Challenges characterizing glacial soil deposits

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ABSTRACT: Glacial soils are heterogenous and contain many different soil types with different drainage behaviour: undrained, drained and partly drained. For the interpretation of field tests assumptions are made that the soil behaves either drained or undrained during the execution of in-situ tests, like CPTU, DMT or others. In glacial soils partly draining behaviour may occur during in-situ testing, which makes interpretation difficult and often unreliable. The testing procedures may have to be adapted such that either drained or undrained behaviour is being measured. Experience from several smaller scale site investigations will be presented that illustrate the difficulties. There are tests that do not measure pore pressure response, in applying these tests, one must be aware that misleading results may be derived. For CPTU tests besides the profiling also dissipation tests must be carried out and the mode of interpretation may be adapted. Similar for DMT the changes in membrane pressure during the entire execution of the tests must be observed. A combination of field tests with laboratory tests may be necessary for a reliable characterization.

Keywords: glacial soils, undrained behaviour, drained behaviour, partly drained, CPTU, DMT

1. General geologic conditions

Glacial soils are complex sediments that may have a range of grains size from clay size to boulders. Yet, on the other side also segregated soil layers with rather narrow grain-size distribution are present that have been formed by water transport, like fluvio-glacial gravels, sands, nearly pure silts and glacial clays. For the characterization of the soils different in-situ techniques and laboratory tests [1,2,3] can be used, as not all in-situ techniques are suited for all soil types. In some geologic conditions and based on the structural requirements combinations of field and laboratory tests are considered useful. For fluvioglacial gravel mainly heavy penetration test are useful, such as the DPSH-B according to EN_ISO 22476-2 [6] or the Standard Penetration test SPT according to EN_ISO 22476-3 [7] can be used; in fine-grained soils the electric Cone Penetration test [5] CPTU (EN_ISO 22476-1), the mechanical cone penetration test [9] CPTM (EN_ISO 22476-12) or the flat Dilatometer test [8] DMT (EN_ISO 22476-11).

2. Field tests in glacial soils

2.1. Dynamic Penetration tests

For coarse grained soils d_{90} > 2 mm in the field practically only the SPT with the cone tip can be used. In Switzerland the tube sampler is never used in practice, as it may be damaged by stones in the ground. Even a cone is heavily used and abused in glacial gravel as shown in Figure 1.

The dynamic penetrometer test DPSH-B may also be used in practice. This test has in gravel the shortcoming that it may become stuck by larger stones. For these soils the use of SPT in boring is advisable, larger stones can be drilled through and a new SPT started.

2.1.1. Extended SPT in boreholes

Our experience has shown that during removal of the drill rod with the sampler the water table may sink below the level of the groundwater table, thus hydraulic failure may occur at the bottom of the boring. The soil below the bottom of the borehole may loosen and the blow count will be too low and misleading.

Figure 1. Closed conical tip of an SPT sounding damaged from driving in gravelly deposits.

When SPT are made according to the standard procedure this may lead to too low values of blow counts. In order to avoid these shortcoming, SPT tests with the closed conical tip are made over larger depth, to 1.05 m depth or deeper until resistance becomes too large.

2.1.2. Examples of extended SPT tests

In Figure 2 to Figure 6 the results of five extended SPT tests with representative values of blow count N from a boring through fluvial deposits in a large alpine river valley are shown. Each series of SPT shows a different pattern of the blow counts and shows the influence of the variable ground conditions.

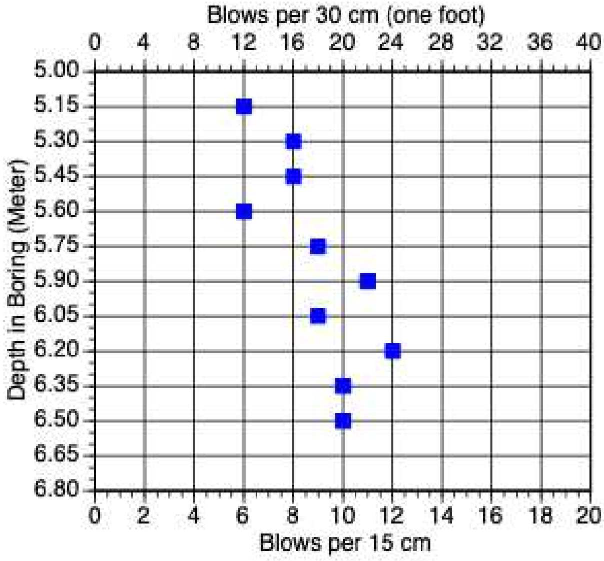

Figure 2. Extended SPT in fluvial deposits from 5,0 to 6.5 m. A value N = 16 is considered representative.

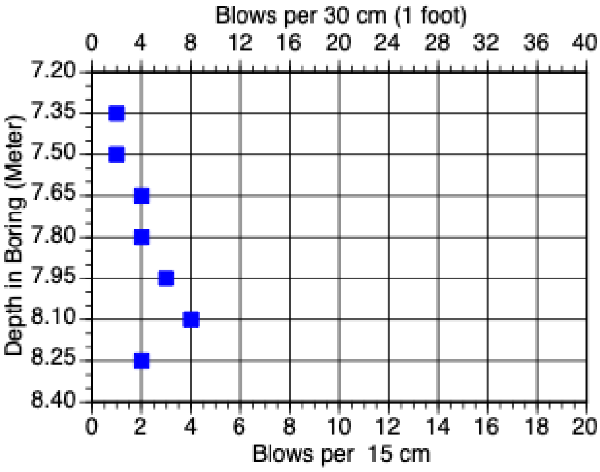

Figure 3. Extended SPT from 7.20 to 8.25 m depth. Deposits are loose with a representative value N = 4.

