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The Use of CPTU and DMT Methods to Determine Soil Deformation Moduli—Perspectives and Limitations

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Abstract: The article presents the concept of determining constrained modulus—M_0, initial shear modulus—G_0, Young modulus—E, and rigidity index—I_R on the basis of parameters from static penetration tests CPTU (Piezocone Penetration Testing), SCPTU (Seismic Piezocone Penetration Testing) and dilatometer tests DMT (Flat Dilatometer Test), SDMT (Seismic Flat Dilatometer Test). The basis for constructing the empirical relationships between the mentioned modules and parameters from the CPTU and DMT studies was to determine the factors that affect these relationships. The article discusses the impact of the following factors; geological and geotechnical conditions, conditions of recording measurements in CPTU and DMT tests, factors relating to the CPTU and DMT testing methods, factors affecting reference parameters from laboratory tests, factors related to subsoil properties. The basis for obtaining the empirical relationships for determining the analyzed modules and rigidity index were extensive research of the soils of various origins, in Poland. Measurement uncertainties and factors influencing the recorded parameters in the CPTU study were documented by the studies of the Norwegian Geotechnical Institute and the former Department of Geotechnics of the Agricultural University in Poznań. In these studies, penetrometers from several reputable manufacturers were used. The article summarizes the established empirical relationships for individual modules, taking into account the effect of overconsolidation. It also comments on the interrelationship between constrained modulus M_0 from CPTU and DMT test for soils in Poland.

Keywords: CPTU; DMT; deformation moduli.

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1 Introduction

Soil deformation moduli play a vital role in the preparation of a geotechnical project for investments, such as road facilities, high-capacity buildings, and wind farms. A very valuable and, at the same time, expected element of the geotechnical design is a complete profile of changes of constrained modulus M_0, Young's modulus E, and small strain shear modulus G_0 in the subsoil. In situ tests, such as cone penetration test CPTU, seismic cone SCPTU, and flat dilatometer tests DMT, SDMT are highly applicable for obtaining the profile of changes of the abovementioned moduli in the subsoil. The fact that these studies are already commonly used in Poland works in their favor (Młynarek, 2010). The interpretation of CPTU and DMT tests has a good theoretical basis (Lunne et al., 1997, Marchetti, 1980) and numerous empirical relationships have been developed to determine the deformation moduli based on the parameters from these tests (e.g., Mayne, 2006, Młynarek et al., 2013, 2015, Robertson & Cabal, 2012). The measurements of building settlements and the extent to which they comply with the settlements predicted based on the deformation moduli determined from CPTU and DMT are also known. Some of these studies showed high compliance of settlements measured with those calculated on the basis of the moduli determined from the CPTU (Młynarek et al., 2013, Rześniczak et al. 2019), as well as from the DMT tests (Monaco et al., 2007). There are also studies that document a significant discrepancy between the predicted and measured settlements and in the case of the moduli from the DMT, predicted settlements are significantly lower than the measured ones. One of the goals of this article is to clarify this interesting issue.

2 Factors influencing the relationship between CPTU and DMT parameters and deformation moduli

2.1 Geological and geotechnical conditions

Parameters from CPTU and DMT tests, which are the foundation for determining Young's modulus and small strain shear modulus G_0, are recorded in the subsoil under strictly defined geological and geotechnical conditions. The randomness of these parameters is related to the variability of soil properties in the tested subsoil and the factors that impact these properties. The second group of factors are measurement uncertainties related to the testing technique. It is important to state Lacasse & Nadim (1994) that these factors cannot be separated in the analysis of the randomness of the determined parameter in in-situ tests. This fact is essential in assessing the quality of the parameter, which is used in defining the relationship between the CPTU test parameter, e.g. cone resistance q_c, q_t and the abovementioned moduli.

The factors that form the geotechnical properties of soils in the subsoil were defined by Powell (2005) as follows:

- geological regime
- hydrological regime
- engineering regime.

The geological regime is associated with the variability of soil grain size of soil, its macrostructure, and its origin. A change in hydrogeological conditions is, among others, caused by changes in the groundwater level, which generates the effect of seepage pressure and a change in the stress state in the subsoil. Engineering regime includes such processes as changes in the stress state in the subsoil as a result of excavation, soil drainage, and the impact of the load on neighboring objects. Each of those factors may generate preconsolidation effects and the abovementioned changes in the stress state in the subsoil, Marchetti (2012) quotes the following formulation by Jamiolkowski "Without stress history impossible to select reliable E, or M_0 from q_c" (cone resistance in the CPTU method). The factors commented above should be taken into account in order to forge the relationship between the parameters from CPTU, DMT, and deformation moduli for soils from Poland. A particular emphasis should be put on identifying the effect of preconsolidation of the subsoil in the studied area for the planned investment.

2.2 Conditions of recording measurements in CPTU and DMT

Introducing a static penetrometer tip or a dilatometer blade into the subsoil causes a change in the stress state in the subsoil. Disturbed areas for both tools were documented by Baligh & Scott (1975) (Fig. 1). The cone resistance in the CPTU is recorded in the limit state (Młynarek & Sanglerat, 1981, Durgunoglu & Mitchell, 1973). Differences in the conditions of recording measurements in CPTU and DMT tests and the reduction of shear modulus with the level of strain were well illustrated by Mayne (2001) (Fig. 2). The preconsolidation effect is closely related to the impact of horizontal stress σ_h on the recorded measurements in both studies (Fig. 3). A detailed explanation of this problem can be found in the publication of Marchetti (1998). The use of such evidence is important, as mentioned in Section 2.1, in constructing accurate relationships between the determined parameters in the CPTU, DMT tests and soil deformation moduli in the subsoil. Figs. 1, 2, and 3 show that the CPTU and DMT methods can identify different values of deformation moduli in preconsolidated soils if standard empirical relationships are used to assess them. This effect also applies to soils with exposed macrostructure, e.g. varved clays (Młynarek et al., 1982).

2.3 Factors related to the CPTU and DMT testing technique

In order to form the relationship between the parameters from the CPTU and DMT tests and the deformation moduli and the small strain shear modulus G_0, one needs to identify the factors that affect the parameters recorded in these tests. These factors will have a significant impact on the quality of the determined relationship between the CPTU and DMT parameters used and the abovementioned moduli. Identification of these factors—random variables can be obtained from the record of functions that describe CPTU and DMT tests (Młynarek, 1978 & 2007).

The physical process in CPTU is defined by the law describing the displacement of the cone in the soil medium and is equivalent to the law describing the process of the displacement of a material point in a medium exhibiting friction (Banach, 1950).

The function describing this process takes the form of:

$$F\,(P, v_p, \theta_1, \theta_2) = 0 \qquad (1)$$

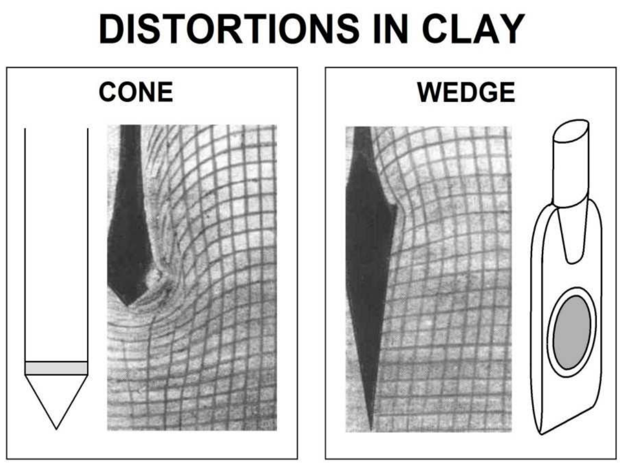

Figure 1: Photographs of deformation grids caused by the penetration in the soil of cone-shaped and wedge-shaped penetrometers (after Baligh and Scott, 1975).

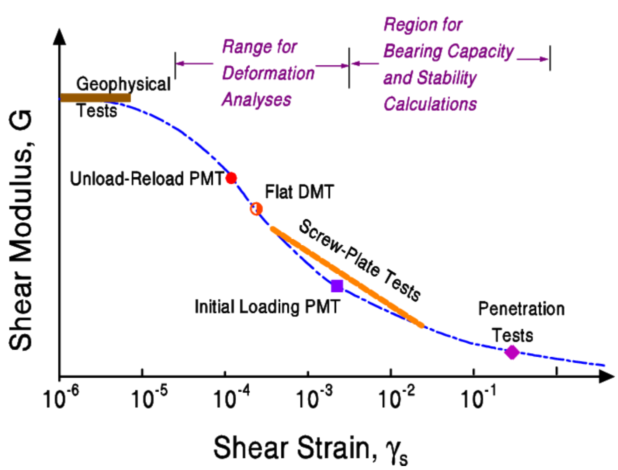

Figure 2: Reduction of shear modulus with level of strain (after Mayne, 2001).

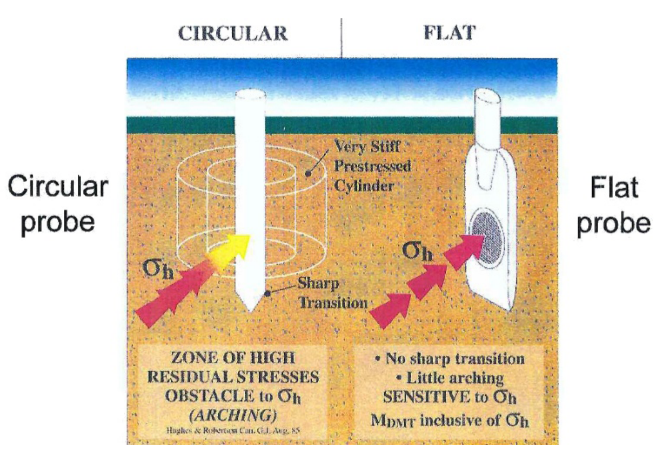

Figure 3: Sensitivity to σ_h of measured parameters in CPTU and DMT tests (after Marchetti, 1998).

where: P—measured parameter of the process, e.g.—q_c,u_2, v_p—rate of penetration, θ_1—characteristics of the soil medium, θ_2—cone characteristics.

Parameter θ_1 is a function of many independent variables, describing the soil medium

$$\theta_1 = f(x_1 .. x_{10}) \tag{2}$$

where: x_1—content of clay fraction in soil, x_2—content of silt fraction in soil, x_3—content of sand fraction in soil, x_4—density, x_5—coefficient of viscosity, x_6—angle of internal friction, x_7—cohesion or an equivalent parameter, according to the adopted form of the description of shear strength, x_8—structure, x_9—constrained modulus, x_8—OCR (overconsolidation ratio), σ_{v0}, σ_{h0} (vertical, horizontal stresses in the subsoil).

Parameter θ_2 is described by a function of the following form (Młynarek, 1978, Lunne et al., 1997)

$$\theta_2 = f(x_1^c .. x_3^c) \tag{3}$$

where x_1—variable describing cone geometry (e.g. h—height, d—diameter), x_2^c—coarseness of cone material, x_2^c—deformation modulus of cone material.

Several important observations may be derived from equation (1). The general solution of equation (1) has not been detected. The solution given using bearing capacity theory, expansion theories is only a specific case of its solution. Variables required for the construction of equations (2) and (3) are latent and discrete, which complicates even a partial solution of equation (1). A change to each of the independent variables affects the solution of the equation. In the engineering approach, it is important whether the effect is significant or nonsignificant. Multivariate analysis of variance may prove helpful when making a decision on the matter (Młynarek et al., 1982).

If the subsoil contains organic soils, the variables in equation (2) must be supplemented with the content of organic matter and the degree of decomposition (Młynarek et al., 2008 & 2015). Equations: (1), (2), (3), lead to several very important conclusions for forming the relationship between e.g. cone resistance q_c, q_t from the CPTU and the constrained M_0, or shear G_0 moduli, namely:

– It is necessary to search for the so-called local correlation for the studied area, where the range of variability of individual variables is strictly defined, e.g. the measurement of q_c in fine-grained soils, coarse-grained soils, and intermediate soils (Lunne et al., 1997).

– Values of the parameter, e.g. cone resistance, must be referred to the most important variable that affects the cone resistance, which is geotechnical stress σ_{vo} , σ_{ho} (Młynarek & Sanglerat, 1982, Lunne et al., 1997).
– Owing to the fact that the penetrometers of various companies are used in Poland (Młynarek 2010) and the research is conducted by several operators even in one company, it is necessary to specify the factors related to the testing technique that affect the recorded parameters in tests, such as CPTU.

The physical process in the case of DMT is defined by the following function (Młynarek et al., 2015)

$$F_2 (P_d, V_d, Q_1^d, Q_2^d) = 0 \quad (4)$$

where: P_d—measured process parameters, e.g. pressure p_0, p_1, V_d—membrane-bearing velocity of the dilatometer Q_1^d—membrane properties.

The parameter Q_2^d is a function of many variables, namely:

$$Q_2 = f (X_1 ... X_{10}) \quad (5)$$

These are identical variables that occur in equation (2) for the CPTU method.

The general form of equation (4), as in the CPTU, is unknown. This problem also generates the need to construct the so-called local relationship for the relationship between moduli M_0, E, and G_0 with the parameters from this study.

There are two factors that affect the level of precision and accuracy of the CPTU or DMT parameters recorded in the study in the group of factors relating to the testing method. The first one is the quality of the measuring system of the penetrometer (CPTU) or dilatometer, the second one is the level of education of the device operator. These elements are particularly important for the assessment of the relationship between, for example, the cone resistance and the deformation moduli of soils from Poland since penetrometers of various manufacturers are, as previously mentioned, in use. The original Marchetti dilatometers are used in Poland in DMT tests.

In order to evaluate precision and accuracy it is necessary to perform a replicate test. For the i-th replication the test value x_i can be obtained (Lee & Lumb, 1974):

$$X_i = \alpha z + \beta + \delta_i \quad (6)$$

where δ_i is a random variable of zero mean and variant $V(\delta_i)$. Expectation value for a large number of replication is

$$E(x) = \alpha z + \beta \quad (7)$$

where α and β express the bias or lack of accuracy, while $V(\delta_i)$ represents the lack of precision. The larger the variance, the lower the precision. The value is most often determined by a calibration or model test, in which some response z can be predicted by a theoretical function

$$z = f (z) \quad (8)$$

The problem with evaluating the quality for a test by determining precision needs to be considered separately for a laboratory analysis, in which an experiment is performed on soil samples, and for in situ testing. In the case of a laboratory analysis, the quality of a sample has a highly significant effect on precision and as a consequence—on an increase or decrease of uncertainty of the reference test. The laboratory test is a necessary reference test for the evaluation of the deformation modulus based, for example, on the relationship between cone resistance and the deformation moduli.

In the case of in situ testing of the factors, which affect the evaluation of quality of the test, includes the performance of testing using nonstandard equipment, performance of testing by several operators differing in their educational background, and their ability to predict. A relationship of the two latter factors and their effect on the parameter may be presented after Lumb (1974) with the use of dependence, which determines the replication of a test on the same sample or in the field on the same soil layer—n, performed by p-operators on q—different apparatus.

The k-th repeat test by the j-th operator on the i-th apparatus can be presented as:

$$X_{ijk} = \zeta + \alpha_1 + \beta_1 + \gamma_1 + \delta_{ijk} \quad (9)$$

i, 1 to q; j, 1 to p; k, 1 to n

where: X – value of investigated parameter, α_1 - represents the Machine Effect $E(\alpha_1) = 0$, β_1 - represents the Effect operator $E(\beta_1) = 0$, γ_1 - represents the interaction between machine and operator, δ_{ijk} - is a random variable of zero mean and variance.

If α_i; β_j; γ_{ij} are considered as random variables, then they will have their own variances $V(\alpha_i)$, $V(\beta_j)$, $V(\gamma_{ij})$. These variants will result in a significant effect on the impression.

