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Geotechnical characterization of soft soil deposits in Northern Poland

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Highlights

- Case study of soil classification and geotechnical properties
- Discrepancies between CPTU, DMT and USCS classifications
- Local empirical correlations for soft soil compressibility parameters
- Shear strength parameters of organic soils from in-situ and laboratory tests

Abstract

This paper presents a geotechnical characterization of deltaic soft soil deposits in the Vistula Marshlands, northern Poland. It shows the limited applicability of organic soil classifications based on Cone Penetration Tests (CPTU) and Dilatometer Tests (DMT). None of the in-situ-based classifications correctly identifies peat. Analysis of the behaviour of contractive/dilative soil layers according to Robertson's updated classification (2016) is shown to be in agreement

with volumetric changes observed during triaxial compression tests. The coefficient of primary compression C_c is found to decrease exponentially with the initial bulk density (ρ_{d0}) and to increase linearly with the in-situ water content (w_c). The presented geotechnical characterization and reference data can be used for foundation design and soil improvement in the soft organic soils of northern Poland.

KEYWORDS: compressibility, shear strength parameters, soil classification, soft soil

1. Introduction

Some regional correlations concerning the interpretation of in-situ tests in the Holocene organic soils of Poland were proposed by Lechowicz (1997), Młynarek et al. (2008, 2010) and Rabarijoely (2018). The geotechnical parameters of Eemian peats were described by Zawrzykraj et al. (2017). However, these studies consider mainly central and western Poland. The aim of this paper is to provide a comprehensive geotechnical investigation concerning the Vistula Marshlands (Figure 1) and to propose some local correlations for geotechnical design parameters in the Gdańsk region.

The study is divided into three parts. The first concerns the determination of soil classification with laboratory tests and using CPTU and DMT classification charts, particularly with regard to soft organic soils such as clays, silts and peats. The second part deals with soil compressibility in one-dimensional conditions and discusses sample disturbances resulting from the probing. Particular attention was drawn to the estimation of compression coefficient C_c and its relation to the physical properties of the soft soil. In the third part, shear strength parameters of the considered soft soils are discussed, taking into account in-situ test results and CIU and CID triaxial tests. The observed dilative/contractive phenomena in the triaxial tests is compared to soil behaviour type according to Robertson's (2016) chart.

2. Study site description

The test site was in Jazowa, next to the S7 expressway in the Vistula Marshlands, near Elbląg and c. 50 km to the Southeast of Gdańsk. Two soft soil deposits interbedded with a sand layer were found (Figure 2). The first one (approximately 3 m thick), at 0.7 m to 4.05 m depth, contained a mixture of low plasticity silts and clays (ML/CL), organic silty clays (OH) and peats (Pt). A layer of loose to medium-dense sand (SM) was found at 4.05 m to 7.05 m depth. The second soft soil layer, between 7.05 m and 14.45 m, contained mainly organic silt (OH) with some peat (Pt). A compacted layer of sand was found below 14.45 m with a 0.25 m inclusion of organic soil at 15.7 m. The water table was approximately 1.7 m below the surface. The analyzed soft soil is generally normally consolidated with slight overconsolidation in the upper part of the deposits, due to water level changes and climate action.

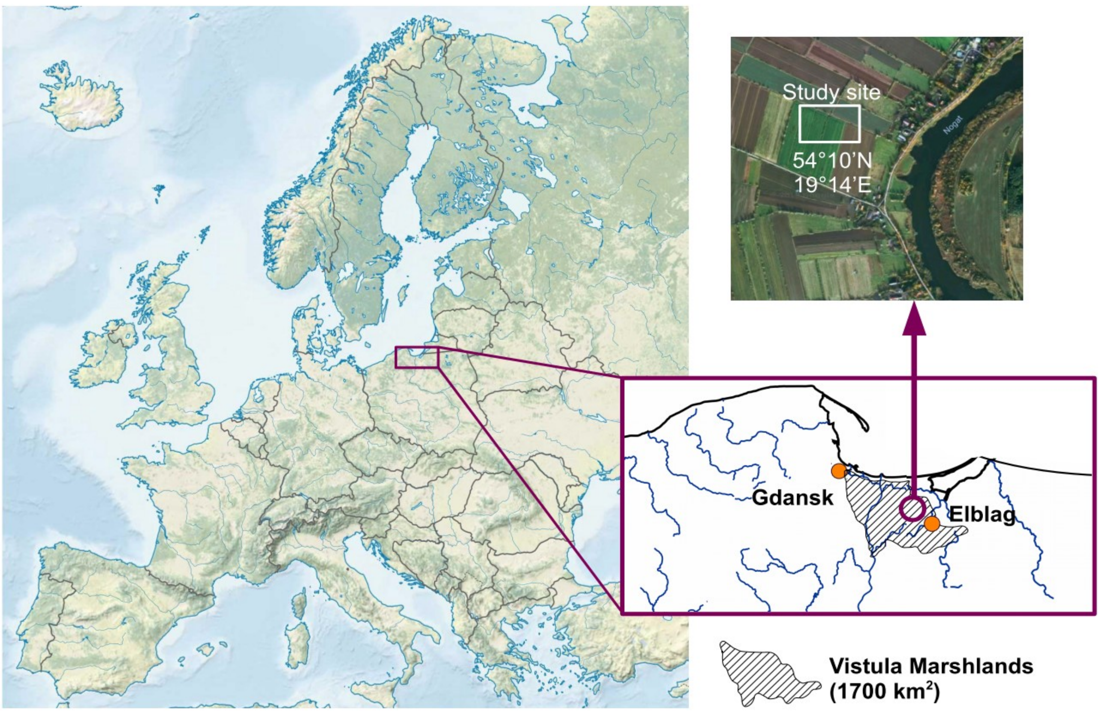

Figure 1. Location of the testing site. [no colour]

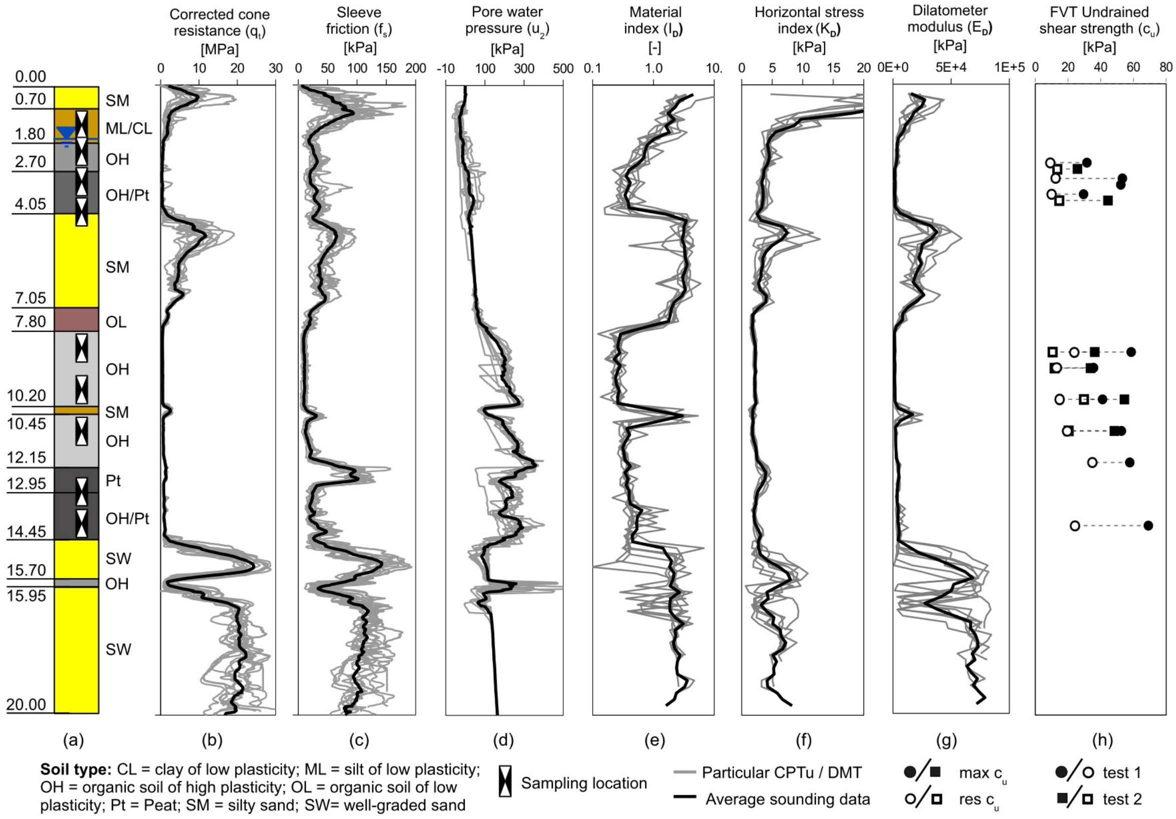

Figure 2. (a) Jazowa site soil profile, (b)-(d) CPTU probing results, (e)-(g) DMT results, (h) FVT sounding results. [no colour]