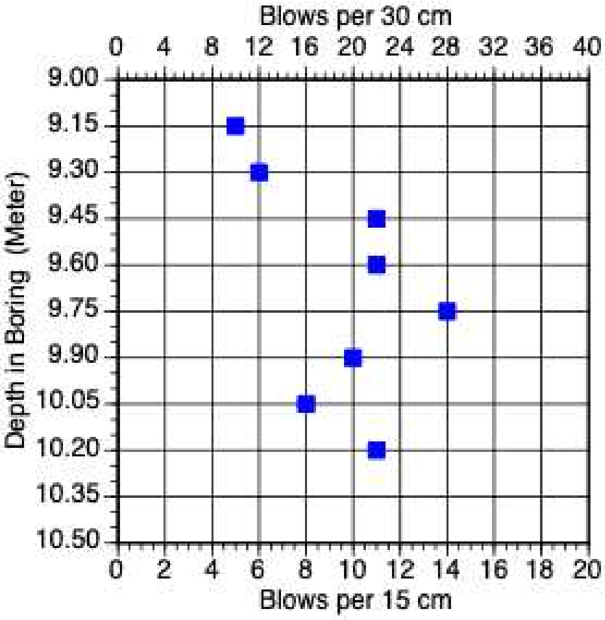

Figure 4: Extended SPT from 9.0 to 10.2 m depth. A value N = 22 is considered representative

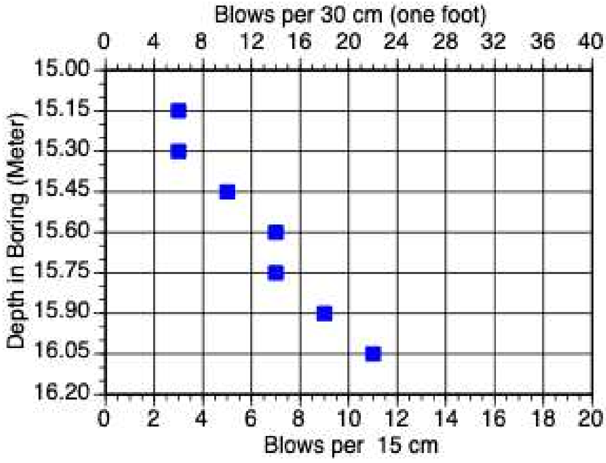

Figure 5. Extended SPT in fluvial gravel from 9.0 to 10.2 m depth

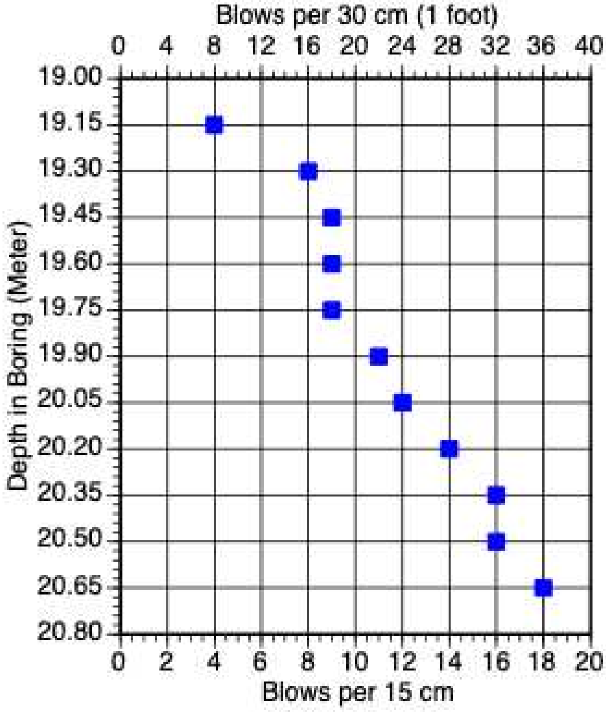

Figure 6: Extended SPT over 1.8 m depth, the representative value of SPT would be N = 18.

In all tests in the top layer (0.45 m) lower blow counts (SPT) were measured than in the deeper layers, these differences are most likely caused by the disturbance of the natural soil below the bottom of the bore-hole. The differences can be important as in the test below 9 m (Figure 4), where the N = 11 is only half the value below 9.45 m, which with N = 22 is considered representative. In the layer from 15.0 to 16.05 m, the blow counts increase below 15.75 m, which may indicate a denser layer with more tip and friction resistance. In the layer between 19.0 and 20.8 m the blow count increase below 19.75 m, also indicating an increase in resistance, probably a different layer. The blow counts from extended SPT must be interpreted with logged core description and classification.

In another case where only 45 cm long SPT tests had been made, very low values were measured, which would have meant to extend the drilled piles into an underlying clay layer, which would have meant much longer piles, up to 35 m long, into the glacial till. Supplementary investigations were carried out, in this case with mechanical cone penetration test (CPT-M) which showed an increase of the tip resistance below 0.45 m depth and the resistance increased more with depth and the test had to be stopped in 1.2 m depth. These new soundings assured safe shorter piles remaining in the top silty sand layer.

2.2. DMT in draining soils

At the same site as the extended SPT also four DMT push-in tests were made in fine-grained soils, sands with only a small fraction of gravel, shown in Figure 7. The tests show the large variability of the density of the fluvial deposits and the associated soil mechanics parameters. In such granular soil it is practically impossible to obtain undisturbed samples, thus in-situ tests like the DMT are the only means to measure soil mechanics properties. The variation of the modulus agrees with the variation of the blow counts from SPT in the gravel layers. These soil layers may contain small angular gravel that may scratch the membrane (Figure 8) or even damage it. The membrane can be easily replaced at small cost. In such alternating soil one must take the risk that a few membranes might have to be replaced. The DMT gives reliable results of modulus and friction angle.

The variations and the zones with loose sand and gravel led to the selection of an internally braced excavation of the large excavation for a four-lane motorway. As there were gravel layers with cobbles and boulders alternating with the depth this dictated the choice of type of side walls for the excavation.

