WARSAW UNIVERSITY OF TECHNOLOGY Index 351733
FACULTY OF CIVIL ENGINEERING
COMMITTEE FOR CIVIL AND WATER ENGINEERING
POLISH ACADEMY OF SCIENCES ISSN 1230-2945

DOI: 10.24425/ace.2025.156183

ARCHIVES OF CIVIL ENGINEERING

Vol. LXXI ISSUE 4 2025

 pp. 483 – 496

Research paper

Assessment of shear strength of soils from in situ research on a regional basis

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Abstract: A correct description of the subsoil is necessary for optimal foundation design and has a practical economic dimension for many geotechnical design problems. Soil strength assessment requires selecting appropriate, standardised tests and validated interpretation methods, based on local research results. However, the correctness of the inference in terms of foundation solutions requires the characterisation of subsoil parameters broken down into units that occur regionally. The article compares the results of shear strength of litho-genetically diverse soils (among others, organic and macroporous, dusty – loess soils) obtained during laboratory and field tests on test plots from the area of Poland. The selected subsoils represent sediments of various lithology, genesis and degree of preconsolidation. The studies included both in situ tests (FVT, CPTU and DMT) and validation laboratory tests. The obtained results made it possible to verify correlation coefficients and formulas, known from the literature, for determining the c_u/s_u parameter obtained by using different methods on a regional basis, as a foundation for determining reliable geotechnical design parameters.

Keywords: geotechnical research, shear strength, methods of interpretation, regionalism, loess soils

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1. Introduction

Proper soil characterisation has a fundamental practical meaning for many geotechnical design problems. The available helpful research methods and techniques make it possible to automate the recording of measurements and to use increasingly sophisticated interpretation tools [1]. However, the correctness of the inference in terms of foundation solutions requires characterising subsoil parameters broken down into units that occur regionally [2]. The possibility of inference on this basis in both field (in-situ) and laboratory studies has already been used and described in numerous publications [3–10]. However, these findings should always be referred to the area of occurrence of a given litho-genetic unit that has been shaped by many processes with limited, local scope, e.g. preconsolidation. The interpretation of geotechnical results requires the use of correlations for particular types of soil; these correlations take into account coefficients given in wide ranges of values, not always corresponding to the litho-genetic units tested [11, 12].

The basic parameter that expresses the bearing capacity of the subsoil is undrained shear strength (c_u)[1]. This parameter can be determined both on the basis of laboratory tests [13, 14] and field (in situ) tests [13, 15]. At the outset, it should be borne in mind that undrained shear strength (c_u) and total stress analysis are accepted in geotechnical engineering as certain simplifications for the purposes of design calculations. In fact, the subsoil failure mechanism – concerning saturated fine-grained soils – is described by the effective strength parameters (φ' and c'), where the excess water pressure in the pore space of the soil (Δu) is taken into account [1]. The udrained shear strength value is defined as the shear stress (τ) at the intersection of the failure envelope (according to Mohr–Coulomb criterion) in the total stress system $(\sigma–\tau)$ [14]. In practice, however, the value (c_u) is affected by many factors, i.e., the determination method, boundary conditions, load path, limited stress level, initial stress state, and other variables.

Undrained shear strength (c_u) can be determined using various methods, on the basis of the adopted patterns of induced loads and failure criteria [15], i.e. plate load test (PLT), back analysis, e.g. in stability analyses (post-factum method), and based on in situ tests: directly measured (FVT) or estimated (e.g. CPTU, DMT). The group of in-situ tests for determining undrained shear strength includes, above all, the FVT (Field Vane Test), which measures directly the resistance value of the vane rotation. The earlier Polish standard [16] provided practically very similar requirements as to the procedure of conducting the field vane test; presently, this method has been standardised at the level of European norms [17]. Modern devices of this type now allow for continuous recording of torque values while maintaining a constant shear speed and take into account the value of force resulting from the friction of the drill column against the ground above the probe tip [18]. The values measured in this way are the most direct and carry little risk of error.

