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EVALUATION OF SOIL PARAMETERS BY IN-SITU TESTS FOR MAPPING

Zbigniew MŁYNAREK

[a] A. Cieszkowski Agricultural University, Poznań, Poland
E-mail address: hebo@wp.pl

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Abstract

The paper presents and discusses several components of such procedures as programming and interpretation of in-situ tests and documentation of their results. These include the assessment of the effect of geological and investment processes on mechanical soil parameters of subsoil, criteria for the selection of in-situ testing techniques to solve the presented geotechnical problem, a synthetic conclusion concerning the determination of present-day concepts and a description of strength and deformation parameters of soils using CPTU, DMT, VST and SCPTU. The paper also briefly presents new in-situ techniques belonging to the full flow group, e.g. T-bar and ball penetrometer tests, as well as theoretical foundations for the determination of representative parameters for the isolation of geotechnically homogenous soil layers in the subsoil. Examples are given of the isolation of homogenous layers using the cluster method and the krieging method, treating the problem as a uniaxial (1-D) and flat (2-D) problem. Moreover, the author's 2-stage concept for clustering data as a quasi three-dimensional (3-D) problem. Data used to isolate layers included parameters from CPTU, normalized cone resistance and a coefficient of friction. The paper also discusses the effectiveness of the applied methods to obtain a 3-D model of subsoil structure, i.e. lithologic and strength models, and a model defining the diverse subsoil rigidity.

Streszczenie

Artykuł przedstawia i dyskutuje kilka problemów związanych z programowaniem i interpretacją badań in-situ, a także dokumentowaniem ich rezultatów. Analiza zawiera ocenę wpływu efektów geologicznych i inwestycyjnych na parametry mechaniczne gruntów, kryteria doboru metody badania in-situ dla rozwiązania problemu geotechnicznego, syntetyczną ocenę odnośnie współczesnej koncepcji opisu parametrów wytrzymałościowych i odkształceniowych gruntów. Artykuł przedstawia także skrótowo nowe techniki badania in-situ, na przykład: badania typu T-bar i „ball penetrometer". W pracy omówiono syntetycznie podstawy teoretyczne dla przygotowania reprezentatywnych parametrów gruntów, które są wykorzystywane do konstrukcji jednorodnych geotechnicznie warstw gruntów oraz podano przykłady wydzielania jednorodnych warstw gruntów w podłoża, traktując problem jako zadanie jednowymiarowe (1-D) i płaskie (2-D). Przedstawiona została także koncepcja: grupowania danych jako zadanie trójwymiarowe (3-D). Danymi do grupowania i wydzielania warstw były parametry z badania CPTU tj. znormalizowany opór stożka i współczynnik tarcia. W artykule przedyskutowano także efektywność zastosowanych metod dla określenia 3-D modelu podłoża tj. modelu litologicznego i wytrzymałościowego oraz modelu definiującego sztywność podłoża.

Keywords: **Interpretation of in-situ tests; CPTU, DMT, SDMT, T-bar, ball penetrometer tests; Methods of mapping.**

1. INTRODUCTION

An important factor in preparing an appropriate in-situ testing program is to meet the expectations of the engineers designing an investment project. These expectations most frequently fall within three basic categories:

- obtaining the subsoil structure profile with representative geotechnical parameters of soil layers and water regime conditions
- presentation of the concept for the foundation of the designed object and the performance of foundation works
- determination of cooperation of the designed structure with subsoil over the entire period of service life of the object.

Frequently, a significant discrepancy is found between the prediction of geotechnical parameters of subsoil and numerous theoretical aspects of soil mechanics, modeling and numerical simulation. At present, it is difficult to find the equilibrium condition for the scope of in-situ tests, supplemented with laboratory experiments, and the necessary analysis, which is covered by modeling and numerical simulation. Some of the criteria for this condition seem to be the category of the investment project, the method and volume of loads transferred onto the subsoil. In some geotechnical situations, such as e.g. constructing an earthen structure on subsoil with poor bearing capacity, these factors may prove insufficient for the determination of an appropriate proportion between the scope of in-situ testing and laboratory analyses for the purpose of the necessary theoretical analysis.

The huge progress in the design of devices used for in-situ testing and the creation of very good theoretical foundations for the interpretation of many tests, e.g. CPTU and DMT, have, on one hand, contributed in the definition of so-called reliable soil parameters, but on the other hand, they require from the designing engineer considerable knowledge manifested in the understanding and appropriate application of parameters, supplied by the geotechnical engineer, which describe the soil medium in terms of strength and deformation aspects. At present, the above mentioned concept of reliable soil parameters is of great importance [33]. This concept comprises several factors, determining the values of a given parameter in in-situ testing and laboratory analysis. These factors include:

- quality, in terms of statistical analysis of the performed in-situ test (the replication test)
- the effect of heterogeneity of the soil medium on the determination of a representative parameter for the isolation of so-called geotechnically homogenous layer in the subsoil
- an appropriate interpretation of the process, which describes applied in-situ test
- quality of a sample for laboratory analysis
- the effect of the unloading and overconsolidation processes of the sample in laboratory analysis on its properties.

Literature sources on in-situ testing written in the last twenty years indicate that research concentrated primarily on the issue of appropriate theoretical interpretation of in-situ tests and identification of factors affecting determined geotechnical parameters of soils using applied in-situ test. Clarification of the issues discussed in this point required extensive field tests, tests performed in calibration chambers, as well as the calibration of in-situ testing results, both using the above mentioned laboratory tests and other in-situ testing techniques [54], [32], [86], [49]. Moreover, in recent years, numerous studies have been performed on sample quality and the effect of the overconsolidation process on the shear strength parameters and constrained moduli deter-mined in laboratory tests, used in the calibration of CPTU and DMT [33], [54], [15], [81], [58].

In the context of achievements in the above mentioned field it may be stated that few studies have been conducted on the effect of the quality of in-situ testing, or heterogeneity of the medium by the performance of replication test, on measured parameters in the applied test [59], [2], [35], [72]. It may be inferred that these factors are being underestimated, thus it is of interest to determine whether the effect of these factors on measured parameters in in-situ testing is significant or rather non-significant. Statistical criteria are assumed to be the measure of significance [44].

2. CRITERIA FOR THE SELECTION OF IN-SITU TESTING METHOD SOLVING PRESENTED GEOTECHNICAL PROBLEM

When programming in-situ tests three aspects are investigated: safety of the construction, performance and economy. A typical phenomenon seems to be the fact that the investor is mainly interested in safety at the lowest costs incurred for geotechnical testing. This underlying discrepancy frequently leads to a considerable limitation of in-situ testing and laboratory analyses, and, as a consequence, to overestimation of settlements of shallow foundations or underestimation of bearing capacity of piles. Practice provides dozens of such examples. Another problem, as it was mentioned in point 1, is the level of knowledge on the part of design engineers, and frequently even geotechnical engineers, of the state-of-the-art methods of in-situ testing and present-day variation in mechanical parameters, describing properties of the soil medium. For this reason a short comment on in-situ testing techniques, their appropriate application and interpretation of measured values from these tests seems to be necessary. Mayne [55] grouped in-situ tests to assess the structure of subsoil and geotechnical parameters as follows:

- geotechnical testing for general mapping across the investigated site
- in-situ tests to assess vertical geostratigraphic and soil parameters
- drilling and sampling to obtain high quality and representative materials for laboratory analyses.

In order to carry out tests for the above purpose there is a large number of available tools, based both on conventional and state-of-the-art technology. A very good characteristic of in-situ tests is presented by a diagram given by Mayne [55], (Fig. 1). This diagram needs to be supplemented with dynamic probing tests (DPT), commonly used in Europe. A very characteristic element which needs to be taken into consideration when forecasting geotechnical parameters is the fact that individual groups of tests during penetration generate different stress and strain paths around the gauging probe tip.

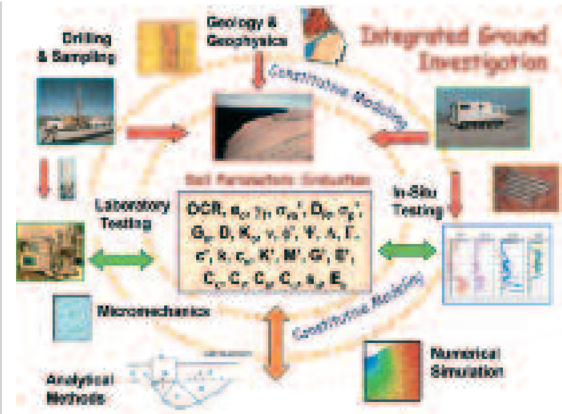

Figure 2.
General concept for integrated approach to foundation design [55]

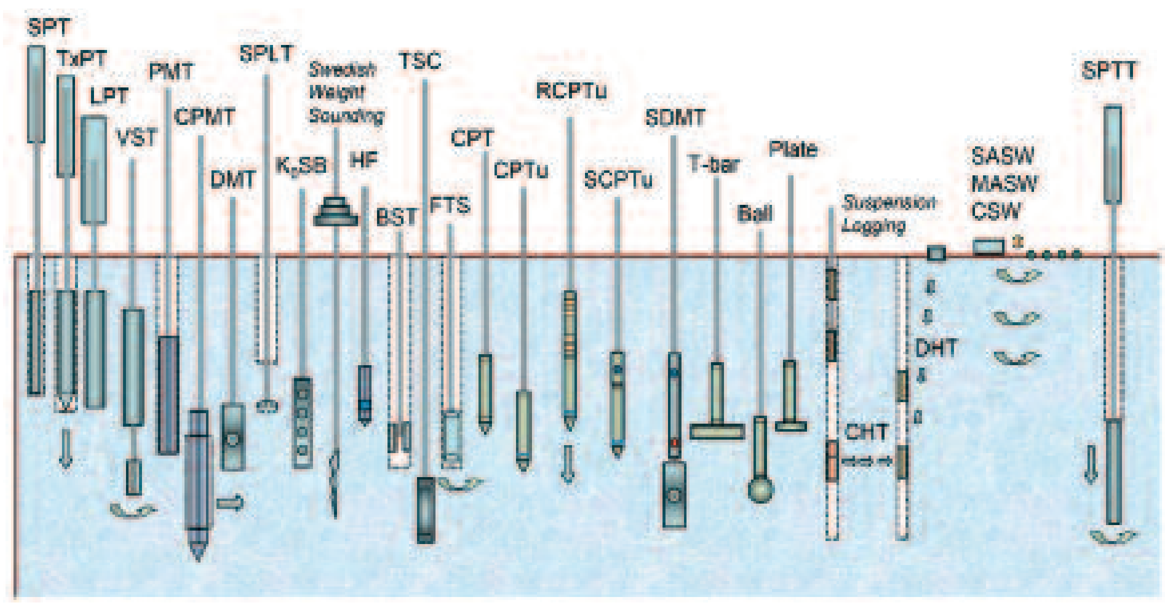

Figure 1.
Available in-situ geotechnical test for determination of soil parameters [55]

This results in a situation when different parameters describing soil strength and deformability of subsoil are obtained from individual tests. Thus, an essential element in programming an in-situ test for the solution of a specific geotechnical problem needs to be the selection of an appropriate method of in-situ testing and the selection of appropriate parameters, which are required and adequate for the solution of the foundation for a planned investment project. This task is perfectly illustrated by the diagram developed by Mayne [55], (Fig. 2).