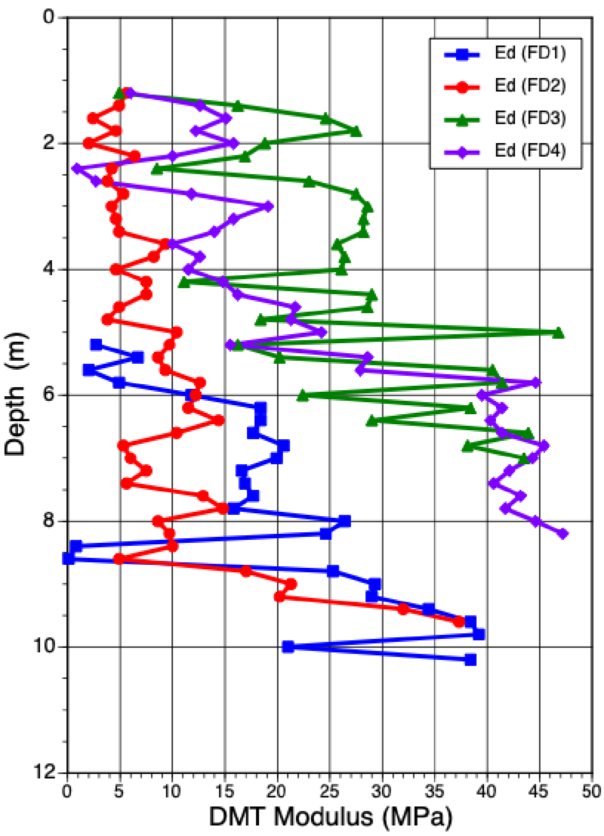

Figure 7.Modulus M obtained for four pushed-in DMT tests showing the large variability of fluvial deposits.

Figure 8: DMT Blade used in fine grained soils with some gravel showing some scratching that may damage the membrane.

Sheet piles had to be excluded and a secant pile were selected. Slurry walls were excluded due to layers with open gravel, tree trunks and differences in hydraulic head in separated aquifers.

At another site with glacial sands above and below the groundwater table DMT (Figure 9) were carried out. In boring RB1 the water table was at 8 m depth, in boring RB2 the groundwater was present below 10 m. In RB 1 the DMT show disturbance of soil in the first meter of the test due to hydraulic uplift during the drilling operation. The tests executed above the groundwater table show less variation of the modulus with depth, i.e. less disturbance the moduli M means of M = 60 to 80 MPa in 12 to 14 m depth are rather high and 90 MPa in 10 to 12 m. and M = 100 to 120 MPa in 6 to 8 m depth.

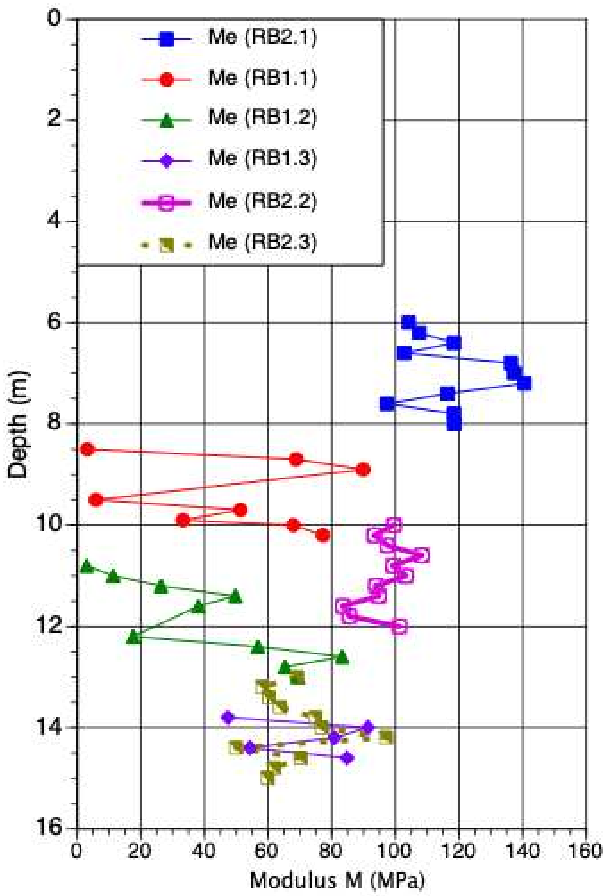

Figure 9. Modulus M for glacial sands, partly below groundwater table at 8 m.

2.3. Undrained soils: clays

With in-situ test, such as CPT and DMT, applied in fine-grained plastic clays with low permeabilities the undrained properties will be determined. Drained properties like confined soil compressibility must be determined with laboratory tests. A combination of both types of tests is beneficial in characterizing the geotechnical characteristics. Some examples are presented that led finally large savings in construction cost.

2.3.1. Normally consolidated glacial clay

A small filled-in glacial lake 20 m deep, but limited in extent, area 120 by 100 m, required new construction. A major highway had been founded on drilled cased concrete piles 20 to 23 m deep into glacial gravel. The piles continued to settle and additional construction near the surface was planned. Several borings were made and DMT and CPTU tests executed. The results from the deepest boring of undrained shear strength from CPTU and DMT tests are shown in Figure 10.

