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DILATOMETER TEST CALIBRATIONS FOR EVALUATING SOIL PARAMETERS

Simon Rabarijoely✉

Institute of Civil Engineering, Warsaw University of Life Sciences – SGGW, Warsaw, Poland

ABSTRACT

Strength and deformation parameters are used on every step of a foundation engineering design. They are necessary for the initial estimation of the subsoil bearing capacity and also for acceptance of the final method of the construction foundation. There are many effective *in situ* methods for the evaluation of these parameters. Based on data from the Marchetti dilatometer (DMT) tests obtained from the experimental Stegny site and the planned WULS-SGGW stadium, analysis and interpretation of results was performed. The uncertainty obtained from DMT tests is shown by mathematical methods. Selection criteria of the testing technique are presented. Limitations concerning the applied methods and the complex interpretation of the results of *in situ* investigations are shown.

Key words: geotechnical parameters, *in situ* investigations, cohesive soil

INTRODUCTION

In the broadly understood engineering activity, the main problem faced by engineers after selecting the location of the facility is the need to determine its foundation. Geotechnnical investigation is a very significant part of the design phase of all engineering structures. Field tests should be used, often simultaneously with laboratory tests (Głuchowski & Sas, 2020; Lech, Skutnik, Bajda & Markowska-Lech, 2020; Wierzbicki et al., 2021). The increase in the popularity of field tests resulted from significant progress in the construction of new test devices, the level of interpretation of the obtained results, the possibility of obtaining a continuous stratigraphic profile of the subsoil, and tests carried out in calibration chambers. However, it is important to understand the quality and meaning of the parameters that are determined by *in situ* testing and to know the limitations of analyzing the factors influencing the measured parameters during the test, as it has significant impact on their correctness (Godlewski & Szczepański, 2012, 2015). Therefore, a precise analysis of the obtained results is recommended each time. The article presents the principles of the selection of sounding and the most frequently used field research methods in Poland. The state of knowledge concerns the methodology and methods of interpretation of the results; furthermore, analysis of research results from the Warsaw University of Life Sciences – SGGW (WULS-SGGW) Campus and the experimental Stegny site are presented. The DMT research was carried out on the WULS-SGGW Campus; it proved that in the subsoil occur thick grey boulder clays of the Odra Glaciation and brown boulder clays of the Warta Glaciation, and the phenomena occurring in the past had significant impact on the complex geological structure of the area. The Marchetti dilatometer (DMT) was also used to examine the area of the experimental Stegny site. The research shows that most of the area below 4 m is covered with interbedding Pliocene clays

Simon Rabarijoely https://orcid.org/0000-0002-4409-223X
✉simon_rabarijoely@sggw.edu.pl

and cohesive loams with a predominance of clays. In a further step, the uncertainty of the results of DMT tests was assessed. The research was carried out on the above-mentioned facilities. The uncertainty of the pressure p_0 and p_1 values from DMT was determined using mathematical methods, and the calculations are included in this paper.

The next chapter contains the methodology of calculations to determine the values of strength and deformation parameters following the approach of Larsson (1989), and examples of calculation results for selected profiles in the experimental Stegny site and in the planned WULS-SGGW stadium site are presented. The aim of this article was, among others, to validate the obtained results of DMT tests, from which, after a thorough analysis, the correctness of the sounding results was assessed.

MATERIAL AND METHODS

The selection and use of field tests should be based on the predicted design and type of structure, e.g. type of foundation, methods of improving or maintaining the structure, location and depth of structure foundation. During the selection of *in situ* methods and the location of research sites, the study results and field control should be taken into account (Table 1). The tests are performed in sites characterized by variable ground conditions with regard to soil, deposits and groundwater.

Soil surveys are usually carried out in stages depending on the problems that arise during planning, design and construction of the actual project. The following phases are distinguished: preliminary research on the location and design of the building;

Table 1. Selection of ground investigation methods in different stages according to ENV 1997-2:2007 (Polski Komitet Normalizacyjny [PKN], 2007)

<table>
<tr><td colspan="6">RECOGNITION</td></tr>
<tr><td colspan="6">Studies based on topographic, geological and hydrogeological maps. Interpretation of aerial photos of a given area. Review of archival materials</td></tr>
<tr><td colspan="6">PRELIMINARY RESEARCH</td></tr>
<tr><td colspan="2">COHESIVE SOILS</td><td colspan="2">NON-COHESIVE SOILS</td><td colspan="2">ROCKS</td></tr>
<tr><td colspan="2">CPT, SS, DP or SPT.
Sampling (PS, TP, CS, OS) PMT, GW</td><td colspan="2">SS, CPT, DP or SPT, SR.
Sampling (AS, OS, SPT, TP) PMT, DMT, GWO</td><td colspan="2">Inspection of the area concerned. Discontinuity map, SE. In weak rocks: DP, CPT, SPT, SR or CS</td></tr>
<tr><td colspan="6">Preliminary selection of the foundation method</td></tr>
<tr><td colspan="6">DESIGN RESEARCH</td></tr>
<tr><td colspan="2">COHESIVE SOILS</td><td colspan="2">NON-COHESIVE SOILS</td><td colspan="2">ROCKS</td></tr>
<tr><td>pile foundations</td><td>spread foundations</td><td>pile foundations</td><td>spread foundations</td><td>pile foundations</td><td>spread foundations</td></tr>
<tr><td>SS, CPT, SPT or SR.
Sampling (PS, OC, CS) FYT, PMT, GWC, PIL</td><td>SS or CPT, DP.
Sampling (PD, OC, CS, TP) FYT, DMT or PMT, GW</td><td>CPT, DP or SPT.
Sampling (PS, OS, AS) PMT, DMT, GW, PIL</td><td>CPT+DP, SPT
Sampling (PS, OS, AS, TP). Possibility PMT or DMT, (PLT) GW</td><td colspan="2">SR
The map of the rifts in TP, CS, RDT (PMT in weathered rocks) GW</td></tr>
<tr><td colspan="6">CONSTRUCTION PROJECT</td></tr>
</table>