Figure 2 effectively highlights the fact that geotechnical engineers, working on a task related to the foundation of an object and preparation of a program of in-situ tests, find themselves in a difficult position. Some general guidelines may be helpful when selecting tools and in understanding of the description of soil strength and deformation properties of subsoil using parameters measured by the applied test. These guide-lines may be as follows:

- Parameters obtained from basic in-situ tests are located in different positions on the non-linear dependence between modulus of shear G and shear strain (Fig. 3). Thus, the determination of an adequate modulus from several tests requires calibration, most frequently through an empirical dependence, and it is crucial to be aware of the limitation of empirical dependencies.

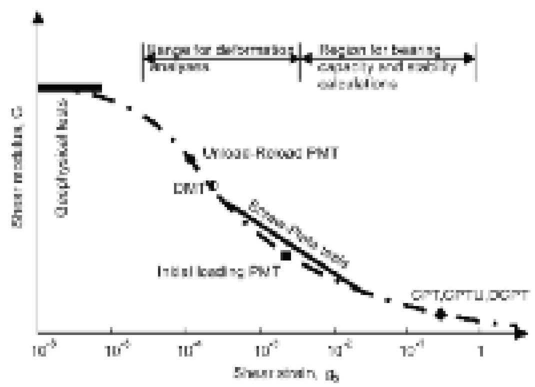

Figure 3.
Location of some in-situ tests on shear modulus σ-shear strain characteristics [55]

- Different methods of introduction of gauging probe tips into subsoil (Fig. 1) require a different theoretical interpretation of individual tests. This fact results in the generation of new parameters, describing properties of soils found in subsoil. These parameters are most often related to empirical dependencies described by standard parameters, defining shear strength and deformation characteristics of soils.
- A starting point for the selection of a testing method is the conventional division of soils in the subsoil into two categories: (I) clays and (II) sands. This division emphasizes different grain size distributions of these soils and drainage conditions. Between these soils there are so-called transition soils [37], for which drainage conditions and determination of measures describing their state are much more complex than for soils of categories I and II.

Traditional division of subsoil into two categories is very convenient for engineers, as it implies a relatively simple formula for the description of shear strength and selection of an appropriate in-situ testing technique. This formula defines shear strength as follows:

– for category I – clays, undrained shear strength (Su) is evaluated. Mayne [55] observed that "based on the framework of critical state soil mechanics (CSSM), all soils in fact are frictional materials and their strength envelope can be best represented by effective stress fiction angle Φ'

– for category II – sand, effective friction angle Φ' is assessed. Determination of Φ' is most frequently done using relative density (D_R), taking into consideration the effects of overconsolidation, the state of stress in subsoil, cementation, aging, grain mineralogy [31], [39] or void range potential ($e_{max} – e_{min}$) [11].

A review of possible applications for individual tools presented in Fig. 1 has been discussed in detail by numerous authors. This problem was presented in a comprehensive, tabular form by Lunne et al [56]. General guidelines for the selection of an appropriate in-situ testing method may be summed up as follows: the peak effective stress friction angle of sands (Φ'p) is used for dimensioning retaining walls, footing bearing capacity, and bearing resistance of deep foundation, as well as pale side friction, and slope stability analysis. For retaining walls and the construction of tunnels, apart from the standard strength parameters, the value of coefficient K_0 is also required. This parameter is best determined using several in-situ testing techniques, due to the basic problems in the in-situ estimation of its value. Recommended methods include CPTU, DMT and CPMT [50]. Preferred tool in the assessment of non-drainage shear strength of organic soils is VST. Such testing techniques used in these soils as CPTU or DMT are still waiting for extensive verification studies to be performed [47], [60].

3. CONCEPTS FOR THE INTERPRETATION OF SELECTED IN-SITU TESTS

The primary sources of information for the interpretation of individual in-situ testing methods are solutions from elasticity and plasticity theory. The classification diagram of models of stiff or elastic plastic soils is pre-sented in Fig. 4, [88]. Several in-situ tests are well-embedded in this classification, e.g. the cone penetration test, the dilatometer test, the pressuremetr test, the vane test, T-bar and ball, the plate test. Especially solution of the problem using the static penetration test has solid theoretical foundations. In individual groups of the diagram there are solutions for CPTU and DMT.

- Limit state theory (LST) is a solution described in the nomenclature adopted by Mayne [53], [54] as CSSM (critical – state soil mechanics) and is represented by studies [80], [18], [21], [30], [84], [94].

- The static theory boundary bearing capacity [93], [76] – solutions which have been widely applied in the interpretation of CPTU, DMT and CPMT.

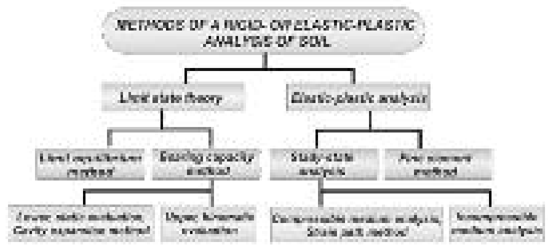

Figure 4.
Systematic of theoretical models describing in-situ tests

Following nomenclature proposed by Mayne [54], hybrid SCE – CSSM model was used to describe dilatometer test, although dilatometer geometry does not directly fulfill the cavity expansion theory. As far as the group of limiting state test (LST) solutions is concerned, within the kinematical approach, studies by [19], [67] need to be mentioned. In these studies a solution was presented for the driving of an unsunk wedge into a non-cohesive soil medium. The kinematical solution leads to the upper limit estimation of limiting load.

In the increment elastic plastic analysis of the cone penetration process, the basic problem is maintaining strict boundary conditions. This is caused by large (completed) plastic deformations and the non-linearity of the stress-deformation characteristic. This implies an assumption of a realistic co-ordinate system, e.g. the Lagranage material co-ordinates (unknown reference system) or Euler solid axes system. In spite of numerical problems, studies worth recommending here include those by Van den Berg [91], [92] who provided a solution in the Euler co-ordinate system, applying the elastic plastic model proposed by Drücker – Prager and Coulomb – Mohr.

In the stress path method two postulates are adopted: non-compressibility of soil and constant penetration velocity. The main element of the solution is calculation of half-speed and stress increments using the iterative method [9], [87].

In the solution, which includes the concept of strain paths, two basic methods are employed, i.e. so-called straight pale method [4] and finite difference method [87].

A highly effective analysis of the cone penetration process requires combination of theoretical and numerical methods [95]. Several solutions have been presented in this group of methods, which effectively determine shear strength parameters, i.e.:

- Strain path method combined with finite element method [87]
- Strain path method accounting for loading paths [23]
- Expansion of the cavity method combined with finite element method [94]
- Strain path method combined with expansion of cavity method [94], [95]
- Steady state analysis combined with finite element method [28], [29].

4. AN OVERVIEW OF PRESENT-DAY INTERPRETATIONS AND ASSESSMENT OF STRENGTH AND DEFORMATION SOIL PARAMETERS USING IN-SITU METHODS

Theoretical solutions and their verification contributed to the situation when the number of soil parameters describing shear strength and deformation of subsoil is at present much larger than the number geotechnical engineers and designers, unfamiliar with the recent advances in soil mechanics, are accustomed to. Following Mayne [54] conservative parameters describing strength and stiffness, may include effective stress friction angle Φ’ according to the simple Mohr – Coulomb strength envelope and an equivalent elastic modulus (E’). These parameters need to be supplemented with additional parameters, used in the solution of a geotechnical problem (Fig. 2). They include constrained modulus ($D' = 1/m_v$), dilatancy angle (Ψ’), Poisson’s ratio (γ’), damping ratio (D), modulus reduction curves (G/G_0), coefficient of secondary compression ($C_{£\epsilon}$), small – strain modulus ($G_0 = G_{max}$) threshold strain (γ_{tn}), lateral stress coefficient (K_0), overconsolidation ratio (OCR), constitutive soil modeling constant related to strength (M), compressibility index (λ, K), state parameter (Ψ_S) and hardening (h).

Sand strength can be attributed to termed critical state friction angle Φ'_{CR}, that is dependent on the mineralogy, particle shape size, roundness, and aging [31]. The measured peak sand strength Φ'_p can be considered as the sum of minimum value (Φ'_{CS}) and an additional component to volume increase or dilatancy effects [75], [6].

Strength and deformation parameters of in-situ tests are used to solve a wide variety of geotechnical problems. Apart from the detailed theoretical solutions mentioned above, parameters are also based on empirical relationships. There are two groups of these relationships, general and local. General empirical relationships treat the soil medium as material independent of the geographical location, and described by general criteria, e.g. mineralogy, variation of grain size distribution, etc. These relationships are most effective in relation to soil category I – sands. Local correlation relationships need to be considered more valuable for the solution of a given geotechnical problem, since they are strongly connected with the area where the geotechnical problem is investigated. Due to the extensive body of literature on the existing empirical relationships and dependen-cies obtained from theoretical solutions, below we present only these formulas which make it possible – using different in-situ techniques – to determine strength and deformation parameters of soils category I and II.

4.1 Measures for cohesive soils

In case of soils of category I, i.e. clays, measures found in empirical dependencies in the variable form include plasticity index I_p, liquidity index LI or moisture content w_n. Atterberg limits are used to calculate the plasticity index or liquidity index. Analysis of factors affecting these limits has been the subject of numerous studies [82], [83], [36]. It is generally assumed that precision of assessment of the liquid limit is described by mean precision coefficient CV = 6.0%, while that of yield point is CV = 8.7% [44]. Values of indexes I_p or LI are introduced to empirical dependencies in numerical form, thus it is important to realize that there are many factors affecting Atterberg limits. Moisture content of clays may rather be used only to local correlation relationships, where the mineralogical type of clays or carbonate contents in clays are known.

