Comparison of pile driveability methods based on a case study from an offshore wind farm in North Sea

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ABSTRACT
Significant research effort has been put into pile driveability analyses with the aim of determining a successful, safe and cost-efficient installation. Driveability analysis involves selection of appropriate hammer, determination of pile makeup details and careful review of soil profile to reach desired penetration or capacity with reasonable number of blows without overstressing the pile.
In this paper, pile driving records from the installation of 6.5 m diameter monopiles at a wind farm in southern North Sea are considered. The ground conditions at the site generally consist of between 10-50 m thickness of over consolidated clay with some layers of sand overlying chalk bedrock. The most important of the variables to establish is the Static Resistance to Driving (SRD). There are proposed procedures for evaluating SRD in sands and clays; however, the knowledge about pile driveability in the chalk at the site is very limited. This makes prediction of the soil response after driving the pile into the chalk layers unreliable. The piling records are used to test how well the existing driveability suit the conditions at this site by comparing the predicted blow counts with results from back-analyses of as-measured pile driving records.

Keywords: pile driving, backanalysis, chalk

1 INTRODUCTION

Continuous growth in need of renewable energy demands new economical and technologically feasible innovations. In order to overcome increasing depths, dimensions of the offshore structures as well as foundations become larger.

According to Karimirad (2014), more than 65% of the offshore wind turbines are monopile structures and significant research effort has been put into accurately predicting the pile response to driving.

Pile driveability denotes the ability of a pile to be safely and economically driven to the required depth without causing excessive fatigue damage. The analysis for a particular set of driving equipment, pile material and dimensions, and a specific type of soil at the site involves a detailed static and dynamic soil resistance input parameters to reflect layers that pile penetrates.

Predicting Soil Resistance to Driving (SRD) has been a challenging task and some

of the methods used nowadays include procedures given by Toolan and Fox (1977), Stevens (1982), Alm and Hamre (2001).

The design of monopile foundations for offshore wind turbines relies heavily on experience and approaches used in the oil and gas industry, however these methods were developed when most of piles installed offshore had a diameter of less than 2 m.

This paper aims to evaluate the accuracy of existing methods for 6.5 m diameter monopiles at the Westermost Rough wind farm in southern North Sea where ground conditions generally consist of over consolidated clay with some layers of sand overlying chalk bedrock. Data from pile installations have been gathered and used as input into back-analysis to test how well present driveability models suit the conditions at this particular site.

2 SITE CHARACTERISATION

The Westermost Rough offshore wind farm is located in the North Sea, around 8 km off the Yorkshire Coast north of Hull and covers an area of approximately 35 km^2 (Figure 1).

Figure 1. Location map of the Westermost Rough offshore wind farm

2.1 Seabed and bathymetry

Geophysical survey indicated that within the area water depths range between 11 m LAT (lowest astronomical tide) and 28 m LAT. The seabed generally shoals to the southwest with gradient less than 1 degree, except where current related features, evaluated as possible relict sand waves or eskers, up to 7.0 m high, were present.

2.2 Geological setting

Based on extensive geotechnical, geological and geophysical logging data from ground investigations, it was recognized that the site consists of quaternary soils overlying chalk bedrock.

Holocene Deposits (HLCN)

Holocene Deposits cover seabed across the area of wind farm and are typically comprised of sand, sandy gravels and low to high strength clays between 0.2 m and 3.7 m thick.

Channel Infill Deposits (CHF)

Channel Infill Deposits consist of very low to low strength silty clays and silty sand, with thickness ranging between 3 m and 8 m along the eastern edges of the wind farm site, locally thickened from 16 m to 22 m in the northern corner of the site.

Bolders Bank Formation - Upper (BSBK_U)

The deposits comprise very stiff, high, very high and extremely high strength, slightly sandy to very sandy gravelly clay, reddish brown, becoming brown and greyish brown with depth.

Bolders Bank Formation – Middle (BSBK_M)

The deposits of thickness between 1 m and 10 m comprise gravelly sands, locally encountered as sandy gravel or cohesive soil with a high proportion of granular material.