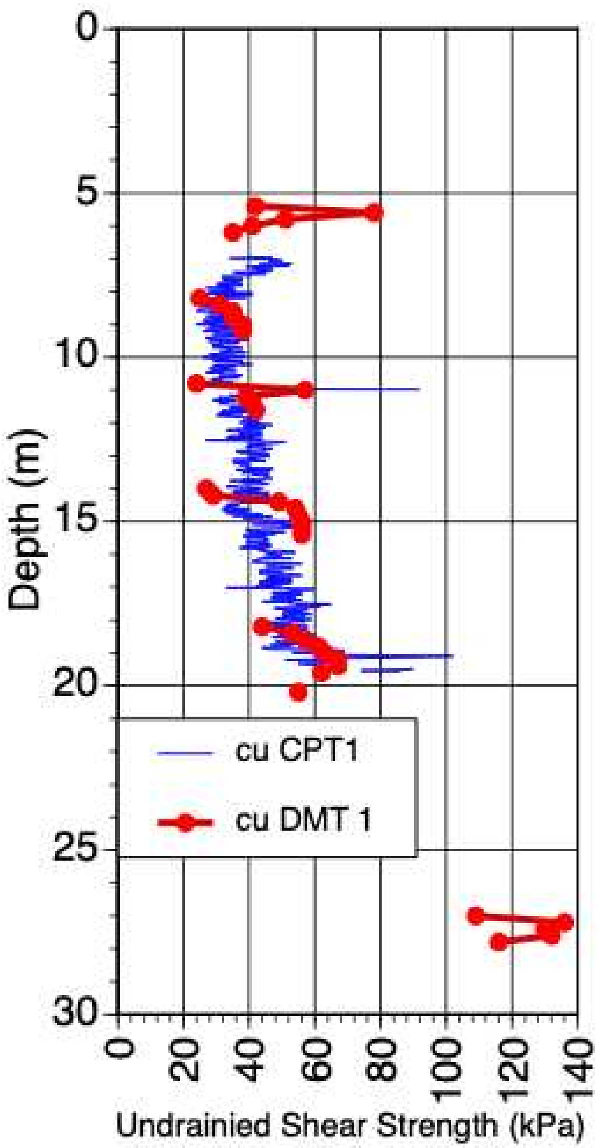

Figure 10: Undrained shear strength in soft glacial clay with 5 m man-made fill on top and intermediate layer of gravel between 23 and 26 m depth

The top layer of 5 m is man-made fill. The upper clay layer from 5 to 21 m depth is normally consolidated. Fluvioglacial gravel is present below 29 m and between 23 and 26 m. The consequence of an advancing, retreating glacier. The clay layer from 26 to 29 m is slightly over consolidated as the undrained shear strength measured by DMT indicates. The cause of the large settlement of the piles could thus be determined. The dynamic penetrometer DPM used for the site investigation in the 1970's for the bridge project could not cross the top gravel layer and the lower clay layer was not detected.

2.3.2. Filled-in Glacial lake

South of the city of Berne a large area of a filled-in glacial basin exists that is 200 m deep at places. The top 40 m are glacial clay, which is slightly over consolidated due to aging. A site investigation for an industrial building, a printing plant, was carried out. One boring to 40 m depth with DMT tests and three static penetration tests CPTU to 25 m were made. The undrained shear strength profiles with depth are shown in Figure 11.

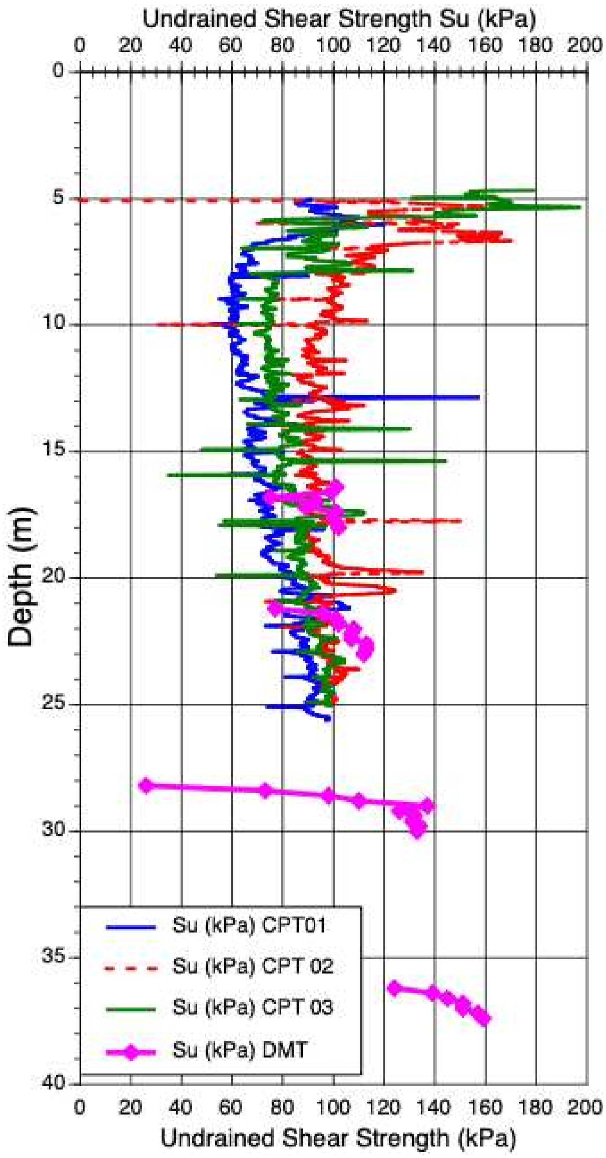

Figure 11. Undrained shear strength for postglacial clay determined with 3 CPTU tests and one boring with DMT test from the bottom of the borehole

The undrained shear strength indicates a slight, but important, over-consolidation of the clay. The results of the three CPTU tests show different undrained shear strength of the clay from 5 to 20 m depth and converge to the same S_u between 20 and 25 m depth. The difference could at first not be explained, but was discovered later. The tests were executed beginning of November and the CPTU tip was stored on site at ambient temperatures, which had dropped during some night to below freezing. When the tips were used in the morning and pushed in the ground the cone-tips warmed up, resulting in temperature strains and in changes in the strains of the load gauges. Changes in the readings are due to warming of CPT tip in the ground, which was different for the three tests, made on different days, as predrilling was required through the top gravel layers.

The measurements with DMT of the undrained shear strength (Figure 11) show that the soil below the bottom of the borehole had been disturbed for several decimetres. The undisturbed strength was reached after 0.8m depth below the borehole bottom.

The clay below 25 m depth appears to be slightly more over consolidated than above. The boring was drilled to 40 m depth, because in a deep boring at some distance a layer of gravel or moraine had been encountered. At this boring no such sediments were found.

For the now built plant it was necessary to have a good knowledge of the pre-consolidation stresses (Figure 12). The over-consolidation stresses between 5 and 10 m were 120 to 400 kPa and from 10 to 25 m depth 120 kPa. The surface loads on the slab foundation are 40 kPa, thus far below the pre-consolidation stresses. A slab foundation was thus a safe and economic solution. In the top layer the poor postglacial deposits (0-2 m) had to be replaced by gravel. A piled foundation was not necessary and could be saved (Cost 1 Mio €).

