The influence of soil structure on CPTu and SDMT results

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ABSTRACT

The natural structure of clays has a significant influence on its mechanical behaviour and can be characterized using in-situ and laboratory tests. It was reported by Robertson (2016) that soil structure leads to an increased tip resistance (q_c) and shear wave velocity (V_s) when performing seismic cone penetration tests. However, only limited studies investigated changes in soil structure by means of in-situ tests. Sensitive, marine clays were investigated within the research project "VIBE – Sustainable Ground Improvement Solution for Oslo" at the Norwegian Geo-Test site Onsøy (Gundersen et al. 2019). Possibilities and limitations of the vibro replacement method were studied for very soft ground conditions based on a full-scale field test. The influence of soil structure on in-situ measurements of piezocone penetration tests (CPTu) and seismic flat dilatometer tests (SDMT) are further studied by intentionally disturbing the soil structure by a vibrator. Results of CPTu and Medusa SDMT, executed before and after treatment, are compared to characterize changes in soil structure. The results indicate that the vibration-induced destructuration led to a significant decrease of CPTu measurements, namely tip resistance (q_c), sleeve friction (f_s) and measured pore pressure (u_2), within medium to high sensitive clays. As the decrease in f_s is more significant compared to the decrease in q_c, a significant decrease in friction ration (R_f) was observed. In analogy, SDMT resulted in a decrease in shear wave velocity (V_s), horizontal stress index (K_D) and dilatometer modulus (E_D) after the vibro treatment. It was further shown that the soil behaviour type chart according to Robertson (2016) leads to no sufficient characterization of soil structure in soft, marine clays.

Keywords: soil structure; soft clay; cone penetration test, seismic flat dilatometer test.

1. Introduction

Soil particle characteristics (e.g., particle size, particle shape, mineralogical composition), their arrangement and interparticle bonds influence the soil compressibility, strength and permeability (Leroueil and Vaughan 1990, Mitchell 1976). While the term *fabric* is often used to describe the arrangement of particles and pore spaces in soils, effects of particle arrangement (fabric) and inter-particle bonding are defined as *structure* according to Mitchell (1976) and Burland (1990). Soils characterized by the same fabric can show differences in stiffness and strength due to different degrees of interparticle bonding. Soil structure is influenced by compositional factors (e.g., mineralogy, particle size and shape), environmental factors (e.g., pore water composition, weathering), chemical processes (e.g., leaching, cementation), physical processes (e.g., consolidation, freezing) and biological processes (e.g., decay of organic matter).

The influence of structure on stiffness and strength has been studied by comparing laboratory results of natural and remoulded soils (e.g., Burland 1990). While structured soils are often characterized by an apparent preconsolidation pressure and a peak strength behavior, this behavior is not observed for remoulded specimens.

A limited number of studies investigated the influence of soil structure on piezocone penetration tests (*CPTu*) and seismic flat dilatometer tests (*SDMT*) (e.g., Schnaid et al. 2004). Robertson (2016) illustrated that soil structure leads to an increase in tip resistance (q_c), shear wave velocity (V_s) and small strain stiffness shear modulus (G_0). However, no studies are available which have attempted to artificially change the soil structure in-situ and to study the respective changes by means of in-situ tests.

The research project VIBE "VIBro rEplacement – Sustainable Ground Improvement Solution for Oslo" was initiated by Keller Geoteknikk and the Norwegian Geotechnical Institute (*NGI*) to investigate possibilities and limitations of the vibro replacement method in Norwegian clays. A full-scale test of the vibro replacement method was performed at the Norwegian Geo-Test site Onsøy (Gundersen et al. 2019), further named here as test site VIBE. In addition, the site was used to study the influence of soil structure on results of *CPTu* and *SDMT*. For the latter research question, a vibrator was used to disturb the natural structure of the Onsøy clay. In-situ and laboratory tests were performed before and after vibro treatment to capture changes in soil structure.

This article compares results of (i) *CPTu* and *SDMT* executed before and after vibro treatment in the clay and (ii) evaluates the performance of the soil behaviour type chart according to Robertson 2016 which was developed to detect soil structure.

2. Methods

2.1. Vibro replacement

Vibro replacement is a widely adopted ground improvement method for soils, ranging from clay to medium sand (McCabe et al. 2009). The bottom-feed and top-feed methods can be used to create vibro stone columns in these ground conditions. The bottom feed method, used at NGTS Onsøy, is further described as it is the proper method to install stone columns in soft clays. An undrained shear strength equal to 4 kPa was defined by Wehr and Sondermann (2013) as the lower boundary for vibro stone installation.