Bolders Bank Formation – Lower (BSBK_L)

The deposits are between 1 m and 12 m thick and comprise very stiff, high, very high and extremely high strength brown, dark brown to reddish brown, slightly sandy, slightly gravelly clay.

Rough Formation (ROUGH)

Rough Formation deposits are found within local channel features cut into the Chalk, with thickness varying between 1.3 m and 13 m.

The deposit comprises low plasticity, very high to extremely high strength, sandy gravelly clays.

Swarte Bank Formation (SWBK)

Swarte Bank Formation deposits locally underlay the Rough Formation deposits. They comprise a light grey diamict with an almost complete absence of clast lithologies other than chalk and occasional flint. The thickness varies between 1.5 m and 13 m.

Westermost Rough Chalk Formation (WMR)

The top of the chalk surface varies along the site. From the central northern part of the site to its southwestern corner, the top of the chalk surface is from 28 m to 40 m below seabed. On the other positions, though, the top of the chalk surface is observed from 10 m to 19 m below seabed. The chalk comprises generally extremely weak and very weak, low density, creamish white and white chalk. However, this chalk has a general absence of flint bands and marl seams, making it different to chalk of similar age encountered onshore. It is assumed that this particular chalk formation has not been previously logged and therefore it is called the Westermost Rough Chalk Formation.

The relevant chalk characteristics for pile design and installation are the intact strength (directly related to porosity/density) and the fracture condition that is defined by the CIRIA grade (Lord, Clayton, Mortimore, 2002). The chalk at WMR is low to medium density and consists of three geotechnical units: structureless chalk (CIRIA Grade D), structured fractured chalk (CIRIA Grade B and Grade C) and structured assumed intact chalk (CIRIA Grade A).

2.3 *Geotechnical profiles at the site*

In addition to the identification of soil layers based on the geological description of soil samples recovered from boreholes at selected locations and the interpretation of the geophysical surveys, the formations were recognized by cone penetration tests that were carried out at all wind turbine locations.

The cone penetration test (CPTs) performed at the site were specified as CPTU tests, i.e. including pore pressure readings. The outcome of the CPT classification is a refinement of the complete soil stratigraphy, determination of specific depths of different geological units and identification of layers with different engineering properties being visible from the increase or decrease in the measured cone resistance and skin friction.

CPT data from several observed locations (P01, P02, P03, P04, P05 and P06) are illustrated in Figure 2 and design soil parameters are specified in Table 1, where γ' (kN/m^3) is effective unit weight, D_d (%) is relative density, φ (°) is friction angle and s_u (kPa) is undrained shear strength. Plasticity index PI (%) is 16-17 for ROUGH and BSBK formations and 8-9 for chalk D, CHF and SWBK formations.