In urban areas we have to take into consideration an additional important factor affecting determined indexes I_p and LI, namely, subsoil contamination with various substances, which may cause significant changes in the chemical composition of groundwater. Bjerrum [5] suggested that groundwater salinity may affect numerical values of Attenberg limits in clays. This hypothesis was confirmed by the results of studies conducted by Młynarek, Tschuschke [63]. Those authors analyzed the effect of strongly salinated tailings water from a copper ore tailings dump, infiltrating the subsoil, on Attenberg limits, and as a result, on calculated values of indexes I_p and LI (Fig. 5). Investigations showed a time-significant effect [12] of salinated water on I_p and LI values of clays. As a consequence of these changes, a diverse assessment of the value of coefficient N_{kt} is obtained, which is used to determine non-drainage shear strength or constrained moduli of clays by CPTU [1], [73].

Changes in the liquidity index of clays in subsoil may be derived using several in-situ testing methods. The CPTU technique seems to be most suitable for this purpose. Based on an extensive body of data, Liszkowski et al. [45] determined a dependence between cone resistance and plasticity index for cohesive soils of different geological origin (Fig. 6). The indirect effect of overconsolidation ratio (OCR) and mineralogical composition of soil on these dependencies was determined in this way.

4.2 Measures for sands

Commonly applied measure of packing in sands is relative density (D_R). The applicability of this index

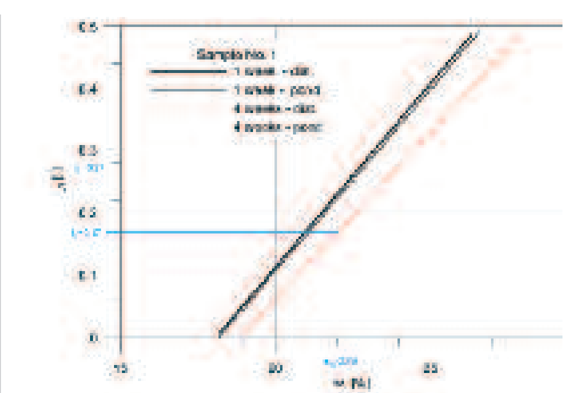

Figure 5.
Changes of liquidity index of clay with respect to changes of moisture content and different water used to estimation of Atterberg limits

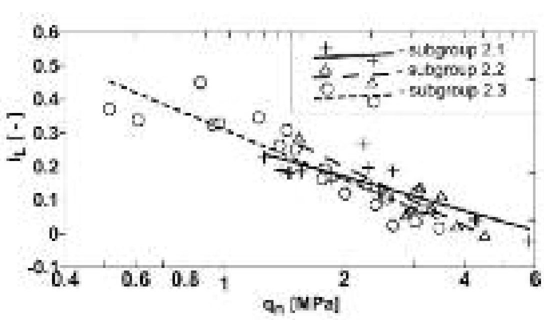

Figure 6.
Liquidity index vs. net cone resistance for genetically differentiated cohesive soils [45]:
subgroup 2.1 (solifluction tills) I_L=0,271-0,147ln(q_n),
subgroup 2.2 (last glaciation tills) I_L=0,310-0,216ln(q_n),
subgroup 2.3 (earlier glaciation tills) I_L=0,375-0,254ln(q_n)

has several limitations, affecting the precision of its assessment under in-situ conditions. These factors include [31], [32], [55], [10], [11] quality of the sample for analysis e_o, cementation, aging, overconsolidation effects, mineralogy and particle shape and size effects, as well as the methodology of determination of e_{max} and e_{min} [48]. Figure 7 presents comprehensively the effect of the applied testing method on assessment of e_{max} and e_{min}.

$$e_{max} - e_{min} = 0.23 + 0.06 / D_{50}\,(mm) \tag{1}$$

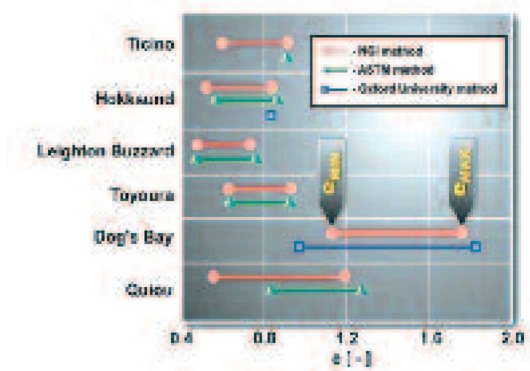

Figure 7.
Max. and min. porosity of 6 sands evaluated by means of different methods [48]

To assess relative density of sands several in-situ tests are applied. The SPT and DPT techniques are commonly used in many countries [57]. However, when applying these tests it must be noted that numerous factors affect measured penetration of the testing probe tip per blow, such as energy efficiency or rod length [16], [68].

Preferred methods to assess D_R include CPTU and DMT. The application of the dependence between cone resistance and D_R (CPTU) or the lateral stress index K_D (DMT) should be preceded by a commentary by Jamiołkowski [31] and Lunne [48]:

- Relative density and the consolidation stress tensor (i.e. the level of effective stress existing in the specimen or in situ) are the most important variables that influence q_c – cone resistance. This fact was confirmed by statistical analysis of calibration chamber (CC) testing results, [90], [80], [25]
- Correlation between q_c and D_s and σ'_{vo} holds only for NC sands. This correlation for OC sands should refer to the effective mean in-situ stress σ'_{mo}
- In case of siliceous sands, their grain shape and crushability play a secondary role in the evaluation of D_R based on q_c data. However, the use of correlations obtained from CC experiments leads to the underestimation of D_R of sands deposits containing more than 5 and up to 20% of fines [32]
- The effect of geological time on porosity, macrostructure and mechanical properties of coarse grained soil deposits and aging influences the correlation D_R vs. q_c
- The CC (chamber size) effect leads, within some boundary conditions, to underestimating or overestimating of the field qc, which may lead to a certain error in the in-situ assessment of D_R, based on dependence q_c vs. D_R determined in the calibration chamber. Details on the problem may be found in studies by [70], [56], [77], [3].

Taking into consideration the above commentary, dependencies proposed by Jamiolkowski et al. [31], Mayne [55] for the determination of D_R on the basis of cone resistance qc (CPTU) and coefficient K_D (DMT) are as follows:

CPT, CPTU

Adapted on the basis of calibration tests

- Schmertmann dependence (1978)

$$q_c = Co \cdot Pa \left(\frac{\sigma'_{vo}}{Pa}\right) \exp\,(C_2 \cdot D_R) \tag{2}$$

$$D_R = \frac{1}{C_2} \ln\left[\frac{q_c / Pa}{C_0(\sigma'_{vo} / Pa)^{C_1}}\right] \tag{3}$$

q_c = measured cone resistance in situ multiplied by CF (kPa, MPa), Pa – atmospheric pressure (kPa), D_R – relative density (as decimal), σ'_{vo} – effective geostatic stress (kPa),

$$CF = a\,[(D_R)^b]^m \tag{4}$$

a, b = empirical coefficient function of R_d,

m = +1 and -1 for BC-1 and BC-3, respectively (for BC available boundary condition, see Jamiolkowski, 2001)

Values of coefficients C_0, C_2, C_1 are available in tables presented in a study by Jamiolkowski et al. (2001)

- Equation proposed by Lancellotta [41]

$$D_R = A_0 + B_0 \cdot X \tag{5}$$

$$\text{where: } x = \ln\left[\frac{q_c}{(\sigma'_{vo})^{\alpha}}\right] \tag{6}$$

A_0, B_0, α – empirical correlation factors, [31]

Correlation relationships, which take into consideration the overconsolidation ratio of subsoil, the mineralogical grain type, the aging effect and grain compressibility, were given by Jamiolkowski [31] and Mayne [55].

CPT, CPTU

- NC siliceous sands

$$D_R = \frac{1}{C_2} \ln\left[\frac{q_c}{C_o(\sigma'_{vo})^{C_1}}\right] \tag{7}$$

$C_0 = 17.68$ $C_1 = 0.50$ $C_2 = 3.10$

- NC and OC siliceous sands

$$D_R = \frac{1}{C_2} \ln\left[\frac{q_c}{C_o(\sigma'_{vo})^{C_1}}\right] \tag{8}$$

$C_0 = 24.94$ $C_1 = 0.46$ $C_2 = 2.96$

- NC sands of different compressibility

$$D_R = -1.292 + 0.268 \ln (q_c \cdot \sigma'_{vo}{}^{0.5}) \tag{9}$$

Jamiolkowski et al [31] proposed the following procedure for determination of mean effective geostatic stress σ'_{mo}:

– in NC deposits the upper limit of K_0 can be taken as $1 - \sin \varphi'_{cv}$ (φ'_{cv} – the angle of shear resistance at critical state)

– in heavily OC sands (i.e. OCR = 15)

K_0 does not exceed 1.0.

Kulhavy, Mayne [39] proposed a dependence q_{t1} vs. D_R, which takes into consideration aging correction, time, sand compressibility and OCR:

$$D_R = 100\left[\frac{q_{t1}}{305 \cdot Q_A \cdot Q_c \cdot Q_{OCR}}\right]^{0.5} \tag{10}$$

$Q_A = 1.2 + 0.05 \log (t/100)$, t – time (years), Q_c = sand compressibility, $Q_c = 0.9$ for high compressibility, $Q_c = 1.0$ for medium compressibility, $Q_c = 1.1$ for low compressibility

$$Q_{OCR} = OCR^{0.2} \tag{11}$$

$$q_{t1} = q_t / (\sigma'_{vo} \cdot \sigma'_{atm})^{0.5} \tag{12}$$

On the basis of detailed investigations conducted at NGI using sands with different compressibility values, for which the percentage of fines (pass No 200 sieve) was less than 5%, Lunne [48] proposed the following correction of the original formula given by Baldi et al [3]: if $D_R < 40\%$, then determined D_R values need to be multiplied by 1.2, for $40\% < D_R < 60\%$ – multiplied by 1.4, while for $D_R > 60\%$ – by 1.5, respectively. This correction refers to sands with $C_C/1 + e_o > 0.05$.

