Mechanical behaviour of an unsaturated compacted residual soil

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ABSTRACT

This paper provides a series of experimental test results performed on a residual soil from a site of the University of São Paulo in São Paulo, Brazil. The focus of these tests were to study the shear strength behaviour by conducting modified unconfined compression tests as well as constant water (CW) content triaxial tests on specimens compacted at different initial water content conditions. The matric suction in the soil specimens was precisely measured during the tests by using high capacity tensiometer (HCT). The experimental results suggest that the shear strength behaviour is significantly influenced by the initial compaction water content. An empirical model is proposed using the results of the present study to predict the 3D failure envelope of the tested compacted residual soil for any condition of the initial stress state.

RÉSUMÉ

Cet article présente une série d'essais expérimentaux menés sur un sol résiduel d'un site de l'université de São Paulo, au Brésil. L'emphase de ces essais a été mise sur l'étude du comportement de résistance au cisaillement lors d'essais de compression non-confinés et d'essais triaxiaux à teneur en eau constante (EC) sur des échantillons compactés sous différentes conditions de teneur en eau initiale. La succion matricielle dans les spécimens a été mesurée avec précision durant les essais en utilisant un tensiomètre à haute capacité (THC). Les résultats expérimentaux suggèrent que la résistance au cisaillement est influencée de manière marquée par la teneur en eau initia l’échec en 3D de le lors du compactage. Un modèle empirique est proposé faisant usage des résultats de la présente étude afin de prédire l’enveloppe de résistance au cisaillement du sol résiduel compacté sous toutes conditions de contrainte initiale.

1 INTRODUCTION

Residual soils formation is attributed to the mechanical and chemical weathering or disintegration of rocks. The behaviour of these soils is dependent on their composition and also on the environmental conditions under which they were formed. The upper layers of the residual soil are typically fine-grained in nature and below which are partially disintegrated parent rocks. In other words, residual soils commonly remain in place over the rocks from which they are originally formed. Such soils are found in many regions of the world and more widely distributed in Brazil. These types of soils are used commonly in the construction of both geotechnical and geoenvironmental structures such as embankments, pavements and soil barriers.

The residual soils show significantly different characteristics in comparison to other natural soils and are difficult to be classified using conventional procedures. The conventional soil mechanics principles also do not offer rational interpretation of residual soils mechanical behaviour because they are typically in a state of unsaturated condition. Therefore, geotechnical engineers are interested in understanding the mechanical behaviour of both natural and compacted residual soils which are sometimes referred in the literature as problematic soils (Rahardjo, et al, 1995, Pereira, et al., 2000; Toll and Ong, 2003; Rahardjo, et al, 2004a, Rahardjo, et al, 2004b; Futai and Almeida, 2005; Matsushi and Matsukura, 2006; Kayadelen, et al., 2007; Jotisankasa and Mairaing, 2010). The residual soils mechanical behaviour is also sensitive to wetting and drying cycles (Toll and Ong, 2003; Rahardjo, et al, 2004a). For this reason, the influence of initial compaction water content as it relates to the soil shear strength behaviour is important and hence investigated in this paper. The soil used in the present study is a residual soil obtained from the experimental site of the University of São Paulo in São Paulo, Brazil.

The unsaturated shear strength of compacted residual soil is studied by conducting unconfined compression tests and constant water content (CW) tests. The CW tests were conducted in the triaxial shear test equipment introducing some modifications to accommodate the high capacity tensiometer (HCT). Along similar lines, HCT was used to measure the matric suction changes during the shearing stage of conducting the unconfined compression tests. The HCT used for the present investigation was built at the University of São Paulo to measure matric suction in the range of 0 to 500 kPa (Oliveira and Marinho, 2008). The shear strength behaviour of the soil under saturated conditions was performed by Oliveira (2004). The experimental results of the present study and that of Oliveira (2004) have been used to develop a 3-D empirical model to predict the failure envelope of compacted residual soils of São Paulo, Brazil for any condition of the initial stress state. The results of the study suggest that the initial moulding water content of the compacted residual soils has considerable influence on the shear strength.

2 SOIL PROPERITES AND EXPERIMENTAL PROCEDURES

2.1 Soil properties

The residual soil sample collected from the site of the University of São Paulo in São Paulo, Brazil was homogeneous in nature. Table 1 summarizes some of the soil properties and Figure 1 shows the representative grain size distribution. The soil has 20% of clay, 46% of silt and 34% of sand-sized particles.

