The use of the seismic flat dilatometer for soil characterisation and geotechnical design of a fjord crossing

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ABSTRACT: The results of flat dilatometer tests with measurement of shear wave velocities were used to characterise the founding soil of the piled piers for a motorway bridge crossing a fjord. The geotechnical investigation comprised several in-situ techniques as well as laboratory tests, the results of which were compared with those of the sDMT tests. The ground conditions comprise soft, fine- and coarse-grained soils and late-glacial or moraine deposits. The tests were undertaken either on the seabed or downhole and the energy source was generated with an underwater hammer. The results of the sDMT test were instrumental for the analysis of various foundation options at illustrative design stage which include large diameter bored piles, and either driven pre-cast concrete or tubular steel piles installed at a rake, particularly to assess the behaviour under the vertical loads and under the lateral loading due to the potential impact of the vessels crossing the navigational channel.

Keywords: seismic flat dilatometer sDMT; underwater hammer; bridge; bored piles; Denmark

1. Introduction

1.1. Fjord Link Frederikssund project

The Fjord Link Frederikssund Bridge (a.k.a. Crown Princess Mary Bridge) is a 1.4 km motorway bridge crossing the Roskilde Fjord, south of the existing road bridge near Frederikssund. The bridge forms part of the Fjord Link Frederikssund project by Vejdirektoratet (the Danish Road Directorate): a 9.3 km dual carriageway highway which connects Skibbyvej to Marbækvej.

The highway cross section at the high bridge is 18.6 m wide, and the bridge has a clear navigation height of 22 m above the water. The substructure of the bridge is founded on large diameter bored cast-in-place piles. The foundations are bearing the vertical loads shed by the bridge structure, as well as lateral loading due to the potential impact of vessels crossing the navigation channel, in addition to lateral loads from to temperature variations, ice load and breaking loads from traffic.

1.2. Topography

The topography of the region is dominated by the glacial history of the area. The terrain in the vicinity of the fjord is generally low-lying grasslands and rural agricultural developments. The coastal area of the fjord is distinctive for its salt marshes, reed swamps and bays.

Site levels within the axis of the main alignment are between +26.5 m DVR90 (Danish Vertical Reference 1990) and -5.6 m DVR90 at the fjord seabed. From the eastern abutment of the bridge at +7.5 m DVR90 the ground slopes down to the banks of the fjord at sea level. On the west bank, the abutment lies at approximately +4 m DVR90 before sloping to sea. On the western bank of the fjord the bridge crosses approximately 200 m of shallow waters (<1m depth).

1.3. Geology and ground conditions

1.3.1. Geological history

The geological history of the fjord is dominated by the sediments deposited during the retreat of the Scandinavian ice-sheet during the Weichselian/ Vistulian glaciation in the Pleistocene. During this period, the area was covered by an ice sheet up to 3,000 m thick, extending from Norway to Germany and to Ireland to the west.

Towards the end of the Pleistocene as global temperatures began to rise and ice began to melt, the fjord was a large meltwater river. After the Pleistocene, the rise in temperatures led to the ice melting rapidly causing the sea level to rise and the continent rising isostatically, draining the fjord as the land level raised higher than the sea.

Following this, the marine transgression began, leading to an approximate sea level rise of 20 m and the fjord becoming a marine environment again. Finally, following a short period of being landlocked as sea levels fell, sea levels rose to the current levels and the fjord once again became connected with Isefjord and hence Kattegat and the North Sea.

1.3.2. Existing ground conditions

The solid geology in the fjord area comprises primarily Danian limestone. This is a limestone deposited in a marine environment during the Danian Stage. The dominant geological feature of the fjord is the Roskilde Fault [1], running approximately North-South through the centre of the fjord, marked by a paleochannel. The Danian limestone depth ranges from approximately -43 m DVR90 at the centre of the fjord, to as shallow as -10 m DVR90 on the eastern and western sides.

In addition to the dominant Danian limestone, Selandian Greensand is also locally present, usually overlaying the Danian limestone as a sand or clay, and has only been identified at the banks of the fjord.

The drift geology in the area is dominated by glacial and postglacial sediments deposited in the form of ridges, moraines and meltwater deposits. The glacial deposits tend to comprise a heterogeneous mix of clay, sand and gravel (known as till). In the postglacial period, clays and peats were deposited in the marine environment.

1.3.3. Stratigraphy

Indicative ground conceptual model is shown in Fig. 1. Generally, the Danian limestone formation is overlain by coarse-grained Meltwater Deposits (sands and gravels), with the exception of some areas on the west side of the fjord where layers of Glacial Tills are found between them. The Glacial Till found in the area is predominantly fine-grained.

In the centre of the fjord, where the overburden thickness is greatest, the coarse-grained Meltwater Deposits are found in thicknesses of up to 20 m. Overlying these coarse-grained deposits are a fine-grained layer of the same era (fine-grained Meltwater Deposits) of up to 8 m thickness. Finally, the postglacial deposits were identified in up to 13 m thickness, comprising Gyttja and Peat, with some thinner layers of Shell materials and Postglacial Clays and Silts.

