Proceedings of the XVIII ECSMGE 2024
GEOTECHNICAL ENGINEERING CHALLENGES
TO MEET CURRENT AND EMERGING NEEDS OF SOCIETY

ISBN 978-1-032-54816-6
DOI 10.1201/9781003431749-411

End bearing of large diameter piles in sand during driving

Résistance de pointe des pieux de grand diamètre lors du battage dans le sable

D. Cathie*
Cathie Group, Brussels, Belgium

S. Raymackers, T. Vergote, G. Haine
DEME Offshore, Zwijndrecht, Belgium

J. Saraiva, A. Burgraeve
Cathie Group, Brussels, Belgium

**david.cathie@cathiegroup.com*

ABSTRACT: Current methods for calibrating the end bearing component of soil resistance to driving (q_u) in sand are based on factoring or capping the cone tip resistance (q_c). Results of a detailed back analysis of driving behaviour of large diameter piles in sand using a large database of pile driving data from several windfarm sites are presented. Self-weight penetration (SWP) records are assessed indicating q_u/q_c~0.5. Signal matching for piles in the PAGE JIP indicated unit end bearing <0.2q_c. Back analysis of three large windfarm sites revealed q_u in the range 0.13-0.25q_c with the lower value being more reliable. Differences between SWP and dynamic end bearing are explained in terms of drained and undrained soil resistance. The undrained resistance is much lower than the drained resistance because the operational undrained shear strength is affected by the effective stresses around the pile tip during dynamic penetration which in turn are governed by hydraulic fracture and cavitation.

RÉSUMÉ: Les méthodes actuelles de calibration de la composante en pointe de la résistance du sol au battage (q_u) dans les sables sont basées sur un facteur de réduction ou le plafonnement de la résistance à la pointe du cône (q_c). Les résultats d'une analyse à postériori détaillée du comportement de battage de pieux de grand diamètre dans le sable à l'aide d'une vaste base de données de battage de pieux provenant de plusieurs parcs éoliens sont présentés. Les mesures de pénétration sous poids propre (SWP) sont évaluées et indiquent q_u/q_c~0,5. L'analyse par calage de signaux sur les pieux effectués dans le cadre du PAGE JIP a indiqué une résistance unitaire de pointe <0,2q_c. L'analyse à postériori de trois grands parcs éoliens a révélé que q_u se situait entre 0,13 et 0,25q_c, la valeur la plus faible étant la plus fiable. Les différences entre le SWP et la résistance en pointe dynamique s'expliquent en termes de résistance du sol drainée et non drainée. La résistance non drainée est beaucoup plus faible que la résistance drainée parce que la résistance opérationnelle au cisaillement non drainé est fonction des contraintes effectives autour de la pointe du pieu pendant la pénétration dynamique qui, elles-mêmes, sont régies par la fracture hydraulique et la cavitation.

Keywords: Pile; driveability; end bearing; cone resistance; undrained.

1 INTRODUCTION

Current pile driveability methods do not always provide good predictions for large diameter monopiles and various authors and groups are addressing this problem (Stergiou et al, 2023, Yenigul et al, 2023; Jones et al, 2020; Vergote et al, 2022; Cathie et al, 2020). The two components of the Soil Resistance during Driving (SRD) are shaft friction and annulus end bearing. The end bearing component is the subject of this paper. Starting with static end bearing assessed from self-weight penetration, the study investigates the end bearing using a detailed back analysis of driving data over the full range of penetration depths and correlation to the cone penetration resistance, q_c.

Three major monopile installation projects are considered in this paper with a total of 190 locations. Pile diameters were 6.7 – 7.8m OD and pile tip wall thickness 56 – 85mm. All sites were in the North Sea in predominantly fine to medium sand, although some locations contain layers of dense silt and clay (which do not dominate the SRD but are useful to indicate rapid changes in end bearing).

An independent approach to assessing unit end bearing is also reported using signal matching of dynamic pile monitoring measurements.

2 END BEARING ASSESSMENT

2.1 Previous work

Methods of assessing the annulus end bearing resistance during driving of offshore tubular piles (q_u) in sand are widely based on the cone tip resistance (q_c) using a ratio q_u/q_c. Alm and Hamre (2001) proposed $q_u = 0.15\ q_c^*$ where q_c^* is $(q_c/\sigma'_{v0})^{0.2}$, which lead to approximately q_u/q_c of 0.35-0.55, while Maynard et al (2019) found q_u/q_c=0.1. Schneider and Harmon (2020) studied three well documented cases and found q_u/q_c =0.35-0.45. Jones et al (2020) adopted q_u/q_c =0.6 for all soils.

