An Efficient Tool to Determine Undrained Shear Strength of Soft Soils

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ABSTRACT: Disturbance encountered when testing soft soils both in laboratory and in-situ conditions makes the determination of the undrained shear strength, S_u, very challenging. This paper introduces a new tool called "Cylindrical Penetrometer" (CP) to measure the undrained shear strength of soft soils. Description of this tool is given, and the related shear test procedure is detailed. The proposed tool offers the advantage to avoid the disturbance of soft soils before the commencement of the CP test. From recorded measurements and based on considerations of the existing shear tests, a specific method of determination of S_u is proposed. The experimental program included laboratory tests by using two sizes of the CP. The recorded results from CP tests, performed on a reconstituted Tunis soft clay, were compared with those obtained from direct shear tests, vane tests, and a consolidated undrained triaxial test. A fair agreement was found between the Cylindrical Penetrometer results with those obtained from the current shear tests.

KEYWORDS: Shear strength, Cylindrical penetrometer, Experiments, Soft soil, Validation.

1. INTRODUCTION

The undrained shear strength of soft soils represents a key parameter for studying the stability of retaining structures, slopes, and predicting the bearing capacity of foundations in short-term condition (Schofield and Wroth, 1968; Gregersen and Loken, 1979).

The special structure of soft soils renders their response quite dependent on the conditions of shear tests (Schofield and Wroth, 1968). Elsewhere, numerous investigations reported that soft clays exhibit structural alterations while extracted in-situ as well as during their extrusion from a sampler. As a result, strength parameters determined from laboratory tests, will be different from those determined from in-situ tests (Bobei and Locks, 2013). Therefore, the question is to suggest a reliable estimation of the undrained shear strength of soft soils from the results of laboratory and in-situ tests.

The accuracy of S_u measurement is affected by three influencing factors: the disturbance, the rate of loading (shear test), and anisotropy.

Regarding the disturbance when performing a laboratory test, it occurs during the preparation of the sample before starting the test. Namely, the extraction from a sampler and the cut of the specimen for installation on the shear apparatus represent the main causes of the induced disturbance.

Similarly, when preparing for in-situ tests, the induced soil disturbance takes place during the boring phase. That is the typical case of the pressuremeter test before the measurement of the limit pressure in the shear phase. Therefore, the induced disturbance during an ordinary pressuremeter test affects the determination of the limit pressure from which one deduces the undrained shear strength of soft soils.

The loading rate factor controls the shear phase. In current soil mechanics dealing with saturated clays, the short-term resistance is determined by performing shear tests at a rapid loading rate. In turn, the long-term resistance is determined at a quite slow loading rate. For a better understanding of the loading rate, many investigators attempted to establish the conformity between the obtained results from in-situ and laboratory tests (Mesri, 1989; Peuchen and Mayne, 2007). Indeed, in early soil mechanics investigations, geotechnical engineers used to consider a unique value of the undrained shear strength mobilized either in a full-scale soil failure or as observed during laboratory tests.

The anisotropy also affects the determination of the undrained shear strength of soft clays. Depending on the assumed failure surface during the shear phase, the mobilized undrained shear strength along a horizontal failure surface differs from the one developed along the vertical failure surface. By taking into account the soil anisotropy, using a correction factor, Wang et al. (2008) introduced the average mobilized undrained shear strength S_u from triaxial compression and extension tests to estimate the undrained shear strength of soft clays and silts.

Further, instead of overcoming related laboratory issues, engineers preferred in-situ tests to estimate the undrained shear strength of soft soils. For instance, they opted to the use of the vane shear test since it is simple and cost-effective. In view of providing a reliable estimation of the undrained shear strength, several contributions aimed to overcome the disturbance accompanying the vane insertion (Westerberg et al., 2015). To ovoid the overestimation of the shear strength, various correlations were proposed; for instance, by Mesri and Huvaj (2007).

Elsewhere, to avoid the use of correlations, which often apply for a certain soft soils type, a non-overestimated undrained shear strength from the vane test can be determined with a limitation of the recorded torque by capturing the soil failure in the range of small strains (Bouassida and Boussetta, 1999; Bouassida, 2006). This approach revealed later on, also applicable to in-situ vane test results (Bouassida and Azaiez, 2018). Despite of this, it is important to note that the induced loading during the vane shear test does not reflect the soil failure surface in field conditions, particularly the assumed cylindrical failure attributed to the blade rotation (Mesri and Huvaj, 2007). Engineers and researchers also attempted to exploit pressuremeter measurements by establishing correlations between the limit net pressure and the undrained shear strength. Frikha et al. (2013) showed that correlations derived from the pressuremeter data mostly overestimate the undrained shear strength of soft soils.

To overcome this limitation for determining the undrained shear strength of soft soils, Bouassida et al. (2022) patented the Cylindrical Penetrometer (CP) test and then presented the obtained results with a reliable measurement of the undrained shear strength of Tunis Soft Clay (TSC).

This paper reports the results obtained from an experimental program aiming to validate the reliability of the Cylindrical Penetrometer (CP) test. The experimental program included the reconstitution of Tunis Soft Clay within consolidation cells of different sizes. This reconstituted soft soil was subject to the CP shear tests, direct shear tests, and vane shear tests to compare different methods of measurement of the undrained shear strength of Tunis Soft Clay.

Interpretation of the results included the determination of the parameter controlling the increase in the undrained shear strength λS_u measured from CP tests with the variation of the consolidation stress.

2. STATEMENT OF THE PROBLEM

North and South Lakes of Tunis City are the most problematic construction sites, in terms of ground conditions, due to the presence of deposited sedimentary soil of the recent quaternary age. Kaàniche et al. (2000) reported that the geological formation of those deposits is Mio-Pliocen clays.

Characterization of such type of soil is delicate because of its weak consistency and strength parameters. In this regard, a big interest was shown after several studies conducted at the soil mechanics laboratory of the National Engineering School of Tunis (Bouassida, 1996; Tounekti et al., 2008; Touiti et al., 2009; Mezni and Bouassida, 2019). Along with these contributions, numerous experimental theoretical and practical investigations on Tunis Soft Clay (TSC), have focused on its behavior, as well as its improvement using different techniques (Bouassida and Porbaha, 2004; Bouassida and Klai, 2012, Jebali et al., 2017, Tabchouche et al., 2017).

From several earlier works: Bouassida (2006), Bouassida and Klai (2012), Klai and Bouassida (2016), and Mezni and Bouassida (2019), the characterization of Tunis soft clay is summarized by the following properties and geotechnical parameters. In areas nearby, the North and South Tunis Lakes, the soil profile shows the first Tunis soft clay horizon from 2 m to 25 m depth and then appears a dense sand layer of thickness approximating 5m. Then, the second Tunis soft clay layer is crossed over a variable thickness attaining a depth from 45 to 60 m with respect to the ground level.

Gradation curves obtained from several experiments indicated that the dimensions of more than 90% of particles are lesser than 80 µm, the clay fraction varies between 30 to 50%, the liquid limit and the plasticity index are in the ranges of 43% to 92%, and 17% to 51%, respectively (Touiti et al., 2009). The total unit weight is around 17 kN/m^3, and the water content varies between 32% and 86% (Touiti et al., 2009). According to the USCS, Tunis soft clay is classified CH.