Less frequently used tests are laboratory methods of direct determination of undrained shear strength, i.e. with a laboratory cone probe or a vane probe, where the basic limitation is the inability to run the test in a specific stress state. This is done more often using the direct

[1]It should be observed that according to the current version of Eurocode 7 (PN-EN 1997-2:2009) [17], undrained shear strength is marked with the symbol "c_u", while in the literature related to in situ research, the symbol "s_u" is also used [16]; and in Polish literature to date, this parameter was described with the " τ" symbol

shear apparatus, and triaxial compression apparatus, less often – the Ring Shear apparatus. Among these tests, "triaxial" studies are often considered to be the only reference studies that validate the other methods [14]. This is due to the assumptions made regarding the conduct of the study under full control conditions. It should be borne in mind that the "triaxial" test also has some limitations. The most important one is the small volume of the test material, which may be unrepresentative of the entire zone, and changes in the structure of the sample during its collection, transport and preparation. The scale effect is also important. However, the correctness of the parameters obtained from in situ tests as a result of interpretation requires the appropriate and conscious selection of the "interpretation key", preferably in relation to local conditions [15].

Below, the article will discuss experiences in the determination of shear strength for organic soils, based on the authors' research and published findings regarding local results. The research part will present the process of documenting and interpreting parameter c_u on the basis of available in situ test methods, illustrated with an example of macroporous soils (loess).

2. Collected previous experience

One way of validating the correlation used is to refer the obtained results to the results of determinations from a method where such measurement can be performed directly [18]. In the context of fast and accurate field tests such as CPTU, the value of shear strength is determined by the cone factor (N_k or N_{kt} depending on the type of test, CPT or CPTU) described in [5], according to Eq. (2.1). Direct point measurements of shear strength values performed in the field using an automated FVT probe allow for validation and clarification of the correlation coefficient used for a given soil type and local conditions [18]. For CPTU tests, Eq. (2.1) is used to determine the shear strength of the soil; it requires determining the N_{kt} parameter.

$$s_u = \frac{q_n}{N_{kt}} \tag{2.1}$$

where: q_n – net cone resistance, N_{kt} – empirical cone factor dependent on soil characteristics.

The values of this parameter described in the literature for individual lithological groups vary widely [1,5,16] and in some cases do not correspond to reference studies. In general, it can be said that the increase in the ratio of the shear modulus (G_0) to the undrained shear strength value translates into a significant increase in the value of the cone factor. This relationship has been empirically confirmed by many researchers in local studies [18–21]. Also, the withdrawn standard PN-B-04452:2002 [16] suggested adopting the N_{kt} value depending on the genetic group of the soil and the tip resistance value (i.e. to some extent on the soil consistency). This document refers to the previously existing genetic division of cohesive soils into four groups and, importantly, suggests the use of significantly lower cone factor values in fluvioglacial deposits (between 6 and 15) and Pliocene clays (between 8 and 14). While observations concerning glacial and fluvioglacial soils are confirmed by other researchers, e.g. [22, 23], the authors' own experience in the study of preconsolidated clays suggests that the presented proposal be approached with caution and higher values be used [24]. Table 1 presents the values of N_{kt} parameter proposed so far, depending on the genetic type as a form of data collected from the area of Poland for lithogenic units that occur regionally.

It should be noted that the above list lacks "young" (Holocene) organic sediments and research results for macroporous fine-grained soils of eolic origin (loess and loess-like soils). This is complemented by the authors' research described in detail in [12, 18], presented in Table 2. In normally consolidated high organic soils (peat), the values of this coefficient are between 8 and 12 [25]. On the other hand, in normally consolidated organic soils (mud), it is recommended that significantly lower values be used, between 3 and 9 [18]. At the same time, in practice, it can be observed that overloading of organic soils, especially mineral-organic soils (gyttjas), and inducing in them a preconsolidation effect results in a marked increase of N_{kt}.

