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Interpreting properties of glacial till from CPT and its accuracy in determining soil behaviour type when applying it to pile driveability assessments

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ABSTRACT: Glacial till comprises a mixture of material which has been transported by a glacier, such as clay, silt, sand, gravel, cobbles and even boulders. Due to this widespread and variable material, accurate driveability predictions can be very difficult to begin with. This study presents a driveability assessment that was performed for the installation of monopiles at a site located offshore of England's East coast where the Swarte Bank glacial till formation can be found. Pile driveability back-calculations showed the influence of assessing the soil behaviour type, the accuracy of interpreting properties of the glacial till from CPTs when applying it to pile driveability assessment, and how CPTs should be treated with more value than borehole description when assessing soil resistance to driving on Swarte Bank glacial till.

1 INTRODUCTION

Glacial till is composed of unsorted material that has been carried out and deposited directly by a glacier. Due the heterogeneity of its composition (clay, silt, sand, gravel, cobbles and even boulders) the classification, and therefore, the behaviour prediction, can be very difficult. An interpretation of the site investigation data was carried out as a basis for the driveability assessment of the monopiles used as foundations of the windfarm. To assess the soil resistance to driving (SRD), the interpretation was based mainly on the CPT results. Borehole descriptions and laboratory tests are also taken into consideration. Driveability back-calculations were performed which showed that interpreting soil properties from CPT results should be treated with more value than borehole descriptions, when assessing soil resistance to driving in Swarte Bank glacial till.

2 GLACIAL TILL AT THE WINDFARM SITE

One of the glacial till formations found at the windfarm site is named Swarte Bank. It is the most abundant of the Quaternary units, forming the thickest sediment overlying a chalk formation, that covers the entire site and forms the infill of subglacial valleys cut during the Elsterian glaciation. It is typically composed of stiff to hard sandy gravelly clay with interbedded sand layers.

Based on the site investigation results, the Swarte Bank formation was divided in two sub-units for further analysis:

– sub-unit 1 sandy, gravelly clay;
– sub-unit 2 sand interbeds which are present within the clay sub-unit 1.

CPTs, borehole descriptions and laboratory tests were analyzed, and geotechnical classification parameters were identified for each of these sub-units.

2.1 *Swarte Bank soil behaviour type*

As described in §2, the Swarte Bank formation can have intermittent clay (cohesive) and sand (cohesionless) layers. The appropriate selection of either cohesive or cohesionless behaviour is critical when undertaking driveability calculations. Soil parameters considered when selecting either cohesive or cohesionless behaviour were the pore pressure and friction ratio. An assessment of the soil behaviour type (SBT) can be performed based on CPT data as shown in past research (i.e. Robertson (1990) as in Figure 1; Robertson and Wride (1998) as in Table 1, etc.).

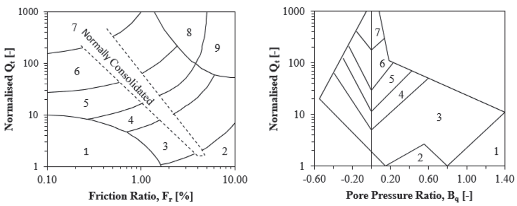

Figure 1. SBT zones Robertson (1990).

Table 1. SBT zones (Robertson and Wride, 1998).

Zone	SBT
7	Gravelly sand to sand
6	Clean sands to silty sands
5	Silty sand to sandy silt
4	Clayey silt to silty clay
3	Clay to silty clay
2	Organic soils-peats

