Estimation of Tunis Soft Soil Undrained Shear Strength From Pressuremeter Data

Estimation de la cohésion non drainée de la vase de Tunis à partir des résultats d'essais pressiométriques

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ABSTRACT : The present paper focuses on the prediction of the undrained cohesion from pressuremeter test. For this purpose, using a data base of measured undrained shear strength from laboratory and in situ tests and pressuremeter results, different existing correlations and analytical approaches are considered. Predictions made by those approaches have been interpreted and evaluated. Some recommendations are given to better estimate of the undrained shear strength of Tunis soft soil using results obtained from pressuremeter test.

RESUME : La prédiction de la cohésion non drainée à partir de l'essai pressiométrique constitue l'objectif du présent article. A cet effet, différentes corrélations et approches sont considérées avec une banque de données de mesures de la cohésion non drainée à partir d'essais de laboratoire et d'essais in situ. Les prédictions faites par ces approches et corrélations sont interprétées et analysées. En vue d'une meilleure prédiction de la cohésion non drainée de l'argile molle de Tunis à partir de l'essai pressiométrique quelques recommandations sont formulées.

KEYWORDS : pressuremeter, undrained cohesion, limit net pressure, correlation, analytical approaches, Tunis soft soil
MOTS CLÉS : pressiomètre, cohesion non drainée, pression limite nette, corrélations, étude analytique, argile molle de Tunis.

1 INTRODUCTION.

Geotechnical Campaigns in Tunisia include systematically pressuremeter tests accompanied with core drilled samples (for laboratory tests) and penetrometer tests. However, there is a notable preference to the pressuremeter equipment that is used as basic tool of investigation for different types of foundations. Statistically, the pressuremeter present 50% of the in situ linear meters tested soil. Testing cores drilling only represents 40% and SPT and CPT tests present 10% of usual geotechnical campaigns. (Haffoudhi et al. 2005).

In the literature several correlation relating pressuremeter data and laboratory test results have been proposed. The most common used in Tunisia are correlations between the limit net pressure and Menard's modulus and shear strength characteristics of the soil, ie, cohesion and friction angle. It is found that existing correlations are mostly unreliable, restrictive as to scope, so that estimated characteristics are unrealistic especially when dealing with the Tunis soft clay that is purely cohesive in undrained condition.

Working principle of the pressuremeter consists on transmission of pressure in borehole through a guiding tube. The applied pressure is done using a variation of volume of expandable membrane mounted on the shaft of a cylindrical probe. The test consist on the measurement of the volume of fluid injected into the membrane as a function of the applied pressure and allows the determination of both ultimate pressure p_l and horizontal modulus E_M. The installation procedure has a major influence on the measured expansion curve, precluding direct theoretical interpretation of soil strength and deformation properties. (Aubeny et al. 2000)

The present paper focuses on the interpretation of undrained shear strength from pressuremeter test.

For this purpose, a data base of measurements of undrained cohesion from laboratory tests and pressuremeter results was established. Different existing correlations and analytical approaches were considered to compare between estimates of the undrained cohesion. Some recommendations are suggested to better estimate the undrained shear strength of Tunis soft soil from pressuremeter data.

2 SOFT SOIL OF TUNIS

Tunis soft clay belongs to the category of problematic soils because of its weak strength characteristics and high compressibility. Then, designing foundation on Tunis soft clay requires a thorough study both for the short term behavior and long term behavior.

According to results of classification tests done by Klai and Bouassida 2009) one can conclude that Tunis soft clay shows high proportion of silt and indicates high plastic muddy soil with varied clay fraction. The classification of saturated Tunis soft clay is highly plastic silt or with very low consistency (Bouassida, 2006).

The percentage of organic matter recorded for reconstituted soft clay was about 3.12 %. Undisturbed soft clay has a higher content matter organic than the reconstituted soft clay which informs low compressibility for it (about 10%).

According to results recorded during the geotechnical campaigns carried out within the framework of Radès La Goulette Bridge project (Nippon Koeï et al, 2001), the Tunis soft clay is characterized by variable percentage of organic matter from 0.8 to 22 % (Klai and Bouassida 2009).

