THE INFLUENCE OF FRICTION FATIGUE ON PILE DRIVABILITY PREDICTIONS FOR LARGE DIAMETER MONOPILES IN THE BOLDERS BANK FORMATION

Davies S.R. Xodus, London, UK, simon.davies@xodusgroup.com
Giuliani G. Ramboll, London, UK, giorgio.giuliani@ramboll.co.uk
Vaziri M. Ramboll, London, UK, mohsen.vaziri@ramboll.co.uk

ABSTRACT

Reliable drivability studies enable appropriate hammer selection, pile geometry optimization and safe, cost-effective pile installation. Friction Fatigue is a general term describing temporary changes in soil resistance around the pile shaft during pile installation. Friction Fatigue is affected by factors such as soil type, geotechnical properties and initial stress state. Impact hammer driving records for large diameter monopile foundations at a Southern North Sea offshore wind farm provide the link between dynamic soil resistance during driving and static soil resistance. The objective of this paper is to evaluate the influence of friction fatigue parameters for the glacial deposits of the Bolders Bank Formation which, in this area, comprised layered sequences of stiff, high strength, over-consolidated CLAY and dense to very dense SAND. The assessment utilizes and expands on the Alm and Hamre (2001) friction fatigue model in conjunction with the drivability software WEAP (2010). The results indicate that friction fatigue is a required component for drivability assessments in these soils and calibration can enable refinement of drivability predictions. Friction fatigue parameters can be modified to develop a drivability envelope for predictions. Different soil behaviour types defined through cone penetration testing, may also require different friction fatigue models to improve drivability predictions.

Keywords: Bolders Bank Formation, pile drivability, friction fatigue, monopile

INTRODUCTION

The successful installation of large diameter driven piles is predicated upon reliable drivability predictions and therefore fundamentally on the drivability ground models. A refined understanding of the site-specific soil resistance to driving (SRD) enables the development of a robust and cost-effective installation strategy, which balances hammer selection, drilling requirements, and driving induced fatigue etc.

This paper investigates friction fatigue effects observed within glacial deposits from the Bolders Bank Formation for an offshore wind farm site located off the Yorkshire coast in the Southern North Sea. Actual driving records are compared with pile drivability predictions based on the Alm and Hamre (2001) cone penetration test (CPT) model in conjunction with friction fatigue model embedded into the drivability analysis program WEAP (2010).

The Alm and Hamre (2001) method is well established within the industry and utilises measured CPT parameters to directly derive the initial and residual SRD, rather than interpreted geotechnical parameters. The method also incorporates a friction fatigue model, which was evaluated to enable implementation into the WEAP (2010) friction fatigue model. The key friction fatigue parameters were varied, and predicted blow counts compared to driving records.

A comprehensive ground investigation was performed at the site, which included seventeen boreholes with adjacent seabed CPTs, fifty-seven CPTs, in-situ testing and laboratory testing covering all seventy-three locations across the site. This provides a robust basis for back-analysis of the driving records and therefore the soil's response to driving.

SITE GEOLOGY

The site is located in the Southern North Sea, approximately 8km off the coast of Yorkshire. The water depths at the site range from 12 to 17m. The geology at the site comprised a veneer of recent sediments over glacial deposits of the Bolders Bank Formation overlying chalk of the Rowe Chalk Formation. As summarised in Table 1, the Bolders Bank Formation comprised three main geological units: the Upper Unit, the Lower Unit and the Channel Unit, which comprised material reworked from both the Rowe Chalk Formation and the Bolders Bank Formation itself. A comprehensive summary of the geotechnical properties for the units comprising the Bolders Bank Formation is presented in Giuliani *et al.* (2017).

Table 1. General Site Geology

Formation	Unit	Description
Bolders Bank Formation	Upper	Layered sequences of stiff, high strength, overconsolidated silty CLAY and medium dense to very dense SAND.
	Lower	Stiff to very stiff, high strength silty CLAY.
	Channel	Dense to very dense SAND and GRAVEL and stiff to very hard, high strength CLAY and SILT.
Rowe Chalk Formation	Grade D	Structureless chalk grades Dm and Dc.
	Grade A to C	Weak, low to medium density chalk with marls seams and rare flints.

The generalized tectono-stratigraphic setting at the site and how it evolved through time is presented in Fig. 1, which also illustrates the complex relation between the Bolders Bank Formation units and the underlying Rowe Chalk Formation at the site.