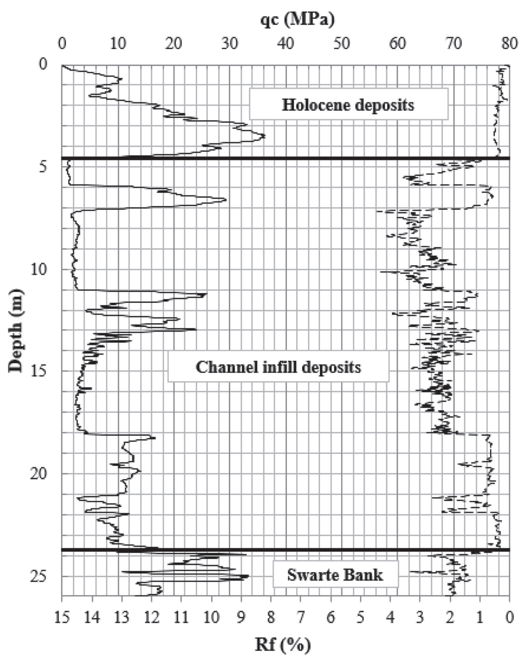

Figure 2. CPT log for Location A (qc—full line, Rf—dashed line).

It should be noted that the Swarte Bank unit was described as clay in the borehole descriptions for the majority of the locations at the site, since most CPTs refused in this unit, due to high resistance, and limited laboratory tests were performed.

Figure 2 and 3 show examples of CPT logs at 2 locations (Location A and B, respectively) where

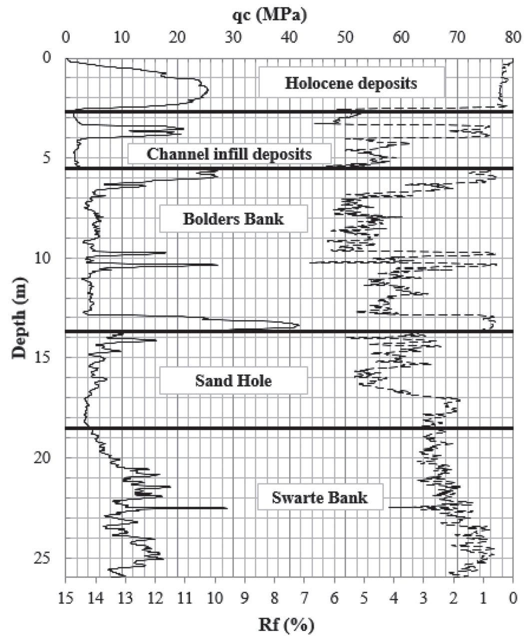

Figure 3. CPT log for Location B (qc—full line, Rf—dashed line).

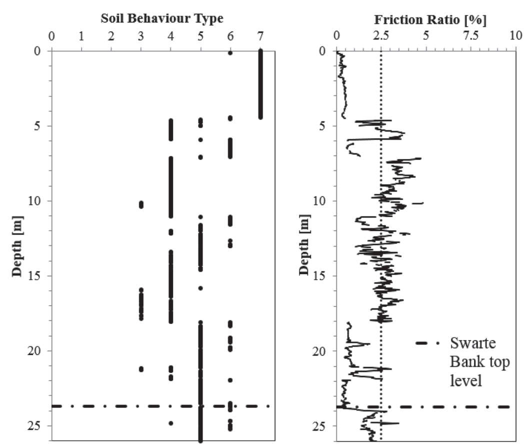

Figure 4. SBT for location A (Robertson and Wride, 1998).

the CPT did not refuse in the first meters of the Swarte Bank formation.

Based on the CPT data the SBT was derived for Locations A and B as shown in Figures 4 and 5 respectively.

Where no CPT data were available, global parameters were used for the Swarte Bank unit. These global parameters were derived based on the available consolidated undrained (CU), uncon-

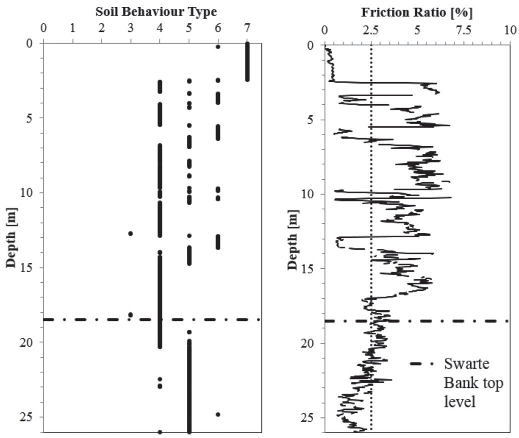

Figure 5. SBT for location B (Robertson and Wride, 1998).

solidated undrained (UU) and direct shear stress (DSS) tests.