In continuation of the previous contributions on TSC, a new investigation started since 2016 to find out a reliable tool-method for determining the undrained shear strength of soft soils. Thus, it resulted the proposal of the CP test. This paper aims to present, in details, the CP, the procedure of the CP test, and, finally, to introduce the method of a reliable determination of S_u.

3. RECONSTITUTION OF TUNIS SOFT CLAY (TSC)

Soil reconstitution included the preparation of specimens and their consolidation in specific cells. The experimental investigation started with sampling TSC blocks at 35 m depth at J. Jaures Avenue, located in Tunis City. The grain size distribution showed that dimensions of 98% of particles are lesser than 80 µm (Jebali et al., 2017).

In order to guarantee both the saturation and the weak consistency of the reconstituted soil, the fraction of fine particles of dimensions lesser than 100 µm was hydrated at a water content equals 1.25 to 1.5 times its liquid limit. Final step comprised the fill in and a smooth vibration of the slurry in a consolidation cell. This typical procedure of soil reconstitution enables obtaining samples of TSC with a uniform soil texture and well-controlled physical parameters, especially the water content (Bouassida, 1996).

Two types of consolidation cells were prepared. First, two big consolidation cells made up of epoxy resin material denoted C_1 and C_2 of inner diameter $D_{in} = 19$ cm and height $H_t = 45$ cm. Figure 1a illustrates such a cell mounted to the loading frame of an oedometer apparatus to ensure the consolidation of the reconstituted soil. Second, four small cells denoted, C_3, C_4, C_5, C_6 of height H_t = 10.5 cm and inner diameter D_{in} = 7.1 cm were also prepared to consolidate the reconstituted soil as shown in Figure 1b.

At the bottom of each consolidation cell, when the reconstituted specimen is loaded, water seepage occurs through holes within a metallic or plastic perforated porous plate. Bouassida and Boussetta (1999) proposed this drainage procedure during their investigation on a reconstituted TSC.

(a)

(b)

Figure 1 Consolidation of Tunis soft clay

Table 1 shows the recorded parameters of the reconstituted TSC used for the present study and those obtained by Bouassida (1996). Incremental applied load to the cell C_1 produced a vertical stress equals 30 kPa, whilst for the cell C_2, it reached a maximum applied stress equals 60 kPa. In turn, for cells C_3, C_4, and C_5 the maximum applied vertical stress was 300 kPa. The maximum applied vertical stress increased up to 100 kPa for the cell C_6.

For each consolidation stress level, average values of the water content and total unit weight were recorded from measurements taken from all tested consolidation cells. The Tunis Soft Clay reconstituted by Bouassida (1996) was extracted at 5 m depth from the Tunis' North Lake, for which the incremental load reached 60 kPa. Its grain size distribution is formed by 100% of particles of dimensions lesser than 80 microns, 87% of dimensions lesser than 20 microns, and 55% particles of dimensions lesser than 2 microns. It is a highly plastic clay.

Worth noticing that the physical parameters of the reconstituted TSC in the present study are quite similar to those proposed by Bouassida (1996).

Table 1 Reconstituted Tunis soft clay parameters

Specimen	C_1	C_2	C_3	C_4	C_5	C_6	Bouassida (1996)
Total unit weight (kN/m^3)	17.0	17.0	16.7			15.6	16.6
Water content (%)	51.3	43.7	32.6			39.8	51.0
Average water content (%)	-	-	34.6			-	-
Specific gravity	2.8						2.6
Liquid limit (%)	55.0						73.0
Plasticity index (%)	27.4						47.0
Consistency index I_c	0.14	0.41	0.82			0.55	0.47
Average consistency Index I_c	0.75						-
Consolidation stress σ_c (kPa)	30	60	300			100	-
Undrained shear strength (kPa)	-	-	-			-	8
Undrained friction angle (°)	-	-	-			-	3

The recorded plasticity index of the two tested reconstituted soft soils confirms its classification as a high plastic clay. Besides, due to the recorded negligible undrained friction angle, the characterization of TSC is a purely cohesive soil with a very low undrained shear strength i.e., less than 12 kPa, (Bouassida and Hazzar 2008).

4. TESTING METHOD

Figure 2 displays the designed Cylindrical Penetrometer (CP) to provide a direct measurement of the undrained shear strength, in comparison to the existing testing methods, especially for soft soils (clays). Note that, before starting the CP test, there is no action to apply to the specimen. Therefore, the CP penetration into the soil (the specimen) corresponds to the applied loading in the shear phase. Hence, the CP test is a one-phase shear test during which the soil disturbance does not take place before the test.

Figure 2 Designed CP: small CP (right side) and big CP (left side)

4.1 CP description

Two sizes of the CP have been manufactured: a big CP and a small CP for which the dimensions are as follows. The big CP has an outer diameter D_{out} = 63.6 mm, an inner diameter D_{in} = 60.5 mm, and a height of 11.0 cm. The small CP has an outer diameter D_{out} = 38.0 mm, an inner diameter D_{in} = 35.2 mm, and a height equals 7.0 cm. The proposed tool is a thin hollow cylindrical tube with a sharpened tip over a short distance: d_0 = 5 mm. Such a shape facilitates the penetration of the CP into the soft soil, at a prescribed vertical penetration rate, over a distance: $d_0 \leq d \leq d_f$ (Figures 3a, 3b, & 3c).

d_0 and d_f are the initial penetration and the final penetration of the CP into the sample; d is the current penetration of the CP during the shear test.

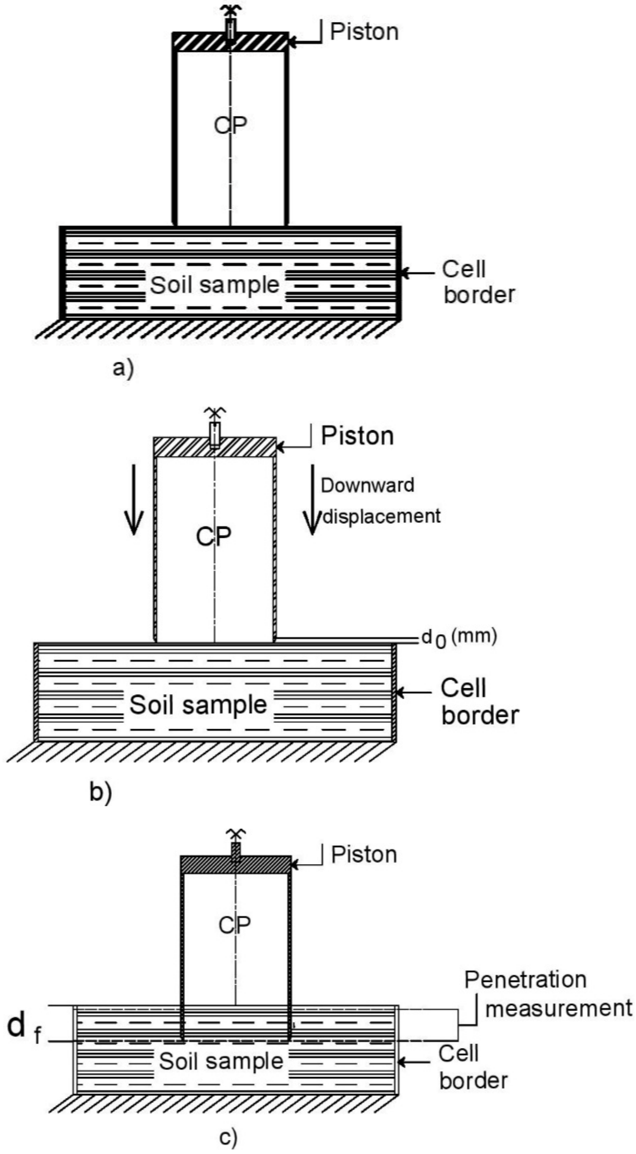
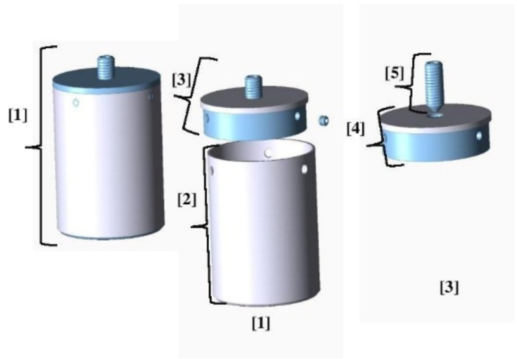


