FOUNDATION ON ORGANIC DIATOMITE

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KEYWORDS

Diatomite, organic, interglacial, strength, deformation, pre-consolidation, foundation, piling.

ABSTRACT

In glacially formed landscapes, organic soils of interglacial age are sometimes found in geotechnical site investigations for building projects. The thickness of such layers is often minor and the pre-consolidation of the layers from e.g. ice transgressions in glacial periods produces soil parameters that are often acceptable for foundations of structures with delimited loads in soils above the interglacial organic layer.

On a building project in the Danish town Vejle, an unusually thick layer of interglacial organic diatomite was found. Furthermore, the geology was surprisingly complex on the ~40 meter wide building site – with large variations over short distances – included layers of soft postglacial organic soils and dense sands with varying thicknesses.

Diatomite is not commonly found in Denmark and maybe as a consequence not well described in the geotechnical literature. This – besides the local found thickness of the formation and significant classification parameters – called for further investigations of the geotechnical properties, also to comply with the demand of testing in geotechnical category 3 in the European geotechnical standard.

In this article, the geotechnical characteristics and achieved knowledge of the found organic diatomite are described. The investigation includes borings, CPTs, laboratory classification tests, oedometer and triaxial tests and pile driving results with pile driving analyses.

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1. INTRODUCTION

This article is initiated by a project in Vejle in Denmark, where a sewage treatment plant in 2023 was to be enlarged by a new tank and an adjacent pumping station.

At first, a borehole in the center of the tank area showed several meters of fill and postglacial organic soils, which brought up the necessity of a piling foundation into subsoils of sand. The following extended geotechnical site investigation for the design of the project revealed unexpected differing ground conditions within the relatively narrow building site, including the presence of relatively thick interglacial layers of organic diatomite. Although diatomite is well-known in this part of Denmark, it is not commonly found and handled. The geotechnical properties of diatomite have not previously been investigated deeply and, as a consequence, are not well described in the literature. As the characteristics of the diatomite were essential for the project, an extended investigation was necessary to comply with the demand for testing in geotechnical category 3 in the European geotechnical standard.

The authors aim with this article to present the knowledge gained about diatomite soils in order to increase the general knowledge for future projects involving diatomite soils.

2. PROJECT AND GEOTECHNICAL INVESTIGATIONS

The project comprised an expansion of an existing sewage treatment plant in the Danish town of Vejle, where a storage tank and an adjacent pumping station were to be built. The tank has a diameter of 34 meters to approx. 3 meters depth at terrain levels +2 to +3 m DVR90 (meter above sea level). The pumping station is 7 m by 8 m and taken to approx. 7 m depth.

Initially, a geotechnical boring (B1) to 20-meter depth had been executed. The boring showed 5 meters of fill and 4 meters of postglacial marine and freshwater deposits of organic soils (including peat and gyttja) over inorganic marine and meltwater dense sands to 16 meters depth. Below this, clay and gyttja of assumed Miocene age were described. Based on these results, a foundation on driven concrete piles was presumed, with pile toes in the sand. The groundwater table was found at a depth of 1.5 meters, which called for groundwater lowering in the construction phase, especially for the pumping station.

Afterward, Geo performed a geotechnical parameter investigation for the project, as shown in Figure 1 (ref. 3). First, three Cone Penetration Tests (CPTu 101 – 103) were performed at the perimeter of the tank area, where a penetration depth of 30 meters was presumed sufficient and abundant. The evaluation of the CPTu's showed some severe variations in the layering and soil firmness compared to the initial boring. It was found necessary to supplement these with two borings (104 and 105), originally solely planned for pumping tests, but deepened to 25 and 40 meters depth and expanded to include fast field vane tests and extraction of soil samples for advanced laboratory testing.