2.2 *Geotechnical properties of the Swarte Bank sub-units*

The general properties of the two Swarte Bank sub-units identified on this site are summarized in Table 2.

The particle size distribution was determined by laboratory sieving analysis. For sub-unit 1, results are as shown in Figure 6. For sub-unit 2 only one test was performed and the result is as shown in Figure 7. Results show a sandy, gravelly clay for sub-unit 1 and a sand for sub-unit 2.

As mentioned in §2.1, where no CPT data were available the undrained shear strength values for the Swarte Bank were considered based on laboratory tests. However, values were scattered and the range of undrained shear strength obtained by laboratory tests was large; therefore, wide bounds for the undrained shear strength were adopted as shown in Figure 8 and Table 3. The best estimate (BE) and upper bound (UB) values were used for the soil profiles as the basis for assessing SRD.

For locations where CPT data were available, the undrained shear strength was derived from the q_{net} using the N_{kt} factors, as shown in equation (1):

$$Su = \frac{qnet}{Nkt} \tag{1}$$

where Su is the undrained shear strength in kPa, q_{net} is the net cone resistance in kPa and N_{kt} is the correlation factor.

Table 2. Properties of the Swarte Bank formation at the site.

Properties		Sub-unit 1 (clay)	Sub-unit 2 (sand)
Moisture content (%)		15	–
Plasticity index (%)		11	–
Effective unit weight (kN/m³)		12	10
Effective angle of friction (°)	Lower bound		34
	Best estimate		38
	Upper bound		42

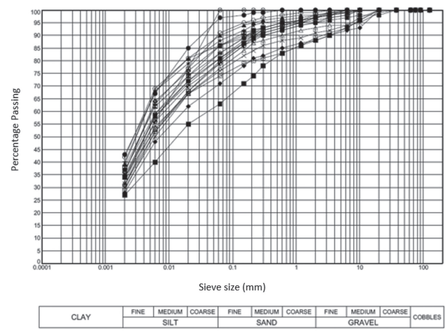

Figure 6. Sieve analysis results for sub-unit 1.

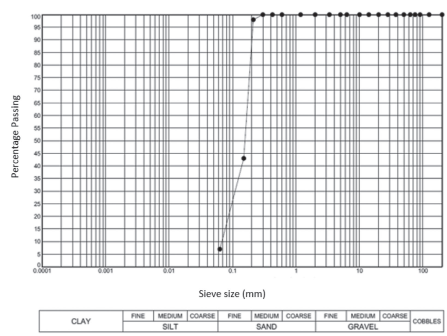

Figure 7. Sieve analysis results for sub-unit 2.

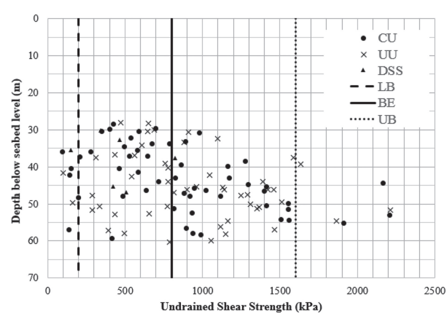

Figure 8. Undrained shear strength for Swarte Bank formation based on laboratory tests.

Table 3. Undrained shear strength bounds adopted for Swarte Bank formation at the site.