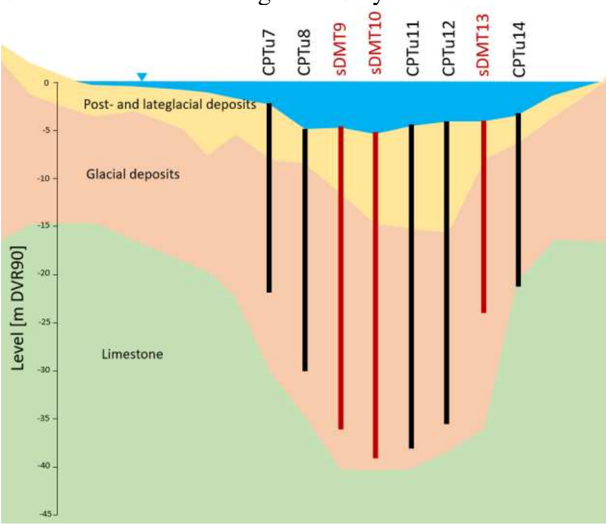

Figure 1. Indicative ground conceptual model with selected investigation locations.

1.3.4. Employer's geotechnical investigations

The Danish Road Directorate has procured several ground investigations associated with the project. One of these investigations, carried out from November 2014 to January 2015, was focussed on investigating ground conditions across the fjord [2].

This investigation consisted of twenty boreholes, seventeen Cone Penetration Test with pore water pressure measurement (CPTu) and three Seismic Dilatometer Tests (sDMT). Within the seventeen boreholes, Standard Penetration Tests (SPT) were carried out, as well as Field Vane Tests (FVT). The location of the in-situ tests analysed in the following is shown in Fig. 1.

In addition to the in-situ tests, laboratory tests were also carried out on retrieved samples. Classification tests were carried out on most samples, and strength and compressibility testing on selected samples. The strength and compressibility tests included seventeen incrementally loaded (IL) oedometer tests, fifteen consolidated anisotropic undrained (CAU) triaxial compression tests, fifteen consolidated isotropic drained (CID) triaxial compression tests, three unconsolidated undrained (UU) triaxial compression tests, and five direct shear (DS) tests.

Further laboratory testing was also carried out as part of this investigation but is not discussed in this paper. The characterisation of the limestone formation is also not covered in this paper.

2. Seismic dilatometer testing

2.1. Description of test

The seismic dilatometer consists of a flat steel blade with a sharp lower edge. The blade has a circular steel membrane mounted flush on one side. The blade is advanced in to the ground using common field equipment, for example a push rig normally used for a CPT test (see Fig. 2).

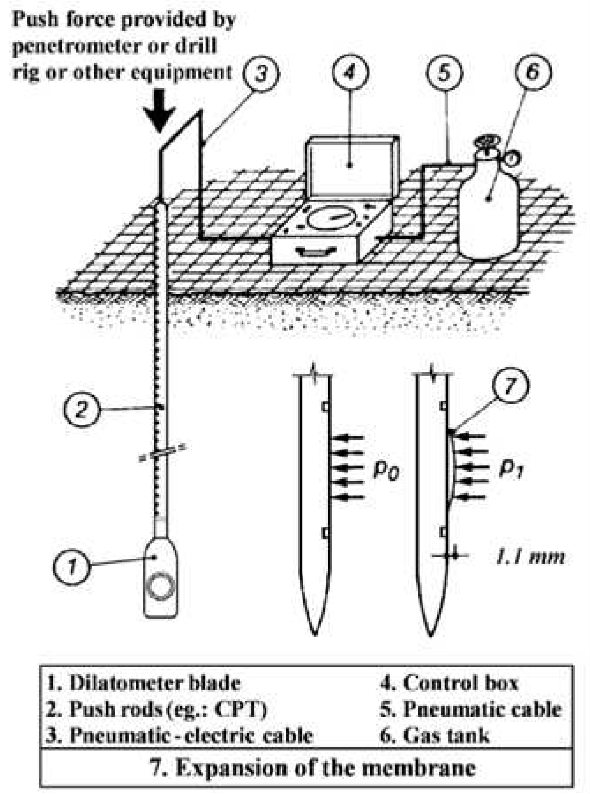

Figure 2. General layout of the dilatometer test **Error! Reference source not found.**.

The test begins by pushing the dilatometer into the ground to the starting depth. The operator then inflates the membrane and takes two readings: p_0, the pressure required to begin to move the membrane, and p_1, the pressure required to move the centre of the membrane out 1.1 mm.

The blade is then advanced further into the ground, typically by 20 cm, and the process is repeated.

The two pressure readings, p_0 and p_1, are used to obtain four index parameters:

- I_d, Material Index;
- K_D, Horizontal Stress Index;
- E_D, Dilatometer Modulus; and,
- U_D, Pore Pressure Index.

Using correlations detailed in [3], these index parameters are used to obtain the following soil parameters:

- M_{DMT}, Oedometer modulus (fine-grained and coarse-grained soils);
- c_u, Undrained shear strength (fine-grained soils);
- K_0, In situ earth pressure coefficient (fine-grained soils);
- OCR, overconsolidation ratio (fine-grained soils); and
- φ, friction angle (coarse-grained soils).