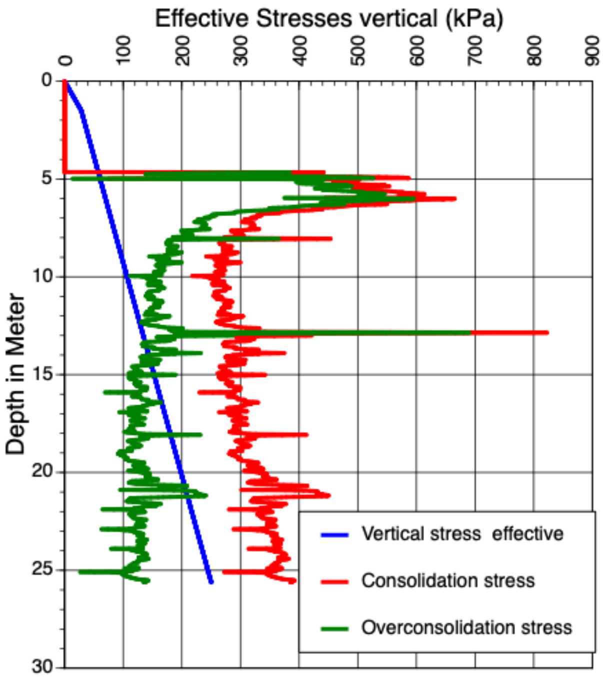

Figure 12: Comparison of consolidation stress with vertical effective stress for deposit of soft postglacial clay.

2.4. Partly draining glacial soil

Site investigation in a fine-grained fluvioglacial sediments, alternating silt, silty clay and sandy silty layers, were carreid out with DMT and CPTU. For the site with two large office buildings of 40 by 100 and 40 by 130, m additional site investigations in these fluvioglacial to glacial sediments of 20 to 25 m thickness were carried out The evaluated data shows some rather large scatter. In this case the main goal was also to estimate the overconsolidation of these soil and to design a slab foundation and save the piles.

The undrained shear strength determined by DMT in borings are shown in Figure 13. Nearly all values start on the level of normally consolidated undrained shear strength and increase over the depth of test series. The shear strength then reach 100 to 150 kPa. These values indicate that the soil is overconsolidated, as shown by the computed overconsolidation pressures (Figure 14) in the range of 300 kPa.

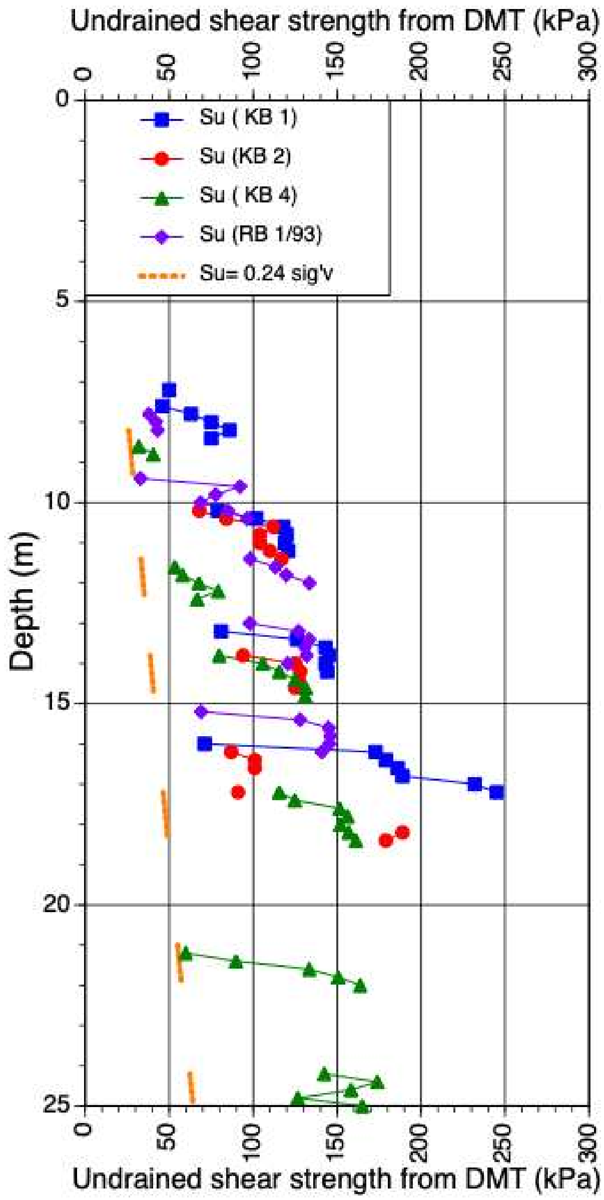

Figure 13: Undrained shear strength in fine grained fluvio glacial deposits from DMT

