (CD)

Nº	Friction Angle (°)	Cohesion Intercept (kPa)
Authors Point 1 – DS	24,33	120,69
Authors Point 2 - DS	54,16	14,63
Stein et al. (2018) - DS	37,7	145
Stein et al. (2018) - Triaxial	31	180
Merighi et al. (1987) - DS	24,40	95,60
Merighi et al. (1987) - Triaxial	37	65

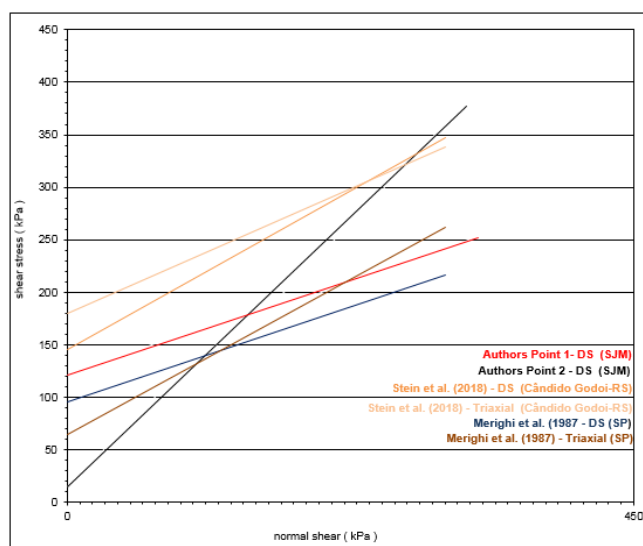


Fig. 16: Different Mohr-Coulomb failure envelope (CD)

## V. CONCLUSIONS

1987, p.29-45, Maceió, Alagoas, Brazil.

The results obtained were very satisfactory. The curves were within the range of expected values, the results being reliable both probabilistically and in the literary comparison. A possible problem, the undisturbed block of soil was well performed, and the soil was characterized as homogenous. Thus, the tests proved to be a good alternative, efficient for the determination of resistance parameters of contaminated compacted tropical soils. For this tropical soil the results were positive, but it is not possible to affirm that other tropical soils, with different proportions of sand and clay, will have the same behavior.

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