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# ASSESSMENT AND VERIFICATION OF CORRELATIONS IN CPTU TESTING ON THE EXAMPLE OF SOIL FROM THE WROCLAW SURROUNDINGS (POLAND)

The paper presents the results of a series of Cone Penetration Test CPTu performed near the city of Wroclaw (Poland). The tests were carried out in 13 testing points located in close distance to each other. To verify the results of the penetration tests, fine-grained soil samples from selected depths were taken for laboratory tests. The study focuses on the evaluation of soil type, unit weight, and undrained shear strength $c_u$, and compression index $C_c$. The grain size distribution of the soil and its mechanical parameters on the basis of a uniaxial compression and an oedometer tests were estimated. A comparison of laboratory and CPTu for selected values is presented. Determination of soil type was carried out on the basis of $I_{SBT}$ and $I_C$ values and good agreement with the granulometric composition was found. For undrained shear strength, commonly used correlations based on $N_k$, $N_{kt}$ and $N_{ke}$ were adopted. However, the values obtained from the CPT are significantly lower than the results from laboratory tests. Therefore, values of cone factors suitable for investigated soil type and reference test were proposed. In the case of the compression index, the coefficient values $\beta_c$ and $\alpha_m$ obtained agreed with those available in the literature. The findings presented in the paper indicate that laboratory tests remain necessary to identify soil properties from CPTu. The presented results are also a contribution to the knowledge of local soil conditions in the Lower Silesia area (Poland).

**Keywords:** cone penetration test, in situ testing, undrained shear strength, uniaxial compression, compression index, oedometer test

# 1. Introduction

Cone Penetration Tests (CPT) and Cone Penetration Tests with pore water pressure measurement (CPTu) have been successfully applied for many decades in geotechnical field investiga-

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tions. They enable the determination of a suite of physical and mechanical soil properties and are currently an important element of in-situ testing. However, the greatest difficulty concerns the interpretation of the measurements obtained. The literature contains many correlations of CPT results with various geotechnical parameters. A broad overview of empirical formulas and correlations for different physical and mechanical parameters can be found, among others, in Kulhawy and Mayne [1], Lunne et al. [2], Karlslund et al. [3], Robertson [4,5], Mayne [6], Eslami et al. [7].

The article focuses on the soil type, unit weight, and mechanical properties i.e. undrained shear strength and compression index. In case of determining the type of soil, correlations based on Robertson work [4,8,9] are commonly used. Unit weight, as the basic physical soil parameter, was studied by e.g. Robertson and Cabal [10], Mayne et al. [11], Ju et al. [12]. For undrained shear strength the issue is more complex and many correlations were introduced. Widely used expressions are those proposed by Lunne et al. [2] employing empirical cone factors. A number of papers have specified the values of these parameters under different conditions for different soils, test procedures, and both normally consolidated and overconsolidated soils [2,13-17]. Many authors have also been involved in the determination of settlements and compression parameters based on cone penetration testing [2,4,18-21]. However, the use of empirical relations requires evaluation of local geotechnical conditions and often also their calibration. Using improper correlations may lead to significant errors in geotechnical engineering. The studies on the empirical dependencies for soils occurring in Poland can be found in the works of, among others, Bednarczyk and Sandven [22], Zawrzykraj et al. [23], Konkol et al. [24].

The aim of this work is to verify commonly used correlations of physical and mechanical properties with CPTu measurements for normally consolidated fine-grained soil. The examination was carried out for a selected soil located in the surroundings of Wrocław in Poland. The analyses focused on the evaluation of soil type, unit weight, undrained shear strength, and compression index. The results of laboratory tests are presented, including the oedometric and uniaxial compression tests. In addition to the selection of appropriate correlations for the basic physical properties, the study also allowed to determine the empirical factors necessary to assess the mechanical properties of selected soils and compare them with those available in the literature.

## 2. Field investigations

The CPTu tests were performed in the town of Bierutów located near Wrocław in Lower Silesia in Poland (Fig. 1). Geographically, it is situated on the Silesian Lowland, in the part called the Oleśnica Plain. Geologically, it is located on the Fore-Sudetic Monocline. Its substrate constitutes the metamorphosed rocks of the Paleozoic. This is overlain by Permian and Triassic rocks and a complex of Tertiary rocks that form its cover. The top layer is composed of Quaternary sediments of various origins with a thickness of 10-50 m. These are Pleistocene formations, mainly related to the Central Polish glaciation: glacial and fluvioglacial units, eolian units (of limited range) and alluvial sediments connected with the North Polish glaciation and recent (Holocene) ones. Glacial till has the greatest spread, sometimes covered with younger deposits [25].

The study area measured 3 by 5 m. The research points were located in places of planned pile foundations. In total, 13 CPTu were performed, and subsequently soil samples were taken for laboratory tests. Fig. 1 presents the location of the CPTu test and the sampling points. CPTu were performed with an electric piezocone of 10 cm$^2$ cross-section area and 15 cm$^2$ friction sleeve area. Registration of the basic values cone tip resistance ($q_c$) in MPa, sleeve friction ($f_s$) in kPa and pore pressure at the shoulder ($u_2$) in kPa were logged every 20 mm. Soil samples of intact structure were collected by drilling at the point marked also in Fig. 1.