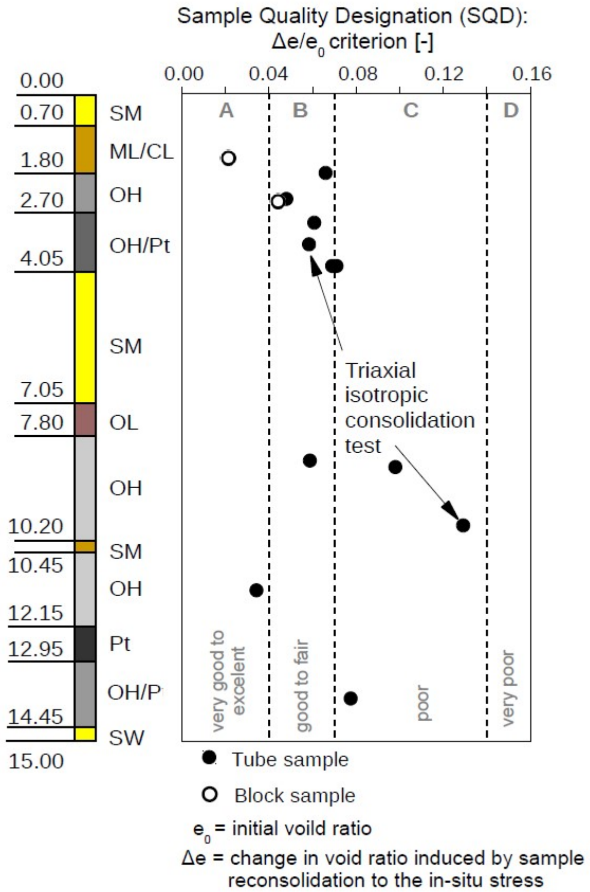

Figure 3. Sample Quality Designation class (A-D) based on $\Delta e/e_0$ criterion. [no colour]

2.1. Field investigation

Fifteen CPTU tests with a 10 cm^2 electric piezocone were carried out down to approximately 20 m, in accordance with ASTM D5778 (2012). The test results for corrected cone resistance (q_t), friction sleeve (f_s) and pore water pressure (u_2) are presented in Figures 2b-d, with average values in bold. The corrected cone resistance is defined as:

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

where: q_c = cone resistance, u_2 = pore water pressure measured at shoulder filter position, a = cone correction factor (equal to 0.84).

Eight DMT tests were conducted in accordance with ASTM D6635 (2015) in the vicinity of the CPTU soundings. The obtained material index (I_D), horizontal stress index (K_D)

and dilatometer modulus (E_D) with calculated average values are presented in Figures 2e-g. The DMT indexes are defined as:

$$I_D = (p_1 - p_0)/(p_0 - u_0) \tag{2}$$

$$K_D = (p_0 - u_0)/(\sigma'_{v0}) \tag{3}$$

$$E_D = 34.7(p_1 - p_0) \tag{4}$$

where: p_0 = corrected first reading, p_1 = corrected second reading, u_0 = in-situ pore water pressure, σ'_{v0} = in-situ effective vertical stress.

Two electrical Field vane tests (FVT) with an unprotected tapered 13.0 x 6.5 cm vane and electronic measurement system were performed in terms of ASTM D2573 (2015). During the tests, the vane instrument rotates the extension rods from the surface. Downhole, the torque is taken up during the first 15 degrees of rotation by the slip-coupling on top of the vane. Thereafter, the torque is transmitted to the vane. The maximum and residual values of undrained shear strength measured at the site are given in Figure 2h.

The sampling depths are presented in Figure 2a. The tube samples were extracted using a Piston Sampler ST:1 equipped with three 170 mm long liners and an inner diameter of 50 mm. The cubical block samples were taken from a shallow excavation. The block samples were carefully separated from the parent material, placed in a rigid box, wrapped with cheesecloth and covered with melted wax.

The quality of the specimens, often denoted as Sample Quality Designation (SQD), is an important aspect when interpreting laboratory data. The undrained shear strength, compressibility parameters and sensitivity are influenced by the sample quality (Karlsrud and Hernandez-Martinez, 2013). The SQD can be verified in one-dimensional or isotropic consolidation tests. Taken into account was the change in the void ratio ($\Delta e/e_0$) due to disturbance during sampling, in accordance with the Lunne et al. (1997) criterion. Eleven oedometer and two triaxial tests were used to evaluate the SQD. A good sample quality

($0.04<\Delta e/e_0<0.08$) was generally observed (Figure 3), apart from the peat (~14 m depth) and organic silt (~10 m depth) specimens, where the ratio was $\Delta e/e_0>0.08$. The excavated block samples were of better quality than the tube samples. The void ratio criterion may not necessarily be valid for silt and peat sample quality, because this criterion was not originally meant for these types of soils. As it was already observed by Ladd et al. (1999), the measured $\Delta e/e_0$ ratios increase generally with depth, see Figure 3. As overburden stress increases, greater stress relief during sampling should be expected, resulting in a larger $\Delta e/e_0$ ratio (deJong et al., 2018).

2.2. Basic soil properties

The basic physical soil properties, such as water content, soil unit weight and specific gravity, were determined at the sampling depths. The dataset of the index properties of the Vistula Marshland soft soils is presented as a result of comprehensive laboratory tests. The organic matter in the soft soil deposits was measured with the loss on ignition method for twelve selected samples according to ASTM D2974 (2014). The consistency limits were determined in terms of ASTM D4318 (2017) on thirteen selected specimens. The soil pH was measured on eight samples, using the reference electrode Sentix 41. The 50 g soil samples were diluted in 100 ml of distilled water and the pH analysis was carried out after 24 hours. A granulometric analysis was performed using the laser diffraction method (e.g., Eshel et al., 2004), which considers five repeated measurements. Consequently, the Particle Size Distribution (PSD) for eight samples was determined.

The soft soils analyzed in this paper are high-plasticity organic silts (silty muds), organic silty clays (clayey muds) and peats. The selected index properties of these soils are summarized in Table 1 and Figure 4. The water content varied from 47% to 76% for muds and was over 170% for peats. The specific gravity of the muds was approximately 2.57 g/cm^3. In the case of peats, the specific gravity was much lower (between 1.6 and 1.7 g/cm^3), on account of its fibrous structure (e.g., Cheng et al., 2007; Mesri and Ajlouni, 2007). The peat layers had a several times higher content of organic matter than the muds. This is in line with most of the classification systems, where the boundary between organic soils (muds) and highly organic soils (peats) is usually fixed at 30% of the organic matter content. The pH ranged from 5.8 (moderately acidic) to 7.9 (moderately alkaline). The majority of the examined soils did not react to hydrochloric acid. Only the highly organic soils from 8 m to 12 m started to effervesce, which indicates a calcium carbonate content of up to 3% at 8.5 m, and over 5% at approximately 12 m, where sea shell traces were noticed. The dataset of the Jazowa site, including the geotechnical parameters, is provided as an electronic supplement to this paper.

