Evaluation of undrained shear strength of cohesive and organic soils from *in situ* tests

La resistance au cisaillement non drainée de sols cohésives et organiques determinée par l'essai *in situ*

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ABSTRACT: The evaluation of undrained shear strength τ_{fu} is of crucial importance in field and laboratory investigations. In cohesive and organic soils the in situ tests e.g. cone penetration test (CPT, CPTU), dilatometer test (DMT) and field vane test (FVT) are typically used to develop profiles of undrained shear strength. The paper presents the results of field and laboratory tests performed on the heavily overconsolidated Pliocene clay and boulder clay, as well as organic mud. The problem of accuracy in evaluation of the undrained shear strength of cohesive and organic soils from in situ tests is discussed.

RÉSUMÉ: L'évaluation de la résistance au cisaillement non drainée τ_{fu} est une tâche importante dans les recherches effectuées sur le terrain et en laboratoire. Dans les sols cohésifs et organiques les tests effectués sur place: ex. les tests de pénétration à l'aide d'une sonde conique (CPT, CPTU), test par dilatomètre (DMT) et par bêche sur le terrain (FVT) sont typiquement utilisés pour le développement des profils de résistance au cisaillement non drainée. Cet article présente les résultats d'analyses faites en laboratoire et sur le terrain sur de l'argile de l'ère Pliocéne surconsolidé, de l'argile glaciaire, de la boue organique. Dans la partie finale de l'article une discussion est faite sur le problème de la précision dans l'évaluation de la résistance au cisaillement non drainée des sols cohésifs et organiques dans les analyses effectuées.

1 INTRODUCTION

In practical geotechnical engineering, the undrained shear strength τ_{fu} is one of the important parameters governing the behaviour of soil deposits. Considerable efforts have been made to develop the interpretation of in situ tests. Although several theoretical and analytical interpretations of these tests have been proposed (Jamiolkowski et al. 1985, Kulhawy & Mayne 1990, Larsson & Mulabdic 1991, Lunne et al. 1997, Marchetti 1999) the estimation of τ_{fu} relies mainly on empirical and local experience. Therefore, the application of the existing approaches to the evaluation of τ_{fu} from cone penetration and dilatometer readings requires taking into consideration the local conditions and history of soil deposits.

The paper presents the results of field and laboratory tests of heavily overconsolidated Pliocene clay and boulder clay, which prevail in the Warsaw region. The results of field and laboratory tests performed on organic mud at the Nielisz site are also presented.

2 INTERPRETATION OF IN SITU TESTS

2.1 *Cone penetration test*

The cone resistance q_c has mainly been used to estimate the undrained shear strength of clays by using empirical correlation and/or theoretical solutions. Due to large variation of stress and strain induced during the penetration process and the very complex (anisotropic, plastic and rate-dependent) soil behaviour, the interpretation of CPT results is a very difficult geotechnical problem. Therefore, the methods of interpretation available to date refer to the empirical correlation or to approximate theoretical solutions based on highly simplified assumptions.

Due to the difficulties in modelling the penetration process, in practice the most commonly used formula for the evaluation of undrained shear strength τ_{fu} is:

$$\tau_{fu} = \frac{q_c - \sigma_{vo}}{N_K} \tag{1}$$

where: q_c = cone resistance; σ_{vo} = in situ vertical stress; N_K = empirical cone factor.

Since water pressure acts on the area behind the cone, the actual cone resistance, q_T, corresponding to the penetration resistance of the cone is:

$$q_T = q_c + u\,(1 - a) \tag{2}$$

where: u= pore pressure behind cone during penetration; a = area ratio (constant for a specific cone).

The undrained shear strength related to the failure caused by cone penetration is then:

$$\tau_{fu} = \frac{q_T - \sigma_{vo}}{N_{KT}} \tag{3}$$

where: N_{KT} = empirical cone factor related to q_T.

Jamiolkowski et al. (1985) show that for very soft to medium clays the cone factor N_c based on τ_{fu} measured during a field vane test decreases with increasing plasticity index and ranges from 9 to 26.

Campanella and Robertson (1988) showed that the cone factor N_{KT} varies between 4 and 30 depending on some factors, such as sensitivity, stress history, stiffness, macrofabric and definition of τ_{fu}.

2.2 *Dilatometer test*

Many studies have been performed to evaluate and improve some of the original correlations proposed by Marchetti (Marchetti & Crapps 1981) but even new correlations are limited only to mineral soils (Briaud & Mirian 1991). The following correlation between the normalized undrained shear strength and lateral stress index K_D was proposed by Marchetti (1980) for cohesive soils (material index I_D < 1.2):

$$\frac{\tau_{fu}}{\sigma'_{vo}} = 0.22\,\left(0.5 \cdot K_D\right)^{1.25} \tag{4}$$

where: σ'_{vo} = in situ effective vertical stress.