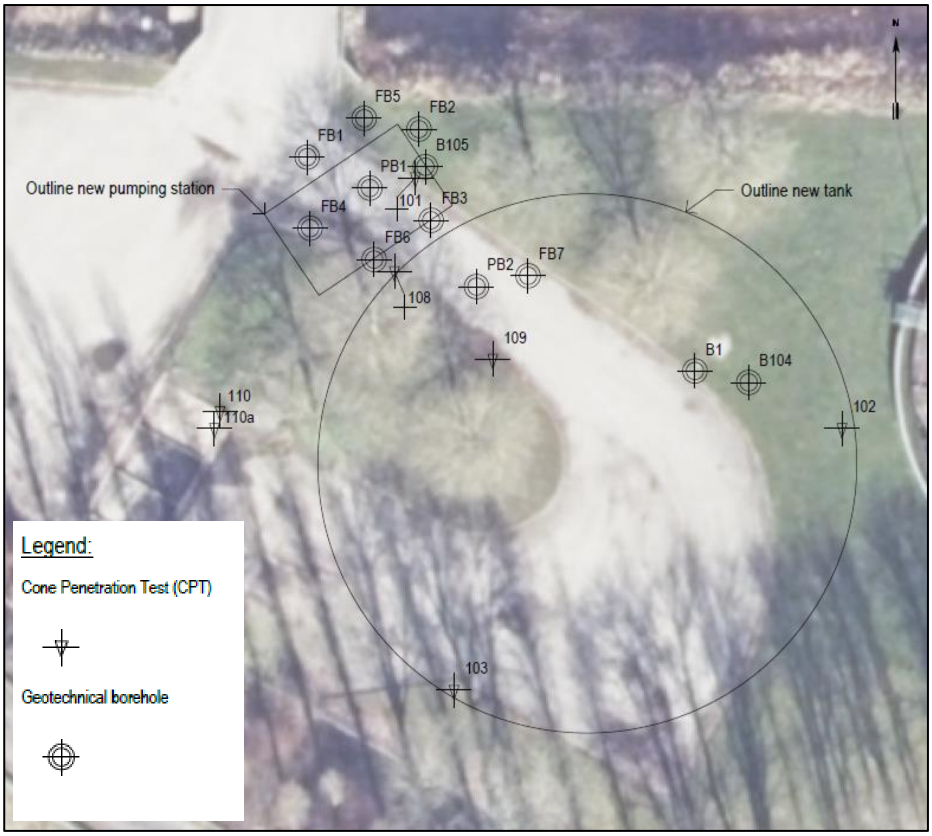

Figure 1: Site plan with borings and CPTu's.

The geological description of the extracted samples showed dominating layers of organic diatomite of interglacial age. Furthermore, the thickness of the sand layer – in which the piling was planned – seemed to thin-out towards one side. After driving of 24 relatively short test piles equally spread in the tank area, three supplementary CPTu's (108 – 110) were performed in a part of the tank area, where the sand layer was very thin or even absent, followed by driving of five extra 30 meter test piles. The long piles were testing with pile driving analyses (PDA) six days after installation with CASE and CAPWAP analyses. For groundwater lowering, additionally nine borings to 13 – 23 meters depth, especially in the area of and around the pumping station, have been executed. Besides a geological description of all extracted samples to establish a geological model for the site, special focus has been on evaluating the geotechnical parameters of the found organic diatomite. Besides a number of classification tests (water content, density, plasticity index and loss of ignition), oedometer tests and later triaxial tests have been made on the extracted soil samples of diatomite. The results of the investigations is further described in the following, focusing on the organic diatomite.

3. DIATOMITE IN GEOLOGIC CONTEXT

In Denmark, freshwater diatomite is found in south-western, se Figure 2, deposited as non-calcareous sediments from differing interglacial or interstadial periods and as marine deposits from the geological Paleogene period in the north-western part. Diatomite is a fine-graded, biogenic sediment that predominantly consists of shells from microscopic algae, also called diatoms. Diatom shells are a so-called opal-A, a silicate with very high water content in the chemical form $SiO_2 – nH_2O$, an inorganic biogenic material. Pure dry diatomite is able to float on water due to its high porosity. Natural diatomite deposits contain variable levels of finely divided organic material and minerals of silt and clay, typically deposited during winter periods between summer algae blooms. If the geotechnical characteristics of the sediment are dominated by high organic content, it is tradition in Denmark to use the term diatomite gyttja.

