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Evaluation of Soil Strength Parameters by the *CPTU* Method

Notation

a – Attraction = $c'/\tan\Phi'$ [MPa] after Senneset-Janbu procedure,
a' – Effective area ratio of cone [–],
Bq – Pore pressure ratio [–],
c – Cohesion [kPa]
C_h – Horizontal coefficient of consolidation [m^2/s],
E – Young's modulus [MPa],
f_s – Sleeve friction [MPa],
f_T – Sleeve friction corrected for pore pressure effects [MPa],
G – Shear modulus [MPa],
I_p – Plasticity index [–],
I_r – Rigidity index = G/Su [–],
k – Pore pressure correction factor [–],
m – Oedometer modulus number [–],
m_i, m_n – In situ modulus numbers [–],

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M	–	Constrained modulus [MPa],
M_i, M_n	–	Tangent oedometer modulus *in situ* [MPa],
n	–	Young's modulus number [–],
N_c	–	Bearing capacity factor [–],
N_{kE}	–	Effective cone factor [–],
N_{kT}	–	Cone factor corrected for pore pressure effects [–],
N_m	–	Cone resistance number [–],
N_q	–	Bearing capacity factor [–],
N_{qc}	–	Bearing capacity coefficient for cone resistance [–],
N_u	–	Bearing capacity factor [–],
$N_{\Delta u}$	–	Cone factor for pore pressure $= \Delta u/Su$ [–],
OCR	–	Overconsolidation ratio [–],
PPD	–	Pore pressure difference [–],
q_c	–	Cone resistance [MPa],
q_{cl}	–	Modified cone resistance [MPa],
q_E	–	Effective cone resistance [MPa],
q_T	–	Cone resistance corrected for pore pressure effects [MPa],
R	–	Radius of the cone [cm],
R_f	–	Friction ratio [–],
R_{fl}	–	Modified friction ratio [–],
S_t	–	Sensitivity [–],
S_u	–	Undrained shear strength [kPa],
$S_{u(FV)}$	–	Undrained shear strength, field vane [kPa],
$S_{u(TX)}$	–	Undrained shear strength, triaxial compression [kPa],
t	–	Time [s],
T	–	Time factor [–],
T^*	–	Modified time factor [–],
U_o	–	Initial *in situ* pore pressure [MPa],
U_1	–	Pore pressure measured at tip of piezocone [MPa],
U_2	–	Pore pressure measured immediately behind cone [MPa],
U_c	–	Total recorded pore pressure [MPa],
U_T	–	Total adjusted pore pressure [MPa],
W	–	Water content [%],
W_L	–	Liquit limit [%],
W_p	–	Plastic limit [%],

α_{FV} – Correlation constant (FVT) [–],
β – Angle of plastification [°]
Δ_{UT} – Excess pore pressure [MPa],
μ – Field vane correction factor [–],
σ_o – Stress = σ_{vo}, σ_{ho}, σ_m [MPa],
σ_a – Reference stress = 98,1 kPa [MPa],
σ_c – Preconsolidation stress [MPa],
σ_{ho} – Total horizontal stress *in situ* [MPa],
σ_m – Mean total stress $\sigma_m = (\sigma_{vo} + 2\sigma_{ho})/3$ [MPa],
σ_{vo} – Total vertical stress *in situ* [MPa],
σ'_{vo} – Effective vertical stress *in situ* [MPa],
ϕ – Effective friction angle [°].

Test types

CAUC – Anisotropically consolidated undrained triaxial compression,
CPT – Cone penetration test,
CPTU – Piezocone penetration test,
FVT – Field vane test,
SCPT – Seismic cone penetration test,
TX – UU– Unconsolidated undrained triaxial compression.

1. Introduction

During the last twenty years, *in situ* soil tests with the *CPTU* (static penetration test with cone supplied with filter for pore pressure measurement) have confirmed the usefulness of this method in prognosing strength parameters of subsoil. This method also enables identification of subsoil stratography (Douglas, Olsen 1981; Olsen, Malone 1988; Schmertmann 1978; Tsuchiya, Muromachi, Sakai, Iwasaki 1988) and reconstruction of its stress history (Lunne, Rad, Lacasse, Decourt 1989; Mayne, Bachus 1988; Mayne, Mitchell 1988; Sugawara 1988; Sully, Campanella, Robertson 1987; Sully, Campanella, Robertson 1988). The latter aspect of the *CPTU* application opens up promising prospects for its utilization in Poland. Geological processes taking place in Poland resulted in considerable differentation of the degree of soil consolidation. This is clearly seen in the example of deposits from the Wielkopolska region, where those originating from several geological formations can be encountered as deep as over ten metres.

