

PAPER • OPEN ACCESS

The Interpretation of CPTu, PMT, SPT and Cross-Hole Tests in Stiff Clay

To cite this article: Boris Kavur *et al* 2019 *IOP Conf. Ser.: Earth Environ. Sci.* **221** 012009

View the [article online](#) for updates and enhancements.

Recent citations

- [Microcrystalline Cellulose from Lignocellulosic Material Activated by Steam Explosion Treatment and Mathematical Modeling of the Processes Accompanying its Preparation](#)
D.B. Prosvirnikov *et al*
- [A. A. Fomin *et al*](#)
- [V. G. Gusev *et al*](#)



IOP | ebooks™

Bringing you innovative digital publishing with leading voices to create your essential collection of books in STEM research.

Start exploring the collection - download the first chapter of every title for free.

The Interpretation of CPTu, PMT, SPT and Cross-Hole Tests in Stiff Clay

Boris Kavur¹, Filip Dodigovic¹, Jasmin Jug¹, Stjepan Strelec¹

¹ University of Zagreb, Faculty of Geotechnical Engineering, Hallerovaaleja 7, 42000 Varaždin, Croatia

boris.kavur@gmail.com

Abstract. The paper presents the comparison of results of CPTu, PMT, SPT and CH (cross-hole) tests performed in stiff, heavily overconsolidated clay at a test location in Zagreb. This clay exhibits very high pore pressures generated during CPTu penetrations despite significant overconsolidation and relatively high values of shear strength and stiffness in its undisturbed, natural state. The clay is classified using the CPTu and CH test results in recently published soil classification charts based on the soil behaviour in (S)CPTu penetrations. The results of classifications indicate that the tested clay probably has a pronounced microstructure that is likely a consequence of the geological processes of cementation and aging. Effects of cementation and aging are manifested on very high shear wave velocities, as measured in cross-hole seismic tests, and consequently very high values of the small-strain modulus. The collapse of the soil structure at higher shear displacements is resulting in volume contractions and softening behaviour after reaching the peak shear strength. The study shows a relatively good agreement between CPTu, PMT and SPT parameters and adequate correlations have been established between the cone resistance (CPTu), limit pressure (PMT) and number of blows (SPT). The in situ state and parameters of strength, stiffness and compressibility of the clay tested are estimated based on the in situ test parameters as well as comparative laboratory test results obtained on undisturbed soil samples. Existing empirical correlations developed for the interpretation of CPTu test results are mainly from young and uncemented soils without microstructure and therefore, the results shown here are important for better understanding of the structured soil behaviour characteristics (stiffness, strength, and compressibility).

1. Introduction

The aim of this paper is to present results obtained by various in situ tests conducted in stiff, heavily overconsolidated and probably structured clay at a test location in Zagreb. The central part of the city is located on the soil in which upper layers are formed of gravel that reaches a depth between 10-20 m below ground level. A thin layer of clay and/or made ground usually covers the gravel layer. Below the gravel layer there is a layer of stiff, highly overconsolidated clay extending to a greater depth (Figure 1). At the test site, the stiff clay appears at a depth of 17 m.

In this study, the Menard pressuremeter test (PMT), the standard penetration test (SPT) and the piezo-cone penetration test (CPTu) were conducted in a highly overconsolidated, stiff clay layer in similar, comparative soil conditions. Throughout the soil profile (figure 1), cross-hole (CH) seismic



velocity measurements were made between boreholes as well. In addition, soil samples were taken for laboratory testing.

The tested clay exhibits high-pressure generation during CPTu testing. In addition, it shows volume contractions at high shear deformations despite significant overconsolidation and relatively high values of shear strength and stiffness in its undisturbed, natural state.

The study shows a relatively good agreement between CPTu, PMT and SPT parameters that has enabled the establishment of correlations between the cone resistance (CPTu), limit pressure (PMT) and number of blows (SPT). The CPTu and CH test results are used in recently published soil behaviour type (SBT) classification charts in order to classify the clay tested and to explain its behaviour. The in situ state and parameters of strength, stiffness and compressibility of the clay tested are estimated based on the in situ test parameters as well as comparative laboratory test results obtained on the undisturbed soil samples.

2. Soil profile at the test site

The test site is located near the Jelačić Square in Zagreb. The soil profile at the test site is shown in Figure 1. It shows changes of the natural moisture content (w_o) together with the plastic (w_p) and liquid (w_L) limits of the stiff clay layer. The consistency index of this stiff clay ranges from 0.96 to 1.12. Figure 1 also shows the profile of SPT blow counts obtained in the B-100 borehole. In the neighbouring C-100 borehole located at a distance of 8m far from the B-100, a piezo-cone CPTu test was performed. These boreholes (B-100 and C-100) were then used for cross-hole seismic tests (CH). The results of shear wave velocity (v_s) measurements are shown in Figure 1. In the surface clay layer (above the gravel layer), the undrained shear strength was measured by field vane tests in the B-100. Undisturbed samples from the stiff clay layers in the B-100 were taken to determine undrained shear strengths in the laboratory. The undrained shear strength profile is shown in Figure 1 as well.