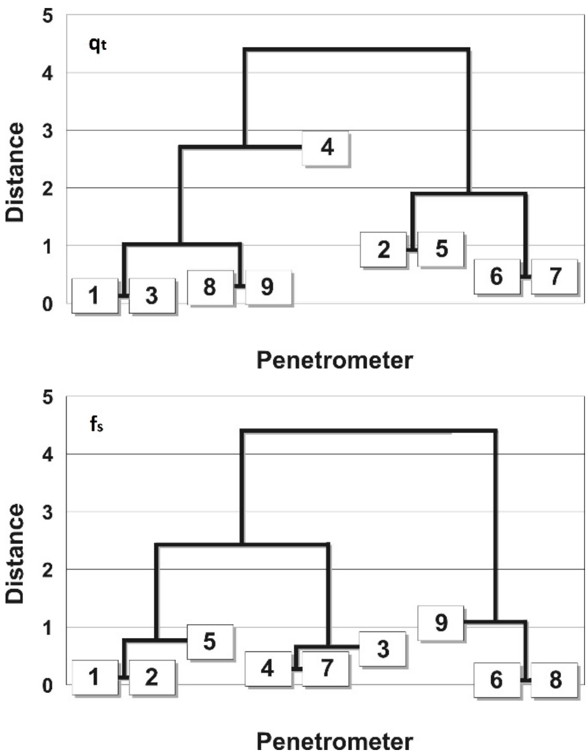

Figure 4: Grouping of the penetrometers after Ward's method (dendrograms for q_t and f_s) (after Gauer et al. 2002).

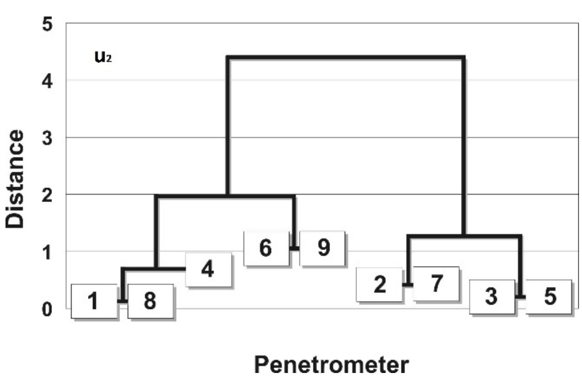

Figure 5: Grouping of the penetrometers after Ward's method (dendrograms for u_2) (after Gauer et al. 2002).

Research on the level of accuracy and precision of 9 penetrometers of various manufacturers was carried out by the Norwegian Geotechnical Institute and the Department of Geotechnics of the former Agricultural Academy in Poznań (Quality of CPTU – report, Norwegian Geotechnical Institute, Gauer et al., 2002). A homogenous group of penetrometers is created using the cluster method based on the Ward criterion (Box et al., 1978, Winter et al., 1991). Figs. 4 and 5 show the grouping of penetrometers recording the least different friction value on the friction sleeve, cone resistance q_c, and pore pressure – u_2.

In the last fifteen years, the manufacturers of penetrometers have significantly modified the measurement systems, hence the obtained values of cone resistance and friction on the friction sleeve may show a different assessment of compliance than the one presented in the first stage of the Norwegian Geotechnical Institute research (Gauer et al., 2002). The Norwegian Geotechnical Institute continued detailed research on this subject (Panigua et al., 2021, Lunne et al., 2018, Lindgard et al., 2018) under various geotechnical conditions with penetrometers from various manufacturers. An example of research carried out at a silt test site in Norway, in which Pagani, Geomil, and Geotech penetrometers were used, is shown in Fig. 9. These studies made it possible to reach several very important conclusions, namely:

– Procedures and operator skills can have a significant effect on test results, in addition to the equipment.
– For all the investigated cones, penetration pore pressure u_2 gave the most repeatable results.
– Corrected cone resistance q_t, generally varies somewhat more than u_2, regarding tests with the same cone, and more than comparing one cone type with another.
– Some of the cone types show good repeatability for sleeve friction f_s readings, while some show a relatively large variation. Owing to significant uncertainties with the f_s readings, one should be careful with using this parameter and the frictions ratio when interpreting soil parameters for design.
– Since the measured u_2 values appear to frequently be the most reliable parameter, it should be used in addition to q_t for deriving soil parameters.

The abovementioned results of the Norwegian Geotechnical Institute research come to another very important conclusion for those interested in buying a penetrometer. This conclusion relates to the use of, e.g. undrained shear strength or constrained modulus, to determine empirical relationships found in literature, which, as previously mentioned, should be calibrated with the results of laboratory tests on samples of high quality. This type of calibration may demonstrate the low usefulness of the adopted empirical dependence, which was designated for soil outside Poland. The ISO standard 22476-1:2022 should be the starting point for the penetrometer quality assessment.

In the case of the DMT, the issue of measurement uncertainties is significantly limited. It is determined by two factors: standard dilatometers are available on the market by mainly one manufacturer and the measurement technique of the dilatometric test is not complicated.

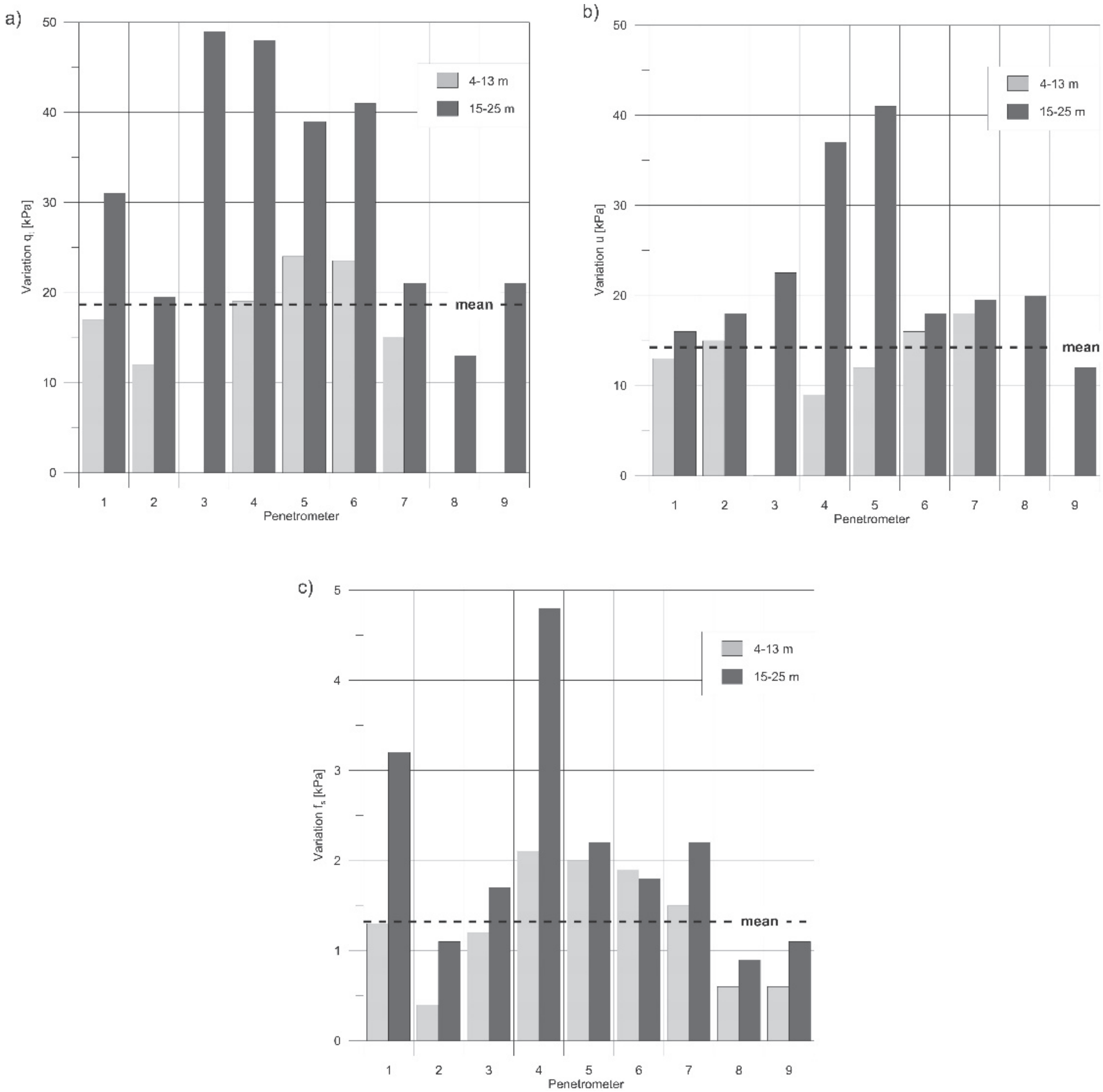

Figure 6: Total variation sum of precision and mean noise level for q_t, u and f_s for different penetrometers (after Gauer et al. 2002).

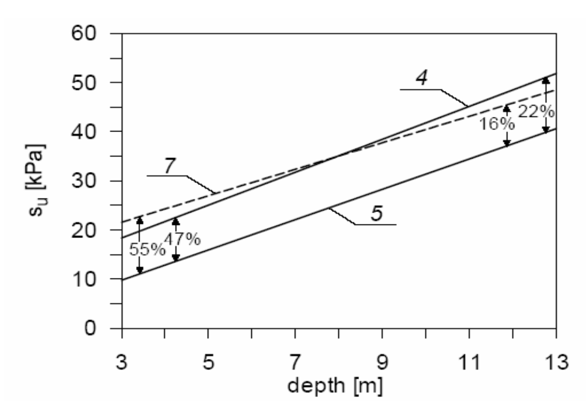

Figure 7. Prediction of undrained shear strength of Onsoy clay by different penetrometers (after Młynarek et al. 2007).

Marchetti (2012), the author of the concept of dilatometer examination, formulated the results of the replication test as follows: "Any operator gets the same results, no need for highly skilled workers."

2.4 Factors affecting reference parameters from laboratory tests

The laboratory reference test for determining the correlation between CPTU and constrained modulus *M* should be performed on high-quality samples (Lunne et al., 2006). Owing to the demonstrated differential registration of the friction coefficient f_s by individual penetrometers, it is also necessary to perform reference tests of grain size distribution of individual soils found in the subsoil. The reference graining test is particularly important if CPTU classification systems are used to identify soils found in the subsoil (Lunne et al., 1997, Robertson, 2012). The influence of the quality of samples obtained by various samplers on the course of deformation characteristics in the sample loading process was documented by Tanaka (2007) and Long (2002). The results of these tests are shown in Figs. 10 and 11.

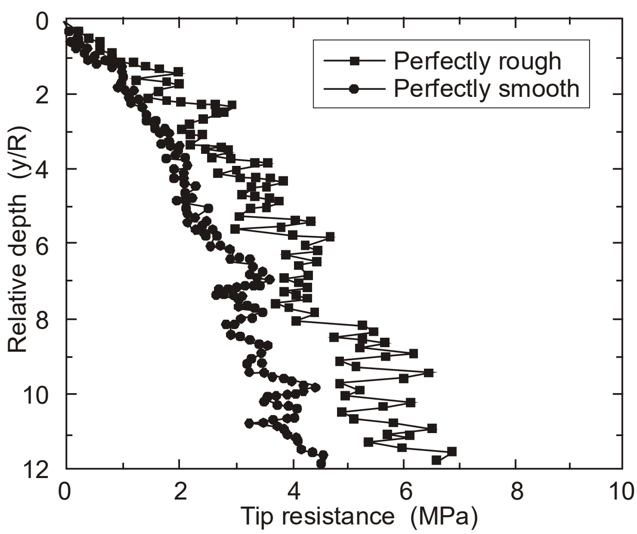

Figure 8: Penetration resistance vs. penetration depth (after Yu, 2004).

The criteria developed by the Norwegian Geotechnical Institute are very useful for assessing the quality of samples (Table 1).

The results of Tanaka (2007) and Long (2002) unequivocally prove that obtaining reliable constrained modulus reference values, which correspond to oedometer moduli, depend heavily on the quality of the tested samples.

2.5 Factors related to subsoil properties

The characteristic features of a certain group of soils in Poland, as mentioned in Section 2.1, are their macrostructure and cementing effect. These elements have a significant impact on the parameters recorded in the CPTU and DMT, which will be used to predict soil deformation moduli in the subsoil. Fig. 12 a and b show the course of the penetration process in varved clay (Młynarek et al., 1982). The test was performed with a mini cone under strictly controlled laboratory conditions. Fig. 13 documents the influence of the lamination direction on the recorded values of the dimensionless cone resistance $q_c/\gamma_d D$.

Fig. 13 clearly shows that the influence of the direction of lamination has a major impact on the cone resistance values, with constant physical parameters of the tested soil.

The second characteristic element for soils from Poland is shown in Fig. 14 (Stefaniak, 2014). The

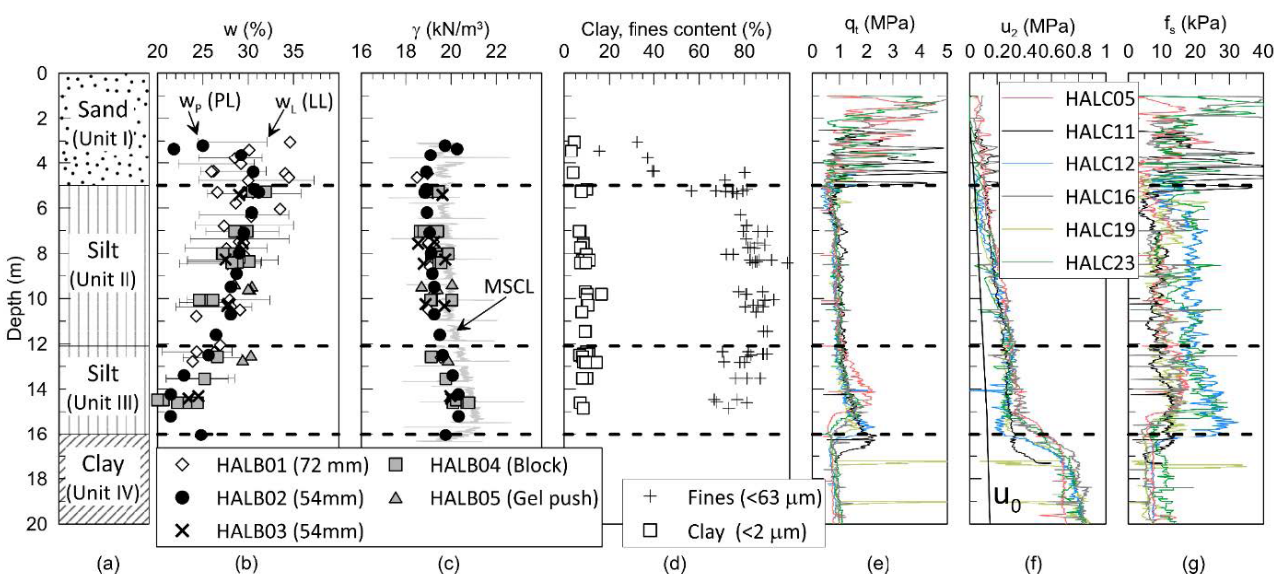

Figure 9: Classification and CPTU data; (a) Soil units, (b) natural water content and Atterberg limits, (c) total unit weight, (d) clay particle and fines content, (e) corrected cone resistance, q_t, (f) pore pressure, u_2, and (g) sleeve friction, f_s (after Paniagua et al. 2021).

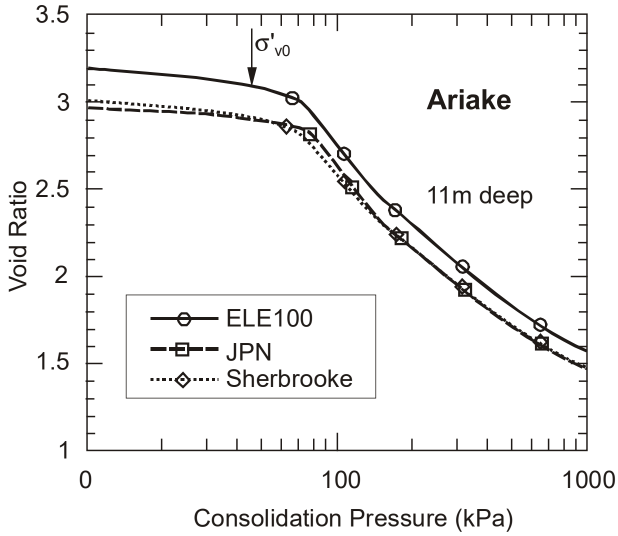

Figure 10: *e*-log *p* relationship for different sample quality for Ariake clay (after Tanaka 2007).