In situ tests: SR – soil and rock sounding, SS – static probing, CPT(U) – cone penetration test with pore water pressure measurement, DP – dynamic probing tests, SPT – standard penetration test, PMT – pressuremeter test, DMT – flat dilatometer test, FVT – field vane test, PLT – plate loading test, SE – seismic measurernents, PIL – pile load test, RDT – rock dilatometer test.

Sampling: PS – undisturbed samples, CS – structure – core sample, AS – spiral drill sample, OS – open probe, TP – sample from an open excavation.

Groundwater measurement: GW – free water table, GWO – measurement in an open system, GHE – measurement in a closed system.

design studies; control and monitoring. In the case where all tests are performed at the same time, initial testing and design testing should be considered simultaneously. Three aspects are taken into account in the selection criteria for the ground testing technique: construction safety, performance and economy. Most investors are guided in their work by safety at the lowest cost of geotechnical research. This erroneous approach very often leads to significant reduction of *in situ* and laboratory tests, and then, for example, to overestimating the expected subsidence of the spread foundations or underestimating the load capacity of piles. This phenomenon may also result partly from the poor knowledge of the designers, often also geotechnicians, about the contemporary field research and contemporary differentiation of mechanical parameters describing the subsoil properties (Młynarek & Wierzbicki, 2007; Marchetti, 2015; Lechowicz, Rabarijoely & Kutia, 2017; Młynarek, Wierzbicki & Stefaniak, 2018; Rabarijoely, 2018; Zawrzykraj, 2019; Tarnawski, 2020).

The article presents only a small range of methods, by which soil parameters can be obtained. When choosing the methods to be developed, I was guided by the universality of their use in the world and in Poland, and the correctness of the research results. Based on the analysis of the test results, the strength and deformation parameters can be determined from correlation dependencies.

Taking into account the physical and mechanical properties of the soils, five geotechnical layers were isolated in the grounds of the WULS-SGGW Campus. Layer I consists of fluvioglacial deposits of the Warta Glaciation ($^{fg}Q_pW$) – medium and fine sands with relative density $D_r = 0.35–0.55$, and clay sands, sandy clays and silt with $I_L = 0.15–0.20$. Layer II represents glacial deposits of the Warta Glaciation ($^{b}Q_pW$) – medium and fine sands with $D_r = 0.30–0.50$, and sandy clay with $I_L = 0.00–0.20$ and clay sands with $I_L = 0.25–0.54$, respectively. Layer III is brown glacial clay from the Warta Glaciation ($^{g}Q_pW$) – sandy clays with $I_L = 0.00–0.11$. Layer IV represents grey glacial clay from the Odra Glaciation ($^{g}Q_pO$) – sandy boulder clay with $I_L = 0.00–0.12$. Layers III and IV are similar in terms of plasticity, but clearly differ in the content of the sand fraction. Sandy clays from Layer III contain a few percent more of the sand fraction, which together with the analysis of the DMT results was the basis for distinguishing these layers in the subsoil. Layer V consists of fluvial sediments of the Mazovian Interglacial ($^{f}Q_pM$) – fine and medium sands, in the top represented by very compact layers with a relative density $D_r = 0.80–0.90$ (Table 2). Boulder clays with *OCR* = 3–7 are similar in terms of plasticity, but clearly differ in the content of the sand fraction (Katedra Geoinżynierii SGGW, 2000–2005).

The experimental Stegny site is located in southern Warsaw; here, a few sedimentation cycles, from sands to clays, were observed in vertical profile. The entire complex of Pliocene clays comprises: clays, silty clays (60–70%), silts (10–25%), and sands (10–20%). The $CaCO_3$ and organic matter contents do not exceed 5% and 1%, respectively. The basic properties of the Pliocene clays are presented in Table 2 and Figure 1.

Table 2. Index properties of mineral soils in the WULS-SGGW Campus and Stegny sites (Katedra Geoinżynierii SGGW, 2000–2005)

Sites	Soil type	Organic content (I_{om}) [%]	$CaCO_3$ content [%]	Water content (w_n) [%]	Liquid limit (w_L) [%]	Density unit weight of soil (ρ) [t·m^{-3}]	Density specific weight of soil (ρ_s) [t·m^{-3}]
Stegny	Pliocene clays	–	–	19.20–28.50	67.6–88.0	2.1–2.2	2.68–2.73
WULS-SGGW Campus	boulder clay Layers III, IV	–	–	5.20–20.10	21.9–26.6	2.0–2.2	2.68–2.73

