Specimen Diameter Influence on Effective Shear Strength Parameters in Triaxial Tests

Fernando Feitosa Monteiro

Master Student, Department of Hydraulic and Environmental Engineering, Federal University of Ceará, Fortaleza/CE, Brazil
e-mail: engffmonteiro@gmail.com

Leila Maria Coelho de Carvalho

Civil Engineer, Department of Civil Engineering, University of Fortaleza, Fortaleza/CE, Brazil. e-mail:leilamaria.carvalho@hotmail.com

Alfran Sampaio Moura

DSc, Professor at the Department of Hydraulic and Environmental Engineering, Federal University of Ceará, Fortaleza/CE, Brazil
e-mail: alfransampaio@gmail.com

Marcos Fábio Porto de Aguiar

DSc, Professor at the Federal Institute of Ceará (IFCE) / University of Fortaleza,/CE, Brazil. e-mail:marcosfpa@hotmail.com

Icaro Rodrigues Marques

Engineering Student, Department of Civil Engineering, Federal University of Ceará, Fortaleza/CE, Brazil. e-mail: icarormarques@hotmail.com

Yago Machado Pereira de Matos

Master Student, Department of Hydraulic and Environmental Engineering, Federal University of Ceará, Fortaleza/CE, Brazil
e-mail:yago_mpm@hotmail.com

ABSTRACT

This paper presents a comparison between effective shear strength parameters measured with triaxial tests on remolded sand soils specimens with 50mm e 38mmdiameters. On this study, twenty-one consolidated undrained (CU) tests were performed on remolded specimens; those triaxial tests were carried out with consideration of confining pressure of 100, 200, and 300 kPa. The results shows that the angle of internal friciton of the 50mm e 38mm specimens presented the value of 33º and 29º.While the results for the cohesion intercept showed values of 18,25 and 68,12 kPa. Analyzing the results that were obtained on this study, it is possible to indicate that the variation of the specimen diameter can influence on effective shear strength parameters. The results show that the specimen diameter influence on effective shear strength parameters for triaxial tests on different diameters specimens is small regarding the internal friction angle. On the other hand, soil cohesion presented a significant variation

KEYWORDS: Triaxial Tests; Remolded specimen; effective shear strength parameters

INTRODUCTION

In geotechnical engineering, the shear strength parameters are crucial and useful for design work to produce safe and economic geotechnical structure design. The shear strength of soil is the maximum resistance to shear expressed as a stress. Soil shear strength derived from two main components: internal friction angle and cohesion. Several factors could affect the shear strength of soil [1]. When the shearing stress reaches its limit value due to the failure of a loaded soil mass, soil deformation is caused. Movement of wedge soil behind a retaining wall or sliding in an earth embankment are some of the forms of shear failure [2]. An improper estimation can constitute a serious damage to both property and life. Cohesion is a component of the shear strength, which is independent of the normal stresses applied, the origin of this phenomenon is due to the grouting between the particles, chemical attraction between clay particles, residual stresses from the original rock and ionic attraction.

The triaxial compression test is the most used test when it comes to the evaluation of the shear strength of the soil, and obtaining its parameters. The test offers a range of possibilities in its conduction, as the option to control the load applied to the sample or the deformation suffered by it. The principle of triaxial compression test is versatile, and procedures may be related to various practical problems such as the investigation of slope stability and the design of retaining walls and foundations optimization [3]. The test can simulate real situations of the field by providing better understanding of the behavior of soils and their properties. The test has remarkable advantages as the control of drainage conditions and the possibility of measuring the pore pressure. No other test that combine these two features is designed to date. On the test, the cylindrical specimen is sealed by a rubber membrane, and confined in a cell with water, which can be subjected to pressure. An axial load is thrown on top of the sample via a piston which controls deviator stress. The connections allow the cell to drain both water and air in the soil voids, or the measurement of pore pressure on condition of undrained test [4].