DMT

$$K_D = C_o (\sigma'_{vo})^{C_1} Po - C_1 \exp (C_2 D_R) \tag{13}$$

$$K_D = A \exp (B\, D_R) \tag{14}$$

K_D = the lateral stress index from DMT

$$K_D = \frac{Po - u_o}{\sigma'_{vo}} \tag{15}$$

Po = lift-off pressure at DMT, u_o = pore pressure, prior to penetration or expansion, σ'_{vo} = vertical effective stress, prior to penetration or expansion, C_0, C_1, A, B = empirical coefficient, see table in [31].

The relationship between wedge resistance in DMT and relative density is as follows [51]:

$$D_R = -1.082 + 0.204 \ln (q_D / \sigma'_{vo})^{0.4} \tag{16}$$

q_D – wedge resistance

SPT

$$C_D' \frac{(N_1)_{60}}{D_R^2} = \frac{11.7}{(e_{max} - e_{min})^{1.7}} \tag{17}$$

where: N_1 – N value adjusted to σ'_{vo} of one atmosphere.

4.3 Recommendations for assessment of shear strength parameters and constrained moduli

4.3.1 Effective stress friction angle of sands

Numerical values of effective stress friction angle of sands (Φ'_p) are made up of two components: the basic friction component (Φ_p) related to mineralogy, particle shape, size and roundness [78], [31] and a dilatancy component related to packing arrangement, relative density, test mode and effective confining stress [75], [6]. In order to determine the values of peak effective friction angle of sands two solutions are applied: relationship between the relative density index and determined value of the peak effective friction angle from the triaxial compression test or empirical dependencies from several in-situ tests. The first group of solutions is represented by the general Bolton relationship [6]:

$$\Phi'_{TX} = \Phi'_{CS} + 3[D_R \cdot \{Q - \ln p_f' / \sigma_{atm}\} - R] \geq \Phi_{CS} \tag{18}$$

where: D_R – relative density (decimal value), Q – empirical term for soil mineralogy and compressibility, R – empirical parameter, p'_f – mean principal effective stress at failure.

A considerable body of literature focuses on values of coefficients. Bolton [6] proposed Φ'_{cs} = 33 deg. Q = 10, R = 1 for quartz sands. Kulhavy, Mayne [39] recommended an adoption of p'_f equal to the double value of effective overburden stress, while Jamiolkowski et al [31] diversified values of constants depending on the mineralogical type of sands: siliceous, quartz, calcareous and glauconitic type mineralogies.

Another method to assess Φ'_p includes empirical relationships, in which parameters measured during the performed in-situ test are found. In case of tests in which penetration per blow (SPT) is a measured parameter, the determined relationship is definitely purely empirical in character. For such tests as CPT and DMT determination of Φ'_p is obtained from theoretical solutions, numerical simulations or empirical trends. The relationship between peak friction angle and parameters from CPT, SPT and DMT, are given below. These formulas are ascribed to specific theoretical solutions and are defined by the assumptions for the applied theory (see p. 3).

- SPT, Schmertman [79]

$$\Phi_p \approx \arctan \left[\frac{N_{60}}{12.2 + (\sigma'_{vo} / \sigma_{atm})} \right]^{0.34} \tag{19}$$

For corrected N – values to an energy efficiency of 60% [26]

$$\Phi_p' = 20^{\circ} + \sqrt{15.4 + (N_1)_{60}} \tag{20}$$

- CPT

Using bearing capacity theories Robertson, Campanella [74] proposed a relationship Φ'_p vs q_t, in the following form:

$$\Phi_p' = \arctan [0.1 + 0.38 \cdot \log (q_t / \sigma'_{vo})] \tag{21}$$

Mayne [55] stated that if $(q_t/\sigma'_{vo}) > 60$, then dependence (21) appears to overestimate Φ'_p. Lunne et al. [50] showed great practical usefulness of this dependence.

Effective vertical stress is only one of the three principal stress directions, the variation of cone resistance is also connected with other components in the state of stress in subsoil, thus dependencies given by Jamiołkowski et al. [31] should be considered valuable. Figures 8, 9 and 10 present dependencies between q_c vs Φ' at different σ'_{mo} levels for the three basic mineralogical types of sands.

Values of Φ'_p may also be obtained from some theoretical solutions, presented in Fig. 4. Two of these solutions are extensively used in practice. One of these solutions, proposed by Vesic [93] and Senneset [85] is a special case of the limit state theory (LST). Solution proposed by Vesic [93] was the determination of the pile end – bearing resistance in terms of the soil friction angle Φ'_p and rigidity index (I_R), where $I_R = G / \varepsilon_{max}$. Rigidity index may be presented in the reduced form of $I_{RR} = I_R / (1 + I_R\, \varepsilon_{vol})$, where ε_{vol} = volumetric strain in the plastic zone. The Vesic solution adapted to CPTU consists of the normalized

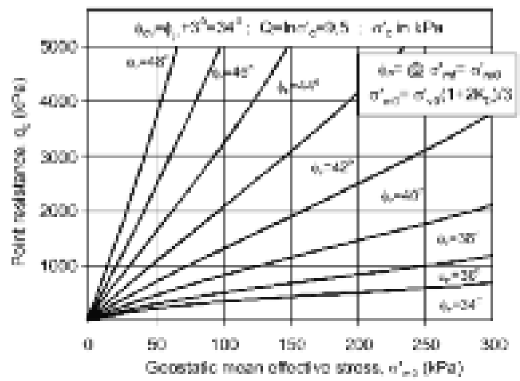

Figure 8.
Peak friction angle from CPT for silica sands using Bolton theory [31], [6]

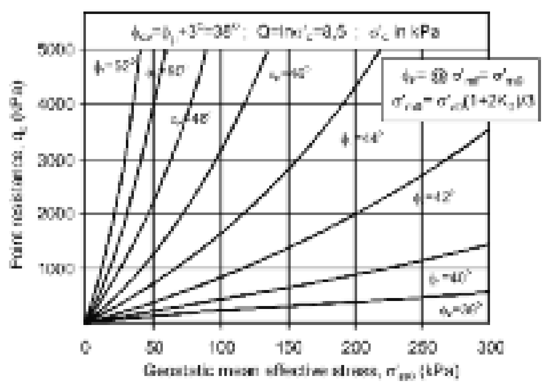

Figure 9.
Peak friction angle from CPT for calcareous sands using Bolton theory [31], [6]

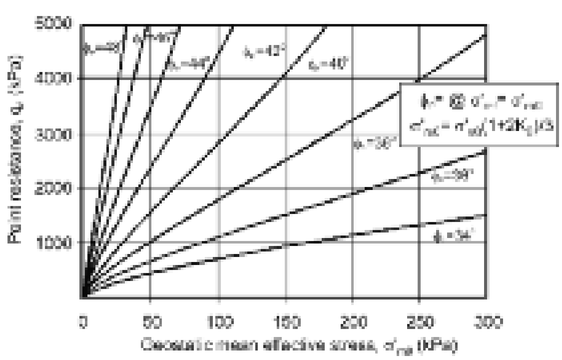

Figure 9.
Peak friction angle from CPT for quartz sands using Bolton theory [31], [6]

cone resistance $q_c/\sigma'_{vo} = Q$ and the notation of the function of both parameters, Φ'_p and I_{RR}, in the following form:

$$\Phi'_p \approx 1.38\,\frac{\log[0.8\cdot Q\cdot I_{RR}^{-0.23}]}{3.3+\log I_{RR}} \tag{22}$$

Parameter I_{RR} may be determined from the relationship proposed by Mayne [55] for sands, irrespective of their mineralogical composition.

$$I_{RR} = \left[\frac{G_o/\sigma'_m}{110}\right]^{2.46} \tag{23}$$

Values of modulus $G_o = G_{max} = \rho_t \cdot v_s^2$ may be easily determined in situ from cone or seismic dilatometer tests [55], [60].
An extensive commentary concerning the application of the solution for sands given by Senneset, Janbu [84] may be found in a study by Mayne [55]. The approximate dependence for the determination of the friction angle of sands, following the theory proposed by Senneset et al [85], is defined by the relationship

$$\Phi' = \mathrm{arctg}\left[\frac{\ln(0.94\cdot(q_t/\sigma'_m))}{4.87+0.035\cdot\beta}\right] \tag{24}$$

$$\text{where: } \beta = 222° - 37.6° \ln(G_{max}/\sigma'_{m0}) \tag{25}$$

In order to prepare an appropriate solution for the foundation of an object on sands, it is necessary to assess the stress-strain strength of sands. A solution to the problem is obtained through simple linear elastic – plastic forms, nonlinear algorithms and constitutive soil models [17], [40], [94]. A detailed and practical commentary concerning these solutions may be found in publications by Yu [94] and Mayne [55].

4.3.2 In-situ tests in clays

The primary component of the rational assessment of strength parameters and constrained moduli of cohesive soils is a known profile of stress history (i.e. OCR). Due to the complexity of processes affecting preconsolidation stress, determination of preconsolidation stress is not an easy task and requires much effort. It needs to be emphasized that, in urban areas, frequently deeper excavations and recreation of the state of stress in the laboratory (reconsolidation) may have a significant effect on the determined non-drainage shear strength [33]. Sample quality may also significantly affect the determination of σ'_p in laboratory analyses [22], and the characteristic of the void ratio vs log. effective stress requires corrections. Consolidation stress ($\sigma'_p = \sigma_{vmax} = p'_c$) is most often defined as the yield point on conventional one-dimensional plots of void ratio vs log effective stress – σ'_v, while OCR is written in the normalized form as σ'_p/σ'_{vo}. The overconsolidation ratio (OCR) is strongly related to strength and stiffness of clay. It needs to be stressed that in reality the vertical preconsolidation stress is merely a single point of an infinite locus of memory on the three–dimensional yield surface [43]. In the description of shear strength of clays there are parameters which are closely connected with the adopted theoretical solution. In turn, the solution should be adapted to the design of the foundation of a given object. In most simple foundation cases, concerning saturated soils, only three soil properties, i.e. Φ', C_C, C_S, are evaluated in addition to the initial state, described by e_o, σ'_{vo} and OCR. The diagram developed by Mayne [55] comprehensively presents concepts for descriptions of these parameters (Fig. 11). This diagram – in the form easily followed by all geotechnical engineers – presents concepts for the description of shear strength parameters of clays. Thus, the exact version of the commentary by Mayne [55] concerning this diagram may justifiably be quoted here:

„In its essence, the premise for CSSM is that all soil, regardless of starting point or drainage conditions, strives and eventually ends up on the critical state line (CSL). In the τ-σ_v' space, this line corresponds to the well-known strength envelope given by the Mohr-Coulomb criterion for c' = 0. Here, shear strength is represented by the maximum shear stress (τ_{max}) and is given by: $\tau = c' + \sigma' \tan\Phi'$. In the e-log σ_v' space, the CSL represents a line parallel to the virgin compression line (VCL with slope C_c) for NC soils, yet offset to the left at stresses approximately half those of the VCL. All NC soils start on the VCL, while all OC soils begin from a preconsolidated condition along the recompression or swelling line (given by slope C_S). Regardless, all shearing results in peak stresses falling on the CSL."