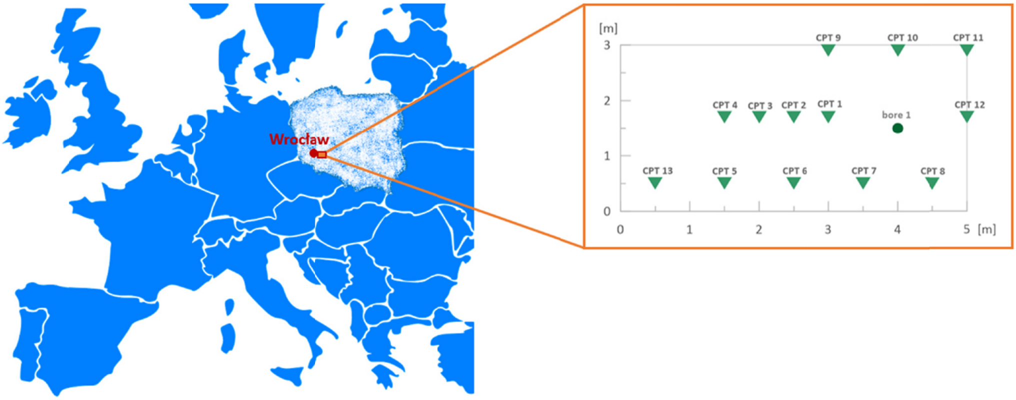


Fig. 1. Location in Bierutów near Wrocław in Poland and arrangement of testing points

All the test records are presented together in Fig. 2. Additionally, for a preliminary analysis of the profile, the friction ratio was determined (cf. [2]):

$$R_f = \left(\frac{f_s}{q_c}\right) \cdot 100\% \tag{1}$$

In the calculations, an 80 mm vertical shift of $f_s$ in relation to $q_c$ was taken into account, which is primarily related to the construction of the cone [2]. For the measurements $q_c$ and $f_s$, comparable values were obtained for all 13 soundings along the entire depth. Only registrations of $u_2$ were not consistent. This is because some of the measurements at various depths were carried out with dissipation tests, which significantly changed the $u_2$ registration.

In the profile, three zones can be distinguished (marked with a dashed lines):

- zone I (from 0 to approx. 2.75 m b.g.l.) – $q_c$ around 2 MPa and $f_s$ below 50 kPa,
- zone II (from approx. 2.75 to 6.25 m b.g.l.) – almost constant $q_c$ of 2 MPa and $f_s$ between 50 and 100 kPa,
- zone III (from approx. 6.25 to 9.0 m b.g.l.) – $q_c$ increases linearly with a depth up to about 6 MPa, $f_s$ is highly variable but always greater than 100 kPa. There were distinguished subzones IIIa to the depth of about 7 m, where $R_f$ is about 4%, and IIIb, where $R_f$ is about 3%.

These differences are also clearly visible in the $R_f$ chart (Fig. 2), where additionally the sampling depths are marked in grey. During drilling in zone I, small layers of saturated medium sand were found. The level of groundwater table was established at 1.3 m b.g.l. In the article, the analysis focuses on the selected fine-grained soil, corresponded to zone II, and for which detailed laboratory tests were conducted. Zones I and III corresponded to coarse-grained formations.

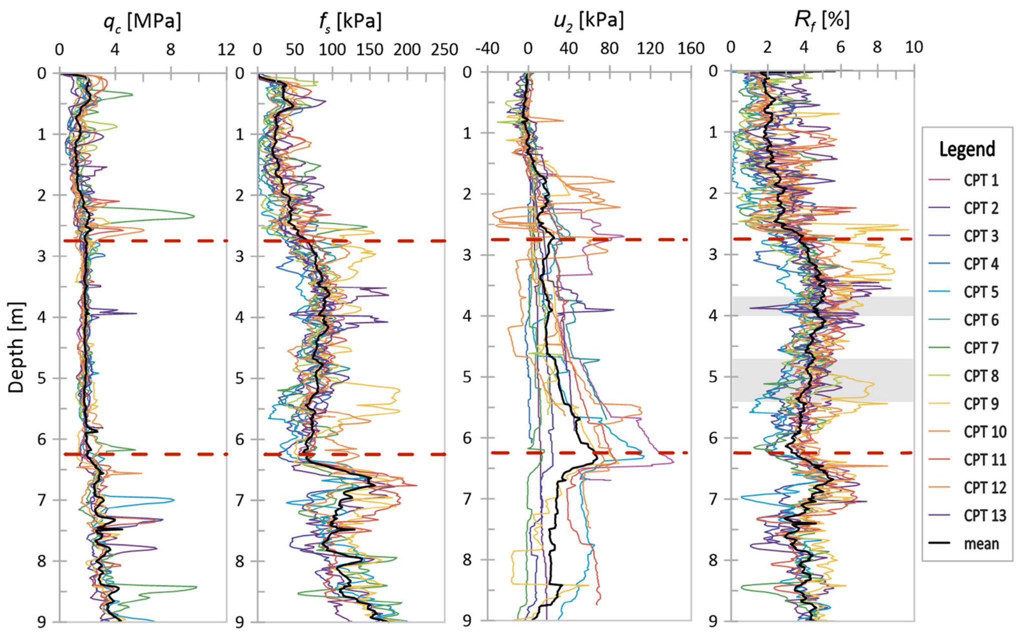


Fig. 2. Records of $q_c$, $f_s$, $u_2$ and $R_f$ results

## 3. Laboratory investigation

Soil samples taken from zone II (cores from depths of 3.7-4.0 and 4.7-5.4 m b.g.l.) were tested additionally in a laboratory. Investigations of basic properties included the evaluation of grain size distribution, water content, bulk density and consistency limits. Mechanical properties were assessed by conducting uniaxial compression and oedometer tests. All tests were performed in accordance with the standard PN-EN ISO 17892 [26].

The evaluation of the granulometric composition was carried out based on standard hydrometer analysis on three samples. All of them had similar texture and represent the same geotechnical layer. The average content of the fractions of clay (<0.002 mm), silt (0.002-0.063 mm), and sand (0.063-2 mm) was 9%, 36%, 54% respectively. Single gravel grains were also observed. According to the standard PN-EN ISO 14688 [27], soil was identified as sandy clayey silt. The soil had an average water content of 9%, a unit weight of 21.6 kN/m$^3$, and a void ratio of 0.34. The plastic limit ($PL$) and liquid limit ($LL$) were identified as 11% and 26%, hence the plasticity index $PI$ = 15% and liquidity index $LI$ = –0.13.