Table 1. Index properties of compacted residual soil

Liquid limit (%)	w_L	47
Plastic limit (%)	w_P	13
Plasticity index (%)	I_p	34
Specific gravity	G_s	2.71

The compaction curve presented in Figure 2 was determined using the Standard Proctor energy. The soil has a maximum dry density, γ_{dmax} of 15.3 kN/m^3 and the optimum moisture content is 24.5%. In the present study, the shear strength behaviour of soil samples prepared at three different water contents was studied. These water contents represent the dry of optimum, the optimum, and the wet of the optimum conditions. The water content and density conditions chosen for the present study are shown as three points on the compaction curve as point D, O and W respectively (see Table 2). Fine-grained soils exhibit different soil structures at these different water contents. Earlier studies have shown that the soil structure can influence the engineering behavior of fine-grained unsaturated soils (Vanapalli et al. 1999). One of the key objectives of the present study is to examine the influence of soil structure associated with different water contents and also the initial stress state on the shear strength.

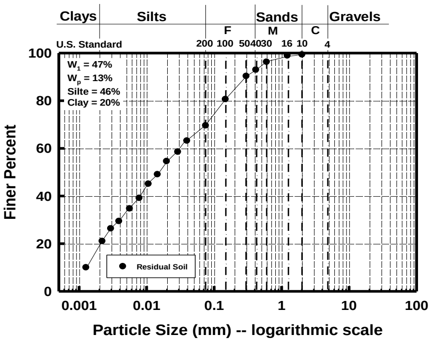

Figure 1. Grain size distribution

Table 2. Properties of the soil at the compaction conditions tested

Point	w (%)	γ_d (kN/m^3)	e	S (%)	θ_w (%)
O	25.3	15.3	0.77	88.9	38.7
D	17.0	14.8	0.83	55.4	25.2
W	28.2	14.8	0.83	92.0	41.7

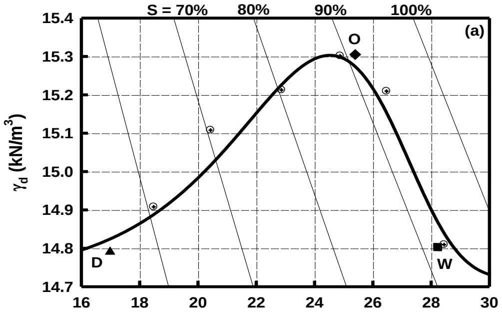

Figure 2. Standard Proctor energy compaction curve

Figure 3 shows the Soil-Water Characteristic Curves (SWCC) for the specimens prepared at the three different initial compaction water content conditions summarized in Table 2. These curves were obtained by using suction plate, pressure plate and filter paper methods for suction measurement. The suction range of SWCC measured using each of these methods is also shown in Figure 3. The SWCC of specimens compacted at dry of optimum conditions desaturate at a faster rate in comparison to optimum and wet of optimum conditions. The specimens compacted at optimum and wet of optimum conditions show approximately similar SWCC behavior suggesting that the soil structure is approximately the same at both these water contents. The SWCC behavior of the residual compacted specimens is consistent with the observations of other investigators on compacted fine-grained soils (for example, Vanapalli et al. 1999). The SWCC results provide valuable information with respect to the shear strength behavior under unsaturated conditions.

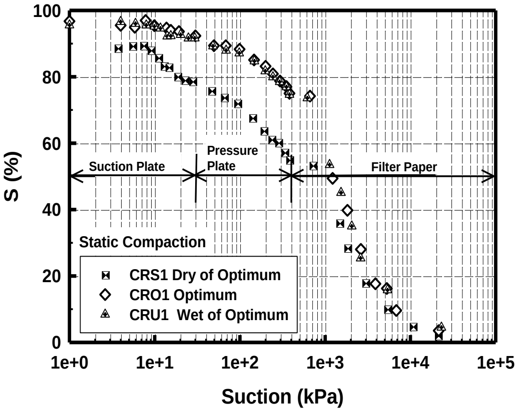

Figure 3. Soil-water characteristic curves for the specimens compacted at different initial water contents.