LITERATURE REVIEW

Shear Strength

The shear strength of soil mass can be defined as the internal resistance per unit area that soil mass can offer to resist failure and sliding along any plane inside it [5]. The shear strength of soil (τ_f) in terms of effective cohesion and effective normal stress at failure:

$$\tau = c' + (\sigma_n - u_w) \tan \phi' \qquad (1)$$

where c' and ϕ' are the effective shear strength parameters (also called the effective cohesion and the angle of shearing resistance respectively); $(\sigma_n - u_w)$ is the effective normal stress. The failure occurs in a plane where there is a critical combination of normal and shear stress, and not a maximum stress value: normal or shear as presented on Figure 1. The Mohr-Coulomb criteria does not take into account the intermediate principal stress, but it still represents the soil behavior well, as the intermediate principal stress has little influence on soil resistance.

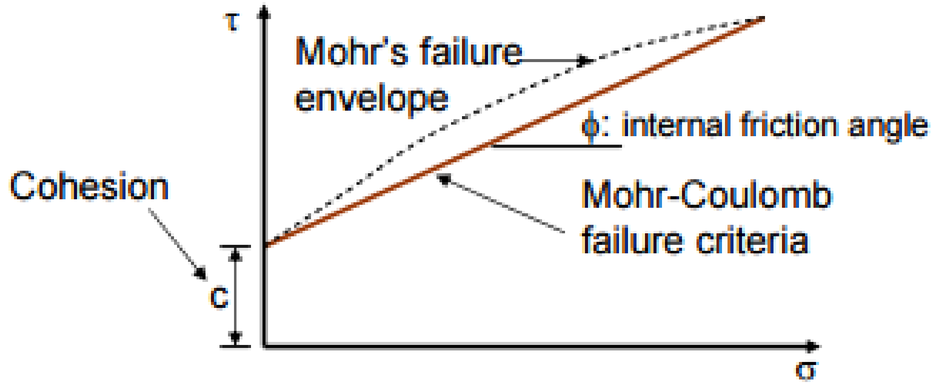

Figure 1: Mohr-Coulomb failure criteria

Mohr´s Circle

The stress state that act on all planes through a point can be represented graphically in a coordinate system, where the abscissas are represented by the normal stresses and the ordinates represented by shear stress [6]. The Mohr circle can be constructed from the normal stress and shear, and two principal stresses (σ_1 and σ_3) as shown in Figure 2.

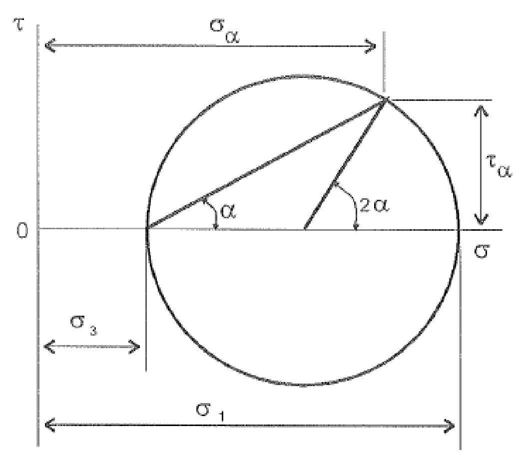

Figure 2: Mohr´s Circle

The stresses (σ_α; τ_α), are represented graphically by a circle in a coordinate system (σ, τ), known as Mohr´s circle. The Mohr´s circle is used to simplify the determination of the stresses and translate the results of the soil shear strength tests. From the principal stresses σ_1 and σ_3, it is obtained the radius and the Mohr circle center indicated by the following equations, where:

$$\text{Radius} : \left(\frac{\sigma_1 - \sigma_3}{2}\right) \tag{2}$$

$$\text{Center}: \left(\frac{\sigma_1 + \sigma_3}{2}\right) \tag{3}$$

Effective Stress State

The stress state can be defined in terms of effective stress. Considering the principal stresses σ_3 and σ_1 and pore pressure, u, in the soil, the circles showed in Figure 3 are obtained [6]. Based on Figure 3, one can emphasize two points:

a) The effective stress circle is moved to the left, relative to the total stress circle of a value equal to the neutral pressure. This fact is due to the neutral pressure acting hydrostatically, reducing of equal value, the normal stresses in all planes. In case of negative pore pressures, the circle moves to the right for the same reason.

b) The shear stresses on any plane are independent of pore pressure, because the water does not transmit shear stress. The shear stresses are due only to the difference between the principal stresses and this difference is the same in both total stress, as in effective stress.