Uniaxial compression tests were performed on three 76mm high and 38 mm diameter specimens. The compression speed was given as 4.0 mm/h. On the basis of the results, the stress-strain curves were plotted and the value of undrained shear strength $c_u$ was determined. The parameter $c_u$ in uniaxial compression is defined as half of the compressive strength $q_u$ i.e. the maximum value of vertical stress. In the tests, the mean value of undrained shear strength was obtained as 472 kPa. The strength established corresponds to the literature data for a fine-grained soil with a certain liquidity index [28].

Oedometric tests were conducted on three samples with a diameter of 50 mm and a height of 21 mm. A standard procedure was used, in which the load was doubled in each subsequent step. The test was performed in the range of primary compression up to 1.6 MPa. Next, the oedometric curves were plotted and compression parameters were determined. The constrained moduli are dependent on the stress range, so this research focused on determining the compression index $C_c$. It is defined as the ratio of the void ratio change to the difference in effective stress taken in log scale. The index relates to the slope of the line in the graph of void ratio versus stress (in log scale). On the basis of the oedometric tests, it was established that the soil is normally consolidated with a $C_c$ of 0.073.

## 4. Soil classification

At the beginning, the soil type was assessed on the basis of CPTu measurements according to two different methods. The first one is based on non-normalized Soil Behavior Type Index $I_{SBT}$ [8], that is defined by the formula:

$$I_{SBT} = \sqrt{\left[3.47 - \log\left(\frac{q_c}{p_a}\right)\right]^2 + \left[\log R_f + 1.22\right]^2} \tag{2}$$

where: $q_c$ – the cone resistance, $p_a$ – atmospheric pressure (0.1 MPa), $R_f$ – friction ratio. For the 13 analyzed CPTu the $I_{SBT}$ value was calculated for the full height. Next, for the depths equal to the sampling depths, i.e. 3.7-4.0 and 4.7-5.4 m b.g.l., the $I_{SBT}$ points were marked on the classification nomogram. The results are presented in Fig. 3. At both depths, the range of the index values corresponds to SBT zones 3 and 4, with average values of 2.90 and 2.88. According to Table 1, the soil should be classified as silt mixtures (zone 4) or clays (zone 3). This partially coincides with the results of the grain size analysis.

TABLE 1

Soil Behavior Type classification ($I_{SBT}$) [8]

| **SBT zone** | **Soil Behaviour Type (SBT)** | $I_{SBT}$ |
|---|---|---|
| 1 | Sensitive fine-grained | — |
| 2 | Clay – organic soil | $I_{SBT} > 3.60$ |
| 3 | Clays: clay to silty clay | $3.60 > I_{SBT} > 2.95$ |
| 4 | Silt mixtures: clayey silty clay | $2.95 > I_{SBT} > 2.60$ |
| 5 | Sand mixtures: silty sand to sandy silt | $2.60 > I_{SBT} > 2.05$ |
| 6 | Sands: clean sands to silty sands | $2.05 > I_{SBT} > 1.31$ |
| 7 | Dense sand to gravelly sand | $1.31 > I_{SBT}$ |
| 8 | Stiff sand to gravelly sand* | — |
| 9 | Stiff fine-grained* | — |

* Overconsolidated or cemented

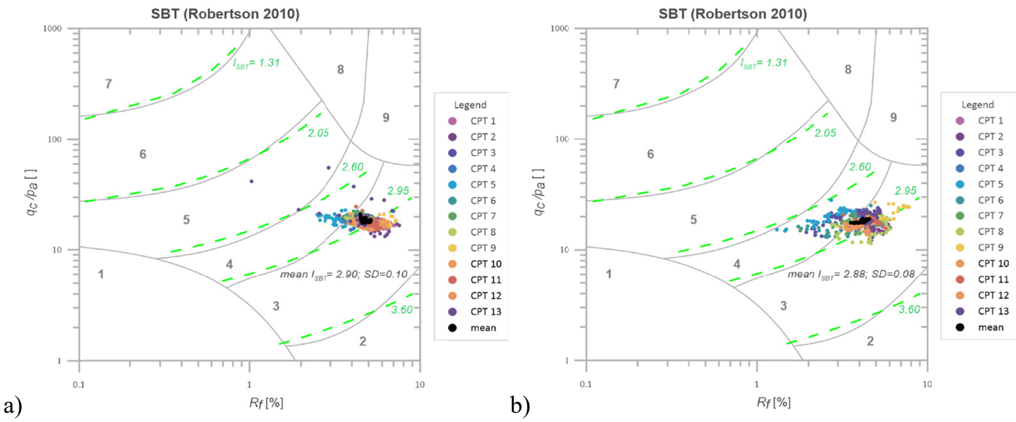


Fig. 3. Soil Behavior Type classification chart with results for depth range:
a) 3.7-4.0 m b.g.l.; b) 4.7-5.4 m b.g.l.

