Paired in situ tests for site characterization and geotechnical design optimization

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ABSTRACT

The paper shows recent experience in performing and interpreting the three most common in situ tests for site characterization: cone penetration test (CPT), seismic dilatometer test (SDMT) and Menard pressuremeter Test (MPT). It is shown that an adequate selection of in situ test methods for site characterization can be used to correctly predict the pile head load-settlement curve. Menard pressuremeter design rules, which were established in the 1st and a draft version of the 2nd generation of Eurocodes, can be applied to SDMT readings to obtain the pile bearing capacity. It was found that corrected lift-off pressure p_0 obtained in the SDMT test is similar to the Menard limit pressure p_l in clay, while pressure p_1 (SDMT) is similar to p_l in sand. All results used in the analysis are obtained on a large-scale project where investigations are strictly controlled and performed in accordance with Eurocode standards.

Keywords: pile static load test, Menard pressuremeter test, seismic dilatometer test.

1. Introduction

The cone penetration test (CPT), the seismic dilatometer test (SDMT) and the Menard pressuremeter test (MPT) are the most frequently used in situ tests for site characterization in recent years in Serbia. This is partly due to increasing investments in the design and construction activities in the civil and mining sectors and partly to legal obligations related to Eurocode. The evolution of the Eurocode from 1st to 2nd generation even more highlights the need for reliable site characterization, use of advanced numerical models, stiffness dependence on strain level, determination of dynamic, cyclic and seismic soil properties, etc. This is why a proper laboratory test program and in situ site investigations form the basis for the safe and reliable design of structures.

Several field investigation campaigns performed as a part of a design of industrial facilities, road and railway infrastructure, residential towers and office buildings made it possible to apply the three mentioned methods for design of various geotechnical structures. The paper will focus on a single pile load-settlement curve prediction using MPT and potentially from SDMT.

1.1. Site description

The site is located on the flood plain of the Sava river close to the town of Obrenovac in Serbia. The geology is relatively uniform across the site. It is formed from alluvial deposits up to 14-15 m deep underlained by thick deposits of Miocene lacustrine clay. The general ground profile is shown in Fig. 1 in terms of SDMT parameters versus depth. In situ investigation consisted of performing: 8 boreholes (up to 35 m depth), 8 MPT's, 5 CPT's and 2 SDMT's. All penetration tests are 30 m deep. Slotted casing was used in the sand to perform MPT tests. Beside usual geotechnical parameters V_{s30} was evaluated from SDMT and it is used to determine ground type according to Eurocode 8.

1.2. Specifics of penetration tests

Three geotechnical units can be distinguished in terms of drainage conditions and mechanical behavior during penetration testing. The upper 5-6 meters consists of silty clay and clayey silt. The water table is at 2.5-3 m below the ground surface. Due to capillary action the first few meters of soil are saturated, while negative pore water pressures influence soil behavior. Below this zone up to 14 m deep the soil consists of silica sand and penetration is fully drained. From 15 m up to investigation depth the soil consists of stiff to very stiff lacustrine clay, which behaves undrained during penetration. Pore pressure parameter Bq equals 0.55, while pore pressure index U_d=0.6-0.65 on average. It was noticed that when cone penetrated into very stiff clay pressure sensors rapidly reached their measuring range of 2.36 MPa. Previous experience at the same location suggests that u_2 measured using a cone from another manufacturer with higher measurement range reached approximately 2.7 MPa. This "range effect" may influence the interpretation of CPT results to some extent since all parameters are functions of measured pore pressure. For this particular clay the u_2/u_0 ratio is $\approx$ 11.