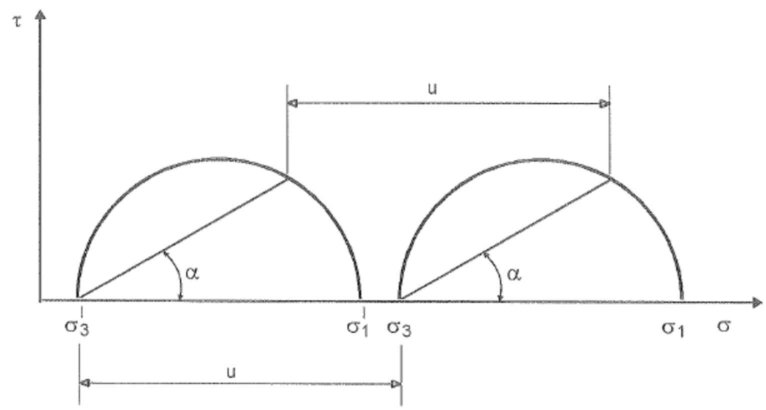

Figure 3: Pore pressure effect on stress state of soil

Triaxial Tests

The triaxial test is carried out in a cell on a cylindrical soil sample having a length to diameter ratio of 2. The usual sizes are 76 mm x 38 mm and 100 mm x 50 mm. Three principal stresses are applied to the soil sample, out of which two are applied water pressure inside the confining cell and are equal. The third principal stress is applied by a loading ram through the top of the cell and is different to the other two principal stresses. A typical triaxial cell is shown in Figure 4.

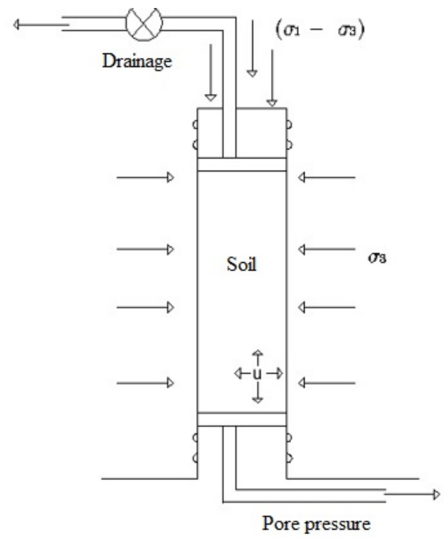

Figure 4: Triaxial test

The soil sample is placed inside a rubber sheath, which is sealed to a top cap and bottom pedestal by rubber O-rings. For tests with pore pressure measurement, porous discs are placed at the bottom, and sometimes at the top of the specimen. Filter paper drains may be provided around the outside of the specimen in order to speed up the consolidation process. Pore pressure generated inside the specimen during testing can be measured by means of pressure transducers.

Overconsolidation

Overconsolidated clays can be made in the laboratory by the consolidation of a sample under effect of an effective stress, therefrom, sample dilatation is allowed under a smaller effective stress. Overconsolidations of great magnitude usually lead to the development of negative pore pressure in undrained tests. When the confining stress is lower than the pre-consolidation stress of the soil, after the first stage of the test, when the sample is consolidated under confining stress, the soil is overconsolidated [6]. When the confining pressure is much lower than the pre-consolidation stress in the CD test, a volume increase occurs during the axial load, which causes the water entering the specimen. In the CU test, with no drainage, the water voids in the soil is subjected to a tensile stress state (in the same way as occurs in a syringe by pulling the piston without allowing ingress of liquid). Figure 5 shows the expectation of the pore pressure behavior in undrained test.