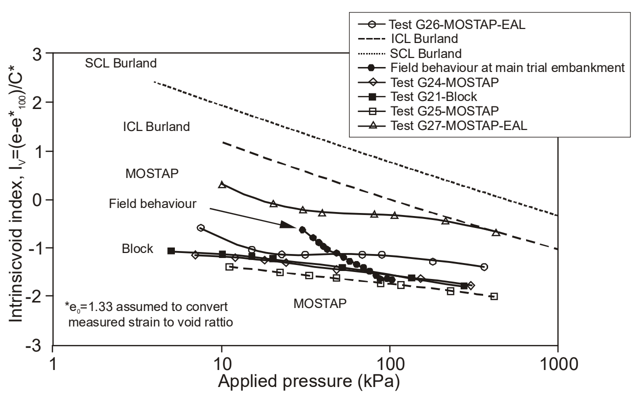

Figure 11: Normalized I_v compression curves—Athlone grey organic clay (after Long, 2002).

Table 1: Criteria of the Norwegian Geotechnical Institute for the evaluation of sample quality

OCR	$\Delta e/e_0$			
	Very good	**Good**	**Average**	**Poor**
1–2	<0.04	0.04–0.07	0.07–0.14	<0.04
2–4	<0.03	0.03–0.05	0.05–0.10	<0.04

where e_0—initial vid ratio under in situ conditions, Δe—volume change when consolidating back to in situ stresses. Note: The above set of criteria are valid for soft marine clays. For other soil types it should be used with great caution.

cementation effect, occurring mainly in silty sediments and fine and silty sands, causes strong stiffening of this part of the subsoil. Cone resistance and parameters from the DMT register this effect very well in these soils, as well as in varved clay. Separate relationships between CPTU, DMT, and constrained modulus and shear modulus G_0 parameters must be searched for these soils (Jamiołkowski et al., 2001; Młynarek et al., 2015).

3 Geological characteristics of the test sites

This article uses the results of research carried out in various regions of Poland, differing in structure and geological history of sediments (Fig. 15).

The largest group of locations is the one in which sediments of the youngest of the Scandinavian glaciations—Weichsel glaciation—were studied. Glacial clays of the Weichsel glaciation from Darłowo, Jarosławiec, Barwic, and Starogard can be divided into two groups of sediments. The first group is glacial clay of an older level, associated with the transgression and regression of the Poznań phase. The second group consists of younger settlements, associated with the transgression and regression of the Pomeranian phase of this glaciation.

The sediments found in Derkacze, Budzyń, Batkowo, Kaźmierz, Chełmno, Rzepin, Poznań, and Lipno are glacial clays and interglacial silts lying in the zone, which the Pomeranian phase of the Weichsel glaciation no longer reached. These sediments are classified as sandy loams and loamy sands because of their texture.

Glacial clays from Jarocin, Krotoszyn, and Koźmin are located in the Riss glacial zone on the outskirts of the Weichsel glacial line. The dominant soils in the profile are sandy loams and loamy sands, which are strongly preconsolidated. A characteristic feature of these clays is the high content of calci carbonate, above 10%, reduced content of the sand fraction, as opposed to the Weichsel glaciation clays, with a simultaneous increase in the content of the silt fraction (up to 40%) (Rząsa & Młynarek, 1968). These clays are also grey–brown in color and are often called grey clays. They are considered to be very good construction subsoil.

Neogene clays were present in the tested profiles in Warsaw and Bydgoszcz. These sediments are strongly preconsolidated as a result of the impact of subsequent Pleistocene glaciations. The consistency of clay is classified as hard or compact, with the exception of top

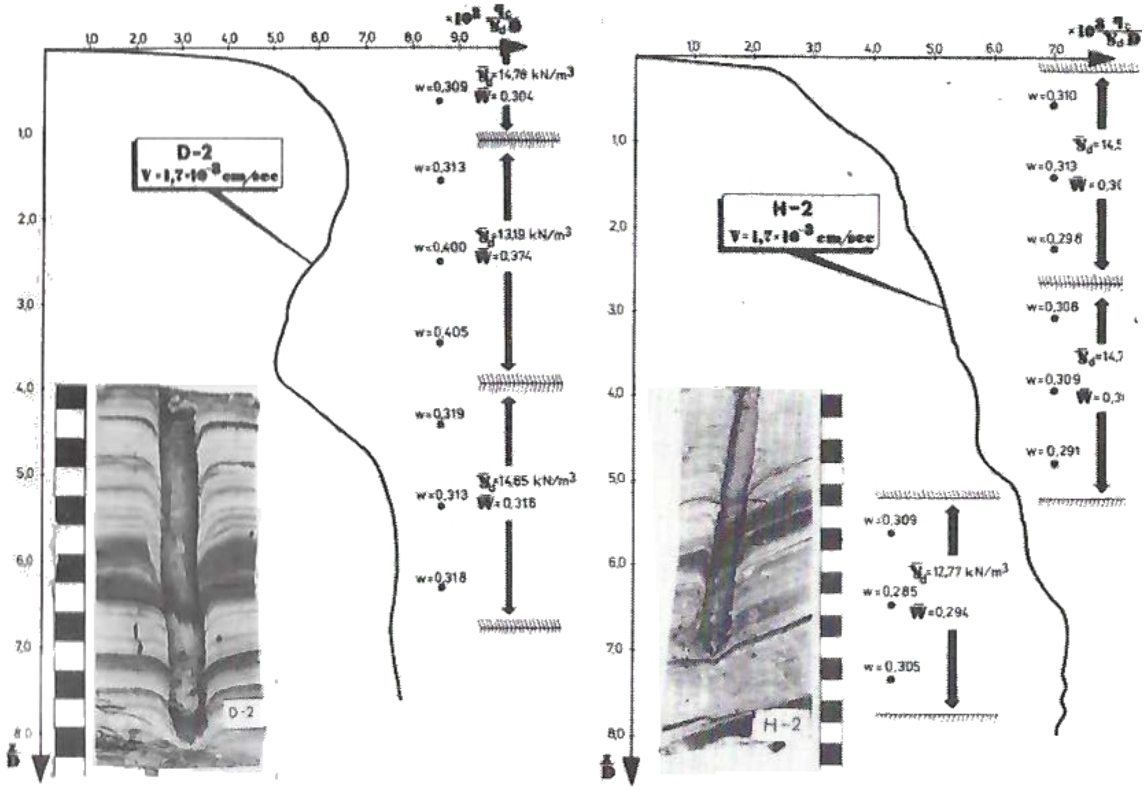

Figure 12“ Static penetration diagram for horizontal (a) and diagonal (b) lamination of clay (after Młynarek et al. 1988), where: *z*—depth of penetration, *D*—cone diameter, g_d—soil dry unit weight.

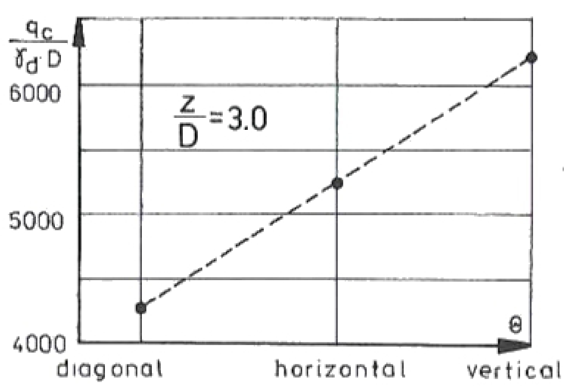

Figure 13: Relationship between mean value of coefficients of cone resistance and direction of lamination (after Młynarek et al. 1988).

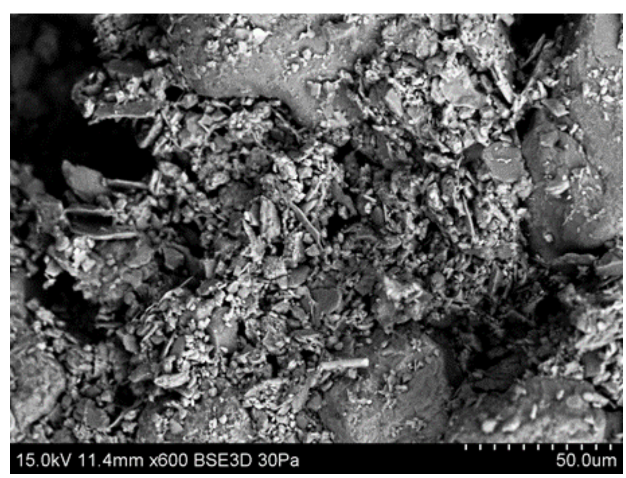

Figure 14: Calcium carbonate cementation of silts (after Stefaniak 2014).

areas in which local yielding of these soils occurs, as a result of the impact of the groundwater deposited on them.

Loess were tested in Łańcut, at the western end of the vast belt of loess covers of the Podolian Upland stretching from Ukraine (Bogucki et al., 2014). The thickness of the loess cover varies in this area from 9 to about 20 meters. These lands, created in the central and upper Pleistocene, rest on older glacial and fluvio-lacial formations associated with the Mindel glaciations. Loess are, in the granulometric sense, silts, sandy silts and sometimes silty clays, i.e. soils corresponding to PN-ISO soils from the range of silt and sand mixtures (saSi–siCl). The individual grain fractions were in the range: 24–33% sand fraction, 55–71% silty fraction, and 7–14% clay fraction. Both

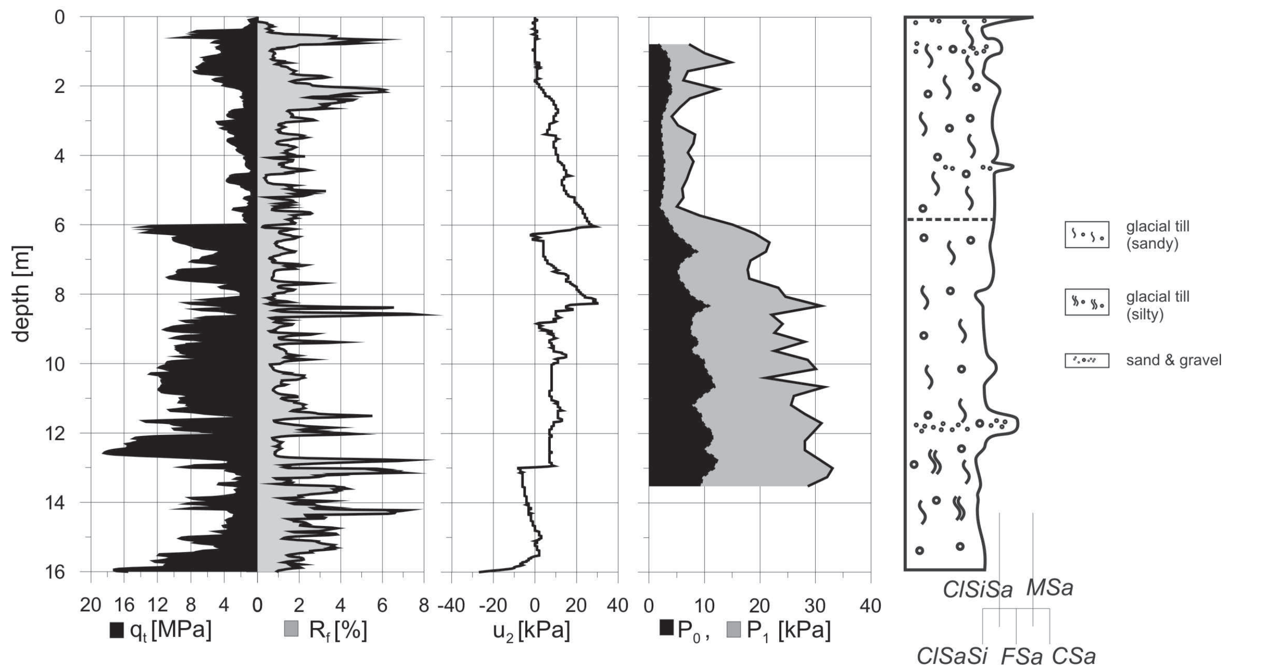

Figure 15: Sample results of CPTU and DMT tests in the analyzed soils against the lithological profile (after Młynarek et al. 2016).

Frankowski et al. (2010) and Bogucki et al. (2014) indicate at least a dichotomy of the loess profile of the Podolian Upland. The top zone of the profile (approx. 3 m in depth), is characterized by a greater possibility to collapse. The lower zone of the soil profile, despite its similar genesis and granulometric composition, is characterized by lower porosity, higher degree of humidity, and clearly lower maturity. The presence of carbonate cementation in the top part of sediments, typical for alluvial silty formations, is an additional element influencing the diversification of the loess profile (Stefaniak 2014).

4 Concept of determining the relationship between deformation moduli, small strain shear modulus G_0, and parameters from CPTU and DMT tests

The introduction of the measuring tip into the subsoil in the CPTU and DMT methods generates excess pore shearing in the subsoil. The dissipation effect of pore pressure is closely related to soil texture. In this context, Lunne et al. (1997) proposed the following subsoil subdivision for the interpretation of penetration characteristics in the CPTU method:

– fine-grained soils
– coarse-grained soils
– intermediate soils.

This division is particularly justified for the subsoil found in Poland, where there are soils with a significantly different origin and grain size composition. The division adopted in this way is also justified according to equation (2) to construct a partial function, which is the relationship between, e.g. the cone resistance and the constrained modulus with other variables established at a constant level. Such division contains the variables x_1, x_2, x_3. Equation (2) requires the condition that observation pairs for this relationship are determined each time at one stress level σ'_{vo} in the subsoil.

4.1 Constrained modulus from CPTU–fine-grained soils

The relationship between constrained modulus M, which is determined in an oedometer test, and cone resistance q_c is expressed with the relationship (Lunne et al. 1997)

$$M = \alpha_m q_c \tag{10}$$

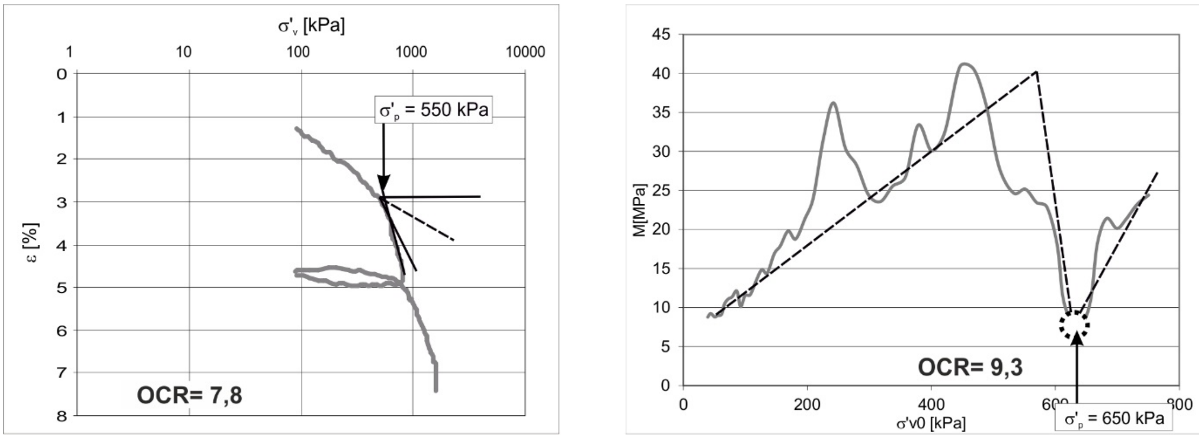

Figure 16: Results of oedometer tests of glacial tills of Posnanian phase and the values of preconsolidation stress, determined via Casagrande (left) and Janbu's (right) methods (after Wierzbicki 2010).