The test procedure described above is used for both the flat dilatometer test (DMT) as well as the seismic DMT, the key difference being that the sDMT has an additional seismic module which measures the shear wave velocity (V_s).

The seismic module consists of an instrumented tube containing two receivers 50 cm apart. The energy source is a hammer striking a base plate pushed flat against the soil surface, which in this case is the seabed. The shear wave velocity is the ratio between the difference in distance between the two receivers and the energy source, and the time delay between the two (upper and lower) receivers.

2.2. Seismic dilatometer tests at Roskilde Fjord

The tests were carried out using the so-called Torpedo method, i.e. starting each set of measurements from the bottom of the borehole casing. At each test location, the first DMT readings were unrealistically small due to soil disturbance. Additionally, at greater depths, the soil was difficult to penetrate so the drill head (driving the push rods) was repeatedly raised and lowered, resulting in disturbance to the penetrated soil.

The seismic tests were carried out when it was possible to penetrate at least 1.0 m past the bottom of borehole casing. Due to the limited penetration rates at greater depths, often the last 10 m of each investigation location has fewer V_s measurements. The interpretation of V_s values was at times problematic due to the soft, nearly liquid, material at seabed surface, and the presence of occasional boulders.

3. Soil characterisation from seismic dilatometer data

3.1. Stratigraphy and soil classification

Table 1 presents the results of the classification tests for the deposits encountered during ground investigations associated with the project. Typical values of the classification parameters are shown in brackets.

Table 1. Main soil classification parameters

Soil type	Water content [%]	Plasticity index [%]	Bulk unit weight [kN/m^3]	Fines content [%]	Clay content [%]
Gyttja and peat	18-289	36-63	10-15	30-82	-
Postglacial and lateglacial coarse-grained deposits	6-28	-	19-21 (20)	0-40 (20)	0-10 (3)
Postglacial and lateglacial fine-grained deposits	9-39	8-15 (10)	18-20 (19.0)	23-88 (30)	18-28 (25)
Meltwater fine-grained deposits	12-34	3-20 (10)	17-21 (20.5)	20-99 (80)	4-50 (25)
Meltwater coarse-grained deposits	2-34	-	17-23 (20.5)	0-65 (3)	0-15 (0)
Glacial fine-grained till	6-24	4-17 (10)	16-23 (21.5)	10-81 (40)	3-24 (15)
Glacial coarse-grained till	5-16	-	20-22 (21.5)	10-40 (25)	3-14 (8)

Seismic Dilatometer Tests (sDMT) were carried out in the central part of the Roskilde fjord. Three locations were selected for sDMT tests: sDMT9, sDMT10, sDMT13. The boreholes adjacent to the sDMT locations allowed the comparison of the stratigraphy based on soil logging to the stratigraphy based on the material index I_D. In Fig. 3 the material index I_D is plotted against depth. From approximately 10 m below seabed, these are generally in agreement with borehole logs where coarse-grained deposits are present.

Between the organic (Postglacial) and coarse-grained material (Meltwater Deposits), fine-grained soils of low plasticity are encountered. Their material index is often similar to those for coarse-grained materials, which makes their identification challenging.