In the first step, the bottom feed vibrator is positioned over the defined location of column installation. A wheel loader fills the bucket with aggregate. The bucket is lifted and empties its content into the air chamber. Once the air lock is closed, the material flows towards the vibrator tip assisted by pressurized air. The vibrator displaces the soil and is lowered to the design depth of the planned column. After reaching this depth, the vibrator is pulled up slightly, causing the aggregate to fill the cavity created. During re-penetration, the aggregate is compacted and pressed into the surrounding soil. The stone column is built up in alternating step to the design level. As a result, a so-called stone column remains in the ground, which is stronger, stiffer and more permeable than the surrounding soil and thus improves the soil behaviour in terms of bearing capacity, settlement response and drainage (McCabe et al. 2009).

2.2. In-situ testing

The piezocone penetration test (*CPTu*) is an internationally established, fast and cost-efficient technique for onshore and offshore site characterization. A cone (usual cross-section area: 10 cm^2, opening angle: 60°) is pushed into the soil at a constant penetration rate of 2± 0.5 cm/s using a hydraulic unit in combination with 1 m long penetration rods. The tip resistance (q_c), the sleeve friction (f_s) and the dynamic pore pressure (u_2) were measured during the penetration procedure. All investigations were executed according to ISO 22476-1:2022.

The updated normalized tip resistance (Q_{tn}), normalized friction ratio (F_r), pore pressure parameter (B_q) and normalized excess pore pressure parameter (U_2) can be derived according to Eqs. (1), (2), (3) and (4) based on the in-situ measurements and can further be used for soil classification and parameter determination.

$$Q_{tn} = \left(\frac{q_t - \sigma_{v0}}{p_a}\right)\left(\frac{p_a}{\sigma'_{v0}}\right)^n \tag{1}$$

$$F_r = \frac{f_s}{q_t - \sigma_{v0}} 100\% \tag{2}$$

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_{v0}} \tag{3}$$

$$U_2 = \frac{u_2 - u_0}{\sigma'_{v0}} \tag{4}$$

were q_t is the tip resistance corrected about water effects, where $q_t = q_c + u_2\,(1\text{-}a)$; a is the cone area ratio; σ_{v0} is the in-situ total vertical stress; σ'_{v0} is the in-situ effective vertical stress; p_a is the atmospheric reference pressure; n is the stress exponent that varies with the soil behaviour type index (Robertson 2016); and u_0 is the in-situ equilibrium water pressure.

The flat dilatometer test (DMT) is an in-situ testing device used for characterizing a wide range of soils, ranging from clays to sands (Schnaid 2009). A steel blade, containing a thin, expandable, circular steel membrane, is pushed into the soil at a constant penetration rate (usually 2 cm/s). The penetration is usually stopped every 20 cm for consecutive pressure readings at defined membrane expansions (*A*-, *B*- and *C*-reading). The expansion of the membrane is performed traditionally using a pneumatic cable, which is connected to a control unit and a gas tank. The enhanced Medusa flat dilatometer (*Medusa DMT*) enables an automatic expansion of the circular membrane without a pneumatic cable. A motorized syringe, driven by an electronic board, hydraulically expands the membrane for pressure readings. The generated pressures are measured using high-accuracy transducers (Marchetti 2018, Marchetti et al. 2019, Monaco 2021). The Medusa equipment was used during test execution at the test site *VIBE*. The penetration procedure was stopped in 20 cm intervals to perform the standard *DMT* (*DMT-STD*) procedure. The standard *DMT* procedure considers *A*- and *B*-pressure readings 15 and 30 seconds after penetration stop, respectively (*A15*, *B30*).

The in-situ readings (*A*, *B*, *C*) are corrected for the membrane stiffness according to Marchetti et al. (2001). The corrected readings (P_0, P_1, P_2) are further used in Eqs. (5), (6) and (7) to derive the material index (I_D), horizontal stress index (K_D) and dilatometer modulus (E_D) which represent the basis for deriving soil parameters.

$$I_D = \frac{P_1 - P_0}{P_0 - u_0} \tag{5}$$

$$K_D = \frac{P_0 - u_0}{\sigma'_{v0}} \tag{6}$$

$$E_D = 34.7 \cdot (P_1 - P_0) \tag{7}$$

The in-situ shear wave velocity (V_s) was determined by using the Marchetti system (Marchetti et al. 2008), which consists of two geophones installed at a vertical distance of 50 cm and mounted between the penetration rods and the Medusa blade (Medusa *DMT*). The penetration was stopped every 50 cm for V_s measurements. The seismic wave, triggered by a hammer blow at the surface and received by both geophones, is amplified, digitized, and send to the computer for real-time data interpretation (Marchetti et al. 2008).