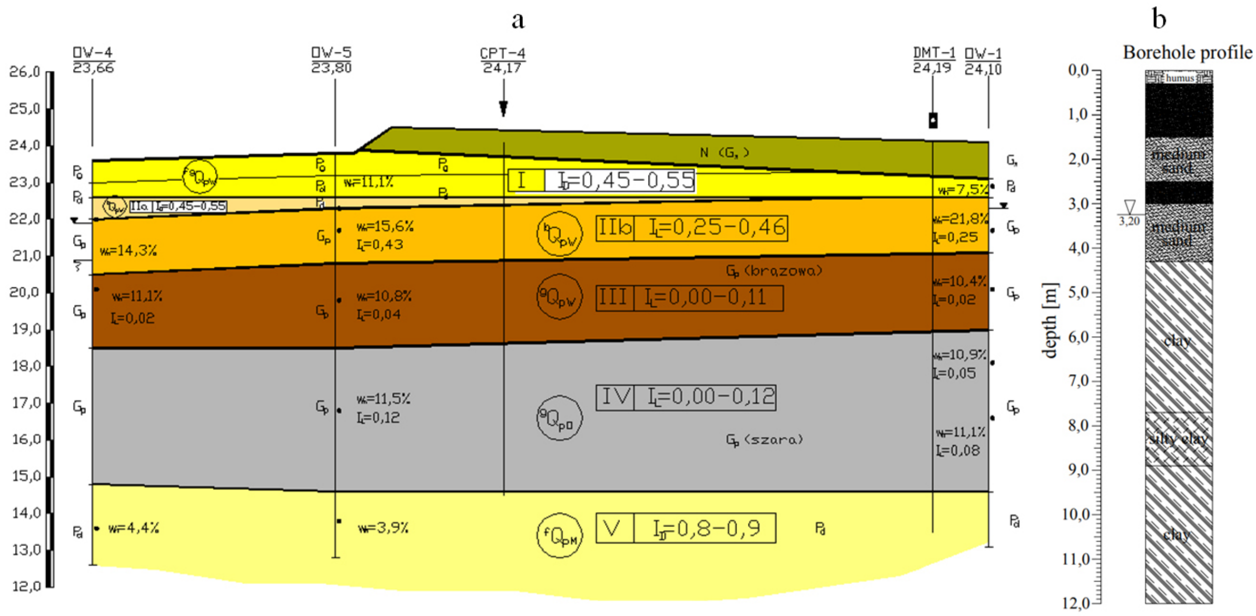

Fig. 1. Typical geological conditions of the foundation at the WULS-SGGW Campus (a) and at the Stegny test site – borehole profile (b)

EVALUATION OF THE UNCERTAINTY OF DMT TEST RESULTS

Introduction

The uncertainty of measurements was assessed for DMT tests on the following sites: the experimental Stegny site and the planned WULS-SGGW football stadium. Both tests were carried out in two research profiles (for Stegny site: Profiles 1, 2 and 3 (DMT8, DMT9 and DMT10), for WULS-SGGW stadium: profiles DMT2 and DMT6). The uncertainty of pressures p_0 and p_1 was assessed. Pressure range (i.e. the difference between the two extreme results obtained at the same level), variance, value of standard deviation, repeatability of the results and uncertainty value (Joint Committee for Guides in Metrology [JCGM], 1993) were calculated.

Determining the uncertainty

Based on field tests, p_0 and p_1 were calculated for the successive insertion depths of the dilatometer blade (Table 3). The $p_{01}, p_{02}, ..., p_{0m}$ are the ordinal results of the value measurements, and $p_{11}, p_{12}, ..., p_{1m}$ are analogous results for p_1. The boundaries of the uncertainty ranges for p_0, p_1 measurements can be described by the following formulas:

$$p_o \pm e_p, \; p_1 \pm e_p \qquad (1)$$

where e_p is expanded uncertainty, provided that the error measurement distribution is normal (with the expected error value equal to zero) (JCGM, 1993).

Estimating the results

The method of conducting the test ruled out the possibility of ensuring the criteria of repeatability and reproducibility, as the measurements were taken once in any given place. In order to analyze the variability, pseudosamples, i.e. pairs of results obtained from adjacent sites, were used. It was assumed that the distribution of the trait is relatively similar in places closed to each other (JCGM, 1993). The tests of these three DMTs were carried out there at a distance of 0.5 m around the boreholetest, and at maximum radius up to 10 m there are a total of 10 DMT profiles (from DMT1 to DMT10). I would like to emphasize that on this site an almost homogeneous layer was observed. A total of 40 samples were collected at the WULS-

-SGGW stadium (OW1–OW14). At the research point analyzed in the article, where two DMTs were carried out (DMT2 and DMT6), five samples were taken (from a depth of 0.5, 1.5, 4.0, 8.0 and 14 m). Due to the failure to meet reproducibility criteria, the uncertainty assessment requires the use of an interval estimation: $e'_p \le e_p \le e''_p$, based on the extreme estimates resulting from the analysis of their variability. The decisive factor in the variability of measurement results is the depth at which the indications were taken. In order to calculate the standard deviation and on this basis estimate the expanded uncertainty e'''_p, a linear regression equation for the dependence of pressure on depth was determined, and then the appropriate pressure value resulting from this equation was subtracted from each measurement result (JCGM, 1993). For pressure p_0 and p_1, the following results were obtained:

– for pressure values p_0 (Stegny site) $\begin{cases} e'_p = 0.0481 \\ e''_p = 0.2720 \\ e'''_p = 0.0471 \end{cases}$

– for pressure values p_1 (Stegny site) $\begin{cases} e'_p = 0.2884 \\ e''_p = 0.8000 \\ e'''_p = 0.2826 \end{cases}$

– for pressure values p_0 (WULS-SGGW stadium: 0.6–2.4 m) $\begin{cases} e'_p = 0.1060 \\ e''_p = 0.1460 \\ e'''_p = 0.1039 \end{cases}$

– for pressure values p_1 (WULS-SGGW stadium: 0.6–2.4 m) $\begin{cases} e'_p = 0.1433 \\ e''_p = 0.1700 \\ e'''_p = 0.1404 \end{cases}$