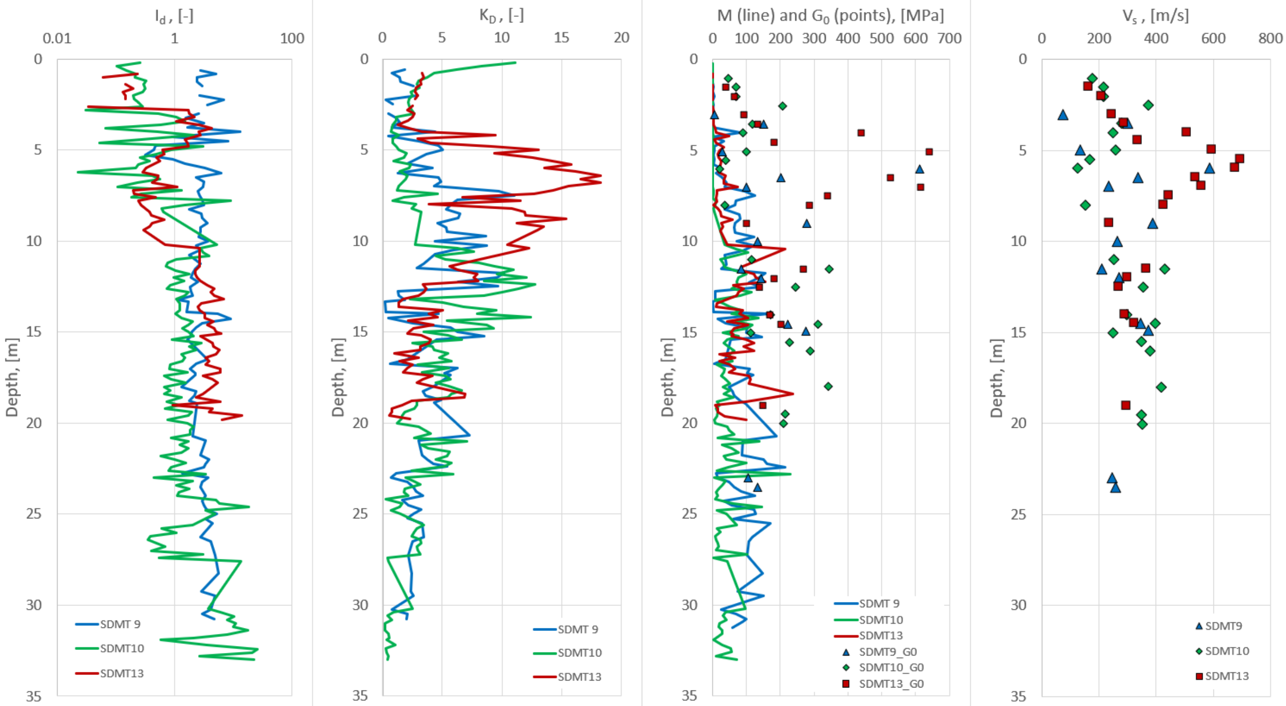

Figure 3. sDMT test results.

Organic soils encountered in the first meters below seabed may be identified by low I_D and K_D values. It should be noted that I_D corresponds more to the soil behaviour rather than to the soil type as defined by classification systems. This is particularly true for fine-grained soils of glacial origin.

Results of CPTu tests and sDMT tests performed in the same layer of coarse-grained deposit are plotted on the CPTu- [4] and DMT-based [5] charts for soil types (Fig. 4 and Fig. 5, respectively). Most of the CPTu results fall into the clean sands to silty sands area. A slightly greater scatter is observed in the sDMT results, however most of those are interpreted as sands, silty sands and sandy silts. Therefore, the two tests give similar results in this coarse-grained deposit.

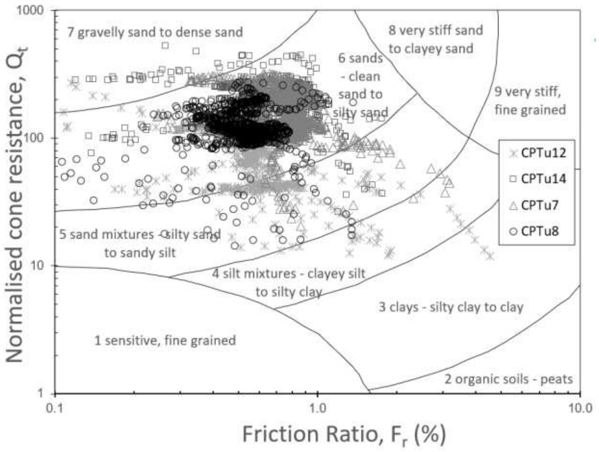

Figure 4. Soil types in accordance with CPTu chart proposed by Robertson [4].

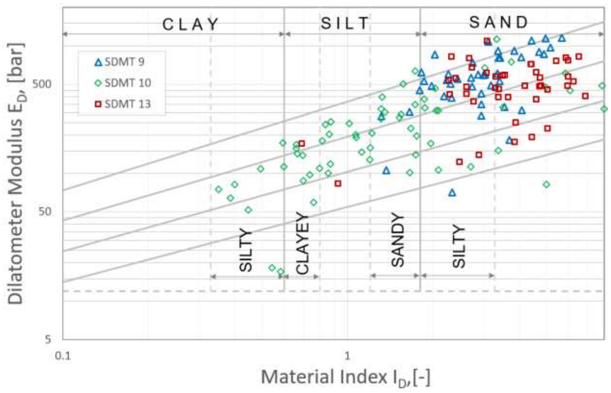

Figure 5. Soil types in accordance with DMT chart proposed by Marchetti and Crapps [5].

It is generally not recommended to perform a sDMT test without associated soil logging since there are specific soils which do not follow the sDMT based chart for soil type. Additionally, and just like for other in-situ tests such as the CPT, the sDMT test results are able to capture the changes in soil behaviour type as opposed to inform on the soil classification.

3.2. Strength

The horizontal stress index K_D can be correlated to the shear strength of soils even though the contact pressure of the membrane does not lead to the failure of the soil. The strength estimated in this manner is comparable to the strength obtained by other methods in which the imposed stresses result in the soil failure.

3.2.1. Undrained strength

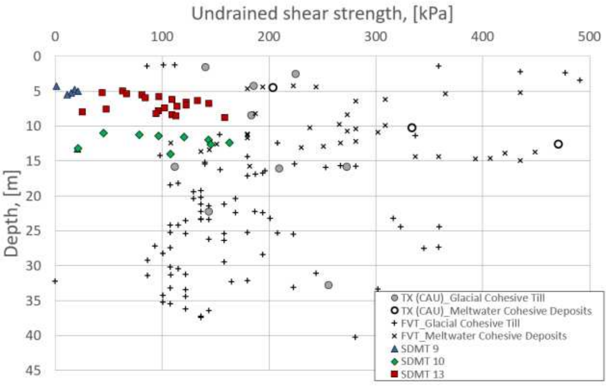

Figure 6. Undrained shear strength obtained from the comparison between sDMT and other types of tests.