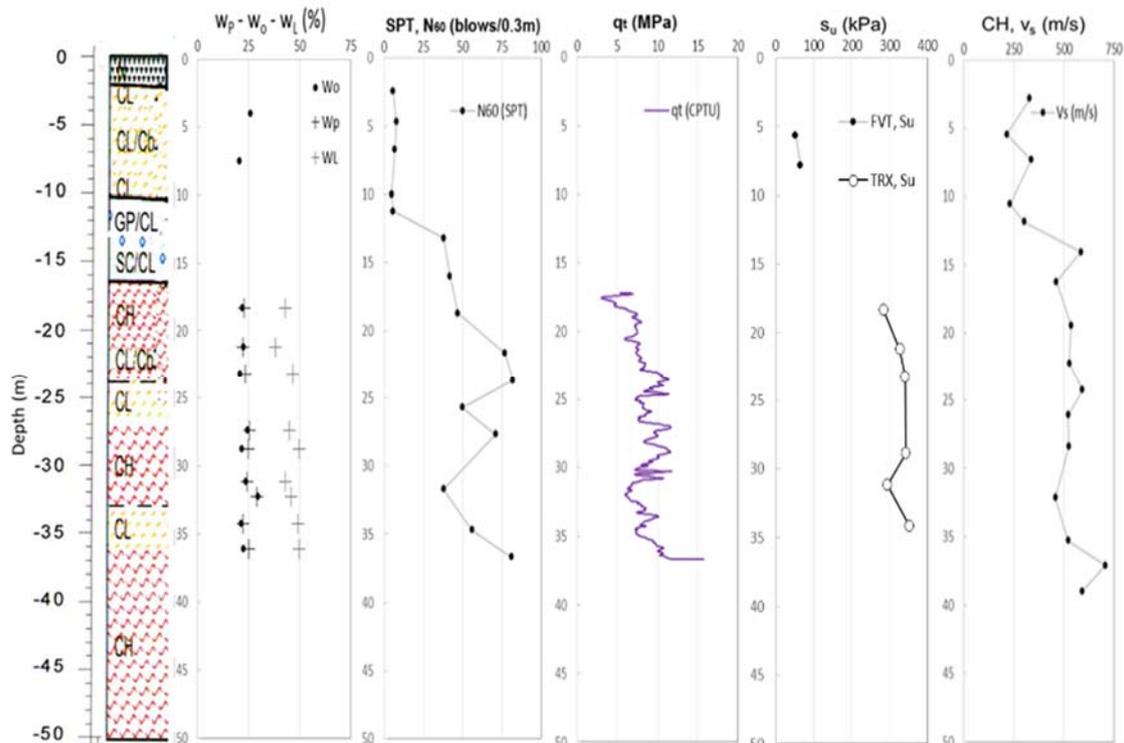


Figure 1. Soil profile at the test site near the Jelačić Square in Zagreb

3. In situ tests

3.1. Piezo-cone penetration test (CPTu)

The piezo-cone penetration test was carried out in the C-100 borehole. The penetration started from the pre-bored and cased hole-bottom at depth of 17.2 m below the ground level. The penetration was facilitated by means of the anchored pusher with 200 kN capacity. Due to the high friction, occurring along pushing rods the penetration had to be done in four steps down to the final depth of 36.7 m. The borehole had to be deepened in intervals of approximately 5 m. The CPTu probe of 10 cm² area was used together with the friction reducer of 15 cm².

Based on procedures suggested by Robertson [1] and [2], CPTu measured parameters (q_c , f_s , u_2) were reduced in order to obtain derived CPTu values of: the normalized cone resistance (Q_{tn}), normalized friction ratio (F_r) and normalized pore pressure ratio (B_q) (Figure 2). The tested soil was classified using different charts shown in Figures 3 and 4. Derived parameters are used for a soil classification based on the Robertson's soil behaviour classification charts ([1], [2] and [3]) (Figure 3) and Schneider et al. [4] Q_{tn} - U_2 chart (Figure 4) which distinguishes soils that dilate ($U_2 < 0$) or contract ($U_2 > 0$) during penetration. The U_2 parameter represents the ratio of the generated pore pressure (the increase of pore pressure above static value) and the effective vertical pressure of soil. Robertson suggested a new Q_{tn} - I_G chart (Figure 3, right) to identify soils with microstructure, where I_G is the small-strain rigidity index ($I_G = G_o / (q_t - \sigma_v)$) [3]. In this chart soils with microstructure are plotted on the right side of the $K_g^* = 330$ limit. K_g^* is a modified normalized small-strain rigidity index defined as:

$$K_g^* = (G_o / (q_t - \sigma_v)) (Q_{tn})^{0.75} \quad (1)$$

The most young and uncemented fine-grained and coarse-grained soils are plotted within $100 < K_g^* < 330$ limits that are shown in Figure 3 (right). According to Robertson [3] soils with $K_g^* > 330$ tend to have significant microstructure and the higher the value of K_g^* , the more microstructure is likely present. Hence, the soil with $K_g^* > 330$ can be classified as structured soil where traditional generalized CPT-based empirical correlations may have less reliability and where local modification may be needed [3].

