Interpretation of CPTU and SDMT in organic, Irish soils

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ABSTRACT: The Limerick South Ring road project started in August 2006 and connects the N7 Dublin Road to the N18 Ennis Road. An extensive laboratory and field soil investigation program was carried out. The soil consists of alluvial (mostly organic) fine clays and silts with thicknesses up to 13m. This paper presents the results of CPTU and seismic dilatometer tests (SDMT) carried out by In Situ Site Investigation (In Situ SI) at two locations along the new embankment in November 2008. It is the first time that the research DMT including shear wave velocity measurements was tested on Irish soils. Effective stress based models developed at the NTNU have been used among several others to interpret the data from the CPTU. The results obtained are compared to the findings from the SDMT.

1 INTRODUCTION

The Limerick Southern Ring Road project consists about 10 km of dual carriageway including permanent embankments of 3 to 8m and temporary surcharge of 2 to 3m height, built on soft alluvial soils. A detailed database of laboratory and field test data is available which were performed prior to any embankment construction, Buggy & Peters (2007). Additional tests have been performed by In Situ SI comprising CPTU and SDMT tests at two locations south and north of the river Shannon in November 2008 (Ch 2+850m and Ch 0+750m respectively). These tests were carried out at 5m distance from the toe of the embankments which were already constructed with temporary surcharge in place.

2 GROUND CONDITIONS

2.1 *Overview*

The main focus of the soil investigation program was on borehole shear vane testing and CPTU as well as series of undisturbed soil samples using piston sampling techniques. The investigations showed that the soil consists of alluvial (mostly organic) fine clays and silts with thicknesses up to 13 m and glacial till and limestone lying underneath, Buggy & Peters (2007). Since the ground conditions show a very constant profile throughout the whole project area it has been decided to focus only on location Ch 0+750m in this paper, see Figure 1.

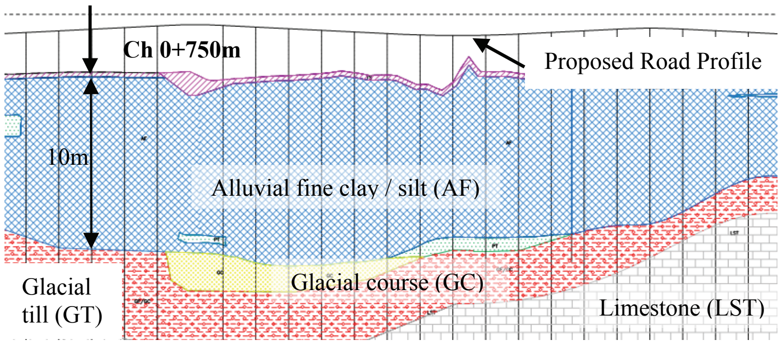

Figure 1. Cross section of the geological profile

2.2 *Existing laboratory and field test results*

Figure 2 shows a summary of the index data available close to Ch 0+750m for the soft alluvial soil, Buggy & Peters (2007). The moisture and organic content decrease with depth and higher values seem to appear in the upper four meters having at the same time lower bulk density. The plasticity index I_p tends to show the same trend being 80 % above 4 m and decreasing below 4 m to an average value of about 40 %.