2.3. Soil sampling and laboratory testing

Undisturbed soil sampling was obtained from a 72 mm thin walled fixed-piston sampler at the test site VIBE. Subsequently, soil specimens with a length of approximately 1 m were scanned by computer tomography to specify the parts of highest quality for laboratory testing. Laboratory testing included the determination of index parameters (particle size distribution, water content, unit weight, Atterberg limits) and the execution of uniaxial compression tests (*UCS*), constant rate strain (*CRS*) oedometer tests, triaxial tests (*CAUC*), bender element tests (*BE*) as well as fall cone test (*FC*). The soil sensitivity (S_t) was derived based on fall cone tests, executed on sampled and remoulded soil specimens. All laboratory tests were performed according to NS-EN 1997-2:2007+NA:2008.

2.4. Detection of structure using CPTu

Eslaamizaad and Robertson (1996) as well as Schnaid et al. (2004) suggested linking G_0/q_t and Q_{tn} to identify soil structure by means of seismic piezocone penetration tests (*SCPTu*). Robertson (2016) introduced the modified normalized small-strain rigidity index (K_G^*) to detect structured soils (see Eq (8)):

$$K_G^* = (I_G) \cdot (Q_{tn})^{0.75} = \left(\frac{G_0}{q_t - \sigma_{v0}}\right) \cdot (Q_{tn})^{0.75} \qquad (8)$$

where I_G is the small-strain rigidity index.

While K_G^* ranging between 100 and 330 indicates soils with no or little structure, structured soils are represented by $K_G^* > 330$ according to Robertson (2016). The latter limit values represent a straight line in the logarithmic Q_{tn}-I_G space (see Fig. 6).

3. Test site VIBE

The test site *VIBE* is located in the southwest (*SW*) of the Norwegian Geo-Test site Onsøy and covers an area of approximately 54x27 m (see Fig. 1). Previous investigations at Onsøy were mainly executed within the shaded areas *south central (SC)* and *southeastern corner (SEC)*, including the boreholes *B01*, *B02*, *B41*, the piezocone penetration tests *C02*, *C11* and the seismic flat dilatometer test *D01* (see Fig. 1). The work steps of the research project *VIBE* can be summarized as follows:

1) The topsoil with a thickness of 30 cm was removed using an excavator and stored at the northern end of the test site.
2) The initial ground conditions were characterized using total soundings (*TOT*), piezocone penetration tests (*CPTu*), flat dilatometer tests (*DMT*) and heavy dynamic probing (*DPH*). The locations of *CPTu-K6* and *DMT-N3* (which are further used in Section 4) are indicated in Fig. 1.
3) Piezometers (*PZ*) and earth pressure cells (*EP*) were pushed into the ground between the planned vibro stone columns to characterize changes in stress during and after the vibro stone column installation. Three piezometers were installed at the locations *PZ2* and *PZ3* (see Fig. 1). Three additional earth pressure cells (*EP3*) were installed close to *PZ3*. Piezometers and earth pressure cells were installed at -2.5 m, -5.5 m and -8.5 m at individual locations. The piezometers and earth pressure cells were measured automatically.
4) Settlement plates (not shown in Fig. 1) were distributed along the working platform and covered by a 70 cm thick working platform (0/63 mm). Two horizontal inclinometers were installed within the working platform to derive a settlement profile. While the horizontal inclinometers were measured automatically, the deformations of the settlement plates were determined with a total station.
5) The vibro work by means of bottom-feed vibrator was executed within a rectangular area of 12.6 x 7.2 m. All elements were executed to bedrock at a centre-to-centre spacing of 1.8 m (see Fig. 1).