– for pressure values p_0 (WULS-SGGW stadium: 2.4–8.0 m) $\begin{cases} e'_p = 0.7679 \\ e''_p = 0.0580 \\ e'''_p = 0.7526 \end{cases}$

– for pressure values p_1 (WULS-SGGW stadium: 2.4–8.0 m) $\begin{cases} e'_p = 1.0967 \\ e''_p = 2.2700 \\ e'''_p = 1.0748 \end{cases}$

The above results confirm that, in addition to the aforementioned depth, the variability of the measurements is primarily influenced by the features of the existing layer. In the case of studies on the WULS--SGGW Campus, the deviations are much greater than in the Stegny site. This is due to the highly consolidated clays deposited during the Odra and Warta glaciations, which influenced the variability of p_0 and p_1 measurements. Smaller deviations in the experimental Stegny site mean that the terrain is more even there, and the layers of the same soil occur at similar depths, in contrast to the WULS-SGGW site, which is located on the Ursynów slope.

VALIDATION OF STRENGTH AND DEFORMATION PARAMETERS OF COHESIVE SOIL

The aim of this paper is to validate *in situ* tests and shorten the time needed for obtaining on-site results and their analysis. The technical tasks included: validation of soil parameters using several experimental comparisons; reduction of the time needed to interpret the results of *in situ* tests; and shortening the time needed to obtain the parameter estimation algorithm. The expected results were: increasing the accuracy of *in situ* test results, resulting in a wider use of the procedures applied; reducing the duration of field studies if the tests are carried out faster, they will be more widely used; reduction of the time devoted for the analysis of the results; and increasing confidence in design methods (Spitler, Yavuzturk & Jain, 1999).

Comparison of the I_D according to Marchetti correlations with $I_{D(corr)}$ according to Larsson approach

The material index is one of the parameters calculated based on the results obtained from *in situ* tests. It is defined as follows (Marchetti, 1980):

$$I_D = f\left(A, B, u_0\right) = \frac{p_1 - p_0}{p_0 - u_0} \tag{2}$$

$$E_D = f\left(A, B\right) = 34.7\left(p_1 - p_0\right) \tag{3}$$

The above formula was presented after observing that the p_0 and p_1 values are relatively similar for clays and completely different for sands. According to Marchetti (1980), the soils can be classified as follows: $I_D < 0.6$ clay; $0.6 < I_D < 1.8$ silt; $1.8 < I_D$ sand. Generally, I_D provides information about the soil type and can describe it well in natural subsoil. It should be mentioned, however, that the material index sometimes wrongly describes clay as loam and vice versa, and a mixture of loam and sand thus describes it as clay. When using the material index to describe the soil type, it should be remembered that it is not determined based on sieve analysis, but is a parameter describing the soil mechanical behavior (being a kind of "stiffness index"). For example, if a clay sample for some reason is "stiffer" than other clays, the sample is likely to be interpreted by the I_D index as clay (Marchetti, 1980).

In order to determine the soil type, based on DMT test results, the Marchetti and Crapps chart (Marchetti & Crapps, 1981) is used (Fig. 2). Based on the relationship between the material index (I_D) and the DMT modulus (E_D) on a logarithmic scale, it is also possible to determine their condition for mineral soils. Additionally, the values of soil bulk density divided by water density are assigned to appropriate intervals.

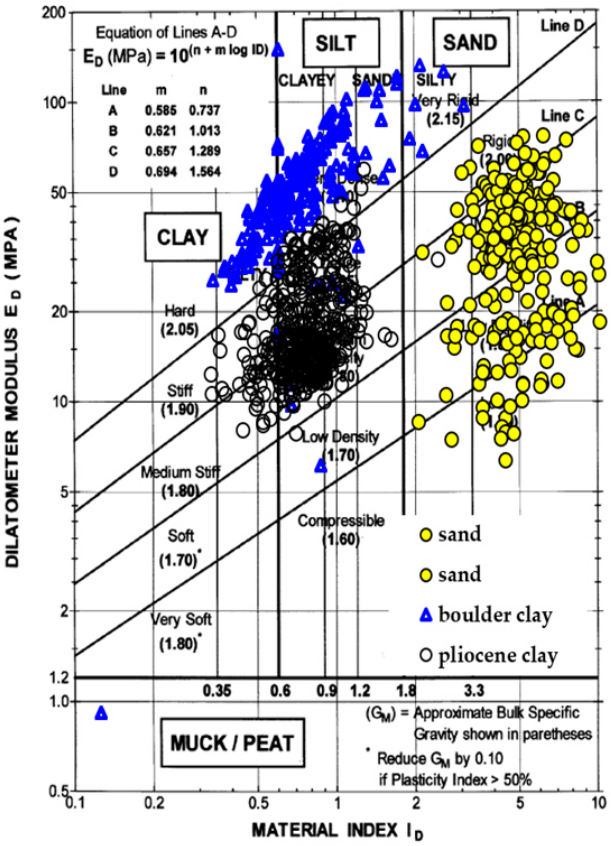

Fig. 2. Chart for estimating soil type and unit weight γ (normalized to $\gamma_w = \gamma$ water) (1 bar = 100 kPa) (Marchetti & Crapps, 1981)

Larsson (1989), on the other hand, derived the material index (I_D) as the corrected material index ($I_{D(corr)}$) after analyzing DMT tests performed for preconsolidated cohesive and organic soils. The influence of preconsolidation on the change of its value is taken into account here. Larsson also took into account the presence of anthropogenic soil in the shallowest layers. According to Larsson (1989), the corrected material index ($I_{D(corr)}$) can be determined from the following relationships:

$$I_{D(corr)} = \begin{cases} I_D \le 0.1, I_D \ge 10, K_D \le 2.5 \rightarrow I_D \\ z \le 2 \text{ and } I_D - 0.075 \cdot (K_D - 2.5) \le 0.1 \rightarrow 0.11 \\ z \le 2 \rightarrow I_D - 0.075 \cdot (K_D - 2.5) \\ I_D - 0.035 \cdot (K_D - 2.5) \le 0.1 \rightarrow 0.11 \\ \text{in otherwise case } I_D - 0.35 \cdot (K_D - 2.5) \end{cases} \tag{4}$$

where z is the depth [m].