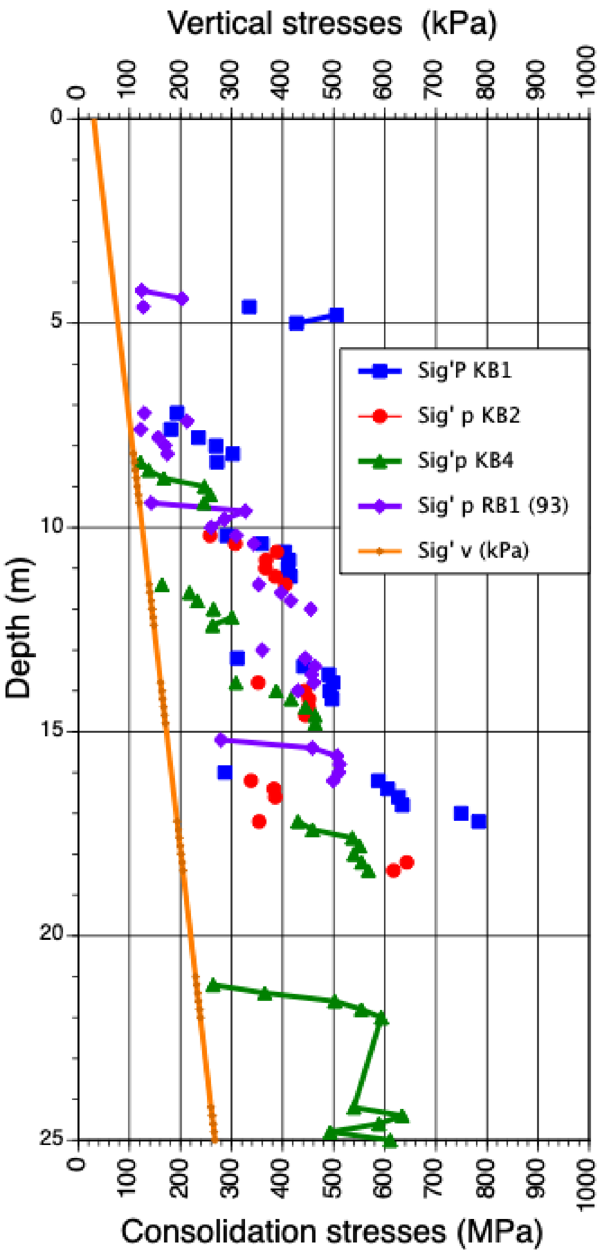

Figure 14; Preconsolidation stresses obtained from results of DMT

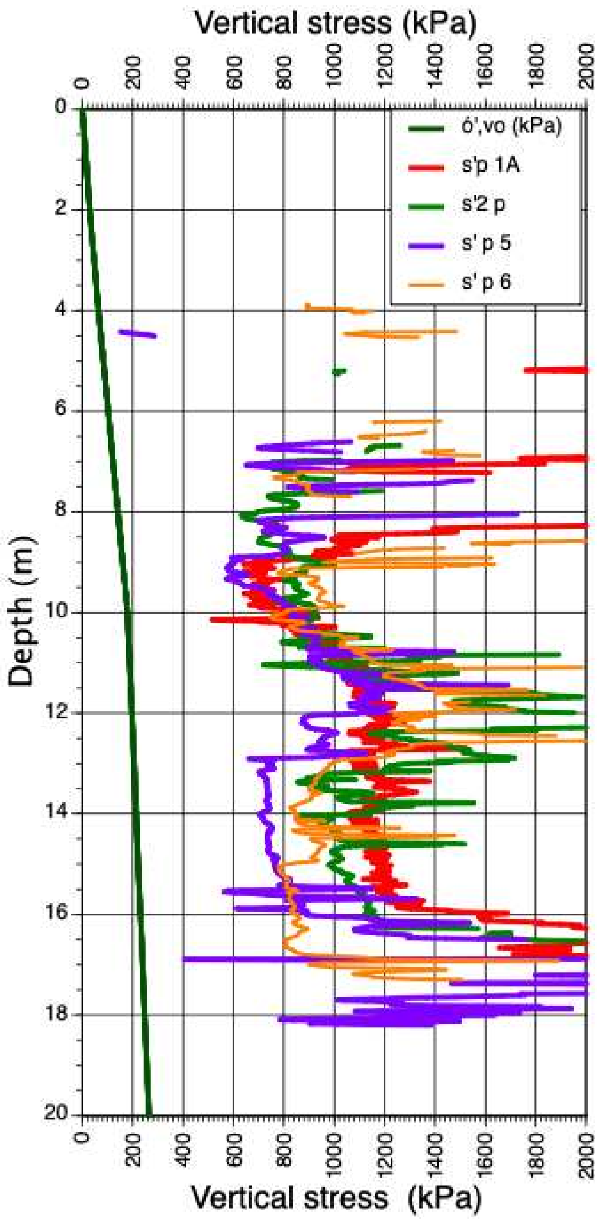

Figure 15: Over consolidation pressures estimated from CPTU tests, lower bound 600 to 800 kPa. The spikes are caused by partial drainage when the test stops to add push rods.

Prebored CPTU were used to determine the preconsolidation stresses (Figure 15). The peaks of the overconsolidation pressures coincide with the depth when a rod (each meter) had to be added to the CPT test equipment. The pushing of the rods was stopped for some ten to twenty seconds and this type of soil could partly drain during the time of the stop. When the pushing started again the cone had to overcome a substantially higher resistance as the soil had gained more resistance due to drainage. We consider the lower limit as representative for the over consolidation stress, which is about 600 kPa and corresponds to the values estimated from DMT.

For the building with a 400 x 100 m footprint five above ground stories and two basements were built and loading distributing structure was then placed on a slab formation that performed well. The coarse estimate with 15 kPa floor load gives a foundation load of 105 kPa. In addition there ia an unloading due to the excavation of about 50 kPa. With a reloading modulus ME = 50 MPa a settlement of 30…40 mm was estimated and observed.

The other building has 8 above ground stories and a different architectural lay-out with an entrance hall with wider spans and higher concentrated loads in some columns and was for structural reasons founded on piles

With the dilatometer a modulus can be directly estimated, this is a modulus under undrained conditions and the deformation of the membrane in the test crosses the preconsolidation stresses. The DMT moduli are shown in Figure 16. These „average“ moduli (Figure 16) are $M_U \approx 10$ MPa do not correspond to the reloading moduli M_E` = 50 MPa that have to be used for settlement computations and estimates.

The settlement estimate has to use information from both in-situ and laboratory tests to obtain a reliable estimate of the stress history (preconsolidation stress) and the deformation characteristics during the reloading phase and the virgin loading phase.