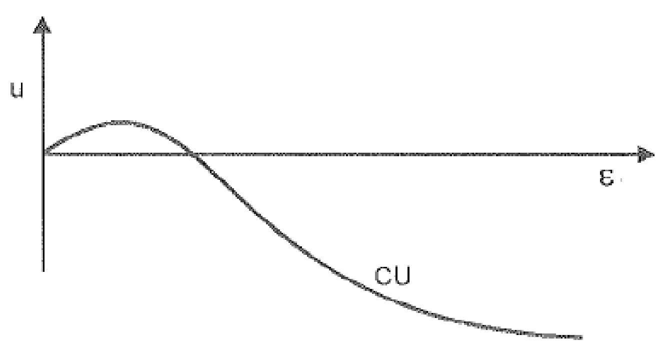

Figure 5: Pore pressure behavior of overconsolidated soils in CU tests

MATERIALS AND METHODS

Disturbed silty sand soil samples were collected from a trench next to the Civil Construction building at the Federal Institute of Ceará. The soil was excavated with a shovel at a depth of 1.5 m below the ground surface, removing the layer of humus and roots, placed in wood boxes, and transported to the geotechnical laboratory of Federal Institute of Ceará. Soil index properties tests such as moisture content, specific gravity, particle size distribution, plastic limit, and liquid limit were performed according to the Brazilian Standard [7]. The triaxial tests were carried out on remolded soil specimens. The remolded specimens for the triaxial tests were prepared using static compaction at a specified moisture content and density as seen in Figure 6.

Figure 6: Static compaction at a specified moisture content and density

The soil samples were compacted in a cylindrical mold with moisture content of 10,5% and 19,17 kN/m³ density as shown in Figure 7. The CU triaxial tests were performed under three different cell pressures of about 100, 200, and 300 kPa using specimens of 50mm e 38mm diameter. The specimens in the CU triaxial test were sheared with a strain rate of 0.083 mm/min.

Figure 7: Triaxial Test Stages

RESULTS AND DISCUSSIONS

The soil used in this study was classified as a SM-SC (silty sand) according to the Unified Soil Classification System. Table 1 shows the basic properties of studied soil.

Table 1: Basic properties of studied residual soil

Moisture Content (%)	Specific Gravity	Particle Size Distribution				Atteberg Limits		Soil Classification (USCS)
		Clay (%)	Silt (%)	Sand (%)	Gravel(%)	Plastic limit (%)	Liquid Limit (%)	
10,5	2,48	14	3	83	0	0	0	SM-SC

Triaxial Test Results

Many tests were performed to obtain consistent results. Because of the relative complexity of the test, some samples were lost, others, presented inconsistent values. For the selection of compatible values, it was held a careful analysis of the various results. Table 2 and Table 3 present results of the performed triaxial tests with 50mm e 38mm diameter samples.

Table 2: Triaxial results for 50mm diameter samples

50mm diameter samples						
Specimen #	σ3 (kPa)	σd (kPa)	σ1 (kPa)	u (kPa)	σ'1 (kPa)	σ'3 (kPa)
3	100	712	812	-163	975	263
5	100	698	798	-171	969	271
2	200	860	1060	-114	1174	314
6	200	758	958	-147	1105	347
23	200	791	991	-2	993	202
24	200	804	1004	-110	1114	310
4	300	1143	1443	-139	1582	439
7	300	1120	1420	-137	1557	437

Table 3: Triaxial results for 38mm diameter samples

38mm diameter samples						
Specimen #	σ3 (kPa)	σd (kPa)	σ1 (kPa)	u (kPa)	σ'1 (kPa)	σ'3 (kPa)
8	100	609	709	-131	840	231
13	100	501	601	-137	738	237
16	100	386	486	-103	589	203
17	100	395	495	-109	604	209
12	200	718	918	-124	1042	324
14	200	547	747	0	747	200
18	200	734	934	-70	1004	270
11	300	699	999	-83	1082	383
15	300	849	1149	-109	1258	409
19	300	611	911	-62	973	362
20	300	948	1248	-84	1332	384
21	300	759	1059	-53	1112	353
22	300	973	1273	-78	1351	378

After analyzing the triaxial test results, it is possible to notice the specimens that presented the most consistent results for each diameter. The choice of the specimen that would be part of the failure envelope was made due to the more consistent results and adjustments. The set composed by specimens 3, 24 and 4 showed the best adjustments for the 50mm diameter samples. Meanwhile, the specimens 17, 18 and 20 presented the best fit for the 38mm diameter samples. From the pick of the best set of specimens, both Mohr-Coulomb failure envelopes were computed as shown in Figures 8 and 9.