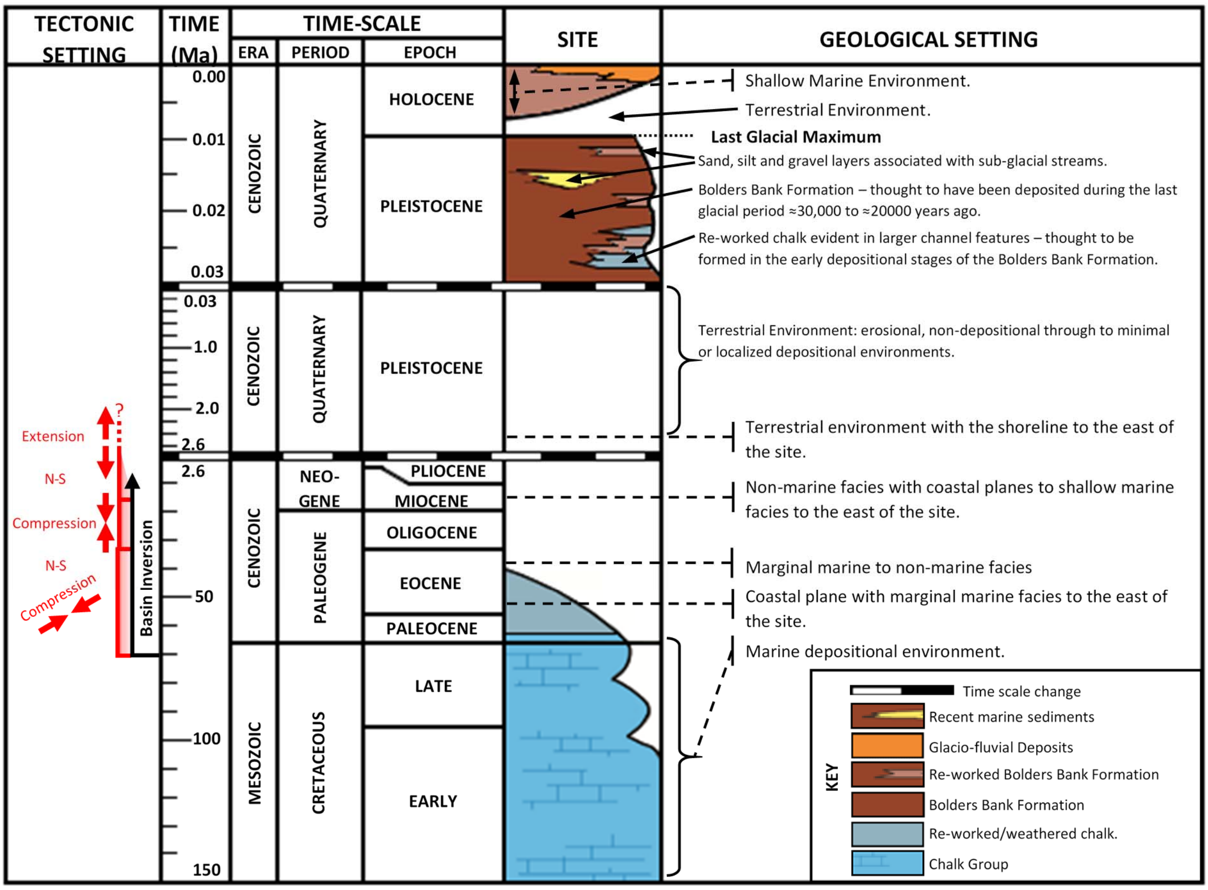

Fig. 1. Generalised Regional Tectono-Stratigraphic Development, adapted after Cameron *et al.* (1992)

FRICTION FATIGUE CONCEPT

It is widely reported in literature that many fine-grained soil types exhibit strength and stiffness degradation during shear disturbance. Pile driving through such soils can remould the soil proximal to the pile shaft, with residual strength and stiffness being achieved at large strains. This is generally a temporary state but is, however, significant for drivability predictions.

The site driving records indicate that remoulding occurs in both the Bolders Bank Formation and the underlying Rowe Chalk Formation. This paper investigates the friction fatigue parameters for the Bolders Bank Formation using the widely available drivability analysis software WEAP (2010), to enable refinement of drivability predictions in similar soils.

The concept of friction fatigue is that as the pile passes through the soil layer, the soil become progressively sheared and remoulded until it reaches its residual strength. As implicit in the name, friction fatigue therefore applies only to shaft friction resistance and not to the end-bearing resistance. This effect has been observed in a number of pile installation cases (e.g. Randolph *et al* (1994), Liu *et al* (2015)).