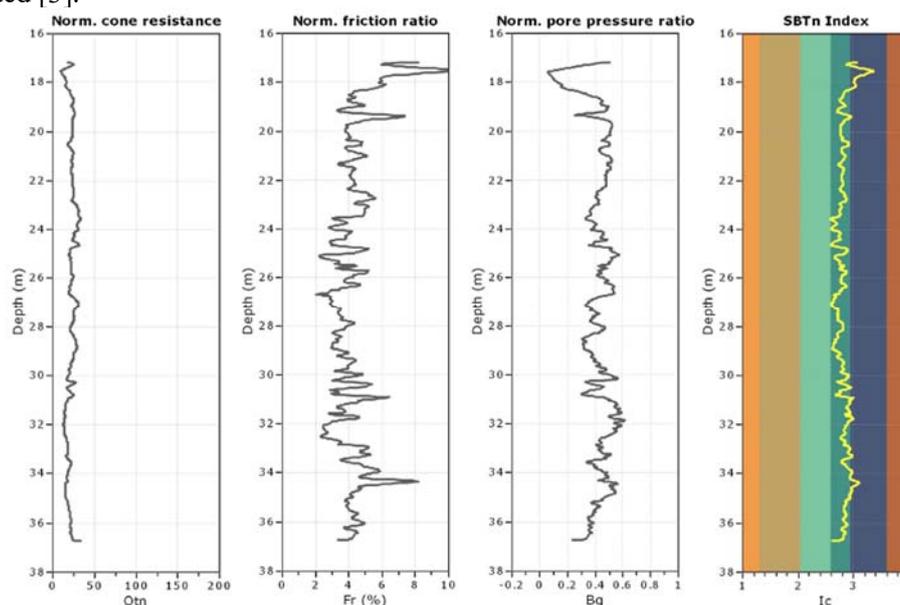


Figure 2. Normalized CPTu parameters and soil behaviour type index (I_c) determined according to procedures suggested by Robertson [2]

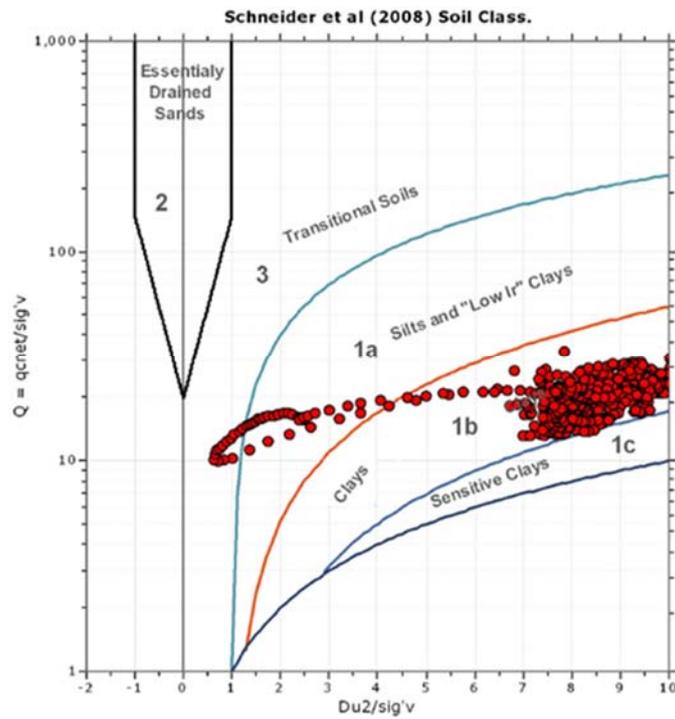


Figure 3. Overconsolidated stiff clay classified according to the $Q_{tn} - F_r$ chart [1], [2] and $Q_{tn} - I_G$ chart [3]