The result in compression index and swelling rate indicate that the reconstituted soft clay is, on one hand, highly compressible and, on the other hand, with non-significant swelling potential.

As observed for the Tunis soft clay it is concluded that is slightly under consolidated.

Pressuremeter data are available thanks to several geotechnical surveys, that also included laboratory tests, conducted at several sites located in the north of Tunis City. Details of the geotechnical sites investigation are presented in Table 1.

Table 1 presents an overview regarding investigations made in seven sites located at north of Tunis City. Collected data refer to pressuremeter and laboratory tests results carried out on undisturbed samples of soft soil extracted at depths between 3m to 96m from the ground surface. Laboratory tests included undrained and unconsolidated (UU) triaxial tests and direct shear tests.

The ranges of the undrained cohesion (c_U) and the limit net pressure (p_l^*) were presented as follows.

- Rades site: (GPL center, Parc B, Rades, Bridge lot number IV, Rades - Lagoulette bridge) $8kPa \le c_u \le 142kPa$ and $200kPa \le p_l^* \le 220kPa$. At depths ranging from 3m to 96 m.
- Lyon street: The undrained shear strength varies between 8 to 24 kPa and the limit net pressure is about 180 kPa to 710 kPa. At depths ranging from .8m to .25m.
- North Urban Centre: The undrained shear strength is about 20 kPa and the limit net pressure varies between 720 kPa to 940 kPa.. At depths ranging from 2m to 21m.
- Bouguatfa station: The undrained shear strength is about 42 kPa and the limit net pressure between 740 kPa to 1440 kPa. . At depths ranging from 4m to 7m.

Considering these data, the measured c_u will be correlated to p_l^* using various empirical and analytical approaches.

Table 1: Overview collected data

Site	Number of tests	Laboratory tests
Rades Center GPL	4	Triaxial tests
Rades Park B	9	Direct shear test
Rades bridge lot IV	1	Direct shear test
Rades Lagoulette bridge	22	Direct scchear & trixial tests
Lyon street	8	Direct shear tests
North Urban Centre	4	Direct shear tests
Bouguatfa station	6	Direct shear test

3. EMPIRICAL APPROACHES

Several empirical correlations have evolved by using the pressuremeter data for the determination of undrained shear strength of purely cohesive soils. Main primary investigations have been suggested by Menard (1957), Cassan (1978), Amar and Jézéquel (1972), Marsland and Randolph (1977), Lukas and LeClerc de Bussy (1972), Martin and Drahos (1986) and Amar et al. (1991). The undrained cohesion is derived from a linear relationship with the limit net pressure defined by $p_l^*=p_l-p_0$, p_0 represents the initial horizontal stress at rest before the execution of cavity and p_l is the measured lateral pressure from the pressuremeter test. This relationship writes:

$$c_u = \frac{p_l^*}{\beta} \quad (1)$$

A number of recommendations for the value of β are available. (β=5.5, (Menard, 1957); β=8 and 15, (Cassan , 1978), β =6.8, (Marsland and Randolph, 1977), β =5.1, (Lukas and LeClerc De Bussy, 1976), β =10, (Martin and Drahos, 1986), β =3.3 and 12 (Clarke, 1995). In Tunisia, the most commonly used value of β is 5.5. Bouassida and Frikha (2007), by using limit analysis solution, suggested the value β = 5.4 that was in good agreement with the correlation proposed by “Ménard, 1957”.

Amar and Jézéquel (1972) suggested equation (2) when p_l^*>300 kPa:

$$c_u = \frac{p_l^*}{10} + 25kPa \quad (2)$$

Baguelin et al. (1978) suggested a non linear relationship between c_u and p_l^*:

$$c_u = 0.67(p_l^*)^{0.75} \quad (3)$$

Table 2 summarizes different correlations that depend on the tested soil.