2.2 Preparation of the compacted specimens

The representative residual soil sample collected at the site of the University of São Paulo in São Paulo, Brazil was mixed using the initial water content conditions (i.e., wet of optimum, optimum and dry of optimum) chosen for the study (see Table 2). The soil sample mixed with the required water content were placed in plastic bags for a period of 48 hours and stored in a humid environment at constant temperature to achieve uniform water content conditions. The soil-water mixture was used for preparing specimens of 38 mm in diameter and 80 mm in height by static compaction into a mould. The purpose of using static compaction as opposed to dynamic compaction is to obtain a more homogenous specimen with respect to density throughout the volume of the specimen (Rahardjo et al., 2004b). The initial matric suction of the specimens were adjusted to the desired value following one of the hydraulic paths: (1) Water was sprinkled carefully over the specimen to reach a degree of saturation that is lower than 100%; (2) The initially compacted specimens were allowed to air dry such that they attain a desired value of matric suction or (3) The specimens were gradually allowed to air dry such that they reach the desired matric suction value after the compacted specimens were allowed to imbibe moisture to achieve saturated conditions. In all the cases, it was assured that the water content throughout the specimen was uniform. This was achieved by storing the specimens overnight in a humid environment chamber. The soil specimens prepared following the above procedures were used in the testing program.

2.3 Experimental procedure

The shear strength of the compacted soil specimens were determined from the unconfined compression and constant water content (CW) triaxial tests. The test program was planned such that the shear strength data can be obtained following different hydraulic paths using specimens that were prepared using different initial compaction water contents. The first phase of each test consisted of a wetting or drying stage during which specimens were brought to desired matric suction, in the range of 0 to 400 kPa, following one of the three paths discussed in the earlier section. Each of the prepared specimens was subjected to shearing under CW allowing the air to drain freely during the testing. The changes in matric suction during the loading stage were measured using the HCT. The HCT developed at the University of São Paulo was placed at the bottom of the specimen for measuring matric suction in the range of 0 to 500 kPa. The design focus of this instrument was to avoid cavitation and measure matric suction values rapidly. The principle of matric suction measurements is based on achieving equilibrium conditions between the pore water pressure in the soil and pore-water pressure in the water of the HCT compartment (Oliveira and Marinho, 2008). The testing procedure for conducting the unconfined and triaxial shear tests are detailed in the following sections.

2.3.1 Unconfined compression tests

A total of 57 unconfined compression tests were conducted on the specimens that were prepared using three different initial water contents (dry of optimum, optimum, and wet of optimum). In other words, 19 unconfined compression tests were conducted for each of the initial compaction water contents. The shear strength tests in the present study were carried out using the modified apparatus with direct matric suction measurement at the base of the specimen using the HCT. The loading was performed following stress or strain controlled procedures. For the strain controlled procedures, the matric suction was measured and recorded at intervals of three seconds. However, for the stress controlled procedures, the matric suction was measured at the same rate but recorded only after achieving equilibration conditions.

2.3.2 Constant water content (CW) triaxial tests

In addition to the unconfined compression tests, a total of 57 CW triaxial tests were conducted on specimens prepared at different initial compaction water contents as summarized in Table 2 (i.e., 19 tests were conducted for each of the three compaction water contents chosen for the study). These tests were conducted in modified triaxial shear equipment following the testing procedures summarized in Fredlund and Rahardjo (1993). The only difference lies in technique of measuring the matric suction. Modifications were made in the apparatus such that the applied matric suction is controlled using the axis translation technique and as well as measured in the soil specimen independently by using the HCT. A desired value of initial matric suction was applied prior to isotropically subjecting the soil specimen to desired confining pressure, which was set at 50, 100, 200 or 300 kPa. The specimens were then sheared by increasing the normal stress (stress controlled) during the test. The changes in the axial strain and the matric suction of the specimens were monitored during each step of the tests.

3 TEST RESULTS AND DISCUSSION

The unconfined compression tests and the CW triaxial shear tests have been performed simulating different hydraulic stress paths. The chosen paths for the study are commonly encountered in engineering practice. The tests in the present study were also carried out to investigate the shear strength behaviour of residual soils at different compaction water contents that are subjected to different normal stresses.

3.1 Unconfined compression tests

The specimens tested can be divided into three groups based on their hydraulic stress paths (see section 2.2 for more details). In this paper, only test results on specimens compacted at optimum moisture (or water) content (OWC) are summarized due to space limitations. The test results of the specimens compacted at the OWC and subjected to the path 1 and 2 under strain controlled, path 3 under strain controlled and path 1 and 2 under stress controlled are summarized in Figures 4, 5, and 6 respectively. The matric suction changes are sensitive to the applied loading until the failure conditions of the tests. However, the rate of matric suction change greatly reduces by the end of the test.