This relationship is empirical and in general linear interpretation models are used to determine the α_m coefficient.

In the case of CPTU, the relationship (10) is as follows:

$$M = \alpha_i\, q_n = \alpha_m\,(q_t - \sigma_{v0}) \tag{11}$$

where:

$$q_t = q_c + u_2(1 - \alpha) \tag{12}$$

u_2 – pore pressure acting behind the cone, α – cone area ratio

In the normally consolidated stress range, Senneset et al. (1989) proposes the relationship α_i – between 4 and 8

A more general relationship suggested Kulhaway & Mayne (1990)

$$M = 8.25\,(q_t - \sigma_{v0}) \tag{13}$$

The Hyson 20Tf static probe from AP van den Berg from the Netherlands was used to carry out detailed tests in order to assess the values of the α_i and coefficients for soils from Poland.

In order to determine the effect of preconsolidation on the relationship recorded by equation (10), the study was conducted in Szczecinek, where the subsoil was characterized by strongly preconsolidated Posnanian phase moraine clays and in the subsurface zone, Pomeranian phase moraine clays (Fig. 16). These subsoil zones differed significantly in the values of overconsolidation ratio—*OCR*. Oedometer reference tests were performed (Fig. 17) to determine values of the constrained modulus from the CPTU, SDMT, and the

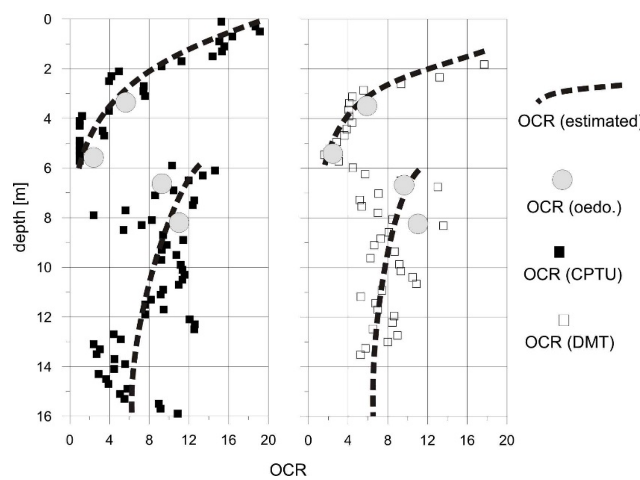

Figure 17: Changes in *OCR* in the glacial till profile (after Młynarek et al. 2016).

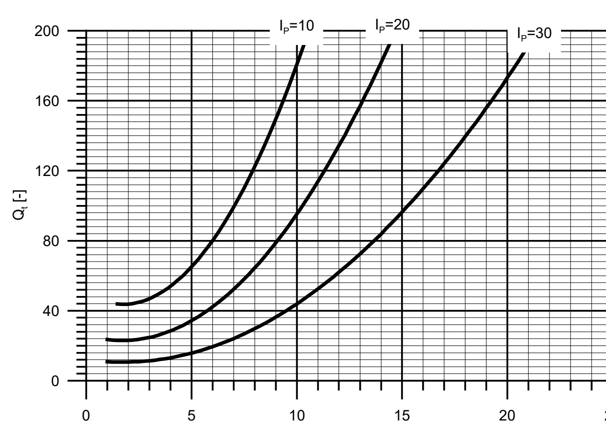

Figure 18: Nomogram for calculating the *OCR* values of cohesive soils with plasticity index I_p<30%, based on the Q_t parameter and the I_p value (after Wierzbicki 2010).

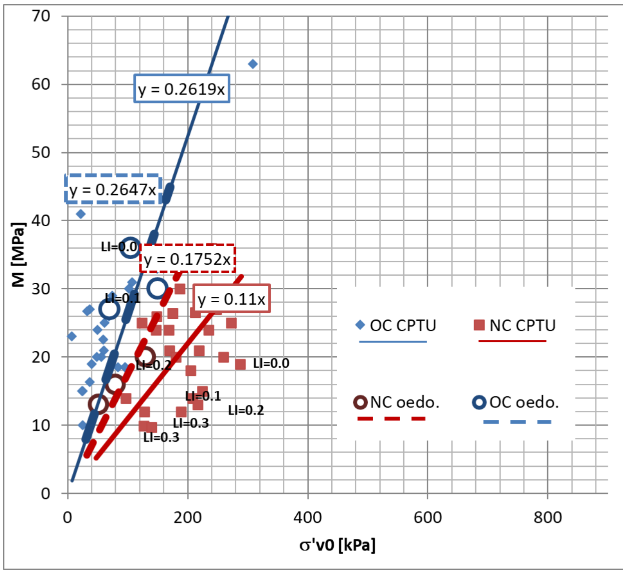

Figure 19: M_{CPTU} and M moduli variation in comparison to σ'_{vo} (after Młynarek et al. 2016).

profile of changes of this modulus in the subsoil. For laboratory tests, samples were taken with the AP van den Berg MOSTAP probe, and oedometer tests were performed in a Geonor oedometer, according to the CRS oedometer method (Sandbaekken et al., 1986).

Fig. 17 shows the assessment of the *OCR* and its changes in the subsoil obtained from the CPTU and DMT and the reference oedometer test. Wierzbicki's nomogram was used to determine the value of the *OCR* from the CPTU (Fig. 18).

The evaluation of the α_i coefficient values in equation (11) can be obtained by analyzing changes in the oedometric compressibility modulus of M_{oedo} and the modulus from the CPTU test, with a change in stress σ'_{vo} in the subsoil. Constrained modulus M_{CPTU} was calculated using the values of $\alpha_i = 8.25$ (equation 13). Fig. 19 well documents the impact of the preconsolidation effect on the value of the coefficient α_i. The conducted research made it possible to formulate a significant statement that has practical recommendations to use the values of the $\alpha_i = 8.25$ coefficient for preconsolidated clays and $\alpha_i = 13.23$ for clays normally consolidated (Młynarek et al., 2016, Wierzbicki, 2010).

4.2 Coarse-grained soils

Soils from the coarse-grained soils group play an important role in Poland. This group consists of sands, gravel, and sandy gravel of different origins. An important element that determines their strength and deformation parameters is the mineralogical composition of grains (Jamiolkowski et al., 2001). A detailed analysis of the relationship between the cone resistance q_c and the M_0 modulus was carried out by the Norwegian Geotechnical Institute (Lunne, Christopherson, 1983). The results from the calibration chamber tests determined the following relationships:

NC sands

$M_0 = 4q_c$ $\quad q_c < 10$ MPa (14)

$M_0 = 2\,q_c + 20$ (MPa) $\quad 10$ MPa $< q_c < 50$ MPa (15)

$M_0 = 120$ MPa $\quad q_c > 50$ MPa (16)

OC sands

$M_0 = 5q_c$ $\quad q_c < 50$ MPa (17)

$M_0 = 120$ MPa $q_c > 50$ MPa (16)

M_0 is the target modulus at in situ stress condition σ'_{vo}. On the other hand, tangent modulus applicable for stress range σ'_{vo} +/- $\Delta\sigma'_{vo}/2$ can be calculated from the relationship (Lunne et al., 1997):

$$M = M_0\,\frac{\sqrt{\sigma'vo + \Delta\sigma'v/2}}{\sigma'vo} \qquad (19)$$

Literature also provides relationships that determine the constrained modulus M_0 on the basis of the cone resistance at different values of the *OCR* (e.g. Eslaamizaad, Robertson, 1996). In-house research gives grounds to conclude that the relationships provided by Lunne and Christopherson (1983) use the constrained modulus to determine the cone resistance for this group of soils from Poland.

4.3 Intermediate soils

Silty and loamy sands qualify for this soil group. It is characterized by heterogenous grain size, as well as origin, and often the effect of cementation. For silty soils, Lunne et al. (1997) recommend the following relationships in order to determine the constrained modulus:

$q_t > 25$ MPa $\quad M_0 = 2q_t$ MPa (20)

$2.5 < q_t < 5$ MPa $\quad M_0 = (4q_t - 5)$ MPa (21)

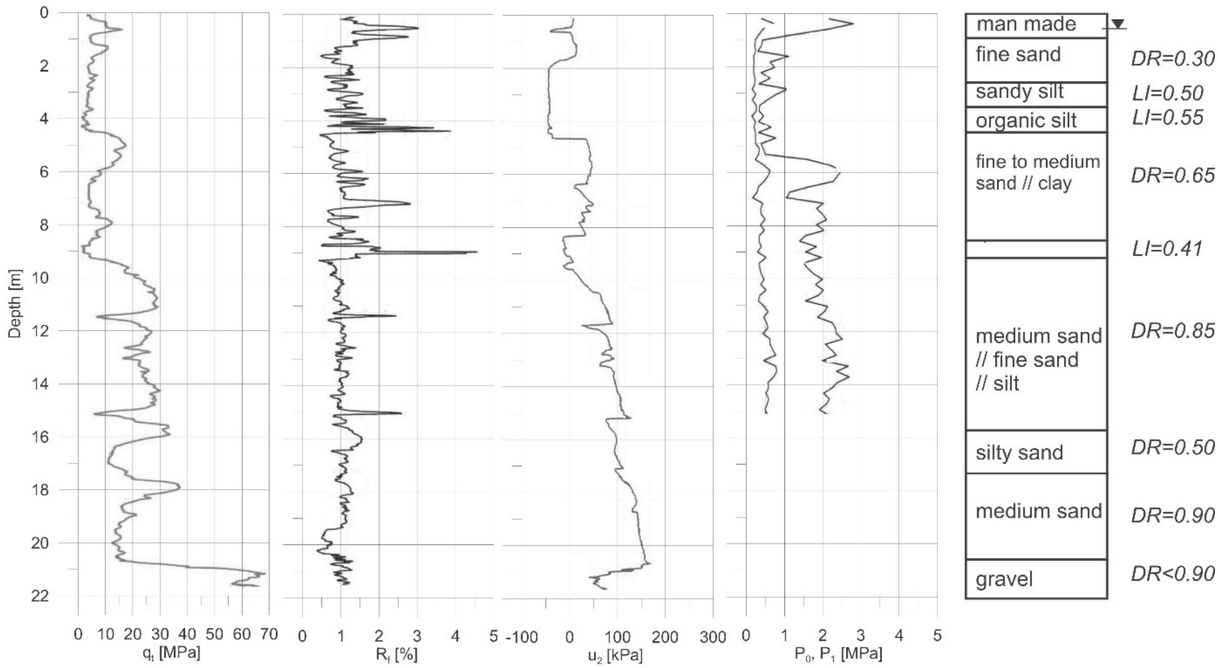

Figure 20: Typical soil profile based on CPTU and DMT test results (Poznań test site), *DR* – relative density *LI* – liquidity index (Młynarek et al. 2012).

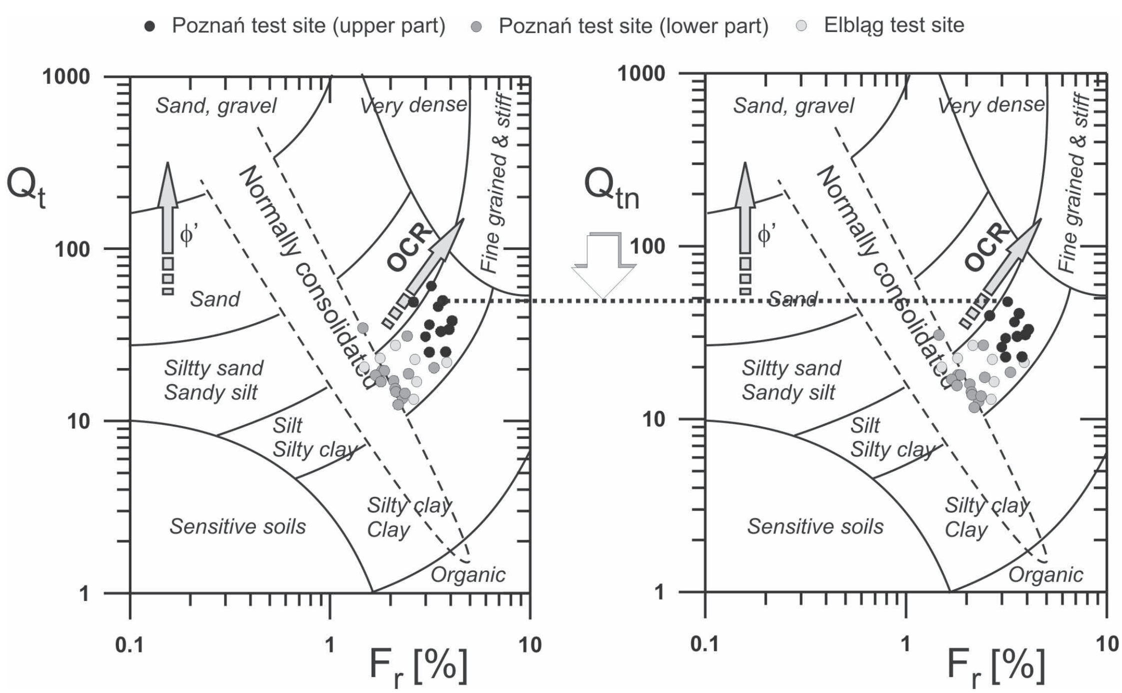

Figure 21: Location of the investigated soils on SBT (left) and normalized SBTn (right) classification charts (Młynarek et al. 2012), where $Q_{tn} = (q_n/\sigma_{atm})(\sigma_{atm}/\sigma'_{v0})^n$, $n = 0.381 I_c + 0.05(\sigma'_{v0}/\sigma_{atm}) - 0.15$ (Robertson 1990).

In-house research has shown that these relationships are very useful to forecast the value of the constrained modulus in the subsoil, where loamy sands and silty sands can be found. As for the silt and clay silt group, greater compliance with the constrained modulus with the oedometer reference tests is obtained using the formula of Mayne (1990)—formula (13).

Alluvial soils, loess and sandy loams, and loamy sands occupy a special position in the intermediate soils group in Poland. In this group of soils there are two

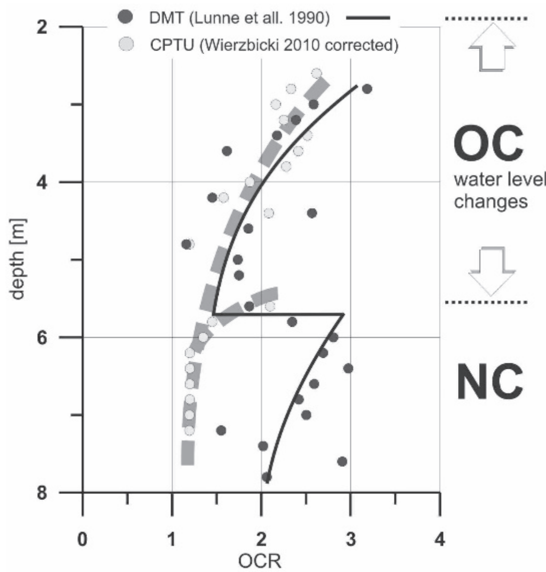

Figure 22: Changes in *OCR* with depth for the Poznań test site (Młynarek et al. 2012).

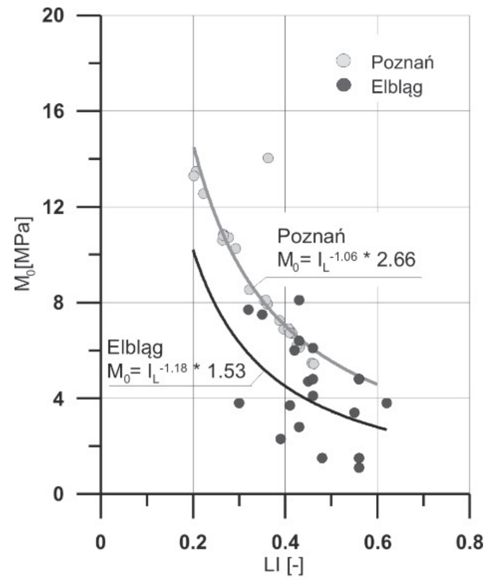

Figure 24: Relationship between constrained modulus M_0 and liquidity index *LI* (Młynarek et al. 2012).