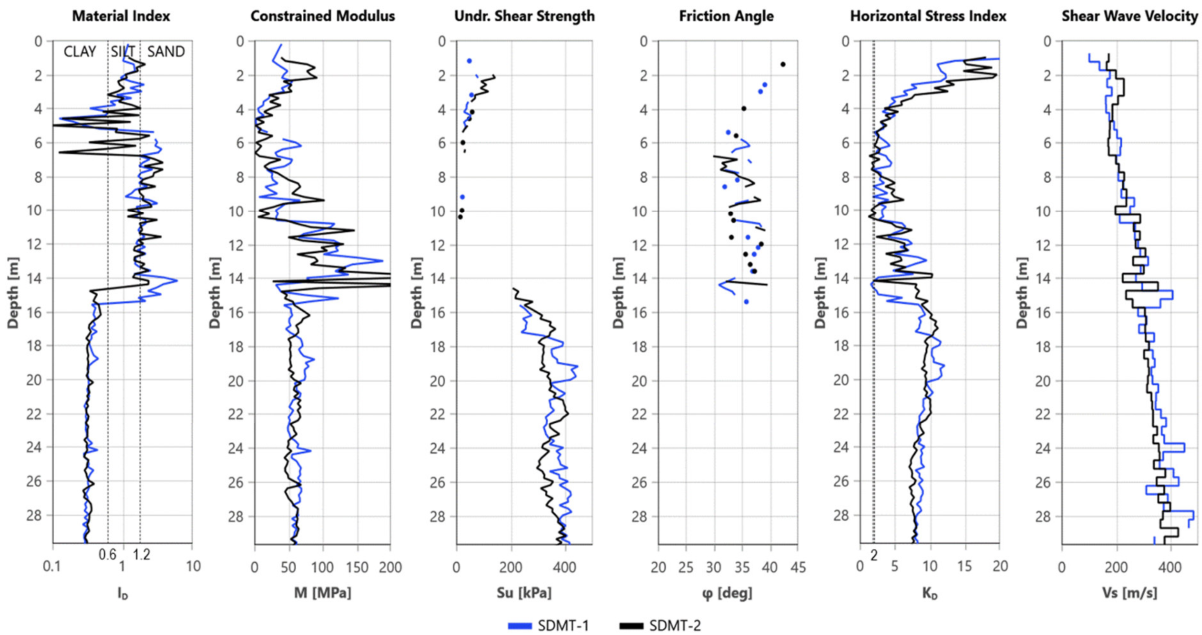

Figure 1. SDMT test results

Robertson (2016) proposed a chart to identify soils with microstructure using the small strain rigidity index (I_G). The combined results of two adjacent CPT and SDMT (V_s) measurements are presented on the normalized rigidity index chart shown in Fig. 2. Lacustrine clay is characterized by KG*≈600 indicating a soil with a microstructure. This may be explained by the presence of a carbonate content up to 15% in the soil mass. In addition, microstructure can be inferred from the SDMT by inspecting the horizontal stress index (K_D) profile. Fig. 1 indicates an almost constant K_D profile below 15 m, which is typical for cemented clays (Marchetti et. al., 2001). Overconsolidation ratio (OCR), or in our case yield stress ratio (YSR), of lacustrine clay interpreted from laboratory test results ranges from 4 to 6, which is in agreement with CPT results and SDMT results using the Mayne (1987) correlation, where YSR=0.5K_D. According to geological findings, upper clay and alluvial sand are normally consolidated.

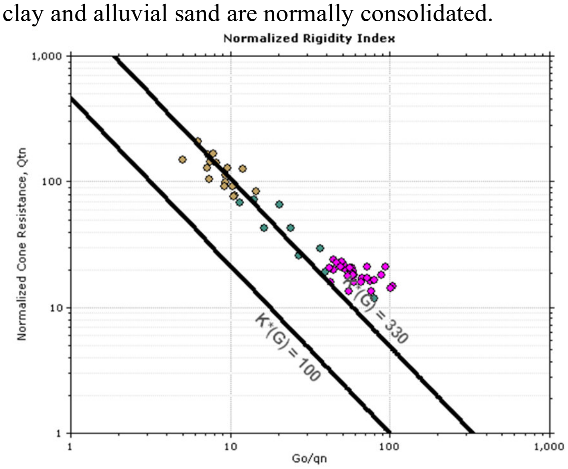

Figure 2. Normalized rigidity index chart (green=silt/clay; brown=sand; pink=lacustrine clay)

The presence of microstructure can be inferred from the comparisson of CPT predicted and measured Vs (SDMT) profiles (e.g., Berisavljević et al., 2014, 2019) as shown in Fig. 3. The correlation used to predict V_s can be found in Robertson (2012).