The specimens with lower initial matric suction values fail quickly in comparison to specimens with higher initial matric suction values. The matric suction changes during the tests also reduce quickly in specimens with the lower values of initial matric suction. Furthermore, as the initial matric suction of the specimen is increased, deviator stress at failure increases, which is reflected as an increase in the shear strength of the specimen. In other words, initial matric suction is one of the key parameters that influence the shear strength behavior of the compacted residual soil tested.

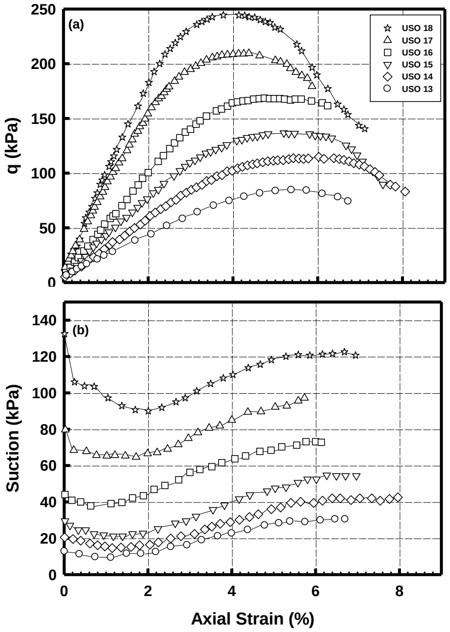

Figure 4. Strain controlled unconfined confined compression tests results for specimens compacted at OWC and following the hydraulic paths 1 and 2 prior the test.

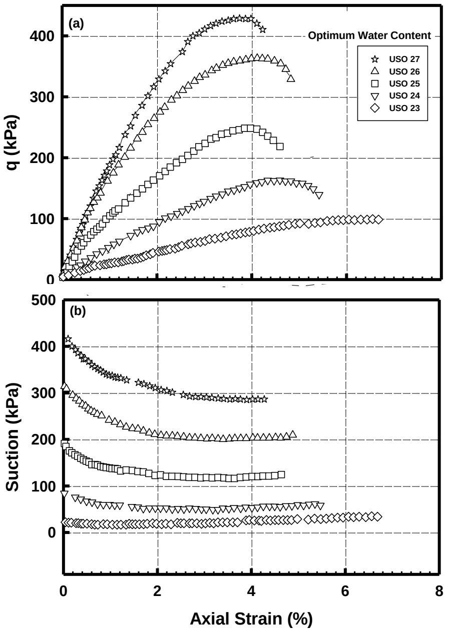

Figure 5. Strain controlled unconfined confined compression tests results for specimens compacted at OWC following the hydraulic paths 3 prior the test.

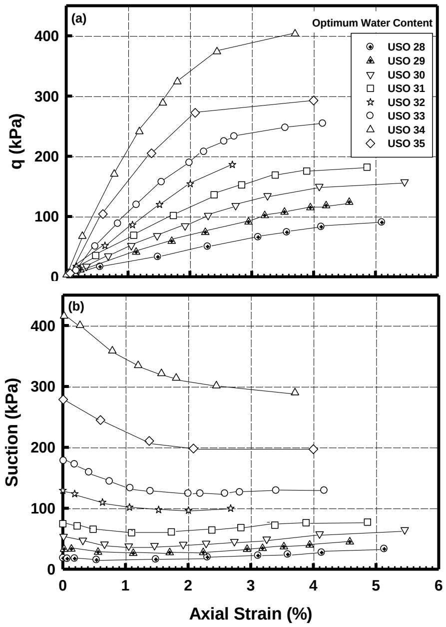

Figure 6. Stress controlled unconfined confined compression tests results for specimens compacted at OWC following the hydraulic paths 1 and 2 prior the test.

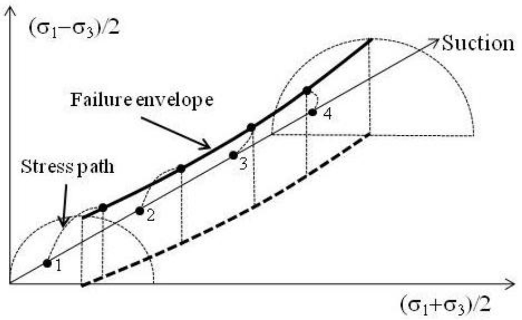

Figure 7. Schematic representation of the stress paths followed by the specimens and failure envelope obtained from the unconfined tests for OWC specimens.