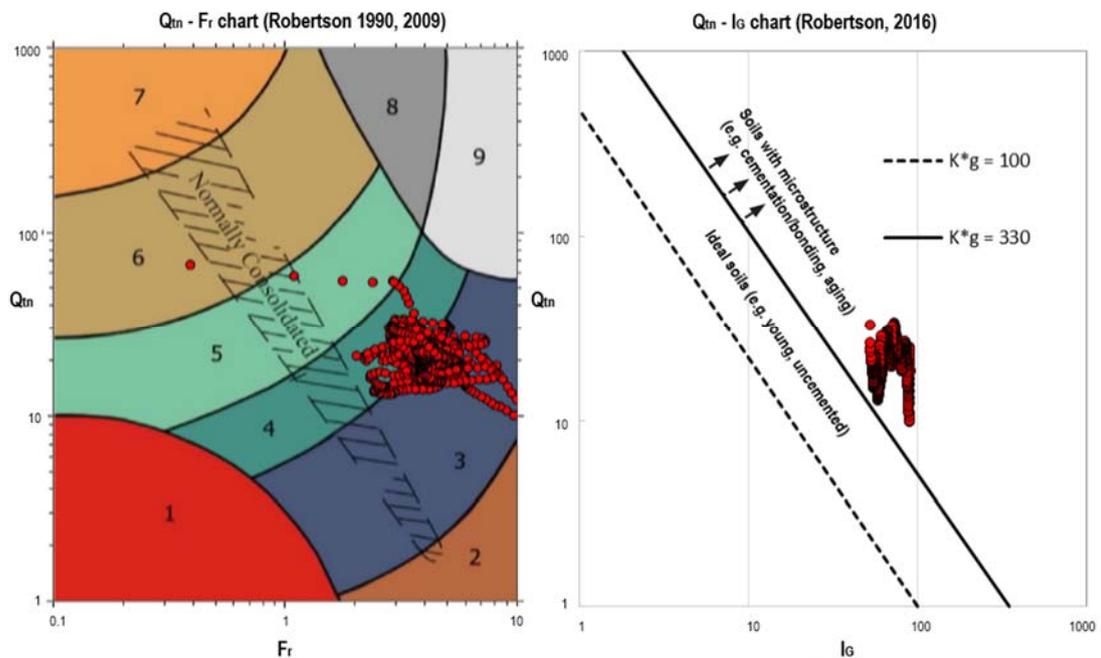


Figure 4. Overconsolidated stiff clay classified according to the $Q_{tn} - U_2$ chart [4]

Key classification parameters obtained in this study are shown in table 1.

Table 1. Key classification parameters obtained from CPTu and CH tests in stiff, heavily overconsolidated clay at the test site in Zagreb.

Soil description	Q_m	F_r (%)	U_2	I_G	K_g^*
OC clay	22 (10-34)	4,2 (2-11)	9,3 (0,6-12,7)	69 (53-91)	696 (403-1029)

Note: OC – overconsolidated; values shown are mean values, with ranges in parentheses.

3.2. Ménard pressuremeter test (PMT)

Ménard pressuremeter tests were carried out in the B-100 borehole that was located 8m away from the borehole C-100.

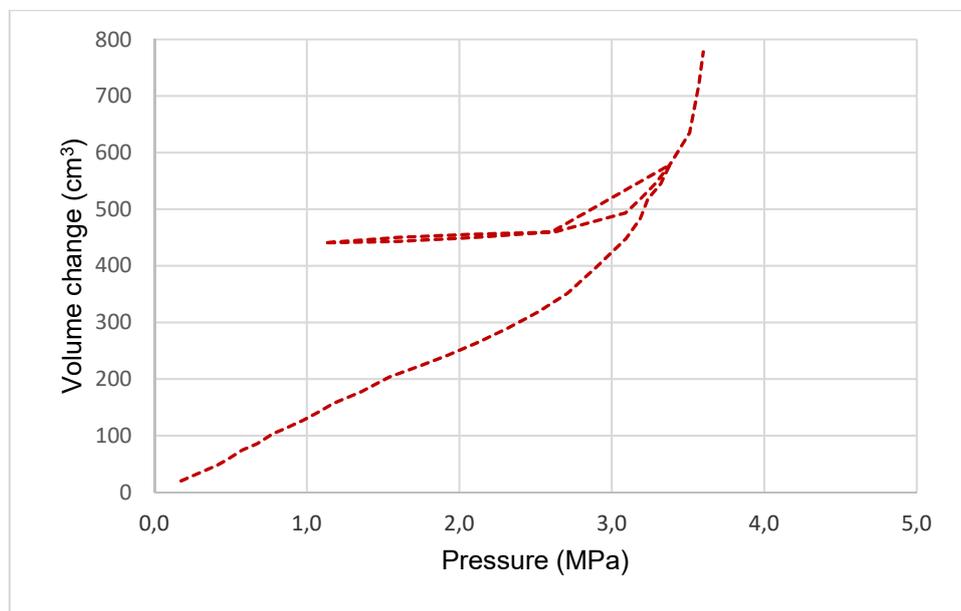


Figure 5. Typical pressuremeter curve (corrected) with one unload-reload cycle obtained in stiff overconsolidated clay in the borehole B-100 at depth of 26.7 m

Pressuremeter tests in B-100 were conducted and evaluated in accordance with the norm EN ISO 22476-4: 2012. Accordingly, the Ménard module (E_M), the limit pressure (p_{lm}) and the creep pressure (p_f) are evaluated from the corrected pressuremeter curve. These parameters should be used in the empirical design of foundations as described in Baguelin et al. [5].

Mair and Wood [6] emphasize that there are generally two approaches how to use PMT parameters, the so-called: french and english approaches. The french approach implies that evaluated parameters (E_M and p_{lm}) should be directly used in a design of foundations. These parameters are empirically linked to soil / foundation conditions and by means of them the bearing capacity and settlement of shallow and deep foundations [7] are estimated. This approach is related on the historical development of the Ménard pressuremeter in France and their view that this test should not be considered as a means of obtaining basic soil properties but rather as a method used strictly empirically [6]. The english approach, on the contrary, implies the use of PMT to assess the basic properties of soil (strength and stiffness), which are then used in usual methods of geotechnical design.

