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Pile driving prediction for monopile foundations in London clay

Prédiction de battage de monopieux dans la formation Argile de Londres

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ABSTRACT: The current industry approach to pile driving predictions consists of developing a model of the hammer-pile-soil system which simulates the relationship between soil resistance to driving (SRD) and blow counts (or pile penetration per blow). The SRD methods traditionally used are broadly based on static pile capacity calculations. The SRD is used in combination with the one-dimensional wave equation model to indicate the anticipated blow counts with depth for specific hammer energy settings. This approach has predominantly been calibrated on relatively long slender piles used in the oil and gas industry but is now being extended to allow calculations to be undertaken for relatively short rigid large diameter monopile foundations.

This paper evaluates the accuracy of current industry practice when applied to a site where large diameter monopiles were installed in predominantly stiff fissured clay. The fissures in the clay were found to influence the soil sensitivity of the material and hence remoulded undrained shear strength values were reported in excess of the initial undisturbed undrained shear strength. Subsequently, through comparison of commonly used SRD calculation methods, it was found that soil sensitivity can have a significant affect on the magnitude of SRD calculated. This paper concludes that CPT based calculation methods are relatively accurate in predicting SRD in these soils and also tentatively proposes an alternative method for SRD calculation which accounts for soil sensitivity when CPT data is not available. Actual geotechnical and pile installation data, including pile driving records and signal matching analysis (based upon pile driving monitoring techniques), were considered for the assessment on the case study site.

RÉSUMÉ: Les méthodes courantes de prédiction de battage de pieux sont basées sur un modèle d’interaction marteau-pieu-sol qui simule le rapport entre la résistance du sol au battage (SRD : soil resistance to driving) et le nombre de coups (ou pénétration du pieu par coup). Les méthodes traditionnelles de prédiction de la SRD sont basées sur l’estimation de la capacité portante statique du pieu. La SRD est combinée avec la théorie de propagation d’ondes unidimensionnelles afin d’estimer le nombre de coups à une profondeur donnée pour une énergie de marteau spécifique. Cette méthode a été calibrée notamment sur des pieux longs et élancés, typiquement utilisés dans l’industrie du pétrole ; néanmoins, la méthode est en train d’être adaptée pour des pieux de grand diamètre (monopieux), généralement plus courts et rigides.

Ce document a pour objectif d’évaluer la précision de la méthode traditionnelle lorsqu’elle est appliquée à l’étude d’installation de pieux de grand diamètre dans des argiles fermes et fissurées. Il a été détecté que les fissures dans l’argile ont une influence dans la sensibilité du sol, car les valeurs de cohésion non-drainée sur sol remanié sont supérieures aux valeurs initiales de cohésion non-drainée sur sol intact. De ce fait, il a été trouvé que la sensibilité du sol a un effet non-négligeable sur la magnitude de la SRD calculée. Cet article conclut que les méthodes de calcul basées sur le CPT sont relativement précises pour prédire la SRD dans ces sols et propose provisoirement une autre méthode de calcul du SRD qui prend en compte la sensibilité du sol lorsque les données CPT ne sont pas disponibles. L'étude a été réalisée avec des données géotechniques et d'installation factuelles, dont des enregistrements pendant le battage et des analyses de calage de signaux (basées sur des techniques de suivi de battage par de capteurs installés sur les pieux).

Keywords: driven piles; fissured clay; London clay; monopiles; offshore foundations.

1 INTRODUCTION

Pile driveability predictions and their accuracy are critical to the installation contractor when selecting an appropriate hammer to successfully advance the piled foundation to its required depth. It is therefore essential to undertake regular back-calculations to ensure that the theoretical and empirical methods used for predictions are appropriate and allow accurate forecasts to be made. Accurate predictions allow more timely and cost-effective installation works.

To allow for accurate predictions of pile driving there are various aspects which must be modeled accurately. Correct assumptions must be made regarding the hammer and pile configuration including pile geometry and hammer energy settings. Albeit, these assumptions become secondary if the interpreted soil response is not representative of the in-situ conditions. Therefore, it is critically important that an appropriate Soil Resistance to Driving (SRD) method is selected and the soil properties are interpreted accurately.