Roque et al. (1988) have shown that the undrained shear strength from the dilatometer test may be obtained from the following formula:

$$\tau_{fu} = \frac{p_1 - \sigma_{ho}}{N_c} \tag{5}$$

where: p_1 = corrected second pressure reading; σ_{ho} = in situ total horizontal stress; N_c = bearing capacity factor for cohesive soils, varies from about 5 to 9.

The analysis carried out by Yu et al. (1992) as well as by Smith & Houlsby (1995) indicates that undrained shear strength can be estimated from the following formula:

$$\tau_{fu} = \frac{p_o - \sigma_{ho}}{N_D} \tag{6}$$

where p_o = corrected first pressure reading; N_D = dilatometer factor for clays, varies from about 4 to 7.

3 DESCRIPTION OF TEST SITES

3.1 *Warsaw region*

The tested sites are located in the central part of the Warsaw Valley, on the post-glacial plateau. Due to this, the subsoil composed of upper Cretaceous deposits filled with Tertiary soils (Oligocene, Miocene, Pliocene) has a configuration generally consistent with the composition of the Warsaw Basin. The top of the Oligocene, Miocene and particularly Pliocene deposits was shaped by processes of erosion and of glacitectonic origin.

In general, the tested subsoil with the exception of surface antropogenic fill consists of the upper moraine deposits, the lower preglacial deposits or Pliocene clays. The highly undulated bed configuration indicates significant soil inhomogeneity.

Nevertheless, the analysis of field and laboratory test results indicated that the tested soils can be classified as stiff sandy clay in the upper Quaternary layers and stiff Pliocene clay in lower Tertiary layer.

Index properties of the tested soils are presented in Table 1. In order to evaluate the undrained shear strength τ_{fu} in cohesive soils and its variability with depth, the comprehensive investigations were undertaken by the Department of Geotechnical Engineering of Warsaw Agricultural University. The in situ testing has been carried out using cone penetration equipment (type HYSON-200 kN) CPT and CPTU. The in situ investigations were supplemented by oedometer and triaxial tests performed on undisturbed samples (Borowczyk & Szymański 1995).

The cone penetration tests showed negative pore water pressure during penetration due to the fact that the tested clays were heavily overconsolidated.

In laboratory, the undrained shear strength was determined in triaxial tests on undisturbed samples. In laboratory tests for samples taken from tested area, a criterion for acceptable volumetric strain for reconsolidation to the in situ effective stress was used to determine the quality of the tested soil specimens.

3.2 *Nielisz site*

Nielisz test site is located in south-eastern Poland in the Wieprz river valley, where in interval 1994 – 2000 an extensive testing programme including laboratory and in situ tests was carried out (Lechowicz & Rabarijoely 1997, Rabarijoely 2000). Due to the appearance of soft soils the main dam embankment of the Nielisz reservoir was constructed in 2-stages with preloading fills (Lechowicz et al. 1998).

At the Nielisz site the soft subsoil consists of mineral and organic sediments. The original thickness of soft soils at the site varied from 1.0 to 5.0 m. The upper 0.5 - 1.0 meter mainly consists of sandy silt or silt. Further down, there is a mud or organic mud layer with 1.0 - 4.0 m thickness divided into two beds by the silt bed. A sand bed occurs below the is found.

The results of the index properties of organic soils are summarised in Table 2. Organic soils classified as organic mud have the organic content between 21% and 35%, with higher values in the upper layer and lower in the lower layer.

Beyond the existing embankment under the downstream berm and the upstream slope the soft soils are overconsolidated with an overconsolidation ratio, *OCR*, decreasing from 3 to 2 with depth. At the end of the first stage, the effective vertical stress was higher than the initial preconsolidation pressure. During the second stage in soft subsoil under the embankment crest, the effective vertical stress exceeded the initial preconsolidation pressure several times. The dilatometer test and field vane test were carried out to obtain profiles of undrained shear strength in organic subsoil. Results of triaxial tests were used to compare with the results of in situ tests. In the evaluation of undrained shear strength from field vane tests the correction factors were used.

Table 1. Index properties of tested soils from Warsaw region.