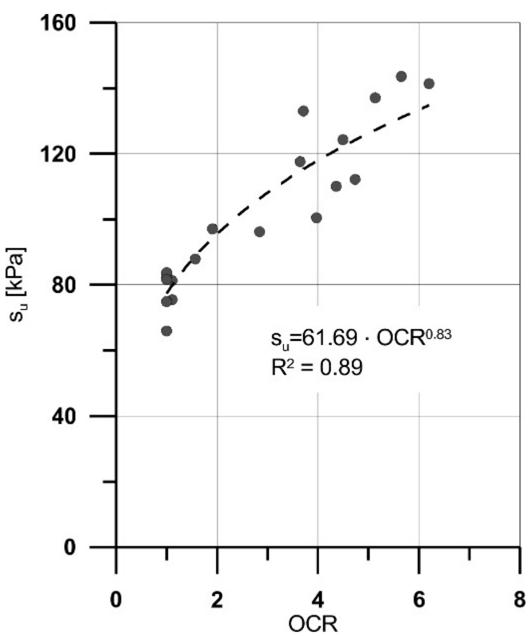

Figure 23: Relationship between constrained modulus M_0 and overconsolidation ratio *OCR* for the Poznań test site (Młynarek et al. 2012).

previously mentioned effects, i.e. preconsolidation and cementation. Detailed test results for these sediments are presented in the works of Młynarek et al. (2012, 2015). In order to determine the impact of the preconsolidation effect on the value of the constrained modulus in the group of intermediate soils represented by sandy loams and loamy sands, tests were carried out in two locations, namely Poznań and Elbląg. Fig. 20 shows a typical profile of the subsoil in Poznań and parameters from CPTU, while DMT in Fig. 21 documents the classification of soil in the subsoil from Poznań and Elbląg test sites to the intermediate soils group. The variability of the effect of subsoil preconsolidation from the Poznań test site is shown in Fig. 22. The OCR coefficient from the DMT was calculated based on the relationship proposed by Lunne et al. (1990)

$$OCR = 0.3\, K_D^{\;1,17} \tag{22}$$

where $K_D = (p_0 - u_0)$ p_0 = corrected pressure from DMT test, u_0 – hydrostatic pressure on σ'_{v0} level for measured parameter p_0.

The *OCR* for the CPTU test was determined from the nomogram—Fig. 18.

Fig. 22 shows that for the zone of normally consolidated subsoil, a differential assessment of the *OCR* coefficient values from CPTU and DMT is obtained. The obtained result justifies the comment presented in Section 2.2 regarding the recording of stress σ_h in CPTU and DMT. The influence of the preconsolidation effect on the change of the constrained modulus in the subsoil is shown in Fig. 23.

A significant relationship between the change in the state of the soil, defined by the liquidity index *LI* and the constrained modulus M_0 for soils from both locations is illustrated in Fig. 24. The influence of both variables, i.e. the *OCR* and the liquidity index *LI* on the change in the

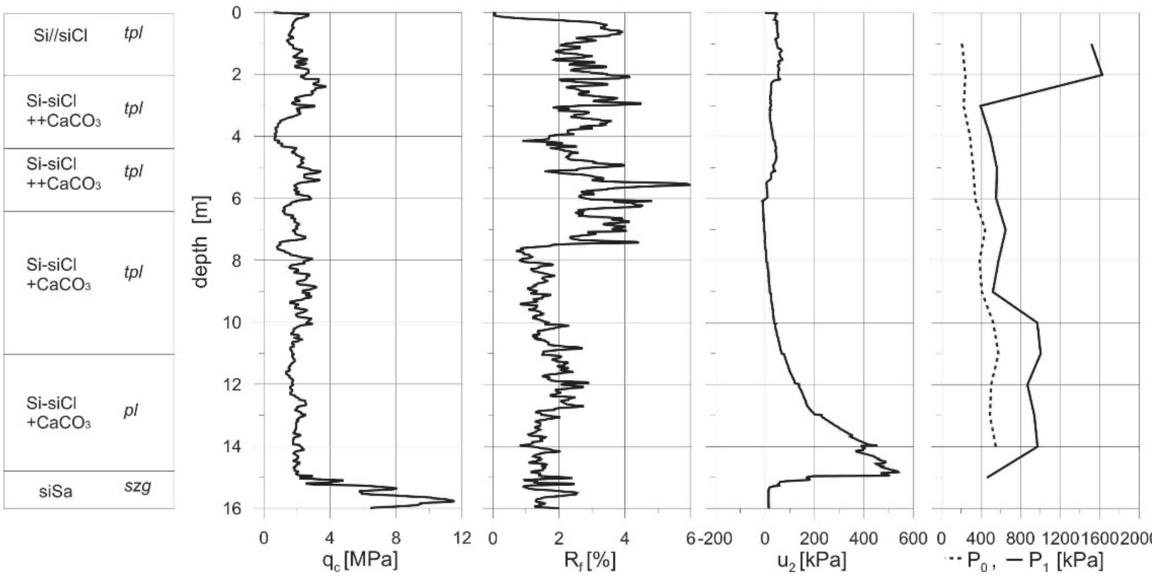

Figure 25: CPTU i DMT results in relation to geotechnical profile at example testing point (after Młynarek et al. 2015).

constrained modulus can be presented in the following empirical relationship:

$$M_0 = 22.16 - 1.16LI - 0.19\,OCR \tag{23}$$

Equation (23) has a significant statistical value. The *OCR* for this relationship was adopted according to Wierzbicki (2010).

5 Interrelationship between constrained modulus M_0 from CPTU and DMT

The following Marchetti procedure is commonly used to determine the constrained modulus M_0 from the dilatometer test (1980):

$$M_0 = R_m\ E_D \tag{24}$$

$$E_D = 34.7\,(p_1 - p_0) \tag{25}$$

$$K_D = \frac{Po - \mu o}{\sigma' vo} \tag{26}$$

$$R_M = 0.14 + 2.36 \log K_D \quad \text{if } I_D \le 0.6 \tag{27}$$

$$R_M = 0.5 + 2 \log K_D \quad \text{if } I_D \ge 3.0 \tag{28}$$

$$R_M = R_{m.0} = (2.5 - R_{m.0}) \log K_D \quad 0.6 < I_D < 3.0 \tag{29}$$

$$R_M = 0.14 + 0.15\,(I_D - 0.6) \tag{30}$$

$$R_M = 0.32 + 2.18 \log K_D \quad \text{if } K_D > 10 \tag{31}$$

where p_1, p_0—corrected pressure of the dilatometer test.

Owing to different load directions in the CPTU—vertical test and the DMT—horizontal test (Fig. 1), in the case of soils with exposed structure and cementation, the influence of these factors on the CPTU and DMT constrained modulus determined from the tests should be taken into account. An example of this type of sediment are loess. The study of these soils was carried out in the vicinity of Łańcut (Mlynarek et al., 2015). Fig. 25 shows a typical geotechnical profile from the test sites, while Fig. 26 shows the location of these soils in the CPTU classification system, DMT. Two zones can be clearly separated in the test medium. The upper zone, which is characterized by high heterogeneity of the macrostructure due to cementation (Fig. 19) and the effect of preconsolidation (Fig. 26).

The lower zone classifies the subsoil as normally consolidated. The influence of the abovementioned factors on the determined values of the constrained modulus from both studies is well illustrated in Figs. 27 and 28. For the lower zone of the subsoil, a relationship was established between the two moduli. This relationship is defined by the empirical relationship:

$$M_{DMT} = 0.021\,M^2{}_{CPTU} + 0.711\,M_{CPTU} \tag{32}$$

The differential impact of the preconsolidation effect on the value of the constrained modulus from CPTU

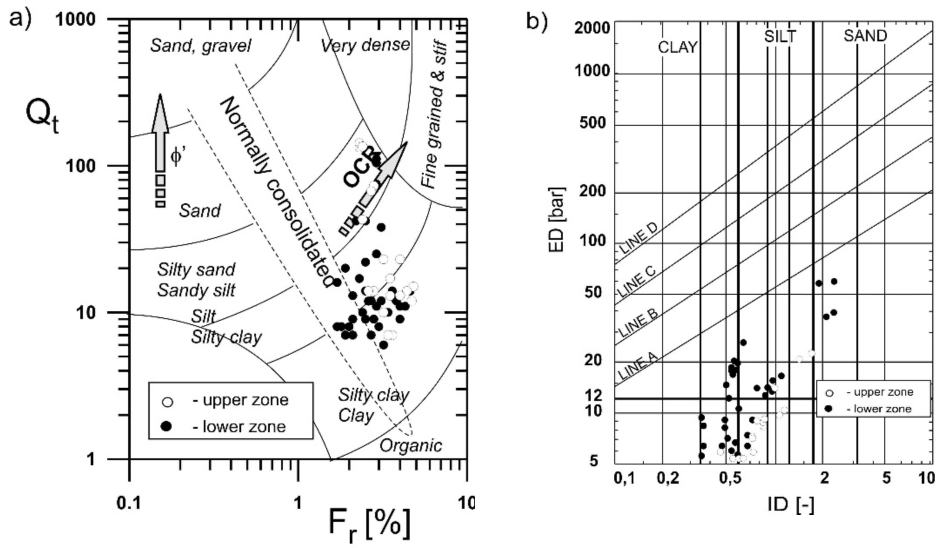

Figure 26: Position of tested loess soils in the CPTU classification system by Robertson (1990) (a) and the DMT classification system by Marchetti-Craps (1981) (b) (after Młynarek et al. 2015).

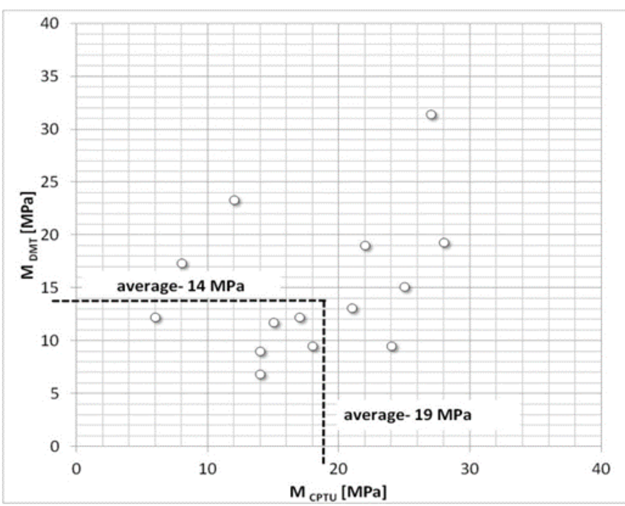

Figure 27: A relationship between constrained moduli from CPTU and DMT for the upper zone of the loess subsoil (after Młynarek et al. 2015).

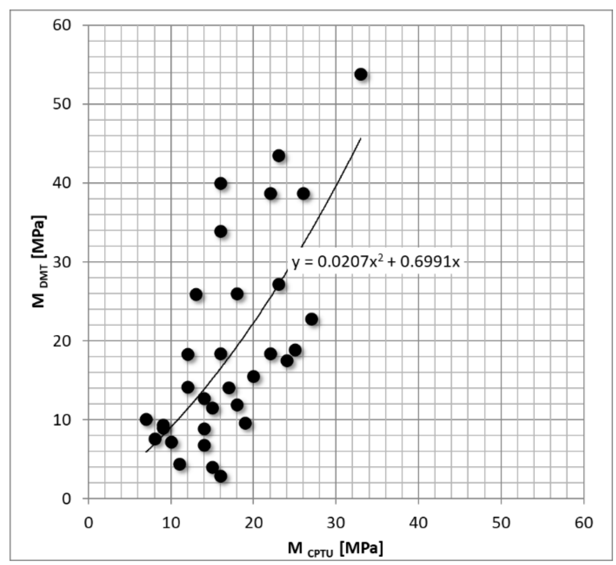

Figure 28: A relationship between constrained moduli from CPTU and DMT for the lower zone of the loess subsoil (after Młynarek et al. 2015).

and DMT tests is also noticeable in moraine clays. This problem is well justified by previously commented research from the vicinity of Poznań. Fig. 29 points out a very interesting observation that in the plastic states of moraine clay, the effect of preconsolidation disappears and the constrained moduli from both studies are very similar. Lechowicz et al. (2011) point out that in order to determine the constrained modulus from DMT in heavily preconsolidated clays, a correction of the R_m coefficient should be made in the formula (24). High values of the constrained modulus obtained from the original formula may lead to underestimate the expected settlement of the structure, as previously mentioned.

The fact that the assessment of constrained moduli from CPTU and DMT for organic subsoil is compatible can be considered an interesting issue. It is also important due to generally known difficulties in obtaining high-quality samples for reference laboratory tests. Thus, CPTU and DMT methods seem to be beneficial for determining the profile of constrained moduli changes in this subsoil. In

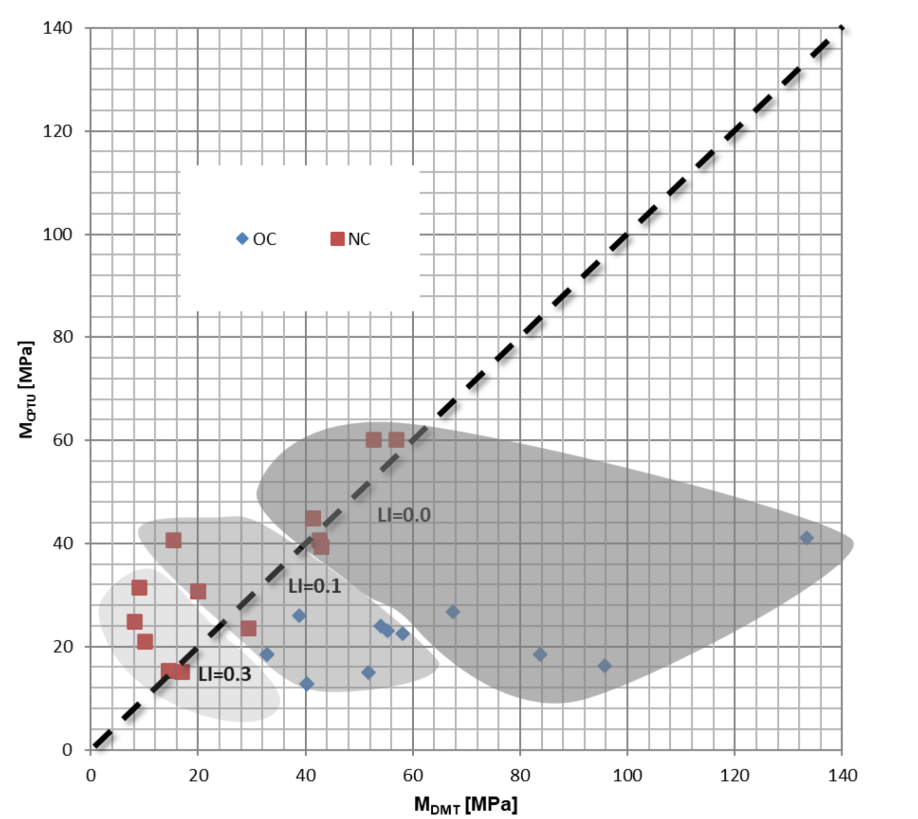

Figure 29: Comparison of M_{CPTU} and M_{DMT} values with M_{oed} modulus (after Młynarek et al. 2016).