To evaluate the accuracy of the current industry practice, back-calculations have been undertaken on a case study site. The site that is discussed in this paper is located in British waters on the south side of the Thames Estuary. The project comprised the installation of multiple monopile foundations for offshore wind turbines generators in predominantly stiff fissured London clay.

2 PILE DRIVEABILITY ASSESSMENT PROCEDURES

Pile driving is normally modelled in two distinct stages, these are as follows:

1) Derivation of the SRD of the encountered ground conditions for a particular pile geometry, normally using best estimate and high estimate soil parameters to derive soil profiles for pile driving prediction purposes.
2) Modelling of the dynamic behavior of the hammer-pile-soil system using a wave equation analysis to estimate the blow count variation with penetration depth, steel tension and compression stresses and pile refusal depth.

Back analyses of the pile driving data from pile driving records or pile driving monitoring (PDM) are also carried out regularly to assess the accuracy of the design and installation feasibility of a piled foundation and the parameters used within these assessments. The back analyses presented in this paper are those carried out using pile driving records; however, back analyses undertaken using PDM data were also considered.

To give accurate predictions, quantification of the SRD, the impact energy generated by the hammer and the dynamic response of the soil (damping and quake parameters), among others, must be modelled precisely.

3 SOIL RESISTANCE TO DRIVING

There are no codified methods for estimating the SRD; however, most of the better-known SRD methods Alm & Hamre (2001); Stevens et al. (1982) are broadly based on modifications to static pile capacity calculations.

The calibration and verification of these SRD methods is 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.

Therefore, a driveability analysis using one of these SRD methods should use the corresponding wave equation parameter sets.

3.1 Alm and Hamre (2001)

Alm & Hamre (1998) presented a SRD prediction model using Cone Penetration Test (CPT) data directly and included a friction degradation concept.

The model originates from the idea that the individual interpretation of undrained shear strength profiles was a significant source of variability in the predictions. Therefore, it was proposed to directly correlate SRD with measured values of CPT tests. When using this approach, it was found that the variability in interpretation of input profiles significantly reduced.

The effect of friction degradation is associated with the continuous displacement of the pile during driving, this phenomenon was systematically observed during pile driving in certain areas. The nature of this mechanism is such that the unit shaft friction decreases for an increasing length of pile passing through a given soil horizon.

The model was established based on the back-analysis of driving data in North Sea soils. Since the first model developed by Alm and Hamre (1998) overpredicted the resistance of deep normally consolidated clays, an enhanced method was developed (Alm & Hamre, 2001).

The database used for the calibration comprises foundation piles with outer diameters between 1828.8mm and 2743.2mm, with the exception of a long 762mm conductor pile. The penetration below the mudline covers a large range; from 35m to 115m.

The back-analysis was calibrated with a single set of dynamic soil parameters. Quake values were taken as 2.5mm for both shaft and tip. The tip damping was taken as 0.5s/m and the shaft damping was 0.25s/m for all case studies.

3.2 Stevens et al. (1982)

This SRD method was originally developed on very dense sand and hard clayey soils in the Persian Gulf. The case studies of foundation piles of 914.4mm and 1066.8mm outer diameter with penetrations of 33m to 50m below mudline were used to calibrate the method.

The unit skin friction during continuous driving in cohesive soils is calculated using a stress history approach that was originally developed by Semple and Gemeinhardt (1981). This method essentially applies an additional multiplication factor to the API RP 2A (2000) skin friction calculation and uses the same assumptions as the API RP 2A (2000) calculation for all other formulae.

The dynamic soil parameters used in this method are those proposed by Roussel (1979). The shaft and tip quake are both assumed to be 2.54mm. With regards to the shaft damping, this is proposed to range from 0.1s/m to 0.36s/m for hard to soft clay respectively. The tip damping is proposed to be 0.5s/m for all soils.

4 DYNAMIC BEHAVIOUR OF THE HAMMER PILE-SOIL-SYSTEM

To allow the modelling of the dynamic behavior of the hammer-pile-soil system, the one-dimensional wave equation, proposed by Smith (1960), is used; this is implemented in commercial software (i.e. GRL WEAP).

In the one-dimensional model, the hammer and pile are modelled as a series of discrete masses and springs. The resistance of the soil at the pile toe is modelled by a point force on the last pile node and the resistance of the soil along the pile shaft is represented by a set of point resistances at each pile node. The magnitudes of these resistances are a function of the elastic displacement (i.e. the quake, Q) and of the damping constant, J.