Table 1. Proposed values of N_k parameter based on PN-B standard [16]

Genetic type of soil	**Minimum**	**Maximum**
Unconsolidated clays and glacial tills from the catchment area of the Vistula River	$q_c = 0.5$ $N_k = 12$	$q_c = 2.5$ $N_k = 25$
Older consolidated glacial tills	$q_c = 1.5$ $N_k = 12$	$q_c = 7.0$ $N_k = 20$
Fluvioglacial soils, Quaternary silty clays, silty clays	$q_c = 1.2$ $N_k = 6$	$q_c = 3.5$ $N_k = 15$
Pliocene and Miocene clays	$q_c = 1.3$ $N_k = 8$	$q_c = 4.5$ $N_k = 14$
Loess soils	no data	
Gyttjas	$q_c = 0.2$ $N_k = 1$	$q_c = 4.0$ $N_k = 6$

Table 2. Selected values of N_{kt} parameter from local research data [12, 18]

Lithological-genetic type	**Value of N_{kt} parameter**		**Author**
	Authors' research	**Values taken from the literature**	
NC mud and peat	6–9	6–15 (fluvioglacial soils)	EC7 standard [13]
		7.0 (mud) 12.9 (peat)	Młynarek et al. [25]
NC fluvisols (soft plastic silty soils)	~3	7–10 (very soft clay) 7–10 (silty soils) 6–15 (fluvioglacial soils)	Lune et al. [5] Stefaniak [26] EC7 standard [13]
OC Gyttja	~20 (OCR~3–15)	30 (stiff clay) 12–20 (clays) 1–6 7.9 gyttja (OCR ~1–1.6)	Lune et al. [5] EC7 standard [13] Młynarek et al. [25]
Loess soils	~25	30–50	Frankowski et al. [7]

Tests that make it possible to indirectly determine the value of c_u in in situ conditions include DMT tests [15]. In the literature [3–5] related to the estimation of the undrained shear strength value, correlations are given (as a function of c_u/σ'_{vo} and K_D) derived from three types of tests, Eq. (2.2)–(2.4), that determine (validate) the reference value (c_u), in relation to Marchetti's correlation [6]: – based on direct in situ determinations from a vane test (FVT),

$$\frac{c_u}{\sigma_{vo}} = 17 \div 21(0.5K_D)^{1.25} \tag{2.2}$$

– based on laboratory determinations and direct simple shear (DSS) test results (according to ASTM D6528-17),

$$\frac{c_u}{\sigma'_{vo}} = 0.14(0.5K_D)^{1.25} \tag{2.3}$$

– based on laboratory determinations and the course of the triaxial compression test (TRX),

$$\frac{c_u}{\sigma'_{vo}} = 20(0.5K_D)^{1.25} \tag{2.4}$$

Empirical formulas Eq. (2.5) developed for dilatometric studies generally allow for a good estimation of these values, especially for uncemented cohesive soils (material index $I_D \leq 1.2$).

$$c_{u,\mathrm{DMT}} = 22\sigma'_{v0}(0.5K_D)^{1.25} \tag{2.5}$$

where: c_u – undrained shear strength; σ'_{v0} – vertical component of effective stresses in the soil; K_D – horizontal stress index from DMT.

In practice, on the basis of reference studies (other direct methods, e.g. FVT), it is necessary to introduce corrections based on local research results to the original Marchetti formulas. Based on the research described in the literature [15, 24], it was confirmed that the undrained shear strength from DMT for normally consolidated (NC) soils has good consistency with the strength determined during "triaxial" and FVT tests. For overconsolidated (OC), cemented and/or cracked soil, further calibration of the standard correlation with DMT is necessary. Data in this regard for overconsolidated (OC) postglacial clays (tills) and (Neogenic) clays can be found in the paper [22], where a new correlation Eq. (2.6) was established on the basis of empirically determined coefficients:

$$c_u = \alpha_0 \sigma'^{\alpha_1}_{v0}(p_0 - u_0)^{\alpha_2}(p_1 - u_0)^{\alpha_3} \tag{2.6}$$

where: $\alpha_0 = 0.18$, $\alpha_1 = 0.14$, $\alpha_2 = 0.20$, $\alpha_3 = 0.15$.

An example of another modification of Marchetti's formula for organic soil from Poland is a formula for undrained shear strength, based on the results of the authors' research [19], in the following form:

$$c_u = \sigma'_{V0} S(0.5K_D)^{1.25} \tag{2.7}$$

where: coefficient $S = 0.35$ for mud, coefficient $S = 0.40$ for organic mud; and for peats and gyttjas in the form of:

$$c_u = \sigma'_{V0} S (0.45 K_D)^{1.20} \tag{2.8}$$

where: coefficient $S = 0.40$–0.45 for limestone gyttja, coefficient $S = 0.50$ for amorphous peat.