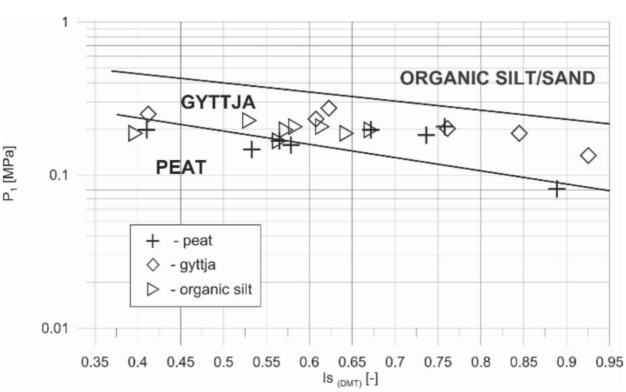

Figure 30: Position of tested soils in the classification diagram by Rabarijoely (2013) (after Młynarek et al. 2015).

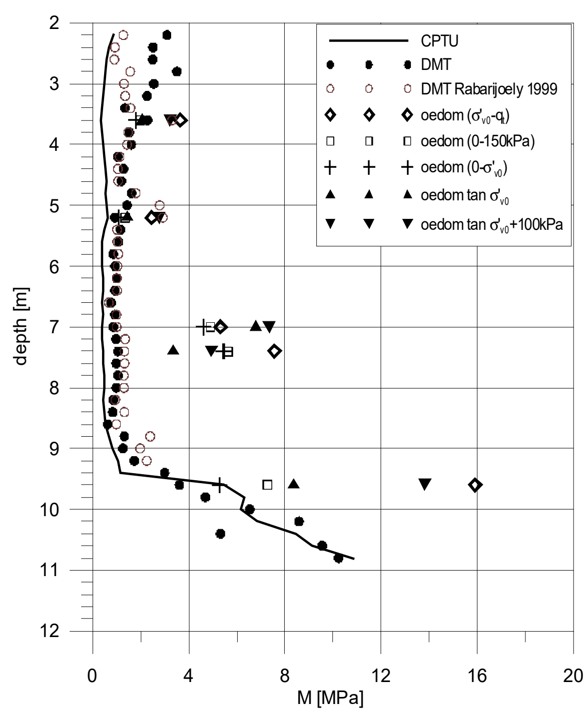

Figure 31: Changes in constrained modulus along with depth, determined using different methods (after Młynarek et al. 2006).

order to recognize this issue, the research was carried out in three locations (Fig. 15), Exemplary characteristics from CPTU and DMT for the Poznań—Bogdanka River location are shown in Fig. 30.

In order to determine the constrained modulus from the CPTU, the α coefficient in equation (11) was taken as Mitchel, Gardner (1975) 1.3 for peats, 1.6 for gyttjas, and 8,25 for silty clays. Constrained moduli for the DMT were

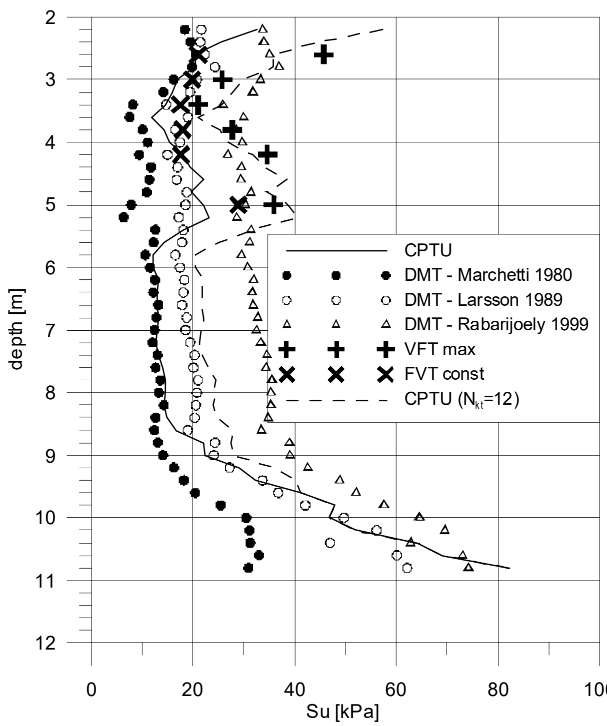

Figure 32: Changes in constrained modulus along with depth, determined using different methods (after Młynarek et al. 2006).

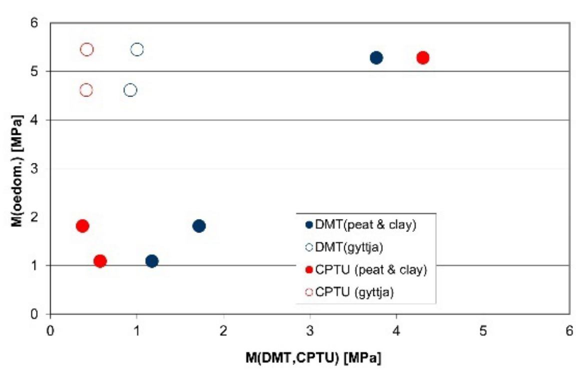

Figure 33: A comparison of moduli of compressibility determined on the basis of oedometer test with that of CPTU and DMT (after Młynarek et al. 2006).

calculated according to the relationship of Marchetti (1980) and Robarijoely (1999). The results of the research from the Poznan—River Bogdanka location (Fig. 31) prove that the adoption of appropriate values of the α coefficient in the equation (11) or the correction of the Marchetti formula (1980) is a complex problem. Although a better prognosis for the assessment of constrained modulus is obtained from the DMT test (Fig. 32), it is necessary to

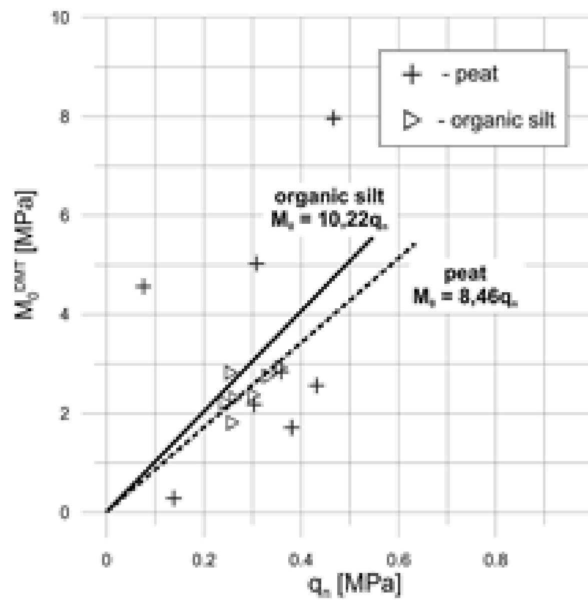

Figure 34: Correlation between constrained moduli M_0 from DMT and q_n value from CPTU Młynarek et al. 2015.

perform a reference laboratory determination for organic soils for both tests.

The cumulative results from the research at the analyzed locations (Fig. 33) confirmed the previously formulated opinion that index "α," e.g. depends clearly on the type of organic soil. The results of these studies suggest that estimated values of this index amounts to 10.2 for peats, while for organic silts (mud) amounts to 8.5. However, Figs. 33 and 34 indicate that this relationship has a relatively low statistical value.

6 The use of CPTU and DMT to assess the G_0 profile in the subsoil

To determine the profile of changes for the shear modulus G_0 in the subsoil, static probing with SCPTU seismic tip and SDMT type dilatometer test are used. These studies are also commonly used in Poland (Godlewski, Szczepański, 2013). The definition of the shear modulus and rigidity index is presented from Mayne (2006) in Fig. 35.

From the SCPTU and SDMT, the small strain shear modulus is determined from the relationship (Lunne et al., 1997)

$$G_0 = \rho V_s^2 \tag{33}$$

where: ρ - soil density, V_s - shear wave velocity.

Empirical relationships between the measured shear modulus with parameters from tests in in situ CPTU or DMT conditions are searched for due to the cost of SDMT and SCPTU. Such dependencies can be a valuable complement to SCPTU or SDMT research and this can limit their number for the upcoming geotechnical project. The second valuable advantage of these relationships is that they make it possible to determine the profile of shear modulus changes in the subsoil and to construct a 3D model of subsoil stiffness (Młynarek et al., 2007 & 2013). In constructing these empirical relationships, variables that affect the variability of shear modulus G should be taken into account (Hardin, 1979, Lee & Stoke, 1986). Functions that identify the relationship between modulus G and G_0 and variables, which describe parameters of soil medium is expressed in the following form:

$$G/G_0 = \mathrm{f}\,(\sigma'_{vo}, e_0, OCR, S, C, K, T) \qquad (34)$$

where: σ'_{vo}—effective vertical stress, e_0—initial void ratio, OCR—overconsolidation ratio, S—degree of saturation, C—grain characteristics, K—soil structure, T—temperature for noncohesive soils.

Empirical relationships for noncohesive soils between the cone resistance q_t and shear modulus G_0 taking into account some variables from equation (34) were presented by Baldi et al. (1989) in the form of

$$G_0 = \rho\,(277\, q_t^{\,0.13}\, \sigma'^{\,0.27}_{vo})^2 \qquad (35)$$

and Hegazy and Mayne (1995) for cohesive soils

$$G_0 = \rho\,(14{,}13\, q_t^{\,0.359}\, e^{-0.479})^2 \qquad (36)$$

where: ρ—soil density.

In order to determine the relationship between the cone resistance from the CPTU and the shear modulus G_0 for the subsoil in Greater Poland, where moraine clays of different degree of preconsolidation are found, tests were carried out in the vicinity of Poznań. A typical subsoil profile is shown in Fig. 36. SDMT tests were performed to determine the shear modulus G_0. Based on the replication test, the following correlation was determined:

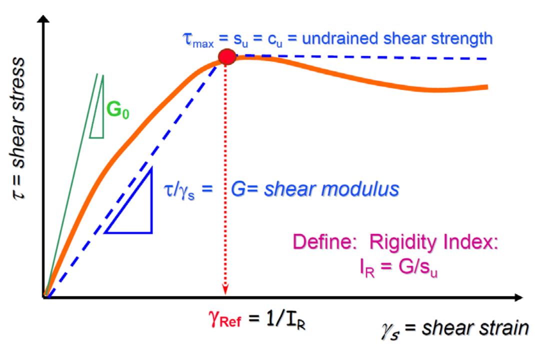

Figure 35: Shear stress vs. shear strain for soils and definition of t_{max}, G, g_s and I_R (after Mayne 2006).

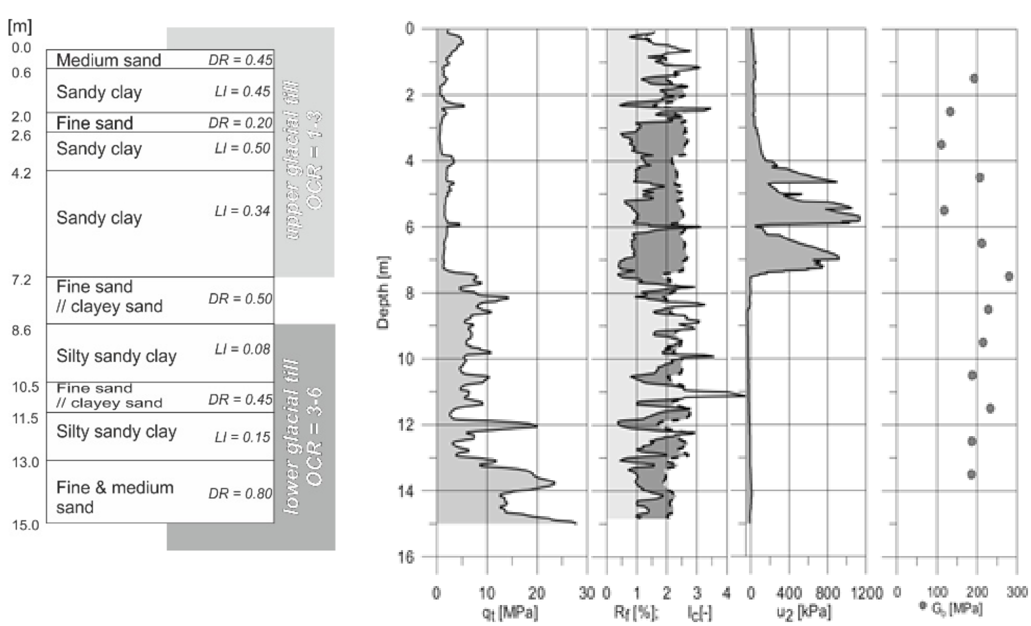

Figure 36: Typical CPTU/SDMT profile from the Poznań test site: q_t - corrected cone resistance, R_f - friction ratio, I_c – soil behavior type index, u_2 – pore pressure behind the cone, G_0 – initial shear modulus (after Młynarek et al. 2013), where LI – liquidity index, DR – relative density.

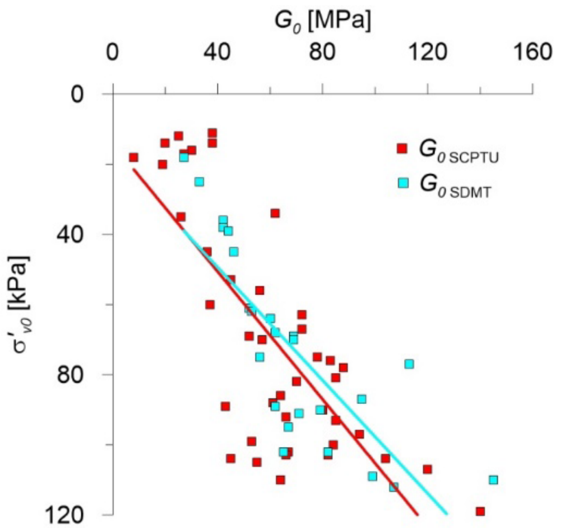

Figure 37: Trend of changes in shear modulus G_0 with depth for SCPTU and SDMT performed in normally consolidated medium sands (after Młynarek et al. 2021).

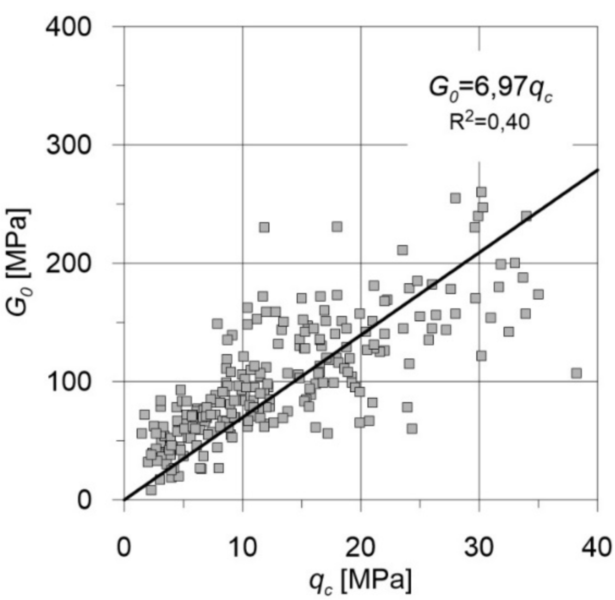

Figure 38: Correlation between shear modulus G_0 and cone resistance q_c for the entire data population (after Młynarek et al. 2021).

$$G_0 = 361 - 323.23\,LI - 0.323\,\sigma'_{v0} - 0.125\,\sigma'_p \qquad (37)$$

Preconsolidation stress values σ'_p at individual levels σ'_{v0}, where G_0 modules from SDMT were recorded, were calculated from the relationships given by Wierzbicki (2010). The basis for these calculations were the values of normalized cone resistance Q_t.