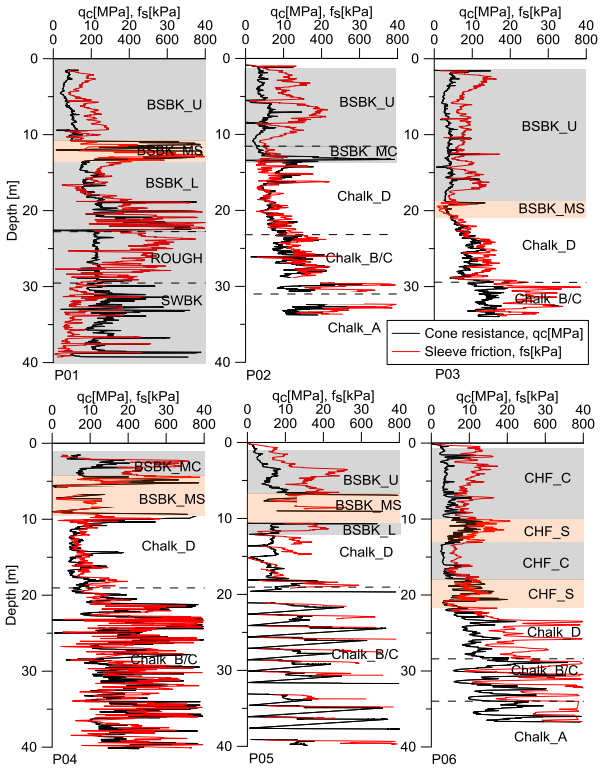

Figure 2. CPT profiles for position P01-P06

Table 1. Soil properties at six positions

	γ' [kN/m^3]	D_r [%]	s_u [kPa] φ [°]
HLCN	7	15[aef], 27[c], 36[b], 46[ad]	35°
CHF_C	11	-	(140-280)[f]
CHF_S	7	(27-40)[f]	(31°-33°)[f]
BSBK_U	11	-	(130-280)[a] (120-330)[b] (130-230)[c] (130-530)[e]
BSBK_L	11	-	(470-1500)[a] 350[e]
BSBK_MC	11	-	(160-240)[b] (240-390)[d]
BSBK_MS	10	15[c] (65-80)[a] (80-100)[de]	(28°)[c] (34°-40°)[ade]
ROUGH	11	-	615[a]
SWBK	10	-	(750-930)[a]
WMR_D	9.3	-	125
WMR_B/C	9.3	-	-
WMR_A	9.3	-	-

*Index a-f corresponds to positions P01-P06, respectively

It should be noted that due to poor CPT readings in sand layers at positions P04 and P05, and in chalk layer of grade B/C at position P05, the values of cone resistance and skin friction at these locations should be taken with caution.

Table 1 also demonstrates how soil parameters can vary significantly from one position to another, even in the same geological unit.

3 DRIVEABILITY ANALYSIS

The total resistance to driving may be divided in a static part, the static resistance to driving (SRD) and a velocity or displacement rate dependent part called the damping. Evaluation and development of correct input of static resistance is of high importance to obtain an accurate model. In order to determine SRD, common practice is to relate it to the Static Soil Resistance; American Petroleum Institute (API) proposes such methods. There are number of methods presented over the years and are still in use in North Sea pile design.

The earliest models like Toolan and Fox (1977) did not include friction fatigue concept, which was presented in 1978 by Heerema who made driveability prediction based on the assumption that skin friction in clay is gradually lost along the pile wall as driving proceeds (Heerema, 1978). Semple and Gemeinhardt's method from 1981 related unit skin friction to clay stress history (Semple and Gemeinhardt, 1981). In 1982, Stevens adopted model by Semple and Gemeinhardt. The methods mentioned above are referred to as traditional methods, while recently developed models are usually based on CPT data (Alm and Hamre, 1998).

Three driveability approaches have been selected for the purpose of this paper, some are slightly modified in order to achieve better estimation of the ground conditions at this particular site and a brief summary of each is described in section below.

3.1 Methodology for estimating SRD

Toolan and Fox (1977)

This SRD model, referred to as Toolan and Fox method in this paper, proposes unit skin friction in clays is equal to remoulded undrained shear strength. However, this parameter is difficult to measure accurately, so a portion of measured undisturbed strength is often assumed, expressed by the factor α. A range of α values were considered in order to determine the most appropriate value for each type of clay at this location and following values were chosen: 0.5 for CHF_C, BSBK_U, BSBK_MC, BSBK_L, and 0.4 for ROUGH and SWBK formations. Unit skin friction is then expressed as

$$f_s = \alpha \cdot s_u \tag{1}$$

The unit end bearing in clay is set equal to the cone tip resistance.

In this study, the unit skin friction for granular soils is not computed according to

original formulation, where it is calculated as fraction of the recorded cone tip resistance (1/300 for dense sand), but according to API (API RP 2A, 1981) as

$$f_s = 0.8 \cdot \sigma'_{v0} \cdot \tan(\varphi - 5°) \quad (2)$$

where σ'_{v0} is the effective vertical stress (kPa) and φ is the angle of internal friction.

Unit end bearing in granular soil is assumed one third of the cone tip resistance. It is generally accepted that the behaviour of large diameter piles is fully coring, implicating that unit skin friction is applied to the external and internal pile wall and unit end bearing to the cross-sectional area of the pile.

The model is also applied for chalk. The grade D chalk is treated as clay. For other grades of chalk, unit end bearing is calculated as 60% of the cone tip resistance, and unit skin friction is set to 20 kPa.