The velocity of the hammer and hence impact of this mass produces a displacement in the discrete masses. The displacement of the subsequent sections leads to a compression or to an extension in the springs between them. The force generated by the two springs produces a force on the weight which in turn results in a reduction of the wave velocity. The transferred force and velocity then becomes an input for the next step. This procedure is ongoing until the velocity reduces to zero.

The model proposed by Smith (1960) is commonly accepted in industry, however, it presents a number of limitations. The main limitations of this model are as follows:

1) The dynamic soil parameters employed in the model are not directly associated with soil properties and hence measured values from common geotechnical investigation methods are not used to define these (i.e. quake and damping parameters).
2) Radiation damping (the radial loss of energy) is not currently accounted for since only one-dimensional effects are considered. Research undertaken by Meynard and Corte (1984) and Simons and Randolph (1985) discusses alterations that could be made to the shaft and tip damping parameters in the one-dimensional model in an attempt to account for this; however, this is not implemented in common practice.
3) In static conditions, inside friction causes plugging and outer friction increases close to the pile tip; these effects are mainly due to end bearing pressure. In the one-dimensional model, the inside friction, the outer friction and the end bearing pressures all remain uncoupled.

Due to these factors not being accurately modelled in this approach, which was developed using the computational resources available in the 1960s, there is a clear necessity for a much-improved method calibrated on both relatively short rigid piles and relatively long and slender piles. It is anticipated that there will be many three-dimensional effects which will be more emphasized for larger diameter monopile foundations when compared to the relatively long slender piles in which the current approach was calibrated for.

5 BACK ANALYSIS OF PILE DRIVING RECORDS AND MONITORING

There are many intricacies of pile driving which are not wholly accounted for in the current one-dimensional approach to pile driveability predictions. Subsequently, the authors consider that it is of crucial importance that back-calculations of pile driving records and monitoring are carried out, at least at locations considered representative of certain ground conditions.

To give an accurate back-calculation of predicted blow count profiles when compared to actual pile driving installation records, a number of factors must be precisely modelled. These factors are the assumed energy delivered by the hammer in to the pile, soil properties, dynamic parameters adopted, hammer efficiency, pile installation or down time effects (Colliat et al., 1993).

The back-calculations presented in this paper use two methods of back-calculation; adapting the stroke height of the hammer to accurately model the variation of energy exerted through the pile, and back-calculating the measured SRD through bearing graphs for different energy levels.

6 CASE STUDY EXAMPLE

6.1 Pile geometry and hydraulic hammer

At the site under consideration, the monopiles were installed using a self-elevating jack-up platform and were installed into the seabed using a double acting hydraulic hammer. The details of the pile geometry and employed hydraulic hammer are exhibited in Table 1.

The monopiles have an outside diameter of 5.0m at the pile toe with a conical section tapering to 4.5m at the pile head. Embedment below seabed level ranged between 40 and 45.5m.

Table 1. Pile and hammer data

Parameter	Value
Pile length	Variable
Pile diameter	5.0m
Wall thickness (pile toe)	46mm
Toe area (open ended)	$0.72m^2$
Specific weight	$77kN/m^3$
Elastic modulus	210GPa
Pile steel grade	S355
Hammer type	IHC S-1200
Rated energy	1200kJ
Equivalent rated stroke	2.02m
Ram weight	600kN
Hammer weight (in air)	1450kN
Anvil weight	625kN
Hammer + sleeve connection ring weight	1025kN

6.2 Ground conditions

The geology across the site comprises a variable thickness of localised surficial sands overlying alluvial sediments. The alluvial sediments overlie the London Clay formation. There are several in-filled paleochannels where the alluvial sediments can extend up to 20m in depth. However, out with these channels, the London Clay is present at relatively shallow depths of less than 2m below seabed level.

The surficial sands are very loose to loose and are present across most of the site; reaching up to 4m in thickness in some areas.

The paleochannel infill comprises a significant amount of heterogeneous soil of variable thickness comprising very soft to soft clays, silty sands, and soft to firm clays. Gravelly sands occasionally becoming sandy gravel with occasional thin layers of cohesive material were also locally identified in the base and flanks of the in-filled channels.