Table 2: Empirical relations between the undrained cohesion and the limit net pressure

c_u	Clay type	Reference
$\frac{p_l^*}{5.5}$	Soft clays	Menard (1957)
$\frac{p_l^*}{8}$	Firm to stiff clays	Cassan (1978)
$\frac{p_l^*}{15}$	Stiff to very stiff clays	
$\frac{p_l^*}{5.5}$	p_l^*< 300kPa	Amar and Jézéquel (1972)
$\frac{p_l^*}{10}+25kPa$	p_l^*>300kPa	
$\frac{p_l^*}{6.8}$	Stiff clays	Marsland and Randolph (1977)
$\frac{p_l^*}{5.1}$	Hard clays	Lukas and LeClerc De Bussy (1976)
$\frac{p_l^*}{10}$	Stiff clays	Martin and Drahos (1986)
$0.67.(P^*_l)^{0.75}$	All clays	Baguelin et al.(1978)
$\frac{p_l^*}{3.3}$	Soft clays	Clarke (1995)
$\frac{p_l^*}{12}$	Stiff clays	

Figure 1 shows the variation of undrained cohesion vs the limit net pressure using different empirical approaches and measurements. It is noticed that the value of the undrained cohesion deduced from empirical approaches is mostly overestimated compared to those measurements from experiments The best correlation result is $c_u=p_l^*/15$.

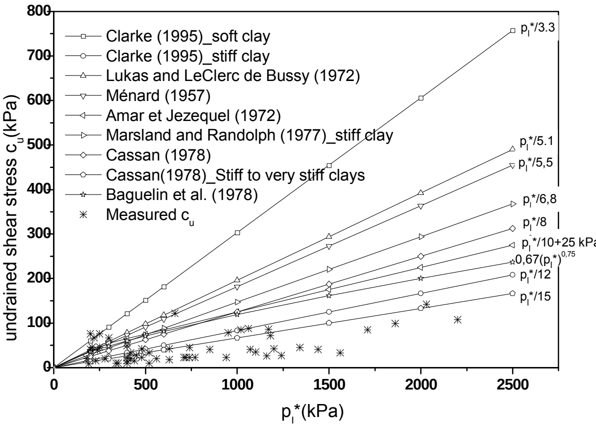

Figure 1: Undrained cohesion vs pressuremeter limit net pressure: comparison between correlations and measurements

In fact, more than 60% of the measured values of the undrained cohesion for clays (with c_u < 50kPa and p_l^* > 300kPa), are less than theoretical values deduced with β = 15. This leads to propose an increase of the value of β in order to give an acceptable estimate of c_u.

In order to locate closely the measured undrained cohesion, one can suggest the range of values β= 8 and β=47 which best fits the measured values as mentioned in figure 2.

Further, on the basis of measurements of the undrained cohesion, four methods of interpolation are suggested in Table 3. It can be observed that all those interpolations do not give an acceptable coefficient R^2 (obtained in the range of 0.16-0.25). If consider the classical empirical correlation given by Eq (1), the best value of β will be β=19.6 which is quite greater than coefficients as suggested by empirical approaches summarized in Table 1.

Table 3: Methods of interpolation

Interpolation	c_u	R^2
Linear	$p_l^*/19.6$	0.1639
Exponential	$20.66\exp(p_l^*/1430)$	0.2515
Logarithmic	18,57 $\ln(p_l^*)$-74.40	0.1745
Power	$2.4(p_l^*)^{0.42}$	0.1691

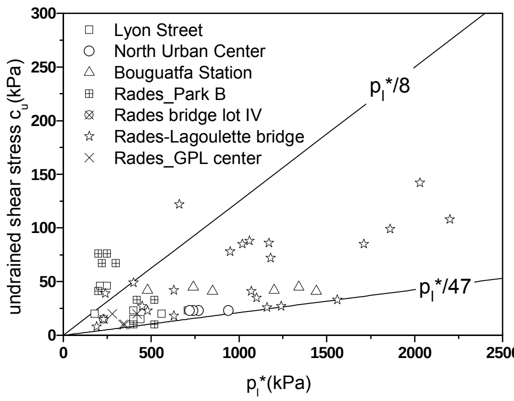

Figure 2: Predicted undrained cohesion of Tunis soft clay

3 ANNALYTICAL APPROCHES

Various methods based on the pressuremeter test are presented to evaluate the undrained cohesion of soft soils.