Table 1. Selected index properties of Jazowa soft soil deposits

Soil symbol	*Sampling depth*	$w_c^{(1)}$	$\gamma^{(1)}$	$G_s^{(1)}$	*PI*	*Clay content*$^{(1)}$	*Silt content*$^{(1)}$	*LOI*$^{(2)}$	*pH*$^{(2)}$	*CaCO$_3$ content*$^{(1)}$
	[m]	*[%]*	*[kN/m^3]*	*[g/cm3]*	*[%]*	*[%]*	*[%]*	*[%]*	*[-]*	*[%]*
ML	1.3÷1.6	30.6	17.07	2.65	21.6	33.1	66.6	5.8	8.0	<1
ML/OH	1.85	45.4	17.6	-	-	-	-	4,4	-	-
OH	2.2	54.4	-	-	49.7	-	-	-	-	-
OH	2.45	68.9	17.0	-	-	-	-	8,3	-	-
OH	2.3÷2.6	75.9	14.22	2.54	63.7	51.3	48.7	11.4	5.8	<1
OH	3.2÷4.2	64.4	14.52	2.61	54.8	44.6	55.1	16.2	6.0	<1
Pt	~4	264.1	11.58	1.71	-	-	-	69.9	5.2	<1
OH	8.10	46.8	16.9	-	-	-	-	6,7	-	-
OH	8.1÷8.9	46.3	15.7	2.59	23.3	5.8	94.2	4.2	7.0	1-3
OH	8.65	57.3	16.9	-	-	-	-	4.5	-	-
OH	9.5÷10.5	45.4	16.3	2.67	27.5	2.3	97.6	-	-	3-5
OH	11.1÷11.9	49.8	15.6	2.54	15.7	1.8	96.8	7.1	7.9	>5
OH	13.0	41,8	-	-	37,4	-	-	-	-	-
OH	~13.4	77.2	15.9	-	28.5	5.6	93.5	7.0	8.2	>5
Pt	~14	179.2	10.5	1.57	-	-	-	87.2	7.0	<1

(1) = average value, (2) = point value, w_c = water content, γ = soil unit weight, G_s = specific soil gravity, PI = plasticity index, LOI= Loss on Ignition

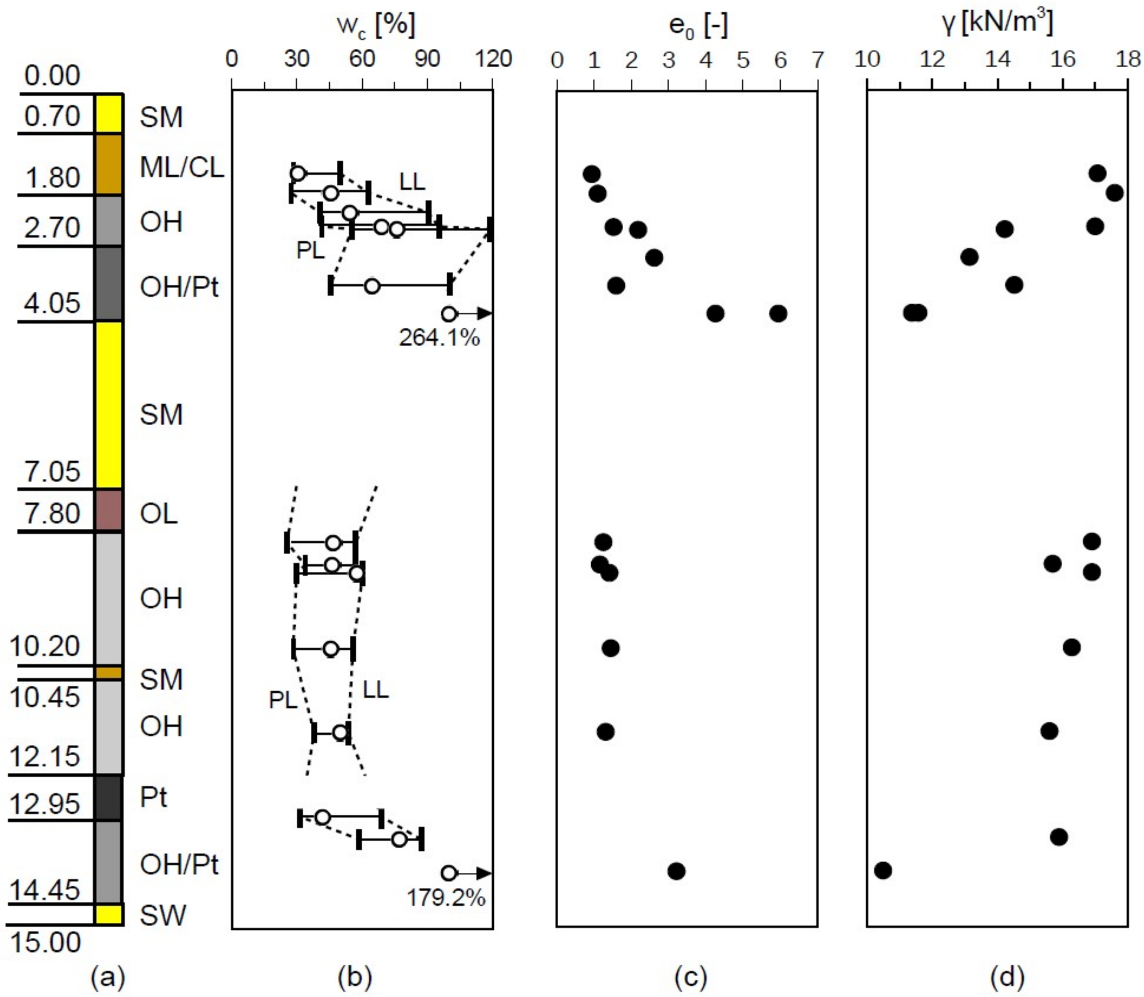

Figure 4. Soil properties of Jazowa soft soil deposits: (a) soil profile, (b) Atterberg's limits, (c) initial void ratio, (d) soil unit weight. [no colour]

3. Soil classification

3.1. Methods

Three different soil classification systems were applied: USCS, CPTU- and DMT-based. The Unified Soil Classification System (USCS), according to ASTM D2487 (ASTM D2487, 2017), uses the Casagrande's plasticity chart. The other two are in-situ behavioural soil classifications.

The soft soils were classified according to the CPTU charts proposed by Robertson (1990), Schneider et al. (2008) and Robertson (2016). The normalized cone resistance Q_t and friction ratio F_r were calculated as follows (Robertson, 1990):

$$Q_t = (q_t - \sigma_{v0}) / \sigma'_{v0} \tag{5}$$

$$F_r = [(f_s / q_t - \sigma_{v0})] \times 100\% \tag{6}$$

where: q_t = corrected cone resistance, σ_{v0} = in-situ vertical total stress, σ'_{v0} = in-situ vertical effective stress, f_s = sleeve friction.

The normalized pore water pressure parameter B_q and net cone resistance q_n are defined:

$$B_q = (u_2 - u_0)/(q_t - \sigma_{v0}) \tag{7}$$

$$q_n = q_t - \sigma_{v0} \tag{8}$$

where u_0 = in-situ pore water pressure.

The contractive/dilative soil behaviour type (SBT) was determined using the updated Robertson's classification (Robertson, 2016), with the following variables:

$$Q_{tn} = [(q_t - \sigma_{v0}) / p_a](p_a / \sigma'_{v0})^n \tag{9}$$

$$n = 0.381 \times I_c + 0.05 \times (p_a / \sigma'_{v0}) - 0.15 \tag{10}$$

$$I_c = \left[(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2\right]^{0.5} \tag{11}$$

where: Q_{tn} = normalized cone resistance, p_a = atmospheric reference pressure equal to 100 kPa, n = variable stress exponent, I_c = soil behaviour type index.

The last classification system was based on the Marchetti and Crapps (1981) DMT chart.

The statistical evaluations of the CPTU and DMT readings were performed for classification purposes. The required parameters, such as Q_t, Q_{tn}, F_r, B_q, q_n, Δu_2 for CPTU and I_D, E_D for DMT, were averaged within the soil layers presented in Figure 2a. Next, the global average values and corresponding standard deviations (SD) for each soil layer were calculated. Finally, the global average values and SD error boxes were plotted in the classification charts.

3.2. Results and interpretation

The Jazowa soft soils, classified according to USCS, are presented in Figure 5. The graph is based on the data given in Figure 4, where Atterberg's limits are shown. The soil layer from 0.7÷1.8 m has a liquid limit (*LL*) of less than 50% and lies below the A-line and can be defined as low plastic silt (ML) according to ASTM D2487 (2017). Soils sampled from the 2.3÷4.05 m and most of the samples from 7.8÷14.45 m have a liquid limit exceeding 50%, and are classified as organic silts (OH). Samples taken from approximately 4 m and 14 m are classified as peats (Pt) due to their high organic matter content (Table 1). The layers at 2.7÷4.05 m and 12.95÷14.45 m contain mixtures of interlaying muds and peats and are denoted as OH/Pt.