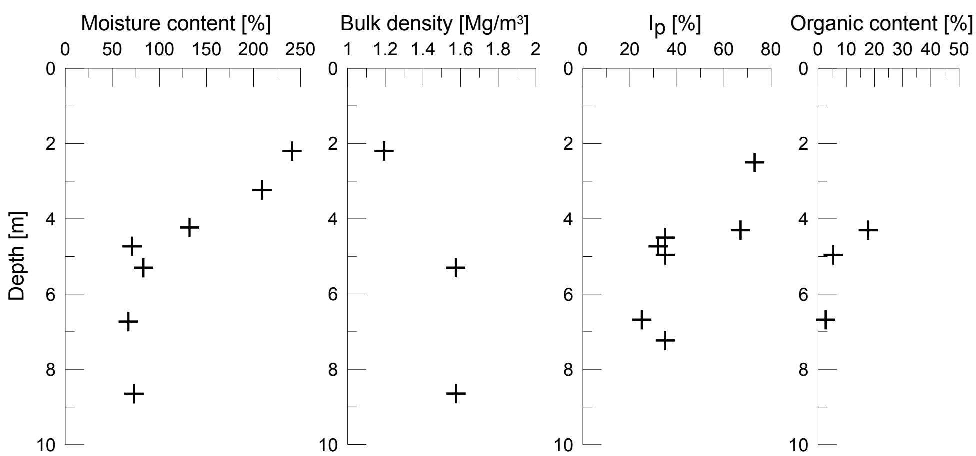

Figure 2. Summary of index test results around Ch 0+750m

Soil samples were taken from different borehole locations and a series of oedometer tests were performed. Unfortunately most of the samples were of poor quality. However the results of the deformation parameters are used to compare these values with the results from the CPTU and SDMT tests. The undrained shear strength values are estimated based on CPTU, borehole vane tests and triaxial tests (unconsolidated undrained (UU) and anisotropic consolidated undrained compression (CAUC) tests). Unfortunately just a few CAUC tests are available and are of varying quality. The triaxial stress paths show friction angles around 34 degrees and attraction values in the order of 4.5 kPa.

3 INTERPRETATION METHODS

3.1 *CPTU*

3.1.1 *Soil classification*

Soil classification is based on the chart originally developed by Senneset & Janbu (1985) and later revised by Senneset et al. (1989). It combines the corrected cone resistance q_t with the pore pressure parameter B_q and allows application even for negative excess pore pressures; see Figure 4a. Using classification charts one has to keep in mind that they are indicators for soil behavior and not indicators for grain size distribution.

3.1.2 *Undrained shear strength*

Generally s_u can be calculated using the net cone resistance q_{net} divided by a cone factor N_{kt} which is determined from a reference value for s_u. In this paper CAUC tests have been used to establish the cone factors. Senneset et al. (1982) developed an expression using the effective cone resistance q_e instead of q_{net}. For $B_q < 0.4$ the s_u values might be questionable due to sensitivity of measured q_e. The relevant expression is

$$s_u = \frac{q_e}{N_{ke}} = \frac{q_t - u_2}{N_{ke}} \quad ; N_{ke} = 9 \pm 3 \tag{1}$$

Karlsrud et al. (2005) established the following correlations for s_u based on high quality Canadian Sherbrooke block samples, depending on the sensitivity S_t of the material. However care has to be taken for values of Norwegian clays of $B_q < 0.6$ since the scatter is much higher due to less data in that range.

S_t<15: $$N_{ke} = 11.5 - 9.05 B_q \tag{2}$$

S_t>15: $$N_{ke} = 12.5 - 11.0 B_q \tag{3}$$

Lunne et al. (1997) recommended using the excess pore pressure approach for very soft soils since there might occur uncertainties in q_t measurements. Different values for $N_{\Delta u}$ can be found in the literature. Janbu (1977) reported theoretical values of $N_{\Delta u}$ in the range of 4 to 8 for use in the equation:

$$s_u = \frac{\Delta u}{N_{\Delta u}} = \frac{u_2 - u_0}{N_{\Delta u}} \tag{4}$$

3.1.3 *Deformation parameters*

Janbu (1974) suggested a linear correlation between the deformation modulus and the net cone resistance utilizing the fact that the initial in situ modulus M_i and the net cone resistance can be expressed by Equation 5 and 6. Eliminating s_u one obtains the equation used for the deformation modulus, Equation 7. Senneset et al. (1989) suggested values of m_i between 5 and 15. Later Sandven (1990) found that m_i decreases for increasing B_q and decreasing q_{net} and OCR.

$$M_i = m_u \cdot s_u \tag{5}$$

$$q_{net} = N_{kt} \cdot s_u \tag{6}$$

$$M_i = m_i(q_t - \sigma_{vo}) \tag{7}$$

3.1.4 *Stress history*

Sandven (1990) established an effective stress approach to determine the preconsolidation pressure σ_c`. Using a conventional bearing capacity expression $q_e$ can be expressed as a function of $\sigma_c$`.