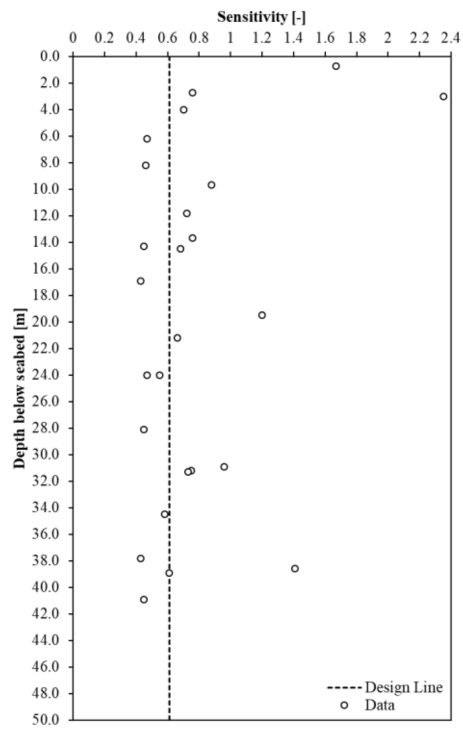

Figure 2. Clay sensitivity distribution with depth

The main formation which makes up most of the wind farm geology is the London Clay formation. This clay is of high to very high plasticity with closely to very closely spaced fissures which vary in orientation. The fissures effect the "sensitivity" (i.e. ratio between the strength of the soil in an undisturbed state and the strength of the soil in a remoulded state at the same moisture content) of the material and hence remoulded strength values were reported (through triaxial testing) in excess of the initial undisturbed undrained shear strength. During the remoulding process, the fissures and pre-defined failure planes are destroyed and removed, and hence the soil becomes more homogeneous with no preferential failure planes. Figure 2 displays the distribution of clay sensitivity with depth across the wind farm site and a Design Line of the mean value.

6.3 Geotechnical Input Parameters

Table 2 below displays an indication of the generic geotechnical parameters derived for the back analysis.

Table 2. Geotechnical input parameters

Unit	Description	Undrained Shear Strength [kPa]	Effective Angle of Friction [°]
I	Surficial SAND	-	28
II	Very soft CLAY	<20	-
	Silty SAND	-	30
	Soft to firm CLAY	30 to 50	-
	Gravelly SAND / Sandy GRAVEL	-	30 to 33
III	Firm to hard London CLAY	50 to 200	-

These parameters give an indication of the unit variation across the site and were derived from cone penetration test (CPT) and laboratory test data.

The available geotechnical data was reviewed and CPT cone factor, Nkt values were derived for the soft alluvial clays and the London clay. The Nkt values were higher than those generally used, however, it was recognised that these soils were of high plasticity and heavily fissured. An assessment of CPT and laboratory testing data resulted in the adoption of an N_{kt} of 32.5 being selected for the London clay.

Figure 3 displays the the results of an assessment of SRD using the methods set out in Section 3 and back calculated values from measured pile driving records (as described in Section 5).

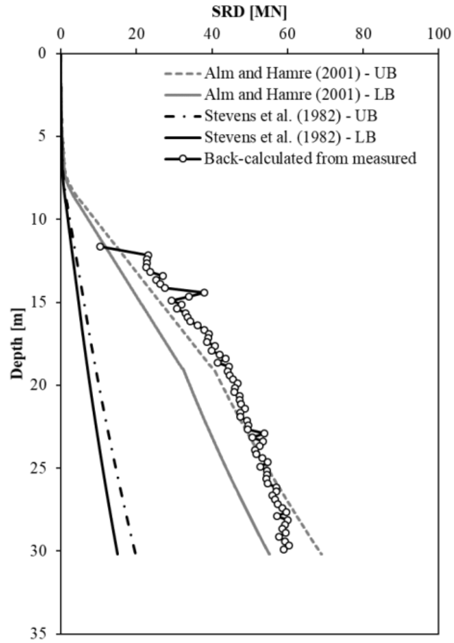

Figure 3. Back-calculations at location X5

As demonstrated in Figure 3, in the case of location X5, the difference in SRD was found to be ~45MN between the lower bound Stevens et al. (1982) estimate and that back-calculated from the measured data – discrepancies of this magnitude were found to be the case on most of the locations where the fissured clay was present in the majority of the soil profile. The Alm and Hamre (2001) estimate at this location gives a much-improved prediction but still underestimates overall. The significant underestimation from the Stevens et al. (1982) prediction is alarming as this could have led to multiple piles across the site refusing prematurely resulting in a significant amount of time and economic loss.