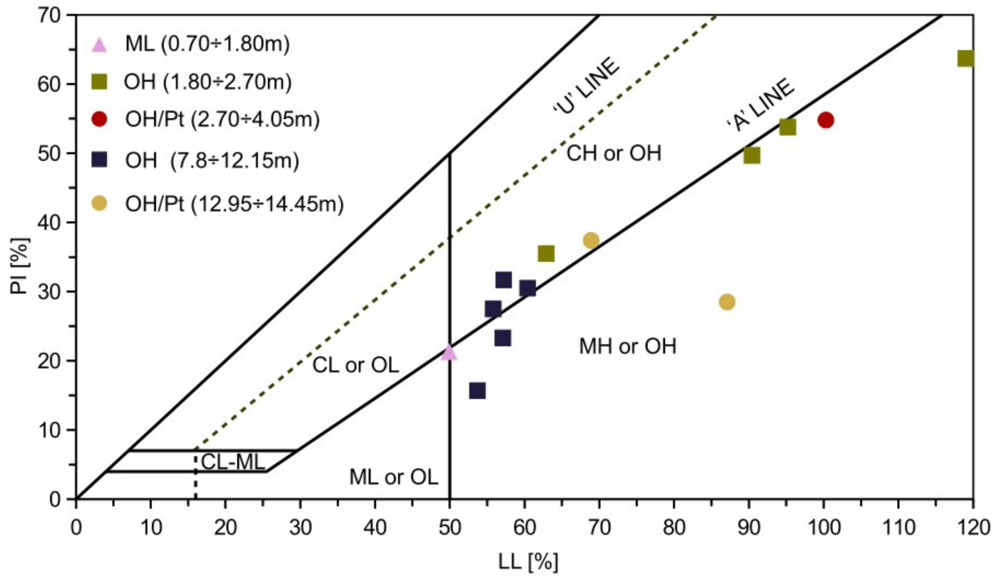

Figure 5. Classification of layers according to ASTM D2487 (2017). [no colour]

Soil classifications based on the Robertson (1990) and Schneider et al. (2008) approaches are presented in Figure 6. Only Figure 6a classifies ML correctly as a mixture of silt and clay, while Figure 6b classifies it as having a high sand content, which was not confirmed in the PSD, see Table 1. As can be seen in Figure 6a, Robertson's chart classifies peat and organic soils as clay, which is only partially correct for the organic silty clay from

1.8 to 2.7 m (Table 1). Moreover, the muds (OH soils) with low clay fraction at this depth range and the soils with high silt fraction content in the 7.8÷12.15 m layer are positioned close to one another in the diagram. The SBTn for Jazowa site according to Robertson (2016) is shown in Figure 6b. Most of the examined soils are clay-like contractive, which was confirmed with consolidated drained triaxial tests (Fig 13). The muds at shallow depths (0.7÷4.05 m) are, however, classified as dilative.

The results for Jazowa soft soils in Schneider's classification are presented in Figure 7. In Schneider's $\Delta u_2/\sigma'_{v0}$ nomogram (Fig.7a), muds are classified as silts and silty clays, the peats and ML soils from shallow depths are denoted as sands. In the $B_q = \Delta u_2/q_n$ nomogram (Fig. 7b), ML soils from a shallow depth are classified as having a high sand content, which was not confirmed in the PSD, see Table 1. Here, muds are classified as silts and silty clays. However, peats are denoted as sand mixtures or clays but not as organic soils.

The results of DMT-based classification (Marchetti and Crapps, 1981) are presented in Figure 8. Similarly to Robertson's classification, the organic clay layer at 0.7÷1.8 m is classified as silty sand/sandy silt. The muds are classified as clayey silts, silty clays or clays, and peats are denoted as silty clays. The dilatometer modulus E_D of the considered muds and peats is considerably higher than the upper bound proposed for such soils according to the Marchetti and Craps diagram.

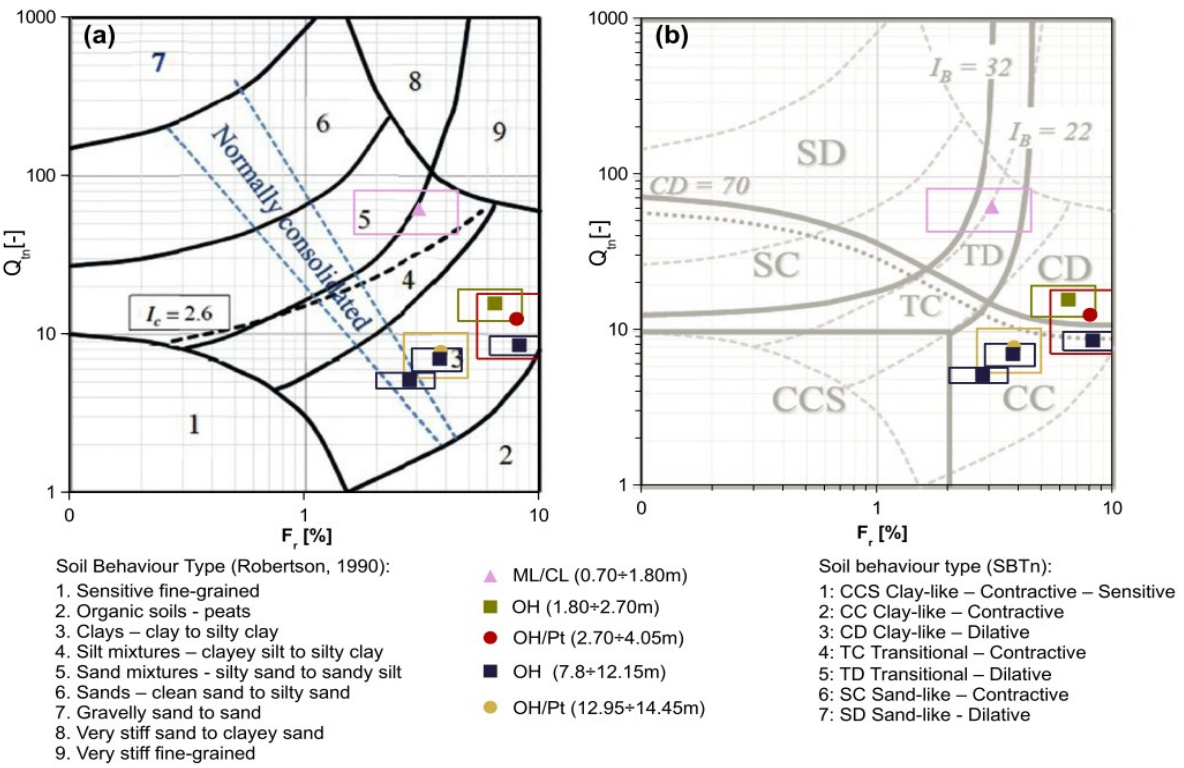

Figure 6. CPTU based classification (error boxes based on standard deviation) for Jazowa soft soil deposits: (a) Robertson's nomogram (1990; 2009) and (b) SBTn charts based on Q_{tn}-F_r (Robertson, 2016). [no colour]

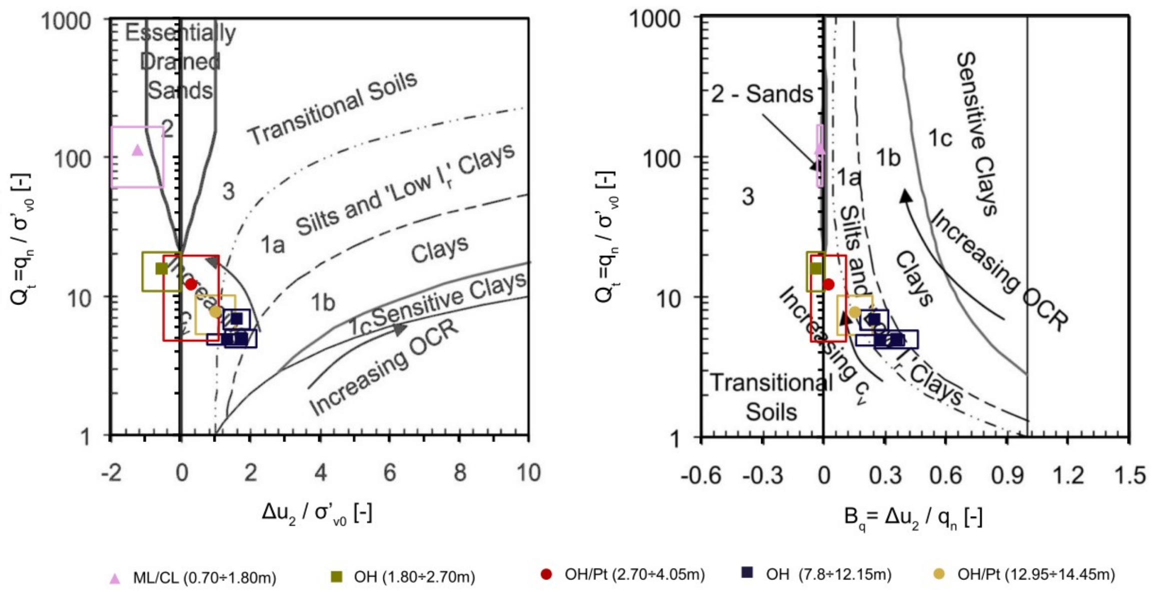

Figure 7. Schneider et al. CPTU classification (2008) (error boxes based on standard deviation). [no colour]