Moghaddam *et al* (2017), note that the degree of degradation is typically related to the distance, p, between the soil layer depth, d_s, and the pile tip penetration depth, d_p, over a maximum distance, $d_{p.max}$, termed the limit length, L_{li}, at which point the soil is fully degraded. This is illustrated by Equation 1 and Equation 2:

$$p = d_s - d_p \qquad [1]$$

$$L_{li} = d_s - d_{p.max} \qquad [2]$$

The degradation from fully intact (where the pile unit shaft friction, τ, is equal to the initial pile side friction, τ_i, at $p = 0m$), to the fully residual pile unit shaft friction, τ_{res} (where $\tau = \tau_{res}$ at $p = L_{li}$), is typically non-linear over p, and will vary with factors such as load cycles, stress state and soil type, (e.g. Alm and Hamre (2001), White and Lehane (2004)). For CPT data, Alm and Hamre (2001) described this non-linearity using the exponential degradation shape factor, k, and noted that for their data set in both fine-grained and coarse-grained soils:

$$k = {((q_t/p_0')^{0.5}}\Big/{C_k} \qquad [3]$$

Where q_t is the normalized cone tip resistance, p_0' is the effective overburden pressure, and C_k is a model constant which, for Alm and Hamre's (2001) database, $C_k = 80$. The degradation shape described by Equation (1) is illustrated in Fig. 3, using $C_k = 80$ for four soil types which were originally presented in Alm and Hamre (2001).

Figure 3 shows the degradation shape from fully intact, 100% unit shaft resistance, i.e. $\tau = \tau_i$ at $p = 0m$ to fully degraded, 0% unit shaft resistance, i.e. $\tau = \tau_{res}$ at $p = L_{li}$. For the purpose of this study, τ_{res} and L_{li} are taken at τ/τ_i= 0.1%, rather than 0% due to the exponential nature of the curve.

Figure 3 illustrates that for a dense sand, degradation occurs rapidly as pile penetration increases with L_{li} reached at ≈28m penetration. For overconsolidated clay, degradation occurs more slowly than for dense sand and loose sand. For normally consolidated clay degradation occurs much more slowly than for overconsolidated clay, with full degradation reached at L_{li} ≈225m. This means that an overconsolidated clay will achieve its residual unit shaft resistance at smaller pile penetration distance, p, than a normally consolidated clay.

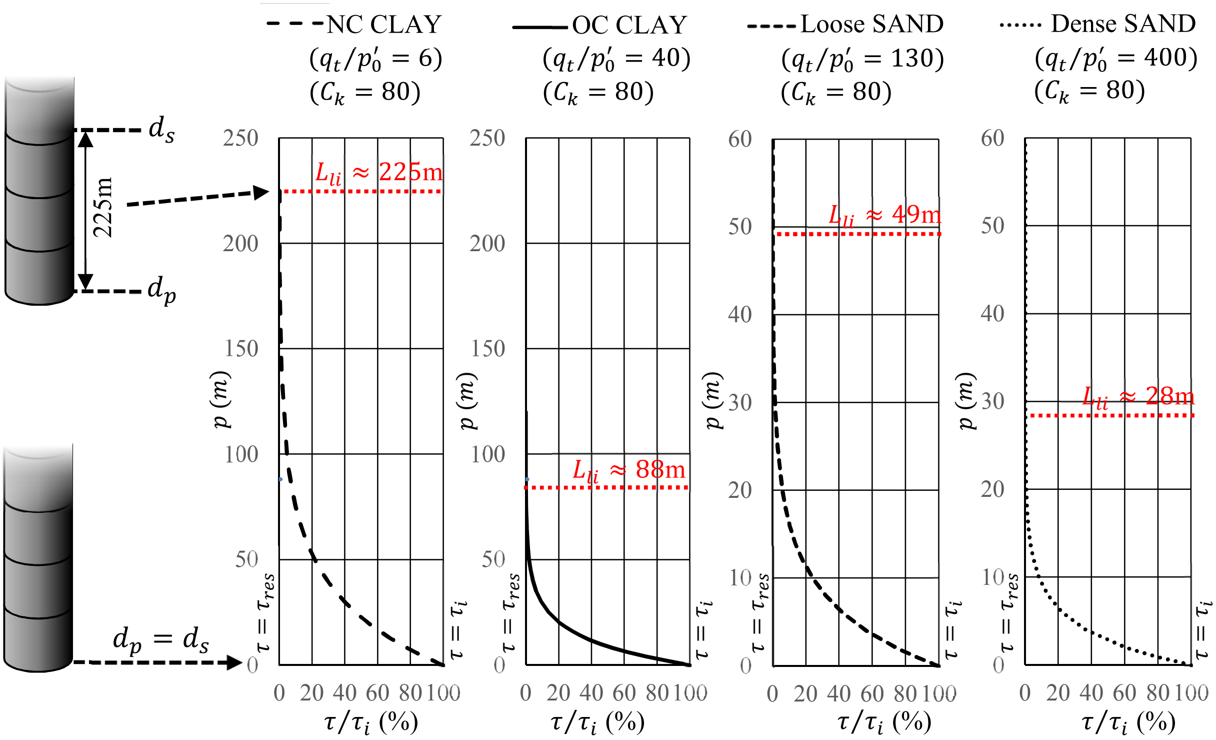

Fig. 3. Shaft Friction Fatigue Degradation Shape for Different Soils (after Alm and Hamre, 2001)

The key friction fatigue parameters in WEAP (2010) are τ_{res}, L_{li}, f_o (the exponential decay shape factor) and fL (the initial d_p below d_s before the onset of friction fatigue - taken as a fraction of the L_{li}). The friction fatigue parameters L_{li} and f_o are inherently related through the Alm and Hamre (2001) degradation shape factor, k. In accordance with Alm and Hamre (2001), it is assumed that friction fatigue occurs as soon as the pile penetrates below the soil layer and therefore the WEAP parameter fL is taken as zero.