Figure 3 CP test procedure: (a) positioning of the Cylindrical Penetrometer, (b): initial CP penetration (before measurement), (c): final CP penetration (end of measurement)

Figure 4 shows that the CP [1] comprises a cylindrical tube [2] fixed to the piston [3] by three headless socket screws. The latters are positioned along three equal bows around the bottom side of the piston (Bouassida and Azaiez, 2021).

The piston [3] is composed by an annular ring [4] that transmits the soil reaction against the imposed vertical displacement by means of a headless socket screw [5] fixed to the load cell (Figure 4).



[1] CP tool - [2] cylindrical tube - [3] piston - [4] annular ring - [5] pressure screw

Figure 4 Components of the designed CP

4.2 Experimental investigation

4.2.1 Preparation of samples

The consolidation cells C_1 and C_2 filled out by the consolidated TSC have total thicknesses equal 37.2 cm and 34.0 cm, respectively. The cell C_1 was cut into three portions. Only results of the two bottom portions from the cell C_1 were reported. The consolidation cell C_2 was cut into two portions. Each portion served to carry out shear tests on both its upper and the bottom sides. Table 2 summarizes, for each cell, the performed shear tests.

The upper and bottom sides of the reconstituted soil portions are denoted: 1US = upper side of the first portion; 1BS = bottom side of the first portion; 2US = upper side of the second portion; 2BS = bottom side of the second portion; 3US = upper side of the third portion; 3BS = bottom side of the third portion.

Figure 5 shows that two tests were performed, using the small CP, and one test using the big CP on each side of a portion from C_1
The upper side of a portion from the cell C_2 served to preform two small CP tests and one vane shear test.

The bottom side of a portion from the cell C_2 served to perform a CP test and to extract a sample to perform a direct shear test.
The small cells C_3, C_4, C_5 and C_6, served for performing tests using the small CP (Table 2).

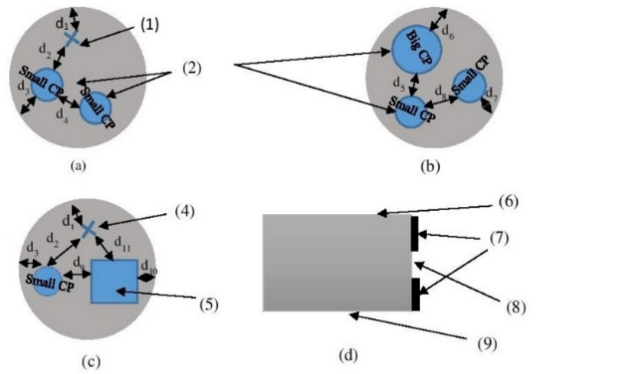
4.2.2 Performing the CP test

Figure 6a shows a portion of the reconstituted sample “1” cut from a big consolidation cell positioned on the loading frame of a triaxial device “2”. The triaxial frame is equipped by an s- type load cell”3” of 2 kN capacity and a displacement transducer “4”, VJT0271 of 25 mm travel distance. This latter records the penetration of the CP “5” when pushed upward toward the sample. The tip of the displacement transducer is positioned on a horizontal support attached to the sample “1”.

A GDS lab software controls all data acquisition. Prior to the test, one check, on the motor drive that the prescribed displacement rate is 1.25 mm/min satisfying the undrained shear condition. After checking the GDS lab connection, the CP test starts by the penetration of the sharpened tip of the tool into the sample, and then, the re-initialization of all transducers’ readings to zero to pursue the CP test.

Table 2 Shear tests performed on the samples of TSC

	Cell designation							
Shear test	C_1		C_2		C_3	C_4	C_5	C_6
CP	Small CP	Big CP	Small CP	Big CP	Small CP	Small CP	Small CP	Small CP
	8	4	6	0	1	1	1	1
Vane	0		4		0	0	0	0
Direct shear	3		4		0	0	0	0



(a) Cross section from an upper side of a specimen portion from C_2: (1) Vane test measurement, (2) CP tests.
(b) Cross section of a specimen portion from C_1: (2) CP tests.
(c) Cross section from a bottom side of a specimen portion from C_2: (4) Vane test measurement, (5) Sample for direct shear test.
(d) Side view of a cell portion: (6) Upper side of a portion (US), (7) 3 cm (C_1) or 5 cm (C_2) height used for laboratory vane test and /or CP test, (8) 7 cm (C_2) to 11 cm (C_1) of intact soil used for direct shear test, (9) Bottom side of a portion (BS).

Figure 5 Location of shear tests performed on a portion from cells C_1 and C_2

d_1 = Distance between the vane shear test and the cell border: 1 cm to 2 cm.
d_2 = Distance between the vane shear test and a CP test performed with the small CP: around 7 cm.
d_3 = Distance between a CP test and the cell border: 2.5 cm to 3.5 cm.
d_4 = Distance between two CP tests performed with the small CP, around 5.5 cm.
d_5 = Distance between two CP tests performed with the small CP and big CP: 4 cm to 5 cm
d_6; d_7 = Distance between the CP test and the cell border of 1.5 cm to 2.5 cm
d_8 = Distance between two CP tests performed with the small CP: 4 cm to 5.5 cm
d_9 = Distance between the implementation of the cutting ring and the cell border: from 1 cm to 1.5 cm.
d_{10} = Distance between the sample prepared for the direct shear test and the CP test about 3 cm
d_{11} = Distance between the sample prepared for the direct and the vane shear tests measurements: from 5 cm to 6.5 cm.