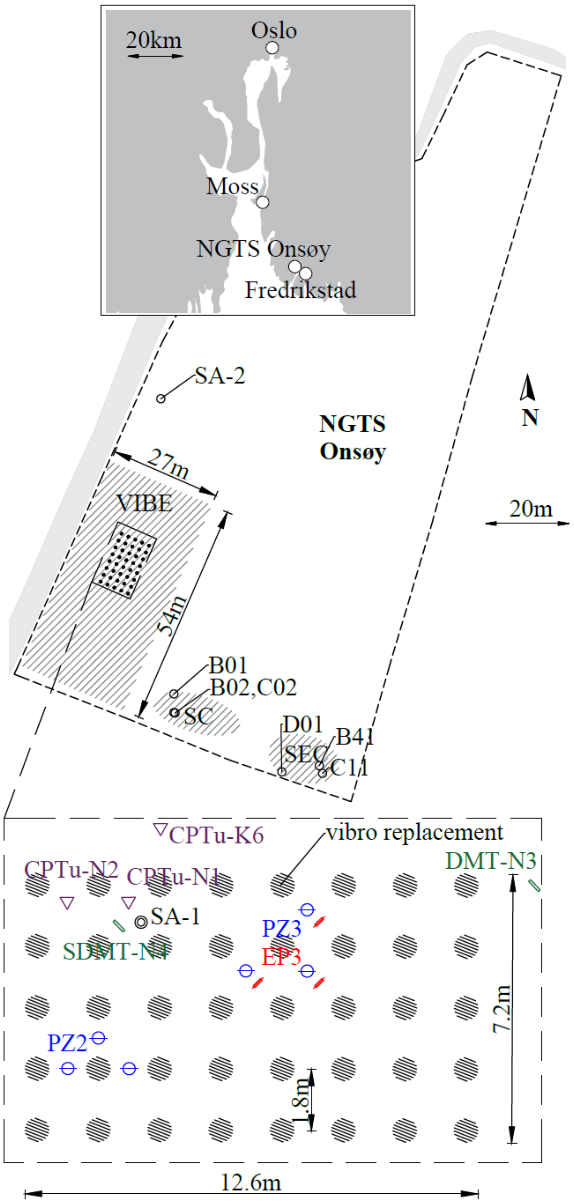

Figure 1. Overview of the investigations at test site VIBE: piezocone penetration tests (CPTu), (seismic) flat dilatometer tests (DMT/SDMT), soil sampling (SA), piezometers (PZ) and earth pressure cells (EP).

6) A second round of in-situ testing and soil sampling was performed four days after the vibro treatment. The piezocone penetration tests (*CPTu-N2*, *CPTu-N1*), seismic flat dilatometer tests (*SDMT-N4*) and soil sampling (*SA-1*) were executed between the columns, as shown in Fig. 1. All test locations had a distance of 50 cm to the closest, respective installation point, enabling a direct comparison of laboratory and in-situ results. Additional soil sampling, namely *SA-2*, was executed slightly north of the test site *VIBE* (see Fig. 1).

4. Results

4.1. Characterization of the Onsøy clay

The ground conditions at *NGTS* Onsøy can be divided into four soil units at *SC* and *SEC*, namely *L1* (clay: dry crust), *L2* (clay: low to very low strength), *L3* (clay: low to medium strength), *L4* (clay: low to medium strength, slightly sensitive) and *L5* (bedrock) (Gundersen et al. 2019). The layer boundaries are indicated as grey, dotted lines in Fig. 2 and suggest that the bedrock surface falls towards the east, leading to a greater thickness of *L2* and *L3* in the southeastern corner. The depth to bedrock amounts to approximately 24 m and 28.5 m at *SC* and *SEC*, respectively. The ground water table was situated 1 m below the surface during the investigations.

At *SC* ad *SEC* the soil unit *L1* is approximately 1 m thick and consists of a weathered clay crust, characterized by a unit weight of about 17.5 kN/m^3. The high plastic clay of L2 is characterized by a water content (w) ranging between 60 and 75%. Its plasticity index (*PI*) (defined as the difference between liquid limit *LL* and plastic limit *PL*) and unit weight amount to 44% and 16.2 kN/m^3, respectively. *L3* is characterized by a higher unit weight and lower water content as well as plasticity index compared to *L2*, leading to average values of approximately γ_{sat} = 17.8 kN/m^3, w = 45% and *PI* = 27%. The top part of the soil unit *L4* has similar properties compared to *L2* but the water content, plasticity index and clay content are expected to decrease towards bedrock. The in-situ undrained shear strength derived from *CAUC* (s_{uC}) can be approximated based on Eq. (9) for depths ranging between 5 and 20 m (Gundersen et al. 2019). A s_{uC} of 15.8 kPa was recommended for areas closer to the surface.

$$s_{uC} = 0.43 \cdot \sigma'_{v0} \tag{9}$$

As an alternative, s_{uC} can be determined for *SC* and *SEC* based on *CPTu* using the cone factors N_{kt} = 9 and $N_{\Delta u}$ = 7.2, leading to s_{uC}/σ'_{v0} ratios of approximately 14.5. The soil sensitivity is generally ranging between 5 and 8 for *L2* and *L3* at *SC* and *SEC*. Soil specimens referred to *L4* exhibit higher sensitivities as shown in Fig. 2j.