The paper presents interpretation of *CPTU* diagrams for the subsoil from the Poznań area. It also gives basic information concerning the methods of determining strength parameters of soil from cone resistance and the values of measured pore pressures in the soil.

2. Site and the Range of Tests

The *in situ* investigations were carried out on a plot located in the western part of Poznań, in the area of the local water-course called Junikowo Stream. It was in this area that the test points in which the following investigations were done were situated: 24 bores-holes 10-15 m deep, yielding undisturbed soil samples for strength tests, 6 penetration tests with Gouda including 4 *CPTU* tests with pore pressure dissipation and 2 *CPT* tests, 3 shears with ITB-ZW type vane apparatus. The soil structure in this area consists of: a complex of sandy holocene soils, pleistocene sands of water-glacial origin and a layer of varved clays (Fig. 1). In the soil profile two levels of ground water table appear one of which is stabilized at a depth of 0.25 m below the subsoil surface while the other strained, related to a floor of varved clays, was found at a greater depth (Fig. 1).

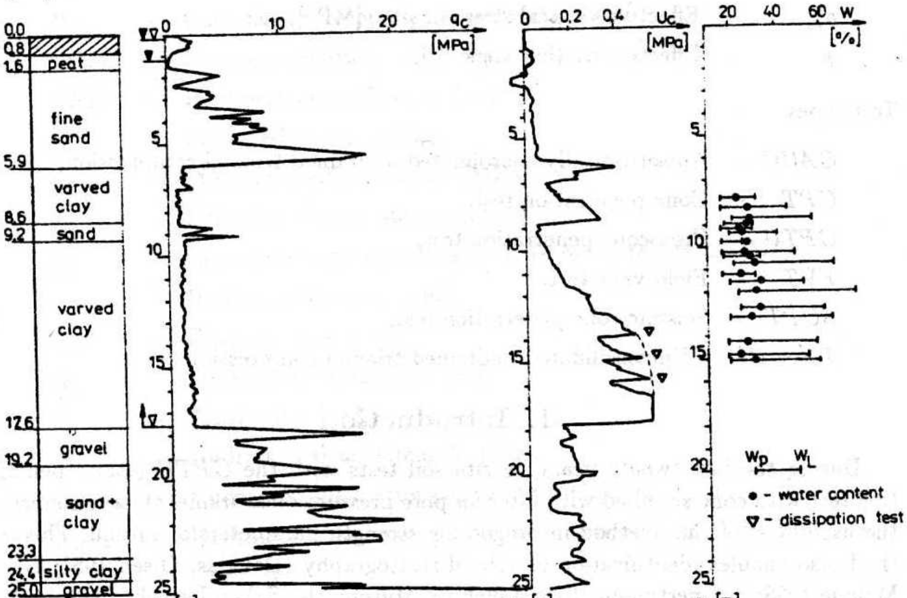

Fig. 1. Typical *CPTU* records and clay properties

3. Methods for Determining Penetration Parameters

In the penetration - *CPTU*, three independent penetration curves are registered, namely changes with depth of cone resistance - q_c, sleeve friction - f_s, and pore pressure - U_c. Knowledge about cone resistance and sleeve friction facilitates calculation of the so- called, friction ratio - R_f:

$$R_f = \frac{f_s}{q_c} 100\% \tag{1}$$

In order to discriminate geotechnical strata from the above parameters, Harder-Bloh's procedure was used (Harder, Von Bloh 1988). For the discriminated layers the procedure of data "smoothing" was applied using a "movable" mean (Werno, Dembski, Młynarek, Sulikowska 1990), due to which the initial number of data and, so-called, "zig-zag" effects were reduced, and replaced by a set of mean values. Hence, the layers discriminated in the complex of varved clays with similar grain size composition, showed differences in consistency. The penetration characteristics averaged in this way were significantly consistent with the profile structure (Fig. 2).