Even though at a first glance it appears that the soil is under compressive loading during pressuremeter test, the soil deforms as result of shear stresses ([6] and [7]). Due to this fact, PMT enables a shear modulus evaluation from test data. The relation between shear stresses in the soil and resulting shear strains is nonlinear. A potential use of the pressuremeter test to define G- γ degradation curve of soil stiffness with shear strains (γ) is suggested in Čolja and Kavur[8] and the flat dilatometer test for the same purpose is suggested in Ivandić et al. [9].

Table 2 shows average values of PMT parameters obtained in stiff, heavily overconsolidated clay at the test site in Zagreb.

Table 2. Average values of interpreted parameters obtained from PMTs in stiff, heavily overconsolidated clay at the test site in Zagreb

E_M (MPa)	p_{lm}^* (MPa)	p_f (MPa)	E_r (MPa)	E_r/E_M	E_o/E_M	E_o/E_r
32	4.5	2.7	220	7	48	7
where are: E_M – Menard modulus from first loading E_r – modulus of elasticity obtained from unload-reload cycle E_o – modulus of elasticity at small-strains (estimated from G_o value) p_{lm}^* - net limit pressure p_f – creep pressure						

3.3. Standard penetration test (SPT)

The standard penetration tests were carried out in the B-100 borehole alternately with PMTs and undisturbed soil sampling. SPT blow counts are corrected with reference to the relevant energy ratio (60%), i.e. as N_{60} values.

3.4. Cross-hole (CH) seismic test

The cross-hole seismic tests were carried out between the B-100 and C-100 boreholes in order to measure velocity of shear waves (v_s). Based on density measurements (ρ) on soil samples, the small-strain shear modulus is determined as:

$$G_o = \rho \cdot v_s^2 \quad (2)$$

Correlations between:

- total cone resistance and net limit pressure:

$$q_t = 2 \cdot p_{lm}^* \quad (3)$$

- net limit pressure and corrected SPT blows (N_{60}):

$$p_{lm}^* \text{ (MPa)} = \frac{N_{60} P_a}{1.4} \quad (4)$$

- total cone resistance and corrected number of SPT blows:

$$q_t = \frac{N_{60} \cdot P_a}{0.7} \quad (5)$$

where:

P_a – atmospheric pressure (equal to 0.1 MPa)

q_t – total cone resistance (in MPa).

The small-strain shear modulus (G_o) is estimated based on:

- normalized cone resistance:

$$G_o = 240 \cdot Q_m \cdot p_a \quad (6)$$

- net limit pressure:

$$G_o = 240 \cdot p_a \cdot \left(\frac{p_{lm}^* - \sigma_v}{\sigma'_v} \right) \quad (7)$$

- corrected number of SPT blows:

$$G_o = 240 \cdot p_a \cdot \left(\frac{(N_{60} \cdot p_a / 0.7) - \sigma_v}{\sigma'_v} \right) \quad (8)$$

where:

σ_v – total vertical stress (MPa)

σ'_v – effective vertical stress (MPa)

p_a – atmospheric pressure (equal to 0.1 MPa).

Calculated values of G_o (based on CH test) are shown in Figure 6.b together with values estimated from Q_m , p_{lm}^* and N_{60} based on correlations (6), (7) and (8) respectively.