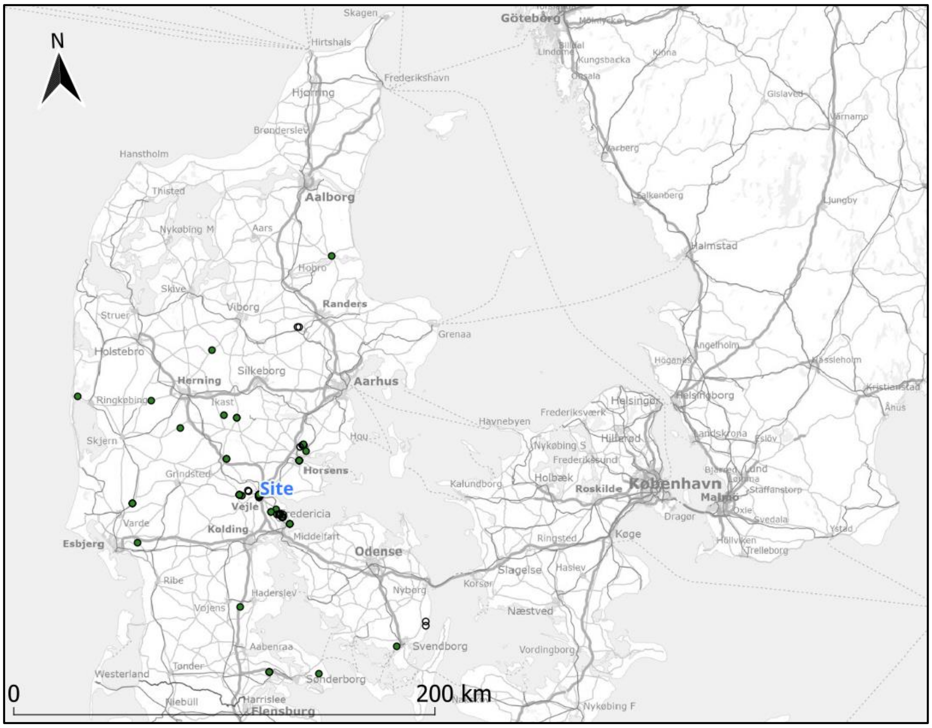

Figure 2: Location of boreholes with interglacial and interstadial freshwater diatomite in Denmark according to the GeoAtlas Live database (green dots), combined with locations of known diatomite pits (open rings). The site location (Figures 1 and 3) is marked with blue text.

Diatomite absorbs water very easily. In saturated state, the water content is very high and the unit weight accordingly low. For the same reason, diatomite is often confused with the more commonly found organic sediments such as gyttja. Like organic material, diatomite holds water in a way that does not contribute to the plasticity. The diatomite shells are hollow, and the encapsulated water increases the plasticity and liquid limits, but pure diatomite is non-plastic.

The site of interest is located on the northern edge of a known diatomite area in the so-called Mølholm Valley, which runs into the broader Vejle Fjord Valley, as shown in Figure 3. In previous boreholes in the Mølholm Valley, the diatomite is typically covered by postglacial organic deposits and occasionally thin layers of tills. Presumably, the diatomite extends at similar depths beyond the sides of the valley, and the postglacial formation of the Mølholm Valley eroded down to the diatomite surface.