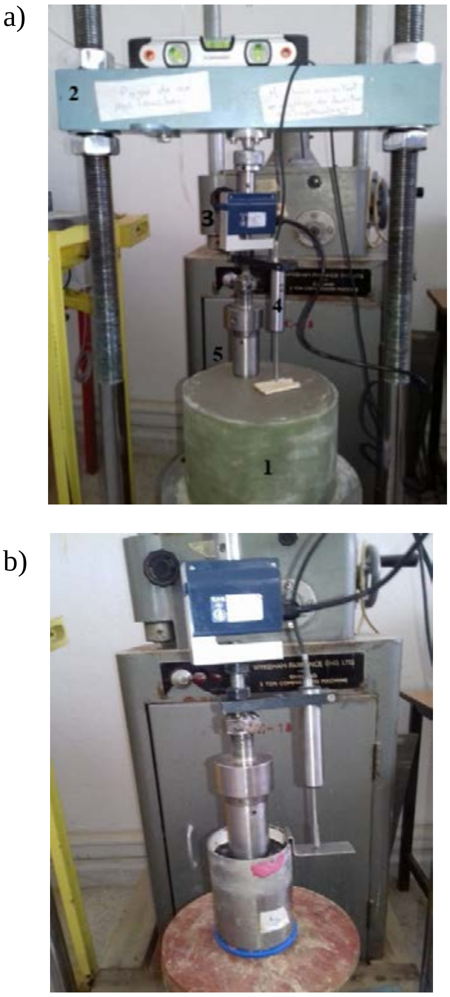


Figure 6 (a) CP test performed on a portion of a big consolidation cell, (b) CP test performed on a small consolidation cell

5. METHOD OF S_U DETERMINATION FROM THE CP TEST

According to the French standard, NF–P 94, from the direct shear test, soil resistance is determined in the range of a horizontal displacement less than or equal to 5 mm in the absence of the peak of load-displacement curve. Therefore, the soil-failure shear strength requires a limitation on the horizontal displacement of the shear box. For estimating the undrained shear strength from the direct shear test, Westerberg et al., (2015) proposed a limitation by setting a maximum distortion angle of 0.15 radians, thus, maximum values of S_u are recorded between 0.10 and 0.15 radians. In their study, they used the shear test as a referential test to calibrate the empirical factor when determining the undrained shear strength from the field vane test, cone penetration test, and the fall cone test.

Earlier, using a similar approach, Bouassida and Boussetta (1999) suggested a limitation on the rotation of the vane blade to determine the maximum torque, and consequently, the estimated undrained shear strength of soft clays.

It proved the convenience of determining the soil undrained shear strength in the range of small strains. Hence, compiling data from the vane shear test, one can point out the difference between S_u determinations depending, or not, on the range of induced strains.

From Figure 7a, dependent less of the blade rotation value, the average peak value of the recorded torque (e.g. S_u) is 0.45 N.m as determined in the range of the blade rotation from 16.5° to 30°. Meanwhile, if consider the limitation on the normalized blade rotation: α (°)/90° ≤ 9%, the average maximum torque approximates 0.35 N.m. Hence, if the condition of small strains does not prevail, it

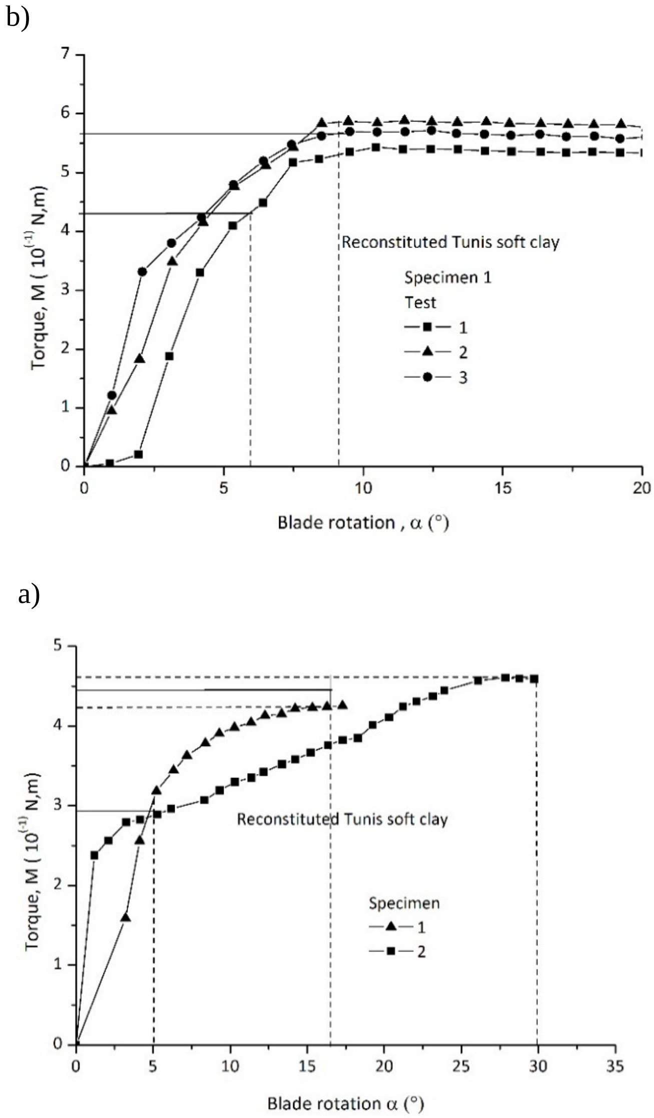


Figure 7 Evolution of the torque vs. the rotation of the blade (Bouassida and Boussetta, 1999)

results an overestimated S_u value by 28.5%, that is non-negligible. Elsewhere, from Figure 7b, considering the same limitation on the normalized blade rotation, α° /90° ≤ 9%, one can suggest two S_u estimations corresponding to the blade rotation values, namely 6° and 9° with respective torque values 0.48 N.m and 0.55 N.m. Thus, one obtains a non-negligible relative difference between the two torque values, i.e., 14.58%.

In addition, when introducing a limitation on the blade rotation, as suggested by Bouassida and Boussetta (1999), this shall allow a correction method for the vane test, which applies to any type of soft soil. Indeed, the correction factor proposed earlier by Bjerrum to avoid an overestimated undrained shear strength was revealed non-applicable to Japanese marine clays (Tanaka, 1994).

Later on, Azaiez et al. (2018) implemented the proposed approach by Bouassida and Boussetta (1999) to interpret in-situ vane test data. From this work, the limitation on the rotation of the vane blade in a prescribed range revealed satisfactory to avoid an overestimated undrained shear strength of river sediments.

When running the CP test, the imposed rate of vertical displacement (penetration d) is identical to the imposed rate of horizontal displacement during the direct shear test. From Figure 8, the sample distortion (shear deformation) is equal to $\frac{2d}{D_{int}}$.

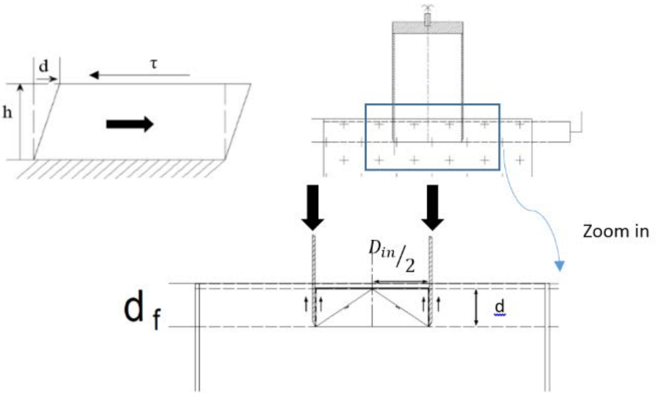


Figure 8 Similarity of shear strains during the DST and the CP test

Based on this consideration, when running the CP test, a limitation of the tool penetration should apply to measure the ultimate vertical force, and then to determine safely the undrained shear strength. This is applicable when the evolution of the force P, during the CP test, does reach a peak value. Hence, the mobilized soil shear strength does not always correspond to the peak of stress-strain (or force-displacement) curve recorded from any shear test (Bouassida and Boussetta, 1999; Bouassida, 2006). Worth noted that the limitation of the penetration d of the CP into the soft soil also applies for the sheared soil-CP contact area, A_{sh}, is given by Equation (1):

$$A_{sh} = \pi \left(D_{in} + D_{out}\right) d \qquad (1)$$

where π is the constant defined as the ratio between a circle diameter and its perimeter.