It can be observed that for the older sediments, below 14 m, predicted V_s are lower compared to measured V_s. In some cases, if predicted Vs is used to calculate V_{s30} the incorrect ground type according to EN-8 could be determined.

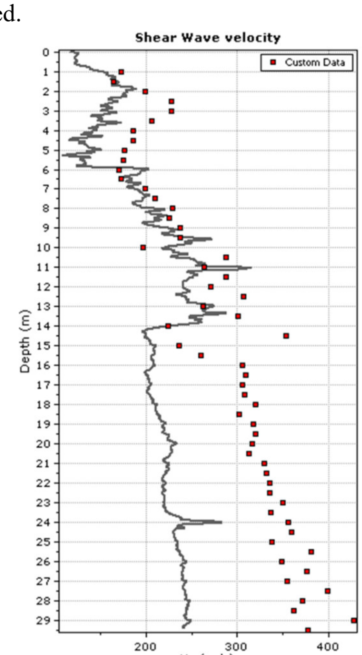

Figure 3. Predicted (continuous gray line) vs measured (red squares) Vs profiles

2. Comparison of SDMT and MPT results

Several references (Marchetti et. al., 2001; Schmertmann, 1987; Lutteneger, 2006) address the fact that there is a good link between SDMT lift-off pressure p_0 and pressuremeter limit pressure p_l in clay. Menard defined the limit pressure for an MPT test as the pressure required to double the initial volume of the cavity. This is equivalent to the 41% cavity strain. Menard limit pressure is a method-specific parameter and depends on disturbance, installation effects and ground properties. The standard SDMT procedure derives p_0 from the assumption of the linear pressure-displacement relation and is back extrapolated from the pressure readings at 0.05 mm and 1.1 mm. Both SDMT and MPT stress soil in a horizontal direction after some disturbance induced by their installation. Fig. 4 shows a comparison between SDMT p_0 and MPT p_l pressures at the Obrenovac site. It can be seen that in upper and lower clay deposits $p_0 \approx p_l$, while in sand $p_1 \approx p_l$. Tests in clay are essentially undrained. Tests in sands are drained since no excess pore pressures are developed. These are important facts for the theoretical interpretation of penetration and pressuremeter tests. Since for this particular site there is a good agreement between SDMT and MPT pressures, it should be possible to apply MPT design rules to SDMT test results. The merits of this approach rely on the fact that SDMT is more simple to perform and measurements are taken at more frequent intervals (usually 0.2 m) compared to MPT. On the other hand, the interpretation of pressuremeter test results relies on a sound theoretical basis. For this approach to be useful, a strong link between SDMT results and Menard modulus (Em) should be established. The Em can be estimated from pl, since the ratio E_m/p_l is well established for various soil types (Clark, 2023; Eurocode 7).

3. MPT design rules

The design of foundations with MPT (Baguelin et al., 1978; Bustamante and Frank, 1999) consists of correlating the base resistance q_b and the shaft resistance q_s to the limit pressure p_l. The correlation between p_l and q_b is supported by the analogy between the expansion of a cylindrical cavity and the mobilization of the base resistance. The correlation between p_l and q_s is more empirical.

Based on 174 full scale pile load tests Burlon et al. (2014) presented a new MPT calculation method for pile bearing capacity. The bearing capacity of the pile is defined as the load that produces the settlement of the pile head equal to 10% of the pile diameter (D). This method is included in the Annex of the 2nd generation of Eurocode 7-3.