$$q_e + a = N_{qc}(\sigma_c' + a) \tag{8}$$

An interpretation diagram has been developed for the bearing capacity coefficient N_{qc} as a function of the friction angle φ which can be found in Sandven (1990). Sandven (1990) stated that this approach gave reasonably good predictions in slightly overconsolidated deposits but seemed to underestimate σ_c` in moderately to heavily overconsolidated materials.

3.2 *DMT*

The three key dilatometer parameters are the material index I_D, the horizontal stress index K_D and the dilatometer modulus E_D expressed by combination of p_1 and p_0 as the first and second pressure reading, Marchetti et al. (2001) as follows:

$$I_D = \frac{p_1 - p_0}{p_0 - u_0} \tag{9}$$

$$K_D = \frac{p_0 - u_0}{\sigma'_{v0}} \tag{10}$$

$$E_D = 34.7(p_1 - p_0) \tag{11}$$

I_D is related to the rigidity index of the material, K_D is dependent on the coefficient earth pressure at rest K_0 and E_D is a function of the stiffness of the soil. Figure 4b shows a soil classification chart including I_D and E_D, Marchetti & Crapps (1981).

The undrained shear strength s_u can be determined by a combination of K_D and the in situ effective overburden stress σ'_{v0} (see Equation 12). The modulus M_{DMT} represents the vertical drained tangent modulus at geostatic overburden pressure corresponding to the oedometer modulus. It is obtained by applying a correction factor $R_M(I_D, K_D)$ on E_D (see Equation 13). Due to observed similarity between the K_D profile and OCR profile the following formulation for OCR_{DMT} for clays have been found (see Equation 14).

$$s_u = 0.22 \cdot \sigma'_{v0}(0.5K_D)^{1.25} \tag{12}$$

$$M_{DMT} = R_M \cdot E_D \tag{13}$$

$$OCR_{DMT} = (0.5K_D)^{1.56} \tag{14}$$

4 RESULTS

4.1 *General*

The CPTU measurements at Ch 0+750m carried out by In Situ SI have been compared to existing data. Figure 3a shows a summary of the corrected cone resistance q_t, measured pore pressure u_2 (including the hydrostatic line) and sleeve friction f_s. SDMT tests have also been carried out during the test series by In Situ SI. The main results for Ch 0+750m are shown in Figure 3b.

The measurements carried out by In Situ SI fit very well with the existing CPTU results. However some variations in cone resistance and sleeve friction can be noted. One has to keep in mind that the existing CPTU measurements have been taken before commencing construction of the embankment. The tests carried out by In Situ SI are carried out after (partly) construction at the embankment toe. This might explain the higher cone resistance and sleeve friction especially in the top 3 to 4 m, probably due to compaction and partly consolidation of the material. However to be able to get better correspondence with existing lab data it has been decided to mainly use the results from CPT583 for comparison with CAUC and oedometer test data. The results from In Situ SI have been used for comparison with SDMT results.