Based on DMT test results carried out in the experimental Stegny site and in the area of the planned WULS-SGGW stadium, the I_D for the local soil was compared with the corrected index. First, index parameters were calculated for each of the holes, including I_D; later, following the method developed by Larsson (1989), the corrected material index ($I_{D(corr)}$) was calculated. According to I_D values, in which the indicator parameters clearly change, two layers were distinguished in the experimental Stegny site and three layers in the WULS-SGGW stadium area. The results are presented in Table 3 and Figure 3.

Table 3. Average value of the material index (I_D) and the corrected material index ($I_{D(corr)}$) depending on the depth in: the experimental Stegny site and the planned WULS-SGGW stadium

Site	Depth [m]	I_D (Marchetti, 1980)	$I_{D(corr)}$ (Larsson, 1989)
Stegny	0.0–4.4	4.2	4.43
	4.4–10.4	0.9	0.79
WULS-SGGW stadium (Layers I, II, III, IV)	0.0–2.2	1.1	1.0
	2.2–4.0	1.6	1.52
	4.0–8.6	0.6	0.56

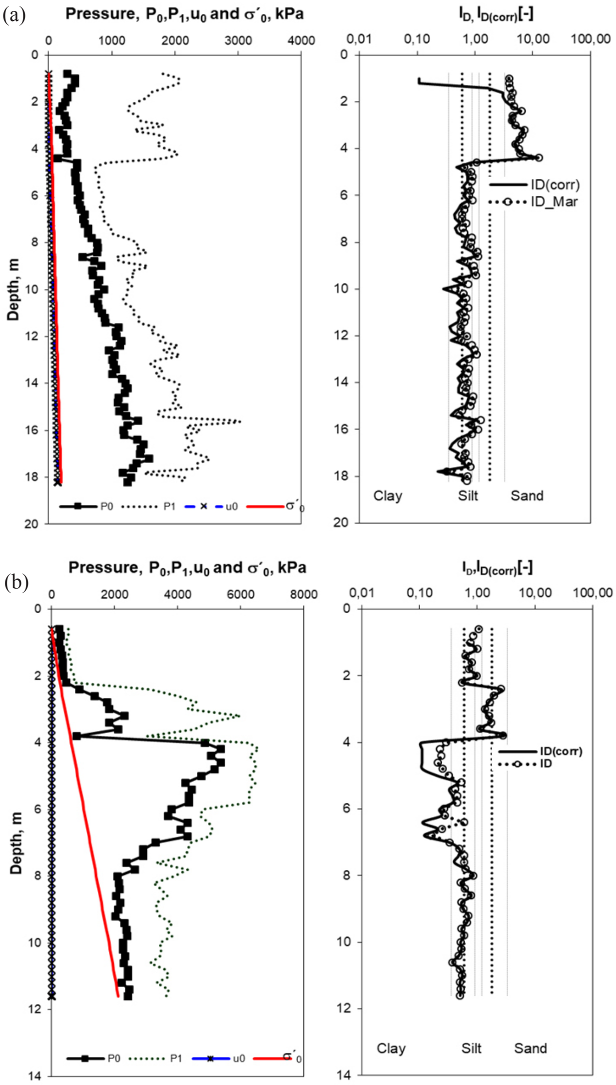

Fig. 3. Profiles of corrected readings p_0 and p_1 and index parameters: the material index (I_D) and the corrected material index ($I_{D(corr)}$) from DMT tests obtained for Pliocene clay subsoil in the Stegny (a) and for boulder clays in the WULS-SGGW stadium (b) sites

From the obtained data and based on the interval values determined by Larsson (1989), it can be concluded that there are two layers of clay in the area of the WULS-SGGW stadium and clay is the deepest deposit. As for the experimental Stegny site, after comparing the results, residual sand and deeper Pliocene clay were determined. The results are not correct in full, which is caused by averaging the indicator values of the strata and the previously mentioned misinterpreted soils of similar "stiffness". The I_D and $I_{D(corr)}$ values are similar, although the $I_{D(corr)}$ values are closer to correct in terms of borehole results.

Determination of strength and deformation parameters

A spreadsheet created by Larsson (1989) was used to determine the strength and deformation parameters of soils based on DMT tests. The program is intended for the presentation and evaluation of DMT test results. The program was constructed by Rolf Larsson at Swedish Geotechnical Institute (SGI) to perform calculations according to the guidelines from the SGI Information 10 (Larsson, 1989) and to check the evaluation of the overconsolidation ratio (*OCR*) and undrained shear strength (c_u and τ_{fu}) presented by Larsson and Åhnberg (2003) in the SGI Report 61 (Larsson & Åhnberg, 2003. The results obtained from the computational approach according to Larsson (1989), PN-B-03020 (PKN, 1981) and from laboratory tests are presented in Table 3.