Others have assessed end bearing driving resistance by adapting the static end bearing capacity (Byrne et al, 2012; Lehane et al, 2022). Lehane et al consider the mobilisation distance for end bearing and select q_u=0.4q_c in their adaptation of the Unified method. How the raw cone resistance profile is smoothed is important for most methods, for example by considering ±1.5D* (where D* is the equivalent diameter) above and below the pile tip (Lehane et al, 2022).

Independent back analysis of a large quantity of driving data by Stergiou et al (2023) led to a correlation: $q_u = (0.15 + 0.35A_r)q_c$. For the monopiles used in the present study, the area ratio, A_r is in range 0.03-0.04 leading to q_u/q_c~0.16.

Cathie et al (2020) in a detailed signal matching study for a large diameter monopile (7.8m OD) imbedded in a very dense sand layer (qc > 100MPa) at 30m depth, in a water depth of 30m, revealed a unit end bearing in the range 6 – 7.5MPa. Using a q_c limit of 50MPa indicated q_u/q_c of 0.12 - 0.15 which was considered to be constrained by cavitation of the pore water during dynamic penetration.

Based on strain measurements during penetration, White (2002) showed that for drained penetration in sand significant grain crushing and particle breakage occurs just below the pile tip in a "nose cone" with zones of vertical compression/horizontal extension, horizontal compression/vertical extension and dilation. Rapid loading tests of piles in sand in a centrifuge by Hölscher et al. (2012) confirm the different zones of contraction and dilation as measured by the excess pore pressure response either positive or negative (suction). The pore pressure response was also dependent on the partial dissipation which occurs during the rapid loading test.

When using CPT data for driveability studies, the pile embedment depth into a stronger layer and the effects of a weaker layer below influence the correlation between q_c and q_u. In dense sand, a "critical depth" to achieve "quasi-stationary" penetration resistance is between 10 – 20 times the governing dimension – either the cone diameter, or the pile diameter for closed or plugged piles (Puech and Foray, 2002). For unplugged piles, the pile wall thickness at the tip may be considered the relevant dimension, which is the case for all piles considered in this paper. Thus, pile penetrations into stronger layers require 1.0-1.5m to develop full quasi-static resistance.

Whether the pile tip response is drained or undrained may be a key consideration for assessing end bearing. The normalised velocity $v.d/c_v$ where v is the pile velocity, d the pile wall thickness, and c_v the coefficient of consolidation of the sand), may be used: when $v.d/c_v$<0.1 drained conditions may be assumed whereas for $v.d/c_v$>30 undrained conditions apply (White et al, 2018). Considering a pile wall thickness of 75mm and a rate of penetration of 2m/s (rise time of about 5ms, penetration 0.01m), c_v limits for drained and undrained conditions are >1.5 or <0.005 m^2/sec, respectively. Many dense fine to medium sands will fall between these limits (partially drained) while dense fine silty/clayey sands may well respond undrained. This may explain the uncertainty which currently exists on the appropriate end bearing resistance.

Accumulation of excess pore water pressure may develop around the shaft in fine sands due to partial drainage under typical blow rates of 30-100 blows/minute. However, this is less likely around the pile annulus despite the highly variable strain field with areas of contraction and dilation (White, 2002). Nevertheless, particle breakage will occur due to the very high stresses associated with driving in dense sand which cause contraction and average pore pressure increases and lower operational undrained shear strength.

Finally, the need to consider the spatial variability of the cone resistance in the vertical direction has led to the various methods developed to smooth the cone resistance for pile resistance calculations (see Lehane et al, 2022). However, the spatial variation of the cone resistance around the perimeter of a large monopile will also be significant. A discussion on spatial variability is outside the scope of this paper and may be considered a scale effect of using the CPT cone resistance as an indicator of end bearing resistance.

2.2 Penetration rates

During initial pile installation (pile stabbing), the pile penetration velocity is a similar order of magnitude to the penetration rate of a standard cone penetration test (0.02m/s) and the normalised penetration rate lies in the drained region for sand. Therefore, this self-weight penetration (SWP) resistance is considered a static, drained resistance. Back analysis of the SWP of piles during stabbing provides a reliable benchmark (drained) resistance for performing back analysis during driving.

During pile driving, the average penetration rate is approximately 0.01m/s during normal driving (25 blows/0.25m) and 0.001m/s during hard driving (125 blows/0.25m) depending on the blow rate indicating drained end bearing response during multiple blows even if the single blow response is partially drained or undrained.