The undrained shear strength, c_u, obtained from the sDMT for overconsolidated soils by using the correlation provided in [3] based on vertical effective stress and K_D is plotted in Fig. 6 alongside with undrained shear strengths obtained from CAU and FVT tests. It can be seen that c_u derived from the sDMT is generally lower than that derived from other laboratory or in situ tests. This may be due to the low plasticity of the tested material (<15%) and/or the validity of the input parameters used in the SHANSEP method which is used as the basis of the correlation.

The undrained shear strength of gyttja layer derived from the sDMT ranges from 1 to 8kPa. This is consistent with the c_u values determined in 3no. unconsolidated undrained triaxial tests (UU) which fall within the 3-5 kPa range. It is noted that in each of the UU tests the failure occurred at stress levels close to the resolution of transducers and therefore the actual undrained shear strength may be lower and therefore closer to the lower bound value obtained from sDMT tests.

3.2.2. Effective strength

The effective friction angle of the tested coarse-grained soils, calculated directly from the correlation between K_D and φ' proposed by Marchetti [6] is to be treated as a lower bound value: this caveat appears to be confirmed by isotropically consolidated drained triaxial tests (CID) as shown in Fig. 7. It should be noted that these triaxial tests were executed on reconstituted samples with relative density estimated from SPT tests.

The effective friction angle from the CPTu tests was also estimated using the equation based on the correlation proposed by Schmertmann [7] for uniform fine sands. This equation incorporates a correlation to derive D_r, using the coefficients and relationship proposed by Baldi et al. [8] for Ticino sand and chosen by the ground investigation contractor [2]. The derived φ' from CPTu tests are higher than those obtained from DMT tests.

Values of φ' based on SPT tests with the Stroud correlation [9] for coarse-grained deposits with OCR equal to 3 are rather similar to the ones obtained based on the dilatometer tests (Fig. 7).

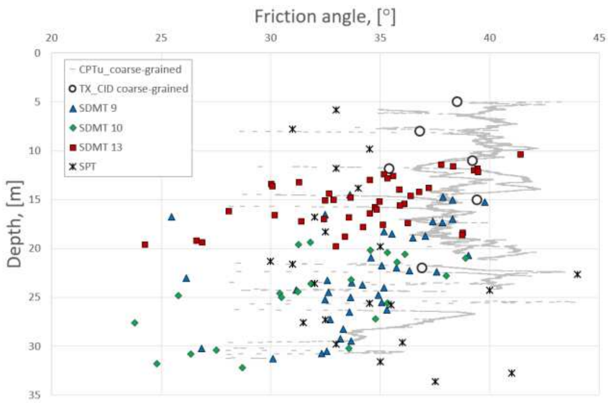

Figure 7. Effective friction angle versus depth obtained from different testing methods

3.3. Compressibility and stress history

One of the advantages of the sDMT tests is the ability to gather information (G_0 from V_s, E_D, and K_D) which allow to estimate soil stiffness and stress history parameters. Due to the lower strain range applied during the test, the sDMT is believed to be much more sensitive to stress history than a CPT test [10].

By combining the results of the standard DMT test with the small strain shear modulus obtained with the sDMT from the shear wave velocity one obtains two points on the shear stiffness degradation curve: one of these points corresponds to the small shear strains and the second to larger shear strains.

Typical ranges of the shear strain and associated stiffness in different soil types have been presented by Amoroso et al. [11, 12] and are briefly discussed herein. These are based on the assumption that the soil follows the theory of linear elasticity with a Poisson's ratio equal to 0.2 in drained conditions. This assumption is made to allow comparison of the soil moduli obtained from sDMT test with those obtained from triaxial and oedometer tests.

In practice the stiffness of the soil varies with the applied load and the stiffness may be different in the horizontal and vertical directions due to anisotropy. All of these remarks should be remembered when comparing values of soil moduli obtained by different methods.

The triaxial and oedometer tests used for comparison with sDMT results were carried out on samples retrieved from boreholes in the fjord area, but not necessarily in the same vertical/depth of sDMT tests.

3.3.1. Stiffness of coarse-grained soils

For coarse-grained deposits, Amoroso et al. [11] suggest that working strains in the range 0.01 % - 0.45 % correspond to the modulus obtained from DMT test. Fig. 8 shows soil shear moduli derived from 3 different methods (V_s measurement, DMT tests, triaxial CID tests), which confirms the conclusions offered by Amoroso et al. [11].

When comparing soil stiffness moduli referred to strain range to the obtained results from triaxial CID tests, it seems that the strain range could be narrowed down, but the extent cannot be clearly defined based on the data available.

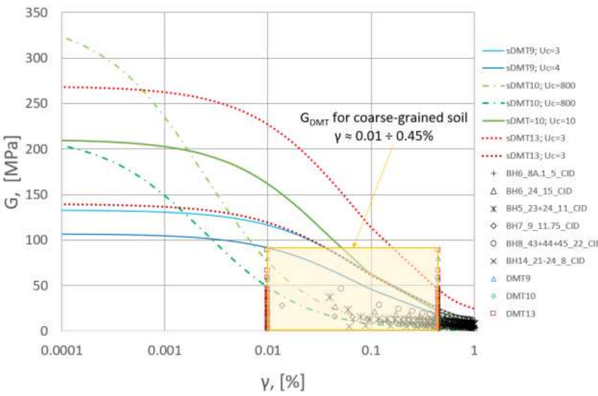

Figure 8. Shear stiffness moduli for coarse-grained deposit based on different testing methods.