Figure 8 shows the Mohr-Coulomb failure envelope for the 50mm diameter samples. From this adjustment, it was found that cohesion intercept has a value of 18, 25 kPa and the internal friction angle of 33º.

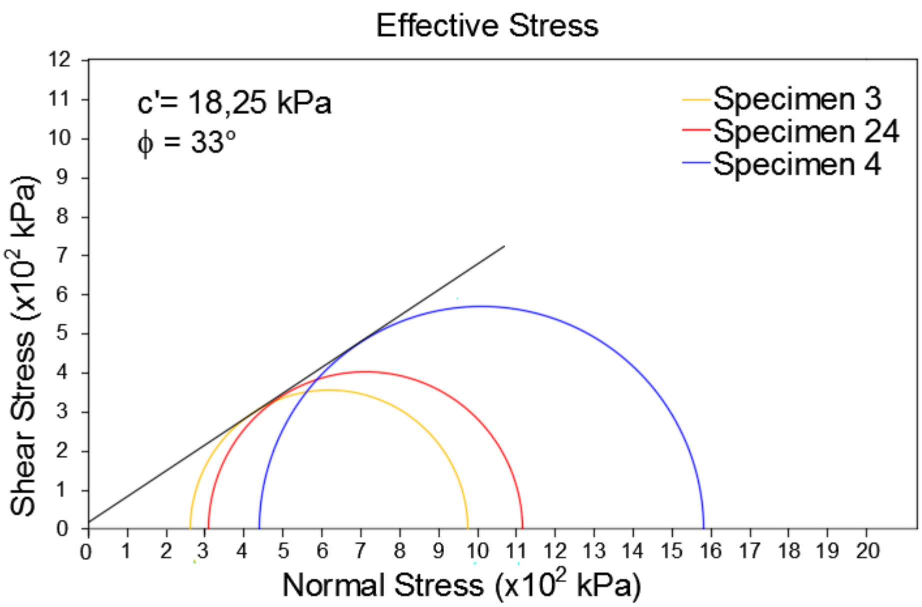

Figure 8: Mohr-Coulomb failure envelope for the 50mm diameter samples

As for the Mohr-Coulomb failure envelope for the 38mm diameter samples, the determined adjustment resulted in a cohesion intercept with a value of 68,12 kPa and the internal friction angle of 29º as shown in Figure 9.

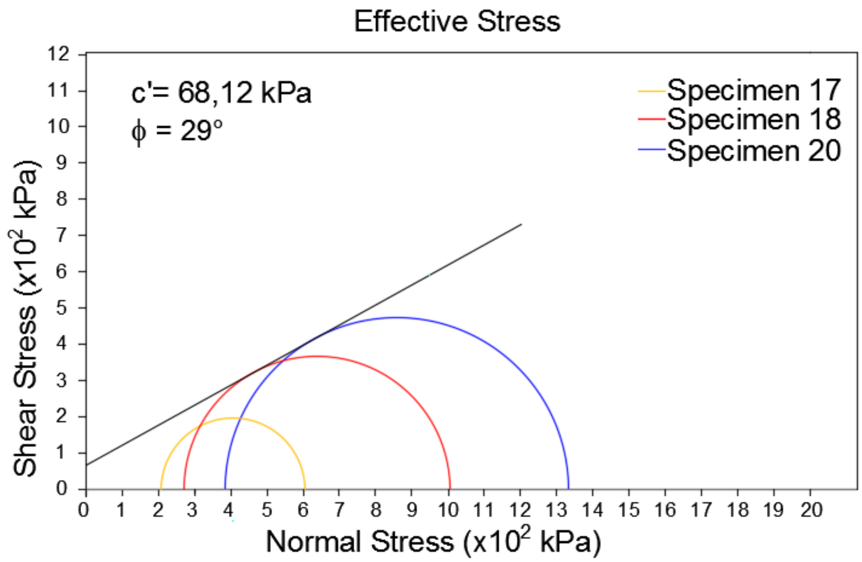

Figure 9: Mohr-Coulomb failure envelope for the 38mm diameter samples

Failure Planes

According to the Mohr-Coulomb failure criterion, failure from shear will occur when the shear stress on a plane reaches a value given by:

$$\theta = 45° + \frac{\phi'}{2} \tag{4}$$

In order to determine the inclination of the failure plane with the major principal plane, where σ'_1 and σ'_3 are, respectively, the major and minor effective principal stresses. The failure plane makes an angle θ with the major principal plane. In Figures 10 and 11 the inclination of the failure plane of the tested soil for both 50mm e 38mm diameter samples is shown. The 50mm e 38mm diameter samples presented inclinations of the failure plane of 62° and 59 °.

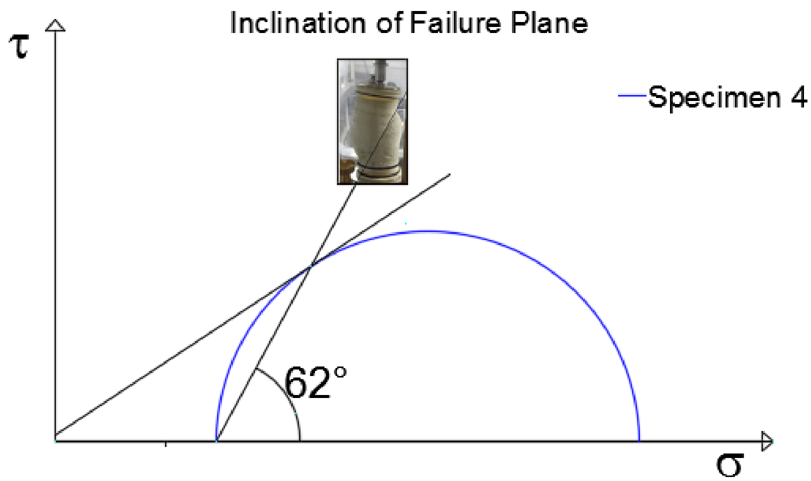

Figure 10: Inclination of the failure plane for 50mm specimen

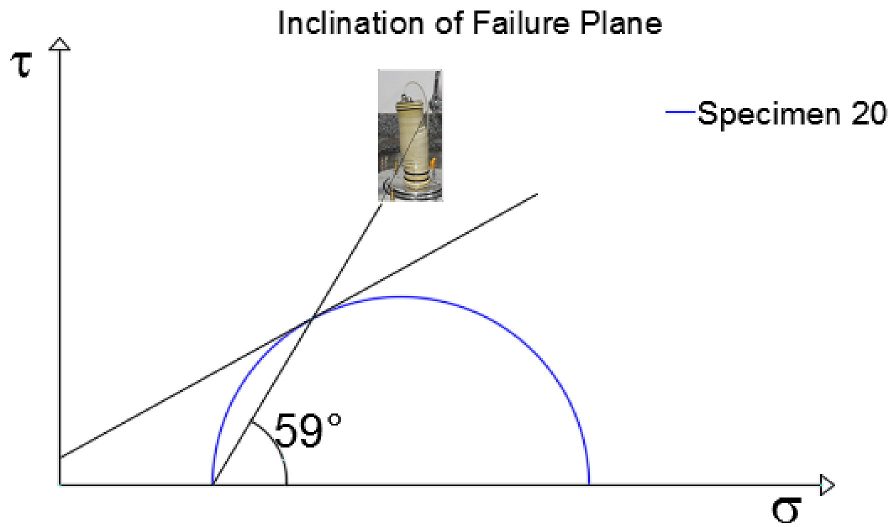

Figure 11: Inclination of the failure plane for 38mmspecimen

Comparison between Triaxial Tests Performed on Different Diameter Specimens

Table 4 shows the effective shear strength parameters from triaxial tests and inclination of the failure plane of 50mm and 38mm diameter samples comparison. The results show that the effective shear strength parameters for triaxial tests on different diameters specimens are not identical, although, the internal friction angle presented similar results. The internal friction angle showed a 12,12% difference between the different diameter specimens. On the other hand, the specimens cohesion presented a 73,20% discrepancy, which is a considerable variance. The shear strength of the soil is affected by the quality of the sample. Remolded samples will usually present lower values of effective shear strength parameters than undisturbed samples because residual soils are sensitive to disturbances and disruptions incurred during sampling that affect the results of the tests [1]. Disruptions in the stability of the soil samples gave a lower value for shear strength due to the collapse of the soil structure as well as increase the value of effective friction angles [8]. During the tests performance, it was noted a great difficulty on working with the 38mm diameter samples, due to its fragility. Some samples were ruined during the use of rubber sheath, to a point that the test would not proceed, because the specimen was damaged. Thereat, the number of triaxial tests needed to achieve consistent results was higher. The inclination of the failure plane of 50mm and 38mm diameter samples varied 4,83%, which was expected due to the relation to with the internal friction angle.

Table 4: Triaxial results for 1,4" diameter samples

Specimen Diameter (mm)	c' (kPa)	ϕ' (º)	θ (º)
50	18,25	33	62
38	68,12	29	59

CONCLUSIONS