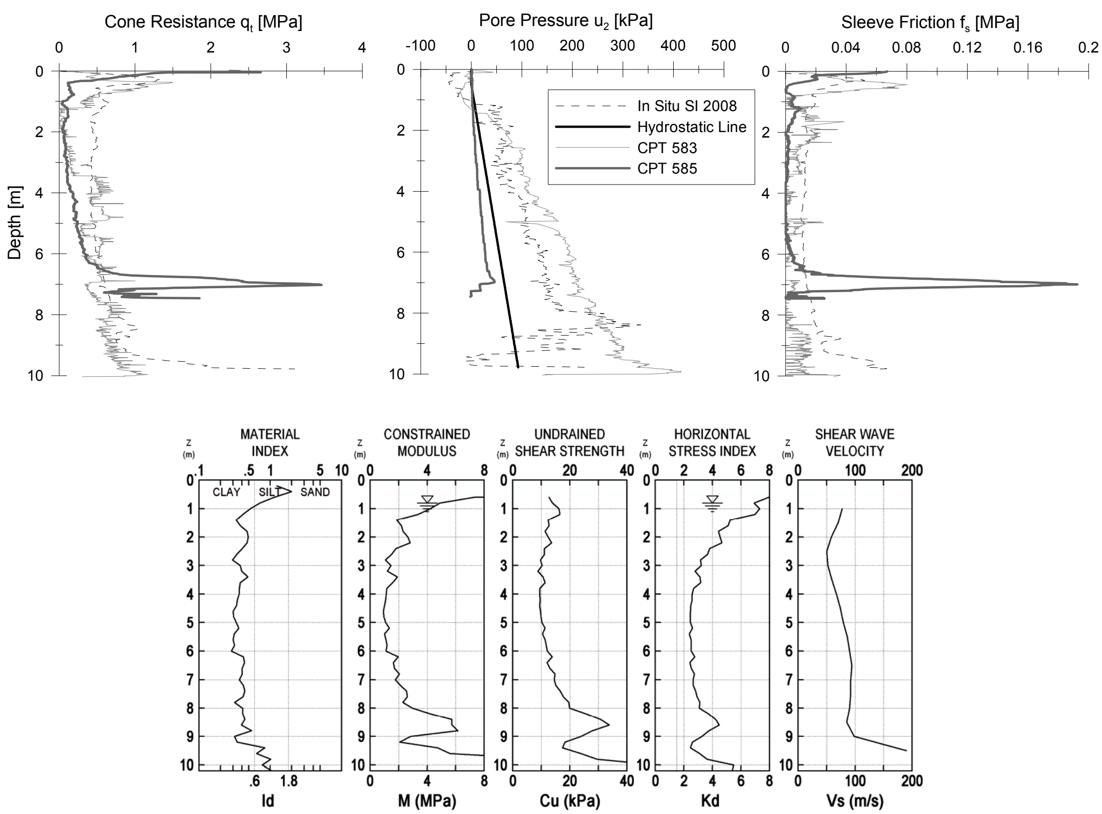

Figure 3. (a) CPTU and (b) SDMT results from Ch 0+750m

4.2 *Soil classification*

Soil classification for CPTU is based on the chart developed by Senneset et al. (1989), see Figure 4a. The alluvial soil is classified as fine silt, medium clay and soft to very soft clay due to high excess pore pressures combined with low cone resistance which is typical for Irish soils.

Figure 4b shows the soil classification for the SDMT results after Marchetti & Crapps (1981). The alluvial soil is identified as silty clay to mud, with very low E_D and $I_D < 1$. The suggested unit weight lies between 15 and 16 kN/m^3 which is consistent with Figure 2. During the project an average value of 17 kN/m^3 has been used which corresponds well with the proposed values from the SDMT, Buggy & Peters (2007).

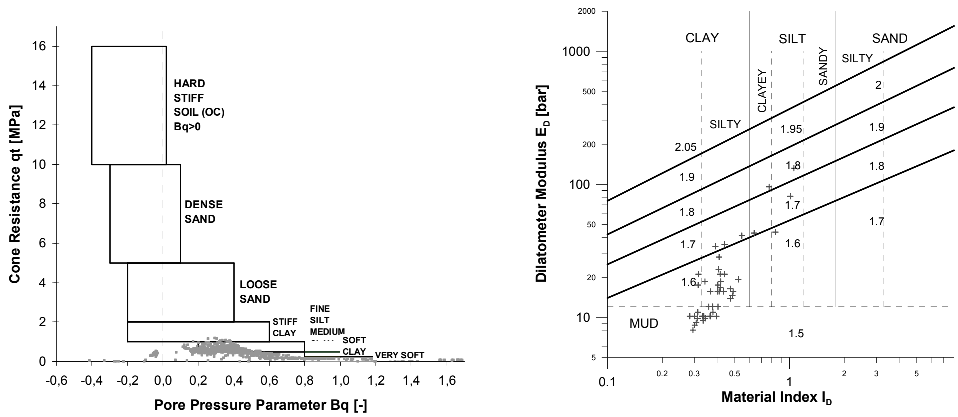

Figure 4. (a) Soil classification after Senneset et al. (1989) and (b) Marchetti & Crapps (1981)