Figures 11 and 12 illustrate discrepancies in understanding of basic terms concerning strength of clayey soils (clays, silts, loams), which are frequently applied by many practitioners and diverge from the principles of critical state soil mechanics (CSSM). Most often, misunderstanding in this context concerns the term of cohesion [55]. This term refers to undrained shear strength ($c = c_u = s_u$) in some situations, while in others, it is used to mean effective cohesion intercept (c'). Figures 11 and 12 explain this problem, where s_u is obtained as the peak shear stress in a stress path of constant volume, while "c' " is obtained by force-fitting a straight line, to represent the Mohr – Coulomb strength criterion ($\tau = \sigma' \tan\Phi' + c'$) from laboratory data. It is generally known that the strength envelope is a more complex problem and is closely related to preconsolidation stress σ'_p [43], [41].

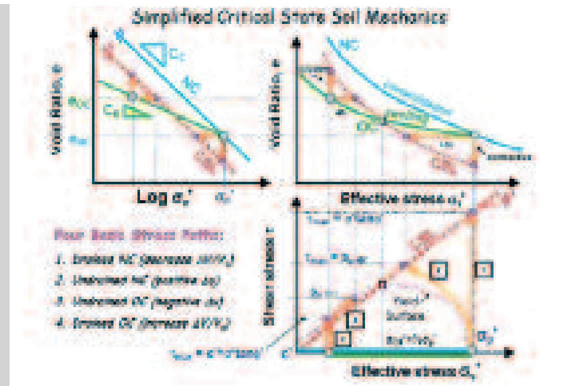

Figure 11.
Simplified critical state soil mechanics showing four common stress path [55]

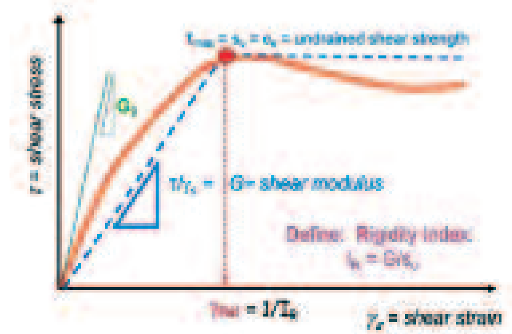

Figure 12.
Shear stress vs. shear strain for soils and definition of τ_{max}, G, γs and I_R [55]

Literature on the subject is truly extensive, thus only recommended relationships, applicable in the design process, are given below.

4.3.2.1 OCR from different in-situ tests CPTU

Most frequently, in order to assess OCR or construct the profile of overconsolidation difference (OCD) in subsoil based on CPTU, the hybrid SCE-CSSM model is applied. The suitability of the dependencies given below, which facilitate the determination of preconsolidation stress σ'p, is best summed up in studies [39], [50], [53], [55].

$$\sigma'_p = 0.33\,(q_t - \sigma_{vo}) \tag{26}$$

$$\sigma'_p = 0.47\,(u_1 - u_2) \tag{27}$$

$$\sigma'_p = 0.53\,(u_2 - u_0) \tag{28}$$

$$\sigma'_p = 0.60\,(q_t - u_2) \tag{29}$$

where: q_t – corrected cone resistance (see Lunne et al), u_1 – pore pressure measured on the cone, u_0 – in-situ pore pressure, u_2 – pore pressure measured behind the cone.

Problems in the assessment of OCR may occur in case of varved clays [14].

DMT

$$\sigma'_p \approx 0.51\,(p_0 - u_0) \qquad [55] \tag{30}$$

$OCR = 0.5\,K_D^{1.56}$ [51], [32], [7]

K_D – dilatometer horizontal stress index.

VST

$$\sigma'_p = \frac{22 \cdot s_{uv}}{\sqrt{PI}} \qquad [55] \tag{31}$$

where: s_{uv} –undrained shear strength from vane test, PI – plasticity index

SPT

$$\sigma'_p \approx 0.47\,N_{60} \cdot \sigma_{atm} \qquad [39] \tag{32}$$

Pressuremeter test (PMT)

$$\sigma'_p \approx 0.5\,s_{uPMT}\ln(I_R) \tag{33}$$

$I_R = G/s_{uPTM}$ – the operational value from the PMT.

T-bar test

$$\sigma'_p = 0.357\,q_{T\text{-}bar} \tag{34}$$

Seismic down hole test

$$\sigma'_p = 0.107\,v_s^{1.47} \tag{35}$$

Seismic SCPTU

$$\sigma'_p = (q_t - \sigma_{vo})^{0.302}\,(v_s/64)^{0.751} \tag{36}$$

4.3.2.2 Undrained shear strength

The undrained shear strength of clays is related to numerous factors, e.g. initial stress state, OCR, strain rate, direction of loading and macrostructure. The effect of initial stress state is included in the analysis by the in-troduction to the dependence of the normalized value s_u / σ_{vo}. Undrained shear strength is determined using constitutive laws based on the SSSM theory or, alternatively, using empirical approaches. Empirical dependencies derived from in-situ tests, performed using different techniques, require calibration [55], [62] . The assessment of s_u based on laboratory analyses is also problematic, as the realization of different stress and deformation paths may lead to different s_u values [13] Fig. 13 and 14.

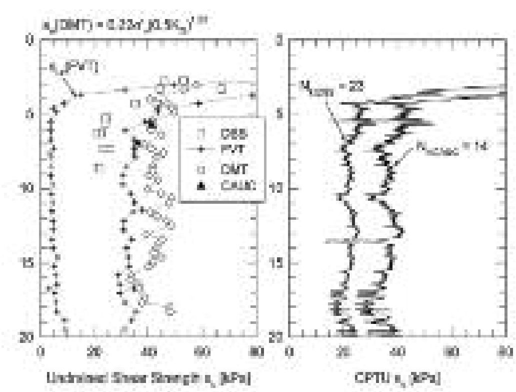

Figure 13.
Undrained shear strength Su of varved clay based on different laboratory and in-situ tests [13]

Referring s_u to direct simple shear (DSS) has many advantages, despite the fact that experience properly meets conditions for stability and bearing capacity analysis. Another problem is inconsistency observed between the interpretations of individual in-situ tests.

Values of s_u from VST are corrected by coefficient μ, which for clays is dependent on liquidity index and OCR [5]. For peats, values of coefficient μ depend on the botanical composition of peats and their degree of decay. The range of changes in the values of this coefficient for peats from the Wielkopolska region in Poland is 0.35 – 0.55 [61]. Many limitations are also found in case of VST [45]. The distribution of shear stress varies and strain paths cause formation of compressed and sheared areas.

Local experience is required in case of coefficient μ. Larsson et al. [42] proposed a formula to determine correction coefficient for clays

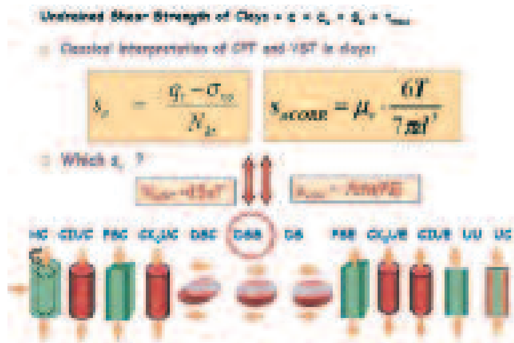

Figure 14.
Dilemma in matching laboratory benchmark mode for undrained shear strength (s_u) with in-situ CPT and VST data [55]

$$\mu = \left(\frac{0.43}{W_L}\right)^{0.45} \quad (37)$$

where: W_L – liquid limit

Dependencies recommended for the determination of undrained shear strength are given below:

Simple direct shear (DSS) [55]

$$\left(\frac{s_u}{\sigma'_{vo}}\right)_{DSS} = 0.22\ OCR^{0.80} \quad (38)$$

$$s_{u(DSS)} = 0.2\ \sigma'p \quad (39)$$

for lightly – overconsolidated clays with OCR < 2

Other dependencies, also for the triaxial compression test, may be found in a study by Mayne (2006).

CPTU

Powell, Lunne [72] and Lunne et al. [50] gave dependencies for the determination of undrained shear strength, verified and applied in site characterization for geotechnical designs.

- s_u evaluation using total cone resistance

$$s_u = \frac{(q_t - \sigma_{vo})}{N_{kt}} \quad (40)$$

N_{kt} – may vary from 10 to 20. It is advisable to verify this coefficient with available archive records for the analyzed area. The diagram presented in Fig. 15 may be useful in the assessment of N_{kt}.