In current SRD assessment methods (Alm & Hamre, 2001; Maynard et al, 2019, Stergiou et al, 2023) the (drained) cone resistance (q_c) is used to estimate the (potentially undrained) pile annulus resistance. This has led to applying limiting values to the measured q_c to avoid overestimating the unit end bearing (Cathie et al, 2020).

3 END BEARING RESISTANCE DURING SELF-WEIGHT PENETRATION

The measured SWP under either pile weight or pile plus hammer weight provides a direct measure under essentially static conditions of the soil resistance. At low penetrations in sand, the soil resistance is dominated by end bearing. By reducing the pile and hammer weight by the estimated shaft resistance, the unit end bearing (q_u) can be evaluated with some confidence.

In this study, the SWP for pile and hammer together has been used since all the studied projects involved scour protection and not all piles punched through the scour protection under their own weight. The additional overburden pressure caused by the scour protection increases the effective stresses in the soil and thus the soil resistance. An overburden correction was applied to the raw CPT data (collected in the absence of scour protection) considering the additional stress increment using elastic theory and the effect of this increase on cone tip resistance using the correlations of Krogh et al (2022). As noted above, the cone resistance was smoothed over a 1m interval (+-0.5m from the depth considered).

The shaft resistance to be deducted from the overall pile plus hammer weight to give the end bearing component was obtained using the Alm and Hamre (2001) method after demonstrating in a preliminary study that the SWP was well predicted.

Figure 1 shows the measured SWP plotted against the calculated unit end bearing, normalized by the cone tip resistance (q_u/q_c) at the pile tip depth. q_u/q_c appears to reduce slightly with depth but is generally within the range 0.4 – 0.6.

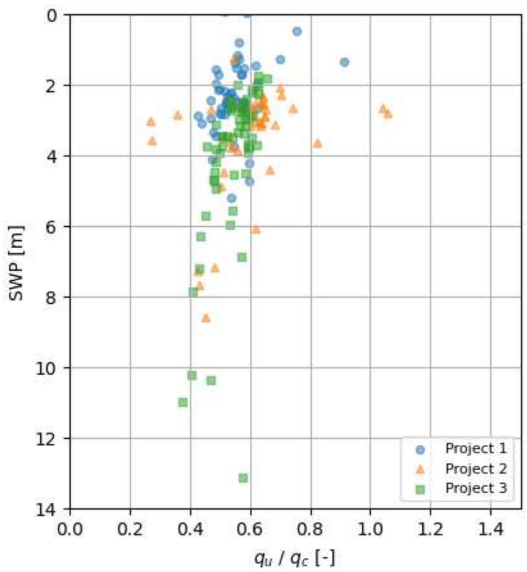

Figure 1. Normalised unit end bearing from self-weight penetration results.

Considering the pile tip wall thickness (t) for the projects in this study, the quasi-static end bearing resistance should be achieved after a penetration of about 1.6m (20*t). Since almost all points exceed this criterion, a correction for shallow penetration is not necessary and therefore the depth dependency is not due to shallow penetration into a layer.

The depth dependency in q_u/q_c appears to be related to the vertical effective stress and the depth dependency disappears when using the Alm and Hamre (2001) formulation for the unit end bearing: $q_u = s_1\, q_c\, (q_c/p_0')^{0.2}$ where p_0' is the effective overburden pressure.

A best fit value s_1=0.15 was derived from this data set which confirms the original formulation of Alm and Hamre is quite appropriate for SWP assessment of large diameter piles although the simpler use of q_u/q_c~0.5 is also satisfactory.

4 END BEARING RESISTANCE DURING DRIVING

4.1 Unit end bearing from signal matching

Since no pile monitoring data was available for the monopile driving for the 3 test sites, the end bearing from the PAGE JIP have been used to provide an independent indicator and backdrop for this study. The PAGE JIP (Cathie et al, 2023) analysed 25 high quality dynamic pile load tests at End of Drive (EoD) and during Restrike by signal matching for piles with diameters between 1.3 - 3.4m. The signal matching process yields the shaft resistance and end bearing at the time of driving. Although signal matching is not as reliable as static measurements or SWP data to assess end bearing, it still provides a realistic estimate of the end bearing resistance for large diameter piles during driving and is the best data available at the present time. End bearing derived from signal matching includes not only the annulus resistance but a component of the shaft resistance very close to the pile toe which cannot be resolved in the wave equation model. In the data presented below, no attempt has been made to correct for the shaft resistance component and therefore the results should be considered a high estimate of the annulus end bearing.