Properties	Boulder clay	Pliocene clay	Pliocene silty clay
Water content w_n (%)	10-14	18-25	14-20
Unit density ρ (t m^{-3})	2.1-2.2	1.9-2.0	2.0-2.10
Plasticity index I_P (%)	10-18	19-64	23-48
Liquidity index I_L (%)	0.0-0.20	-0.10 - 0.15	-010 - 0.10

Table 2. Index properties of organic soils

Properties	organic mud	mud
Water content w_n (%)	120 - 200	65 – 120
Unit density ρ (t m^{-3})	1.2 - 1.3	1.3 - 1.5
Specific density ρ_s (t m^{-3})	2.1 - 2.3	2.3 - 2.5
Liquid limit w_L (%)	130 - 220	70 – 130
Organic content (%)	21 - 35	8 – 20

4 TEST RESULTS AND DISCUSSION

Based on the results of field and laboratory tests, the accuracy of the evaluation of undrained shear strength from cone penetration tests and dilatometer tests was estimated. In interpretation of cone penetration test and dilatometer test the above mentioned and proposed formulae were used.

In order to evaluate the usefulness of empirical formulae, the following parameters were taking into consideration:

$$MRD = \max_{i=1,2,\dots,m} \left| \frac{y_i - \bar{y}_i}{y_i} \right| \cdot 100\% \tag{7}$$

$$MSRD = \sqrt{\frac{1}{m} \sum_{i=1}^{m} \left(\frac{y_i - \bar{y}_i}{y_i} \right)^2} \cdot 100\% \tag{8}$$

where: MRD = maximum relative deviation; MRSD = mean square relative deviation; y_i = experimental value; $\breve{y}_i$ = value obtained from empirical formula.

Table 3. Results of statistical analysis of the empirical equations.

Soil	Equation 1			Equation 9			
	N_K	MRD %	MSRD %	α	N_K	MRD %	MSRD %
Boulder clay	30	18.1	7.7	0.70	21	13.4	6.8
Pliocene clay	35	19.7	8.0	0.75	25	15.0	7.0
Pliocene silty clay	40	17.7	9.0	0.90	35	15.3	8.0

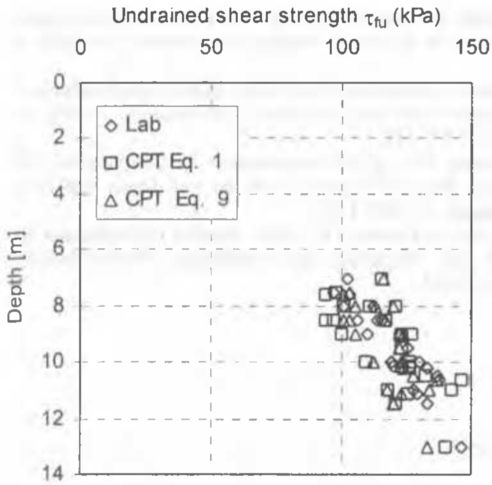

Figure 1. Undrained shear strength obtained from CPT and triaxial tests for boulder clay.

4.1 *Cone penetration test*

The evaluation of undrained shear strength τ_{fu} of cohesive soils based on the measurement of cone resistance q_c using the existing relations presented in the literature can be used for estimating of τ_{fu}. More research works are required for the study of cone factor N_K for heavily overconsolidated soils.

The analysis of results indicates that for evaluation of undrained shear strength from cone penetration test, thc following formula can be proposed (Masoud 2000):

$$\tau_{fu} = \frac{(q_c - \sigma_{vo})^{\alpha}}{N_K} \tag{9}$$

where: α = empirical coefficient.

The results of statistical analysis of the empirical equations used in the interpretation of cone penetration tests are shown in Table 3.

In case of the tested soils the obtained values of N_K are in the range of 30 for boulder clay and 40 for Pliocene silty clay. It is worth mentioning that the empirical cone factor have been calculated basing on laboratory τ_{fu} results obtained from triaxial compression tests. Predicted undrained shear strength values from CPT readings compared with laboratory τ_{fu} values are shown in Figures 1, 2 and 3. Analysis of these results indicate that the cone penetration tests give a good coincidence in these parameters obtained in triaxial tests.

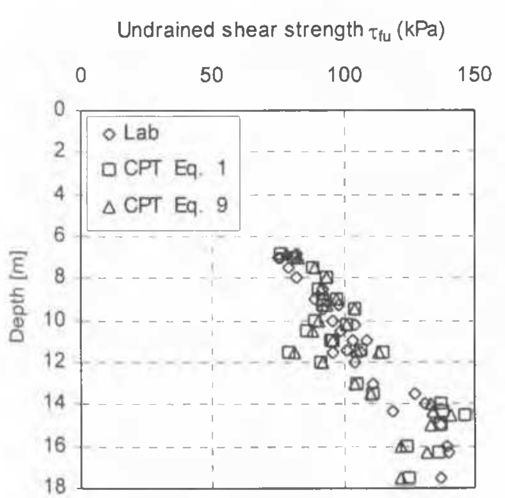

Figure 2. Undrained shear strength obtained from CPT and triaxial tests for Pliocene clay.