The second classification method is based on normalized Soil Behaviour Type Index $I_C$ [4,29], which is determined from the formula:

$$I_C = \sqrt{(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2} \tag{3}$$

where $Q_{tn}$ is normalized cone penetration resistance and $F_r$ is normalized friction ratio. These values can be calcualated from the expressions:

$$Q_{tn} = \left[\frac{q_t - \sigma_v}{p_a}\right]\left(\frac{p_a}{\sigma'_{v0}}\right)^n \tag{4}$$

$$F_r = \left[\frac{f_s}{q_t - \sigma_{v0}}\right] \cdot 100\% \tag{5}$$

where:

$$q_t = q_c + u_2(1 - a_{net}) \tag{6}$$

$$\sigma'_{v0} = \sigma_{v0} - u_0 \tag{7}$$

where: $q_c$ – cone resistance, $q_t$ – corrected cone resistance, $f_s$ – sleeve friction, $u_2$ – pore water pressure, $u_0$ – in situ equilibrium pore water pressure, $\sigma_{v0}$ and $\sigma'_{v0}$ – total and effective vertical stresses, $p_a$ – atmospheric pressure, $a_{net}$ – area ratio of the cone (0.58), $n$ – variable stress exponent. Following the recommendations of Robertson [4,9], $n$ was assumed as 1. To evaluate the vertical stresses it is necessary to apply the unit weight of the soil. The values established with CPTu measurements in accordance with (9) [30] were used as the most adequate for tested soil which is explained in the next section. The $I_C$ value was determined for the full height of the profile and the results from selected depths were subjected to further analyses. The obtained points were plotted on the Robertson [4] classification chart in the graphic representation of

Mayne [6] (Fig. 4). Most of the $I_C$ values are located in zone 4, which corresponds to silt mixtures (Table 2). Again, this is consistent with the results obtained in the laboratory tests and slightly better compared to the $I_{SBT}$.

TABLE 2

Soil Behavior Type classification ($I_C$) [4]

| SBTn Zone | Soil Behaviour Type (SBTn) | $I_C$ |
|---|---|---|
| 1 | Sensitive fine-grained | — |
| 2 | Clay – organic soil | $I_C > 3.60$ |
| 3 | Clays: clay to silty clay | $3.60 > I_C > 2.95$ |
| 4 | Silt mixtures: clayey silty clay | $2.95 > I_C > 2.60$ |
| 5 | Sand mixtures: silty sand to sandy silt | $2.60 > I_C > 2.05$ |
| 6 | Sands: clean sands to silty sands | $2.05 > I_C > 1.31$ |
| 7 | Dense sand to gravelly sand | $1.31 > I_C$ |
| 8 | Stiff sand to gravelly sand* | — |
| 9 | Stiff fine-grained* | — |
| * Overconsolidated or cemented | | |

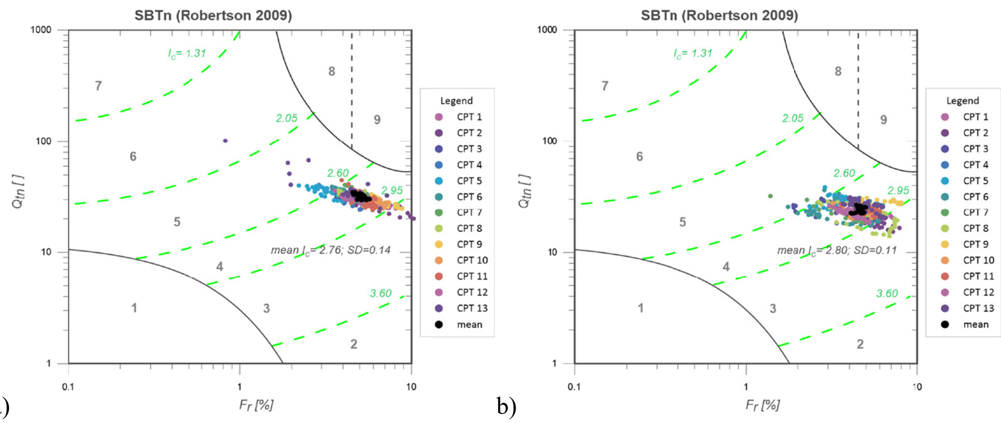


Fig. 4. Soil Behavior Type classification chart with results for depth range:
a) 3.7-4.0 m b.g.l.; b) 4.7-5.4 m b.g.l.

## 5. Unit weight

An average unit weight derived from CPTu was determined by two methods. The first one is based on Robertson and Cabal [8], in which the unit weight can be calculated from the expression:

$$\frac{\gamma}{\gamma_w} = 0.27 \cdot \log R_f + 0.36 \cdot \log\left(\frac{q_t}{p_a}\right) + 1.236 \tag{8}$$

where: $\gamma$ – unit weight of soil, $\gamma_w$ – unit weight of water, $p_a$ – atmospheric pressure. In Fig. 5 a comparison of the results for selected depths with the classification chart is presented. Mean unit weights at selected depths is 18.35 and 18.16 kN/m$^3$ respectively. In laboratory tests, a value equal to 21.6 kN/m$^3$ was found, which is significantly higher than that obtained by this method.

Therefore, a different approach has been used to derive unit weight from CPTu, using a correlation proposed by Bagińska [30]:

$$\gamma = 11 + 2.4 \cdot \ln(f_s + 0.7) \tag{9}$$

By applying the above formula, values were calculated for zone II. Fig. 6a illustrates the established values for the entire zone. Figures 6b and 6c show the histograms for the depths of 3.7-4.0 and 4.7-5.4 m b.g.l. separately, and Fig. 6d shows the histogram of the unit weight values established cumulatively for the two depth ranges. The value of a single interval for all histograms is 0.2 kN/m$^3$ and Gauss distribution is used to describe the results. For depths from 3.7 to 4.0 m b.g.l. an average weight of 21.70 kN/m$^3$ was obtained, for 4.7 to 5.4 m b.g.l. – 21.37 kN/m$^3$, and for both ranges altogether 21.47 kN/m$^3$. These values are very similar to 21.6 kN/m$^3$ obtained in laboratory analyses. This allows to recognize the above correlation as adequate for the investigated soil.