The soil layering at test site *VIBE* can be subdivided into three main units, namely *L1* (clay: dry crust), *L3* (clay: low to medium strength) and *L5* (bedrock). The unit weight (γ_{sat}) ranges between 17 and 18 kN/m^3, leading to a good agreement with results from *SC* and *SEC* (see *L3*). The results of the water content present a larger scatter in Fig. 2c but approximate on average 45%. Soil sensitivity (S_t), derived from fall cone tests, increases with depth. While S_t ranges between 1 and 5.3 within the upper 3.5 m, high sensitive clays (S_t = 24 to 91) are determined in greater depths. Low and high sensitive clays are indicated by *L3.1* and *L3.2*, respectively in Fig. 2a.

4.2. Comparison of in-situ results executed before and after vibro treatment

Changes in soil structure due to the vibro treatment are evaluated by comparing results of *CPTu* and *SDMT*, executed before and after treatment. In-situ measurements and normalized parameters are presented for *CPTu-K6* (before treatment) and *CPTu-N2* (after treatment) in Fig. 3. A disturbance of the soil structure decreased q_t, f_s and u_2 within all the soil units. Since the decrease in f_s was more pronounced compared to the decrease in q_t, R_f (= $f_s/q_t \cdot 100$) significantly decreased. When comparing the u_2 measurements (executed before and after treatment), it is evident that the data after the induced vibrations (red line) show a smoother pattern compared to the data before the treatment was conducted (blue line). After normalizing the in-situ measurements by the in-situ stress state, *CPTu* results (Q_t, F_r, B_q, U_2) executed after inducing vibrations remained smaller compared to results from before the treatment (see Fig. 3e to Fig. 3h).

Results of *DMT*, executed before and after the treatment, are compared in Fig. 4. It is evident that the corrected in-situ measurements (P_0, P_1) decreased due to vibrations (see Fig. 4a). This reduction is evident in all soil units, like for *CPTu* results. While the destructuration of the soil led to a significant reduction of the intermediate parameters K_D and E_D, results of I_D were in good agreement before and after the vibro treatment.

Changes in shear wave velocity due to changes in soil structure could only be assessed to a limited extent at Onsøy, as no seismic investigations were executed before the vibro treatment at the test site *VIBE*. Fig. 5 compares the results of *D01* (executed within *SEC*) and *SDMT-N2* (executed within *VIBE*). As the soil layering differs between the two locations, a comparison was only possible to a limited extent. The blue and red dotted lines indicate the soil layering at SEC and VIBE, respectively (see Fig. 5). Red symbols (*SDMT-N2*), representing the results after vibro treatment at *VIBE*, were on average smaller compared to the blue line (*D01*, executed within *SEC*), even though *L3.1* and *L3.2* are characterized by a higher strength than *L2*. Especially when comparing tests below 5 m a reduction in shear wave velocity due to destructuration is obvious.

5. Discussion

The soil structure was intentionally disturbed by a vibrator around *SA-1*, *CPTu-N1* and *SDMT-N4*, leading to a decrease of in-situ measurements (q_t, f_s, u_2, P_0, P_1) and normalized parameters (Q_t, F_r, B_q, U_2, K_D, E_D). Consequently, the results are in good agreement with the trends reported by Robertson (2016) and Schnaid et al. (2004). It was observed that the high sensitive, low strength clay was strongly disturbed around the four installation points during the vibro treatment. The voids of the installed gravel were possibly filled by the remoulded clay, leading to a low permeability. It is therefore assumed that the reduction of u_2 (in Fig. 3c) is not related to a shorter drainage path but a reduction in strength and stiffness. The u_2 profile, measured after the vibro treatment, were smoother compared to the initial state indicating that the soil was homogenized during the vibro treatment (see Fig. 3c).