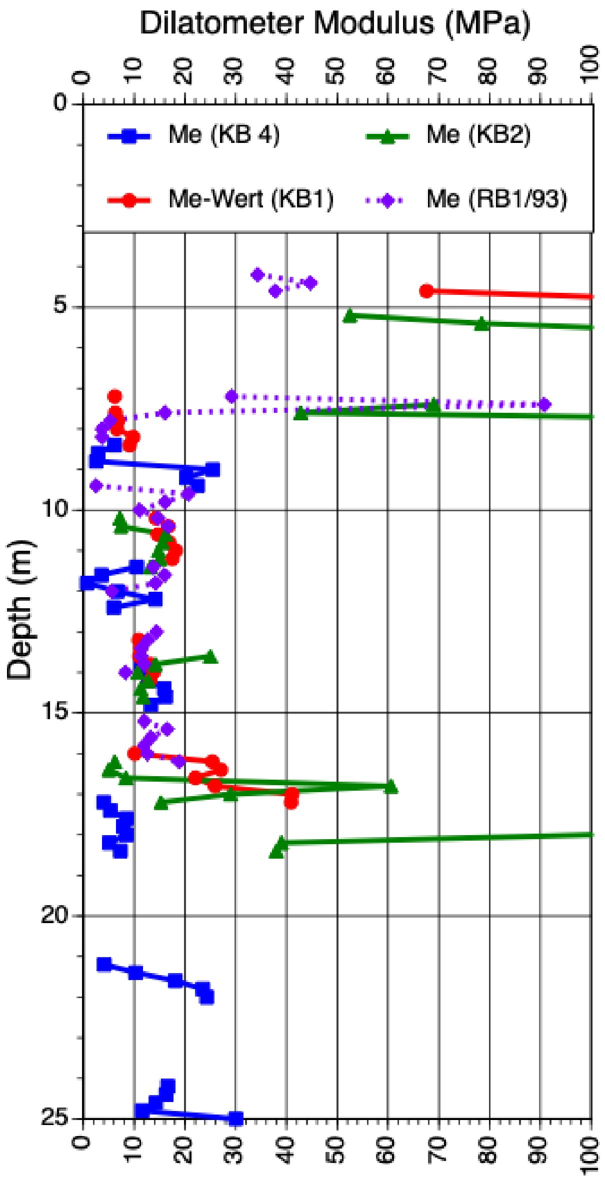

Figure 16. Directly measured DMT moduli

3. Comparison of parameters from in-situ and laboratory tests.

Some soil parameters can be determined by in-situ tests and laboratory tests. The following examples show that sometimes the in-situ test gives more reliable result, as example the preconsolidation stress. In another case different properties are detemined as in case of the vertical consolidation in the laboratory or the pore pressure dissipation test with the CPTU that gives the horizontal consolidation coefficient and the horizontal permeability. This is very important in anisotropic varved clays, but also in visually homogeneous clay.

3.1. Comparison of pre-consolidation stress

In the case of the softglacial clay [3] shown above also two incremental consolidation test in the oedometer were carried out with the usual 24 h increments. The determined pre-consolidation stresses (Figure 17) are subtantially lower about 50 kPa rather than the 120 kPa determined by CPTU.

Within a research project [4] also constant rate of strain oedometers were carried out on samples from the boring. One test at 24 m depth shows similar preconsolidation stresses as the those from CPTU. The preconsolifdation stresses from CPTU give a profile that allows also to judge their plausibility.

The parameter N_k for computing undrained shear strength data from CPTU data that are the basis for computation of the over-consolidation have been calibrated with the undrained shear strength from DMT.

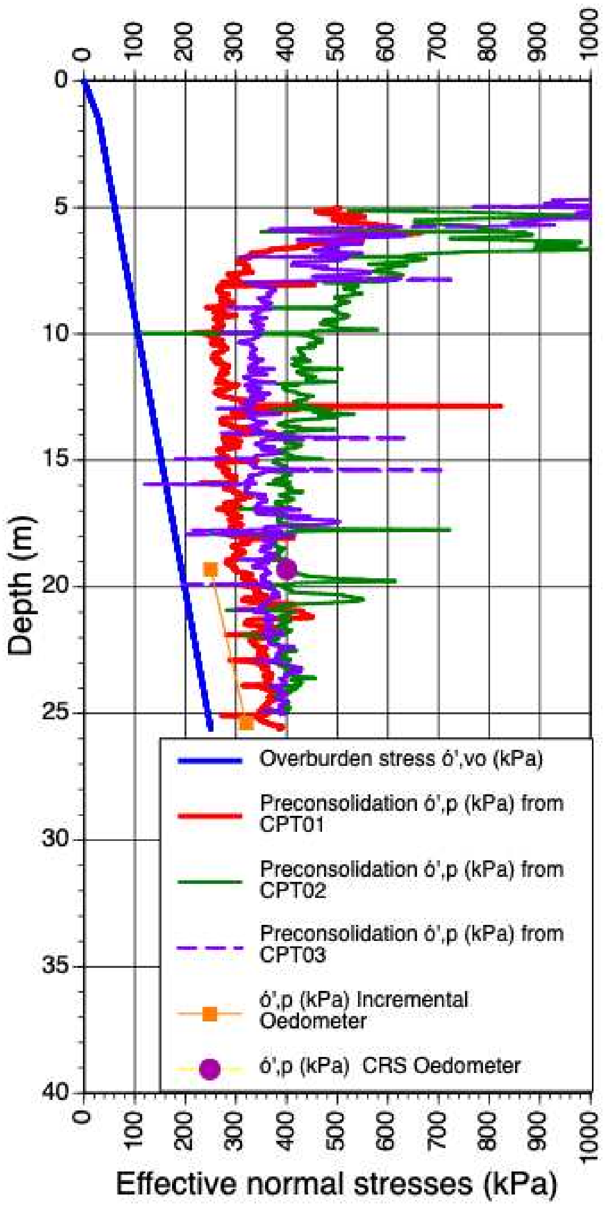

Figure 17: Preconsolidation stresses obtained from CPTU tests, incremental oedometer and CRS, constant rate of strain, oedometer tests

3.2. Comparison of permeabilities

From oedometer test the vertic al permeability and consolidation coefficient are determined. From CPTU dissipation tests the radial of the horizontal permeability or consolidation coefficient can be determined. The degree of anisotropy is an important charcteristics.

In case of a varved clay [2] the horizontal permeabilities were hundred to a thousand times larger, which was a key factor in planning the drainage scheme of the foundation (sand-drains).