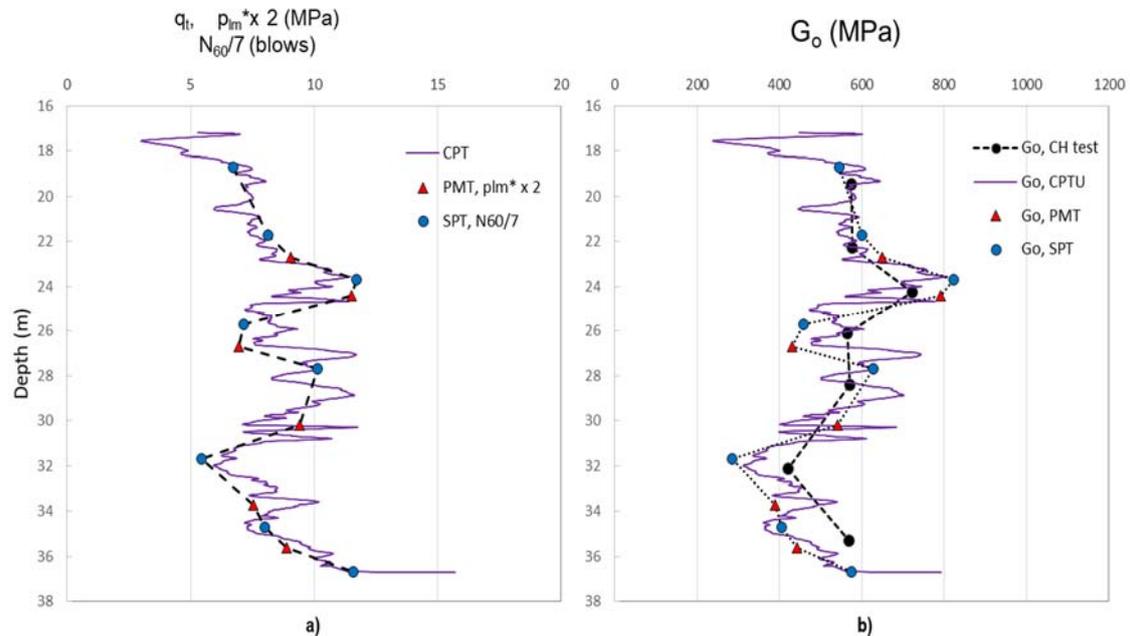


Figure 6. Comparison of CPTu, PMT, SPT and CH test parameters: a) total cone resistance (q_t), net limit pressure (p_{lm}^*) multiplied by 2, and corrected number of SPT blows (N_{60}) divided by 7; b) small-strain shear modulus (G_o) obtained from cross-hole (CH) test, and G_o values estimated from normalized cone resistance (CPTu), normalized net limit pressure (PMT), and normalized number of blows (SPT).

4. Estimation of geotechnical parameters and discussion of results

The in situ state for fine-grained soils is usually defined in terms of the overconsolidation ratio (OCR) that is defined as the ratio of the maximum past effective consolidation stress and the present effective

overburden stress. Such definition is appropriate for young, overconsolidated soils but for aged and/or cemented soils, as probably the clay in this case study, the OCR represents the ratio of the yield stress and the present effective overburden stress. The yield stress will depend on the direction and type of loading [2].

In this study, the OCR is estimated from the normalized cone resistance (Q_m) using the correlation suggested by Kulhawy and Mayne [10]:

$$OCR = k \cdot \left(\frac{q_t - \sigma_v}{\sigma'_v} \right) = k \cdot Q_m \quad (9)$$

where k is a preconsolidation cone factor.

Based on oedometer test results obtained on samples taken from the B-100 borehole, k value of 0.2 is taken. It should be noted that oedometer tests have to be taken deeply to the virgin compression line in order to get meaningful values of OCR in such very stiff, overconsolidated clays. The correlation (9) is used also to estimate the OCR from the net limit pressure (p_{lm}^*) and corrected SPT blows (N_{60}) in a way that they were expressed as q_t using correlations (3) and (5). Figure 7.a shows OCR profiles estimated from in situ tests together with oedometer test results.

The profile of the undrained shear strength is estimated from the total cone resistance according to:

$$s_u = \frac{q_t - \sigma_v}{N_{kt}} \quad (10)$$

where N_{kt} is a cone factor.

Based on triaxial CIU test results obtained on samples taken from the B-100 borehole, N_{kt} value of 20 is taken. The equation (10) is used also to estimate s_u from the net limit pressure and corrected SPT blows in a similar way as above for OCR . Figure 7.b shows s_u profiles estimated from in situ tests together with triaxial test results.

In order to enable estimations of consolidation settlements, the one-dimensional constrained tangent modulus, M , was estimated from the normalized cone resistance using the correlation:

$$M = \alpha_M (q_t - \sigma_v) = \alpha_M Q_m \quad (11)$$

where α_M is the constrained modulus cone factor. [2] suggested α_M value of 14 when $Q_m > 14$ (as it is here). Figure 7.c shows M profiles estimated from in situ tests together with oedometer test results. A reasonable agreement between laboratory test results and estimated M profiles is obtained using $\alpha_M = 14$ in this case.

With regard to the results obtained in this study, a good agreement between CPTu, PMT and SPT parameters is achieved as it can be seen in Figures 6 and 7. The piezo-cone penetration test proved as a valuable tool that can provide a reliable continuous soil profile. Unfortunately, the CPTu test was not a practical and cost-effective solution in such ground conditions due to the tremendous friction along

rods that prevents penetration. The CPTu test had to be done alternately with drilling in four steps from depth of approximately 17 to 37m. Nevertheless, the CPTu results provided the valuable and reliable basis for the interpretation of SPT, PMT and geotechnical parameters as well as soil behaviour. According to classification procedures performed using SBT (soil behaviour type) charts shown in Figures 3 and 4 clay layers tested belong to:

- zones 3 and 4 in chart Q_m-F_r that are classified as clay and silt-mixtures respectively,
- soils with microstructure in chart Q_m-I_G and
- 1b zone in chart Q_m-U_2 that indicates *clays* with contractive behaviour.

Such classification results correspond well with the observed soil behaviour in field and laboratory tests. The Q_m-I_G and Q_m-U_2 indicate that the clay probably has a pronounced microstructure that is likely a consequence of the geological processes of cementation and/or aging. The effect of cementation and aging is manifested on very high shear wave velocities, as measured in cross-hole seismic tests, and consequently very high values of the small-strain modulus (G_o).