The results are plotted in Figures 4 through 6 as a relationship between the deviator stress versus axial strain (Figures 4a-6a), and matric suction changes versus axial strain (Figures 4b-6b). These data are useful in the analysis of the unconfined compression test results. Failure conditions were well defined In Figures 4a-6a. In other words, well defined peak stress was observed from the stress versus strain relationships. In order to critically investigate the mechanical behavior of the residual soil, the relation between stress state variables was plotted. Figure 7 presents a schematic of the possible stress paths during the unconfined shear strength tests and the failure envelope which represents the failure points of each of the tests performed with specific initial stress state conditions is shown. It can be observed that the stress paths showing the failure condition indicate a non-linear variation of the shear strength with respect to matric suction. The shear strength continuously increases non-linearly without showing a decrease after certain in value of matric suction value. Such a behavior can attributed to the high percentages of fines and also due to the dilatancy behavior of the high density OWC specimens tested under unconfined conditions (see Fig. 4). Other CW specimens tested with confining pressures have shown a non-linear increase followed with a decrease in the shear strength which is consistent with their volume change behavior. These results are not shown in this paper because of space limitations.

3.2 Constant water content triaxial tests

For the CW triaxial tests, specimens were tested following path 1 or 2, using stress control method. The specimens were subjected to confining pressure of 50, 100, 200 and 300 kPa. The matric suction was monitored during the application of the confining pressure using the HCT. After achieving equilibrium conditions with respect to matric suction under each applied confining pressure, the deviator stress was increased until the specimen failed. Although the triaxial tests were performed using specimens compacted at different initial water contents (see Table 2), only results corresponding to specimens compacted at optimum water content (OWC) are shown in this paper. Also, due to space limitations, the typical triaxial test results conducted on specimens with OWC using confining pressures of 100, and 300 kPa are presented (Figure 8 and 9).

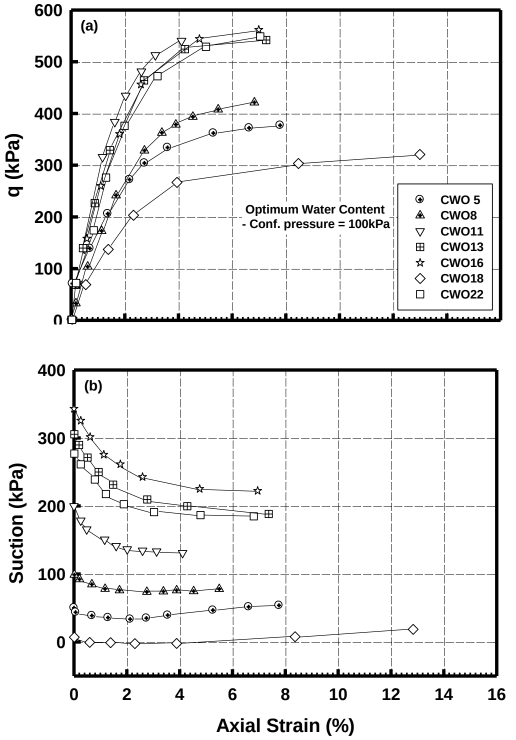

Figure 8. CW triaxial test results conducted with a confining pressure of 100 kPa for specimen compacted at OWC.

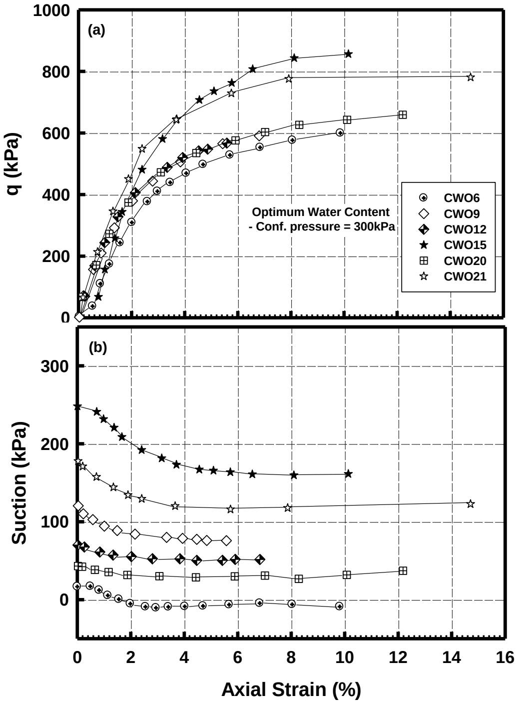

Figure 9. CW triaxial test results conducted with a confining pressure of 300 kPa for specimen compacted at OWC.