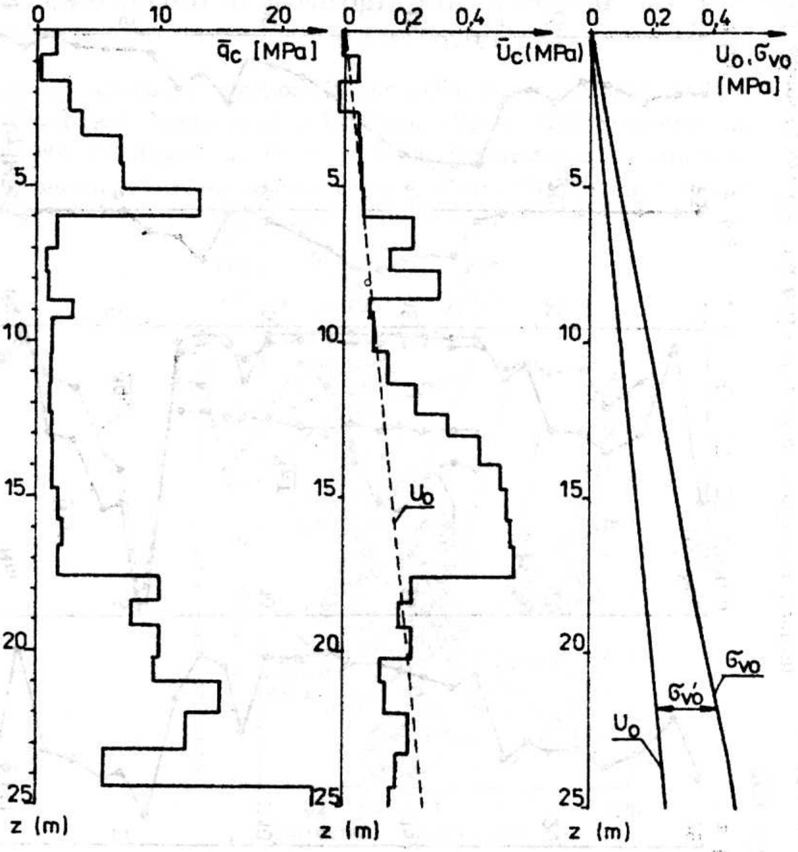

Fig. 2. The mean values of penetration parameters accepted for analysis

Evaluation of pore pressures in subsoil is essential for quantitative interpretation of a *CPTU* diagram. The distribution of pore pressure around the cone during penetration is complex, and the registered values of pore pressure depend on location of the filter on the cone. The measured pressure – U_c is most often corrected with the coefficient – k to the value of:

$$U_T = U_o + k(U_c - U_o) \tag{2}$$

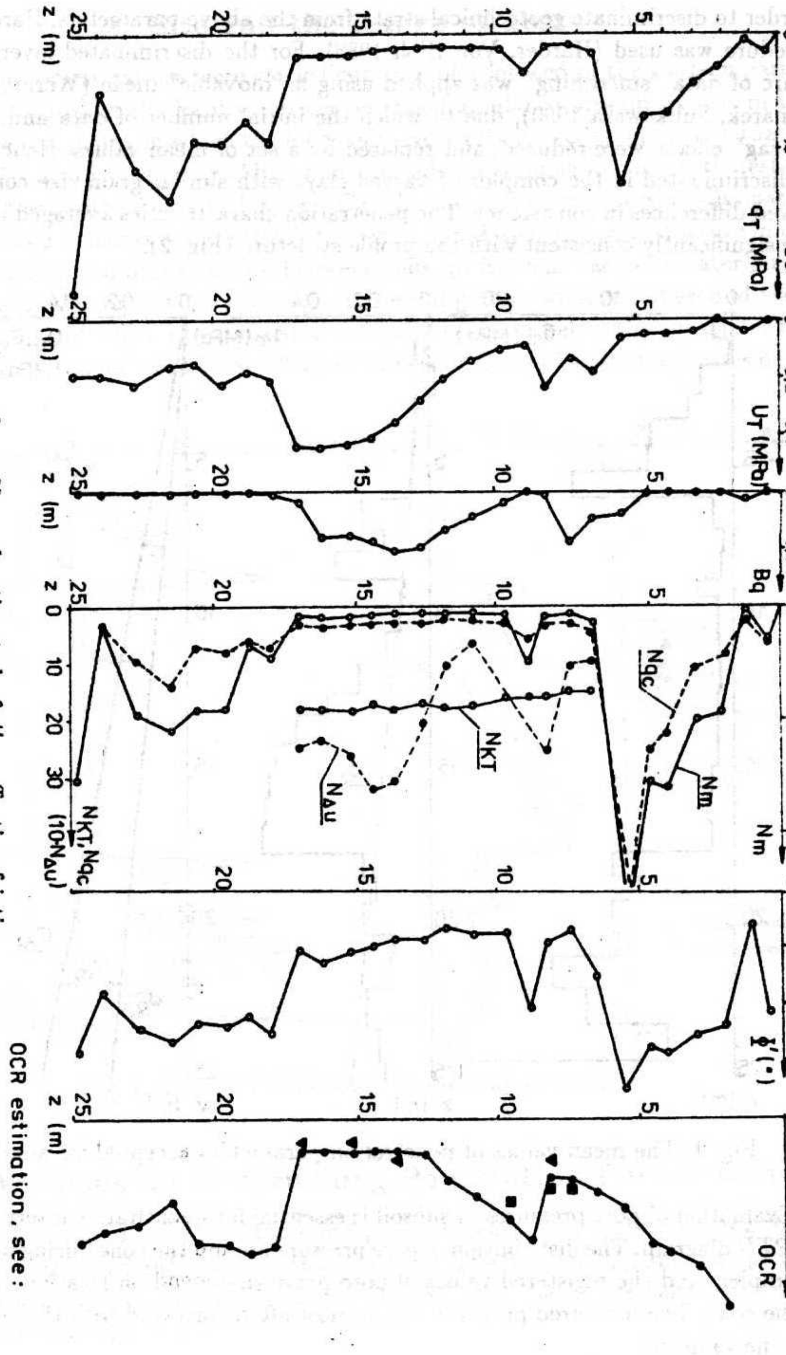

Fig. 3. Corrected values of q_T, U_T and estimated of ϕ' – effective friction angle and OCR – overconsolidation ratio

Since pore pressure can also act in the direction opposite to that of penetration, thus lowering the actual cone resistance, the cone resistance values are corrected with the coefficient – a' in the following way:

$$q_T = q_c + (1 - a)U_T \tag{3}$$

Figure 3 presents corrected values of cone resistance and pore pressure which were used for further interpretation of geotechnical parameters.

4. Evaluation of Stratographic Structure of Soil by the *CPTU* Method

Among numerous suggestions (Campanella, Robertson 1978; De Ruiter 1982; Douglas, Olsen 1981; Lunne et al. 1989; Olsen, Malone 1988; Schmertmann 1978; Tschuschke 1988; Tsuchiya et al. 1988) for evaluating stratographic structure of soil profile form penetration curves, the most useful method of facilitating, besides the evaluation

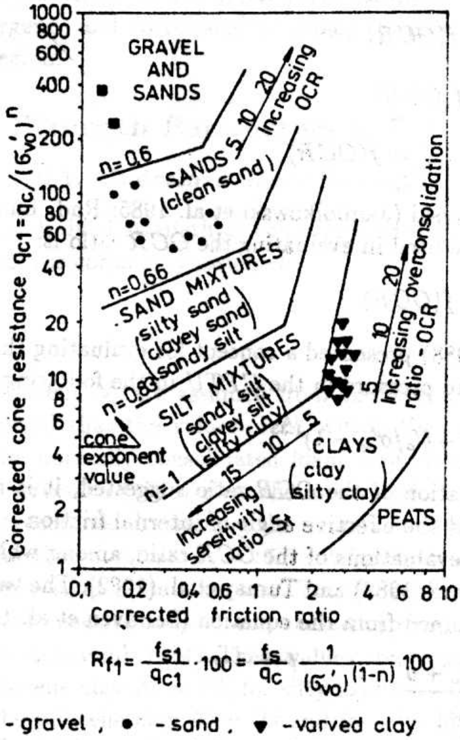

Fig. 4. Cone penetration test soil characterization chart (Olsen, Malone 1988)

of the soil type, the obtaining of general information on the state of the tested soil, is that of Olsen and Malone (Olsen, Malone 1988). It results from the Fig. 4 that the soil layers defined by a Olsen – Malone method show strong convergence with the assessment of grain size composition from the grain size analyses.

5. Stress History of the Subsoil

The stress history can be described by the maximum historical vertical stress – σ_c, or soil overconsolidation ratio OCR. To estimate these values, the penetration curve from the $CPTU$ can be used, particularly when the pore pressure measurement is carried out parallel to the measurement of cone resistance. Sully (Sully, Campanella, Robertson 1987; Sully, Campanella, Robertson 1988) suggested the following empirical relationship to estimate the OCR ratio from the $CPTU$ test:

$$OCR = 0.66 + 1.43(PPD) \tag{4}$$

where:

$$PPD = (U_1 - U_2)/U_o \tag{5}$$

Another possibility of assessing the OCR ratio is Wroth's method (Wroth 1988), which gives relationships between this index and three parameters of penetration curves:

$$(q_T - \sigma_{vo})/\sigma'_{vo} = f(OCR) \tag{6}$$

$$f_T/(q_T - \sigma_{vo}) = f(OCR) \tag{7}$$

$$(U_T - U_o)/(q_T - \sigma_{vo}) = f(OCR) \tag{8}$$

According to Jamiołkowski (Jamiołkowski et al. 1985; Rad, Lunne 1988) a fourth relationship which can be used in evaluating the OCR ratio is:

$$(U_T - U_o)/\sigma'_{vo} = f(OCR) \tag{9}$$

Mayne and Bachus (1988) presented a concept for evaluating the OCR ratio from the measured excess of pore pressure in the $CPTU$ in the following way:

$$OCR = 0.38(U_T - U_o/\sigma'_{vo} - 1)^{1.33} \tag{10}$$

However, in the evaluation of the OCR ratio suggested, it is necessary to know the rigidity index – I_r, and the effective angle of internal friction – Φ. The literature also presents other simple evaluations of the OCR ratio, among which the most useful are the concepts of Sugawara (1988) and Tumay et al. (1982). The value of preconsolidation stress – σ_c can be obtained from the equation (Sandven et al. 1988):

$$\sigma'_c + a = \frac{q_T - U_o + a}{N_{qc}} \tag{11}$$

In the above equation the values of the coefficient a are determined from the Senneset-Janbu's procedure (Senneset et al. 1982) depending on the state and kind of soil.

The OCR ratio of the layers of the analysed subsoil were determined by Sandven's method (Sandven et al. 1988). It results from the Fig. 3 that the complex of varved clays was characterized by only slight overconsolidation by the mean OCR value of 1.7. The increase of the OCR ratio in the top zone of the clays, is most probably due to the effect of the upper layers of water-glacial sands with different stress history. The sands constituting the soil profile in the deeper part of the subsoil also indicate a considerable increase in the OCR ratio with depth. Similar results of the evaluations of the OCR ratio were obtained from Rad and Lunne's suggestion (Rad, Lunne 1988). A test for the OCR values from the $CPTU$ can be the results of the other *in situ* test. Mayne and Mitchell (1988) indicated that the value of preconsolidation stress – σ_c can be estimated from the shear strength from the field vane test (FVT) corrected on the basis of the knowledge of the plasticity index as follows:

$$\sigma'_c = \alpha_{FV} \cdot S_{u(FV)} \tag{12}$$

where:

$$\alpha_{FV} = 22\ J_p^{-0.68} \tag{13}$$

Figure 3 proves that the values of the OCR ratio estimated by different methods indicate strong convergence and correspond to geological processes which took place in the Wielkopolska region.

6. Strength Parameters of Subsoil

6.1. Undrained Shear Strength

To determine the undrained strength of cohesive soils – S_u from cone resistance, the following relationship is commonly assumed:

$$S_u = \frac{q_c - \sigma_o}{N_c} \tag{14}$$

The values of the coefficient N_c are assumed from the vane test or from literature. De Ruiter (1982) reported that the values of N_c within the range of from 10 to 15 can be assumed for the normally consolidated loam, and 15 to 20 for the cohesive, overconsolidated. Schmertmann (1975) gives the interval of N_c changes from 5 to 70. Battaglio et al. (1986) recommend assuming the mean values of N_c depending on the kind of test; $N_c = 14$ for the vane test, $N_c = 17.5 \pm 5$ for the triaxial compression test, and $N_c = 20 \pm 10$ for the rigid plate slate test. The N_c coefficients were determined for the overconsolidated cohesive soils of the varved type by Młynarek (Młynarek, Sanglerat 1983; Młynarek, Kaszub 1984). These values are, respectively: from the vane test (FVT) for the pliocene clay $N_c = 20$, for silty loams $N_c = 24$, from the triaxial test ($TX - UU$) for the pliocene clay $N_c = 23$ and for silty loams $N_c = 40$. Lacasse and Lunne (1982) state that for soils with the rigidity index $I_r = 6 \div 400$, the value of the N_c index is 16 ± 2. Lunne et al. (1985) introduce into the description of the undrained shear strength from the $CPTU$ the corrected value of cone resistance – q_T, according to the formula

$$S_u = \frac{q_T - \sigma_{vo}}{N_{KT}} \tag{15}$$

According to Rad and Lunne (1988), the value of the coefficient N_{UT} changes within the limits of from 8 to 29 with respect to the test in the triaxial apparatus according to the $CAUC$ procedure. Aas et al. (1986) suggest that the values of the N_{UT} coefficient increase from 8 to 16 with the increase of plasticity index I_p from 3 to 50%. Another method for estimating the undrained shear strength from the $CPTU$ is by using the excess pore pressure – ΔU, instead of the cone resistance (Battaglio et al. 1981; Massarsch, Broms 1981; Campanella et al. 1985) then:

$$S_u = \frac{U_T - U_o}{N_{\Delta u}} = \frac{\Delta U}{N_{\Delta u}} \tag{16}$$

The values of the $N_{\Delta U}$ coefficient can vary from 2 to 20 (Campanella, Robertson 1988; Lunne, Christoffersen, Tjelta 1985). Other suggestions for evaluating the shear strength parameter are also known, e.g. (Konrad, Law 1987; La Rochelle et al. 1988; Senneset, Janbu, Svano 1982; Houlsby, Teh 1988).