In terms of determination of undrained shear strength of organic soils, the structure of these sediments is an additional element that affects its value. The presence of organics (fibres) may increase the value of c_u parameter. Additional coefficients can be used in this regard [27], or, when large datasets are available, methods that use machine learning tools can be applied [28].

3. Materials and methods of geotechnical investigation

Further on in the article, an example of undrained shear strength tests that were performed at the Elizówka site in Lublin using the available in situ test methods will illustrate the description of a multi-criteria analysis aimed at determining local correlations.

In the Lublin region, there is a loess cover that consists of loess soils from various facies. The main form of the subsoil are typical loess soils of the aeolian facies. As far as their granulometric characteristics are concerned, they are macroscopically homogeneous silts with fine sandy interbeddings. Typically, they are light beige or yellow, with solid or hard-plastic consistency and low moisture content. In some areas in Lublin, in deeper parts of the subsoil, older aeolian loess soils can be identified as sandy silt. The deepest layer, under typical loess soils, consists of deposited aeolian-alluvial loess soils [11]. The facies is much less homogeneous and has a form of silty clay or silt with sandy interbeddings. Inhomogeneity and high variability are evidenced in the parameters measured during in-situ tests.

The highly diversified stiffness of loess subsoil, which is observed, importantly, alongside a relatively homogeneous macroscopic structure, is a vital aspect of the tested soils. In-situ tests are an excellent stiffness identification method [8, 9], including but not limited to cone penetration tests (CPTU), which provide quasi-continuous data, and dilatometric tests (DMT), which enable determining deformability in natural stress conditions. The tests are supplemented with seismic measurements in which the initial modulus of non-dilatational strain is identified based on the wave propagation velocity measurement in the soil. Studies revealed that for typical solid loesses (liquidity index $I_L \leq 0.0$) that are most common in the Lublin region, the q_c cone resistance values range from 4 to 12 MPa [12]. Such a broad range of values testifies to the high diversity of loess subsoil and validity of separating the geotechnical layers not only for I_L, but also for the q_c cone resistance, which reflects the subsoil's stiffness and offers a statistically broad data set.

The tests were conducted at the Elizówka site in Lublin in two research points (Fig. 1). Each point consisted of CPTU, SDMT and PMT tests as well as an automatic (FVT) and a manual (mFVT) vane test. In the engineering practice, hand-held vanes are still utilised, hence the decision to use this variant of the method as well. It should be remembered that due to the manual nature of the measurement, it is difficult to maintain a low and constant shear speed [18]. In this case, the c_u values determined using the automatic set were about 25%

higher than manual measurements. The tests were carried out to a depth of 8 m, in accordance with the applicable standards [13]. Furthermore, boreholes were drilled to collect samples for laboratory tests and basic parameters of the studied soil are presented in Table 3. For the purposes of the analysis in CPTU and SDMT tests, values representative of the depth at which the FVT was performed were derived. For this purpose, the averaged q_c from the measurement zone, i.e. from the segment of 0.5 m (±0.25 m) and parameters from SDMT tests were determined as a mean from the depth of ±0.2 m (i.e. from 2–3 measurements).

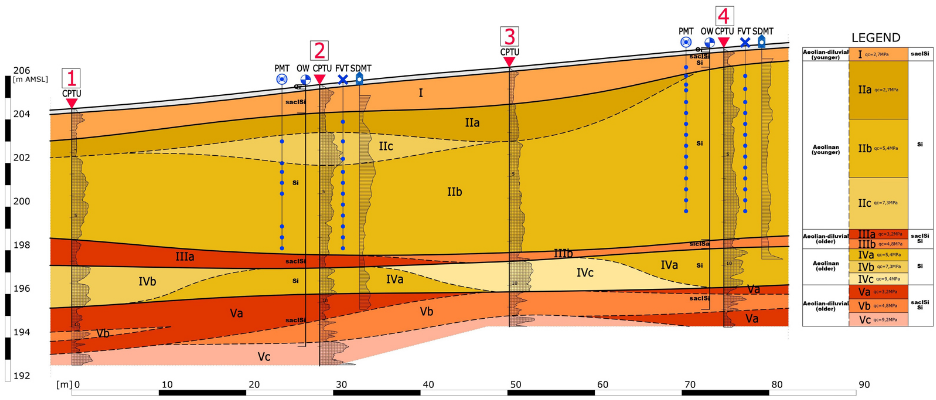

Fig. 1. Geotechnical cross-section from the Elizówka test site, with location of research points