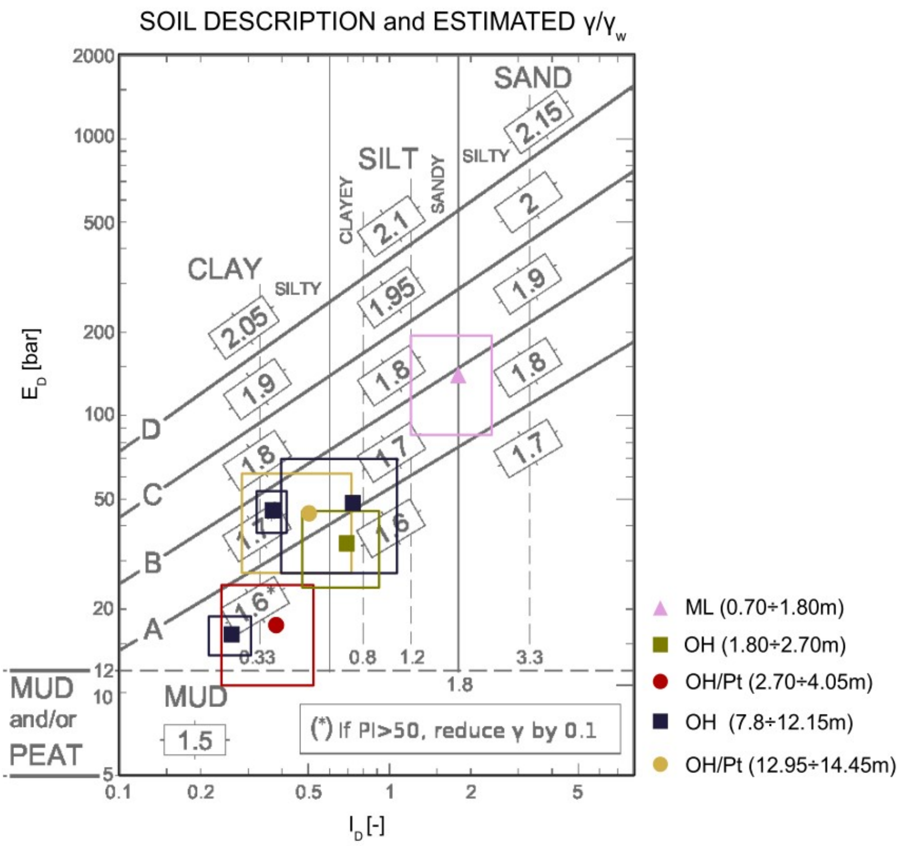

Figure 8. DMT-based classification (Marchetti and Crapps, 1981) for Jazowa soft soil deposit (error boxes based on SD). [no colour]

3.3. Discussion

One should keep in mind that CPTU and DMT soil classifications describe soil type behaviour and are not directly related to the soil granulometry. The difficulties in the soil type identification according to CPTU-based classifications were already described by Młynarek et al. (1997) and Tumay et al. (2011). One can notice that the CPTU-based classification has no relation to the USCS in the case of Jazowa soft soils. Robertson's graph (1990) better classifies shallow soil layers (up to 2.7 m), whereas the Schneider et al. nomogram (2008) results better match the soil behaviour type for deeper muds. In all cases, muds are not recognized as organic soils but as silts and silty clays. As noticed by Młynarek et al. (2008), the CPTU system illustrates well the effect of preconsolidation. Slightly overconsolidated muds from the first layer are generally situated above the NC zone on the Robertson (1990) chart (Figure 6a). DMT classification slightly better reflects the soft soil type than the CPTU classifications. However, peats are improperly identified here as silty clays. Similar discrepancies between CPTU and DMT classifications in the case of organic soils were already found by Młynarek et al. (2008), who argued that organic soils require a separate classification system. For instance, an attempt of new DMT soil classification including organic soils was proposed by Rabarijoely (2018). Consequently, only a combined approach, including local geological maps, drillings, CPTU or DMT classifications and engineering judgement, will allow for appropriate soil type recognition in the considered organic soils.

4. Compressibility

4.1. Methods

The compressibility of soft soil deposits was determined with triaxial apparatus using the isotropic consolidation procedure (ASTM D4767, 2011; ASTM D7181, 2011) on specimens from approximately 4 m and 10 m. An almost fully saturated soil sample (Skempton's parameter $B \geq 0.95$) was subjected to isotropic multi-stage consolidation (e.g., ASTM D7181, 2011) up to 800 kPa. The preconsolidation pressure σ'_p , swelling index (κ) and compression index (λ) were evaluated.

The coefficients of the consolidation (c_v), recompression (C_r), compression (C_c) and swelling (C_s) indices and the vertical preconsolidation pressure σ'_p were determined on the basis of eleven oedometer tests (ASTM D2435, 2011). The end of primary consolidation (EOP) was determined using the Casagrande method or rectangular hyperbola method (Sridharan et al., 1987). An extensive study concerning the application of different methods for the determination of preconsolidation pressure in oedometer tests was presented by Grozic et al. (2003). Determination of the vertical preconsolidation pressure (σ'_p) is not always simple, especially in disturbed low plastic overconsolidated clays, and may include subjective interpretations (Grozic et al., 2005). As the interpretation of consolidation curves requires experience, preconsolidation pressure can be difficult to define, especially when graphical methods are used. That is why Paniagua et al. (2016) recommend to evaluate this parameter using at least three different methods. In this study, the preconsolidation pressure is interpreted using five methods: Casagrande (1936), Onitsuka et al. (1995); Becker et al. (1987); Boone (2010) and Pacheco-Silva's (1970). Casagrande (1936) is a traditional semi-logarithmic method, which is probably applicable only for clays with an overconsolidation ratio between 1 to 3 and 1 to 4 (Lacasse et al. 2008). The Onitsuka et al. (1995) bilogarithmic and Becker et al. (1987) work method were recommended by Grozic et al. (2003) in a study concerning overconsolidated glaciomarine clays of low plasticity. The Boone (2010) approach uses a bilinear approximation of the compressibility curve in the e-log(σ'_{v0}) plane. It is based on a simple slope-intercept mechanism and does not require subjective or graphical interpretations. Pacheco Silva's (1970) method, widely used in Brazil, is considered to be less influenced by subjective interpretation (Grozic et al., 2005).

The overconsolidation ratio (*OCR)* was also estimated with DMT results (Marchetti, 1980) for I_D<1.2:

$$OCR = (0.5 \times K_D)^{1.56} \tag{12}$$

as well with CPTU-based preconsolidation pressure assessment (Chen and Mayne, 1996):

$$\sigma'_p / p_a = 0.86 \times \left(\frac{q_t - \sigma_{v0}}{p_a} \right)^{0.93} \times PI^{-0.28} \tag{13}$$

4.2. Results and interpretation

The principal consolidation parameters for the Jazowa site are summarized in Table 2. The presented values are typical for similar soils. For instance, the c_v for San Francisco Bay mud is between 0.6÷1.2 m^2/year (Lee et al., 1983), while for Jazowa organic silty clay it is between 0.61 and 0.89 m^2/year. The c_v = 0.34÷0.62 m^2/year for Jazowa organic silt is close to the values given by Lowe et al. (1964), c_v = 0.6÷3.0 m^2/year. The c_v = 0.21÷2.66 m^2/year for peats are within the range reported by Mesri and Ajlouni (2007) for fibrous peats (c_v = 0.8÷8.1 m^2/year). The C_c range (0.249÷0.638) for Jazowa OH soils is also typical for this kind of soil. For instance, O'Kelly (2006) reports C_c = 0.29÷1.4 for Irish silts and clays. The derived C_c/C_r ratio for muds in Jazowa varies generally between 7 and 15, which is slightly higher than that reported by Das (2013) for natural inorganic clays of low to medium sensitivity. In natural organic sensitive clays, this ratio may exceed even 15 (e.g, Koskinen and Karstunen, 2004; Mataic et al., 2015).