It should be noted that the program calculates the undrained shear strength in two ways: (1) parameter τ_{fu} is calculated according to the Swedish experience presented in SGI Information 10. This parameter is often recommended here for normally consolidated soils or heavily preconsolidated clays; (2) parameter c_u is calculated according to generally accepted empirical formulas, i.e.

$$c_u = 0.22 \cdot \sigma_0' \cdot OCR^{0.8} \tag{5}$$

The formula uses effective vertical stress and overconsolidation ratio, determined on the basis of the DMT test. This parameter is recommended for preconsolidated clays, but generally understates the shear strength of organic soils. The program also determines two values of the constrained modulus: the *M* parameter is calculated for sands, clays and preconsolidated clays and can be used in normal settlement calculations; and the M_C parameter is calculated for normally consolidated clays and only moderately preconsolidated clays. This parameter can only be used for settlement calculations for a strength below the preconsolidation stress. The calculation is performed automatically after all the data have been entered. The p_0 and p_1 parameters, i.e. corrected A and B readings due to the membrane inertia resistance, are obtained from the following formulas:

$$p_0 = f(A,B) = 1.05(A - z_m - \Delta A) - \\ - 0.05(B - z_m - \Delta B) \tag{6}$$

$$p_1 = f(B) = B - z_m - \Delta B \tag{7}$$

where z_m is zero pressure gauge [bar].

Then the water pore pressure (u_{hydr}) at depth (z), i.e. every 20 cm, is calculated from the formula

$$u_{hydr} = z \cdot \gamma_w \left[\mathrm{kN \cdot m^{-2}}\right] \tag{8}$$

According to Eqs. (2) and (3) given above, the following parameters are calculated: I_D and E_D, i.e. material index and DMT modulus.

The first calculation of the adjusted material index consists of calculating a new I_D taking into account the following ranges:

$I_D < 0.25$; $0.25 < I_D < 0.6$; $0.6 < I_D < 1.8$; $I_D > 1.8$ and adjusting the value depending on E_D,

- determination of vertical total stresses

$$\sigma_{tot} = h \cdot \gamma \left[\mathrm{kN \cdot m^{-2}}\right] \tag{9}$$

- determination of vertical effective stresses

$$\sigma'_{tot} = \sigma_{tot} - u_{hydr} \left[\mathrm{kN \cdot m^{-2}}\right] \tag{10}$$

- calculation of the lateral stress index according to the Eq. (2),
- calculation of the corrected material index according to the Eq. (4).

Larsson (1989) then proposed to perform three iteration versions of the $I_{D(corr)}$ calculation in the same way as above, however each subsequent iteration uses the newly computed material ratio.

Next, more complicated formulas are used to calculate the strength and deformation soil parameters, i.e.: M, OCR, c_u and τ_{fu} and φ.

- τ_{fu} [kPa] (undrained shear strength)

$$\tau_{fu} = \begin{cases} I_{D(corr)} > 1.2 \rightarrow \tau_{fu} = 0 \\ I_{D(corr)} > 0.6 \rightarrow \tau_{fu} = \dfrac{F^*}{10.3} \\ E_D > 10^{\left(0.3944 \ln I_{D(corr)}\right) + 0.5438} \rightarrow \\ \quad \rightarrow \tau_{fu} = \dfrac{F^*}{10.3} \\ I_{D(corr)} < 0.25 \text{ and} \\ \quad E_D > 10^{\left(0.4642 \ln\left(I_D(corr) + 5.438\right)\right)} \rightarrow \\ \quad \rightarrow \tau_{fu} = \dfrac{F^*}{10.3} \\ \quad \text{in otherwise} \rightarrow \tau_{fu} = \dfrac{F^*}{9} \end{cases} \tag{11}$$

where:

$$F^* = p_1 \cdot 98.1 - \sigma_{eff} \cdot K_0 - u_0 \tag{12}$$

- c_u [kPa] (undrained shear strength before dehydration of anisotropic clay)

$$c_u = \begin{cases} I_{D(corr)} \leq 1.2 \rightarrow c_u = 0.22 \cdot \sigma_{eff} \cdot \left(OCR^{0.8}\right) \\ \quad \text{in otherwise} \rightarrow c_u = 0 \end{cases} \tag{13}$$

- φ [°] (internal friction angle soil)

where:

$$\varphi = \begin{cases} I_{D(corr)} \leq 1.2 \rightarrow \varphi = 0 \\ I_{D(corr)} \cdot A^{**} < 100 \rightarrow \varphi = 0 \\ \text{in otherwise} \rightarrow \varphi = I_{D(corr)} \cdot \\ \quad \cdot \left(A^{**} - 100\right)^{0.5} \cdot 0.19 + 25 \end{cases} \tag{14}$$

$$A^{**} = \begin{cases} \sigma_{eff} \left\langle 50 \text{ and } A^* \right\rangle 500 \rightarrow A^{**} = \\ \quad = \dfrac{A^* - 500}{\left(\dfrac{A^* - 500}{1{,}500}\right) + 1} + 500 \\ \text{in otherwise} \rightarrow A^{**} = A^* \end{cases} \tag{15}$$

$$A^* = \frac{1{,}000 \cdot E_D}{\sigma_{eff}} \tag{16}$$

– *OCR* [-] (overconsolidation ratio)

$$OCR = \begin{cases} B \neq 0 \,\text{and}\, B > 1 \rightarrow OCR = B \\ C \neq 0 \,\text{and}\, C > 1 \rightarrow OCR = C \\ \text{otherwise} \rightarrow OCR = 1 \end{cases} \tag{17}$$