Table 3. Basic parameters of the studied soil [11]

Property	Value/range (average)	Determination method
Natural water content w_n (%)	13.06–17.76	ISO 17892-1
Liquid limit, w_L (%)	(27)	ISO 17892-12
Plastic limit, w_P (%)	(19)	ISO 17892-12
Plasticity index I_P (%)	(8)	ISO 17892-12
Specific gravity, G_s	(2.66)	ISO 17892-3
Clay fraction (%)	(11)	ISO 17892-4
Silt fraction (%)	(88)	ISO 17892-4
Sand fraction (%)	(1)	ISO 17892-4

4. Results

Figure 2 presents the direct results of determinations for the tests used, as a function of depth, while maintaining the measured parameters or resistance values for each method.

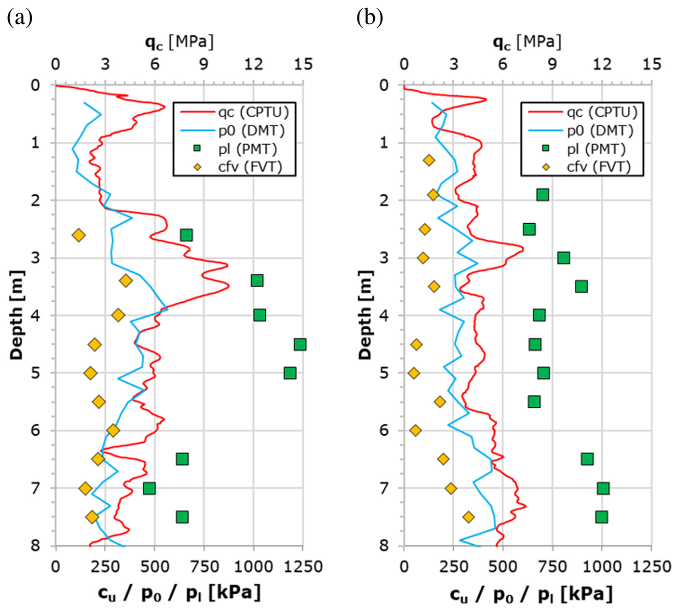

Fig. 2. Summary of test results obtained at the Elizówka site, location: (a) 1, (b) 2

The following charts (Figs. 3 and 4) present the most important data from the measurements, i.e. values of shear strength c_{fv} (FVT) against cone resistance q_c (CPTU) – Fig. 3(a), against limit pressure p_l (PMT) – Fig. 3(b), and against p_0 pressure (DMT) – Fig. 4(a).

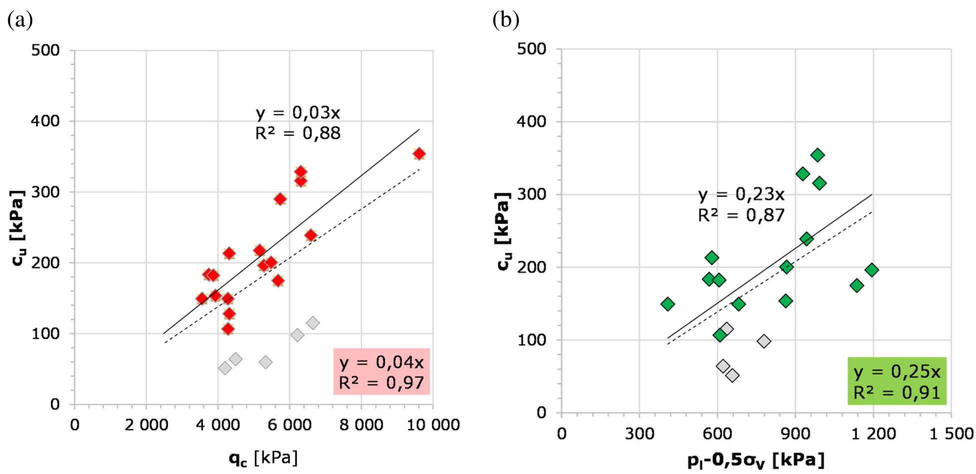

Fig. 3. Summary of test results obtained at the Elizówka site in relation to the results of direct measurements of shear strength s_u: (a) with a static probe (CPTU), (b) PMT test