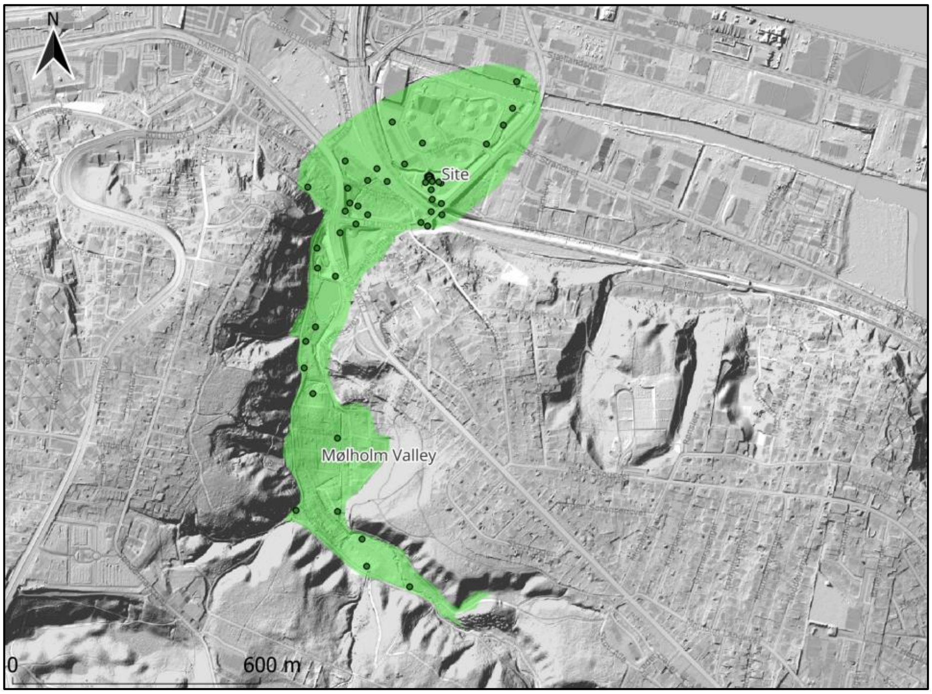

Figure 3: The extent of diatomite in the Mølholm Valley in Vejle. Green dots mark boreholes with diatomite found in GeoAtlas Live, and the green shading marks the presumed extent of shallow diatomite occurrences, covered by thin younger deposits.

The larger extent agrees with interglacial diatomite findings from similar sites in the Vejle Fjord area (ref. 1 and 2), where it is dated through pollen species assemblages to Holstein interglacial (424,000 – 374,000 yr BP) or from an intermittent warmer period within the Saale glacial complex (373,000 – 130,000 yr BP, ref. 1 and 2). The most recent glacial period, Weichsel (115,000 –

11,700 yr BP) resulted in three glacial advances over the area, forming the main Vejle Valley through subglacial meltwater erosion, making the diatomite deposits susceptible to glacial deformation and tectonism.

4. CLASSIFICATION TESTS

The geotechnical classification parameters of the found diatomite are listed in Table 1. Density is measured on intact tube samples at advanced laboratory tests, water contents are measured by dry-out in an oven at 105 °C, plasticity tests are performed by the fall cone method, and loss of ignition measured as weight loss after four hours at 550 °C (on oven-dried material).

Table 1: Classification parameters of diatomite.

Parameter	No. of measurements	Measured values
Density, ρ (g/cm^3)	12	1.18 – 1.26
Water content, w (%)	107	24 – 248
Plasticity limit, w_P (%)	7*	41 – 231
Liquid limit, w_L (%)	7*	22 – 212
Plasticity index, I_P (%)	7*	16 – 29
Loss of ignition, LOI (%)	13	7.4 – 17.5

Note *: Seven of 13 tested samples were found plastic (six were non-plastic).

Figure 4 shows a Casagrande plasticity chart, and all tests are positioned below the so-called A-line. Nearly all shown points represents samples from borehole 105, while nearly all tested samples from borehole 104 were non-plastic. This demonstrates – besides various CPT cone resistances – a variability of the diatomite across the site. The plasticity may be affected by a content – or absence – of e.g. clay and organic material. The loss of ignition shall be seen as the sum of chemically bounded water and organic material.

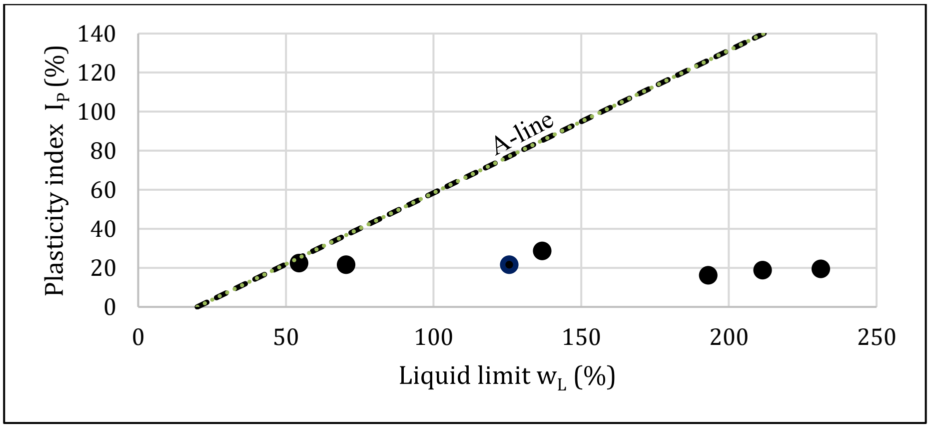

Figure 4: Casagrande plasticity chart for organic diatomite (with A-line).