Stevens et al. (1982)

Four cases are normally studied for this method, lower and upper bound coring, and lower and upper bound plugged, but in this analysis, only coring will be considered. In the original paper (Stevens et al., 1982) lower bound assumes that internal skin friction is 50% of the external skin friction, and upper bound assumes they are equal. This analysis considers best estimate case as original upper bound case, where equal skin friction acts on the inside and outside of the pile wall. In granular soils, both unit skin friction and unit end bearing are calculated using static pile capacity procedures.

$$f_s = 0.7 \cdot \sigma'_{v0} \cdot \tan(\varphi - 5) \quad (3)$$

$$q_{tip} = 40 \cdot \sigma'_{v0} \quad (4)$$

For cohesive soils, unit skin friction is computed using stress history approach presented by Semple and Gemeinhardt (1981), and unit end bearing as defined in the API (API RP 2A, 1981).

$$f_s = \alpha \cdot 0.5 \cdot (OCR)^{0.3} \cdot s_u \quad (5)$$

$$q_{tip} = 9 \cdot s_u \quad (6)$$

where *OCR* is overconsolidation ratio and α is parameter calculated using the expression from API (1981).

This model is also applied for chalk and uses the same procedure as Toolan and Fox model. The method is based on best estimate soil parameters, factors are then applied to both calculated skin friction and end bearing according to original paper to obtain different driveability cases. Further on, the method is referred to as Stevens method.

Alm and Hamre (2001)

The model was first introduced in 1998 and updated in 2001 to offer a direct correlation for unit end bearing and skin friction with the CPT. Since major contribution to SRD is due to side friction, this method includes friction fatigue concept, a reduction in unit skin friction with increasing pile penetration. The unit skin friction for cohesive soils is

$$f_s = f_{sres} + (f_{si} - f_{sres}) \cdot e^{k \cdot (d-p)} \quad (7)$$

where f_{si} is the measured cone skin friction and f_{sres} is the residual friction, calculated as

$$f_{sres} = 0.004 \cdot q_c \cdot (1 - 0.0025 \cdot \frac{q_c}{\sigma'_{vo}}) \quad (8)$$

and shape degradation factor is expressed as

$$k = (q_c/\sigma'_{v0})^{0.5}/80 \quad (9)$$

where *d* (m) is depth to the soil layer, *p* (m) is pile tip penetration and q_c (kPa) is cone tip resistance. Unit end bearing is calculated as 60% of the cone tip resistance.

The unit skin friction for granular soils is computed in the same way as for the cohesive soils, however the initial skin friction f_{si} is calculated as

$$f_{si} = K \cdot p'_0 \cdot \tan(\delta) \quad (10)$$

where K is calculated as

$$K = \frac{0.0132 \cdot q_c \cdot (\sigma'_{v0}/p_a)^{0.13}}{\sigma'_{v0}} \quad (11)$$

The residual friction is calculated as 20% of the initial friction, which is equal to measured cone skin friction. The end bearing is computed as

$$q_{tip} = 0.15 \cdot q_c \cdot (q_c/\sigma'_{v0})^{0.2} \quad (12)$$

The chalk is treated as clay for both skin friction and end bearing. The details of this model are given in the original article (Alm and Hamre, 2001). Further on, the method is referred to as Alm and Hamre method.

3.2 Methodology for backanalysis

To simulate the actual driving conditions, the hammer stroke is adjusted according to the driving energy used during installation. Normally a driveability analysis is performed using the full hammer stroke to evaluate if the selected hammer is able to drive the pile to target depth. By adjusting the hammer stroke, the actual hammer energy recorded in the driving log at the time of installation is used to demonstrate how the predicted SRD suits soil conditions. Bearing in mind that the pile experiences both static and dynamic resistance during driving, the method relies on the wave equation analysis program GRLWEAP (Pile Dynamics, 2010), where dynamic forces are represented by damping parameters. Smith (Smith, 1960) gave the total resistance mobilized during dynamic loading as

$$R_d = R_s(1 + J \cdot v) \quad (13)$$

where R_d is dynamic soil resistance, R_s is static soil resistance, J is a damping constant and v is velocity of a pile segment during a given time interval.