The pressuremeter tests were also difficult to perform in such ground conditions. Briaud [11] stresses that making a quality borehole is the most important step for obtaining reliable test data. Drilling of a good PMT borehole is very different and almost opposite to drilling for soil sampling. The PMT borehole pocket should be made in one continuous run of the drill-bit without cleaning soil cuttings from the hole-bottom. After that, the PMT probe should be installed in the pocket as soon as possible and test started.

In PMT tests performed in this study there was a large difference between Menard modulus obtained in the first loading and the unload-reload modulus (Table 2), which is obviously a consequence of significant soil disturbance due to the process of installing the PMT probe, which cannot be avoided in such a test. Briaud [11] strongly discourages the use of the unload-reload modulus and stresses that E_r is not precisely defined since it depends on the strain amplitude over which the loop is performed. As such, E_r varies widely and it cannot be relied upon for standard calculations unless the strain amplitude and stress level have been selected to match the problem at hand [11].

In any case, the aim during the pressuremeter test should be to deform the soil as much as possible further from the pseudo-elastic phase in order to enable a hyperbolic extension of the PMT curve and to determine reliable values of the limit pressure (p_{lm}). Values of this parameter obtained in this study proved to be in a very good agreement with the total cone resistance (CPTu).

Corrected blow counts (N_{60}) provided by standard penetration tests in this study suit well with the total cone resistance (CPTu) as well. However, it is well known that this parameter can be severely affected by energy inefficiencies in the drop hammer system, as well as additional influences such as borehole diameter, hammer system, rod length and other factors [12].

The SPT is used quite often in geotechnical practice in Croatia (e.g. [13]) and therefore an attempt to establish a relationship between the CPTu and SPT parameters was done in this study. In addition, the SPT proved to be very suitable for conditions of the ground as they are in the test site.

With regard to laboratory tests performed, it should be noted that undisturbed soil samples taken in this study were not of the best but acceptable qualities according to the criteria set up by Laccasse et al. [14]. This criterion correlates to sample disturbance with its initial elastic stiffness based on results of oedometer tests. Processes of borehole drilling, sampling, sample extrusion, and trimming to form a specimen for testing, change the effective stress condition in the soil sample that causes some disturbance. Undrained shear strengths were obtained from four triaxial consolidated-undrained (CIU) and two unconsolidated-undrained (UU) tests. Due to the recompression and restoration of void ratio,

CIU tests are less affected by specimen disturbance than UU tests. Unfortunately, some sample disturbance is inevitable and therefore the profiles of undrained shear strength and overconsolidation ratio are probably underestimated.

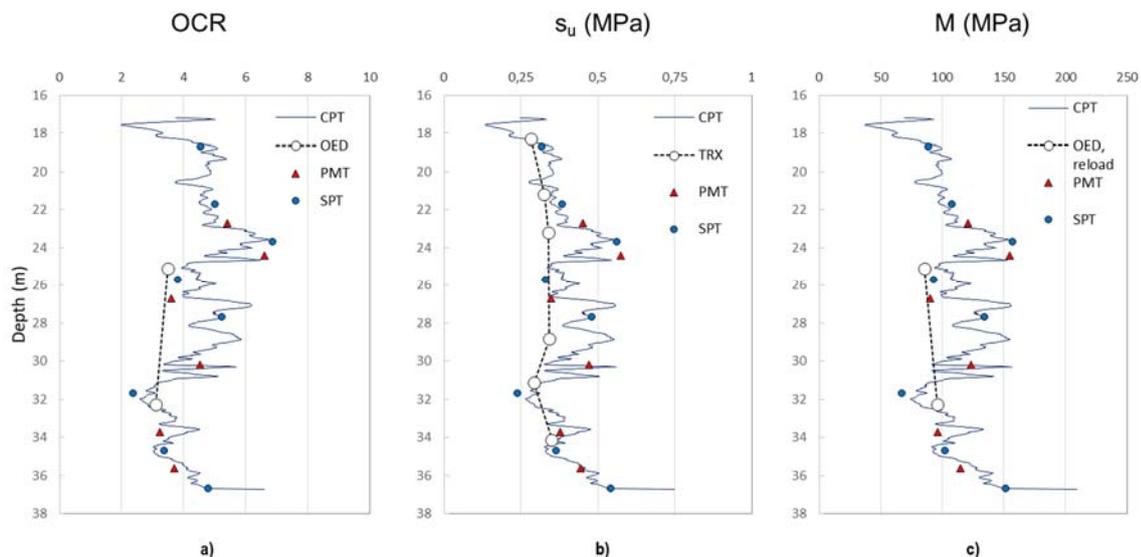


Figure 7. Interpreted geotechnical parameters: a) overconsolidation ratio (OCR); b) undrained shear strength (s_u) and c) constrained modulus (M)

5. Conclusions

In this study, the clay is classified based on measurements of cone resistance (q_c), sleeve friction (f_s), pore pressure (u_2), and velocity of shear waves (v_s), using recently published soil behaviour type classification charts. According to the Q_m-U_2 and Q_m-I_G charts, the clay tested probably has a pronounced microstructure that is likely a consequence of the geological processes of cementation and aging. Effects of cementation and aging are manifested on very high shear wave velocities, as measured in cross-hole seismic tests, and consequently very high values of the small-strain modulus. Soil structure collapses at higher shear deformations due to volume contractions and development of very high pore pressures and softening behaviour after reaching the peak shear strength. Unfortunately, during the preparation of this paper, the geologic history and analysis of the clay structure, that would be supporting the above observations and classification results, were uncertain and unavailable respectively.