Fig. 8 also presents an attempt to recreate the shear stiffness modulus degradation curve for coarse-grained deposits proposed by Menq [13]: this is based on small strain shear modulus G_0 (calculated from V_s, which corresponds to the maximum value in Fig. 8), the uniformity coefficient U_c and the mean effective stress σ'_m. The small strain shear stiffness modulus, G_0, calculated based on V_s measured during the sDMT tests is also plotted versus depth in Fig. 3; its dependence on the changes of the effective stresses is clearly appreciable. The shape of the shear stiffness moduli degradation curves proposed by Menq [13] depends on the uniformity coefficient (U_c). As it can be seen in Fig. 8, some of the V_s measurements were made in heterogeneous deposits with sand layers characterised by a higher amounts of fines (U_c=800). Menq's [13] relationship for coarse-grained materials suggests that for material with higher U_c values, the stiffness modulus degradation starts at lower shear strains.

The stiffness moduli obtained from CID triaxial tests generally fall between the lower and the higher U_c values, though closer to the lower curve. The comparison is affected by several factors, such as the reconstitution of the sand structure with a pre-determined relative density, the impossibility of re-tracing the exact stress history, or the imposing of isotropic stresses. These are some of the known disadvantages associated with laboratory testing on reconstituted coarse-grained soil samples.

3.3.2. Stiffness of fine-grained soils

Fine-grained inorganic materials within the Roskilde Fjord comprise glacial till and meltwater deposits, both of low plasticity (plasticity index typically less than 15%). In order to present stiffness degradation curves more clearly (Fig. 9), the fine-grained soils were grouped in accordance with their depositional environment. The variation of the small strain shear stiffness (G_0) versus depth is shown in Fig. 3. As it may be observed, there is more than one value shown per soil type: this is possibly due to the uncertainty in deriving shear wave velocity V_s and also due to the variable nature of the soil or mean effective stress. The largest differences in small strain shear moduli values are observed for glacial till for which only 7no. measurements are available on a 4.3 m thick layer; the inherent heterogeneity of this type of deposit could also be affecting the data scatter.

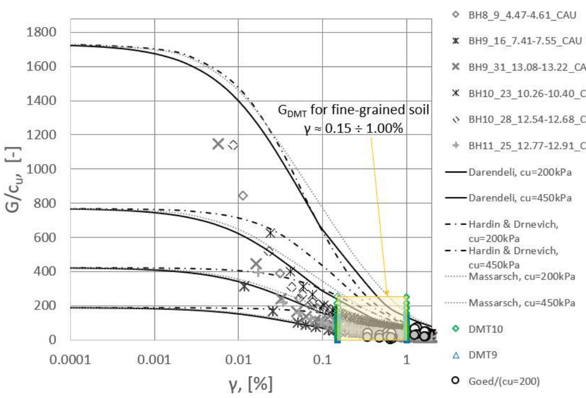

Figure 9. Normalised shear stiffness for fine-grained meltwater deposit based on different testing methods.

Due to the limited number of sDMT measurements in glacial till, only the modulus based on V_s data for fine-grained meltwater deposits are presented in Fig. 9. The stiffness moduli from IL oedometer tests were calculated for stress increases between the in situ stress and 600 kPa, and between the in situ stress and two times the in situ stress.

The stress-strain relationship is most commonly described by hyperbolic models incorporating additional soil parameters and modified by many researchers like Hardin and Drnevich [14], Massarsch [15], Darendeli [16] and Menq [13], shown herein. The stiffness moduli values shown in Fig. 9 are normalized by the undrained shear strength of the soil, which in the case of the triaxial CAU tests refers to the values obtained from the tests, while the stiffness moduli obtained from shear wave velocity, DMT and oedometer tests are normalized by upper (450 kPa) and lower (200 kPa) bound values chosen for the analysed material based on triaxial CAU and FVT data. It should be noted for clarity that the re-evaluated stiffness moduli degradation curves are based only on the upper and lower bound values of G_0 calculated for the measured shear wave velocity values. A plasticity index (PI) of 10 % and an over-consolidation ratio (OCR) of 8 have been assumed (based on IL oedometer tests) as representative values for use in the Massarasch [15] and Darendeli [16] correlations.

Based on the findings from [11, 12], the modulus from DMT tests should correspond to working shear strains ranging from 0.15 % to 1.00 %. As presented in Fig. 7 the shear strain range for analysed deposits of the moduli obtained from DMT tests is similar to that found by Amoroso et al. [12].

The most pronounced non-linearity of the stiffness is observed between 0.02 and 1 % shear strain for the analysed soil type.

Marchetti [17] stated that a large number of case histories have generally proven the favourable comparisons between observed settlement and DMT-predicted primary settlements, thereby supporting the use of DMT as operative constrained modulus. It is noted that this could allow more cost effective ground investigations.