Table 2. Consolidation parameters obtained from oedometer and TX tests

Soil type	*Sampling depth* *[m]*	σ'_{v0} *[kPa]*	$c_v^{(1)}$ *[m²/year]*	C_r *[-]*	C_c *[-]*	C_s *[-]*	λ *[-]*	κ *[-]*
ML/CL	1.3÷1.6	22.6÷27.8	2.91	0.016	0.249	0.036	-	-
ML/OH	1.85	30.6	1.56	0.044	0.392	0.025	-	-
OH	2.45	33.2	0.61	0.044	0.517	0.030	-	-
OH	2.3÷2.6	32.5÷33.8	0.83	0.056	0.638	0.084	-	-
OH	3.2÷4.05	36.7÷40.5	0.89	0.085	0.719	0.146	0.227	0.042
Pt	~4	~40.2	2.66	0.180	1.585	0.104	-	-
OH	8.1÷8.9	73.9÷78.7	0.54	0.029	0.286	0.018	-	-
OH	8.65	77.3	0.62	0.062	0.403	0.019	-	-
OH	9.5÷10.5	82.3÷89.5	-	-	-	-	0.194	0.038
OH	11.1÷11.9	93.1÷97.8	0.34	0.012	0.465	0.025	-	-
Pt	~14	~108.4	0.21	0.059	2.249	0.164	-	-

(1)= values for in-situ stresses

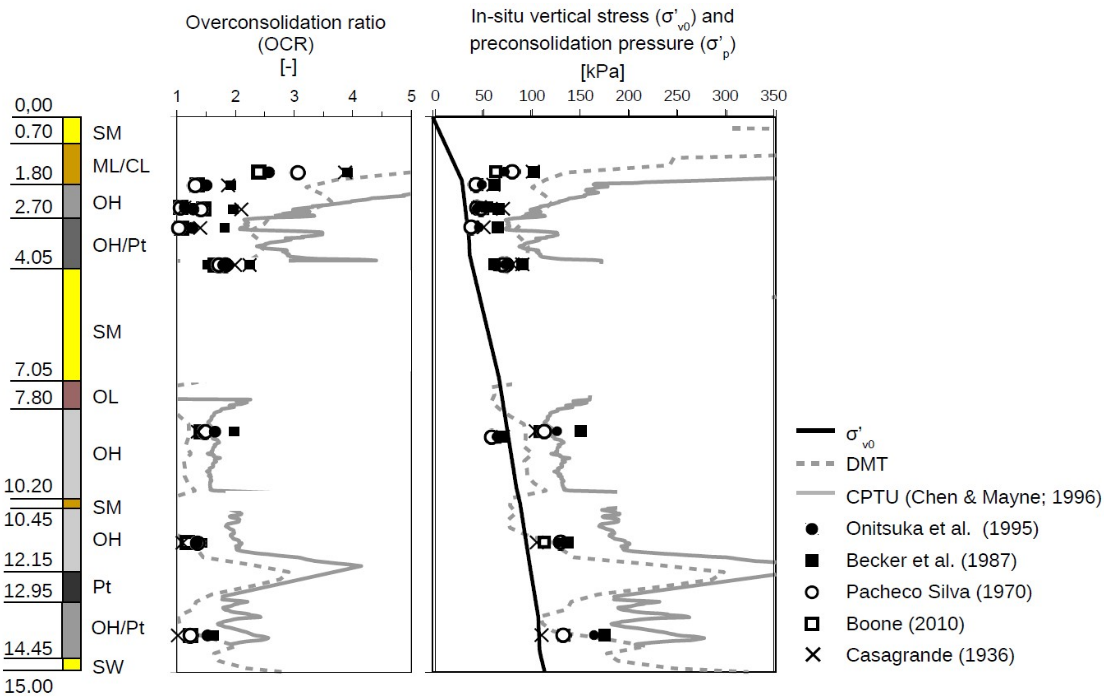

Figure 9. *OCR* profile for Jazowa site. [no colour]

The distribution of *OCR* with depth is presented in Figure 9. According to the oedometer test results, the soil is slightly overconsolidated in the upper layers and practically normally consolidated at larger depths. The *OCR* interpreted from oedometer tests supports the geological history of the Vistula Marshlands, where both soft soil deposits are normally consolidated or slightly overconsolidated sediments. The *OCR* distribution obtained from the DMT-based estimation overlaps the results from the oedometer tests for deeper layers, whereas both the Chen and Mayne (1996) proposal and DMT estimations surpass the results at shallow depths. The *OCR* values interpreted using the five methods based on oedometer tests are fairly consistent. Pacheco Silva's and Casagrande's methods generally give the smallest *OCR* values, whereas Becker's method gives the upper bound of this ratio.

4.3. Discussion

Different correlations between C_c and physical properties of soft soils were examined and the most promising relations were presented. For the Jazowa site, the relation between C_c and initial dry density ρ_{d0} (see Figure 10a) is:

$$C_c = 0.4682 \times \rho_{d0}^{-1.2601} \tag{14}$$

The relation between C_c and water content w_c (see Figure 10b) is:

$$C_c = 0.0075 \times w_c \tag{15}$$

These equations allow for a rough C_c estimate at the site when only a limited geotechnical investigation is carried out. The coefficient of 0.0075 in Eq. (15) is lower than that proposed by Bowles (1984) for organic silts and clays ($C_c = 0.0115 w_c$).

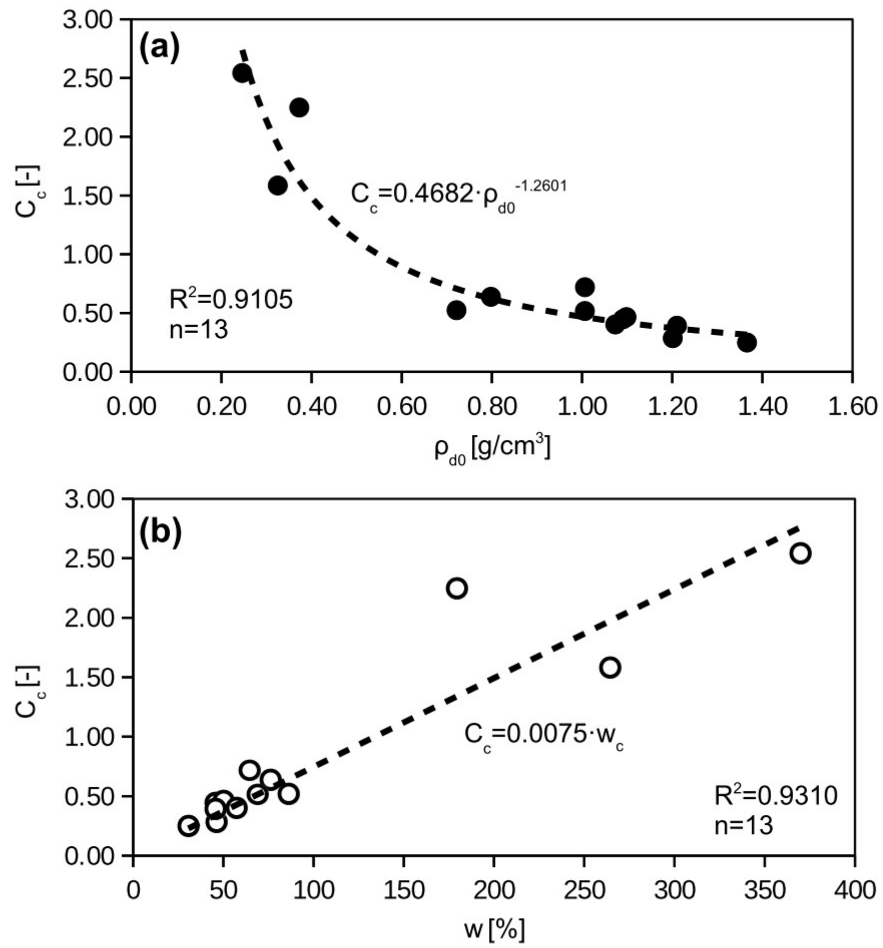

Figure 10. The relationships: (a) C_c versus ρ_{d0} and (b) C_c versus w_c.