The analytical methods are developed to determine the limit pressure of cylindrical cavity with assumption of varied behaviour laws of medium around the cavity, linearly elastic (Lamé, 1952) or elastic perfectly plastic without taking into account the volume variation (Hill 1950, Ménard 1957, Gibson et Anderson 1961, Chadwick 1959, Windle and Wroth 1977) or with inducing the volume variation (Ladanyi 1963, Salençon 1966, Vésic 1972, Carter et *al.* 1986, Manassero 1989, Yu et Houlsby 1991,…). Main purpose of those contributions was the prediction of the mechanical characteristics of soils from the pressuremeter data and the prediction of the ultimate bearing capacity of deep foundations.

The analytical contributions were adopting for the soil behaviour two hypotheses whether is the range of small strains (Hill, 1950, Bishop et *al.* 1945, Ménard 1957, Gibson et Anderson 1961, Windle et Wroth 1977, Ladanyi 1963, Vésic 1972, Hughes et *al.* 1977) or by adopting the range of large strains (Chadwick 1959, Carter et *al.* 1986, Yu et Houlsby 1991 …). Also, it is noticed that Hughes et *al.* 1977 suggested to neglect the elastic strains in the plastic zone, in turn Carter et *al*. 1986 justified that for a deformation that exceeds 10%, the elastic strain should not be neglected, as considered by their method.

This section exploits some analytical result to predict the undrained cohesion of the Tunis soft soil. The main parameters used and measurements from the pressuremeter test are the net limit pressure p_l^* and the Menard modulus *E*. the Poisson's ratio is taken equal υ=0.3.

The first equation is derived from the contribution by Menard (1957), Bishop, Hill and Mott (1945) as well as by Houlsby and Withers (1988), using ideal Mohr-Coulomb elastic-plastic assumptions:

$$p_l^* = c_u\left[1 + \ln I_r\right] \tag{4}$$

Where

$$I_r = \left[\frac{E}{2c_u(1+\upsilon)}\right] \tag{5}$$

Cao et *al.* (2002) examined the non-linear elastic response prior to yielding on the expansion of cylindrical cavity in a soil that is modelled as a non-linear modified Cam Clay (MCC) material. Large strain formulation is adopted for the elastic region and the plastic one as well. In the elastic region, the non-linear behavior which corresponds to the variation in stiffness with strain can be expressed by a power-law function or by a hyperbolic stress-strain curve. Using a power law function, the variation of shear stress and shear strain may be expressed as:

$$\tau = G_l \gamma^{\delta} \tag{6}$$

G_l and δ are non-linear elastic parameters with $0 < \delta \le 1$. The secant shear modulus G_s is given by:

$$G_s = G_l \gamma^{\delta - 1} \tag{7}$$

Noted that G_l is equal to the shear modulus of a linear material if δ=1. Based on the power-law function, the limit cavity pressure is (Cao et *al.*, 2002):

$$p_l* = \frac{c_u}{\delta} + c_u Ln I_r \tag{8}$$

Gupta (2000) analyzed the expansion of cylindrical cavity using the classical hypothesis. Only difference is by consider the no volume change in the plastic zone and the radial displacement at the interface of plastic and elastic zone:

$$R^2 - R_0^2 = r_p^2 - \left(r_p - \xi_{rp}\right)^2 \tag{9}$$

R and r_p are respective radii of the cavity and the plastic zone.

$$\xi_{rp} = \frac{r_p}{2I_r} \tag{10}$$

After Gupta (2000) the limit pressure is:

$$p_l* = c_u + c_u \ln\left[4I_r^2/(4I_r - 1)\right] \tag{11}$$

Frikha and Bouassida (2013) generalized the contribution of Salençon (1966) related to expanded cylindrical cavity within a medium governed by an elastoplastic constitutive law with variable flow. This problem was solved by dividing the medium around the cavity in two zones. The first zone, close to the cavity border, is assumed as plastic medium with variable flow law. The second zone is assumed elastic. The plastic zone is divided into "n" flow zones, each one being characterized by its own plastic radius ci and coefficient of compressibility k_i (i = 1, n).