3.3.3. Stress history

As mentioned previously, the stress history of the soil is another key parameter that can be obtained from DMT tests for soils with material index I_D lower than 1.2, i.e. for fine-grained deposits. The DMT tests results obtained for organic deposits (gyttja) confirmed its normally consolidated state (Fig. 10). Where an OCR greater than 1 is estimated, this may have been caused by the insertion of DMT blade and local consolidation of this deposit near the fjord bed. A large variability of the OCR values obtained from DMT tests was found for the inorganic fine-grained soils as would be expected for an over-consolidated deposit at shallow depths. This variability seems to decrease with the depth of the test. It is noted that several OCR values shown in Fig. 10 relate to soil with an I_D close to 1.2 which indicates a material with a significant content of silt within coarse-grained units, as informed by soil logging. Overall, the OCR values obtained from DMT tests fit reasonably well within the upper bound values obtained from the IL oedometer tests.

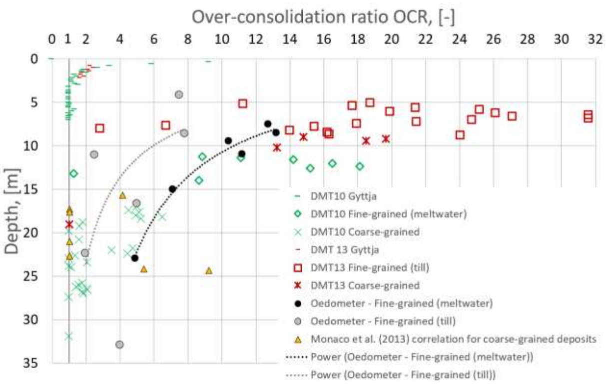

Figure 10. Over-consolidation ratio based on different approaches.

Monaco et al. [18] proposed a correlation for coarse-grained deposits between OCR and the ratio of constrained modulus obtained from DMT and the corrected cone resistance from CPT test (M_{DMT}/q_t). The M_{DMT} and q_t values have been selected to verify the applicability of this correlation to the coarse-grained meltwater deposits of this project. Only data values from soil layers of significant thickness and with I_D values above 1.8 have been analysed. For the analysed coarse-grained meltwater deposits the M_{DMT}/q_t ratio is usually below 5. In Fig. 10 OCR values for coarse-grained deposits are shown only for the few data with the M_{DMT}/q_t ratio above 5, as the correlation proposed by Monaco et al. [18] is not intended for lower M_{DMT}/q_t ratios. The fact there are ratios below 5 may be explained by higher relative density D_r of the analysed coarse-grained meltwater deposits compared to D_r of the deposits analysed by Monaco et al. [18] and high q_t values compared to M_{DMT}. With reference to [3] M_{DMT}/q_c ratios below 5 would not be expected for coarse-grained deposits, especially for this project were these layers are generally overconsolidated based on the geological history of this area and the results of the oedometer tests performed on samples of overlying fine-grained meltwater deposits (see Fig. 10) at depths shallower than 20 m. The dependence of the stress level, the relative density, sand type and cementation on the correlation in between M_{DMT}/q_t and OCR for coarse-grained deposits, as indicated by Jamiolkowski et al. **Error! Reference source not found.** shall be studied further.

4. Conclusions

The results of the sDMT tests carried out for the Fjord Link Frederikssund project were compared to borehole logs, laboratory testing and other in situ testing. In particular, soil classification, undrained shear strengths, effective friction angles, shear stiffnesses and stress histories were examined where results were available.

Soil classification in the DMT tests is based on I_D, the material index. It was found that deeper than 10 m below seabed there was good agreement between soil type based on I_D and stratigraphy encountered in adjacent boreholes. In comparing the material indices to those obtained from the CPTu tests it was found that, while the DMT results showed a slightly greater scatter, both CPTu and DMT results described the majority of soil material as silty sands and sandy silts.

The undrained shear strength of over-consolidated soils derived from the DMT correlation were generally found to be lower than those obtained from direct laboratory or field tests. This may be due to the low plasticity of the investigated materials, or the parameters selected for the SHANSEP correlation which constitutes the basis for the DM correlation. While this suggests that the DMT results yield conservative undrained shear strengths, further assessment is recommended.

The effective friction angles obtained from the DMT for coarse-grained deposits were also found to be lower than those obtained from laboratory triaxial tests as well as from CPTu correlations. However, the triaxial tests were carried out on reconstituted samples, which may not be fully representative of the in situ conditions. The effective friction angle obtained from SPT tests using the Stroud correlation are similar to those from the DMT tests.

As regards stress history the OCR values obtained from the DMT tests for fine-grained deposits were generally found to fit well within the ranges of the values obtained from the IL oedometer test. The possibility of estimation of the OCR values based on DMT tests shall be studied further for coarse-grained deposits with high relative density.

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References

[1] Håkansson, E., Pedersen, S. A. S. "Geologisk kort over den Danske Undergrund, 1:500 000, kort", (Bedrock geology of Denmark, 1:500 000, map), Varv, Copenhagen, Denmark, 1992.

[2] Pedersen, A. B., Galsgaard, J., Regel, J. B. "Roskilde Fjord Forbindelsen - Geotechnical investigations, Geotechnical factual report – final report", GEO, Kongens Lyngby, Denmark, Rep. 38042, 2015.