In a last step, the soil structure is discussed at Onsøy using the soil behaviour type chart according to Robertson (2016). While K_G^* of the natural, structured clay was derived based on the in-situ measurements of

D01 (V_s) and *C11* (q_c, f_s, u_2), *SDMT-N4* and *CPTu-N2* were used to characterize the disturbed clay (after vibro work). The natural clay led to K_G^* values ranging between 140 and 240, indicating no structure according to Robertson (2016) (see blue symbols in Fig. 6). Results after the vibro treatment led to slightly reduced K_G^* values (K_G^* = 140 - 190). The results indicate that the proposed transition between structured and unstructured soils (K_G^* = 330, Robertson 2016) is not applicable for Norwegian, marine clays. This result is in good agreement results of fine-grained Alpine deposits (Oberhollenzer 2022).

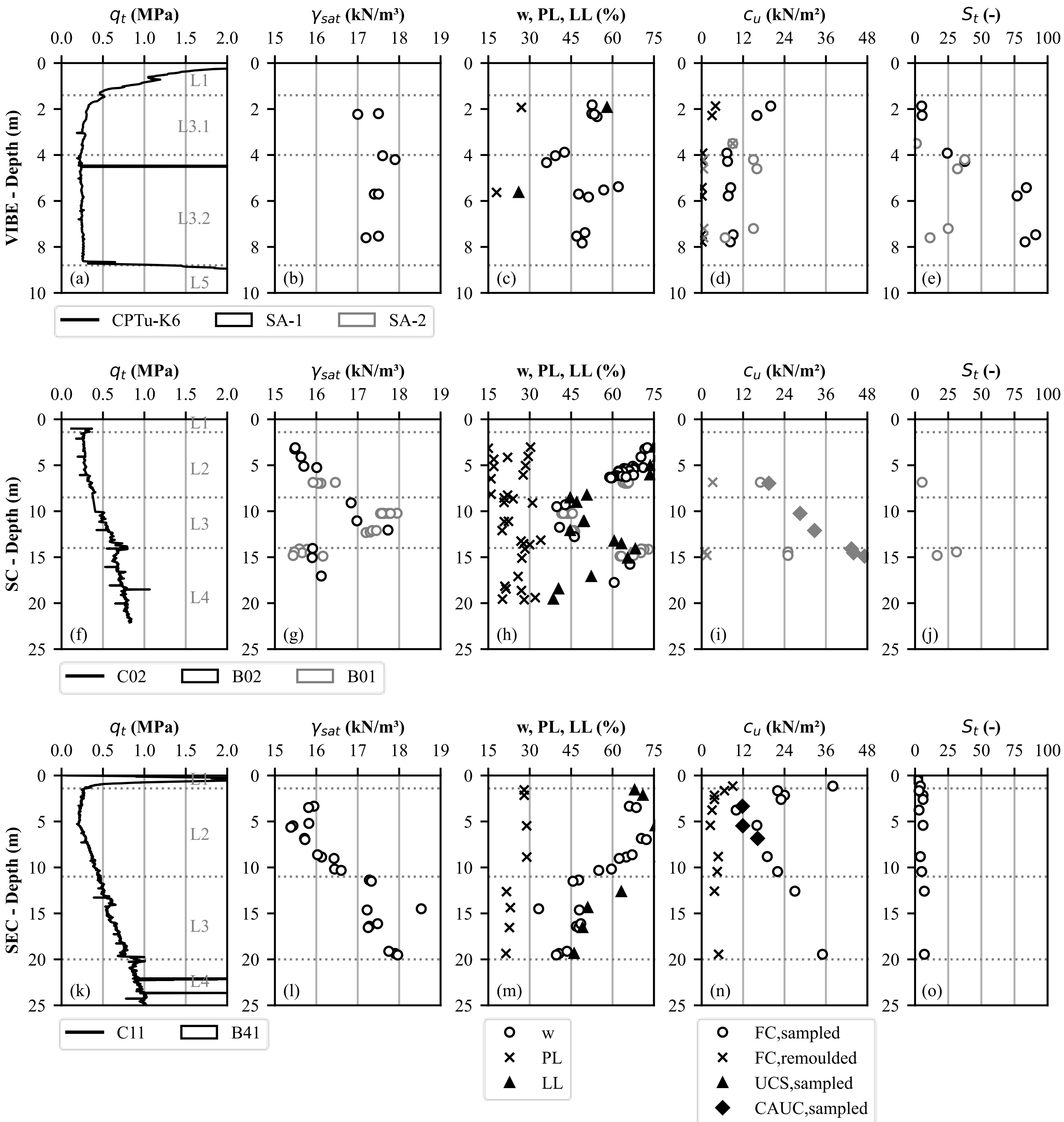

Figure 2. Characterization of the Onsøy clay based on CPTu and laboratory tests: Summary of results for *VIBE* (top row), *SC* (middle row) and *SEC* (bottom row).