The study shows a relatively good agreement between CPTu, PMT and SPT parameters and adequate correlations have been established between the cone resistance (CPTu), limit pressure (PMT) and number of blows (SPT). The in situ state and parameters of stiffness, strength and compressibility of the clay tested are estimated based on the in situ test parameters as well as comparative laboratory test results obtained on undisturbed soil samples. Existing empirical correlations for the interpretation of CPTu test results have been developed mainly from young, uncemented soils (e.g. [15]) without such pronounced microstructure and therefore, the results shown here are important for better understanding of the structured soil behaviour characteristics.

References

- [1] Robertson, P.K. 1990. Soil classification using the cone penetration test. *Canadian Geotechnical Journal*, 27(1): 151–158. doi:10.1139/t90-014.
- [2] Robertson, P.K. 2009. Interpretation of cone penetration tests – a unified approach. *Canadian*

- Geotechnical Journal, 46(11): 1337–1355. doi:10.1139/T09-065.
- [3] Robertson, P.K. 2016. Cone penetration test (CPT)-based soil behaviour type (SBT) classification system — an update. *Canadian Geotechnical Journal*, 53:1910-1927, <https://doi.org/10.1139/cgj-2016-0044>.
- [4] Schneider, J.A., Randolph, M.F., Mayne, P.W., and Ramsey, N.R. 2008. Analysis of factors influencing soil classification using normalized piezocone tip resistance and pore pressure parameters. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, 134(11): 1569–1586. doi:10.1061/(ASCE)1090-0241(2008)134:11(1569).
- [5] Baguelin, F.; Jezequel, J.F.; Shields, D.H. 1978. *The Pressuremeter and Foundation Engineering*, Trans Tech Publications.
- [6] Mair, R.J., Wood, D.M. 1987. *Pressuremeter Testing, Methods and Interpretation*. CIRIA 1987.
- [7] Vrkljan, I., Kavur, B., Fifer-Bizjak, K. 2006. Dilatometarska ispitivanja u inženjerskoj mehanici stijena / Dilatometer testing in engineering rock mechanics. *Građevinar* 58 (2006) 3, 187-197.
- [8] Čolja, D. and Kavur, B. 2017. Primjena Menardovog presiometra za procjenu G- γ krivulje krutosti sitnozrnatog tla / Use of Menard pressuremeter to assess G- γ stiffness curve of fine grained soil. *Inženjerstvo okoliša (2017) / Vol.4 / No.2*, 87-92.
- [9] Ivandic, K., Spiranec, M., Kavur, B., Strelec, S. 2018. Determining G- γ degradation curve in cohesive soils by dilatometer and in situ seismic tests. *World Academy of Science, Engineering and Technology International Journal of Geotechnical and Geological Engineering*. Barcelona: Waset, 2018. 144-151.
- [10] Kulhawy, F.H. and Mayne, P.W. 1990. *Manual on estimating soil properties for foundation design*. Report EL-6800, Electric Power Research Institute, Palo Alto, 306 p.
- [11] Briaud, J.L. 2013. The pressuremeter test: Expanding its use. *Proc. of the 18th Int. Conf. on Soil Mechanics and Geotechnical Engineering. Honour Lectures*. Paris, 2013. pp. 107-126.
- [12] Mayne, P.W. 2007. *Synthesis 368: Cone Penetration Testing*. NCHRP, Transportation Research Board, National Academy Press, Washington, DC, 118 p.
- [13] Kavur, B., Vrkljan, I., Mulabdić, M. 2008. Interpretation of CPTU results in terms of soil design parameters at Satnica Džakovacka, Croatia. *The Third International Conference on Site Characterization (ISC'3) "Geotechnical and Geophysical Site Characterization"*: proc. Huang, V ; Mayne, P.W., London: Taylor & Francis ; Balkema, 2008. 403-408.
- [14] Lacasse, S., Berre, T. and Lefevbre, G. 1985. Block sampling of sensitive clays. *Proc. of the 11th ICSMGE*, San Francisco, pp. 887-892.
- [15] Kavur, B., Mulabdić, M., Minažek, K. 2010. Evaluation of piezocone characterization in deltaic deposits. *2nd Inter. Symp. on Cone Penetration Testing, Huntington Beach, CA, USA, Vol. 2 & 3: Technical Papers, Session 2: Interpretation, Paper No. 19.*