The developed shear strength over the area A_{sh} depends on the adhesion and the frictional angle of the interface existing between the CP and the sheared soil. It is noted that the soil-CP outer contact area, "π d D_{out}" is visible to the operator.

But the assumed internal contact area "π d D_{int}" is not visible. Hence, confirmation of such an assumption is necessary by observing the behavior of the sheared soil inside the cylinder to avoid any underestimation of the S_u value from the CP test.

Unfortunately, the present designed version of the CP does not allow observing what happens inside the cylinder. In undrained conditions, for soft soils (e.g., soft clays), those interface failure parameters reduce to the undrained shear strength since their undrained friction angle is nearly zero. Table 1 confirms those undrained shear strength parameters recorded for Tunis soft clay.

The manufactured cylindrical penetrometer is made up of a referenced 316 commercial steel of a roughness equal 0.8 microns. Thus, when penetrating a soft clay, the developed resistance along the soil-CP interface reduces to the S_u in short-term condition.

Using the Mohr-Coulomb law, it is almost obvious to consider the full shear strength value since the cylindrical shaft area is uniform over depth. Therefore, there is no need to introduce a factor of correction on the shear strength similar to that considered for the tip resistance measured from the cone penetration test over a non-uniform conical contact area within the penetrated soil.

Figure 9 depicts the recorded vertical force P versus the CP penetration in the third portion of the cell C_1. This figure shows the method of determination of the ultimate force, P_{ult}, to estimate the undrained shear strength of the tested soft soil S_u^{CP}. The value of P_{ult} corresponds to the starred dot that intersects the first non-linear portion of the force-penetration curve and its asymptotic and quasi-linear portion. Using the captured value P_{ult} and by taking account of Equation (1), the undrained shear strength is calculated from Equation (2) in which d_{ult} denotes the penetration corresponding to the captured P_{ult} value on the force-penetration curve shown in Figure 9.

$$S_u^{\,cp} = \frac{P_{ult}}{\pi(D_{in}+D_{out})\, d_{ult}} \qquad (2)$$

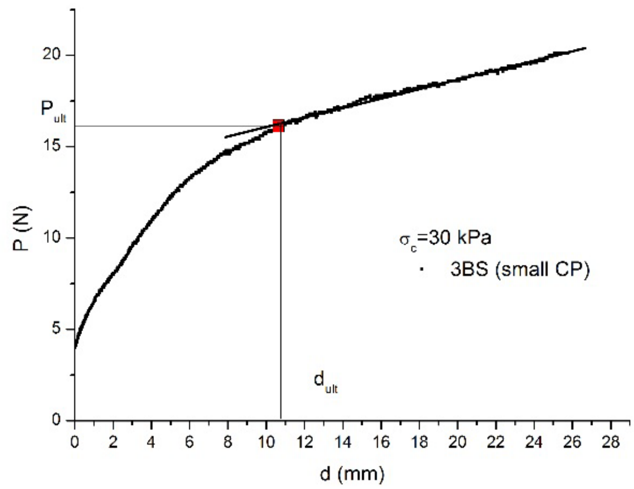


Figure 9 Variation of the vertical force *P* vs. the penetration d in the third portion of the consolidation cell C_1

The curves showing the evolution of the recorded force P versus the CP penetration d are detailed as follows.

Figures 10 and 11 correspond to measurements recorded with the consolidation cell C_1. Whilst Figures 12 and 13 correspond to measurements recorded with the consolidation cell C_2. It is noted that the evolution of the force P versus the penetration, d, in Figure 12 depicts a peak value denoted P_{ult}.

Figure 14 shows the evolution of the force P versus the penetration, d, as recorded for cells C_3, C_4 and C_5 consolidated under the vertical stress σ_c = 300 kPa.

Besides, Figure 15 shows the evolution of the force P versus the penetration d, recorded for the cell C_6 consolidated under the vertical stress σ_c = 100 kPa.

One notes similar evolutions for the majority of measurements obtained from the CP tests.

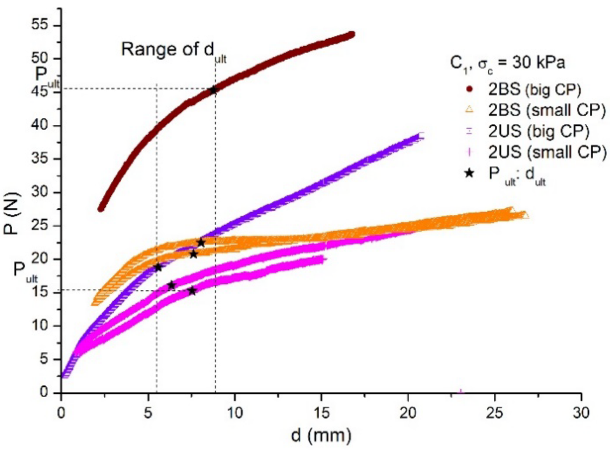


Figure 10 Evolution of the vertical force P vs. the penetration d in the second portion of the cell C_1

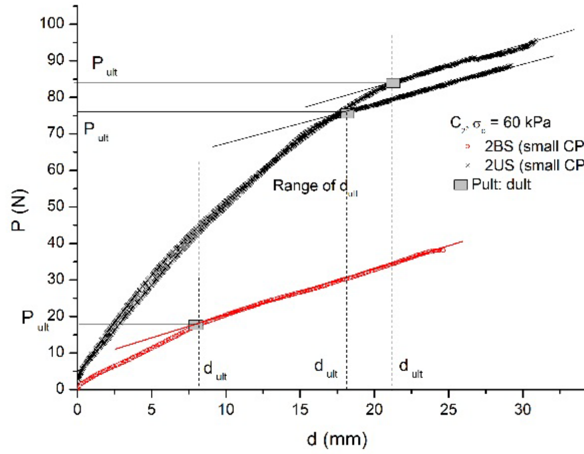


Figure 13 Evolution of the vertical force P vs. the penetration d in the second portion of the cell C_2