5. OEDOMETER TESTS

To evaluate the deformation parameters in the diatomite, three oedometer tests are performed on extracted soil samples in Shelby tubes. Initially, tests have been made at incremental loading up to 3200 kPa, primarily for estimation of the pre-consolidation stress, according to methods by Casagrande, Akai and Becker/Janbu. Afterwards, new partial samples from the same three Shelby tubes are tested to evaluate the oedometer modulus at the actual vertical stress level by incremental loading up to near/below the pre-consolidation stress, stepwise unloading to a stress level (150 kPa) slightly below the evaluated vertical in situ stress and stepwise re-loading to the pre-consolidation stress. The main results are listed in table 2. The tests are performed at room temperature 22 °C.

Table 2: Results of oedometer tests.

Borehole no.	104	105	105
Level (m DVR90)	-17.4	-23.0	-25.3
Water content, prior to testing, w (%)	152/159	171/164	178/180
Loss of ignition, LOI (%)	11.3	13.8	16.2
Plasticity index, I_P (%)	NP	16.1	19.4
Measured fast field vane strength, c_{fv} (kPa)	(400?)	160	170
Estimated effective in situ stress, σ'_0 (kPa)	200	220	225
Evaluated pre-consolidation stress, σ'_{pc} (kPa)	1100	900	675
Over-consolidation ratio, OCR (-)	5.5	4.1	3.0
Compression ratio, C_{CR} (% per load decade), virgin line, max.	44.2	31.5	32.5
Coefficient of secondary compression, C_α (% per time decade), virgin line, max.	1.97	1.49	1.56
Oedometer modulus, re-loading, E_{oed} (kPa)	47100	40100	35700

The oedometer tests show that the diatomite are over-consolidated by a ratio OCR = 3.0 – 5.5. The oedometer modulus (for re-loading) E_{oed} = 35 – 47 MPa indicates a relatively high stiffness, despite the high water content (w = 152 – 180 %) and the high loss of ignition (LOI = 11 – 16 %). The drainage of the diatomite during primary consolidation progressed relatively fast, and the coefficient of secondary compression during re-loading was relatively minor.

These findings correspond well with the presumption that the diatomite has a skeleton of silicon shells, where pore water and organic material mainly are located in cavities inside the shells.

6. TRIAXIAL TESTS

Six triaxial tests have been carried out on two intact samples from borehole 104 at levels -21.5 and -23.5 m DVR90. Hereof, three tests are performed drained (CID_c) and three undrained (CIU_c) after isotropic pre-consolidation.

The tests are carried out by principles given in DS/EN ISO 17892-9 and Danish tradition. According to Danish tradition the test conditions are height H = diameter D = 70 mm, with smooth end platens and drainage both from the ends and radially. The specimens are installed in the triaxial apparatus and saturated with backpressure (500 kPa) at the isotropic stress state $\sigma'_1 = \sigma'_3$ = 20 kPa. Afterwards the specimens are consolidated at a cautious mean effective pre-consolidation stress determined as $\sigma'_{m,pc} = 1/3\ (\sigma'_{pc} + 2K_0\sigma'_{pc})$ in order to approximately re-established the effect of the ice-load etc. and minimize sample disturbance, where σ'_{pc} is estimated from the oedometer tests and K_0 is estimated to 0.4. At this stress level, time curve is recorded and shear rate to be used in the shear phase is derived using the principles in DS/EN ISO 17892-9 for each test. The shear rates used are 1%/hour in all tests. Before shearing, the specimens are unloaded isotropically to selected stress levels in the range of the in situ stress using $K_{0,pc}$ = 1.0. The shearing is stopped at approximately 20% additional vertical strain. The effective stress paths during shearing for all the tests are plotted in figure 5.