The dynamic soil parameters are an integral part of any pile driveability assessment and it is common for an SRD model to have a set of associated quake values and damping factors.

In all cases, the associated side and toe quakes are 2.5 mm and toe damping J_p is 0.5 s/m. The selected parameters are in accordance with the best practice (Pile Dynamics, 2010). The damping parameters used in this analysis are presented in Table 2.

Table 2. Damping parameters

Soil unit	Method	Skin Damping J_s [s/m]
CHF_C BSBK_MC BSBK_U BSBK_L	Toolan&Fox	0.66
	Stevens	0.23
	Alm&Hamre	0.25
HLCN CHF_S BSBK_MS	Toolan&Fox	0.25
	Stevens	0.16
	Alm&Hamre	0.25
ROUGH SWBK	Toolan&Fox	0.23
	Stevens	0.23
	Alm&Hamre	0.25
WMR Chalk	Toolan&Fox	0.65
	Stevens	0.65
	Alm&Hamre	0.25

4 BACKANALYSIS

The main objective of this paper is to show the results of predicting pile driveability based on the methods commonly used in the industry today. Due to the complex local site conditions, the analysis resulted in a significant overestimation of soil resistance to driving in chalk layers and slightly underestimation in clay or sand layers above. The results from only six positions (CPT data illustrated in Figure 2) out of 35 that were analysed, will be discussed below (Figures 4-9).

It is important to outline that the primary concern of analysis done in this paper is prediction in chalk, so only positions that penetrate this formation are referred to as good/poor predictions. Positions located from northwest to northeast generally give poor prediction, especially ones where water depth is larger (indicated with red rectangle in

Figure 3). However, there are exceptions, for example position P06 (discussed later in the paper).

It is stated in the API (API RP 2A-WSD, 2010) that the exact definition of refusal for a particular installation should be defined in the installation contract and should be adopted to the individual soil conditions, hammer and pile dimensions. At this specific location refusal is encountered when one of the following criteria is met: 125 blows per 0.25 m in six intervals of 0.25 m (500 bl/m), 200 blows per 0.25 m in two intervals of 0.25 m (800 bl/m), 325 blows per 0.25 m in one interval of 0.25 m (1300 bl/m) or 325 blows per 0.25 m in two intervals of 0.25 m (1300 bl/m).

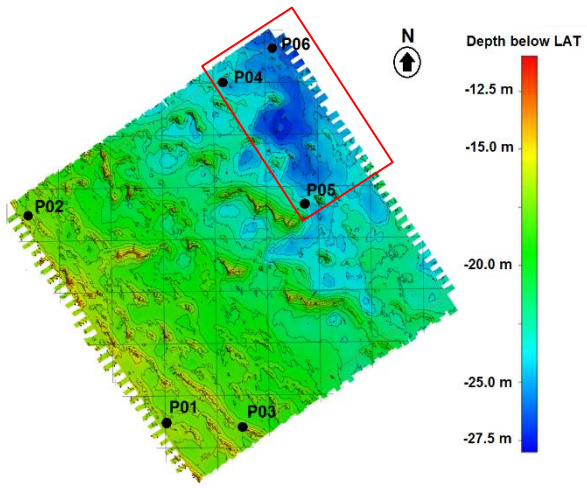

Figure 3. Layout of the windfarm and water depths

Information about pile make up and penetration are given in Table 3. The hammer used in installation process was IHC-S2000, with the rated energy of 2000 kJ and the stroke of 2.02 m.

Position P01 presented in Figure 4 differs from other positions chosen for analysis in this paper because it reaches the target depth without penetrating into the chalk formation.