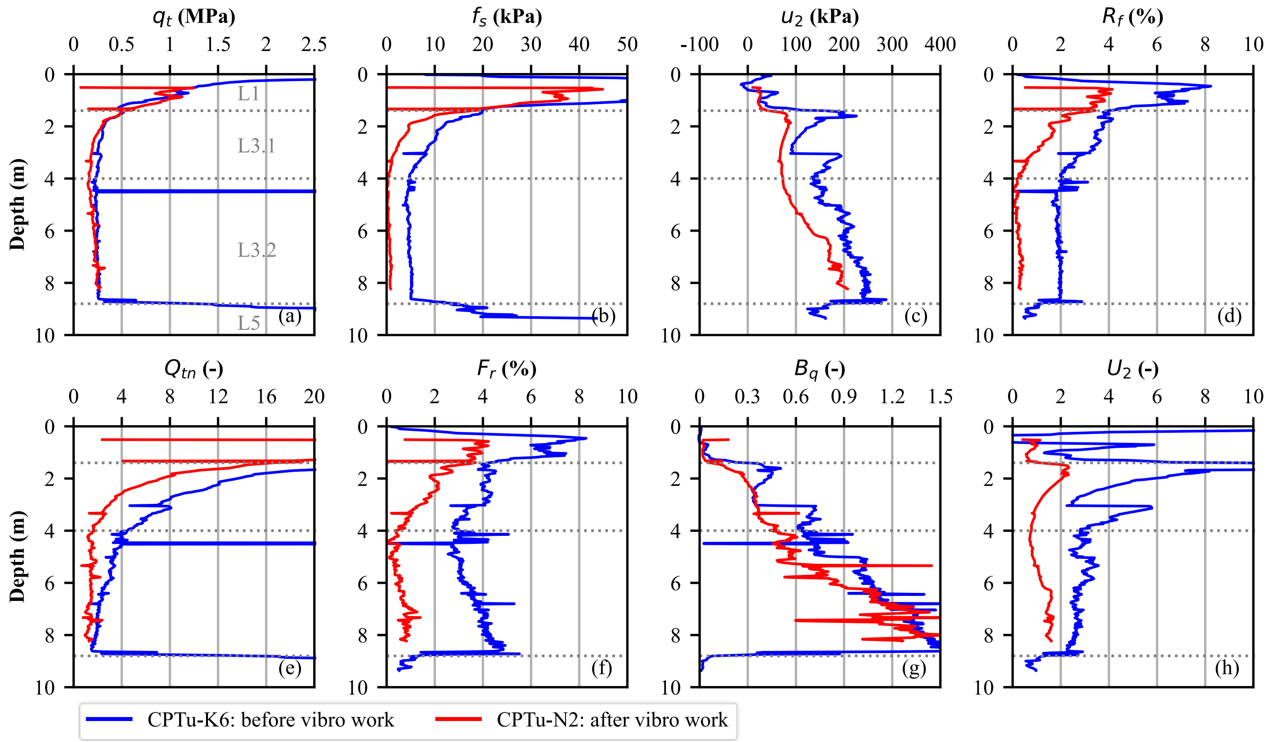

Figure 3. Comparison of *CPTu* results executed before (blue) and after (red) treatment.