Figures 8a and 9a illustrate the stress versus strain relationships for the compacted unsaturated soil specimens under confining pressures of 100 and 300 kPa respectively. The deviator stress increases at a faster rate at low axial strains. However, well defined peak stresses are not observed. A wide range of constant rates of change in deviator stress with axial strain can be observed.

Figures 8b and 9b show the variation of matric suction of the specimen during the period of shearing after the confining pressure was applied. The matric suction in the specimens reduces during the shearing stage. The matric suction value however reaches a constant value or slightly increases at failure or beyond failure conditions. The matric suction value at failure was typically lower than the initial value. As expected, the values of the deviator stress, which in turn reflects in the values of the shear strength of specimens, increase with an increase of the initial matric suction and the confining pressure.

Figure 10 presents schematic representation of the failure envelope associated with the compacted specimens tested using the three moulding water content conditions (dry, optimum, and wet of optimum conditions) for a specific confining pressure. In this analysis, it is assumed that the friction angle, ϕ' is independent of the matric suction value (see Table 4). The saturated shear strength behaviour of the compacted soil under similar conditions was studied by Oliveira (2004).

In order to obtain the failure envelope obtained from CW triaxial tests, the experimental values were projected to a plane (following the stress point form (t =(σ_1- σ_3)/2) versus the mean stress (p=(σ_1+ σ_3)/2) as shown in Figure 10. The points A, B and C defined the failure envelope on the plane (t) versus (p). These points are projected to the vertical line of the confining pressure used, leading to the points A', B' and C'. The angle used for this projection is the friction angle, β which is constant for all values of matric suction (Oliviera 2004). The value of the projection (t) for a specific confining pressure is obtained by multiplying (p) by (1-tanβ).

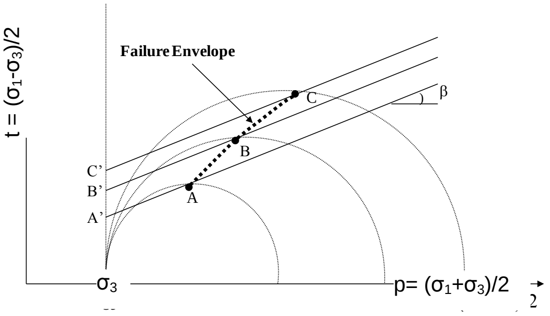

Figure 10. Schematic representation for the projection of the failure envelope for a specific confining pressure.

4 ANALYSIS OF THE RESULTS

Both the unconfined compression tests and the CW triaxial tests were conducted allowing air to drain freely and not allowing the water to drain (i.e., undrained conditions). The results obtained from the unconfined compression tests under saturated conditions are summarized in Table 3. These results are used to obtain the failure envelope for each compaction moulding water content condition.

Table 3 Shear strength parameters obtained from unconfined test at saturated condition for the three different moulding water content conditions (from Oliveira, 2004)

Moulding Condition	d' (kPa)	c' (kPa)	α' (^{o})	ϕ' (^{o})
Optimum	10.3	12	27	31
Dry of the optimum	3.6	4.1	26	29
Wet of the optimum	7.7	8.8	26	29

Determination of the shear strength behaviour unsaturated residual soils using the axis translation techniques or the vapour equilibration technique are expensive, difficult and time consuming. Therefore, empirical equations to estimate the 3D failure envelope of compacted residual soils of São Paulo region of Brazil are developed using the results obtained from the present study results of unconfined compression and CW tests. These results can be applied for any condition of the initial stress state.