Table 3. Pile details

	Penetration depth [m]	Penetration into chalk [m]	Wall thickness at tip [mm]
P01	21.66	0.0	73
P02	26.96	13.56	72
P03	31.06	10.26	72
P04	25.96	15.16	72
P05	31.06	20.86	72
P06	28.46	6.76	72

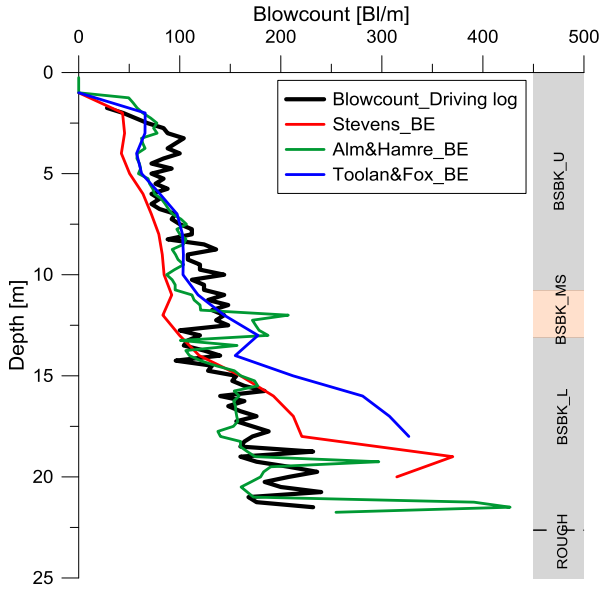

Figure 4. Driveability predictions for P01

As can be observed in Figure 4, both Stevens and Toolan and Fox methods show underestimation in the upper clay layers, but they tend to overestimate number of blows in the lower layers of clay, reaching refusal at 20.9 m and 20.1 m below seabed, respectively. At these depths, the s_u profile, derived from the net cone resistance and a cone factor N_{kt} of 18.5, gives extremely high values of undrained shear strength.

Alm and Hamre method, which relies entirely on CPT data, provided a good best estimate prediction, with a slightly overestimated number of blows in sand layer.

Figures 5-6 show driveability predictions in positions P03 and P06 where chalk formation is found at depth of 20.75 m and 21.7 m below seabed.

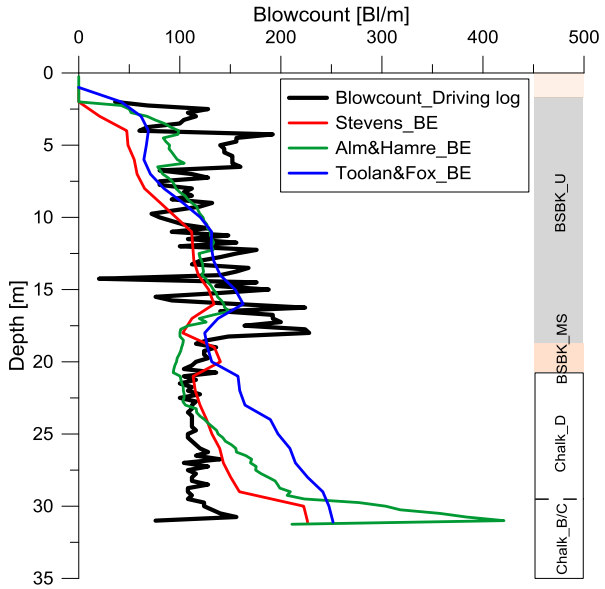

Figure 5. Driveability prediction for P03

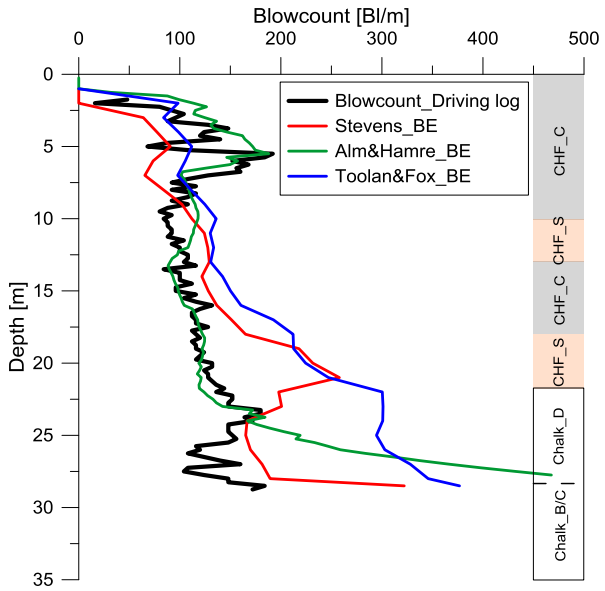

Figure 6. Driveability prediction for P06

These positions are considered to have a reasonable prediction of number of blows in chalk layers by Stevens method. In clay, both Stevens and Toolan and Fox methods underestimate the blowcount, while overpredicting it in sand (from 18.0 to 22.0 m in Figure 6).