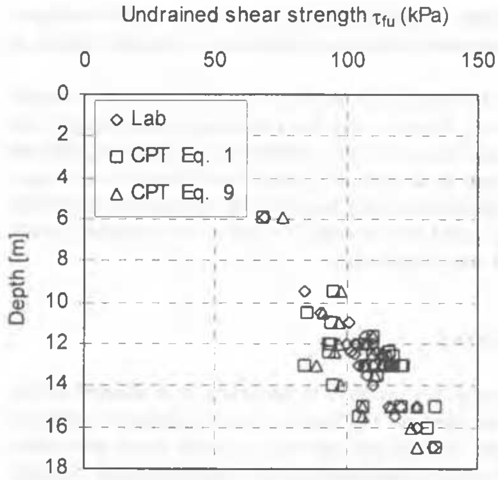

Figure 3. Undrained shear strength obtained from CPT and triaxial tests for Pliocene silty clay.

4.2 *Dilatometer test*

Experience from organic soils indicates that for the evaluation of undrained shear strength τ_{fu} from dilatometer test, the following formula can be proposed (Rabarijoely 2000):

$$\tau_{fu} = \alpha_0 \, \sigma'^{\alpha_1}_v \cdot (p_0 - u_0)^{\alpha_2} \cdot (p_1 - u_0)^{\alpha_3} \tag{10}$$

where: σ'_v = effective vertical stress; u_o = in situ pore water pressure; α_0, α_1, α_2, α_3 = empirical coefficients.

The analysis of test results indicates that the obtained values of empirical coefficients for organic mud from the Nielisz site are $\alpha_0 = 0.70$, $\alpha_1 = 0.08$, $\alpha_2 = 0.11$, $\alpha_3 = 0.58$.

To obtain the undrained shear strength from dilatometer tests the formulae 4, 5 and 10 were used. A comparison between undrained shear strength obtained from triaxial tests and in situ tests for organic soils in cross-section hm 4+50 is shown in Fig-

Table 4. Results of statistical analysis of the empirical equations.

Equation 4		Equation 5		Equation 6		Equation 10	
MRD %	MSRD %	MRD %	MSRD %	MRD %	MSRD %	MRD %	MSRD %
50.7	44.2	19.8	17.6	33.9	19.5	5.0	2.5

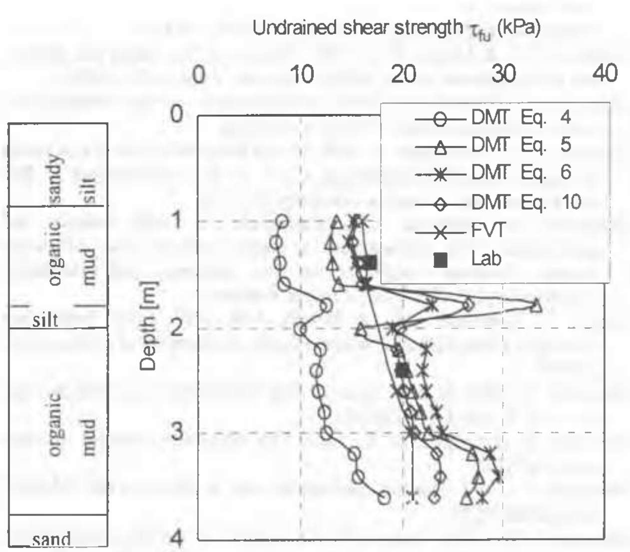

Figure 4. Profiles of undrained shear strength in virgin soft subsoil in cross-section hm 4+50.

ure 4. The results of statistical analysis of the empirical equations used in the interpretation of dilatometer tests are shown in Table 4.

A comparison between the calculated undrained shear strength using formulae 4, 5 and 6 with the corrected shear strength obtained from field vane tests and triaxial tests indicates significant differences. It can be seen from Figure 4 and Table 4 that in general there is a good agreement between the calculated undrained shear strength based on formula 10 and values obtained from field vane tests and triaxial tests.

5 CONCLUSIONS

The paper presents the problem of accuracy in evaluation of the undrained shear strength of heavily overconsolidated cohesive soils from cone penetration test and organic mud from dilatometer test. A comparison between the undrained shear strength from cone penetration test calculated from formulae used in cohesive soils and undrained shear strength evaluated from laboratory tests indicates that the differences are not significant when the empirical cone factor N_c is properly selected. In case of organic mud, the values of undrained shear strength from the dilatometer test calculated from empirical relations elaborated for cohesive soils significantly differ from values evaluated from triaxial and field vane tests. The analysis of test results indicates that for evaluation of undrained shear strength of organic mud from dilatometer test, proposed formula can be used.

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