BACK-ANALYSIS METHODOLOGY

General Approach

The WEAP (2010) friction fatigue parameters L_{li} and f_o are modified and the resulting blow count predictions compared with the blow count data from actual pile driving records. τ_i and τ_{res} are derived using the Alm and Hamre (2001) methodology and therefore the study focusses on the manner in which the soil degrades from τ_i to τ_{res} through Alm and Hamre (2001) degradation shape factor, k. For the purpose of this study, the unit shaft resistance, τ, is, taken as per Alm and Hamre (2001), using Equation 4:

$$\tau = \tau_{res} + (\tau_i - \tau_{res}) \cdot e^{k \cdot (d-p)} \qquad [4]$$

Alm and Hamre (2001) note that for fine-grained soils, τ_i can be taken as the recorded CPT sleeve friction, f_s. From Equation 4, it therefore follows that, where the pile has not penetrated below the soil layer depth, $d-p$ is equal to zero, $e^{k \cdot (d-p)}$ is therefore unity, so τ will be equal to τ_i, and τ_i will be equal to f_s.

For coarse-grained soils, Alm and Hamre (2001) calculate τ_i using the following equation:

$$\tau_i = \left(0.0132 \cdot q_t \cdot \left({p'_0}/{p_a}\right)^{0.13}\right) \cdot \tan \varphi_{cv} \qquad [5]$$

Where p_a is the reference pressure, taken as 100kN/m^2, and φ_{cv} is the constant volume friction angle, which is calculated through CPT correlations with relative density, D_r, (ISO, 2007: p. 483) and angle of internal friction, φ', (Lunne *et al*, 1997: p. 90), using Equation 6:

$$\varphi_{cv} = \left[34.5 + 0.1\left({}^{1}\!/_{2.93} \cdot ln\left({}^{q_c}\!/_{205\left(p_m'\right)^{0.51}}\right)\right)\right] - 5 \quad [6]$$

The residual unit shaft resistance, τ_{res}, is, for the purpose of this study, taken as per Alm and Hamre (2001), using Equation 7 for fine-grained soils, and Equation 8 for coarse-grained soils:

$$\tau_{res} = 0.004 \cdot q_t\ (1 - 0.0025 \cdot q_t/p_0') \text{ for fine-grained soils} \quad [7]$$

$$\tau_{res} = 0.2 \cdot \tau_i \text{ for coarse-grained soils} \quad [8]$$

The unit tip resistance for fine-grained and coarse-grained soils is also determined using Alm and Hamre (2001) as defined by Equation 9 for fine-grained soils and Equation 10 for coarse-grained soils:

$$q = 0.6 \cdot q_t \text{ for fine-grained soils} \quad [9]$$

$$q = 0.15 \cdot q_t \cdot (q_t/p_0')^{0.2} \text{ for coarse-grained soils} \quad [10]$$

Using equations 4 to 10 mean that the unit shaft resistance is directly linked to the CPT data, although some site-specific calibration of parameters such as D_r, φ' and φ_{cv} may still be required for other sites.

The location specific CPT data is used to derive soil behaviour types (SBT) for the ground model using Robertson (2010). The SBTs are then grouped into fine-grained soils (SBT zones 1,2,3,4 and 9) and coarse-grained soils (SBT zones 5,6,7 and 8) in order to apply the methodologies previously described. It is acknowledged that SBTs from CPT are not directly related to a textural-based soil type classification (e.g. Robertson, 2016).

The WEAP (2010) friction fatigue parameters f_o and L_{li} were initially derived through k and the relationship between q_t/p_0' and L_{li}, assuming that τ_{res} is achieved at 0.1% τ/τ_i. On this basis, L_{li} can be defined by Equation 11:

$$L_{li} \approx C_L \cdot (q_t/p_0')^{-0.5} \quad [11]$$

Where C_L is a model constant which, for Alm and Hamre's (2001) database, $C_L = 560$, as illustrated in Fig. 4, which shows the L_{li} for four soil types presented in Alm and Hamre (2001) and their relation to q_t/p_0' using Equation 3 with a C_k of 80.