Bound	Undrained shear strength (kPa)
Lower Bound	200
Best Estimate	800
Upper Bound	1600

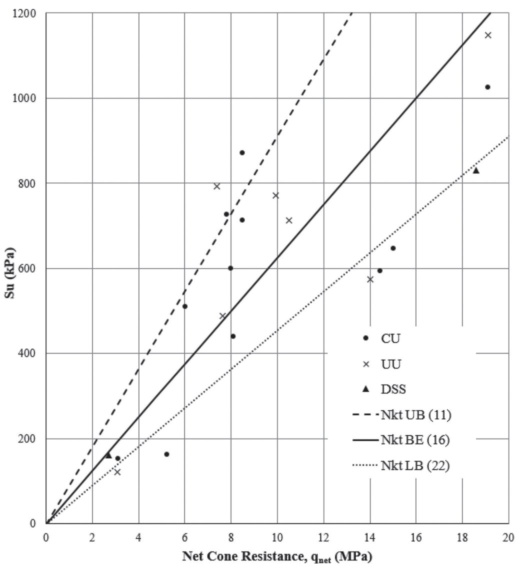

Figure 9. Calibration of N_{kt} factor for the Swarte Bank unit.

For setting up the bounds for the N_{kt} factor 11 CU, 7 UU and 2 DSS tests were available. N_{kt} factors were derived as shown in Figure 9.

3 DRIVEABILITY IN GLACIAL TILL

Monopiles were the chosen foundation solution for the wind turbines installed at the site. In order to select the most suitable hammer to be able to drive the piles to target depth, a driveability assessment was carried out. The prediction of pile driveability consists of developing a model of the hammer-pile-soil system (a wave equation model) which simulates the relationship between SRD and blow count (or pile penetration per blow).

In a separate pile analysis, the SRD was calculated based on a CPT method and used as input to GRLWEAP software (Pile Dynamics, 2010) which was used to perform the driveability assessment. In-house shaft and toe Quake and Damping parameters were used as shown in Table 4.

Table 4. Quake and damping values used in the driveability assessment.

Parameter	Value
Tip Quake	2.5 mm
Side Quake	2.5 mm
Tip Damping	0.5 s/m
Side Damping in CLAY	0.6 s/m
Side Damping in SAND	0.15 s/m

Theoretical refusal criteria significantly lower than the mechanical one was applied. This is in order to allow for some variations in soil conditions, short interruptions during driving (soil set-up) and economical pile driving.

3.1 *SRD*

There are no standardised methods for estimating SRD. Most of the better known SRD methods (Toolan and Fox, 1977; Stevens et al., 1982; Alm and Hamre, 2001) are broadly based on modifications to static pile capacity calculations. Calibration and verification of the SRD methods are based on the use of field data (measured hammer energy and blow counts) combined with wave equation modelling of the hammer-pile-soil system which provides the relationship between blow count and SRD for a given piling situation.

For this offshore wind farm SRD was assessed using a CPT method based on Alm & Hamre (2001) which calculates SRD from CPT cone tip resistance (q_c) data. It has a different method for both sand and clay and the selection of the soil type of the different layers is a manual input.

3.2 *Modelling the Swarte Bank formation*

An evaluation to assess the sensitivity of the CPT interpretation is undertaken. Back-calculations are performed by considering two different approaches for modelling the Swarte Bank formation:

Approach 1: Assess Swarte Bank as a cohesive material (clay), in line with the borehole descriptions.

Approach 2: Assess the Swarte Bank as clay when friction ratio values exceeded 2.5% and when SBT is between 2 and 4 according to Robertson and Wride (1998) (Table 1), and as sand when friction ratio does not exceed 2.5% and when SBT is between 5 and 7 according to Robertson and Wride (1998) (Table 1).

A comparison between the two approaches is presented below.

3.3 *Driveability assessment results (back-analysis)*

The pile driving records of the installed monopiles at the site were subjected to back-analysis. The stroke height was rescaled to match the applied hammer energy on site. With the rescaled theoretical stroke height a new GRL-WEAP simulation is made. Afterwards, the obtained blowcount vs depth curve is compared to the calculated ones.