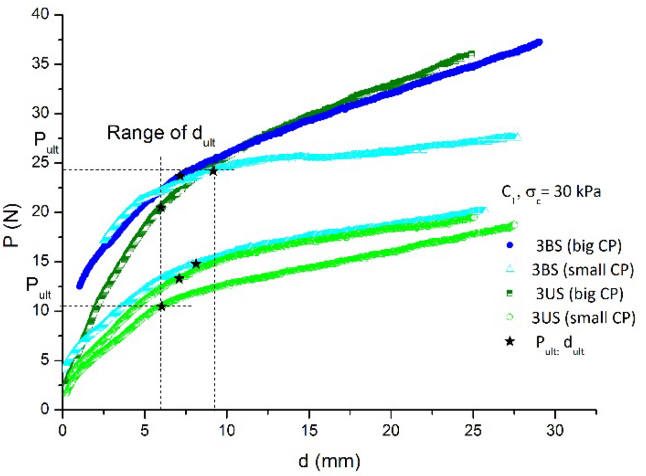


Figure 11 Evolution of the vertical force P vs. the penetration d in the third portion of the cell C_1

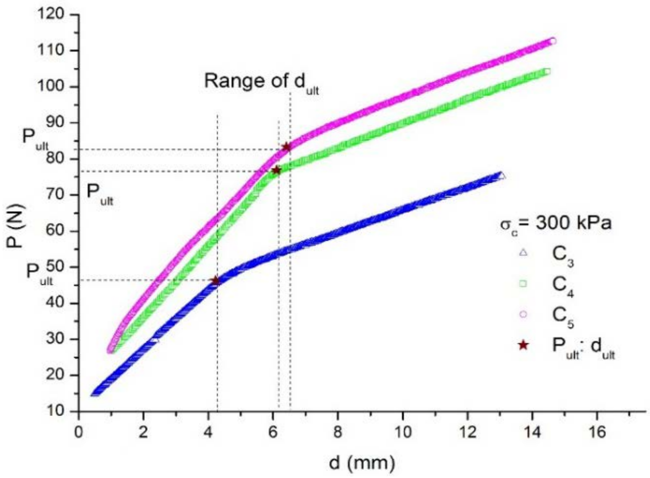


Figure 14 Evolution of the vertical force P vs. the penetration d for cells C_3, C_4, and C_5

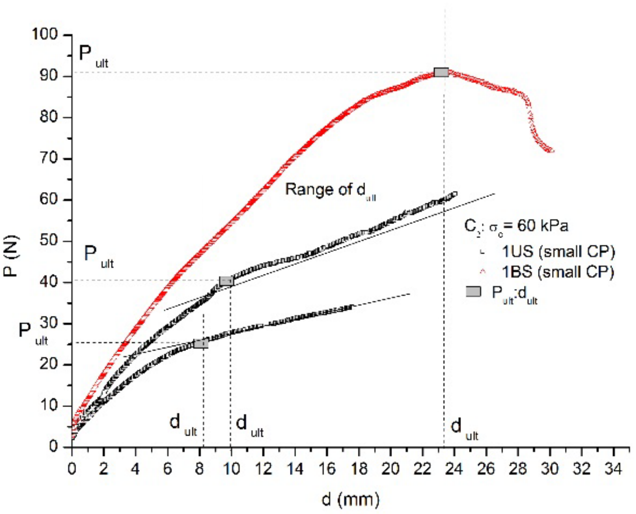


Figure 12 Evolution of the vertical force P vs. the penetration d in the first portion of the cell C_2

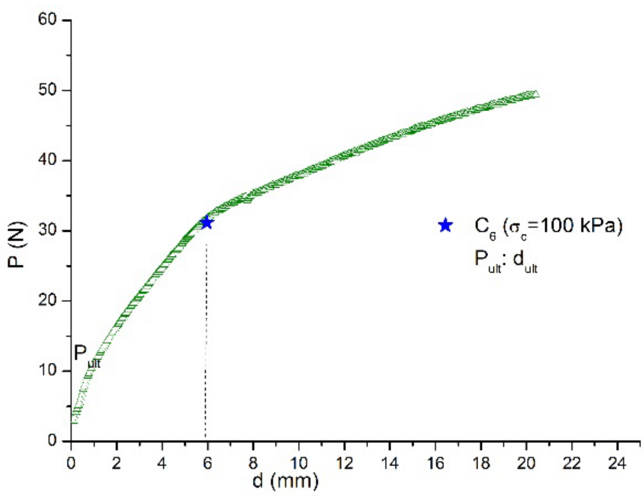


Figure 15 Evolution the vertical force P vs. the penetration d for the cell C_6

6. SHEAR TEST RESULTS

Table 3 presents the recorded ultimate vertical force P_{ult} and the corresponding estimations of S_u, from the CP tests (S_u^{CP}) and the results of DST (S_u^{DST}), all carried out on sampled portions from the reconstituted soft soil in the consolidation cell C_1. Assessment of the proposed method to determine the undrained shear strength of TSC from the twelve performed CP tests in the consolidation cell is processed. First, one determines the average of S_u^{CP} values obtained from the CP tests performed on each side of a cell portion. Table 3 displays those values, i.e., 9.4 kPa, 12.5 kPa for the upper and bottom sides of the second portion of the cell C_1, and 7.9 kPa and 9.2 kPa for the upper and bottom sides of the third portion. Second, Table 3 presents S_u^{DST} values recorded from three undrained DST performed on samples cut from the consolidation cell C_1. From Figure 16 showing those S_u values one obtains S_u^{DST} = 9.9 kPa.

Then, using Equation (3), one obtains the calculated relative difference RD between, average S_u values determined from the CP test and the DST.

$$RD = \frac{|S_u^{DST} - S_u^{CP}|}{S_u^{DST}} \tag{3}$$

Consider the consolidation cell C_1, the relative difference between S_u^{DST} and S_u^{CP} equals 0.8% is negligible for the second portion. Meanwhile, the recorded RD for the upper and bottom sides of the third portion were of 24.7% and 12.8%, respectively. Consequently, one can assume that S_u values from the CP test and the direct shear test are in a better agreement for those recorded values for the second portion than those of the third portion of the cell C_1.

Such relative difference between S_u values is acceptable when measuring the undrained shear strength (Van Impe and Verastegui, 2007).

In addition, the TSC subjected to a vertical stress of 30 kPa reveals not fully consolidated.

Measurements from the proposed DST herein, summarized in Table 3, led to values of average S_u^{DST} equal to 9.4 kPa and 10.5 kPa in the second and third portions of the consolidation cell C_1, respectively.

It resulted a first average value S_u^{DST} = 9.9 kPa for the overall reconstituted soil within the used consolidation cells.

From the latter, the average value from CP tests: S_u^{CP} = 9.8 kPa that is in good agreement with the average S_u value recorded from the DST data. Table 4 summarizes the obtained values of the undrained shear strength from the CP test (S_u^{CP}) conducted on the cell C_2 using the proposed method to determine the values of P_{ult} and the corresponding penetration d_{ult}.

Figure 5 shows that for each section from a reconstituted soil portion in the cell C_2, a vane test (ASTM D4648/D4648M-16) and two tests by the small CP were performed.

From Table 5, measured S_u^{CP} values in the upper side of the first portion of the cell C_2 are 17.8 kPa and 13.6 kPa. It resulted the average value S_u^{CP} = 15.7 kPa.