Fig. 4 also presents the relationship between q_t/p_0' and L_{li}, for C_k values of 20, 60 and 70. For each C_k value, a corresponding C_L value was derived. The resulting relationship between C_k and C_L is presented in Fig. 5 and defined by Equation 12:

$$L_{li} \approx 7 \cdot C_k \quad [12]$$

This enables calibration of the friction fatigue parameters f_o and L_{li} from $C_k = 0, C_L = 0$ to $C_k = 80, C_L = 560$. Calibration beyond $C_k = 80, C_L = 560$ was not required for this study, although it is hypothesized that the C_k, C_L relationship could be extrapolated.

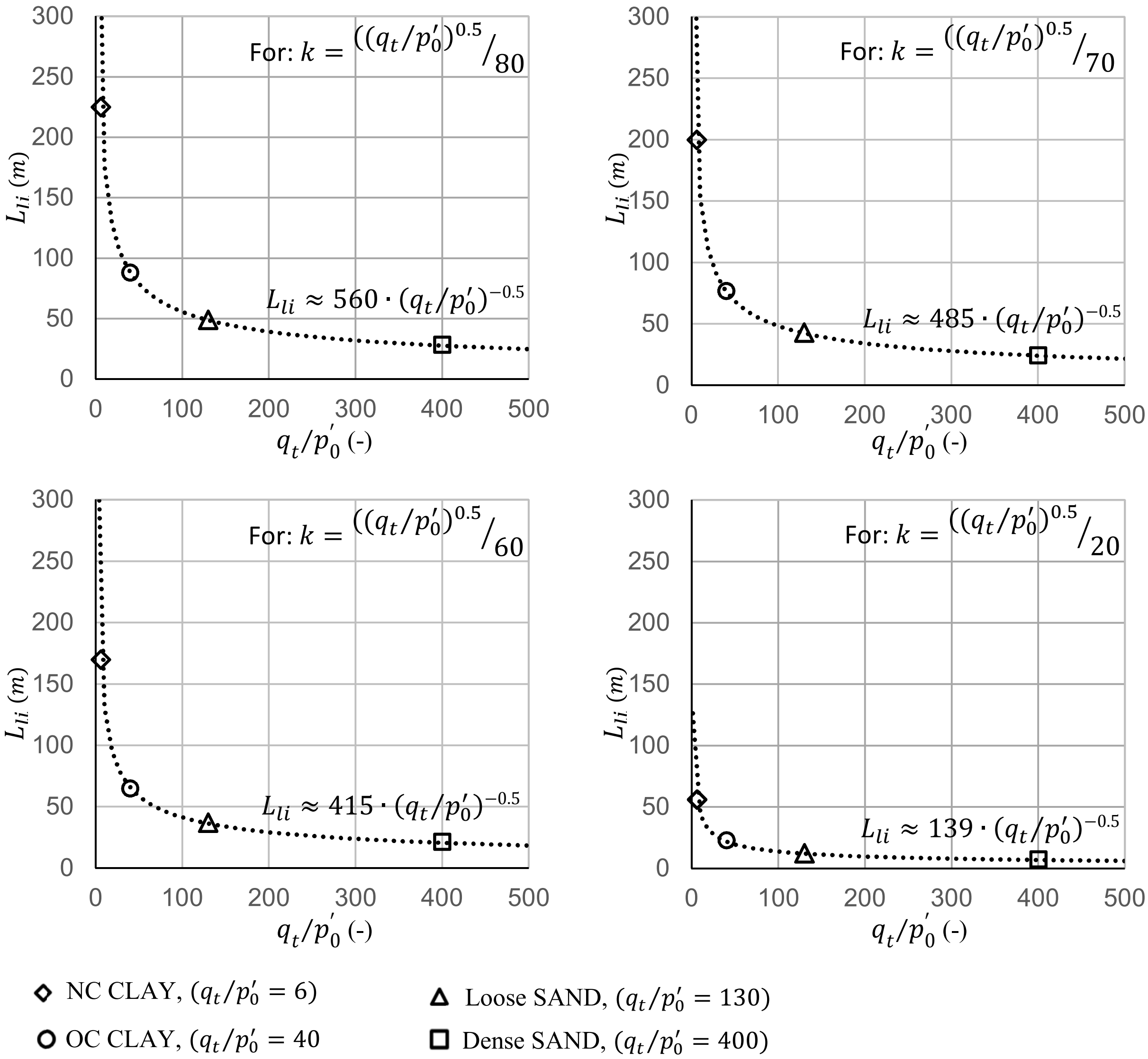

Fig. 4. Relationship between q_t/p'_0 and L_{li} for different values of C_k

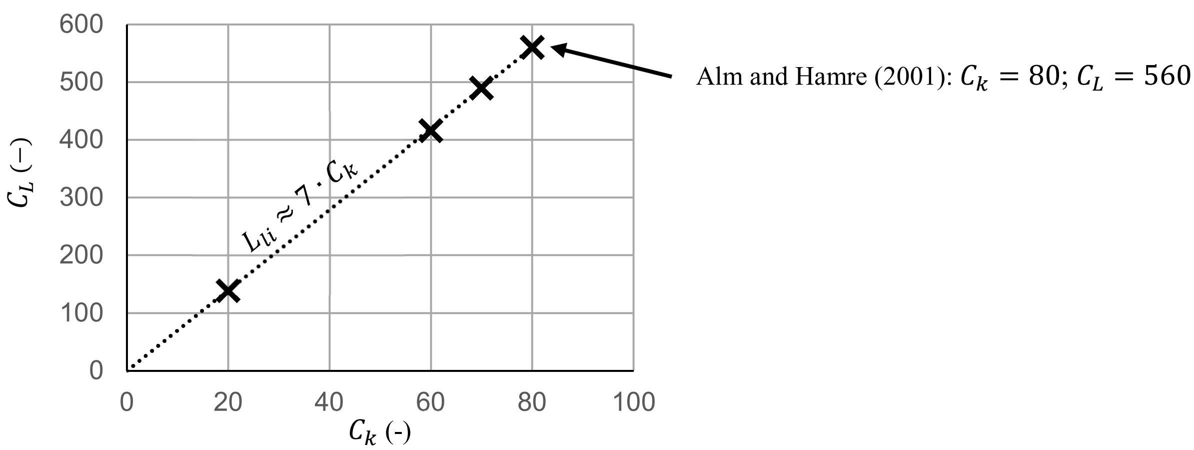

Fig. 5. Relationship between C_k and C_L

The quake and damping factors assumed for the back-analysis are summarised in Table 2 (WEAP, 2010).

Table 2. Assumed Quake and Damping Parameters

Parameter	Fine-grained Soil	Coarse-grained Soil
Shaft Damping (s/m)	0.65	0.16
Toe Damping (s/m)	0.50	0.50
Quake (mm)	2.50	2.50

The ground model was split into 0.4m layers, with average τ, q, τ_{res}, f_o and L_{li} values used over this interval. This enabled sufficient resolution to capture variation in soil layering and properties for both the CPT and pile driving data.

Drivability Database

The database consists of 4.74m diameter, open-ended steel monopiles with wall thicknesses ranging from 60mm to 80mm. Pile penetrations vary from 23m to 35m; with penetrations through the Bolders Bank Formation generally between 15m and 25m.

A total of seventy-three piles have been driven with a Menck MHU 1900-S hammer, which has the following properties: equivalent stroke, h_s: 2.1m; hammer efficiency: 95%; and helmet weight: 1275kN.