Results are presented for the two locations introduced before (Locations A and B), representative of the general trend of results of the back-calculations performed for locations where the pile penetrated the Swarte Bank Unit.

The top of the Swarte Bank layer is at 23.7 m and at 18.5 m depth at Locations A and B, respectively.

3.3.1 *Swarte Bank sub-unit 1 and 2 as cohesive material*

The back-calculations performed for these two locations showed that blowcount is overpredicted on the Swarte Bank formation if this formation is assessed as clay, especially when upper bound driving conditions are considered, as shown in Figures 10 and 11.

3.3.2 *Swarte Bank sub-unit 1 and 2 depending on SBT and the friction ratio value*

For this approach, the friction ratio values, the 2.5% boundary and SBT were considered. CPT data show that at location A the friction ratio within the Swarte Bank formation (23.7 m to 26.3 m) was found to be around 2.0% and SBT was found to be between zones 5 and 6; hence this soil was modelled as sand. For Location B, since the friction ratio was found to be around 2.5% from 18.5 m to 20.5 m and thereafter to decrease to a constant value of around 2.0% for the rest of the unit, and SBT was found to be mainly in zone 5, the Swarte Bank was modelled as a clay above 20.5 m and as a sand below 20.5 m.

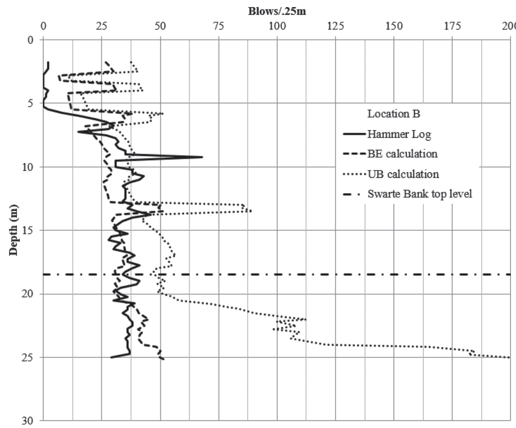

Figure 11. Back calculation location B—Approach 1.

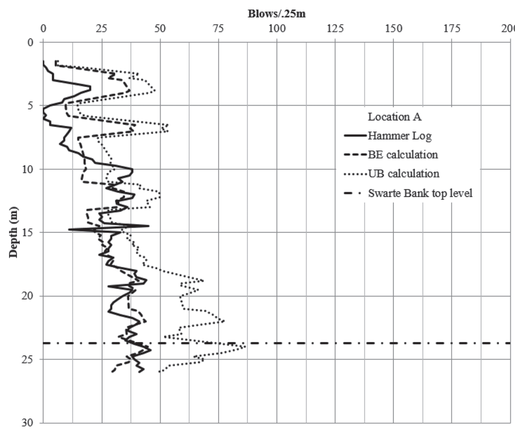

Figure 12. Back calculation location A—Approach 2.

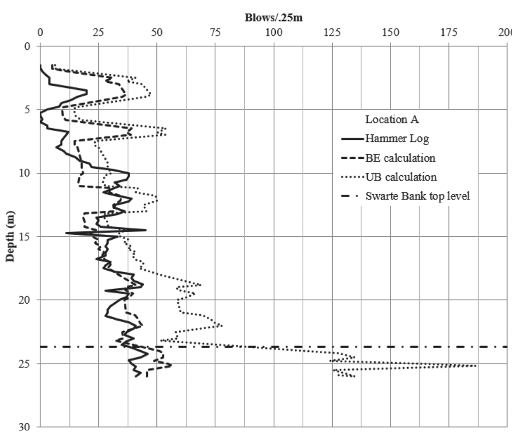

Figure 10. Back calculation location A—Approach 1.

The results are as shown in Figures 12 and 13.