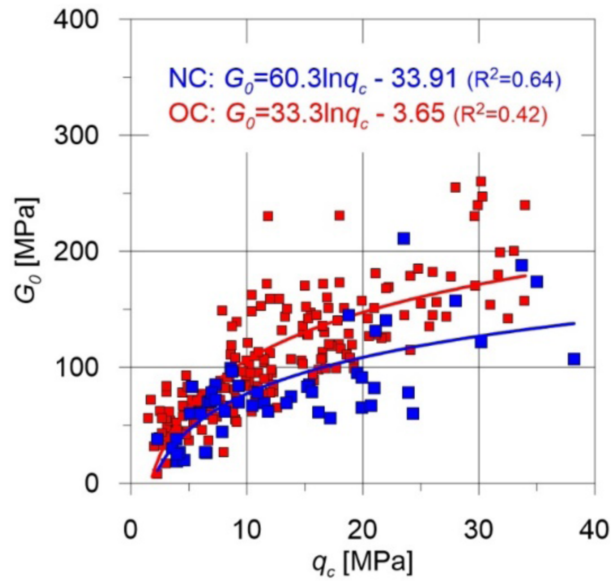

Figure 39: The correlation between modulus G_0 and cone resistance qc taking into account the division into normally consolidated (blue dots) and preconsolidated (red dots) soils (after Młynarek et al. 2021).

Owing to the significant amount of noncohesive soils of different origins in the subsoil in Poland, in order to determine the relationship between the cone resistance q_c and the constrained modulus M, we carried out tests at six locations (Fig. 15), namely: Darłowo, Derkacze, Gnojewo, Rzepin, Warsaw, and in Norway—Holmen (Młynarek et al., 2021).

The analysis consisted of 238 measurements of G_0 shear modulus values determined from SDMT and SCPTU at various levels σ'_{vo} in the subsoil. In order to eliminate the fact that the instrument affected the relationship between the cone resistance q_c and shear modulus G_0, the trend of modulus G_0 with depth was analyzed (Fig. 37).

The analysis showed that the trend of effectiveness does not significantly differ in terms of statistics. The construction of the empirical relationship between the G_0 modulus and the cone resistance q_c was carried out in three stages. The first step was to examine the basic correlation q_c—G_0 for the entire population (fig 38). This relationship is linear, but its statistical significance is not high. In accordance with equation (37), preconsolidation stress σ'_{v0} was introduced into the analysis in the second stage. The entire population of soils was divided into normally consolidated NC and preconsolidated OC (Fig. 39). The statistical assessment of this dependence was also not high. The third stage analyzed the relationships G_0 = f(q_c) in individual soil groups with the adopted division into OC and NC soils. Fig. 40 illustrates an

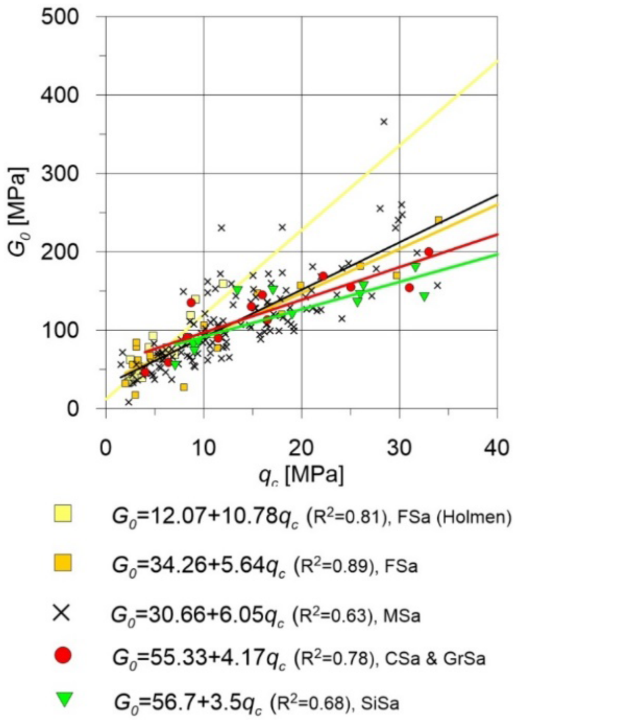

Figure 40: The correlation between modulus G_0 and cone resistance q_c for normally consolidated soils taking into account the type of soil (after Młynarek et al. 2021).

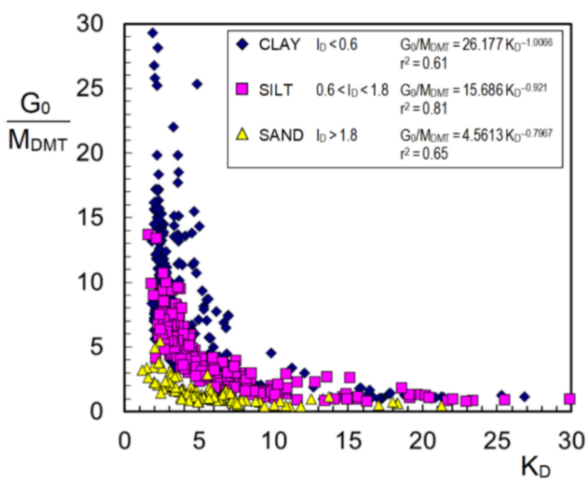

Figure 41: Relationship between the ratio G_0/M_{DMT} and K_D according to Marchetti et al. (2008) (from Monaco et al. 2009).

example of this relationship for normally consolidated soils. Preconsolidation stress σ'_p was calculated based on the Wierzbicki relationship (2010):

$$\sigma'_p = \sigma'_{v0}\,(5.52\ln Q_t\text{-}14.97) \tag{38}$$

A multivariable dependency model which highly assesses the shear modulus G_0 prognosis based on cone resistance and preconsolidation stress for individual soil groups was adopted in the third stage. For this model, the recommended dependencies are as follows:

– fine sands NC

$$G_0 = 26.197 + 0.648q_c + 0.29\,\sigma'_p$$
$$R^2 = 0.85,\ n = 43 \tag{39}$$

– - medium sands NC

$$G_0 = 12.329 - 0.23q_c + 1.06\,\sigma'_p$$
$$R^2 = 0.72,\ n = 128 \tag{40}$$

– - silty sands NC

$$G_0 = 27.316 - 0.02q_c + 0.942\,\sigma'_p$$
$$R^2 = 0.83,\ n = 14 \tag{41}$$

– - coarse sands and gravels NC

$$G_0 = 76.816 + 0.214q_c + 0.583\,\sigma'_p$$
$$R^2 = 0.51,\ n = 11 \tag{42}$$

– - fine sands OC

$$G_0 = 46.712 + 0.785q_c - 0.02\,\sigma'_p$$
$$R^2 = 0.53,\ n = 17 \tag{43}$$

– - medium sands OC

$$G_0 = 17.424 + 0.537q_c + 0.355\,\sigma'_p$$
$$R^2 = 0.68,\ n = 25 \tag{44}$$

where: G_0 [MPa], q_c [MPa], σ'_p [kPa].

A convenient way to supplement the G_0 profile obtained from the SDMT is the empirical relationship between the G_0 modulus and the parameters from the standard DMT. This type of relationship for three basic groups of soils, i.e. clay, silt, and sand was provided by Marchetti et al. (2008) (Fig. 41). This relationship has been presented in the form of the following equation

$$\frac{G_0}{M_{DMT}} = A(K_D)^B \tag{45}$$

where: G_0 – initial shear modulus, obtained from the measured shear wave velocity (eq. 33), M_{DMT} – constrained modulus (eq. 24), K_D – horizontal stress index (eq. 26).

Soils studied by Marchetti (2008) qualified as normal and slightly preconsolidated soils. To assess the suitability of Marchetti equations for soils from Poland, tests were carried out at five test sites: Kazimierz, Lipno, Jarocin, Kaźmierz, and Łańcut (Fig. 15) The analysis was carried out based on 989 SDMT test results obtained from soil deposits of different origin, macrostructure, and *OCR*. The procedure for determining correlation relationships was adopted in the same way as for the relationship $G_0 = \mathrm{f}\,(q_c)$.

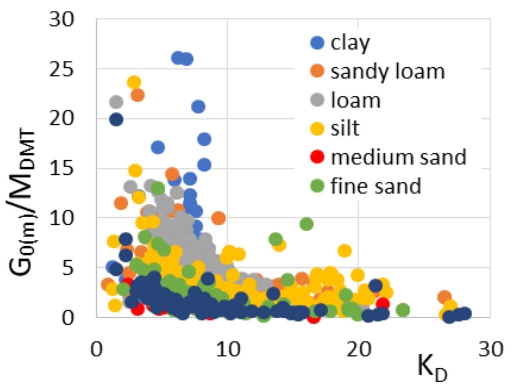

Figure 42: Relationship between the ratio $G_{0(m)}/M_{DMT}$ and K_D in different soil types from all investigated sites (after Młynarek et al. 2022).

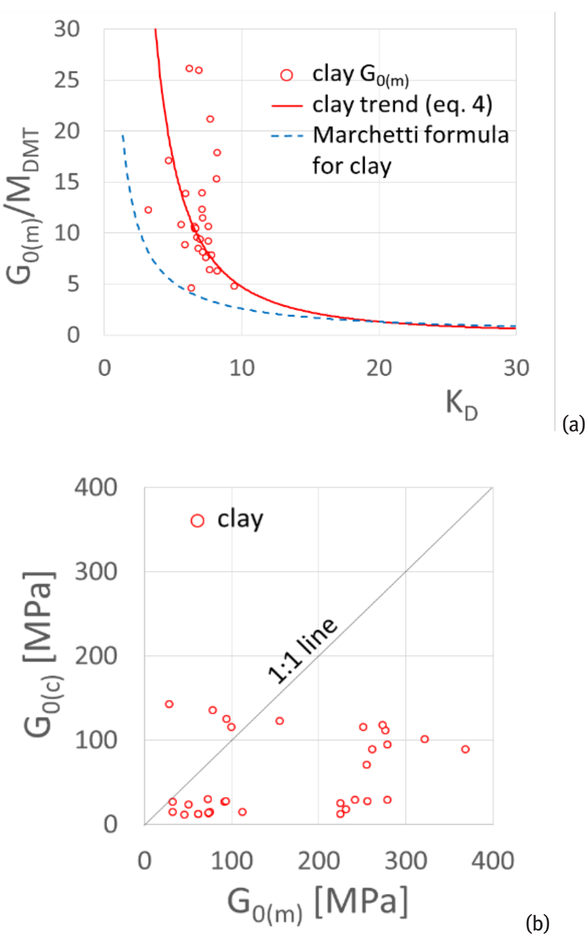

Figure 43: (a) Relationship between $G_{0(m)}/M_{DMT}$ and K_D in clay. (b) Comparison between $G_{0(m)}$ obtained from measured V_s and $G_{0(c)}$ calculated according to Marchetti et al. (2008) for clay (after Młynarek et al. 2022).

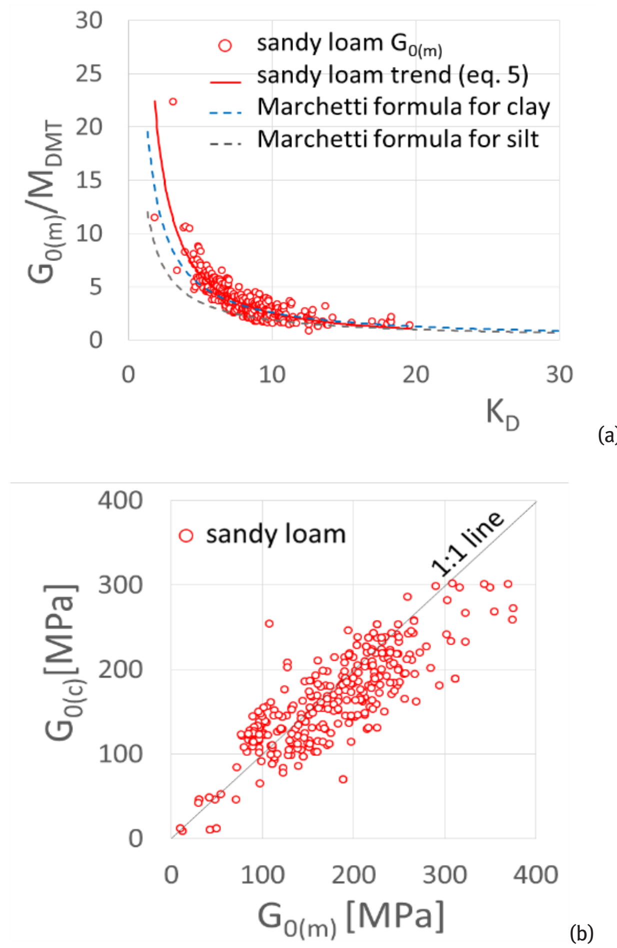

Figure 44: (a) Relationship between $G_{0(m)}/M_{DMT}$ and K_D in sandy loam. (b) Comparison between $G_{0(m)}$ obtained from measured V_s and $G_{0(c)}$ calculated according to Marchetti et al. (2008) for clay (after Młynarek et al. 2022).

In the first stage, general relationships between the $G_{0(m)}$ modules determined based on the measured V_s and the K_D coefficient were forged. Fig. 42 confirms the conclusion formulated by Marchetti et al. (2008) that functional correlation between these variables should be formed for specific groups of soils, which at least distinguish between fine- and coarse-grained soils. The conducted research showed that this correlation is also affected by the preconsolidation effect, which can be defined by σ'_p or *OCR*. This conclusion is presented well in Fig. 43, where for the strongly preconsolidated clay the value of the calculated G_0 modulus from the Marchetti formula is outside 1:1 line. A high compliance between the $G_{0(m)}$ modulus and $G_{0(C)}$ was obtained for less preconsolidated sandy loom and sands. Examples of such a relationship are shown in Fig. 44.

Table 2: Parameter values for equation (45) and coefficients of determination according to different soil types.

Soil type	Parameters values of equation (45)		Coefficient of determination
	A [-]	*B* [-]	R^2
Clay	342.75	-1.861	0.6102
Sandy loam	48.785	-1.294	0.7341
Loam	50.096	-1.114	0.6603
Silt	22.608	-0.998	0.7083
Sand	8.7499	-1.283	0.7684
Fine/silty sand	16.716	-1.184	0.6582

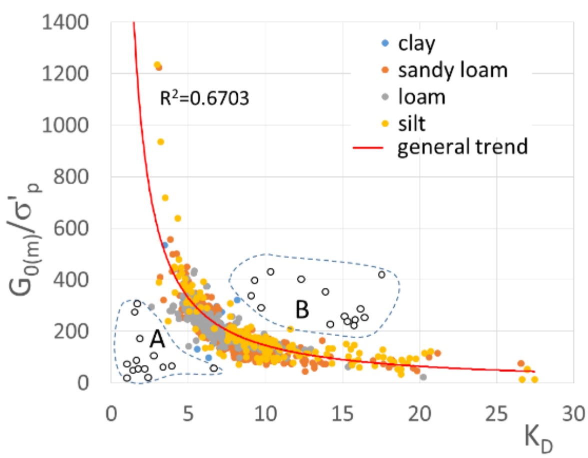

Figure 45: Relationship between $G_{0(m)}/\sigma'_p$ and K_D for sandy loam, loam, clay, and silt (after Młynarek et al. 2022).

After the calibration process, the obtained coefficients for the overall empirical dependence (45) in individual soil groups are presented in Tab. 2.

Table 2 shows that the determined correlations are of high statistical value and can be recommended for practical use. An empirical relationship, which takes into account the impact of preconsolidation stress σ'$_{p,}$ can be formed for preconsolidated cohesive soils (sandy loom, loom, clay, and silt) (Fig. 45):

$$\frac{\sigma_{o(m)}}{\sigma'_p} = 1548 K_D^{-1.058} \qquad R^2 = 0.6703 \qquad (46)$$

Zones A and B in Fig. 45 related to fissured clays and cemented silts confirm the previously formulated opinion that these soils need separate interpretation.

Table 3: Guidance for assessment of Poisson ratio.