5. Shear strength parameters

5.1. Undrained shear strength

5.1.1. Methods

The undrained shear strength profiles were obtained from field tests, including CPTU, DMT and FVT, as described below:

1. The CPT-based c_u estimation was calculated using the following empirical formula:

$$c_u = (q_t - \sigma_{v0}) / N_{kt} \tag{16}$$

where: c_u = undrained shear strength of soil, σ_{v0} = vertical in-situ total stress, N_{kt} = cone factor. For the analyzed deltaic soft soils, a local correlation (with FVT as a reference) linking the cone factor with the friction ratio was used (Bałachowski et al., 2018):

$$N_{kt} = 1.242 \times F_r + 7.803 \tag{17}$$

For the considered soils, N_{kt} varies between 11 and 18 according to Eq. (17). This is similar to the values for normally consolidated marine clays obtained by Lunne and Kleven (1981), who propose an average N_{kt} value of 15 with reference to FVT. The recommended values of the N_{kt} factor depends on the shearing mode. Aas et al. (1986) suggest that the correlation between the cone factor and the average laboratory undrained shear strength obtained from triaxial compression, triaxial extension and direct simple shear tests ranges from 8 to 16. Assuming average shearing mode, an N_{kt} value of 14 can be assumed for soft soils (Robertson, 2009).

2. The DMT c_u estimation was calculated using Lechowicz (1997) formula for soft organic soil deposits in Poland, which is a modified version of Marchetti (1980):

$$\frac{c_u}{\sigma'_{v0}} = S \times (0.45 \times K_D)^{1.20} \tag{18}$$

where: S = normalized undrained shear strength; S is equal to 0.4 for normally consolidated organic soils (Lechowicz, 1997).

3. FVT c_u measurements are affected by several factors, including rate effects and anisotropy (e.g., Bjerrum, 1972; Chandler, 1988). Therefore, the measured undrained shear strength was corrected with the factor μ_v (Chandler, 1988):

$$c_u = \mu_v c_{u-FV} \tag{19}$$

where $c_{u\text{-}FV}$ = undrained shear strength measured directly with FVT, μ_v = correction factor. For muds, the μ_v was defined as:

$$\mu_v = 1.05 - b \times (PI)^{0.5} \tag{20}$$

where: b = time to failure coefficient equal to 0.045, as suggested by ASTM 2573 (2015), which corresponds to time to failure equal 10 000 minutes. For peat μ_v = 0.5 (e.g., Gołębiewska, 1983).

5.1.2. Results

The undrained shear strength profiles obtained from the CPT, DMT and FVT tests are consistent (Figure 11). The c_u values are generally constant in the upper lightly overconsolidated layer and increase with depth in the lower normally consolidated deposit.

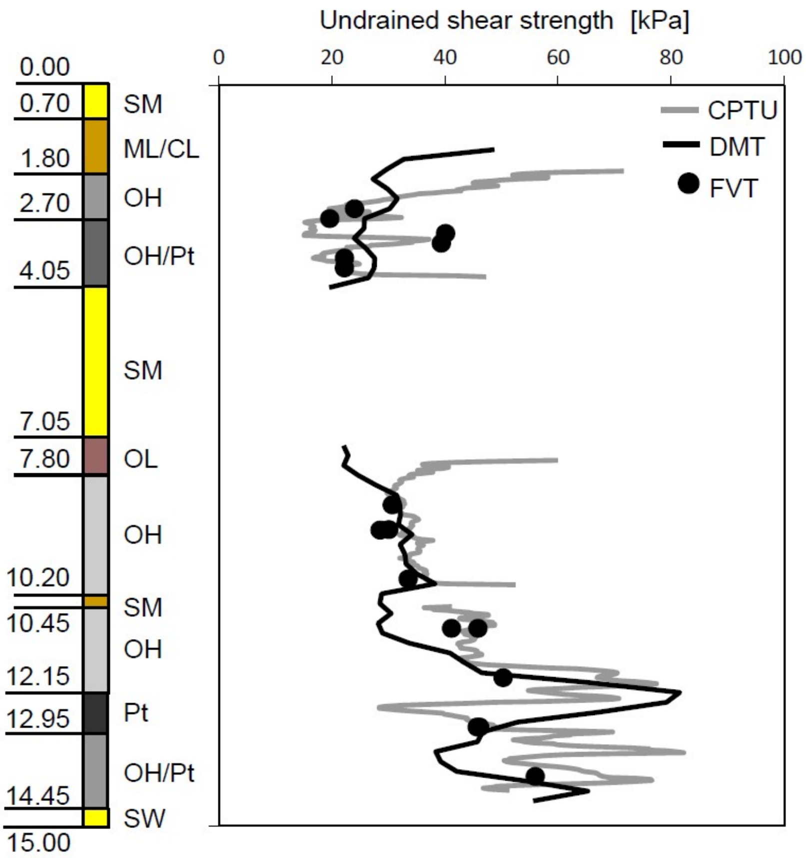

Figure 11. Undrained shear strength profiles

5.2. Frictional strength parameters

5.2.1. Methods

A CIU triaxial compression test (ASTM D4767, 2011) was conducted on mud samples taken from 3.2÷4.0 m and 9.5÷10.5 m. Specimens were sheared at a rate of 0.011 mm/min. Three CIU tests on mud were performed in both samples. The CID triaxial compression test (ASTM D7181, 2011) was conducted only on specimens taken from 8.1÷8.9 m (organic silt) with a shearing rate of 0.002 mm/min. All the tests were carried out using a standard triaxial device and the maximum deviatoric stress criterion was used to determine the angle of internal friction. The angle of internal friction φ' is related to the stress ratio M (slope of failure surface in the p'-q plane, where p' = mean effective stress and q = deviatoric stress) as:

$$M = \frac{6 \cdot \sin\varphi'}{3 - \sin\varphi'} \tag{21}$$

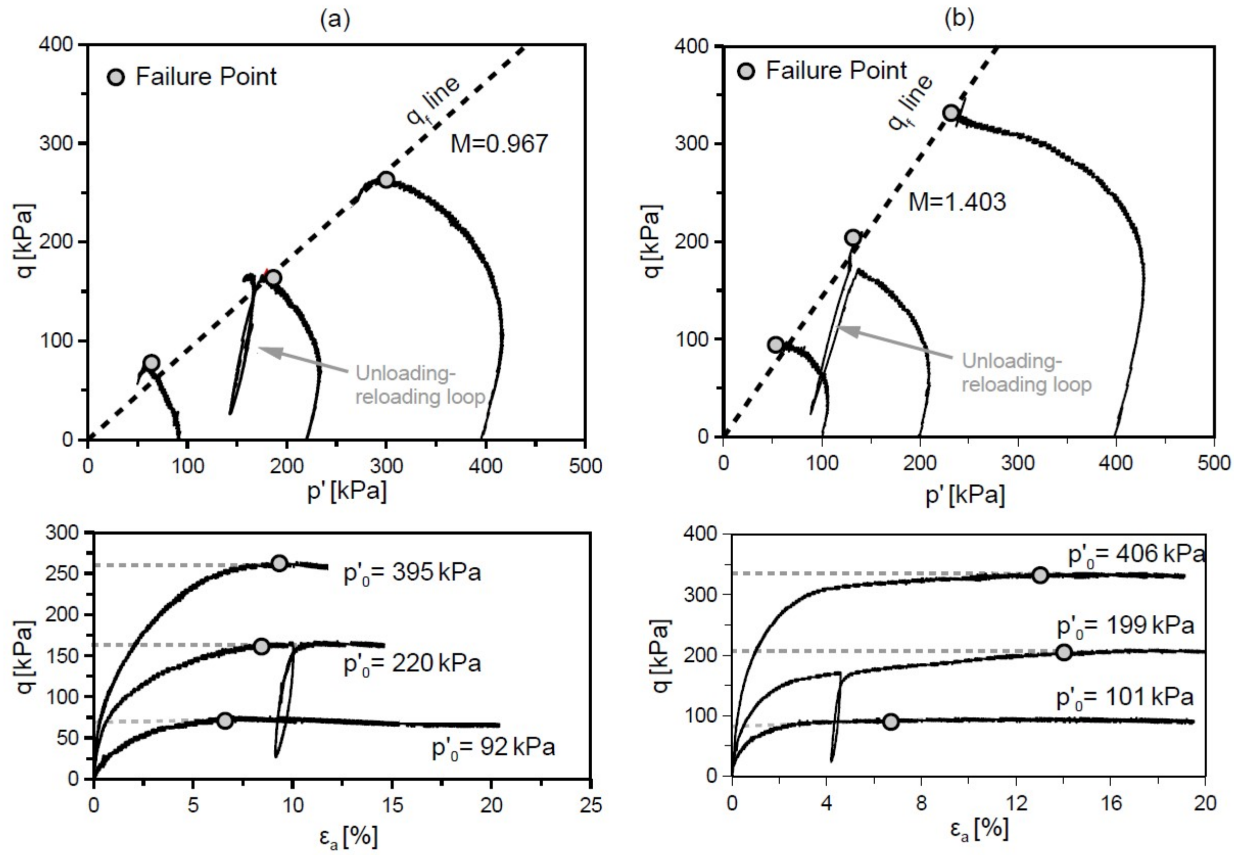

Figure 12. The CIU test results for (a) organic silty clay and (b) organic silt.