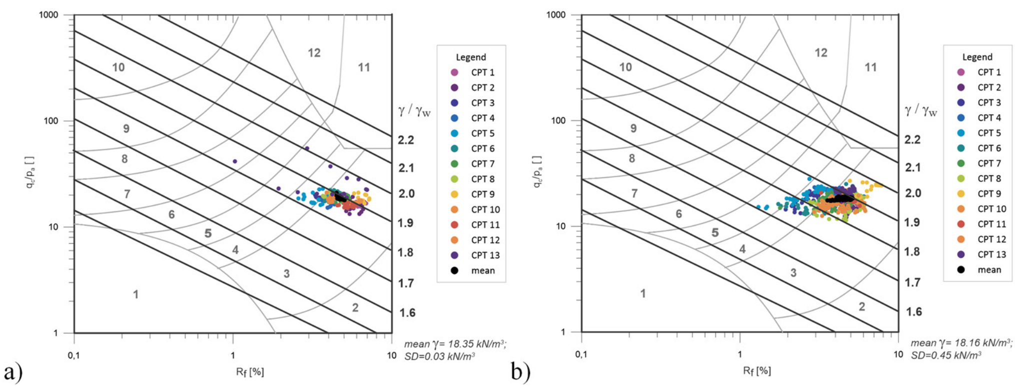


Fig. 5. Unit weight chart with results for depth range:
a) 3.7-4.0 m b.g.l.; b) 4.7-5.4 m b.g.l.

Based on the presented results in this and the previous paragraph, it can be stated that comparable values are obtained for both analyzed depth intervals within zone II. Therefore, it is confirmed that the soil from both depths belongs to one geotechnical layer and all further calculations were performed for both ranges jointly.

## 6. Undrained shear strength

In this study 3 commonly deployed expressions for undrained shear strength from Lunne et al. [2] and references therein were considered:

$$c_u = (q_c - \sigma_{v0})/N_k \tag{10}$$

$$c_u = (q_t - \sigma_{v0})/N_{kt} \tag{11}$$

$$c_u = (q_t - u_2)/N_{ke} \tag{12}$$

where $N_k$, $N_{kt}$, $N_{ke}$ are empirical cone factors. These methods differ in the values and factors from which strength is determined. Cone factors $N_k$, $N_{kt}$, and $N_{ke}$ take a wide range of values and depend on both the type of soil and the reference method from which $c_u$ is derived. Most of the existing test-based correlations of the factors refer to the results of triaxial compression (e.g. [31]), direct shear or the mean of the tests: triaxial compression, extension and direct shear. These type of experiments are time-consuming and require significant financial effort. Therefore, this work focused on determining cone factors for selected soil types using uniaxial compression testing. This is a simple and quick test for evaluating undrained shear strength, which significantly simplifies the calibration procedure for empirical formulas. The evaluation of cone factors with uniaxial compression tests was carried out by e.g. Cheshomi [32]. However, it should be noted that the application of a specific reference test has a potential large impact on the obtained values of $c_u$ [33], and consequently on the values of cone factors. Therefore, only limited values for the $N_k$, $N_{kt}$ and $N_{ke}$ can be used for particular tests.

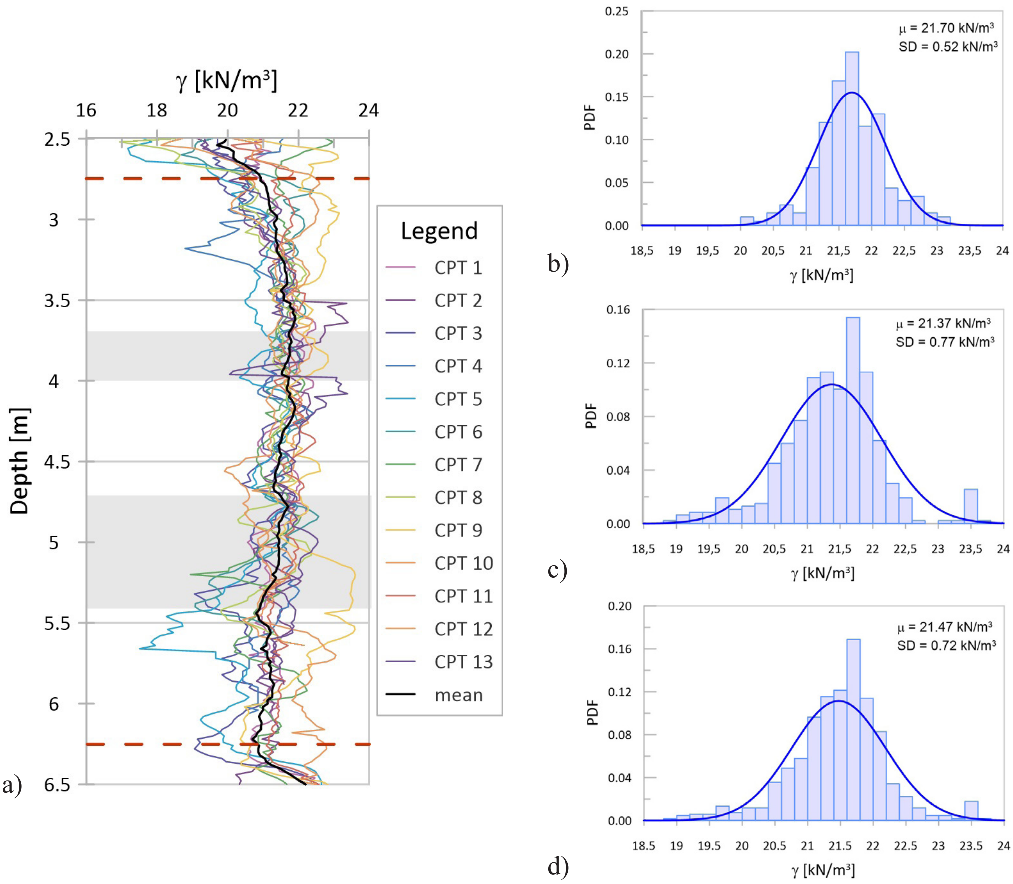


Fig. 6. Unit weight results: a) for zone II; histograms for depth range:
b) 3.7-4.0 m b.g.l.; c) 4.7- 5.4 m b.g.l.; d) 3.7-4.0 and 4.7-5.4 m b.g.l. in total