Records of Blow Count per penetration interval have been obtained for each pile. These are compared with predicted blow counts over the same penetration interval. As blow counts are a function of the hammer input energy, the hammer energy variation during driving has been modelled in WEAP (2010) through estimated hammer strokes using the following equation:

$$h_a = (E_{blow}/E_k) \cdot h \qquad [13]$$

The driving records typically provided: pile penetration per quarter meter interval; number of blows per quarter meter interval; average energy per blow; energy per quarter meter interval; and cumulative blow count. Twelve positions, which represented a range of ground conditions, were reviewed.

COMPARISON OF PREDICTED AND MEASURED BLOW COUNTS

A comparison of blow counts per 0.25m pile penetration for twelve locations is presented in Fig. 6. The figures present three cases: Case 1 - without friction fatigue; Case 2 - standard friction fatigue, $C_k = 80$; $C_L = 560$ (Alm and Hamre, 2001); and Case 3 - modified friction fatigue, $C_k = 20$; $C_L = 140$.

Case 1, the Alm and Hamre (2001) method without friction fatigue over-predicts soil resistance to driving in all but four locations. In these four locations, Case 1 exhibits a closer fit to the driving record, although the magnitude of over-prediction increases with depth and also with some SBTs, such as Zone 7 and Zone 9. There is a wide variation in the magnitude of the over-prediction.

Case 2, the Alm and Hamre method with friction fatigue illustrates that there is generally a need for friction fatigue to be implemented. In four locations it provided a good fit with the driving record, although the pattern of over-prediction is as per Case 1.

Case 3, where the Alm and Hamre degradation shape factor is modified, indicates that a significant reduction in C_k is required to capture the lower range observed in the driving records. This suggests the soils remold relatively quickly with respect to those of the Alm and

Hamre (2001) database. This provides a reasonable fit with eight of the locations but under-predicts on the remaining four.