From the PAGE dataset, the unit end bearing is plotted as a function of the cone tip resistance in Figure 2. Cone tip resistance in this figure is the average q_c over +-1.5D from pile tip. The best fit q_u/q_c is 0.20 for the signal matching results.

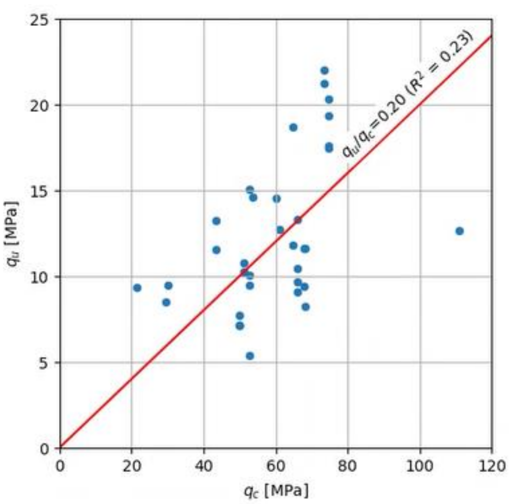

Figure 2. Unit end bearing from signal matching.

4.2 End bearing resistance from back-calculated SRD

Performing a back analysis using a wave equation solution of the three study projects using measured blowcounts, hammer energy and taking account of energy losses due to submergence (Vantomme et al, 2019) leads to a continuous back figured SRD v depth. In the back analysis, a fixed set of quake and damping factors was adopted (quake: 2.5mm, damping shaft and toe: 0.5s/m) to give consistency of SRD with that found from signal matching in sands.

Using the continuous SRD v depth plots, the change in SRD when transitioning from a stronger to a markedly weaker layer may be used to approximately assess the annulus end bearing change (Figure 3). While the shaft resistance is cumulative and only changes relatively gradually with depth, the end bearing resistance reduces rapidly as it penetrates a stronger or weaker soil. The reduction in SRD (ΔQ) is primarily due to a change in end bearing (although a small component of this change is due to the incremental increase in shaft resistance due to the additional penetration and the decrease due to friction fatigue). This can be compared to the reduction in cone resistance (Δq_c) at the layer change, which can be used to develop the approximate relation between Δq_u / Δq_c where $\Delta q_u = \Delta Q/A$ and A is the pile annulus area.

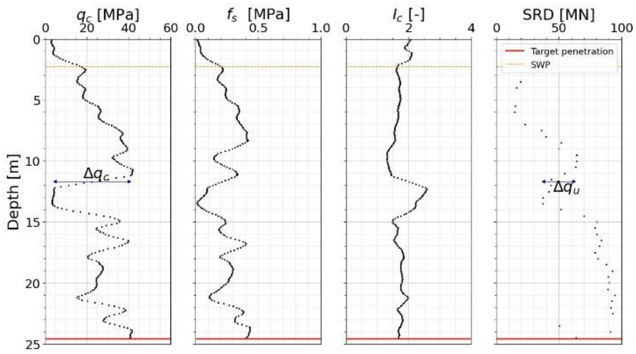

Figure 3. Example of back analysis results and selection of Δq_c and Δq_u.

A manual approach selecting clearly defined cases, and an automatic approach were undertaken. For the manual approach, the main criteria adopted for selecting the locations and depth windows included a) a well-defined change in q_c (typically at least 20MPa), and a sufficiently deep layer of weaker material that the annulus end bearing during driving would reach a quasi-static value (taken as 2m). An example is shown in Figure 3.

There is some subjectivity in the selection of data points in the manual procedure. To exclude any bias, the whole dataset was analysed numerically comparing $\Delta q_u/\Delta q_c$ over a 2m window (selected to obtain a significant change in q_c and q_u, but not too large to be unduly affected by the change in shaft resistance. For the same reason, the dataset was filtered on the soil behaviour index, selecting only cases for Ic>2.4 at the bottom of the window, and Δq_c<-10MPa. The two sets of results are shown on Figure 4 and the histograms on Figure 5.

The manual approach on a much smaller dataset took account of depth mismatches between changes in q_c and q_u, indicates a median $\Delta q_u/\Delta q_c$=0.25, i.e. q_u=0.25q_c. Quite a large scatter of Δq_u may be seen including some locations where there is very little change in SRD despite cone resistance changes in excess of 30MPa.