The assessment of cone factor values was based on formulas (10)-(12). The results of laboratory tests were used for $c_u$, and values for $q_c$, $q_t$, $\sigma_{v0}$ i $u_2$ were determined from CPTu tests, and cone factors were treated as unknown (cf. [15]). Therefore, variability of these coefficients within the analyzed depths was obtained. Figures 7, 8 and 9 show the outcomes of calculations for $N_k$, $N_{kt}$, and $N_{ke}$ respectively. On the left hand side there is a comparison of results from all CPTu tests for the zone II. On the right hand side there are histograms showing determined cone factors for selected depths. The value 0.2 was taken as the width of a single interval for all histograms and the Gauss distribution is used for the description.

The mean values of cone factors $N_k$, $N_{kt}$, and $N_{ke}$ obtained are 3.67, 3.65, and 3.80, respectively. These values are much lower than those recommended in the literature for this type of soil. A summary of the cone factors values proposed for clayey soils by various authors was presented e.g. by Cheshomi [32]. The values of $N_k$ vary from 5 to 28, $N_{kt}$ from 4 to 29, and $N_{ke}$ from 1 to 18. Most of the results are based on triaxial compression tests so the differences are probably related to both the characteristics of the examined soils and the type of reference test, e.g. partial saturation of the specimen in unconfined compression test.

The next step was to evaluate a widely applied calculation formulas, which allows to specify the most common cone factors. The expressions used to calculate $N_k$ proposed by Larsson and Mulabdic [34]:

$$N_k = 13.4 + 6.65 \cdot LL \tag{13}$$

Shin and Kim [35]:

$$N_k = 0.285 \cdot PI + 7.636 \tag{14}$$

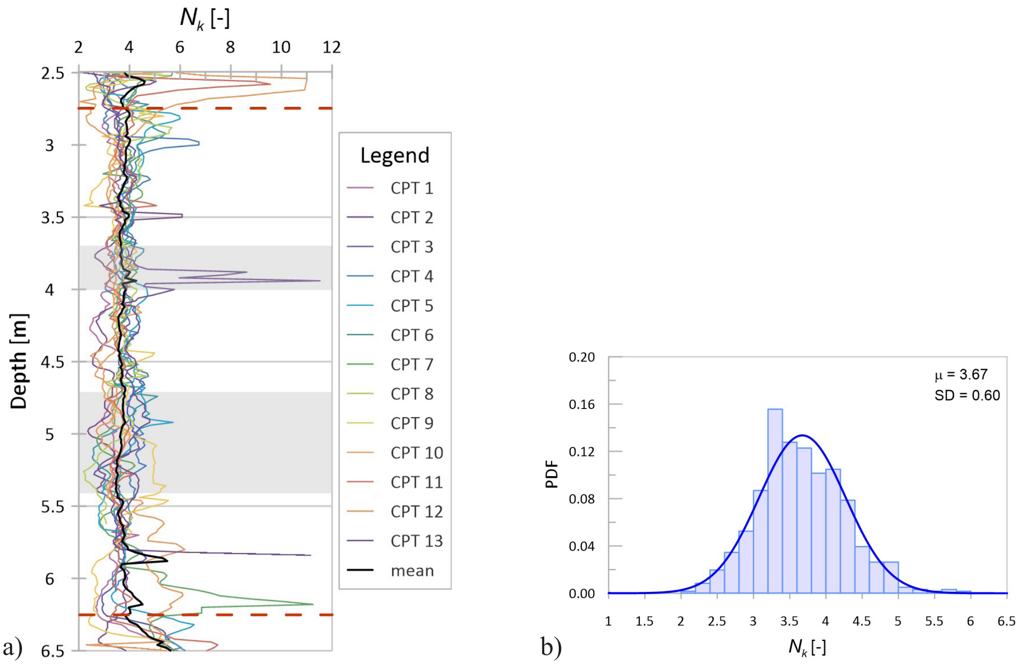


Fig. 7. Cone factor $N_k$ results: a) for zone II; b) histogram for the selected depth range (in gray)