As shown in Figures 10 to 13, addressing the Swarte Bank formation based on the friction ratio values and SBT (Approach 2), gives a much better fit with the measured driving results than when based on the borehole logs only (Approach 1). This is especially noticeable for Location B where a good fit with the hammer log is found by modelling the Swarte Bank formation as clay (from 18.5 to 20.5 m) and as sand (below 20.5 m).

It should also be highlighted that when the Swarte Bank formation is modelled correctly, the BE blow vs depth curve obtained by using the CPT method gives a reliable prediction.

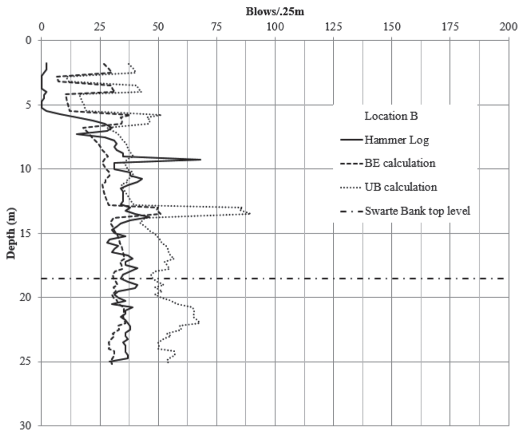

Figure 13. Back Calculation Location B—Approach 2.

4 CONCLUSIONS AND RECOMMENDATIONS

Glacial till comprises a mixture of unsorted material that has been carried by a glacier. One of the glacial till formations found at the windfarm site is named Swarte Bank. In many instances this formation is considered as "borderline" material with intermittent clay (cohesive) and sand (cohesionless) layers, which presents a challenge when describing this formation while logging the borehole cores. The selection of cohesive or cohesionless behavior for this material must be performed with caution since it can lead to an inaccurate prediction of soil resistance which is critical when undertaking driveability calculations.

Referring to the outcome of this paper, the use of the Robertson and Wride (1998) SBT index together with the friction ratio parameter, which can be applied with depth, gave an accurate representation of the method (sand/clay) to be used when predicting soil resistance in glacial till, particularly in the case of the Swarte Bank formation. By considering the SBT and the friction ration as a primary input to model the Swarte Bank formation, a better fit between the measured and the predicted blow count has been achieved. Hence, the use of CPT, especially in challenging materials such as glacial till, is promoted as it will have a positive effect on the selection of an appropriate hammer energy.

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REFERENCES

America Petroleum Institute (2000). *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms—Working Stress Design. Recommended Practice 2A-WSD (RP 2A-WSD).* Washington: API Publishing Services.

Alm, T. & Hamre, L. (2001). Soil Model for pile driveability predictions based on CPT interpretations. *Proceedings XV-th International Conference on Soil Mechanics and Geotechnical Engineering* Vol.3: 1297–1302

Lunne, T., Robertson, P.K. & Powell, J.J.M. (1997). *Cone Penetration testing in geotechnical practice*. London: Blackie Academic and Professional.

Pile Dynamics (2010). *GRLWEAP 2010 Background Report*. Cleveland: Pile Dynamics.

Robertson, P.K., (1990). Soil classification using the cone penetration test. *Canadian Geotechnical Journal* Vol. 27(1): 151–158.

Robertson, P.K. & Wride, C.E. (1998). Evaluating cyclic liquefaction potential using the cone penetration test. *Canadian Geotechnical Journal* Vol. 35(3): 442–459.

Stevens, R.S., Wiltsie, E.A. & Turton T.H.(1982). Evaluating pile driveability for hard clay, very dense sand and rock. *Proceedings 14th Offshore Technology Conference* paper OTC no. 4205.

Toolan, F.E. & Fox, D.A. (1977). Geotechnical planning of piled foundations for offshore platforms. *Proceedings of the Institute of Civil Engineers* Vol. 62 (1): 221–244.

Trenter, N.A. (1999). *Engineering in glacial tills*. London: CIRIA.