4.3 *Undrained shear strength*

Figure 5a shows the undrained shear strength profiles using cone factors of N_{kt}=17, N_{ke}=14 and $N_{\Delta u}$=5, established from Equation 1 and 4. All three approaches show consistent s_u-profiles which fit reasonably well with the existing CAUC test data.

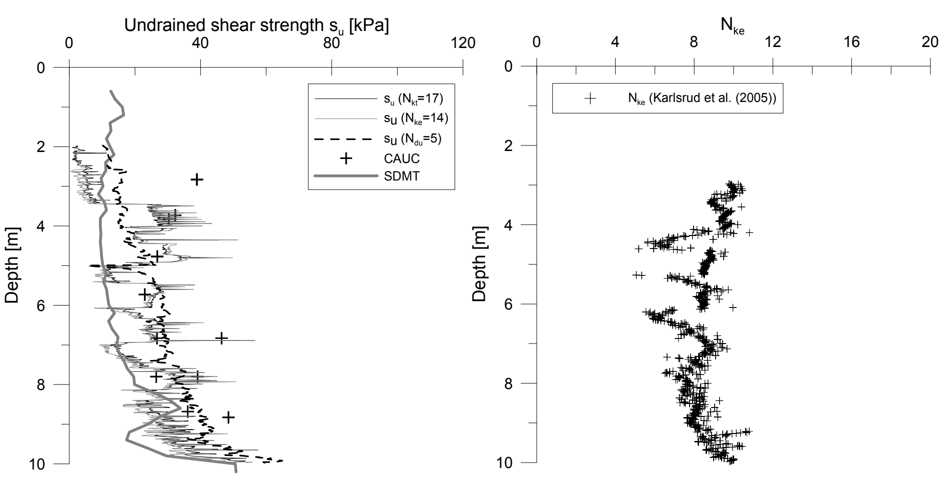

Figure 5. (a) Undrained shear strength profiles and (b) cone factors after Karlsrud et al. (2005)

The s_u-profile suggested by the SDMT shows lower values in the upper part of the alluvial deposit than the CPTU results, however converging at larger depths. Due to varying quality of the CAUC tests some data points lay far outside the suggested s_u-profiles. Karlsrud et al. (2005) suggests lower cone factors established from Equation 2 in the range of N_{ke}=8-10, leading to somewhat higher s_u values, Figure 5b.

4.4 *Deformation parameters*

Figure 6 shows the deformation modulus derived from the SDMT and CPTU together with the available oedometer test data. Choosing a value of $m_i = 5$ gives the best fit with the laboratory data and is in agreement with the suggested range by Senneset et al. (1989) corresponding to very soft soil. The modulus curve fits reasonably well with the SDMT curve. However due to poor quality of the laboratory data, a wide scatter cannot be avoided.

Having in mind that the SDMT was carried out at the toe of the partly constructed embankment, the dilatometer results are also compared to In Situ SI results. It can be seen from Figure 6 that there is a good agreement between the two testing methods.