In the present study, the plastic zone only comprises two flow zones (figure 3). In the external zone II, the condition of no volume variation at infinity is assumed (Salençon, 1966), then no plastic volume variation, hence k_2=1. However, the interior plastic zone I is characterized by its potential flow (k_1 = k).

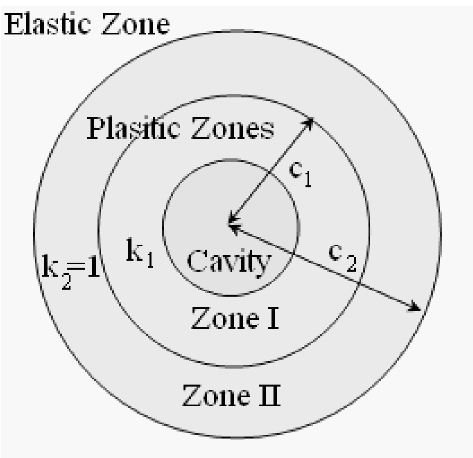

Figure 3. Definition of zones around the cavity subjected to radial expansion (Frikha and Bouassida, 2013)

Further, it is assumed that the ratio of the variation in time of plastic radii $\alpha = \frac{c_1}{c_2}$ is constant. According to these assumptions, in case of purely cohesive soil, the net limit pressure is:

$$p_l^* = c_u\left(1 + \frac{2}{k+1} Ln\left(\frac{E}{4\alpha^{k-1} c_u (1-\upsilon^2)}\right)\right) \tag{12}$$

Where the deformation occurs without volume variation (k_1=k=1), from Equation (1), the limit pressure as found by Salençon (1966) writes:

$$p_l^* = c_u\left(1 + Ln\left(\frac{E}{4c_u(1-\upsilon^2)}\right)\right) \tag{13}$$

The coefficient of compressibility depends on the angle of dilation denoted Ψ and is written:

$$k = \frac{1-\sin\psi}{1+\sin\psi} \tag{14}$$

Interpretations of results:

Taking into account the net limit pressure and the Ménard's modulus measured from the pressuremeter test, the undrained cohesion was predicted by using the approaches presented above. The obtained theoretical values are then compared to the measurements from experiments.

Figure 4 shows the results of undrained cohesion predicted from result of pressuremeter trest and using different approaches proposed by Menard (1957), Salençon (1966), Gupta (2000) and Cao et al. (2002). These predicted c_u are compared with those measured from laboratory test.

Figure 4 clearly indicates that the predicted values of the undrained cohesion using Menard (1957), Salençon (1966) and Gupta (2000) approaches are almost greater than measured values. However, if we compare between these three approaches, it appears that Menard (1957) gives better estimation of the undrained cohesion and the best agreement was observed for measured values of c_u < 50 kPa that corresponds to the category of soft clays. The overestimation is within an average of 26% than measured value.

Referring to Cao et *al.* (2002) approach, the calibration of coefficient δ introduced by Eq (8), under the condition 0 < δ ≤ 1, was carried out. Successive iterations were performed to provide the best fit with measurements of the undrained cohesion. When consider low values of δ, Cao et *al.* (2002) approach leads to more realistic results. The obtained mean value of δ is about 0.16. Accordingly, the undrained cohesion value predicted by using Cao et *al.* (2002) is plotted in figure 4 when the value of the coefficient δ =0.16 is considered. One can notice that a good correlation is observed with measured undrained cohesion less than 50 kPa, but Cao et *al.* (2002) approach still overestimates the realistic value.