- s_u evaluation using effective cone resistance

$$s_u = \frac{q_e}{N_{ke}} = \frac{(q_t - u_2)}{N_{ke}} \quad (41)$$

N_{kt} = 9 ± 3 and appears to be correlated with the pore pressure parameter Bq [85]

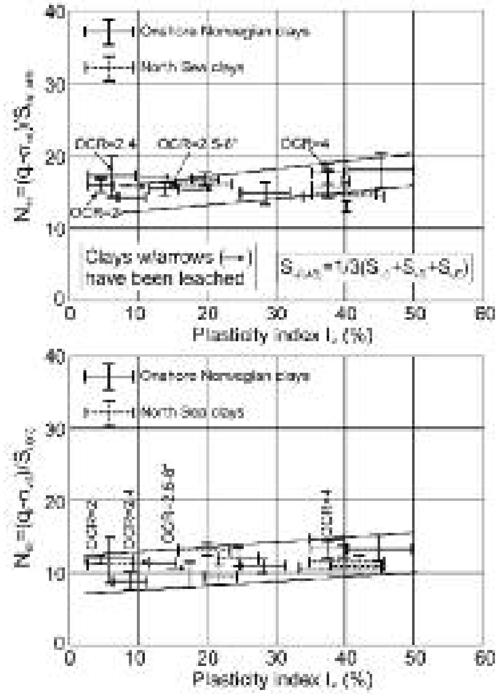

Figure 15.
Computed cone factor N_{kt} vs. I_p [50]

- s_u evaluation based on excess pore pressure

$$s_u = \frac{\Delta u}{N_{\Delta u}} \qquad \Delta u = u_2 - u_0 \quad (42)$$

Coefficient $N_{\Delta u}$ changes within a wide range from 4 to 10 [50]. For documentation purposes Powell, Lunne [72] recommended the following procedure:

– For deposits, where little experience is available, estimate s_u using the total cone resistance (q_t) and preliminary cone factor values (N_{kt}) from 15 to 20. For more conservative estimate, select a value close to the upper limit. For normally and slightly overconsolidated clays, N_{kt} can be as low as 10, and in stiff fissured clay it can be as high as 30. In very soft clays, where there may be some uncertainty with regards to the accuracy of q_t, estimate su from the excess pore pressure (Δu_2) measured behind the cone, using   from 7 to 10. The approach using N_{ke} can also be used in soft clays.

– For larger projects, where high quality field and laboratory data may be available, site-specific cor-rela-tions should be developed based on appropri-ate and reliable values of s_u.

DMT

DeGroot, Lutenegger [13], [15], on the basis of extensive in-situ tests as well as a simple direct shear test and an anisotropically consolidated undrained triaxial compression test (CAUC), all carried out on block samples, showed suitability of the original dependence proposed by Marchetti for the assessment of s_u:

$$s_{u(DMT)} = 0.22\,\sigma'_v\,(0.5\,K_D)^{1.25} \tag{43}$$

Roque et al. [after 55] proposed use of the bearing capacity formula to estimate undrained strength

$$s_u = \frac{P_1 - \sigma_{ho}}{N_C} \tag{44}$$

where: P_1 – corrected β – pressure, N_C for brittle clay and silt 5, medium clay 7, non-sensitive plastic clay 9.

T-bar ball penetrometer tests

Lunne [50] recommended the following dependencies for determination of undrained shear strength. They correspond to s_u^{CAUE} (anisotropically consolidated undrained triaxial test) or $s_{u,av} = 1/3\ (s_u^{CAUE} + s_u^{CAUE} + s_u^{DSS})$

$$s_u = q_{T\text{-}bar} / N_{T\text{-}bar} \tag{45}$$

$$s_u = q_{ball} / N_{ball} \tag{46}$$

Values of coefficients $N_{T\text{-}bar}$ and N_{ball} are presented in table 1. Lunne [48] reported that the coefficients were based on tests on clays, with values of soil plasticity ranging from 33 to 45% and OCR from 1.3 to 1.8.

Table 12.
Recommended CPTU and cone factors [48]

In-situ test	Empirical factor	Undrained shear strength, kPa	Recommended range
CPTU	N_{kt}	$s_{u,CAUC}$	9-13
		$s_{u,av}$	12-17
	$N_{\Delta u}$	$s_{u,CAUC}$	6-9
		$s_{u,av}$	7-12.5
T-bar	$N_{T\text{-}bar}$	$s_{u,CAUC}$	8-11
		$s_{u,av}$	10-13

In urban areas, engineering designs of many investment projects require information on in-situ horizontal stress σ'_h or the coefficient of lateral stress K_0.

Lunne et al. [50] proposed 3 methods to determine σ'_h or K_0 from CPTU:

1. Based on geological evidence

$$\frac{K_{o(OC)}}{K_{o(NC)}} = OCR^m \tag{47}$$

where: $K_{o(NC)} = 1 - \sin\Phi'$, m – 0.45, m – 0.65 [56]

2. Based on relative density

$$\frac{q_c}{p_c} = A\left(\frac{\sigma'_{ho}}{p_c}\right)^{0.6} \tag{48}$$

p_c – reference stress 100 kPa, A – constant depending on D_R, [56]

$$\frac{\sigma'_{ho}}{p_a} = \frac{(q_c / P_a)^{1.25}}{35\exp(D_R / 20)} \tag{49}$$

Numerical values of D_R may be determined from the dependencies given above.

For documentation purposes Mayne [55] proposed a dependence

$$K_o = 0.35\ OCR^{0.65} \tag{50}$$

Moreover, the dependence given by DeGroot, Lutenegger [13] should also be considered suitable in this respect. It was constructed on the basis of studies with the use of an instrumented oedometer ring with strain gauges for measurement of lateral stress during consolidation

$$K_o = 0.60\ OCR^{0.91} \tag{51}$$

In urban areas the processes of loading – unloading of subsoil as a result of deep excavations cause preconsolidation effects, when the lateral stress coefficient (K_o) reaches the passive value (K_p), then the K_0 value increases with OCR. If the passive condition is described by general condition, then K_0 cannot exceed [55]:

$$K_p = \frac{1+\sin\varphi'}{1-\sin\varphi'} + \frac{2c'}{\sigma'_{vo}}\sqrt{\frac{1+\sin\phi'}{1-\sin\phi'}} \tag{52}$$

c' = 0.02 σ'_p may be applicable for this relationship.

Parameters required for the geotechnical design include assessed deformation characteristics. In urban areas the processes of mechanical loading – unloading affect also the value of the constrained modulus – undrained Young's modulus or small strain shear modulus. Results of numerous backward analyses, performed during or after the construction of an object, have shown that values of moduli determined from tests and defined on the basis of field observations may differ dramatically.

Recommended methods for the assessment of deformation characteristics include CPTU, SCPTU and SDMT. Especially SCPTU and SDMT are universal in the assessment of deformation characteristics [52], while high consistency in the assessment of modulus Go with the use of both methods is stressed in a study by Młynarek et al. [60]. It is very important to notice that there are many limitations for the application of empir-ical relationships for site characterization. There limitation can be grouped in the following way:

- limitations resulting from dimensional analysis
- limitations resulting from statistical replication test
- limitations resulting from quality of samples for laboratory tests

A detailed information concerning these; limitation and assessment of the quality of empirical relationships may be found in the paper by Młynarek [64].

5. GEOTECHNICAL DOCUMENTATION (GEOTECHNICAL MAPPING)

5.1 Concepts for the documentation of subsoil structure

Geotechnical parameters determined using different in-situ techniques are a collection of oriented points, most frequently using the Cartesian coordinates, in the 3-D half-space of subsoil. A standard approach in clustering of these points is the isolation of so-called geotechnically homogenous subsoil layers. It is generally known that clustering criteria vary considerably. It is believed that the simplest one is formation of layers on the basis of changes in grain size distribution of soils, taking into consideration morphological aspect (geological regime) [71]. For a design engineer the most valuable is the criterion which supplements the geological regime with properties of points defining strength and deformability of soils in subsoil. Generally, geotechnical documentation, which actually is equivalent to the clustering of geotechnical data, may be divided into two stages: general mapping and local mapping.

5.2 General mapping

In urban areas, general mapping consists of several concepts of map generation. The first are so-called geological maps, which document the structure of subsoil in terms of deposit formation. These maps most often concern the whole area of a given country. Material for the generation of these maps includes most frequently data from geological drillings supported by extensive information on geology. The aim of the second concept for the documentation is formation, within the conurbation, of so-called geological engineering maps. Data for these maps come from the results of various in-situ tests, although, most frequently, from borings performed to determine the structure of subsoil for the planned investment project. Quality of these data varies considerably, it is especially doubtful if this type of maps contains information on strength parameters of soils. In the formation of the first and second type of maps no method of statistical data clustering was used. The third concept comprises maps documenting hydrological conditions. The last two categories of maps are maps documenting subsoil contamination and the underground infrastructure in subsoil. Maps documenting the spatial range of subsoil contamination at present are of paramount importance if a realization of a construction investment project is planned in the area, since legal and other issues are likely to arise. For the identification of subsoil contamination many sophisticated in-situ tests are recommended [69], [74].

5.3 Local mapping

The main aim of local mapping is to prepare the profile of subsoil structure for the planned investment project. This type of documentation is generally known and, in the most part, consists in the isolation in subsoil of the above mentioned homogenous soil layers and the preparation of the geotechnical profile. Preparation of these profiles may include several criteria, e.g. variation of soils found in subsoil, variation of strength or deformation parameters of soil layers and the history of subsoil formation. This type of documentation is frequently prepared in a relatively primitive way, using solely data from different in-situ testing techniques, and no data from laboratory analyses, while the construction of homogenous layers consists in estimated determination of the course of these layers differing in terms of the above mentioned criteria. It is assumed that soil parameters for subsoil, obtained from in-situ tests and laboratory analyses, are random variables, and, consequently, the isolation of homogenous layers should consist in clustering of observations, similar or non-similar, according to statistical criteria. Quality of clustering and, as a consequence, the number of isolated layers of soils, will depend on several factors, e.g. the number of observations and the applied clustering method. As it was explained in point 6, the quality of the parameter selected for clustering is another significant factor.

A special case of map is a risk map. These maps enable to evaluate the risk of planned investment in the view of costs, selection of the type of the foundation, "social opinion" etc.

Strategy of data clustering is closely related to the applied in-situ testing technique. Specially advantageous method in this respect is CPTU, supplemented with SCPTU or SDMT. A very large body of data is obtained from CPTU, and SDMT supplements the point score of assessment of subsoil parameters and may also constitute an element of calibration for both tests, thus increasing the reliability of observations introduced to clustering. This strategy should be based on the general principle that clustering needs to be initiated from the uniaxial (1-D) model through a flat (2-D) model, finally going to the three-dimensional (3-D) model. The 3-D model gives a three-dimensional picture of changes in subsoil properties and is the most expected model when designing the foundation for a given object.

5.3.1 1-D Subsoil model

The starting point for the isolation of homogenous soil layers is one of several parameters measured during the in-situ test. If testing is performed using a static probe (CPTU), then the most advantageous solution for clustering is to record 3 standard characteristics, i.e. changes in depth q_c (q_t), R_f, u_2. For a uniaxial clustering the following methods are applied:

- Harder-Bloh procedure [24] or a procedure modified with the sequence test [89],
- Clustering theory method [27], [20], [65].