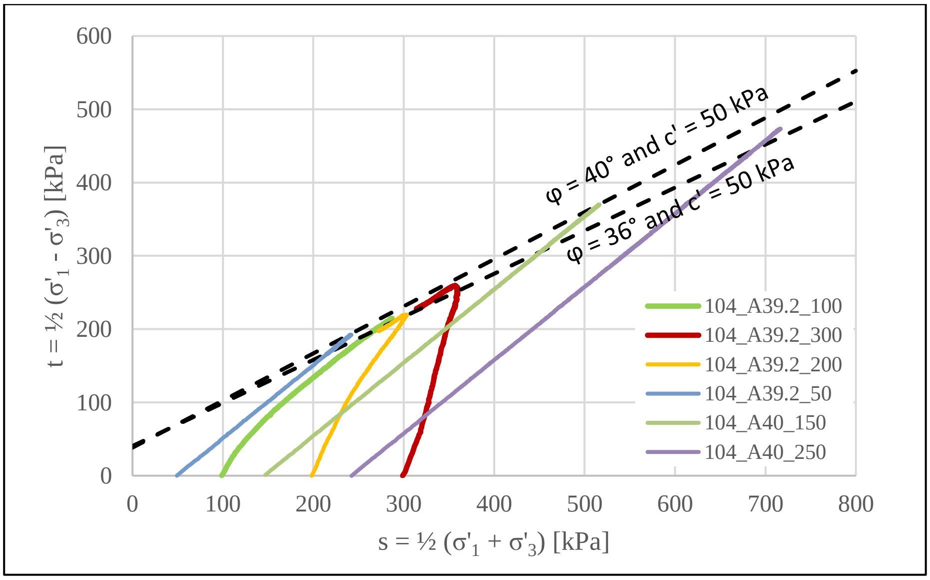

Figure 5: Effective stress paths in shearing for drained and undrained tests.

As seen in figure 5 a common failure criterion for both drained and undrained tests (Mohr-Coulomb: $\tau = c' + \sigma' \cdot \tan\varphi$) can be described using a friction angle φ' = 36° to 40° and an effective cohesion c' = 50 kPa. It is noticeable that the drained test with σ'_3 = 50 kPa and the undrained test with σ'_3 = 100 kPa show dilative behavior (volumetric expansion) during shearing, while the other tests only show tendency to volumetric contraction during shearing, although all the tests are performed on pre-consolidated specimens and therefore expected to show dilative behavior during shearing.

The drained tests were made on two samples from level -21.5 and -23.5 m DVR90. After isotropical pre-consolidation and unloading to axial stress approx. 50, 150 and 250 kPa, drained compression tests were taken to failure measuring the axial stress σ_A, radial stress σ_R and pore pressure.

The undrained tests were made on one sample from level -21.5 m DVR90. After isotropical pre-consolidation and unloading to axial stress approx. 100, 200 and 300 kPa, undrained compression tests were taken to failure. The undrained shear strength is calculated traditionally at 10% additional axial strain as $c_u = ½(\sigma_A - \sigma_R) = 213 – 259$ kPa.

It is notable, that the plasticity index was aimed measured on the tested sample, but that the diatomite in these samples was found to be non-plastic. A friction angle φ' = 36 – 40° seems reasonable for non-plastic soils, the found cohesion c' = 50 kPa (maybe due to a curved failure line at small stresses) and the measured undrained shear strength collides by principle with this.

7. CPT

Six CPTu's have been performed at the site, all partly or fully penetrating the organic diatomite. In the diatomite, all CPTu's show friction ratios $R_f = f_s/q_c$ in the range of typically 2 – 5 % and evaluated soil behavior index (I_{SBT} = 2.6 – 3.2), which – according to normal evaluation procedures – indicates that the diatomite is to be seen as a cohesive soil. This contradicts partly with the evaluation of non-plasticity of some samples, where less friction ratios and frictional behaviour would be expected.

Figure 6 shows the cone resistance q_c in layers, which are interpreted as diatomite. A remarkable variation is seen across the 40-meter wide site with the lowest resistances towards north-west.