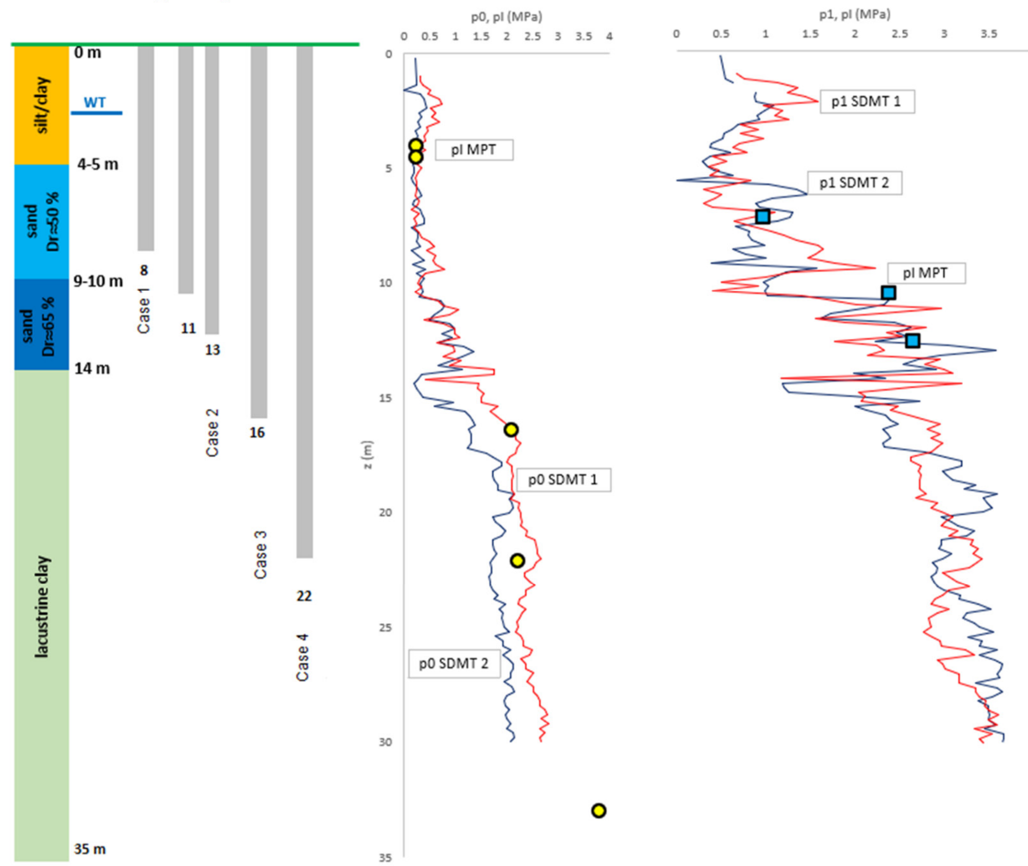

Figure 4. Ground model with p_0 and p_1 from SDMT vs p_l from MPT

According to this method, unit shaft resistance is calculated using equation 1.

$$q_s = k_{s,}(ap_l + b)(1 - e^{-cpl}) \leq q_{s,max} \tag{1}$$

where k_s is dimensionless parameter that depends on pile type and ground type. Parameters a, b and c depend on the ground type and can be found in Burlon et al. (2014) or in the draft version of Eurocode 7-3.

Unit base resistance is calculated according to equation 2.

$$q_b = k_b p_{le} \tag{2}$$

where k_b is a dimensionless parameter that depends on pile type and ground type; p_{le} is the equivalent net limit pressure defined as:

$$p_{le} = \frac{1}{b+3a}\int_{L-b}^{L+3a} p_l^*(z)dz \tag{3}$$

$$a = \max\left\{\frac{D}{2}; 0.5\right\} \tag{4}$$

$$b = min\{a; h\} \tag{5}$$

where p_l* is net limit pressure (p_l-σ_h); z is the test depth, L is the total embedded pile length; h is its length in the resting layer and D is the pile diameter.