(1) The results show that the specimen diameter influence on effective shear strength parameters for triaxial tests on different diameters specimens is small regarding the internal friction angle. On the other hand, soil cohesion presented a significant variation.

(2) The results of the triaxial tests showed that 50mm diameter samples presented a cohesion intercept of 18, 25 kPa and the internal friction angle of 33º. Meanwhile, the 38mm diameter samples presented a 68,12 soil cohesion and a 29º internal friction angle.

(3) The internal friction angle exposed a 12,12% difference between the different diameter specimens.

(4) The specimens cohesion showed a 73,20% divergence, which is a considerable variance.

(5) The inclination of the failure plane values of 50mm and 38mm diameter samples are respectively 62º and 59º, presenting a variation of 4,83%, the small variation is explained by its relation to the internal friction angle

(6) The usage of 38mm diameter specimens is not recommended due to its fragility when testing silty sand soils, which can be easily damaged during the rubber sheath handling, in that way, occasioning inconsistent results.

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REFERENCES

1. Rohaya Alias, Anuar Kasa, Mohd Raihan Taha: “Effective Shear Strength Parameters for Remolded Granite Residual Soil in Direct Shear and Triaxial Tests” *Electronic Journal of Geotechnical Engineering*, 2014 (19.R):4559-4569. Available at ejge.com

2. Sushama Kiran, Bindhu Lal: “Modelling of Soil Shear Strength Using Neural Network Approach” *Electronic Journal of Geotechnical Engineering*, 2016 (21.10): 3751-3771. Available at ejge.com

3. HEAD, K.H. Manual of soil laboratory testing. Chichester: John Wiley & Sons Ltd, v.2, 1994

4. BISHOP, A. W., HENKEL, D. J. The measurement of soil properties in the triaxial test, 2.ed, Londres, UK: Edward Arnold Ltd., 227 p., 1962.

5. Terzaghi, K. (1936). The Shear Resistance of Saturated Soils. Proceedins of 1st International Conference of Soil Mech.Found. Eng. Cambridge.1, 54-56.

6. PINTO, Carlos de Souza. Curso Básico de Mecânica dos Solos. 3.ed São Paulo: Oficina de Textos, 2006

7. Associação Brasileira de Normas Técnicas. (2016). Amostras de solo - Preparação para ensaios de compactação e ensaios de caracterização: NBR 6457. Rio de Janeiro, Brasil.

8. Marto, A. and Kasim, F. (2003) “Characterisation of Malaysian Residual Soils for Geotechnical and Construction Engineering,” Project Report, Vote No: 72256, UTM, Malaysia