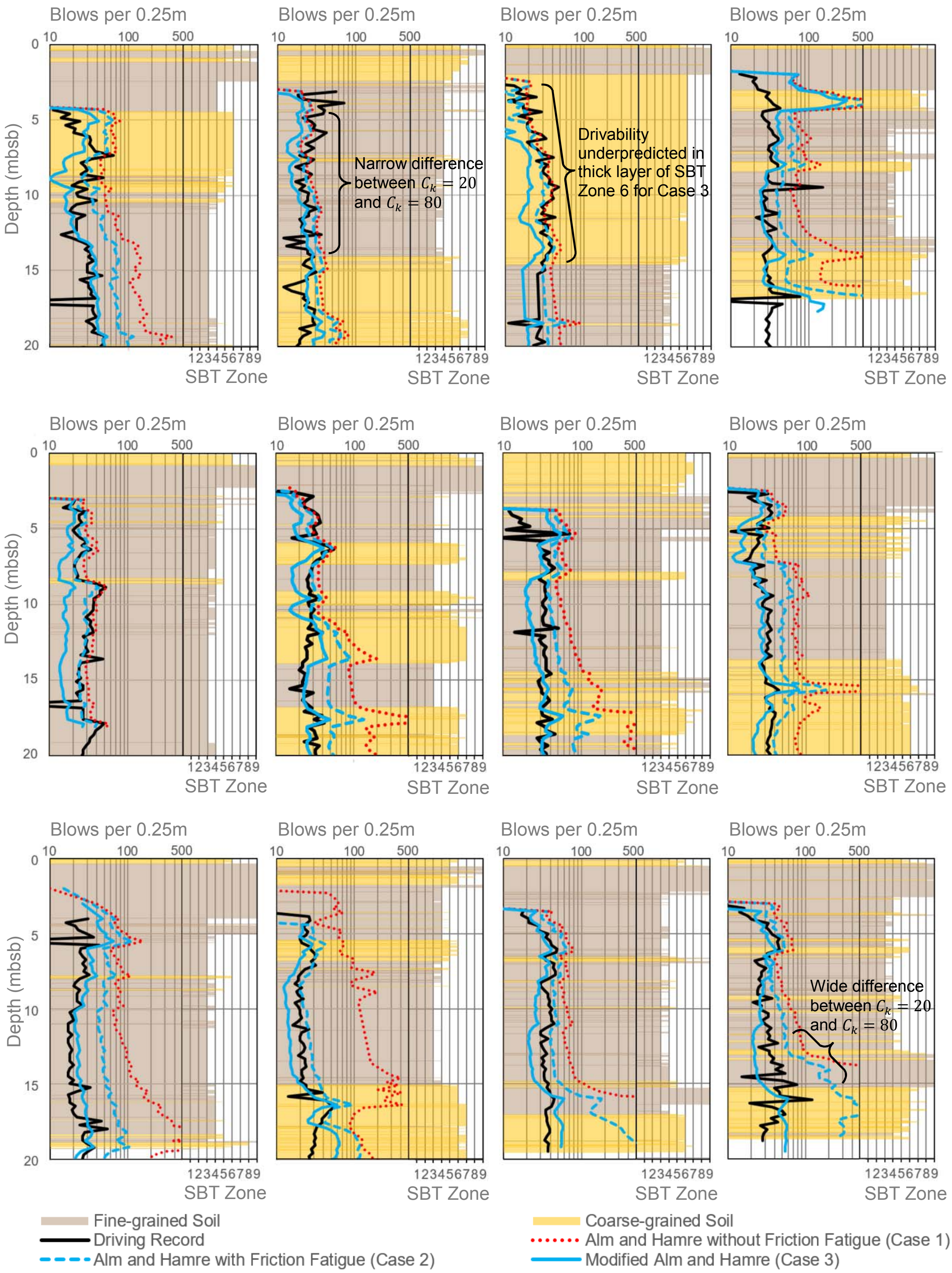

Fig. 6. Comparison of Actual Blow Counts with Predicted Blow Counts for Different Friction Fatigue Cases

It can be seen that a C_k range of 20 to 80 generally captures the envelope for the driving response observed at these locations. There is a notable variation in the difference between predictions using $C_k = 20$ and $C_k = 80$, with some locations exhibiting a wide difference and other locations exhibiting a narrow difference, as indicated in Fig. 6.

The variation observed reflects the fact that the glacial deposits at the site are highly variable soils. Different soil behavior types as derived from CPT may also need different k factors, and in some cases different k factors could be beneficial within an SBT class, for example SBT zones 3 and 4 variation exhibit variation in consolidation state. The soils proximal to the chalk may also have a high carbonate content and be weakly cemented. This is particularly evident for soils classified as channel deposits (Giuliani, 2017), which include reworked chalk and Bolders Bank soils. SBT zone 9, for example, exhibits a different driving response at shallow elevations than at depth, where it is proximal to the chalk. Generally, over or underpredictions in a particular layer lead to discrepancies in predictions for underlying layers.

It is demonstrated that a simple approach using Alm and Hamre with a single k factor for all SBTs can be used to develop a first estimation of the drivability envelope in these soils. However, generally the results suggest that different friction fatigue methods for different SBT zones could improve drivability predictions further i.e. a multiple degradation shape factor approach could be better than a single degradation shape factor approach.