It is likely that various geological processes at the particular site have had a major impact on the development of a complex geological soil scene, which is difficult to reveal in detail. The site is located at the edge of a known glacial tunnel valley, where erosion in a bed of previously deposited interglacial/interstadial layers at the edge of the tunnel is followed by depositing of sands under differing sub-glacial drainage patterns. Furthermore, the area may have been feed with eroded material from surrounding areas, and wash-downs and solifluction of soils may have taken place.

Another obvious reason for the variation may be differences in plasticity and differing pre-consolidation stresses, where large q_c-values are typically found in the area with overlying (heavy) sand and non-plastic diatomite, and minor values in area with absence of overlying sand.

The geological complexity is also seeing in the over-lying postglacial deposits, which is interpreted to consist of a variability of marine and freshwater deposits (in this inner part of Vejle Fjord), ranging from in-organic sands to organic deposits (peat, gyttja etc.) to various depths.

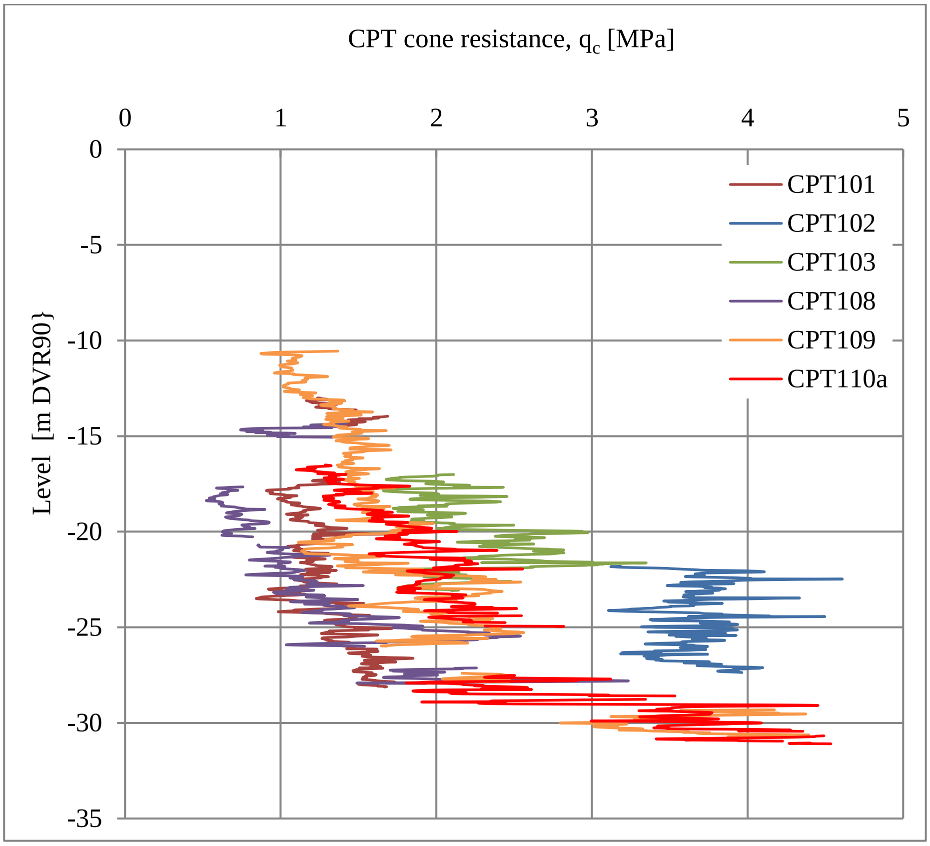

Figure 6: Cone resistance q_c in soil layers interpreted as organic diatomite.

It is notable that major variations in soil profiles are found even over short distances, which in the case of CPT 101 and borehole 104 was less than 2 meter horizontally. The relatively higher cone resistances in CPT 102 may be affected by variations in the plasticity (or even non-plasticity).

8. FAST FIELD VANE TESTS

Fast field vane tests have been performed in the diatomite in two geotechnical boreholes (104 and 105). The tests are performed according to the procedure in the reference sheet from the Danish Geotechnical Society with measurements of undisturbed (c_{fv}) and remoulded (c_{rv}) resistances. The calculated sensitivity is $S = c_{rv}/c_{fv} \approx 0.5$, which normally indicates a mainly cohesive soil.