where:

$$B = \begin{cases} I_{D(corr)} > 0.9 \rightarrow B = 0 \\ K_D \leq 5 \rightarrow B = 10^{(0.16(K_D - 2.5))} \\ K_D \geq 7.5 \rightarrow B = 0.24 \cdot K_D^{1.32} \\ \text{in otherwise} \rightarrow \\ \rightarrow B = 2.51 + 0.368 \cdot (K_D - 5) \end{cases} \tag{18}$$

$$C = \begin{cases} I_{D(corr)} \leq 0.9 \rightarrow C = 0 \\ I_{D(corr)} \geq 2 \rightarrow C = (0.67 \cdot K_D)^{1.91} \\ I_{D(corr)} \geq 1.2 \rightarrow C = \\ = \left(K_D \cdot 0.5 + 0.17 \cdot \dfrac{I_{D(corr)} - 1.2}{0.8}\right)^{1.56 + 0.35 \cdot \frac{I_{D(corr)} - 1.2}{0.8}} \\ \text{in otherwise} \rightarrow C = (0.5 \cdot K_D)^{1.56} \end{cases} \tag{19}$$

– *M* [MPa] (constrained modulus)

$$M = \begin{cases} Q = 0 \rightarrow M = 0 \\ Q < 0.85 \cdot E_D \rightarrow M = 0.85 \cdot E_D \\ \text{in otherwise} \rightarrow M = Q \end{cases} \tag{20}$$

where:

$$Q = \begin{cases} X \neq 0 \rightarrow Q = X \\ Y \neq 0 \rightarrow Q = Y \\ Z \neq 0 \rightarrow Q = Z \end{cases} \tag{21}$$

$$X = \begin{cases} I_{D(corr)} \leq 0.6 \text{ and } K_D \geq 4.38 \rightarrow \\ \rightarrow E_D \cdot (0.14 + 2.36 \log K_D) \\ \text{in otherwise} \rightarrow X = 0 \end{cases} \tag{22}$$

$$Y = \begin{cases} I_{D(corr)} > 0.6 \text{ and } I_{D(corr)} \leq 3 \rightarrow \\ E_D (0.14 + 0.15 (I_{D(corr)} - 0.6) + \\ + \left\{2.5 - \left(0.14 + 0.15 (I_{D(corr)} - 0.6)\right)\right\} \cdot \log K_D) \\ \text{in otherwise} \rightarrow Y = 0 \end{cases} \tag{23}$$

$$Z = \begin{cases} I_{D(corr)} > 3 \rightarrow E_D \cdot (0.5 + 2 \log K_D) \\ \text{in otherwise} \rightarrow Z = 0 \end{cases} \tag{24}$$

ANALYSIS OF THE OBTAINED RESULTS

By analyzing the distribution of parameters according to the Marchetti algorithm, the following values were obtained: τ_{fu} = 56, 385, 360 kPa; c_u = no data; σ'_p = 0.41, 3.7, 2.5 MPa; *OCR* = 7, 6, 5; *M* = 55, 114, 77 MPa for boulder clay, while for Pliocene clay at a given depth, it was found that the Larsson algorithm gave values of τ_{fu} = 71, 92, 117, 211 kPa; c_u = 38, 56, 82, 89 kPa; σ'_p = 0.44, 0.58, 0.74, 1.48 MPa; *OCR* = 4.9, 5.1, 5.4, 6.0; *M* = 43, 35, 30, 37 MPa, while on the basis of laboratory tests the following results are obtained for boulder clay τ_{fu} = no data; 273, 240 kPa; c_u = no data; σ'_p = no data; *OCR* = 2; *M* = 48, 80, 80 MPa; and for Pliocene clay are: τ_{fu} = 79, 54, 144, 84 kPa; c_u = no data; σ'_p = 0.14, 0.30; *OCR* = 2; *M* = 22.5, 30 MPa.

In the computational approach developed by Larsson (1989), several zones corresponding to different types of soil were distinguished in the part concerning cohesive soils (Table 3). The analysis of the test results carried out for selected types of cohesive soils shows that for engineering purposes it is advisable to limit the number of areas separated according to the soil type, while focusing on defining zones characterized by different conditions. The proposed modification of the Larsson computational modification, including the separation of two areas: 1 – clay / silt, 2 – peat / gyttja, and zones of different state, determined based on the undrained shear strength (τ_{fu}) and the constrained modulus, are presented in Eqs. (6)–(24). When analyzing the distribution of the arithmetic mean values of the τ_{fu} for boulder clays at the foundation depth, it was found that the value of τ_{fu} is as-

Table 3. Value of parameters obtained from DMT tests taking into account $e'_p \le e_p \le e''_p$ according to Larsson, 1989) (A); laboratory tests (B), and according to Marchetti, 1980 (C) for the WULS-SGGW and Stegny sites