The parameter k_b shown in equation 2 is influenced by the effective embedded depth L_{ef}. In our case k_b is influenced by L_{ef} only in Case 1 and Case 2 as shown later in the paper.

4. Static pile load tests

During 2019 and 2020 thirteen full-scale static pile load tests (SLT) were performed on the test site. Eleven piles were vertically loaded in compression, one in tension and one was loaded horizontally. We will consider the results of nine piles loaded in compression, since two had problems with their integrity as shown by the sonic integrity test method. Results are discussed in terms of four cases, where each case considers piles with different lengths, as shown in Fig. 4. The number of piles for each case and their respective lengths are shown in Table 1. The cases have been chosen based on the geotechnical unit where the pile is embedded. The nominal diameter of the piles is 880 mm. All piles are drilled with temporary casing.

Table 1. Pile cases considered

	pile length (m)	No. of piles
Case 1	1x8	1
Case 2	2x11 2x12 1x13	5
Case 3	2x16	2
Case 4	1x22	1

4.1. t-z curve from MPT

Frank and Zhao (1982) presented a method (also called FZ method) to obtain the t-z curve using MPT test results. Net limit pressure p_{le} is used to obtain both q_s and q_b using equations 1 and 2. The stiffness of the t-z (q-z) curve is dependent only on the Menard modulus E_M.

$$k_s = \frac{\alpha_s E_M}{B} \tag{6}$$

$$k_b = \frac{\alpha_b E_M}{B} \tag{7}$$

Where α_s=2 and α_b=11 for fine-grained soils and α_s=0.8 and α_b=4.8 for coarse-grained soils. More details about the method can be found in Abchir et al. (2016). The FZ method is simple to use and can give correct results (e.g., Clark, 2023; Bohn et al., 2013). However, it doesn't capture strain softening or strain hardening effects, which were noticed on some sites close to Belgrade.

4.2. MPT prediction of SLT results

Since cases 1 to 4 consider different pile lengths embedded in different geotechnical units, it was possible to evaluate the applicability of the FZ method and MPT results to obtain a load settlement curve for a single pile.

The load transfer method (Coyle and Reese, 1966; Kraft et al., 1981) is frequently used to predict the load-settlement curve of axially loaded piles. In this method, the pile is divided into discrete elements and for each element the soil is modeled by a set of load-transfer curves that represent the soil resistance as a function of pile displacement at several discrete points along the pile, including the pile toe. This method is far from perfect since it neglects coupling effects on shaft resistance. However, it is frequently used in practice and great experience is gained with this method.

In Fig. 5 to Fig. 8 the load settlement curve of the pile head obtained by assigning to each segment the t-z (q-z) function defined by the FZ method is shown.

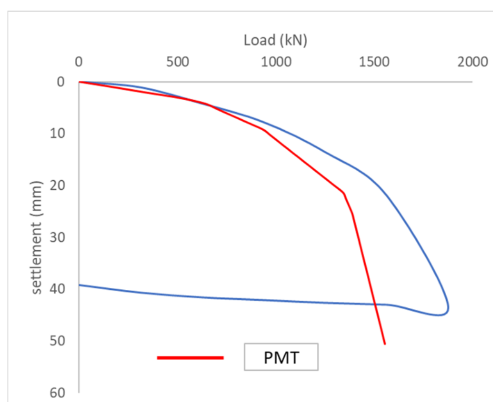

Figure 5. Case 1 prediction L=8 m

5. Discussion

Figure 4 shows that p_l can be estimated from p_0 (SDMT) in clay and p_1 in sand. This indicates that one can apply MPT design rules directly to SDMT pressures in order to obtain the pile bearing capacity. Pile head settlement as a function of applied load can be predicted from SDMT test using MPT FZ method in cases where a reliable estimate of Em is possible.