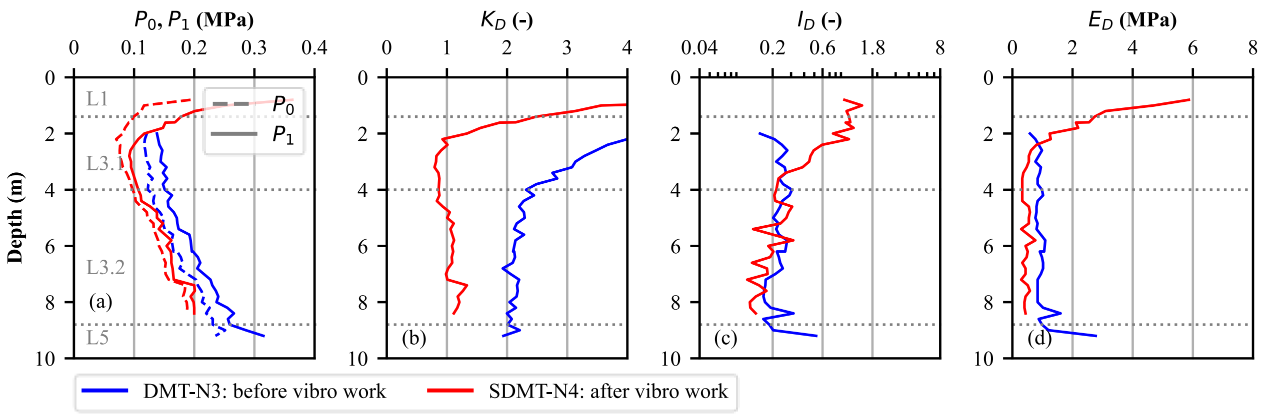

Figure 4. Comparison of *DMT* results executed before (blue) and after (red) treatment.

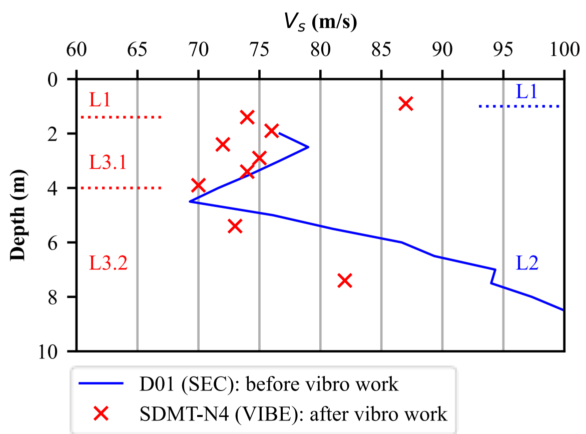

Figure 5. Comparison of downhole seismic results executed before (blue) and after (red) treatment.

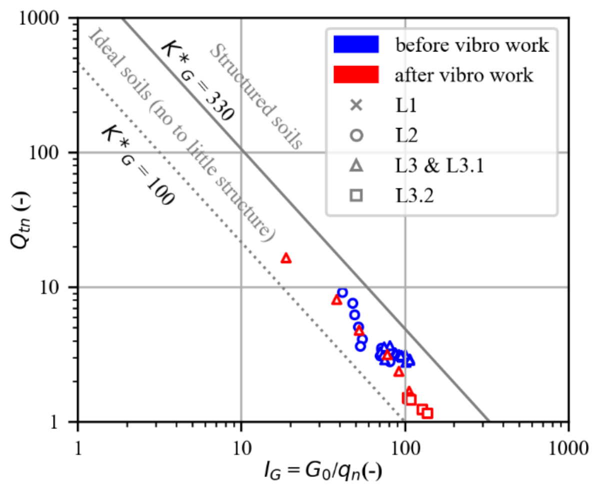

Figure 6. Detection of microstructure according to Robertson (2016): Comparison of results before (blue) and after (red) treatment.

6. Conclusions

This article investigated the influence of soil structure on results of piezocone penetration tests (*CPTu*) and seismic flat dilatometer tests (*SDMT*). Therefore, the natural structure of a marine clay was disturbed using a vibrator (which is normally used to install vibro stone columns) at the Norwegian Geo-test site Onsøy. The unexpected but locally very high sensitive ground conditions led to a liquefaction of the clay situated around individual installation points.

In-situ testing and soil sampling for laboratory testing was performed before and after the vibro treatment. Based on the comparison of both investigation campaigns it was shown that soil structure leads to increased *CPTu* measurements (q_t, f_s, u_2) and normalized parameters (Q_{tn}, F_r, B_q, U_2). The corrected *DMT* readings (P_0, P_1) showed a similar increase leading to a rise in the intermediate parameters K_D and E_D. The shear wave velocity (V_s) increased due to soil structure, as was previously reported by Robertson (2016). However, the results suggest that the soil behaviour chart developed by Robertson (2016) to detect soil structure cannot be used successfully in Norwegian clays as structured clays are characterized by K^*_G values ranging between 140 and 240. These values are significantly smaller compared to the suggested transition (K^*_G= 330). For this reason, it is suggested to elaborate alternative concepts to distinguish between structured and non-structured clays using in-situ tests.

The present conclusions are only valid for the vibro stone column installation in medium- to high-sensitive clays. Further investigations are planned to characterise the installation effects in non- to low-sensitive soils.

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