Table 4 Measurements by the small CP performed on the cell C_2

Cell portion	d_{ult} (mm)	P_{ult} (N)	S_u^{CP} (kPa)
1US	9.8	40.0	17.8
1US	8.1	25.3	13.6
1BS	23.1	90.7	17.1
2US	21.2	83.9	17.2
2US	18.1	76.1	18.3
2BS	8.1	18.0	9.7

Table 3 Recorded S_u from the CP tests performed on the cell C_1

Samples and used CP models	d_{ult} (mm)	P_{ult} N)	S_u^{CP} (kPa)	S_u^{CP} (kPa) (Average)	S_u^{DST} (kPa)	Relative Difference (%)	$\frac{S_u^{DST}}{S_u^{CP}}$
2US big CP	5.6	19	8.7	9.4	9.4	0.8	1.01
2US small CP	7.6	15	8.6				
2US small CP	6.4	16	10.9				
2BS big CP	8.8	45	13.2	12.5	-	-	-
2BS small CP	8.1	23	12.4				
2BS small CP	7.6	21	12.0				
3US big CP	6.0	20	8.6	7.9	10.5	24.7	1.33
3US small CP	7.1	13	8.0				
3US small CP	6.0	10	7.2				
3BS big CP	7.2	24	8.6	9.2	10.5	12.8	1.14
3BS small CP	8.1	14	7.5				
3BS small CP	9.2	24	11.4				
Total average				9.8	9.9	12.7	1.16

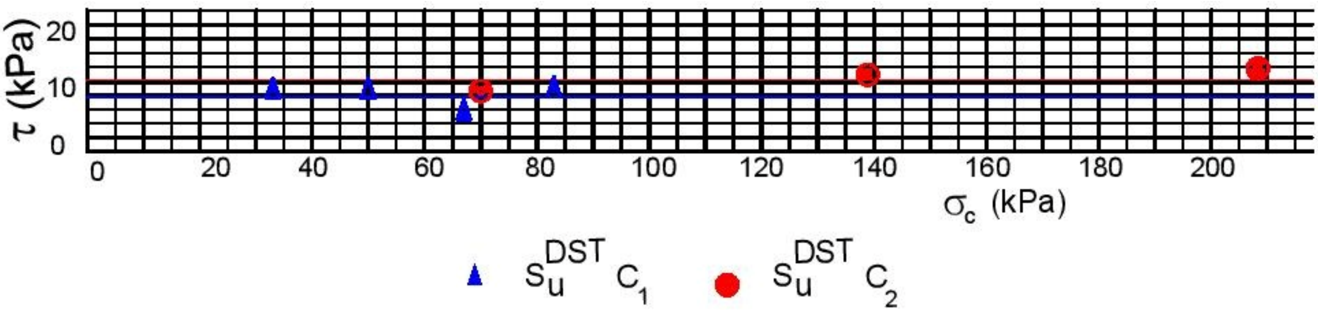


Figure 16 Results of direct shear test performed on cells C_1 and C_2

Table 5 Comparison between S_u values from the CP test, vane shear test and direct shear test measurements with the consolidation cell C_2

Cell C_2 portion	1US small CP	1US small CP	1BS small CP	2US small CP	2US small CP	2BS small CP	Average
S_u^{vane} (kPa)	8.7		13.2	12.3		9.2	10.8
w (%)	45.4		44.3	43.1		49.3	-
S_u^{CP} (kPa)	17.8	13.6	17.1	17.2	18.3	9.7	15.6
S_u^{CP}, (average) (kPa)	15.7		17.1	17.8		9.7	-
w (%)	43.1		44.3	45.4		48.7	-
$\frac{S_u^{vane}}{S_u^{CP}}$	0.55		0.77	0.69		0.95	0.72
RD (%)	79.8		29.5	44.3	-	5.5	48.9
S_u^{DST} (kPa)	10.7		-	13.6	14.7	9.6	12.2
$\frac{S_u^{DST}}{S_u^{CP}}$	0.68		-	0.77	0.89	0.99	0.78
RD (%)	46.40		-	23.30	20.50	0.40	28.20

Consider the measured S_u^{vane} (undrained shear strength measured using the laboratory vane shear test) and S_u^{CP}, the relative difference is RD = 80%; the corresponding ratio is $\frac{S_u^{vane}}{S_u^{CP}} = 0.55$.

The highest recorded values of S_u from the vane tests (S_u^{vane}) were found in the middle of the consolidation cell C_2 at the bottom side of the first portion and the upper side of the second portion where TSC has consolidated the most. Measured S_u^{vane} were only 0.77 and 0.69 times the measured S_u^{CP}. It has resulted the relative differences 30% and 45%, respectively.

The middle portion from the cell C_2 also holds the highest recorded value of S_u^{CP}. It is worth to emphasize that the obtained peak force evolution happened at the bottom side of the first portion, which coincides with that recorded at the middle of this portion.
Considering the least recorded relative difference in Table 5 between S_u^{vane} and S_u^{CP}, determined at the bottom side of the second portion of the cell C_2 (RD = 5.5%), where $\frac{S_u^{vane}}{S_u^{CP}}$ equals almost one, that corresponds to a quasi-equal water content between 49.3% and 48.7% (Table 5).

Figure 17 shows the variation of S_u values from the DST, vane, and CP tests versus the water content. From this figure, one notices that all the S_u values are obtained for a quasi-constant water content, at the average value of 45%. Besides, from the CP tests the undrained shear strength S_u^{CP} are rather comparable to S_u^{vane} i.e measured from the vane tests. In turn, from the DST, the undrained shear strength of TSC is lower than the recorded values from the vane and CP tests.

Table 5 indicates the location of the performed direct shear test measurements, as the corresponding specimen is retrieved from the indicated portion in the cell C_2. It also compares S_u^{DST} to middle of the cell C_2 corresponds to the highest recorded values of S_u^{DST} (13.6 kPa and 14.7 kPa).

Measurements of the undrained shear strength determined from direct shear test performed on TSC reconstituted in the cell C_2 (Figure 16) vary between shear tests 9.6 kPa and 14.7 kPa.

Therefore, the average undrained shear strength from the direct shear test is S_u^{DST} = 12.2 kPa, which is 0.78 times the S_u^{CP}average. Table 5 shows the corresponding relative difference is RD = 28.2%.

6.1 Estimation of S_u evolution as obtained from the CP tests

Table 6 summarizes all measurements, with consideration of an average value of S_u^{CP} from the performed CP tests on specimens collected from the two big consolidation cells C_1 and C_2 the selected soil strength P_{ult} and the CP penetration d_{ult} adopted for S_u^{CP} estimation for the six small consolidation cells C_3 to C_8 as the resulting S_u^{CP}. Average values of the undrained shear strength (S_u^{CP} average) were also considered for the samples related to their consolidation stress.

Table 6 Undrained shear strength of TSC obtained from CP tests

Cell	σ_c (kPa)	d (mm)	P_{ult} (N)	S_u^{CP} (kPa)	S_u^{CP} (average) (kPa)
C_1	30	--	--	9.8	9.8
C_2	60	-	-	15.6	15.6
C_3	300	4.2	46	47.3	52.8
C_4	300	6.1	77	54.8	
C_5	300	6.4	83	56.3	
C_6	100	6	31	22.6	22.6

The parameter predicting the increase in the undrained shear strength in function of the consolidation stress corresponds to the ratio $\lambda\, s_u$ expressed by Equation (4):

$$\lambda\, s_u = \frac{\Delta s_u(\sigma_c)}{\Delta \sigma_c} = \frac{s_u\,(\sigma_c(i)) - s_u\,(\sigma_c(j))}{\sigma_c(i) - \sigma_c(j)} \qquad (4)$$

λS_u is the ratio of the difference between two S_u values by the difference between the corresponding values of the consolidation stress σ_c. The latter corresponds to an effective stress.