The primary concept in the methods mentioned above is to cluster CPTU parameters by the investigation of statistical significance of differences between cosines of regression lines, so-called accumulation curves. Accumulation curves may be constructed for values of cone resistance q_c or q_t, depending on the intended classification system for identification of soils found on the penetration path. An example of application of the modified Harder-Bloh procedure to identify variation in grain size composition of post-floatation deposits embedded in the embankments of the Żelazny Most dump is presented in Figs. 16, 17, and 18. Obtained representative parameters from CPTU (Fig. 18) may be used to construct the 2-D subsoil model of the strength or deformation type.

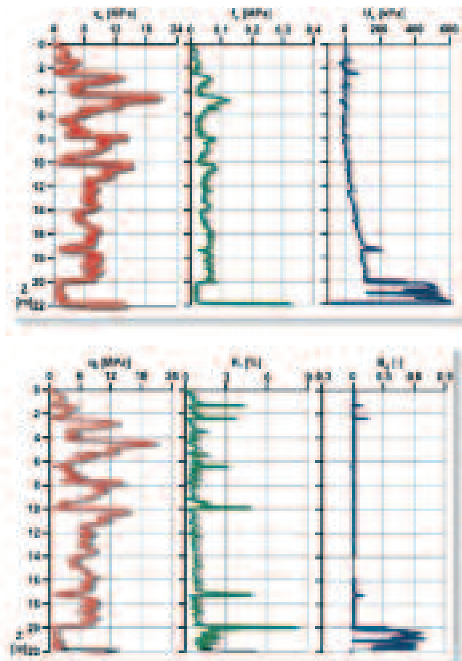

Figure 16.
CPTU characteristics in mine tailings (Żelazny Most Reservoir, Poland)

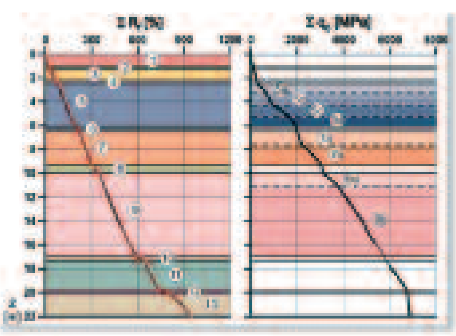

Figure 17.
Cumulation curves for CPTU characteristics in mine tailings (Żelazny Most Reservoir, Poland)

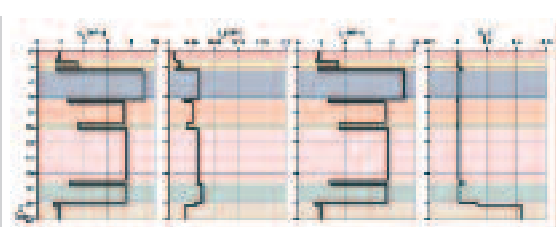

Figure 18.
Representative parameters of CPTU test for selected layers of mine tailings (Żelazny Most Reservoir, Poland)

5.3.2 2-D Interpolation model

A characteristic feature of 2-D model is the fact that it is constructed at levels of geotechnical stresses $\sigma'vo$, selected in 1-D model, separating homogenous layers according to the criteria applicable for the construction of 1-D model. 2-D model, as it was mentioned earlier, is a starting point for the description of 3-D structure of the subsoil. Thus, the construction of 2-D model is highly flexible, as this model may document changes in the type of soils found in subsoil if 1-D model was constructed, based on parameters q_c, q_t, R_f. Strength model of subsoil or a model defining subsoil rigidity may be constructed using representative parameters obtained from CPTU from 1-D model, and next, relationships given in 4.3.

5.3.2.1 Measures and methods to construct 2-D models

Similarity measure, or rather dissimilarity measure, constitutes primary elements in cluster analysis. Dissimilarity measure of objects (e.g. measured parameters of the in-situ test) refers to the function

$$p : X \times X \rightarrow \mathbb{R} \tag{53}$$

if: 1. $\rho(x_r, x_s) \geq 0$,

2. $\rho(x_r, x_s) = 0$ if and only if $x_r = x_s$,

3. $\rho(x_r, x_s) = \rho(x_s, x_r)$,

where x_r and x_s are ρ-dimensional vectors of observations of the r-th and s-th object (r, s = 1,2,......,n). A detailed discussion of different dissimilarity measures may be found in numerous studies, e.g. Kaufman, Rousseeuw [34]. Among many dissimilarity measures we need to mention two, which are most frequently applied in hierarchical clustering methods: the Euclidean distance

$$\rho_1(x_r, x_s) = ((x_r - x_s)^T(x_r - x_s))^{1/2} = \left(\sum_{i=1}^{p}(x_{ri} - x_{si})^2\right)^{1/2} \tag{54}$$

and the Mahalanobis distance, which takes into consideration correlations between characteristics

$$\rho_2(x_r, x_s) = ((x_r - x_s)^T S^{-1}(x_r - x_s))^{1/2} \tag{55}$$

where ρ is an estimator of the covariance matrix. General principle of cluster analysis, which should also be applied in the isolation of geotechnically homogenous layers in subsoil, is the rule that objects belonging to a given cluster should be "similar" to one another, while objects belonging to different clusters should be possibly markedly "dissimilar". The simplest algorithm for clustering of subsoil properties according to the method of cluster analysis is to adopt number of clusters and an optimal division of objects. Such a concept means that in the isolation of subsoil layers the clustering strategy needs to be performed in stages, starting from 1-D model, as it was already explained in point 7.3. A significant problem is connected with the selection of the final number of clusters. This problem may be solved by direct intervention of a geotechnical engineer, who considers obtained division of subsoil to be sufficiently effective for the solution of the foundation design for a given construction object. Another method to determine the final number of clusters may be through statistical formulas. Two such formulas are very effective in the isolation of homogenous subsoil layers, i.e. so-called Caliński-Harabasz [8] index and Hastie, Tibshirani, Walther index [38].

Dissimilarity measures are used in three basic groups of methods: hierarchical cluster analysis and nonhierarchical cluster analysis. The first group includes single linkage (nearest neighbour), complete linkage (furthest neighbour), average linkage, median method and Ward's method. Representatives of the second group of methods are the algorithm of K – means and the krieging method. The third group is composed of algorithms, in which the primary task is the assumption concerning probabilistic distribution of observed variables (e.g. the EM algorithm). Application of the above mentioned methods as a 2-D problem may be very effective in solving significant problems in everyday geotechnical practice. Figures 19, 20 and 21 present an interesting example of application of the cluster method in ascription of CPTs for borings made in the North Sea. The CPTs were to constitute the basis for strength and deformation analysis of subsoil, as well as determination of sampling sites for the collection of samples for laboratory analyses. Due to the homogeneity of the subsoil, i.e. the North Sea clay, cone resistance q_c was a sufficient parameter for clustering. Clustering was performed at the levels of σ'_{vo} = 50, 75, 100, 150 kPa. Figure 19 presents the results of routine clustering of CPTs according to the criterion of the smallest distance of the boring site located in subsoil. In turn, Figure 20 shows differences in the formed agglomerations based on the Mahalanobis distance in the system of two canonical co-ordinates. However, a different picture of subsoil division is generated when cone resistance q_c is the clustering parameter. Figure 21 proves that groups with the least different cone resistance values according to the centroid method, or the group average method, are not completely consistent with the division based on the closest distance from the boring site. This fact shows that, when planning in-situ tests, an advantageous strategy needs to include, first of all, the performance of CPTs, followed by the determination of boring sites for the collection of samples for laboratory analyses.

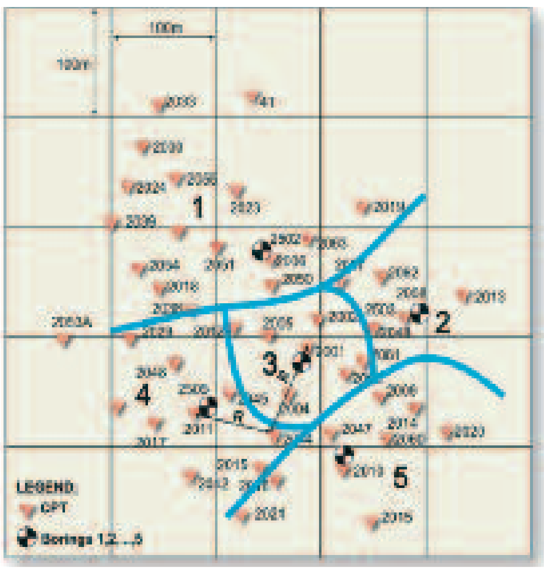

Figure 19.
Location of CPT and subsoil division into zones according to the method of min. value of distance R for identifying boring

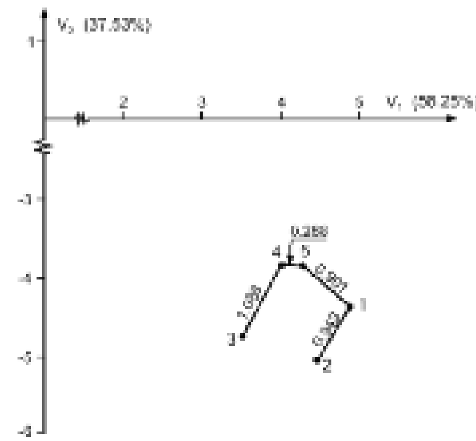

Figure 20.
Configuration of CPT agglomerations in the system of two first canonical variables [55]

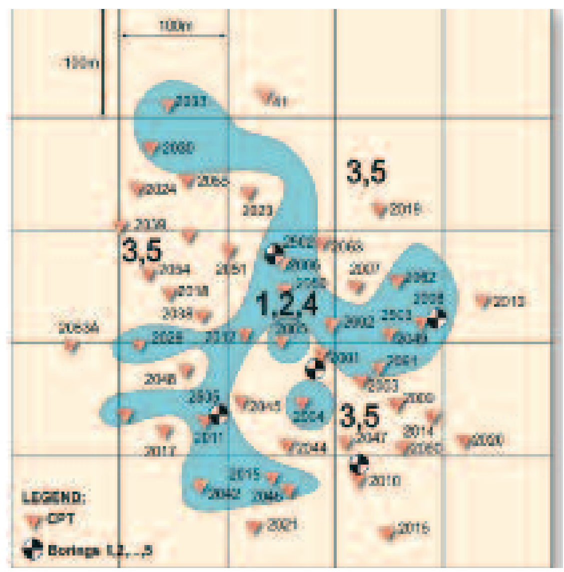

Figure 21.
Subsoil division on agglomerations of CPT according to the centroid method

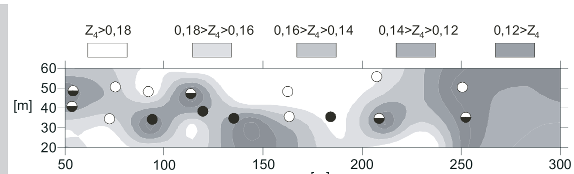

Figure 22.
Comparison between VSS test results and distribution of z0 coefficient

Figure 22 presents application of the kriging method to isolate zones with uniform bearing capacity, determined by means of VSS plate and consistency of these zones with the Proctor criterion, noted with coefficient $Z_4 = \varsigma_{dp} \cdot w^{-1}$ (w – optimal moisture content after Proctor criteria). An interesting adaptation of 1-D model for quasi 2-D model may be construction of so-called geotechnical profile. The geotechnical profile combines 1-D models established in subsoil between at least two in-situ tests. In this model the course of homogenous layers is established graphically, generally with no statistical justification. This method is commonly applied in geotechnical practice.