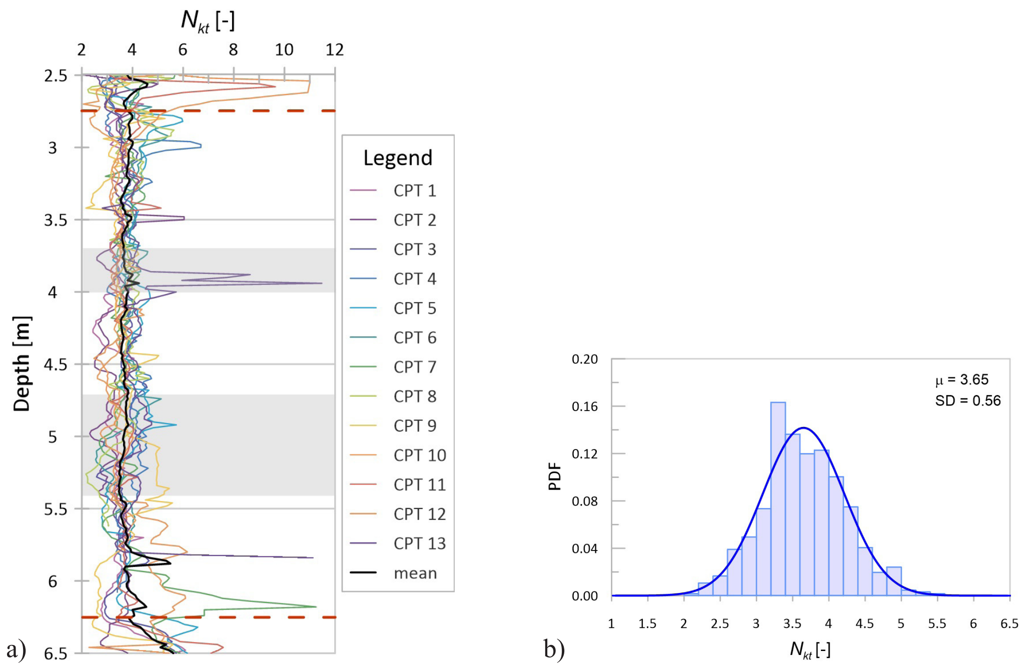


Fig. 8. Cone factor $N_{kt}$ results: a) for zone II; b) histogram for the selected depth range (in gray)

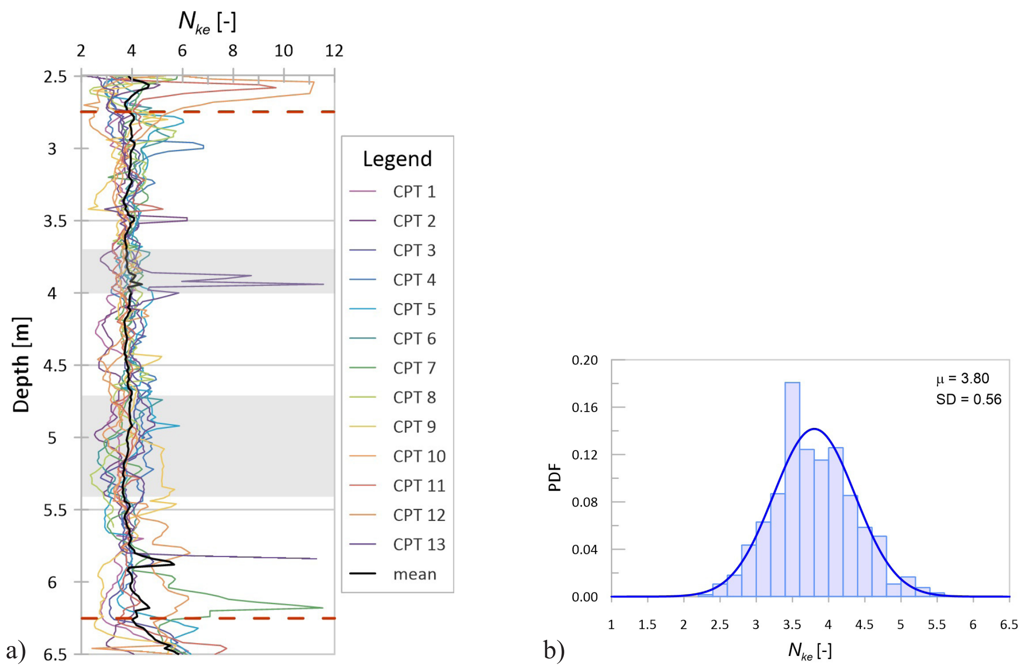


Fig. 9. Cone factor $N_{ke}$ results: a) for zone II; b) histogram for the selected depth range (in gray)

and El-Bosraty et al. [36]:

$$N_k = 27.3 - (3.6 \cdot LL/PL) \tag{15}$$

were verified. They are derived from Atterberg's limits ($PL$, $LL$) and the plasticity index ($PI$). For $N_{kt}$, the formulas proposed by Lunne et al. [2]:

$$N_{kt} = 10.5 + 7 \cdot \log F_r \tag{16}$$

$$N_{kt} = 10.5 - 4.6 \cdot \ln(B_q + 0.1) \tag{17}$$

and Bałachowski et al. [37]:

$$N_{kt} = 1.242 \cdot F_r + 7.803 \tag{18}$$

were evaluated. $F_r$ is normalized friction ratio (5) and $B_q$ is pore pressure parameter calculated from:

$$B_q = (u_2 - u_0)/(q_t - \sigma_{v0}) \tag{19}$$

To determine $N_{ke}$ the formula based on $B_q$ proposed by Hong et al. [38]:

$$N_{ke} = 22 - 22 \cdot B_q \tag{20}$$

was used. The cone factor values calculated from formulas (13)-(20) are summarized in Table 3. All factors are substantially higher than those obtained from laboratory test, and as a consequence, the $c_u$ established that way would be much lower. Therefore, presented equations are not recommended for the selected soil type and uniaxial compression as a reference test. To specify which of these factors has a greater impact, it would be necessary to carry out additional triaxial tests.