Figure 11 is obtained using the experimental data for unsaturated soil specimens compacted at the OWC. In order to obtain empirical equations for the 3-D failure envelope, the relationship between the stress point form $(\sigma_1$- $\sigma_3)/2$ and matric suction at failure is described using the planar form (i.e., 2D) (Figure 11). A linear shear strength envelope is assumed for low matric suction values below the air-entry value. The air-entry value can be estimated from the SWCC. Beyond the air-entry value, the shear strength behavior is represented as a power function for higher matric suction values. The starting point of the linear adjustments was defined by failure envelope obtained from the saturated shear strength tests data (from Oliveira, 2004). Oliveira (2004) determined the shear strength for the same three water contents used in the present study. It can be observed up to the air-entry value, the angle of the fitting line for different confining pressure is constant and equal to the friction angle for saturated condition (i.e., 27^o) (see Table 3). The first adjustment was performed using the unconfined compression tests. This starts with a value of $(\sigma_1$ - $\sigma_3)/2$ which is equal to 10.3 kPa. This value is the cohesion value from saturated soil specimens defined from the unconfined compression tests in stress point form (see Table 3). A manual linear adjustment was performed up to the point where the experimental data starts to diverge. The point where this occurs for the OWC condition is related to the air-entry value and is equal to 56 kPa) (obtained from the SWCC in Figure 3). From that point onwards, the contribution of matric suction towards the shear strength is non-linear. These results are consistent with the studies of other researchers (for example, Vanapalli et al. 1996). The increase in shear strength follows a non-linear behavior and a power function is fit for matric suction values higher than 56 kPa.

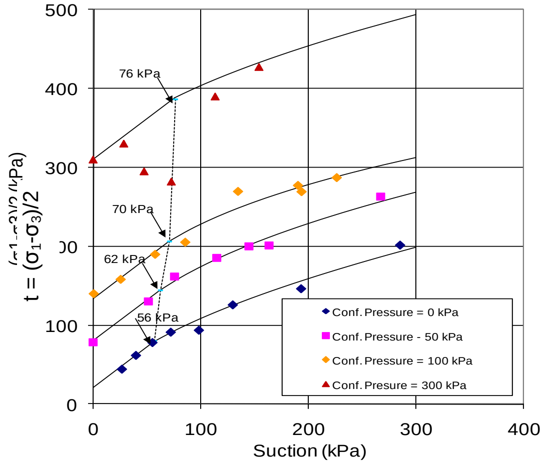

Figure 11. The stress point form as a function of matric suction for different confining pressures for specimens compacted at OWC.

The results obtained for shear strength data using confining pressures of 50, 100, 200 and 300 kPa were adjusted by applying the same methodology detailed for unconfined compression tests data earlier. The adjustment was applied to the experimental data of specimens compacted at dry of optimum, optimum water content, and wet of optimum water content conditions.

Figure 12 presents the failure envelope of the São Paulo region compacted residual soil. The results suggest that the shear strength behavior of specimens compacted at the dry side of optimum differ from the optimum and wet of optimum conditions. The shear strength behavior is lower for specimens compacted at dry of optimum conditions because the effective wet contact area for matric suction to contribute towards the shear strength is lower. This behavior can be attributed to the influence of soil structure and can also be observed from the SWCC behavior (Figure 3).

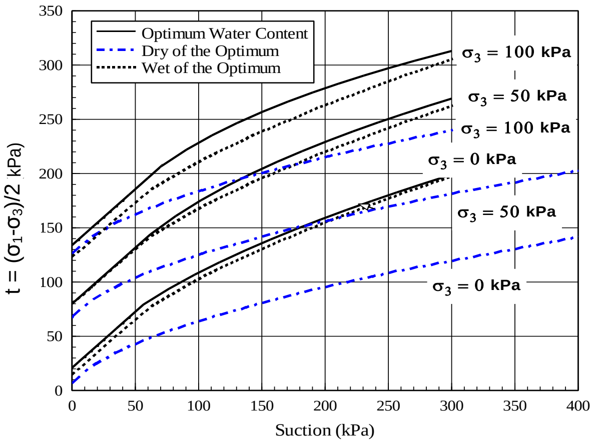

Figure 12. Failure envelopes according to the compaction condition and confining stress.

The developed failure envelopes can be represented by equations in the stress point as a function of matric suction (see Oliveira, 2004)

Figure 13 shows the schematic representation of the failure surface for specimens compacted at optimum water content (OWC). The line (AB) and the non-linear function (BC) were obtained by the projection of the equations of the line presented the failure envelope from the unconfined shear tests for specimens compacted at OWC (Figure 12) to zero confining pressure.. Line (AD) is the failure envelope from the strength tests on saturated soil specimens. The points from which the desaturation occurs and how the matric suction contribution effectiveness reduces are represented by line (BE). The equations for the segments shown in Figure 13 are summarized in Table 3.