Comparison of field vane tests and cone resistances is made by the formula $c_{fv} = (q_c – \sigma_{v0})/N_k$. Borehole 105 is located close to CPT 101 (at approx. 1.5 meters inter-distance), where data is available from level -21 to -28 m DVR90. Here, $c_{fv} \approx 150$ kPa and $q_c \approx 1.2$ to 1.5 MPa produces $N_{kt} \approx 7 – 10$, which is in an expected range compared to normal cohesive soils.

Furthermore, field vane results in borehole 104 may be compared to CPT 102 (at 6 meter inter-distance). Here, a direct comparison is weakened due to interlayered sand, though. The field vane tests show predominantly $c_{fv} \approx 310 –$ 410 kPa (from -16 to -23 m DVR90), and the CPT shows $q_c = 3.1 – 3.8$ MPa at the same levels, but with a layer of sand between level -18 and -22 m DVR90. It seems reasonable to adopt the devisor $N_k \approx 7 – 10$ from above.

A comparison of field vane tests with the undrained triaxial tests from borehole 104 shows $c_{fv} \approx 350 – 407$ kPa at levels around the performed undrained triaxial tests with $c_{u,tr} \approx 230$ kPa (average value). Using the formula $c_{u,tr} = \mu \cdot c_{fv}$, this produces a factor $\mu \approx 230 / (350 – 407) \approx 0.57 – 0.66$. This seems to meet a usual understanding of $\mu \leq 1.0$, but the data is evaluated dubious for further elaboration, e.g. regarding the found non-plasticity of the tested sample.

9. PILE BEARING CAPACITIES

The pile foundation of the actual tank was planned on relatively short pile (mostly 11 – 13 meter), primarily toe bearing in the sand (above the diatomite). According to normal practice, the axial bearing capacity was evaluated on the basis of the dynamic pile driving resistance and the Danish pile driving formula. Settlements of the piles – including contributions from the diatomite below – was evaluated negligible for the actual loads, according to the performed oedometer tests.

Contrarily to this, the geotechnical investigations had revealed that the layer of sand was practically absent in approx. one fourth of the tank area towards north-west. Here, the postglacial deposits were interpreted directly underlain by organic diatomite. Furthermore, the CPTu's had shown that several meters of the diatomite were relatively soft, but seemed to be firmer at depth, where the diatomite were underlain by sands and presumably Miocene mica clay. Due to a risk of differential settlements for the tank as a whole, it was recommended not to have pile bases in the organic diatomite, and it was decided – for one fourth of the tank area – to use longer piles with base in level -27 m DVR90 in order to penetrate the diatomite and reach harder soils. Although the diatomite was relatively soft, cohesional contributions to bearing capacity from the penetrated diatomite was included. The pile bearing resistance was estimated by geostatic calculation, using the usual formula for shaft resistance $R_s = m\, r\, c_u\, A_s$. Here, $m\, r = 0.4$ and c_u based on CPT and a cautious evaluation of N_k was chosen in the diatomite. Furthermore, the piles were cautiously presumed to have pile base in cohesional soil.

The driving resistance of the longer piles were relatively low, and to document the expected development by time in cohesional bearing capacity in the evaluated cohesional diatomite, pile driving analyses with CASE and CAPWAP analyses were performed on the five test pile six days after installation. These tests showed shaft resistances of 1 – 2 times the conservatively calculated resistance in the diatomite. Furthermore, the base resistance was 2 – 3 times the calculated resistance. Consequently, the pile design was proven acceptable.

10. CONCLUSIONS

This article aims to present the knowledge gained and geotechnical parameters for the found organic diatomite for future projects involving diatomite. The findings are intended to contribute to understanding the geotechnical behavior of diatomite, and the work has revealed issues that should be investigated further. For example, the diatomite seems to be a predominantly cohesive soil. Nevertheless, some parameters contradict traditional thinking, e.g. by varying plasticity and general behavior in the laboratory. Also, a traditional approach of evaluating water content, loss of ignition and maybe CPTu's may be challenged and subject for additional investigation. Furthermore, the complex local geology at the project site was a particular issue that required certain robustness in the foundation design.

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