The increase in blowcount is visible after depth of 29.5 m (P03) and 28.3 m (P06), which can be related to change in calculation procedure for chalk grade D and B/C.

Alm and Hamre method follows the blowcount trend from driving log but does not predict well number of blows in chalk. It is important to keep in mind that the method was originally developed only for sand and clay, nevertheless in this paper it is also used for chalk under assumption it behaves as clay.

Figures 7-9 show backanalysis results for positions where head of the chalk unit is found at 13.4, 10.0 and 12.1 m below seabed.

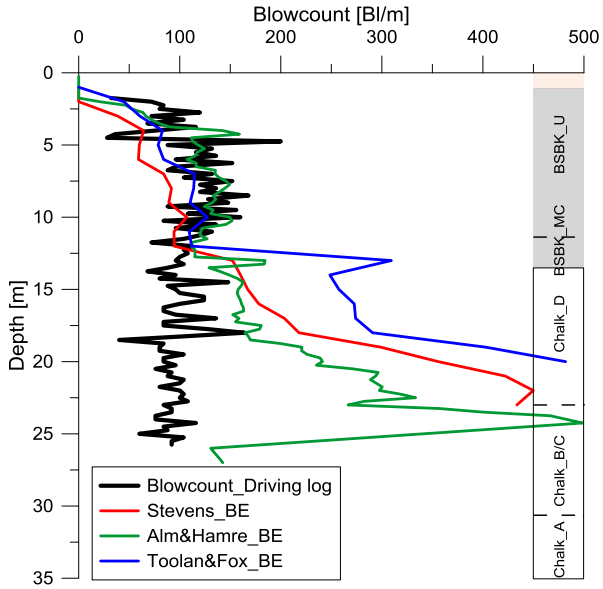

Figure 7. Driveability prediction for P02

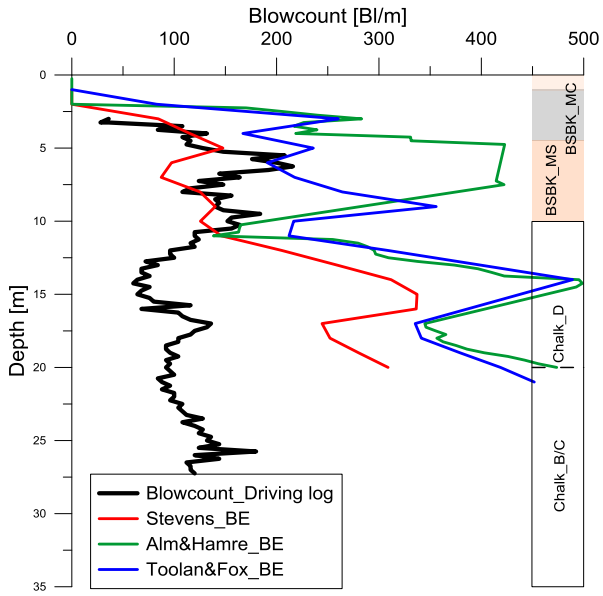

Figure 8. Driveability prediction for P04

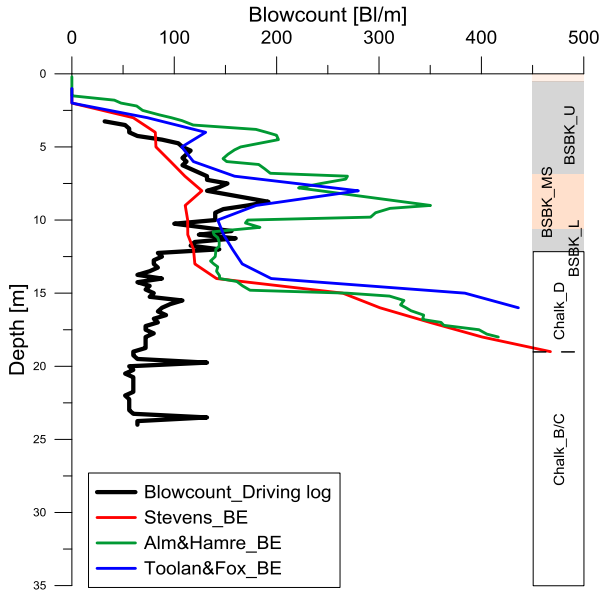

Figure 9. Driveability prediction for P05

Stevens best estimate method gives underestimation of number of blows in clay and sand layers, but then tend to overestimate it greatly in chalk layers below. Refusal is encountered at 24.4 m (P02), 22.1 m (P04) and 19.9 m (P05). Since major part of SRD is due to skin friction, especially for chalk of grade D, the overestimation in results indicates that soil showed much less resistance than expected. Figure 10 representing energy used by the hammer during driving confirms this assumption.

Good prediction of blowcount in clay is found at positions P02 and P05 with Toolan and Fox method, but it tends to overestimate number of blows in sand layer (also seen at P04). Overestimation in chalk at these positions is large, accompanied by reduction of energy used by the hammer.

Alm and Hamre best estimate method captures well blowcount prediction in clay and sand layers at P02, but overestimates it in chalk before meeting refusal at 25.7 m. The same method does not provide good results for sand layers at positions P04 and P05, overestimating the number of blows by up to 100%, what can be explained by poor CPT data found in those layers, since the method relies directly on measured skin friction and cone resistance. The refusal on these locations is met at 5.51 and 8.3 m below seabed. The hammer energy at P04 and P05 was low, around 13 and 18%, meaning that encountered resistance was not high.