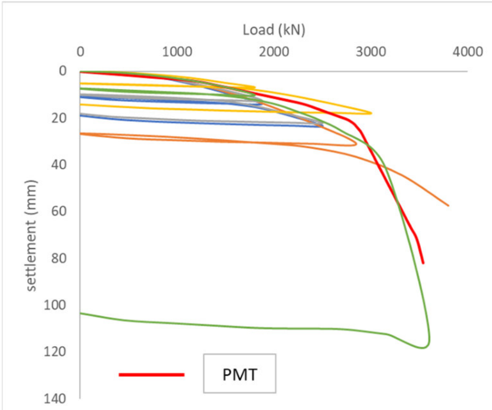

Figure 6. Case 2 prediction L=11-13 m

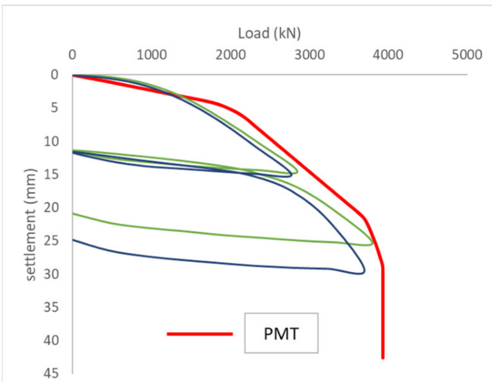

Figure 7. Case 3 prediction L=16 m

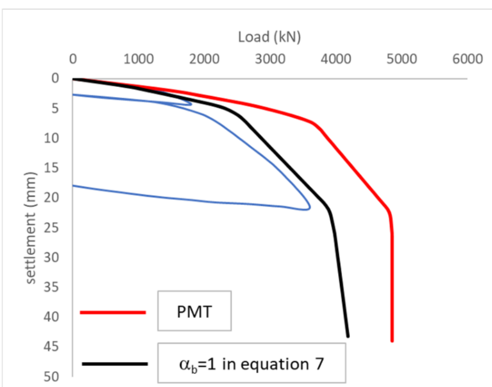

Figure 8. Case 4 prediction L=22 m

The benefit of the MPT method is that it provides a pressure-volume curve or pressure-cavity strain curve that can be used to interpret fundamental soil properties using theory.

Figures 5 to 7 indicate a good prediction of the load settlement curve for design purposes using MPT test results. It should be mentioned that no restrictions on q_s were made. These restrictions are valid only in the sand 2 layer since q_s calculated according to equation 1 is on average 110 kPa. The prescribed limit should be 90 kPa (2nd generation Eurocode 7-3). However, we consider that when analyzing full-scale tests, there are no such limitations on shaft friction. Case 4 (Fig. 8) shows a poor prediction of the load settlement curve. A better fit could be obtained if α_b is reduced to 1 in equation 7. This means that the stiffness of the mobilization curve (t-z curve) is very low, indicating that very low pile base capacity is mobilized in the lacustrine clay for the movements in the range accepted for the SLS design.

6. Conclusion

Based on the observations from the paired in situ tests and SLT, the following may be concluded:

- Menard limit pressure p_l can be predicted from SDMT corrected pressures p_0 and p_1. For clay, $p_l \approx p_0$, while for sand, $p_l \approx p_1$,
- For cases 1 to 3 MPT FZ method can be used to reliably estimate pile head settlement for SLS design,
- For long piles (case 4) with length in the resting layer of more than 9 pile diameters, pile head settlement curve is poorly predicted by the MPT FZ method. A better prediction is obtained when base stiffness is reduced and the pile is considered to behave as a frictional pile (no base resistance),
- Lacustrine clay, which is considered the resting layer for this site, is characterized by a large u_2/u_0 ratio of ≈ 11 and Q_{tn}=18 on average. These two parameters may be used to evaluate the reliability of the MPT FZ method for piles with significant length in the resting layer,
- Soil with significant microstructure can be discerned by comparing measured to estimated Vs profiles and
- The findings presented in this paper are applicable to this particular site and further research is needed.

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