In the complex of varved clays analysed the values of undrained shear strength were determined by the following methods: Aas (Aas et al. 1986) and Lunne, Christoffersen, Tjelta (1985), and were compared with the values obtained from the triaxial compression test and vane test (Fig. 5b). The mean values of the undrained shear strength

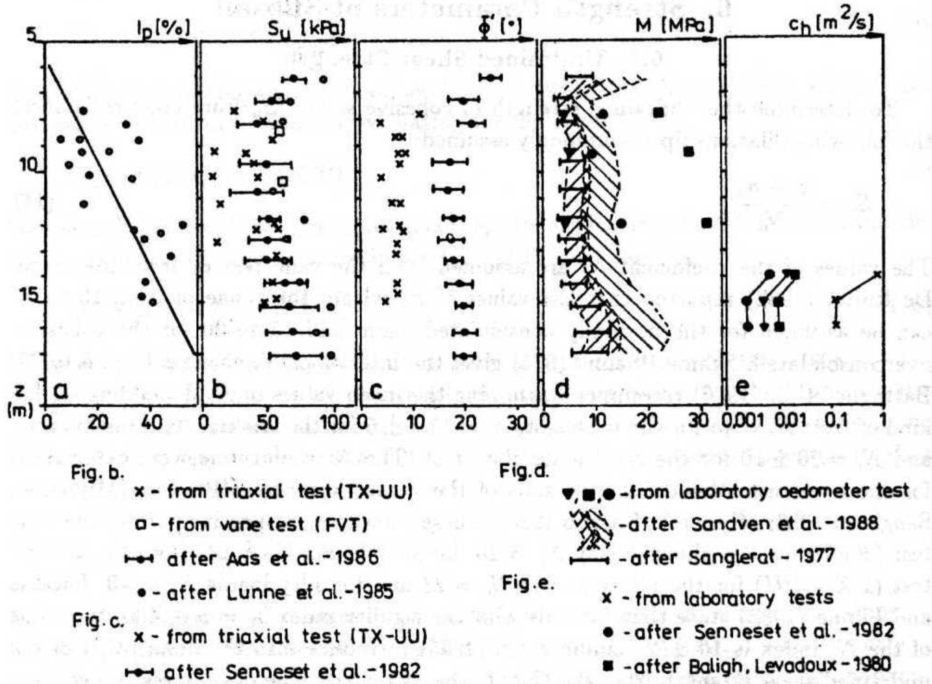

Fig. 5. Strength parameters of varved clay from $CPTU$ test

determined by Aas's method (Aas et al. 1986) changed from 50 to 80 kPa, showing a trend to increase with depth. Significant variability of shear strength at each depth is related to local weakening occurring in the varved clays. The great variability of the clay strength is also confirmed by the results of the triaxial test which are generally consistent with those obtained from interpretation of the penetration characteristics. The higher shear strength values determined from the measured pore pressures and calculated according to formula 16 are 20% greater than those determined with the Aas method (Aas et al. 1986). The shear strength can also be determined, as mentioned earlier, from the vane test, according to the relationship:

$$S_u = \mu \cdot S_{(FV)} \tag{17}$$

The values of shear strength calculated from equation 17 showed substantial conformity with the shear strength estimed from the *CPTU* method according to Lunne et al. (1985).

6.2. Undrained Shear Strength Expressed in Effective Stresses

The measurement of pore pressure during penetration facilitates the interpreting of shear parameters in effective stresses. To determine the relationship between cone resistance and shear parameters it is possible to use the Senneset-Janbu method (Senneset, Janbu, Svano 1982; Senneset, Janbu 1984). The equations are:

$$q_T - \sigma_{vo} = (N_q - 1)\,(\sigma'_{vo} + a) - N_U \cdot \Delta_{UT} \tag{18}$$

$$N_q = \tan^2\left(45^\circ + \frac{\Phi'}{2}\right) e^{(\pi - 2\beta)\tan\Phi'} \tag{19}$$

where:

$$N_U = 6\tan\Phi'(1 + \tan\Phi') \tag{20}$$

$$\Delta_{UT} = B_q(q_T - \sigma_{vo}) \tag{21}$$

$$a = \frac{c'}{\tan\Phi'} \tag{22}$$

The following parameters are obtained from equations 18 ÷ 22:

$$N_m = \frac{q_T - \sigma_{vo}}{\sigma'_{vo} + a} \tag{23}$$

$$B_q = \frac{U_T - U_o}{q_T - \sigma_{vo}} \tag{24}$$

The strength parameter analysis was limited to evaluation of the effective value of the angle of internal friction due to measurement uncertanities related to determination of the value a. It results from Fig. 3, that the calculated values of the angle of internal friction in the case of sandy soils varied from 30° to 45°, while for the complex of varved clays, from 17° to 22°. The mean values of the angle of internal friction in the clay complexes correspond well to the standard values expressed as effective stress (Fig. 5c).