Soil type	Poisson ratio
Dense sands	0.25 – 0.30
Loose sands, stiff clays	0.35 – 0.45
Satisfied clays	~ 0.50

7 Young modulus *E* and rigidity index I_R from SCPTU

In order to prepare the geotechnical design for many investments, including wind farms (Guidelines for Design of Wind Turbines—DNV/Riso 2002) and road facilities—guidelines for soil stabilization with rigid columns, Wydawnictwo Naukowe PWN, parameters such as Young modulus E and rigidity index I_R are necessary. CPTU and SCPTU methods are a convenient way to determine the profile of changes in these parameters on the tested subsoil. There are numerous empirical relationships between the cone resistance q_t and Young modulus E (e.g. Robertson & Cabal, 2012, Mayne, 2001, Lunne et al, 1997). An example of this is the relationship established for uncemented silica sands by Robertson (2012):

$$E = \alpha_E (q_t - \sigma_{v0}) \qquad (47)$$

where: $\alpha_E = 0.015\,[10^{\,0.55Ic + 1.86}]$, $I_c = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2]^{0.5}$, F_r—normalized friction ratio

From the solution from elastic theory, the relationship between shear modulus G and Young's modulus E is written as follows:

$$E = 2\,G\,(1+\ \nu) \qquad (48)$$

where: ν – Poisson's ratio, G – shear modulus G assigned to the appropriate strain level (see Fig. 34).

Shear modulus ratio G/G_0 is used to determine the shear modulus G. The numerical value of this ratio depends on shear strain (Mayne 2001).

Robertson (2012) states that for typical engineering structures, the G/G_0 ratio can be assumed in the ranges of 0.30 to 0.38. The instruction "Guidelines for design of wind turbines—DNV/Riso" recommends for deformations of 10^{-4} ratio G/G_0 = 0.35. This manual also gives more detailed values of the Poisson ratio (Table 3).

For wind turbine foundation projects, the Instruction for design, calculation, installation, and inspection of wind turbine foundation Rev. Francisse de Geotechnique no 138 (2012) recommends the following values G/G_0 ratio for deformation 10^{-3} to 10^{-4}:

– Clayey soils compact material – 0.33
– Compact sandy/ gravel soil – 0.50

This manual also introduces the concept of "static moduli" for deformation 10^{-2} and dynamic moduli for deformation about 10^{-6}.

The presented information proves that the registration of shear modulus G_0 in the SCPTU or SDMT test process at individual stress levels σ_{vo} in the subsoil will make it possible to determine the Young's modulus in the profile of changes in the studied area.

Another parameter that determines the rigidity of the subsoil is the rigidity index I_R presented by formula (47) (Lunne et al., 1997)

$$I_R = G/Su \tag{49}$$

where: s_u – undrained shear strength.

Determination of this parameter using CPTU and DMT is given a lot of attention in the literature.

Massarsch (2009) indicates that the $\alpha = G/G_0$ coefficient depends significantly on the soil type and the value of plasticity index PI in particular.

This coefficient denoted as R_M (Massarsch, 2009) will be determined from the formula:

$$R_M = 0.043\, PI + 0.103 \tag{50}$$

In order to obtain the profile of changes of the rigidity index in the subsoil, the most advantageous method is the CPTU method. Next, the value of s_u is written by the formula (51) Lunne et al. (1997):

$$s_u = \frac{q_t - \sigma_{v0}}{N_{kt}} \tag{51}$$

A detailed analysis of the factors affecting the values of the rigidity index for soils from Poland is presented in the work by Mlynarek et al. (2018). For this purpose, the study was conducted in 10 locations in northern, western, and southern Poland. CPTU, SCPTU, or DMT and drilling was performed at each location. The oldest studied sediments were preconsolidated Pleistocene clays found in the vicinity of Bydgoszcz and Warsaw. The group of preconsolidated sediments includes Warta glaciation clay found in the area of Derkacze (Fig. 15). The analysis also included glacial formations of the youngest glaciation, preconsolidated sediments of the Poznań phase (Jarocin and Maryszew villages) and normally consolidated soils of the Pomeranian phase from the vicinity of Bartek, Boryszew, and Rzepin. The dominant soils in this group are sandy loams and loamy sands. Another analyzed group of soils were eolic loess deposits from the vicinity of Łańcut. The last analyzed group of sediments were cohesive and organic soils, deposited in the conditions of proglacial ponding, which was found in northern Poland. Such grouping of soils allowed to analyze, as in the case of the analysis of M_0 and G_0 moduli, the impact of texture, the effect of preconsolidation and macrostructure and the effect of cementation on the rigidity index. The analysis also included the liquidity index LI, which, as generally known, has a significant impact on the change of the parameter determining the shear strength s_u and shear

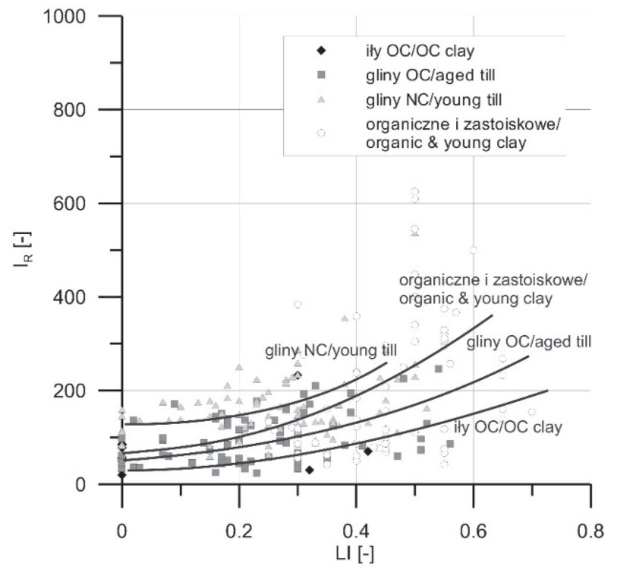

Figure 46: Rigidity index (I_R) vs. liquidity index (LI) (after Młynarek et al. 2018).

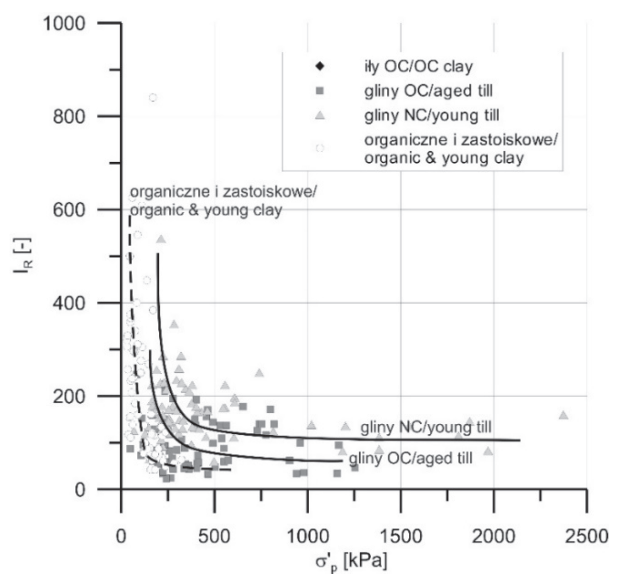

Figure 47: Rigidity index (I_R) vs. preconsolidation stress (σ'_p) (after Młynarek et al. 2018).

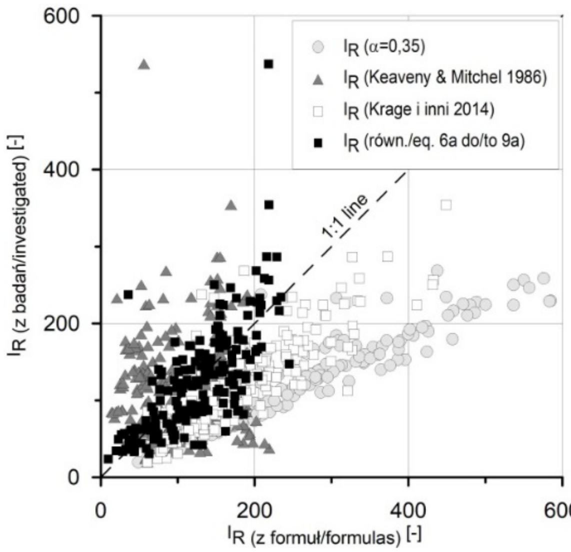

Figure 48: Comparison of rigidity index I_R determined with different formulas and obtained from investigations (after Młynarek et al. 2018).

modulus G_o. Fig. 46 shows the relationship between the rigidity index I_R and the liquidity index LI, while Fig. 47 shows the relationship between the rigidity index I_R and the preconsolidation stress σ'_p. The total impact of these variables on the rigidity index I_R was determined using multiple linear regression (Draper & Smith, 1981). The coefficient R_M was used according to the equation (50) for the calculation of the value I_R.

As a result of this analysis, the following relationships were obtained:

OC clay:

$$I_R = -38.16 + 3.81PI + 231.78LI$$
$$R^2 = 0.64 \qquad (52)$$

OC loam

$$I_R = -7.21 + 8.80\,PI - 7.89OCR + 149.19LI$$
$$R^2 = 0.64 \qquad (53)$$

NC loam, sandy loam

$$I_R = -128.05 + 386.05\,LI + 0.60\,\sigma'_{v0} + 9.45OCR + 4.75PI$$
$$R^2 = 0.26 \qquad (54)$$

Organic and alluvial soils

$$I_R = -14.04 + 4.73PI - 4.78OCR + 151.74LI$$
$$R^2 = 0.51 \qquad (55)$$

Equation (52) does not contain the OCR, because its values in the clay zone changed very little, hence the impact of this variable on the rigidity index was statistically insignificant. The obtained values of the multiple regression coefficient R^2 for NC sandy loam by 0.26 and 0.51 for organic and alluvial soil confirm that the rigidity index depends on the effect of cementation and anisotropic macrostructure of these sediments. In the case of NC sandy loam, the anisotropy of the macrostructure of these soils is associated with thin interbedding of sands. The need to search for separate relationships between the rigidity index and the parameters from the CPTU for soils from Poland is presented by Fig. 47, which shows the values of the rigidity index determined from the CPTU, equations 52–55 and the formulas by Keaveny & Mitchell (1986)—equation (56), and Krage et al. (2014) —equation (57):

$$I_R = 0.26\left(\frac{Go}{\sigma_{v0}}\right)\left(\frac{1}{0.33(0.33Q_t)^{0.75}}\right) \qquad (56)$$

where: Q_t -normalized cone resistance

$$I_R \approx \frac{\exp\left(\frac{137-PI}{23}\right)}{1^{+}1n\left[1+\frac{(OCR+1)^{3.2}}{26}\right]^{0.8}} \qquad (57)$$

Fig. 48 shows that the results for the established soil groups are well placed on the 1:1 calibration line, while the calculated values of the I_R from the Keaveny and Mitchell formula and Krage et al. are largely outside the calibration line.

8 Use of constrained modulus M_0 and shear modulus G_0 from SCPTU and SDMT to create a soil rigidity model.

One of the important tasks of the geotechnical project is to separate areas of similar strength and rigidity in the subsoil in the area of planned investment. Continuous measurement of parameters in the CPTU and DMT predisposes these methods to solve this issue. The theoretical basis for the use of the inverse distance weighting (IDW) method, the clustering of data from the CPTU or DMT and then the separation of "homogeneous" zones in the subsoil in the context of constrained modulus

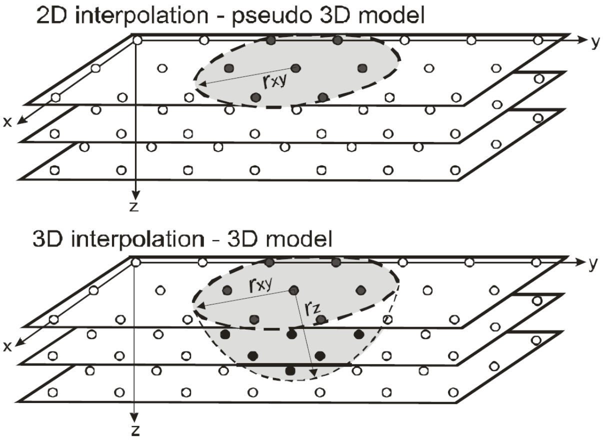

Figure 49: Scheme of quasi 3D model and 3D model for interpretation of CPTU data (after Młynarek et al. 2007).

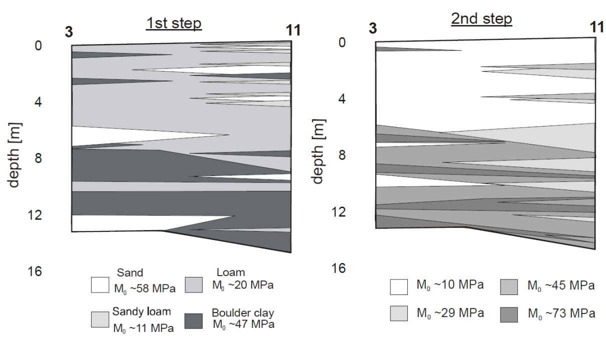

Figure 50: Deformation profile of the subsoil constructed in the 1st step and 2nd step of clustering (after Młynarek et al. 2007).

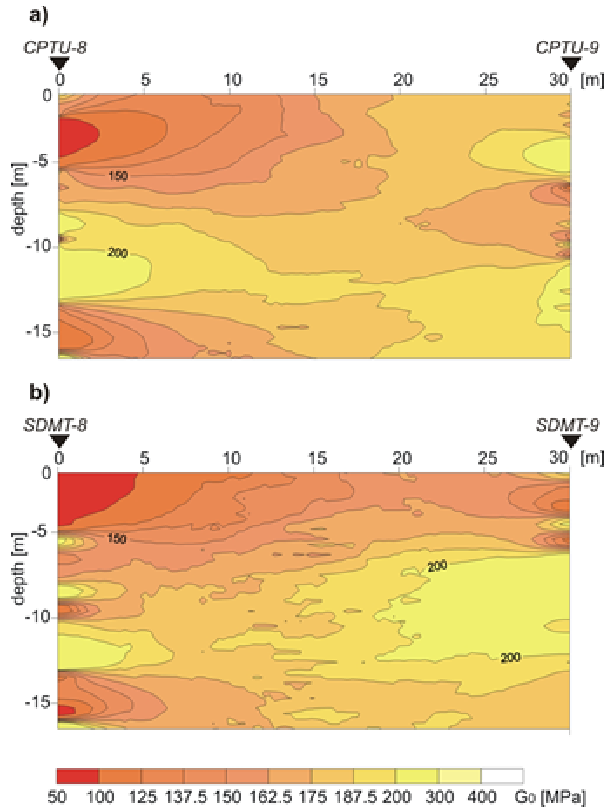

Figure 51: The model of subsoil stiffness calculated on the basis of G_0 values from CPTU results (a) and SDMT results (b) (after Młynarek et al. 2012).

M_o or shear modulus G_0 are found in the work of Młynarek (2005), Młynarek et al. (2007 & 2013).

Fig. 49 shows a diagram of data grouping from the CPTU for the construction of a quasi-3D model and 3D model. The grouping of the cone resistance values q_t allows to use the IDW method to create a model of the tested subsoil rigidity based on the value of constrained modulus M_0. Fig. 50 shows a fragment of this model. For the wind farm construction project, a subsoil rigidity model with the shear modulus G_0 (Fig. 51) was also created.

9 Summary and conclusions

The current state of knowledge justifies the advisability of using the static penetration method and dilatometer test to assess the stiffness of the subsoil very well. Even in complex geological conditions like in Poland, these methods allow to obtain the profile of changes for constrained modulus and initial shear modulus and the rigidity index for the tested subsoil. Undoubtedly, the advantage of these methods is the possibility of constructing a model of subsoil rigidity as one-dimensional, flat, and 3D. Such models allow to distinguish areas of heterogenous or similar rigidity in the area of the planned investment and to select an appropriate foundation system of the facility.

The quality of the obtained constrained modulus, initial shear modulus from these studies is influenced by several factors analyzed in the article. The quality of the CPTU and SCPTU is closely related, as the Norwegian Geotechnical Institute studies have shown, to the level of operator education and equipment quality. For this reason, the obtained moduli based on the parameters from these tests should be verified by laboratory tests of high-quality soil samples. Literary empirical relationships for the determination of the constrained modulus, initial shear modulus and rigidity index require analogous verification.

Local empirical relationships for soils from Poland, which take into account the specificity of these sediments and their genesis, are very helpful for the purpose of this verification. The search for these relationships is still a current research problem.

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