The procedure used to construct this type of profile on the basis of 1-D model and data from CPTU was presented in a study by Młynarek et al. [66]. Figure 23 presents the division of subsoil into homogenous layers in terms of qn and FR, as well as shows the effectiveness of subsoil division using the cosine method and the K-means method. In turn, Figure 24 presents the division of subsoil in terms of constrained modulus M. Values of moduli were ascribed to individual soil layers, established in step 1 of clustering (Fig. 23). Numerical values of the constrained modulus were calculated from correlation relationships, given in chapter 5.3. In these dependencies representative parameters q_n fom step 1 of clustering were used. Figure 24 presents the final stiffness of the subsoil and the differences in subsoil division in step 1 and step 2 of clustering. Subsoils, for which results of analyses are shown, are composed of uncertified Pleistocene and Holocene deposits, where loams and silts with sandy interbeddings are predominant. The consistency of these deposits is defined as plastic to soft plastic.

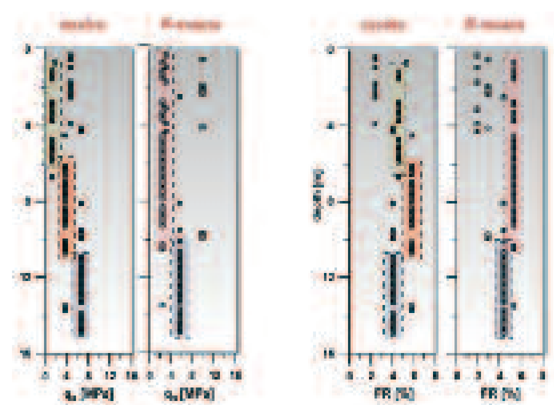

Figure 23.
Representative values of q_n and FR obtained after cluster cosine method and K-means method (First step for construction of geotechnical profile)

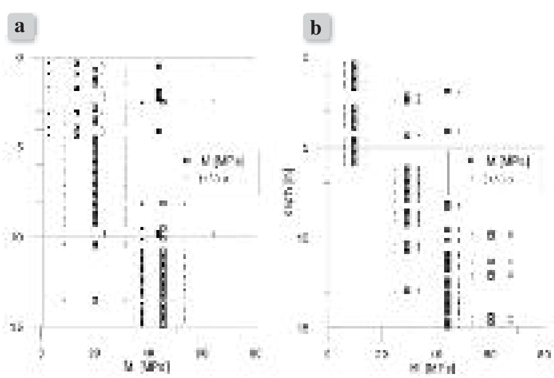

Figure 24.
Comparison between representative values of constrained modulus M, obtained on the basis of 1-st step of clustering (a) and 2-nd step of clustering (b)

5.3.3 3-D Interpolation model

Successive construction of 2-D model at consecutive planes σ'_{vo} (quasi 3-D model) may simulate the three-dimensional variation of CPTU parameters and as a consequence the three-dimensional variation of strength and deformation parameters of subsoil. However, this model does not include the effect of the 3-D variation of parameters recorded between sites of performed CPTU. Differences between quasi 3-D model and 3-D model and the concepts for the construction of 3-D model are shown in Fig. 26. The concept of 3-D interpola-tion model was presented in a study by Młynarek et al. [65]. It is based on the Inverse Distance Weighting (IDW) method, which includes statistical distribution of a given characteristic. For this method the interpolated value of a characteristic (e.g. a parameter from CPTU) in a given point with coordinates (x_0, y_0, z_0) is established on the basis of values defined by coordinates x_i, y_i, z_0. Each of these values affects the interpolated value w_0 with the weight, which is inversely proportional to the distance between these points. The formula used in IDW takes the form:

$$v_0 = \frac{\sum_{i=1}^{|N(v_0)|} w_i v_i}{\sum_{i=1}^{|N(v_0)|} w_i}, \tag{56}$$

where $|N(v_0)|$ denotes the number of observations from the neighbourhood of v_0, and weight

$$w_i = \frac{1}{(d_i + s)^p}. \tag{57}$$

d_i denotes the Euclidean distance between points (x_0, y_0, z_0) and (x_i, y_i, z_0) following the formula (54); $s>0$ and p (typical values are 1,2,3) serve the role of smoothing parameters.

Młynarek et al. [66] proposed the following modification of 2-D neighborhood $N(v_0)$, replacing it with its 3-D equivalent, where distance di between point (x_0, y_0, z_0) and points (x_i, y_i, z_0) is calculated using the equation:

$$d_i = \sqrt{(x_0 - x_i)^2 + (y_0 - y_i)^2 + \zeta (z_0 - z_i)^2}, \tag{58}$$

where ζ ($\zeta>0$) determines the effect of observations in the direction of axis z, i.e. depth or σ'_{vo}. An important role in the construction of 3-D model is played by parameter ζ. If the effect of the measured CPTU parameter along axis z is big, then it may be assumed that $\zeta = 10$. If the effect of this effect is smaller, then $\zeta = 30$. Figure 27 presents the effect of the construction of homogenous zones of cone resistance q_t and coefficient FR in subsoil using quasi 3-D and 3-D models in the adopted profile with coordinate y = 45 m. 3-D model is more objective and effective model than 2-D model. 3-D model, similarly as quasi 2-D mode, constitutes the basis for the transition from parameters from CPTU – Q_i, FR, through the identification of soils in subsoil using a selected classification system [50], to the construction of the strength and deformation model of subsoil, (Fig. 25). Presented example (Fig. 27) of the construction of subsoil structure pertained to subsoil composed of sandy loam, overconsolidated loam and fluvioglacial sands. The obtained interpretation of subsoil structure was highly consistent with the results of test borings in this area [66].

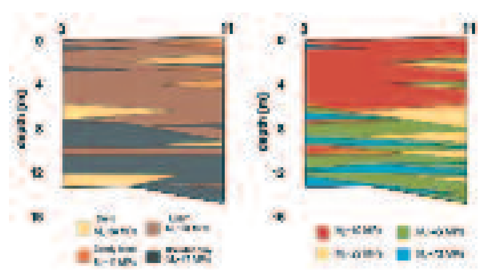

Figure 25.
Deformation profile of the subsoil constructed in the 1-st step and 2-nd step of clustering

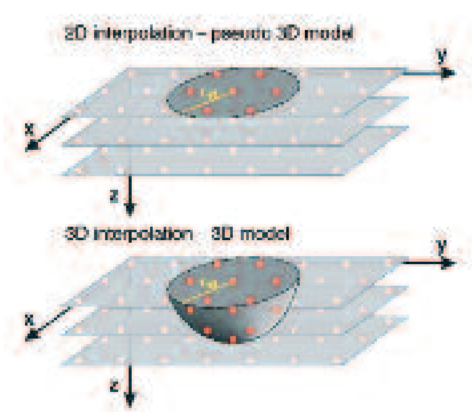

Figure 26.
Scheme of quasi 3D model and 3D model for interpretation of CPTU data

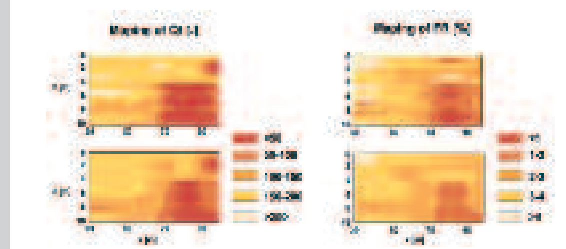

Figure 27.
The profile of homogenous zones of Q_x and FR in the plane x-z at fixed y = 45 m, according to 2D and 3D models

6. CONCLUSION AND GENERAL REMARKS

The keynote of this paper was to present the definition of applied interpretation methods of in-situ tests in a nutshell and to give formulas, used to determine strength and deformation parameters of subsoil. Another guiding idea for the preparation of this paper was to highlight the fact, frequently neglected or underestimated, of the limitations in the application of dependencies derived from theoretical solutions and adapted for the interpretation of different in-situ testing methods. These two elements are combined in the concept defined by Lunne et al. [50] as "reliable soil parameters". In view of this definition a certain philosophical question arises on the role in the determination of reliable soil parameters, which is played by advanced interpretation methods of in-situ tests, quality of the sample collected for the laboratory calibration test and importance of the quality of applied equipment, standard of education and care with which an operator performed the test, or finally random character of parameters measured during the in-situ test. A review of advances in in-situ testing allows to emphasize several additional aspects, which will undoubtedly be of interest for researchers and as a consequence also design engineers. Such issues include:

1. Quantitative and qualitative identification of factors affecting determination of so-called reliable soil parameters.
2. Determination of criteria for quality of samples to be used in laboratory calibration tests and the effect of reconsolidation in the laboratory on properties of the sample if reconsolidation includes loading and unloading processes occurring in urban areas.
3. Interpretation of in-situ tests in soils, which do not have such obvious drainage conditions as the two basic groups, sands and clay. These are soils, which

are frequently found in urban areas. Measures for the description of the state of these soils are ambiguous, their assessment may be affected by the state of subsoil contamination, whereas the effect of partial saturation may significantly limit the transformation of available correlation relationships for the determination of shear strength parameters or description of deformation characteristics. This group of soils includes fluvial deposits: silts and a wide range of organic soils and gyttja.

4. Effect of layered soil on in-situ test results and the effect of this effect on the determination of rational criteria for clustering of data and determination of representative parameters for the construction of correlation relationships in these soils. Statistical methods prove highly useful in this respect. Studies are being conducted on the subject in several research centers.

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