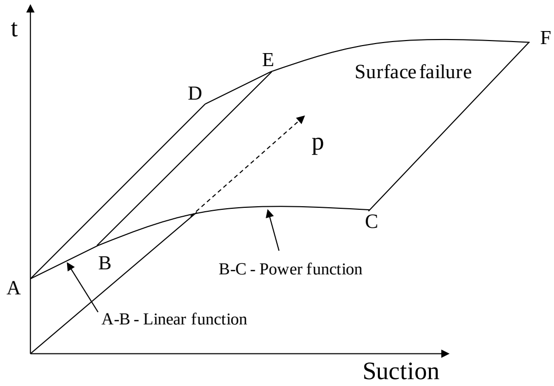

Figure 13. Schematic representation of the surface failure envelope for specimens compacted optimum water content condition.

Table 4. Equations defining the surface failure for moulding condition at OWC.

Segment	Range (kPa)	Equation
AB	(u_a-u_w) = 0 - 56	$t=10.3+0.5057\ (u_a-u_w)$
BC	(u_a-u_w) = 56 - 300	$t=4.2035(u_a-u_w)^{0.5512}$
AD	p= 0 - 300	$t=10.29+0.5057p$
BE	p= 0 - 300	$t=38.82+0.5057p$

As shown in Figure 13, the lateral plane represents the failure envelope for a saturated condition where the matric suction is zero. The extended stress point form is planar up to the air-entry value and is curved after the air-entry value. In other words, the surface failure up to the air-entry value of the soil was approximated to be linear. However, with increase in the matric suction, above the air-entry value, the surface failure was approximated to be non-linear.

The equation to represent the plane (ABDE) was obtained by combining equations for the segments (AB) and (AD). Similarly, equation for the surface (BCEF) is represented by a combination of segments of equations (BE and BC). Equations 1 and 2 represent the 3D failure envelope for specimens compacted at OWC for São Paulo residual soil (Figure 14).

$$t = 10.3+0.5057(p+(u_a-u_w)) \text{ for } (u_a - u_w) \leq 56.4 \text{ kPa} \qquad [1]$$

$$t=38.82+0.5057p+(4.2035\ (u_a-u_w))^{0.5512} \text{ for } (u_a - u_w) > 56.4 \text{ kPa} \qquad [2]$$

where, t or q = $(\sigma_1-\sigma_3)/2$; p = $(\sigma_1+\sigma_3)/2$.

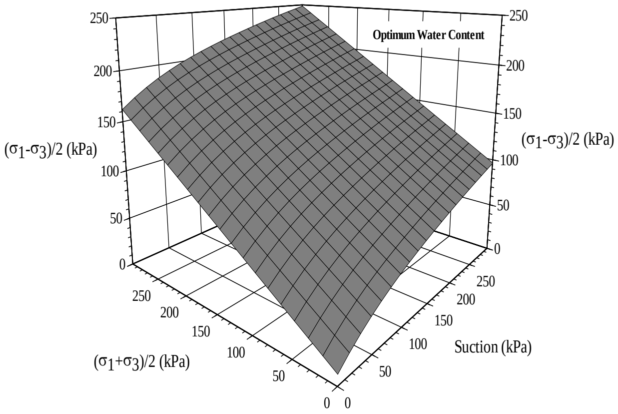

Figure 14. Failure envelope surface for specimens compacted at OWC.

5 CONCLUSION

A series of unconfined compression and CW triaxial shear tests were conducted on compacted residual soil specimens of São Paulo, Brazil. The tests were conducted introducing modifications to both unconfined compression and triaxial testing apparatuses such that matric suction could be measured reliably using the HCT. The study shows that the initial compaction water content significantly influences the shear behaviour of the residual soil. It was observed that the contribution of matric suction to the shear strength of the soil varies linearly with matric suction up to the air-entry value, but varies non-linearly with matric suction beyond the air-entry value. Due to the difficulties and limitations in the experimental determination of the shear strength data of the unsaturated soils, empirical equations of 3-D failure envelope were proposed in this paper. Such empirical equations will be adequate for practicing engineers for estimating the shear strength of the compacted residual soil of São Paulo, Brazil.

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