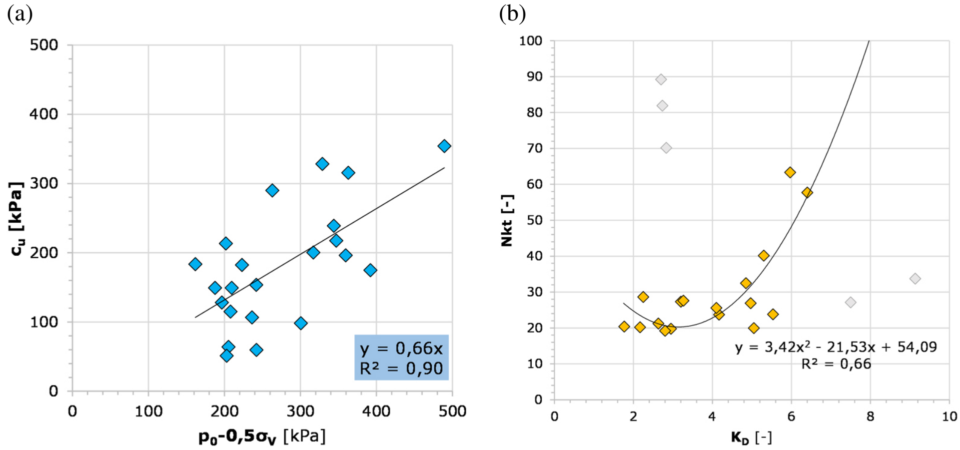

Fig. 4. Summary of test results obtained at the Elizówka site in relation to the results of direct measurements of shear strength s_u: (a) for the DMT test, (b) the auxiliary correlation for determining the N_{kt} parameter from the DMT test (K_D index)

5. Discussion

The primary purpose of the research and analyses was to determine the N_{kt} coefficient of loess subsoil, used to determine shear strength from CPTU tests. Most often, this coefficient takes values in the range between 10 and 20, but for some soils, both lower and higher values are used. From the field tests, N_{kt} coefficient was determined for each FVT measurement; its value varies in a wide range from 19 to 89. After rejecting the extreme results, which deviated significantly from the trend (the likely effect of interbeddings with a higher proportion of sandy fraction content), the average value was 29 and the median was 26. Pairs of parameters q_n–s_u are shown in the chart (Fig. 3a). The trend line was determined (with a forced intercept of 0,0), thanks to which the means $N_{kt} \sim 24$ for filtered results and $N_{kt} \sim 25.5$ for all results were obtained.

The representative value of the coefficient adopted for further analysis was $N_{kt} = 25$. This is a relatively high value compared to standard coefficients adopted for other soils [16]; however, the results of studies on loess conducted in the Lublin Region [7] indicated higher possible values of the coefficient, even in the range of $N_{kt} = 30$–50.

In the case of DMT studies, using Marchetti's original formula [6], the obtained $c_{u.DMT}$ values are much lower than those measured (Fig. 4a). However, it should be borne in mind that this formula is intended for soils with $I_D \leq 1.2$, and for the analysed silts this index is about 2.0. The original formula is based on effective vertical stresses σ'_{v0} and K_D (i.e. p_0, σ'_{v0} and u_0). Due to the fact that the tested loess soils occur in an unsaturated state (well above ground water), for further analyses, simplifications in the form $u_0 = 0$ can be assumed, and undrained shear strength can be conditioned only on the p_0 pressure minus the horizontal pressure expressed in the Marchetti formula Eq. (2.5). In view of the above, the following formula was derived:

(5.1)
$$c_u = 0.66(p_0 - 0,5\sigma_{vo})$$

Taking into account both the derived value $N_{kt} = 25$ and the spread of values in the obtained dataset, additional factors may be sought for more accurate determination of N_{kt}. When analysing N_{kt} against DMT results (Fig. 4b), a correlation with the K_D factor can be noticed. The value of N_{kt} increases with the increased of K_D. Therefore, for soils with $K_D \geq 4$, instead of assuming a constant value, it is better to determine N_{kt} according to the following formula:

(5.2)
$$N_{kt} = 3.4K_D^2 - 21.5K_D + 54, \quad \text{for } K_D \geq 4$$

In pressiometric measurements, there are several propositions for determining shear strength that are generally based on the value of the limit stress p_{LM} minus the lateral pressure, divided by coefficient β. Coefficient β can takes values from 3 to 15 [29]. In the basic Menard formula [30] β is 5.5, and formula assumes the following form:

(5.3)
$$c_u = \frac{p_1 - p_0}{5.5}$$

Another frequently used relationship is the formula proposed by Amar and Jézéquel [31]:

(5.4)
$$c_u = \frac{p_1 - p_0}{10} + 25 \text{ kPa}$$

The value of 5.5 is a coefficient analogous to N_{kt} (it can be called N_{PMT}), which should be determined individually for particular soils. As in the DMT analysis, lateral pressure was assumed as $0.5\sigma_{vo}$ and the following correlation was derived:

(5.5)
$$c_u = 0.23 \div 0.25(p_{\mathrm{LM}} - 0.5\sigma_{vo})$$

Depending on whether individual measurements are taken into consideration (Fig. 3b, where rejected results are marked grey), formulas were obtained that correspond to the average coefficient $N_{\mathrm{PMT}} = 4.35$. Thus, the value is lower than in Menard's method Eq. (5.3).

In the summary of the results, the shear strength values obtained by different methods were compared (Fig. 5). The derived values are in line with the general trend. Three FVT measurements (point 4 – at a depth of 4.5 m, 5.0 m, 6.0 m) deviate from trends (effect of interbeddings with a higher proportion of sandy fraction content). Locally, measurements from a single method deviate from other methods, e.g. point 2 at a depth of 2.7 m (CPTU).

In the chart (Fig. 6), c_u values obtained using different methods are presented. There was a very high consistency of the overall trend; however, one should keep in mind the dispersion of individual results. Dotted lines indicate a range that differs by 30% from the reference value.

With the formulas used, a reliable, averaged value of undrained shear strenght can be obtained. This is simpler and less risky with research methods with large numbers of measurements (e.g. CPTU). However, individual values of c_u may differ in such methods and in methods with a small number of measurements (e.g. PMT), which is mainly related to local variability of the subsoil (e.g. sandy interbeddings, more strongly cemented layers, etc.).

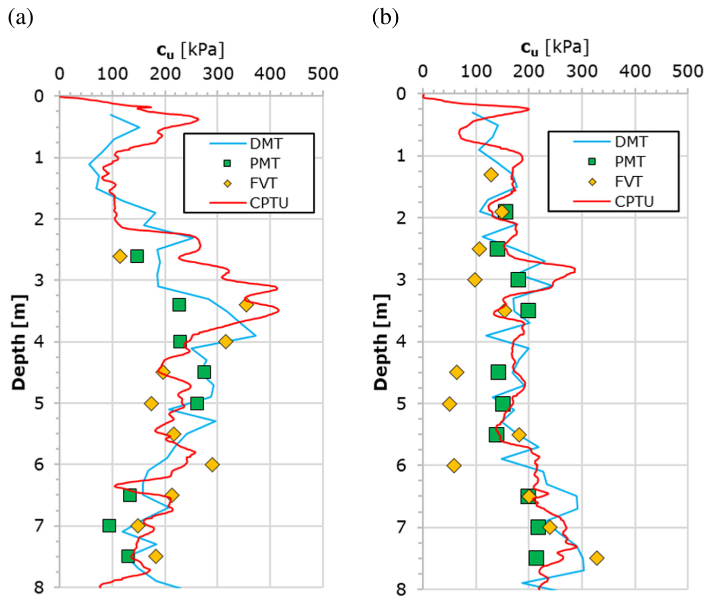

Fig. 5. Summary of test results after corrections of general correlations, location: (a) 1, (b) 2