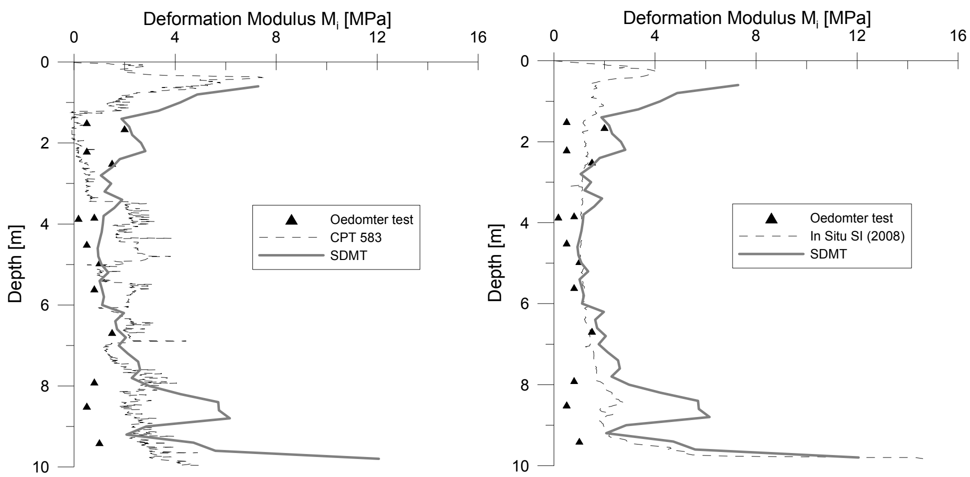

Figure 6. Deformation modulus M_i : Results for CPT 583 and In Situ SI (2008)

4.5 *Stress history*

Using a friction angle of 34 degrees as proposed by the triaxial test results, bearing capacity coefficients N_{qc} between 5.5 and 6.0 have been used for the interpretation of the preconsolidation pressure. Figure 7 shows results for both CPT 583 and In Situ SI together with SDMT and oedometer test results.

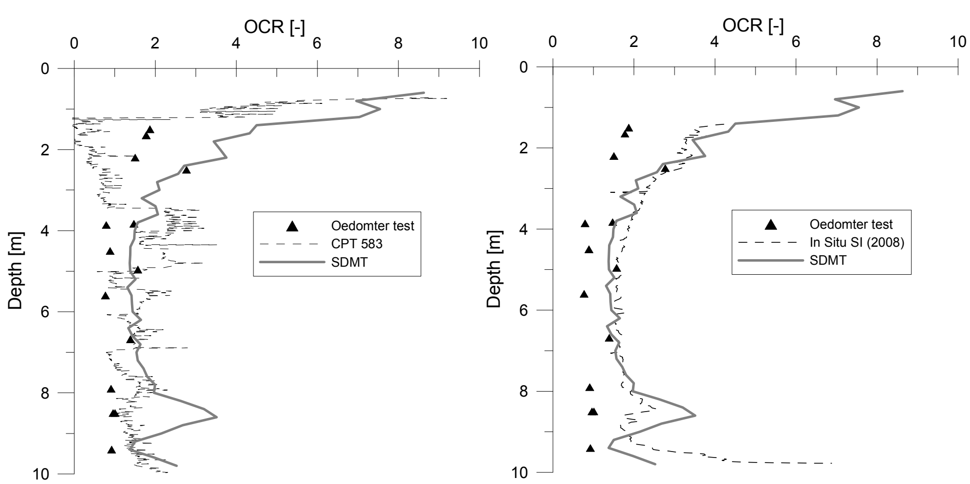

Figure 7. Overconsolidation ratio

The results for CPT 583 fit better with the oedometer data than the In Situ SI test. However due to the earlier mentioned poor quality of the oedometer test results a constant consistency can not be reached. Comparing the CPTU with the SDMT results it can be noticed that especially the results from the In Situ SI test fit very well.

5 CONCLUSIONS

The use of SDMT tests on Irish soils has been studied in this paper as well as the application of effective stress based interpretation models for CPTU developed at NTNU. Despite the complexity of the organic material the models appear very useful for estimating in situ stresses, strength and stiffness characteristics giving parameters which are at least suitable for preliminary designs. It would be useful to perform tests further away from the toe of the embankment to achieve better comparison with the existing laboratory data and collect data from higher quality samples (e.g. block samples) than were available here. Although it was one of the first trials, the SDMT shows potential for use in Irish alluvial soils. It leads to good results especially for deformation and stress history parameters. However more work needs to be done on sites with different soil properties to gain a wider data basis.

6 ACKNOWLEDGEMENT

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