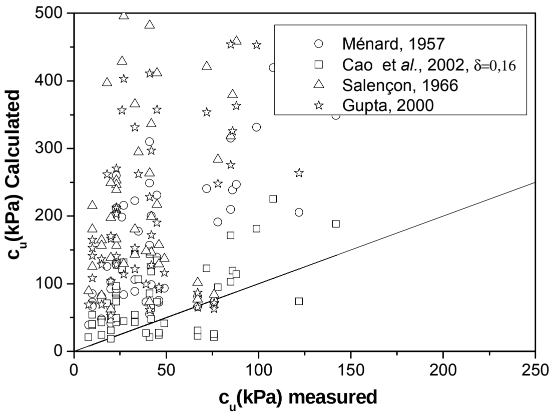

Figure 4: Measured undrained cohesion values vs the theoretical predictions by Menard (1957), Salençon (1966), Gupta (2000) and Cao et *al.* (2002).

Figure 5 presents the prediction of undrained cohesion using Frikha and Bouassida (2013) approach for different values of the parameters α and k (Eq.12). Considering a thin compressible zone around the cavity (Zone I) = 10%, 0.1% and 0.01% of c_2, the parameter α corresponds respectively to 0.1, 0.001 and 0.0001. Considering these values of parameter α the best fit gives, according to Eq (14), a contractive soft soil in Zone I, hence the value of the parameter k must be equal to 1.2 (the dilatancy angle is equal Ψ = -5°). A comparative value of k= 0.9 (Ψ = 4°) is adopted in order to study the effect of the coefficient k. It is mentioned, for k higher than one, i.e. the dilation angle Ψ is negative and the corresponding plasticity is a contractive, which is not compatible with the Mohr-Coulomb criterion. This study presents a case which is beyond the theoretical limit of the plasticity.

It appears from figure 5 that Frikha and Bouassida (2013) approach also underestimates the undrained cohesion. On the other hand, by considering k=1.2, the smallest value of α (=0.0001) leads to better estimation and by reducing the value of k to 0.9, the obtained value of c_u is much higher.

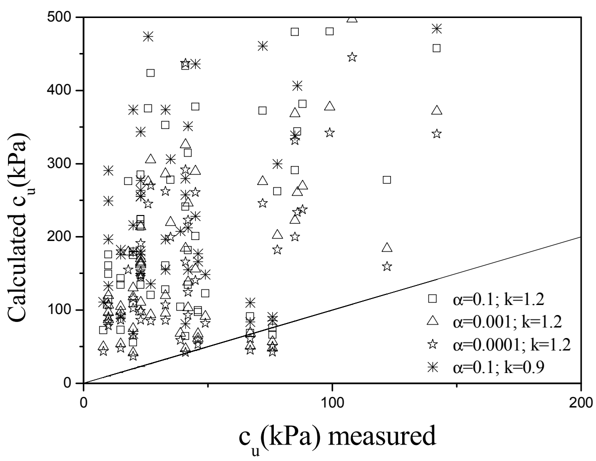

Figure 5: Measured undrainesd cohesion values vs predicted values after Frikha and Bouassida (2013) approach

4 CONCLUSION

The results presented in this paper for both empirical and theoretical results give a poor agreement compared to measured c_u. This disagreement and overestimation can be explained by: the disturbance of different samples between soft and stiff clay, the inaccuracies in evaluating the initial horizontal stress, the non homogeneity of the soil, the sensitivity of the clay, the length to diameter ratio of the pressuremeter probe, the probe anisotropy and the borehole disturbance and unloading. (Baguelin et al. (1978))

The classical correlation of current practice in Tunisia to estimate the undrained shear strength of Tunis soft soil from pressuremeter tests should be adopted carefully. The present study has led to propose an increase of the value of the coefficient β in order to give an acceptable estimate of c_u. One can suggest the range of values β= 8 and β=47 that fits better the measured values.

The suggested analytical contribution, rarely used in practice, also overestimates the undrained shear strength. This paper showed that the hypothesis of nonlinear elasticity does not provide a different estimation compared to the elasticity model. Cao et *al.* (2002) approach still overestimates the realistic value. The approaches of Frikha and Bouassida 2013 can gives a more acceptable result if the zone around the cavity is modeled as contractive soil. This zone must be very thin.
Meanwhile, it is recommended to discuss the results presented in this paper by considering other experimental results available in the literature.

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