Alm and Hamre (2001) recommend that the upper bound drivability is taken as 25% increase of the best estimate prediction. This would appear to be a reasonable strategy for the soils present at this site.

SUMMARY AND CONCLUSIONS

Refinements to the drivability analysis is the key to providing a cost-effective and successful pile installation. However, there are many factors which can affect pile drivability and therefore drivability predictions should be defined by lower and upper bound drivability envelopes to ensure that both the structural integrity, for example driving-induced fatigue, and the installability are adequately considered with a sufficient margin of safety.

The objective of this study was to investigate the friction fatigue effects in the Bolder Bank Formation. The Alm and Hamre (2001) friction fatigue concept, which uses measured CPT parameters to directly derive the initial and residual SRD, was evaluated and developed for implementation into the drivability software WEAP (2010). Twelve locations which represented the general range of ground conditions were assessed. The study concludes that friction fatigue is a key concept to improve the accuracy and reliability of pile drivability predictions. It enables assessment of a progressively changing SRD along the pile length during driving, which is more representative of actual driving conditions in these soils.

The use of the friction fatigue method embedded into WEAP (2010) enables rapid assessment of drivability without the need to undertake multiple analysis iterations. The Alm and Hamre (2001) friction fatigue method provided a good approximation, although significant modification of the shape factor was required to get a reasonable envelop of drivability at this site. Further work to more closely relate SBT from CPT to degradation factors is considered beneficial for the industry. For this site there are instances where different f_o and L_{li} models for different SBTs may reduce the magnitude of over and under-predictions of drivability. The use of measured CPT parameters rather than interpreted geotechnical parameters will improve consistency of predictions for soils with a particular CPT response; this would then be more applicable to other projects where similar soils are encountered.

This study only considers large diameter piles and a single hammer. Alm and Hamre (2001) considered slender jacket piles with approximate diameters ranging from 0.75m to 2.75m. A more comprehensive understanding of the potential implications of diameter effects and hammer energy would also be beneficial to the industry. The impact of load cycles may also influence friction fatigue and the effect of variation in hammer energy during driving is therefore an area for further assessment. The Alm and Hamre (2001) method appears to be suitable for initial drivability predictions, although the magnitude of the change from fully intact to fully remolded could merit further study.

ACKNOWLEDGEMENTS
The soil data and pile driving records used within this study have been provided by Eon and the authors would like to acknowledge permission to publish this data.

REFERENCES

Alm T and Hamre L (2001) Soil Model for Pile Driveability Predictions Based on CPT Interpretations. *Proceedings of the 15th International Conference on Soil mechanics and Foundation Engineering, Istanbul, Turkey*. A.A. Balkema, Rotterdam, the Netherlands, Vol. 2, pp. 1297–1302.

API (American Petroleum Institution) (2014) RP2A-WSD. Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Working Stress Design, Twenty-Second Edition. API, Washington, DC, USA.

Cameron, T.D.J., Crosby, A., Balson, P.S., Jeffery, D.H., Lott, G.K., Bulat, J., & Harrison, D.J., 1992. United Kingdom offshore regional report: the geology of the southern North Sea. London: HMSO for the British Geological Survey.

Giuliani *et al* (2017) Large Diameter Driven Piles in the Bolders Bank Formation, a Case Study. *Proceedings of the 8th International Conference on Offshore Site Investigation and Geotechnics: Smarter Solutions for Future Offshore Developments.* Society for the Underwater Technology, London, UK, Vol. 1, pp. 468–480.

Liu R *et al* (2015) Evolution of driving resistance during driven pile installation. *Frontiers in Offshore Geotechnics III, Oslo, Norway* (Meyer (ed)) Taylor & Francis Group, London, UK.
Lunne T *et al* (1997) *Cone Penetration Testing in Geotechnical Practice*. Taylor & Francis Group, London, UK.

Moghaddam RB *et al* (2017) Friction Fatigue and Drivability Analysis of Open-ended Pipe Piles Based on Cone Penetration Testing Results. *DFI Annual Conference, New Orleans, Proceedings of the 42nd Annual Conference on Deep Foundations LA, USA*. pp. 468–480.
Randolph *et al* (1994). Design of driven piles in Sand. *Geotechnique*, 44(3): 427-448.

Robertson PK (2010) Soil Behaviour Type from the CPT: An Update. *International Symposium on Cone Penetration Testing, Huntington Beach, CA, USA* Vol.2, pp. 575–583.

Stevens RS *et al* (1982) Evaluating Pile Driveability for Hard Clay, Very Dense Sand and Rock. *Offshore Technology Conference Houston, Texas USA*. pp. 465–481.

WEAP (2010). Wave Equation Analysis Program. Version 8. https://www.pile.com/products/grlweap/ (accessed 02/01/2020).