The observed evolution of S_u^{CP} versus the consolidation stress, illustrated in Figure 18, is governed by Equation (5).

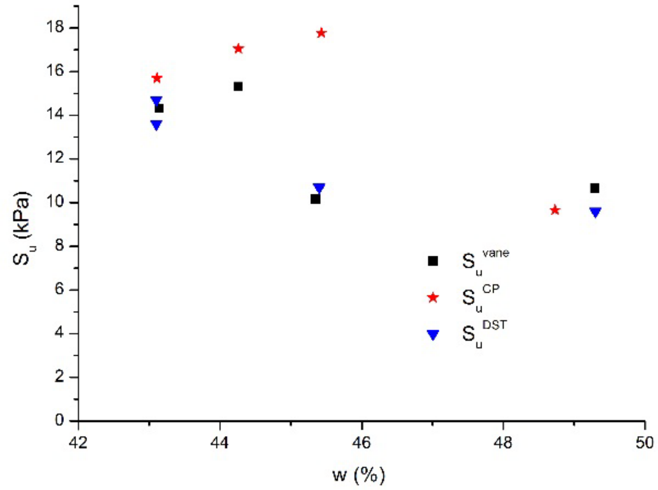


Figure17 Evolution of S_u versus water content from CP tests laboratory vane tests and direct shear tests

$$S_u^{CP}(\sigma_c) = 5.66 + 0.16\ \sigma_c \tag{5}$$

After Equation (5), one deduces that $\lambda S_u^{CP} = 0.16$. Meanwhile,

Earlier, Bouassida (1996) proposed for a reconstituted TSC subjected to an undrained triaxial test $\lambda S_u = 0.27$. Comparing those two values, one concludes that the increase in the undrained shear strength versus the consolidation stress, σ_c, is more pronounced from the triaxial test results than that deduced from the CP test results.

Figure 19 illustrates the variation of S_u^{CP} versus the water content as predicted by Equation (6) with a regression coefficient $r^2 = 0.99$.

$$S_u^{CP}\ (\text{kPa}) = 7.00 + 6095.47 \times\ 0.86^{w}\ ;\ w\ (\%) \tag{6}$$

The evolution of S_u^{CP} versus the water content almost confirms the observed evolution of S_u from the laboratory vane test versus the water content reported by Azaiez et al. (2018). This finding gives more credibility to perform the CP test for estimating the undrained shear strength of soft soils.

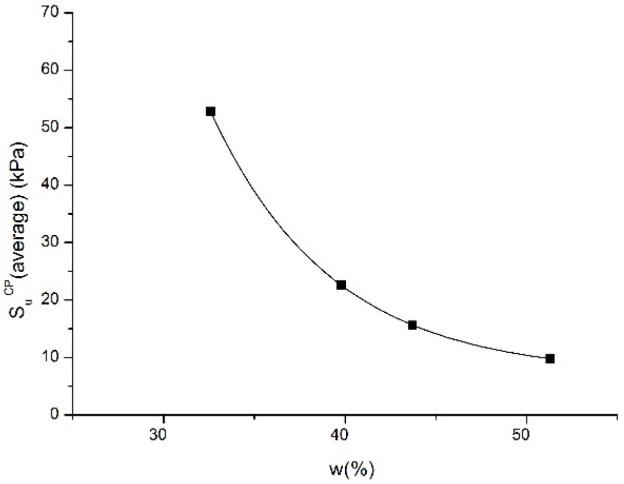


Figure 19 Variation of S_u from the CP tests versus water content

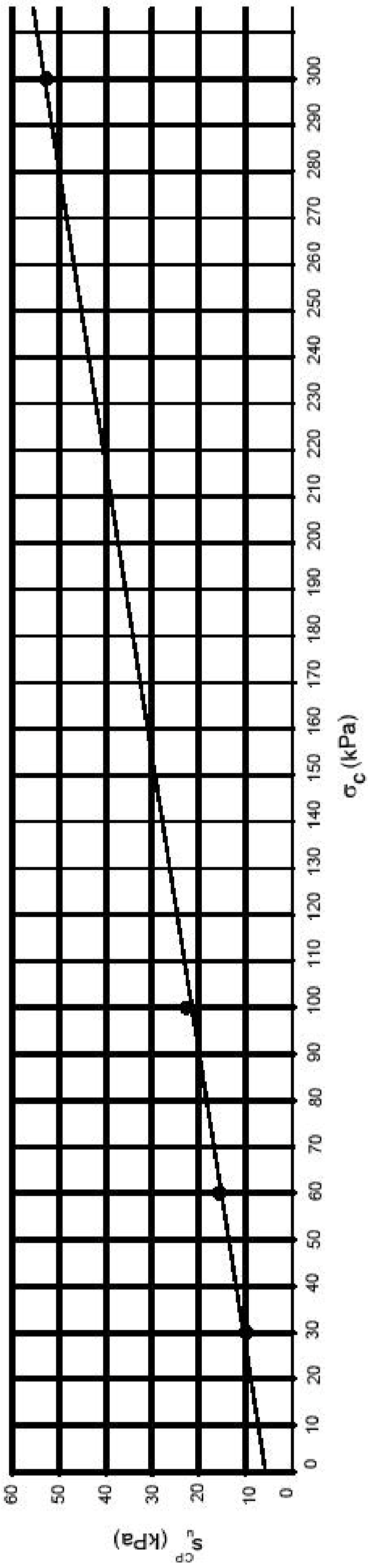


Figure 18 Evolution of the undrained shear strength from CP tests versus consolidation stresses of the tested TSC

7. CONCLUSIONS

The present paper dealt with the determination of the undrained shear strength of soft soils using the called “Cylindrical Penetrometer” (CP). The merit of the newly designed CP was to avoid the soil disturbance that often occurs before the commencement of existing in-situ tests (e.g., vane shear, pressuremeter, etc.) and laboratory tests.

The conducted experimental program first included the reconstitution of Tunis soft clay samples in several consolidation cells. Then, the CP manufactured into two models: a small size and a big size, is described in details. The procedure of the shear test using the CP was introduced, as well as the experiments conducted on reconstituted Tunis soft clay (TSC) samples.

Consider the direct shear test measurement made on the reconstituted specimens and existing TSC data, namely the undrained shear strength determined from a triaxial test, the assessment of CP test results was proceeded.

Note that the measured undrained shear strength does not dependent on the diameter of the CP.

Main finding from the present investigation is that the CP test results underestimate the DST results of TSC when consolidated under 30 kPa, with a relative difference of 12.7%. In turn, results of the CP test overestimate the DST results of TSC when consolidated under 60 kPa, with a relative difference of 28.2%.

The parameter predicting the increase in the undrained shear strength with depth is proposed.

The evolution of the undrained shear strength measured with the CP is correlated to the water content.

The above primary findings suggested from the CP investigations need further assessment by testing other soft soils. Nonetheless, investigations by the CP are in progress to determine the shear strength parameters of other soft soils as well as performing the in-situ CP test.

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