TABLE 3

Comparison of calculated cone factors $N_k$, $N_{kt}$, and $N_{ke}$

| Reference | Equation | $N_k$ [-] | $N_{kt}$ [-] | $N_{ke}$ [-] |
|---|---|---|---|---|
| Larsson & Mulabdic (1991) | (13) | 15.13 | — | — |
| Shin & Kim (2011) | (14) | 11.91 | — | — |
| El-Bosraty et al. (2020) | (15) | 18.79 | — | — |
| Lunne et al. (1997) with $F_r$ | (16) | — | 15.10 | — |
| Lunne et al. (1997) with $B_q$ | (17) | — | 21.29 | — |
| Bałachowski et al. (2018) | (18) | — | 13.81 | — |
| Hong et al. (2010) | (20) | — | — | 22.08 |
| Examination results | — | 3.67 | 3.65 | 3.80 |

## 7. Compressibility

Most studies on the deformation parameters focus on determining the constrain modulus. However, this study concentrates on the determination of compression index $C_c$, which characterizes the soil behavior for virgin compression, as is more reliable for soil description. The expression used to determine $C_c$ is [39]:

$$C_C = (1 + e_0) \cdot 2.3 \cdot \beta_c \cdot \sigma'_{v0} \cdot \left( \frac{1}{q_n} \right) \tag{21}$$

$$q_n = q_t - \sigma_{v0} \tag{22}$$

where: $e_0$ – initial void ratio, $q_n$ – net cone resistance, $q_t$ – corrected cone resistance, $\sigma_{v0}$ and $\sigma'_{v0}$ – total and effective vertical stress, $\beta_c$ – coefficient. Here, the $\beta_c$ value was treated as unknown, $C_c$ and $e_0$ values were taken from laboratory tests and $q_n$ and $\sigma'_{v0}$ were derived from CPTu. In Fig. 10, a compilation of results from all CPTu is shown for zone II and a histogram of the cone factor values. The width of a single interval is 0.07 and the Gauss distribution is adopted for the description. The mean value of $\beta_c$ established in this procedure is 0.60. In the literature most of the correlations refer to a coefficient $\alpha_m$ that corresponds to $1/\beta_c$. According to Sanglerat [40], this value for similar soil type is between 2 and 3, which coincide with $\beta_c = 0.3$-$0.5$. According to Mayne [41], the value of $\alpha_m$ depends on the soil type and ranges between 1 and 10, and lower values relate to soft clays. The results of investigations ($\beta_c = 0.60$ and accordingly $\alpha_m = 1.67$) correspond to the data from the literature.

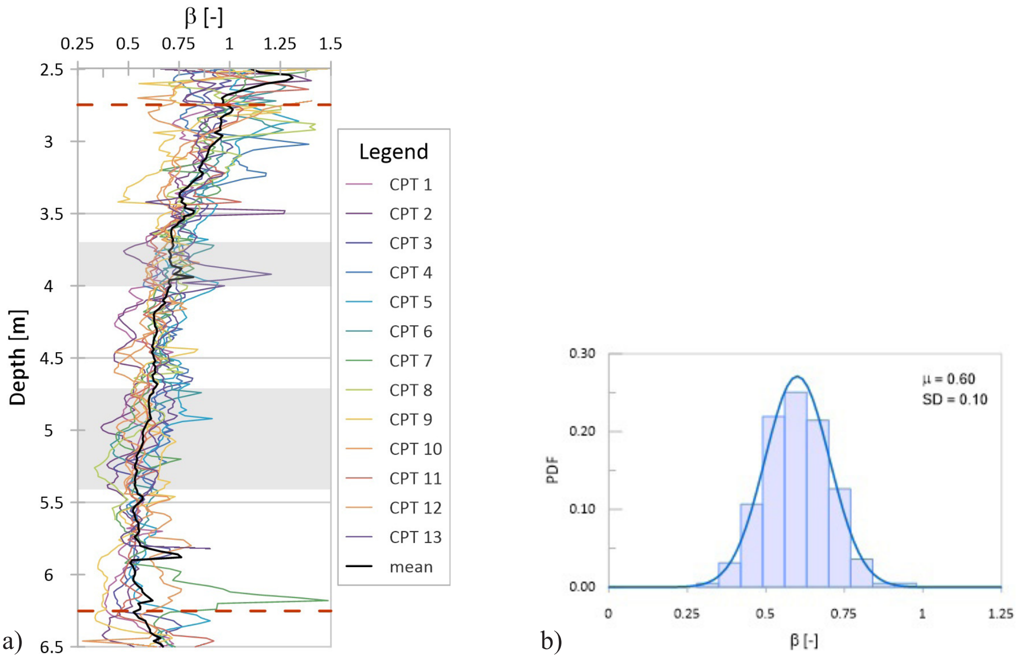


Fig. 10. Cone factor $\beta_c$ results: a) for zone II; b) histogram for the selected depth range (in gray)

## 8. Conclusions

The paper presents the results of a series of CPTu tests for fine-grained soil with laboratory evaluation of physical and mechanical properties. The commonly used empirical formulas used to determine the basic soil properties were verified. On the basis of the laboratory tests the cone factors $N_k$, $N_{kt}$, $N_{ke}$, and coefficients $\beta_c$, and $\alpha_m$ adequate for the tested soil were established. This allowed for comparison of results and validation of selected correlations.

In the case of soil classification, the highest compliance was obtained with Soil Behaviour Type Index $I_C$ and for unit weight with the empirical formula proposed by Bagińska [30]. Determination of cone factors $N_k$, $N_{kt}$, $N_{ke}$ for undrained shear strength was carried out using a uniaxial compression test, which allows for quick strength assessment. The established values of factors are much lower from those presented in the literature and derived from commonly used formulas. The considerations presented should be treated as preliminary ones due to the small number of samples tested. However, even on this basis, significant differences in the values of cone factors can be seen. Those can be related to specificity of the soil and the selection of the unconfined compression as a reference test. For better understanding of the phenomena and determination which element have a crucial influence, further examination should be undertaken. Evaluation of cone factors $\beta_c$ and $\alpha_m$ for compression parameters was performed with an oedometric test. The outcomes are comparable with those presented in the literature.

The findings clearly show that additional laboratory tests are necessary when estimating substrate properties based on CPTu sounding. Such verification is essential for proper calibration of the correlations used with actual local conditions. A direct application of the values contained in the literature may be appropriate in the case of soil with similar qualitative and quantitative characteristics to ground from the area. Attention should also be paid to the manner of conducting the laboratory investigations, as they can have a significant impact on the outcomes.

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