7. Characteristics of Subsoil Deformations

From the value of cone resistance it is possible to estimate the following characteristics of deformation (Lunne et al. 1989): Young's modulus – E, constrained modulus – M, and shear modulus – G. To evaluate the shear modulus it is necessary to know the measurement made by, so-called, "seismic – piezocone". To determine the constrained modulus empirical relationships between Young's undrained modulus – E and undrained shear strength – S_u (Dayal 1982; Verbrugge 1982) or tangent constrained modulus M and cone resistane – q_c (Sandven, Senneset, Janbu 1988; Sanglerat, Janbu, Svano 1977; Senneset 1982; Senneset et al. 1988) are used:

$$E = n\ S_u \tag{25}$$

$$M = m\ q_c \tag{26}$$

According to Sanglerat (1977) the modulus M corresponds to the value of tangent modulus determined for the stress range $\sigma_{vo} + 100$ kPa. The relation between the tangent modulus and the effective stress – σ' is described by Sandven et al. (1988):

$$M = m\ \sigma_a \left(\frac{\sigma'}{\sigma_a}\right)^{1-a} \tag{27}$$

For cohesive soils the constrained modulus within the range of loads which are below the preconsolidation stress can be determined from the equation:

$$M_j = m_j\ q_n = m_j\ (q_T - \sigma_{vo}) \tag{28}$$

Within the range of normally consolidated stresses the modulus value can be estimated from the equation:

$$M_n = \frac{m}{N_m}\ q_n = m_n(q_T - \sigma_{vo}) \tag{29}$$

According to Sandven et al. (1988) the value of the coefficients m in equations 28 and 29 should be assumed in the ranges of: $m_i = 7 \div 13$, $m_n = 4 \div 8$. From the laboratory tests for the samples of varved clay, the tangent, primary and secondary constrained moduli were determined (Fig. 5d). The analysis of the results showed that the primary constrained modulus values for the stress range from 25 to 200 kPa were within the range of variability for the tangent oedometer moduli whose values were calculated from equation 28. According to the Sanglerat method, the values of the tangent constrained modulus, which are presented in Fig. 5d, were determined from the curve of primary constraint curve within the range from $\sigma_{vo} + 100$ kPa. The values of coefficient m which varied from 3.8 to 4.7, were determined from equation 26. According to Sanglerat (1977), for this kind of soils (Casagrande's classification – group CH), the value of the coefficient m should be assumed as being from 2 to 6, which indicates the great usefulness of this method for evaluating correctly defined modulus.

If during the $CPTU$ test the piezocone penetration is stopped, then it is possible to carry out the dissipation test of pore pressure. The rate of dissipation depends on the coefficient of permeability of the soil surrounding the cone. The cone pressure dissipation is determined mainly by vertical permeability (Sills, Almeida, Danziger 1988). To estimate the consolidation coefficient from the *CPTU* test several analytical solutions can be used basing mainly on the theory of "expansion of cavity" (Senneset, Janbu, Svano 1982; Torstensson 1977) and on application of the consolidation theory where the strain path method was used (Baligh, Levadoux 1980; Houlsby, Teh 1988; Kabir, Lutenegger 1990). The horizontal consolidation coefficient is determined from the equation:

$$C_h = R^2 \frac{T}{t} \tag{30}$$

The solution of Baligh and Levadoux (1980) was given for loams of the Boston Blue type, which are characterized by the rigidity index of $I_r = 500$. The Houlsby tests (Houlsby, Teh 1988) showed that with changing soil rigidity index from $50 \div 500$ different curves of pore pressure dissipation are obtained, giving as result a different evaluation of the time factor. Since the excess of pore pressure is mainly in the plastic deformation areas with the radius proportional to the $R\sqrt{I_r}$ value, Houlsby (Houlsby, Teh 1988) suggested taking the rigidity index into account in evaluating the consolidation index from the formula:

$$C_h = R^2 \sqrt{J_r} \frac{T^*}{t} \tag{31}$$

In the complex of varved clays three tests for pore pressure dissipation were made (Fig. 1) from which the values of horizontal consolidation coefficients were determined. For interpretation, the following methods were used: Baligh-Levadoux (1980), Senneset-Janbu-Svano (1982), and Houlsby-Teh (1988), while the results of the calculations are given in Figure 5e. The highest values of the index C_h, decreasing in the range of from 1.0×10^{-2} to 1.92×10^{-2} cm^2/s were obtained for the method which did not take into consideration the changes in the soil rigidity index, i.e. for the Baligh-Levadoux method. The lowest values were obtained from the Senneset-Janbu-Svano (1982) method, which varied in the profile from 4.35×10^{-3} to 8.78×10^{-3} cm^2/s. The Houlsby and Teh method which used the so-called modified time factor, through the soil rigidity index – I_r, in evaluation of the C_h coefficient, yielded indirect results which varied from 7.40×10^{-3} to 1.43×10^{-2} cm^2/s. The results obtained from theoretical solutions were also compared with those from laboratory tests. The analysis of test results showed that in the typical, strongly stratifield soil medium which constitutes varved clays drainage conditions play the essential role in evaluation of the consolidation coefficient. The sandy interbeddings decide as to the relatively high value of the coefficient of permeability, while this, in turn, plays the decisive role in evaluation of the consolidation index which is calculated from the solution of the differential equation of the consolidation theory by, e.g. Terzaghi. From laboratory tests the obtained values of the coefficient of consolidation were 10 times higher as compared with the Baligh-Levadoux method. The remaining methods yield evan greater discrepancies in evaluating the coefficient of consolidation.

8. Conclusions

The tests confirmed the concept expressed by many researchers that the static penetration test made by so-called piezocone is a universal test from which it is possible to identify the kind and state of the soil, to determine its strength parameters and deformation characteristics of genetically differentiated subsoil. The results facilitate confirmation of several detailed conclusions:

a) in a subsoil composed of soils considerably differentiated with respect to grain size distribution, e.g. clays and sands, for identification of substratum layers the Harder-Bloh method (Harder, Von Bloh 1988) can be used, assuming the friction ratio – R_f as a leading feature;

b) to estimate the value of preconsolidation stress – σ_c or the over-consolidation index – OCR in the varved clays from the $CPTU$ test it is possible to use the Sandven-Senneset-Janbu (1988) or Rad-Lunne (1988) methods;

c) among the methods which are used for prognosing undrained shear strength, in case of the varved clay, the Aas et al. (1986) and Lunne et al. (1985) methods showed great conformity with the laboratory test;

d) to evaluate the deformation characteristics from the $CPTU$ test it is possible to use simple correlative relations suggested by Sanglerat (1977) and Sandven-Senneset-Janbu (1988). Using these methods it should be remembered that there are assumptions which require definition of the kind of modulus and the stress range;

e) dissipation of pore pressures in the dissipation test in these soils is governed by very thin laminations of silty sands and silts due to which the mechanism of this phenomenon does not meet the assumptions of the analysed analytical methods (Baligh, Levadoux 1980; Houlsby, Teh 1988; Senneset et al. 1982);

f) modern solutions basing on connecting the theory of modified cavity expansion, the strain paths and the large strain finite element showed that the invarible significant factor of the evaluation of strength parameters and deformation characteristics of soils is, often neglected in empirical solutions, soil rigidity expressed by so-called rigidity index – I_r (Houlsby, Teh 1988; Lunne et al. 1989). So far, there are not any relations which would facilitate deformation of the value of the soil rigidity directly from the piezocone test. To determine this index it is necessary to carry out special *in situ* investigations or using a special cone, a so-called seismic – piezocone, in the $CPTU$ test.

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Summary

The paper presents evaluation of strength parameters of subsoil composed of water-glacial sands and varved clays by the static penetration method. The Harder-Bloh's method was used to interpret the penetration curve . The paper contains: evaluation of the undrained shear strength, the constrained modulus and the coefficient of clay consolidation. Fig. 3 also gives the method of evaluating the stress history of subsoil with the *OCR* ratio.

Streszczenie

Ocena parametrów wytrzmałościowych podłoża metodą *CPTU*

W artykule przedstawiono ocenę parametrów wytrzymałościowych podłoża zbudowanego z piasków wodno-lodowcowych i iłów warwowych metodą statycznego sondowania. Do interpretacji krzywej penetracji wykorzystano metodę Hardera-Bloha. Praca zawiera ocenę niedrenowanej wytrzymałości na ścinanie, modułów ściśliwości i współczynnika konsolidacji iłów. Na rys. 3 przedstawiono także ocenę historii obciążenia podłoża za pomocą wskaźnika *OCR*.