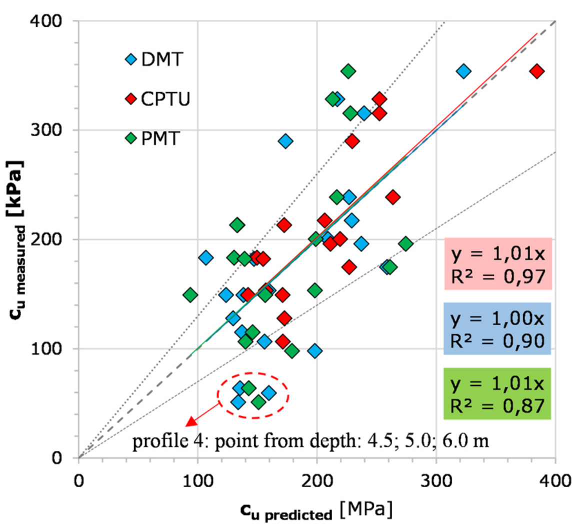

Fig. 6. Measured results versus results interpreted from tests (legend missing)

The authors are aware of the limitations of the final conslusions based on the limited amount of showing measurement data. Nevertheless, the indicated procedure for determining local relationships from in situ tests is correct. The determinated empirical coefficients are very similar with results from other publication (eg. [7]) but it still nesesery to collected more data from further research on the test sites.

6. Conclusions

Analysis of CPTU, DMT, PMT and FVT test results at the same test points showed the existence of significant regional correlations (organic soils from central part of Poland and loess from south-eastern Poland) in the determination of undrained shear strength.

From the measurements taken, an average coefficient $N_{kt} = 25$ was derived for eolic typical loess soils (silty, unsaturated, with a degree of saturation $Sr \leq 0.60$). Furthermore, since 0.22 coefficient from the Marchetti formula is correlated with the cone factor N_{kt}, a new correlation has been proposed for the DMT test results, based on the 0.66 coefficient.

The advantage of field tests is certainly obtaining fast and objective (uniform procedures of execution) information on the subsoil conditions obtained in real conditions (*in situ*). The FVT test is fast and provides a direct undrained shear strength value. Therefore, theoretically, it is ideal for validating other test methods, e.g. N_{kt} values for CPTU tests. However, it should not be forgotten that the value of the "*reliable geotechnical parameter of the soil*" is affected by several factors, among others: the quality of the equipment used for the test, the operator's level of education and diligence in conducting the test, the randomness of the parameters measured during the test, the quality of the samples for validation tests in the laboratory. This cognitive process concludes with the selection of an appropriate method of interpretation, which requires the establishment of a correlation developed for similar conditions, supported by laboratory research and regional research data.

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Ocena wytrzymałości na ścinanie gruntów z badań in situ w ujęciu regionalnym

Słowa kluczowe: badania geotechniczne, wytrzymałość na ścinanie, metody interpretacji, regionalizm, lessy

Streszczenie:

Poprawny opis podłoża jest niezbędny do optymalnego zaprojektowania posadowienia oraz ma praktyczny wymiar ekonomiczny dla wielu problemów projektowania geotechnicznego. Ocena wytrzymałości gruntu wymaga doboru odpowiednich, zestadaryzowanych badań oraz zwalidowanych metod interpretacji, w oparciu o doświadczenia lokalne. Niemniej poprawność wnioskowania w zakresie rozwiązań posadowienia wymaga wypracowania charakterystyki parametrów podłoża budowlanego w podziale na jednostki o zasięgu regionalnym. W artykule porównano wyniki wytrzymałości na ścinanie gruntów zróżnicowanych lito-genetycznie (między innymi organicznych i makroporowych, pylastych – lessy) uzyskane w trakcie badań laboratoryjnych i terenowych na poletkach badawczych z obszaru Polski. Wybrane podłoża reprezentują osady o różnej litologii, genezie i stopniu przekonsolidowania. Badania obejmowały zarówno badania in situ (sondowania FVT, CPTU i DMT) jak i walidujące badania laboratoryjne. Uzyskane wyniki pozwoliły na zweryfikowanie znanych z literatury współczynników korelacyjnych i wzorów dla określenia parametru cu/Su uzyskiwanego różnymi metodami w ujęciu regionalnym, jako podstawa do wyznaczania wiarygodnych do projektowania parametrów geotechnicznych.

Received: 2024-07-16, Revised: 2024-08-20