[3] TC16 2001 "The flat dilatometer test (DMT) in soil investigations", International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) TC16, Rep. TC16 DMT Report, 2001.

[4] Robertson, P. K. "Soil classification using the cone penetration test", Canadian Geotechnical Journal, 27(1), pp. 151-158, 1990, https://doi.org/10.1139/t90-014
[5] Marchetti, S., Crapps, D. K. "Flat Dilatometer Manual", Internal Report of GPE Inc., Gainesville, Florida, USA, 1981.
[6] Marchetti, S. "The flat dilatometer: design applications", In: Third geotechnical engineering conference Cairo University, Cairo, Egypt, 1997, pp. 421-448. Available at: http://www.marchetti-dmt.it/wp-content/uploads/bibliografia/marchetti_1997_SOA_cairo.pdf Accessed: 30 September 2019
[7] Schmertmann, J. H. "Guidelines for CPT performance and design", Federal Highway Administration, Washington, DC, United States, Rep. FHWA-TS-78-209, 1978._
[8] Baldi, G., Bellotti, R., Ghionna, N., Jamiolkowski, M., Pasqualini, E. "Interpretation of CPT's and CPTU's. 2nd part: Drained penetration of sands", In: Fourth international geotechnical seminar field instrumentation and in-situ measurements, Singapore, 1986, pp. 143-156.
[9] Stroud, M. A. "The Standard Penetration Test – its application and interpretation" In: Proceedings of the geotechnology conference organized by the institution of civil engineers, Thomas Telford Limited, London, United Kingdom, 1989, pp. 29-49.
[10] Schmertmann, J. H. "Comparing DMT with CPT in NC/OC Sand Bucket Tests", GPE Inc., Gainesville, Florida, USA, Rep. DMT Digest No. 4, 1984.
[11] Amoroso, S., Monaco, P., Marchetti, D. "Use of the Seismic Dilatometer (SDMT) to estimate in situ G-γ decay curves in various soil types", In: Coutinho R. Q. & Mayne P. W. (eds.) Geotechnical and Geophysical Site Characterization 4, 1st ed., Taylor & Francis Group, London, United Kingdom, 2013, pp. 489-497. Available at: https://www.researchgate.net/publication/264905759_Use_of_the_Seismic_Dilatometer_SDMT_to_estimate_in_situ_G-g_decay_curves_in_various_soil_types Accessed: 30 September 2019
[12] Amoroso, S., Monaco, P., Lehane, B. M., Marchetti, D. "Examination of the potential of the Seismic Dilatometer (SDMT) to estimate in situ stiffness decay curves in various soil types", Soils and rocks, 37(3), pp. 177-194, 2014. Available at: https://www.researchgate.net/publication/275960356_Examination_of_the_Potential_of_the_Seismic_Dilatometer_SDMT_to_Estimate_In_Situ_Stiffness_Decay_Curves_in_Various_Soil_Types Accessed: 30 September 2019
[13] Menq, F. Y. "Dynamic Properties of Sandy and Gravelly Soils", Ph.D. dissertation, The University of Texas at Austin, USA, 2003. Available at: https://repositories.lib.utexas.edu/bitstream/handle/2152/779/menqf032.pdf?sequence=2&isAllowed=y Accessed: 30 September 2019
[14] Hardin, B. O., Drnevich, V. P. "Shear modulus and damping in soils: design equations and curves", Journal of Soil Mechanics and Foundations Division, ASCE, 98(SM7), pp. 667-692, 1972.
[15] Massarsch, R. "Deformation properties of fine-grained soils from seismic tests" In: International conference on site characterization ISC'2, Porto, Portugal, 2004, pp. 133-146. Available at: https://www.academia.edu/9979356/Deformation_properties_of_fine-grained_soils_from_seismic_tests Accessed: 30 September 2019
[16] Darendeli, M. B. "Development of a new family of normalized modulus reduction and material damping curves", Ph.D. dissertation, The University of Texas at Austin, USA, 2001. Available at: https://repositories.lib.utexas.edu/bitstream/handle/2152/10396/darendelimb016.pdf Accessed: 30 September 2019
[17] Marchetti, S. "Some 2015 updates to the TC16 DMT report 2001", In: The 3rd international conference on the flat dilatometer DMT'15, Rome, Italy, 2015, 43–65. Available at: http://www.marchetti-dmt.it/wp-content/uploads/marchetti-2015-update-TC16-DMT15.pdf Accessed: 30 September 2019
[18] Monaco, P., Amoroso, S., Marchetti, S., Marchetti, D., Totani, G., Cola, S., and Simonini, P. "Overconsolidation and Stiffness of Venice Lagoon Sands and Silts from SDMT and CPTU", Journal of Geotechnical and Geoenvironmental Engineering, 140(1), pp. 215-227, 2014, https://doi.org/10.1061/(ASCE)GT.1943-5606.0000965
[19] Jamiolkowski, M., Ghionna, V. N., Lancellotta, R., and Pasqualini, E. "New correlations of penetration tests for design practice", In: 1st international symposium on penetration testing, ISOPT-1, Orlando, USA, 1988, pp. 263-296, https://doi.org/10.1016/0148-9062(90)95078-F