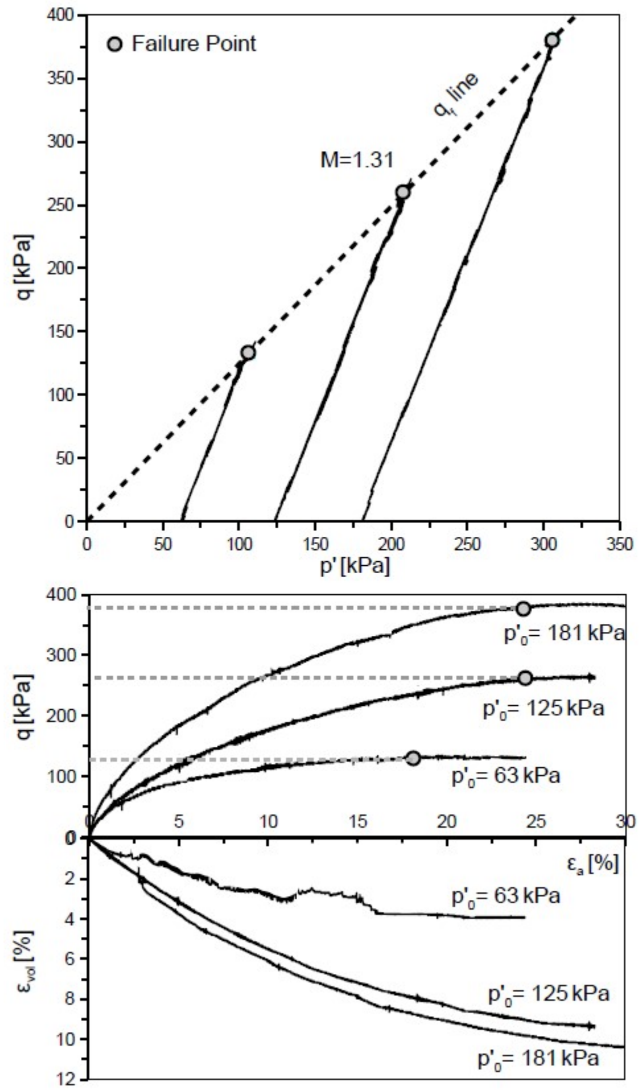

Figure 13. The CID test results for organic silt.

5.2.2. Results and interpretation

The results of the CIU triaxial compression tests are presented in Figure 12 in terms of deviatoric stress (q) versus axial strain (ε_a) and stress paths in the p'-q plane. Full strength mobilization in the organic silty clay required an axial strain of approximately 10%. The elastic modulus E_{50} (secant modulus corresponding to $q = 0.5q_f$) normalized with respect to undrained shear strength ($c_u = q_f/2$, where q_f is deviatoric stress at failure) ranges between 76 and 102. The ratio between the unloading-reloading modulus E_{ur} and the E_{50} is equal to 1.74. The $M = 0.967$ corresponds to an angle of internal friction of 24.6°. The maximum strength of the organic silt (Figure 12b) is also mobilized at large values of axial strain. Stress ratio M = 1.403 results in an angle of internal friction equal to 34.6°. The E_{50}/c_u ratio for organic silt ranges between 180 and 234, while E_{ur}/E_{50} = 2.17. The CID triaxial compression tests on organic silt (Figure 13) give almost the same failure envelope as the one obtained from the CIU tests, with M = 1.31 and φ' = 32.6°. The response of specimens during shearing is clearly contractive (Figure 13) and confirms the results of the updated CPTU classification according to Robertson (2016).

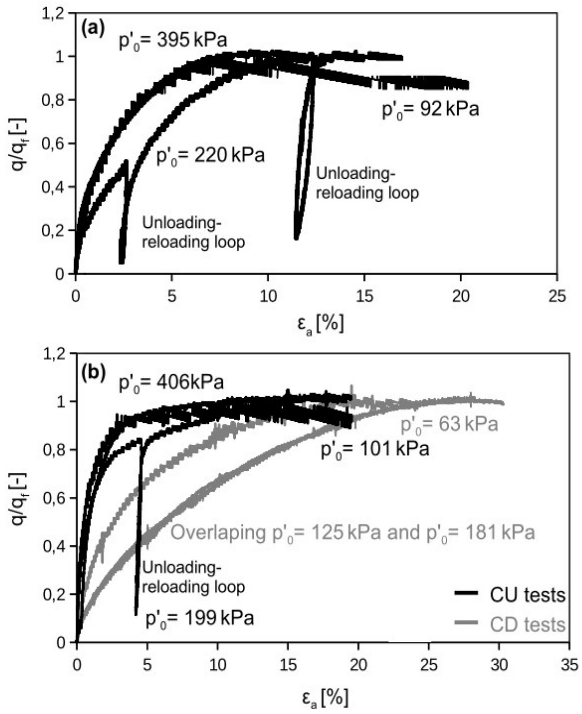

Figure 14. Mobilization of deviatoric stress ratio with axial strains; (a) organic silty clay and (b) organic silt.

5.3. Discussion

The organic soft soil in Jazowa has similar frictional parameters to soils from other sites. For instance, φ' = 28° for alluvial clayey silt is reported by Lambson et al. (1993), and φ' = 23÷57° for Juturnaiba organic clay is given by Coutinho and Lacerda (1989). However, the angle of internal friction in soft deltaic soils is mobilized at an axial strain larger than 10%, see Figure 14. The reported E_u/c_u ratios are slightly higher than those given for similar soils. For instance, for Osaka Bay mud, $E_u/c_u \approx 40$ (Watabe et al., 2002). The organic silt behaviour in the CID triaxial compression test was contractive, which confirms the Robertson (2016) CPTU classification. This classification may, therefore, be considered a practical tool for qualitative descriptions of SBTn and comparison between drained and undrained shear strength.

6. Conclusions

A comprehensive geotechnical characterization of Vistula Marshlands deltaic soft soil from the Jazowa test site has been presented in this paper. The investigation concerned organic silty clay, organic silt, and peat interbeddings. The wide scope of the combined field and laboratory tests allows for the following conclusions to be drawn:

1. In case of organic soils, the CPTU and DMT classification test results do not match the USCS soil types. The discrepancies are smaller in the case of the Schneider et al. (2008) charts. It is worth noting that none of the in-situ-based classifications identifies peat as an organic soil.
2. The *OCR* values derived from oedometer tests are consistent with DMT and CPTU estimations. Five interpretation methods were applied to oedometer test results to determine preconsolidation stress. Pacheco Silva (1970) generally gives the lowest values of OCR, whereas the Becker et al. (1987) method gives the highest values.
3. Local empirical correlations between C_c and basic physical parameters (water content and bulk density) are proposed.
4. Similar c_u values are obtained using CPT, DMT and FVT soundings. The soil sensitivity based on FVT varies from low to moderate, i.e. 1.5 to 4.5.

5. The effective angle of internal friction equals 24.6° (CIU) for organic silty clay and is between 32.6° (CIU) and 34.6° (CID) for organic silt. The maximum shear strength in the considered organic soils is mobilized at an axial strain higher than 10%.
6. The Robertson's SBTn (2016) updated chart for dilative/contractive soil behaviour type can be used for soft soils in the Vistula Marshlands.

The above described research will improve geotechnical design in the Vistula Marshland area. It presents a wide range of geotechnical properties for deltaic soft soil deposits. The geotechnical characterization can be employed as reference data for foundation design and soil improvement in the soft organic soils of northern Poland.

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