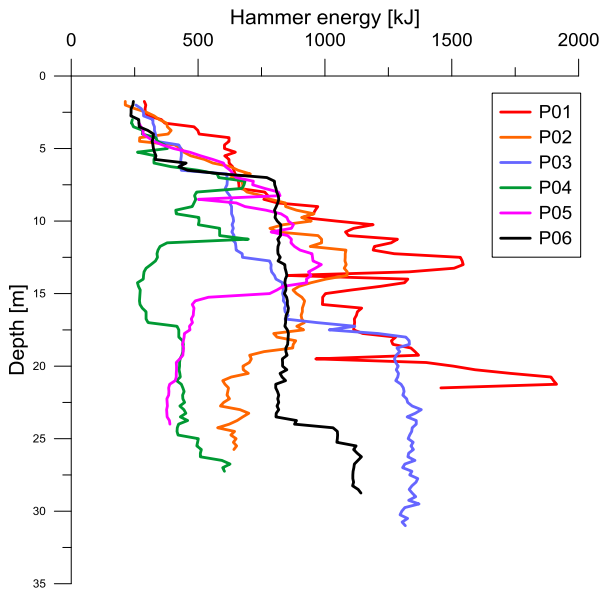

Figure 10. Energy used by the hammer

5 OBSERVATION

One of the possible reasons for deviations in backanalyzed number of blows should be discussed within the energy domain of driveability analysis. Future work will therefore be focused on inspection of static resistance curve that is being used in GRLWEAP model, as the authors' opinion is that analysis with quake and damping settings presented in paragraph 3.2 might work best only for high energy close to rated hammer energy.

6 CONCLUSIONS

Driveability approaches used in industry today were developed for relatively small diameter piles. According to analysis presented in this paper, using these methods to predict behaviour of large offshore monopiles does not provide good estimation, especially when found in complex site conditions. The comparison is done for Toolan and Fox, Stevens and Alm and Hamre

methods, 35 piles were analysed in the original study, but only six of them were discussed in detail.

In general, Stevens best estimate method predicts lower number of blows in the first 10-15 meters, while CPT based Alm and Hamre gives quite a good fit, on condition that CPT profile is reliable.

However, both methods show poor prediction in chalk where it looks as if piles penetrating these layers encountered very low resistance from the surrounding soil.

From the study observed above, it is recommended that correlating soil resistance in chalk directly to CPT measurements should be taken with extreme caution. Further work is required in order to refine calculation procedures to predict the behaviour of piles in chalk layers.

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