The reasoning behind the significant underestimation when using the Stevens et al. (1982) method was deemed to most probably be due to the method not accounting for the sensitivity of the clays and hence increase in undrained shear strength as the fissures within the soil are destroyed and removed through remoulding of the soil during driving.

The Stevens et al. (1982) method essentially applies a pile capacity factor, F_p to the API RP 2A (2000) static capacity calculation in clays. Therefore, one of the main assumptions is the use of an alpha factor, α which is capped at unity. Subsequently, in the case of the site presented, this may not capture the soils response to driving since the soil sensitivity suggests the soil may increase in strength as the soil is remoulded. This appears to have been somewhat captured by the Alm and Hamre (2001) approach through the use of raw CPT parameters.

When applying an assumption that the α value is substituted for the inverse of soil sensitivity, S_t, similar to those calculations used for suction anchor installation in clay DNV-RP-E03 (2005), it was found that the Stevens et al. (1982) approach gave a much closer match with the measured driving data. This method was assessed for more than a dozen locations across the site where the fissured London clay was present. The design line shown in Figure 2 was used as the input of S_t for each analysis.

The resulting alteration to the original Stevens et al. (1982) formula for unit skin friction calculation in clay is displayed below (1).

$$f_s = \frac{1}{S_t} \cdot S_u \cdot \mathrm{F}_p \tag{1}$$

Where, f_s is unit shaft friction, S_t is soil sensitivity, S_u is undrained shear strength and F_p is the pile capacity factor.

Figure 4 and Figure 5 exhibit example back-calculations where the inverse of soil sensitivity was applied as an equivalent alpha factor. As can be observed, this gives a relatively good match with the upper bound estimate for both cases and hence suggests that this is a factor which should be considered when carrying out further pile driving predictions in fissured clay where the sensitivity of the soil is below unity.

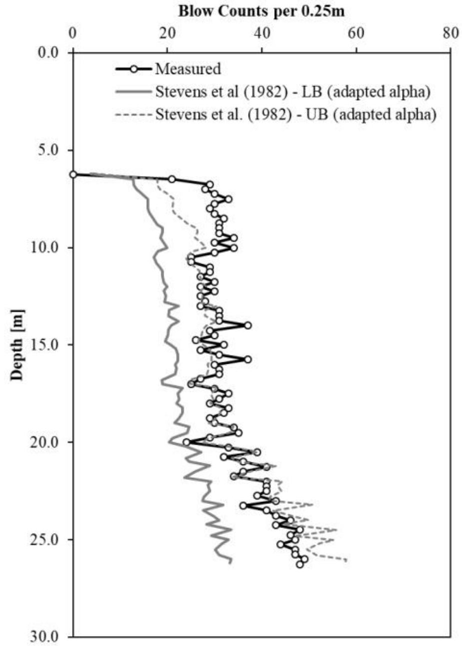

Figure 4. Back-calculations at location X2

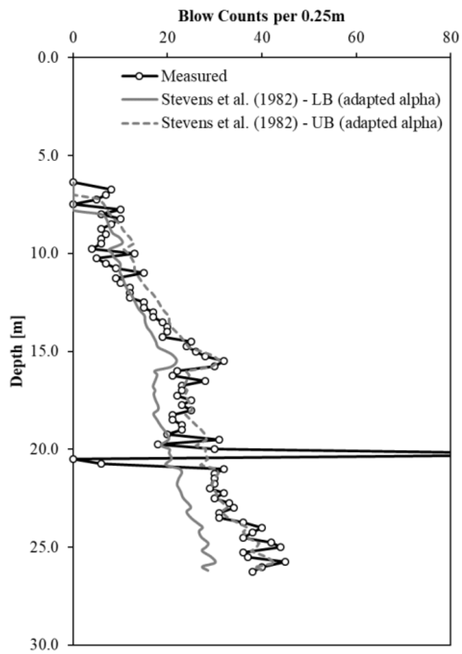

Figure 5. Back-calculations at location Y5

Nevertheless, it was also observed that this approach still slightly underestimated when compared to the measured data - lower bound prediction with best estimate soil parameters falling slightly below measured values. This has not been completely quantified and may be due to some spatial variation of the soil sensitivity; however, it was noted through PDM analysis that there may be some magnitude of radial damping present which is most likely due to the pile geometry and potentially a result of the clay fissuring also. This three-dimensional effect is not accounted for in the one-dimensional approach that is used within GRLWEAP (2010) and hence may be an additional reason behind the underestimation.