Site	z [m]	w_n [%]	c [kPa]		τ_{fu} [kPa]			σ'_p [MPa]			c_u [kPa]	φ [°]			OCR [-]			K_0 [-]			M [MPa]		
		(B)	(A)	(B)	(A)	(B)	(C)	(A)	(B)	(C)	(A)	(A)	(B)	(C)	(A)	(B)	(C)	(A)	(B)	(C)	(A)	(B)	(C)
WULS-SGGW Stadium Layers I, II, III, IV	0.0–2.2	13.7	n.d.	n.d.	31	n.d.	56	0.3	n.d.	0.41	46	10.71	n.d.	n.d.	1.6	n.d.	7	0.6	n.d.	1.8	11	48	55
	2.2–4.0	9.9	n.d.	n.d.	150	273	385	2	n.d.	3.73	309	18.5	n.d.	n.d.	3.5	n.d.	6	0.7	n.d.	1.5	180	80	114
	4.0–8.6	9.6	n.d.	n.d.	461	240	360	1.5	n.d.	2.53	291	n.d.	n.d.	n.d.	1.5	n.d.	5	0.5	n.d.	2.4	122	80	77
Stegny Pliocene clay	6.0–6.4	26.03	n.d.	30	71	79	71	0.22	0.14	0.44	38	n.d.	21	n.d.	3	2	4.9	0.9	n.d.	1.2	46	22.5	43
	9.0–9.4	28.53	n.d.	20	120	54	92	0.32	n.d.	0.58	56	n.d.	14	n.d.	3	n.d.	5.1	0.4	n.d.	1.3	46	n.d.	35
	12.0–12.4	19.84	n.d.	19	156	144	117	0.50	n.d.	0.78	82	n.d.	27	n.d.	4	n.d.	5.4	0.4	n.d.	1.3	46	n.d.	30
	15.0–15.4	21.37	n.d.	27	152	84	211	0.50	0.30	1.48	89	n.d.	21	n.d.	3	2	6.0	0.4	n.d.	1.6	46	30	37

sumed to be 305 and 300 kPa for c_u, while for Pliocene clays the value of τ_{fu} is assumed to be 154 and 86 kPa for c_u. Taking into account the specific M values of the constrained modulus of the boulder clays and Pliocene clays, they reach 151 and 46 MPa, respectively. The Larsson approach can be used to determine the undrained shear strength distribution and the constrained modulus in the subsoil of the designed building from the calculations of arithmetic averages.

CONCLUSIONS

The results of DMT tests according to Marchetti and Larsson after comparing them to laboratory values (laboratory tests in the experimental Stegny site in 2012, research of the Department of Geotechnics on the WULS-SGGW Campus in 2004) allowed for formulating the following conclusions:

– The validation of the deformation and strength parameters of cohesive soils proves that the most reliable parameter, which complies with the laboratory values, is undrained shear strength. From the values calculated according to Larsson (1989), the least similar to the laboratory values are the values of the constrained modulus. The results indicate that the correction of the material index (I_D) to corrected material index ($I_{D(corr)}$) due to preconsolidation proposed by Larsson (1989) gave more similar results to the correct results in the case of the WULS-SGGW stadium site, while the Stegny site gave more differring values (The difference can be noticed in the values of strength and deformation parameters: ($\tau_{fu\_\mathrm{Larsson}} > \tau_{fu\_\mathrm{Marchetti}} > \tau_{fu\_\mathrm{laboratory}}$; $M_{\_\mathrm{Larsson}} > M_{\_\mathrm{laboratory}} > M_{\_\mathrm{Marchetti}}$; $OCR_{\_\mathrm{Larsson}} \approx$ $\approx OCR_{\_\mathrm{Laboratory}} < OCR_{\_\mathrm{Marchetti}}$; $\sigma'_{p\_\mathrm{laboratory}} <$ $< \sigma'_{p\_\mathrm{Marchetti}}$).
– Using the dependencies developed by Larsson (1989), it is possible to significantly shorten the time of interpreting the results of *in situ* tests, as parameter estimation is performed by using algorithms, although it should be emphasized that this does not increase the accuracy of the design methods if the correctness of the results is not considered carefully enough. Considering the influence of the strength of c_u before dehydration of isotropic clay, corresponding in the first approximation to the anisotropic (natural) clay strength obtained from direct simple shear tests tests without drainage, is important in geotechnical design. The basis for estimating the value of τ_{fu}, as a reference for the correlation according to Larsson, is usually

an *in situ* shear test (field vane test – FVT) or a laboratory triaxial test of undisturbed soil sampling (USS) samples, and anisotropycally consolidated to *in situ* stress, tested in undrained (CAUC).

In the future, it is recommended to use the Larsson spreadsheet to analyze the deformation and strength parameters of organic subsoils.

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KALIBRACJA BADAŃ DYLATOMETRU MARCHETTIEGO DO OCENY PARAMETRÓW GRUNTOWYCH

STRESZCZENIE

Parametry wytrzymałościowe i odkształceniowe wykorzystywane są na każdym etapie posadowienia obiektów inżynierskich. Są niezbędne we wstępnej ocenie nośności podłoża, jak również w przyjęciu ostatecznego sposobu posadowienia konstrukcji. Interpretacja danych z badań geotechnicznych wymaga ujednolicenie podejścia wyników raportu badań *in situ*, aby parametry gruntowe były oceniane w sposób spójny i komplementarny z wynikami laboratoryjnymi. Istnieje wiele skutecznych metod otrzymywania parametrów geotechnicznych. W artykule przedstawiony jest badanie dylatometryczne Marchettiego (DMT). Na podstawie wyniku badań *in situ* z poletka doświadczalnego na Stegnach oraz na terenie projektowanego stadionu piłkarskiego SGGW przeprowadzono analizę i interpretację uzyskanych wyników. Ukazano również metodami matematycznymi niepewność wyników otrzymanych z badań dylatometrem Marchettiego. Zaprezentowano w tej pracy kryteria doboru techniki badania, ograniczenia dotyczące stosowanych metod oraz kompleksową interpretację wyników badań *in situ*.

Słowa kluczowe: parametry geotechniczne, badania *in situ*, grunty spoiste