3.3. Core logging and classification

In glacial soils for practically all site investigations core borings are necessary. In-situ tests. like DMT and CPTU require pre-boring. The borings should be logged and classified. The visual classification for fine-grained soil is often difficult and not precise, therefore classification tests should be made.

In order to judge the in-situ test it is important to classify the soil with the standard classification tests, i.e. grain-size distribution, Atterberg limits and water content, according to EN_ISO 14688-1 [10].

4. Some unresolved issues in glacial silts

In an area with nearly pure glacial silts four borings were drilled and in each several sets of DMT were made over about two meters length. The results from the four borings (Figure 18) show very low constrained moduli and undrained shear strength decreasing with depth to values below normally consolidated undrained shear strength, which is physically not possible. The determination of some of the parameters does not work as desired.

Dissipation tests with CPTU indicate a consolidation time of the ground around the test of 20 to 80 s, which means that soil around the DMT device is draining during execution of the tests. The measured parameters (A and B) will vary during the tests and with the manual measurements one cannot clearly define an undrained or a drained behaviour of the soil. Attempts have been made to measure the parameters A and B after drainage, which did not result in satisfactory results of the computed geotechnical parameters.

The measurements should be able to distinguish clearly between undrained and drained behaviour and might require automatic electronic recording of the parameters.

This limitation also applies to CPTU tests carried out with the standard test procedure with a rate of penetration of 2 cm/s. The diffficulty in glacial soils are that the soil type varies with depth along the sounding, it is thus not possible to adapt the rate of penetration, as may be possible for a homogeneous deposit of silt.

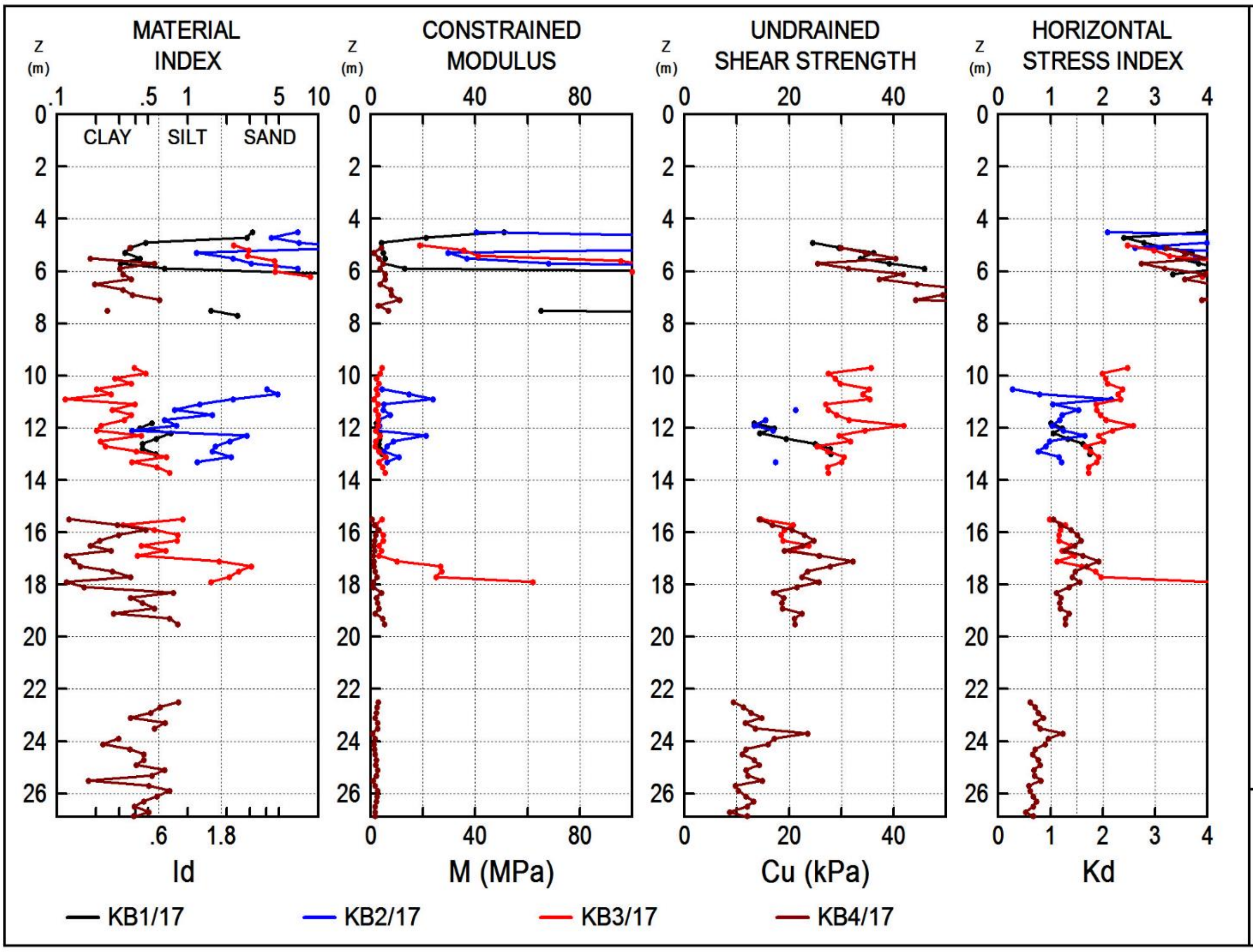

Figure 18. Results of DMT measurements from four borings in pure silt

5. Conclusions

Glacial soils pose challenges for geotechnical characterization, as they range over the entire range of grain size and have fine-grained components that have varying charactersitics from non platic to plastic fines, i.e. from silt to clay. The use of different in-situ tests combined or with laboratory tests is often required for a reliable characterisation. The use of a single type of in-situ test, like the CPTU is no panacea, the limits of the tests have to be considered.

The use of some parameters directly determined with correlations from in-situ tests may be misleading, like the Modulus for undrained behaviour, where the effect of preconsolidation is not considered.

Particular challenges remain for partly draining soils, which form a large part of glacial soils, namely silts and mixture of silts or structured soils, and also varved clays.

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