To improve the prediction of the dynamic response whilst pile driving, three-dimensional effects should be researched further and implemented in current practice. The current prediction approach uses knowledge developed in the 1960’s (Smith, 1960) that only accounts for one-dimensional effects. With short rigid larger diameter monopile foundations becoming the most common offshore foundation type for wind turbines, it is important that these effects (i.e. soil plug behaviour, radiation damping and radial flexibility), which are anticipated to be more emphasised for these larger foundations, are accounted for.

7 CONCLUSIONS

Current pile driving prediction methods for driven monopile foundations have been evaluated on a site which comprises mainly fissured London clay. The results have indicated that soil resistance and hence blow counts can be severely underestimated if certain approaches or assumptions are made in relation to the soils response to driving.

It is tentatively concluded that the reasoning behind the significant underestimations that were seen when using the Stevens et al. (1982) approach was mainly due to the method not accounting for the increase in soil strength in relation to the sensitivity of the clays and hence increase in undrained shear strength as the fissures within the soil are destroyed and removed during driving.

Overall, to ensure the soil resistance to driving is accurately represented, it is proposed that the soil sensitivity should be considered and SRD calculations should be compared to assess the magnitude of difference to allow a decision to be made on their suitability for the given site. In the case of this site, the Alm and Hamre (2001) approach and the use of an equivalent alpha factor (inverse of soil sensitivity) when using the Stevens et al. (1982) gave much improved predictions when compared to the measured data.

8 ACKNOWLEDGEMENTS

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9 REFERENCES

T. Alm and L. Hamre, “Soil model for driveability predictions” Offshore Technology Conference 8835, Houston, May 1998, pp. 13.

T. Alm and L. Hamre, “Soil model for driveability predicitions based on CPT interpretations”, 15th International Conference on Soil Mechanics and Foundation Engineering, Istanbul, Aug. 2001, pp. 1297-1302.

API, API RP2A WSD Recommended practice for planning, designing and constructing fixed offshore platforms – working stress design, 2000.

J. L. Colliat, P. Vergobbi and A. Puech, “Friction degradation and set-up effects in hard clays offshore Congo and Angola”, Offshore Technology Conference 7192, Houston, 1993, pp. 627-635.

DNVGL, DNV-RP-E03 Geotechnical Design and Installation of Suction Anchors in Clay, 2005.

A. Meynard and J. Corte, “Experimental study of lateral resistance during driving”, 2nd

International Conference on the Application of Stress-Wave Theory of Piles, Stockholm, 1984.

PDI (Pile Dynamics Inc.), GRLWEAP, 2010.

H. J. Roussel, 1979, "Pile driving analysis of large diameter high capacity offshore pipe piles", (unpublished work – PhD thesis, Tulane University, School of Engineering).

R. M. Semple, and J. P. Gemeinhardt, "Stress history approach to analysis of soil resistance to pile driving", Offshore Technology Conference 3969, Houston, 1981, pp. 165-169.

H. A. Simons and M. F. Randolph, "A new approach to one dimensional pile driving analysis", 5th International Conference on Numerical Methods in Geomechanics, Nagoya, 1985, pp. 1457-1464.

E. A. L. Smith, "Pile-driving analysis by the wave equation", Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, vol. 86, no. 4, Aug. 1960, pp. 35-61.

R. S. Stevens, E. A. Wiltsie and T. H. Turton, "Evaluating pile driveability fr hard clay, very dense sand and rock", Offshore Technology Conference 4205, Houston, 1982, pp. 465-469.

F. E. Toolan and D. A. Fox, Geotechnical planning of piled foundations for offshore platforms, ICE Proceedings 62 Issue 2, May 1997, pp. 221-244.