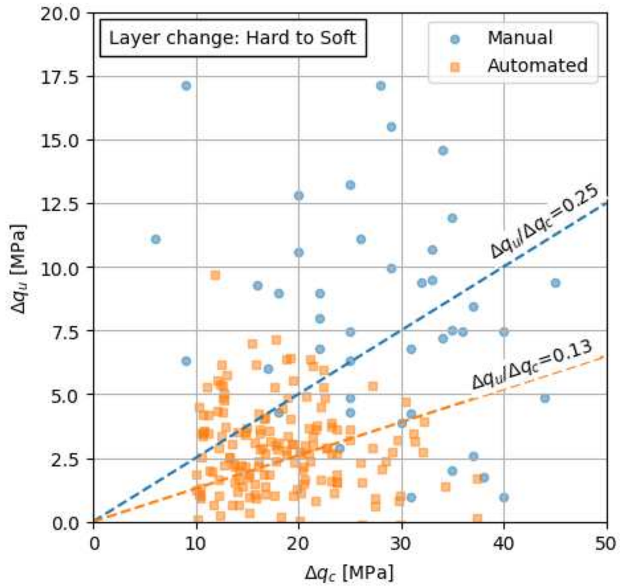

Figure 4. Change in cone resistance and end bearing for reducing q_c (absolute values plotted)

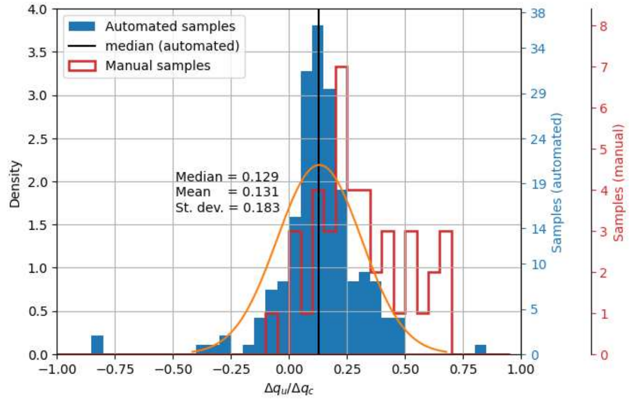

Figure 5. Change is cone resistance and end bearing for reducing q_c (sand into silt/clay) using 2m window applied to full dataset

The automated approach did not account for a depth mismatch between CPT and SRD data but overall, a clearer trend is found compared to the manual method with q_u= 0.13q_c being the best fit. Both approaches assume that the CPT data is representative for the entire pile diameter.

5 DISCUSSION

Based on the static resistance during SWP, the q_u/q_c ratio found for the sand profiles in this study approaches 0.4 below 10m penetration. From signal matching the ratio is less than 0.2, while from back analysis we have 0.13 - 0.25, with the lower 0.13 corresponding to the better data set. The difference between static and dynamic penetration are believed to be due to the different mechanisms occurring during drained static penetration (pile during SWP and CPT) and undrained or partially drained dynamic penetration (pile). In soft clays, House et al (2001) demonstrated a factor of 3.4 difference between drained and undrained penetration resistance while recently Carotenuto et al (2022) have shown by variable rate penetration testing in silts a ratio between cone resistance in drained and undrained conditions respectively of 3.0. Both studies are for contractive soils. In contrast, White et al (2018), in considering free-fall penetrometers, suggests higher undrained resistance for all but very loose sands due to dilation-induced suction pressures based on theoretical considerations.

Applying the White et al undrained strength model for a silica sand with relative density of 90% leads to a mean effective stress at failure of 7.2MPa, an undrained shear strength of 4.7MPa, and q_u of 35MPa. This is not consistent with the results of this study.

The White et al model does not take account of pore water cavitation during driving as discussed by Cathie et al (2020). If the cavitation limits are respected, at 30m penetration and 30m water depth, the mean effective stress at failure is limited to 0.99MPa, the undrained shear strength 0.63MPa and q_u 4.8MPa which is well aligned with the back analysis results.

Lower undrained end bearing during driving in sand may also result from extremely high instantaneous hydraulic gradients around the pile tip. Carotenuto et al (2022) indicated pressure differences between the cone tip and shaft that exceeded u/σ'_{v0} of 3 at a penetration rate of 0.1m/s and could exceed 4-5 during a blow (2m/s). This would translate to upward hydraulic gradients far greater than needed for hydraulic fracturing of the soil and temporary relief of the effective stresses around the pile tip area as the pressure wave in the pore water propagates outwards from below the pile tip. Such pore pressure gradients can also be inferred from the high volumetric strain rate gradients around a pile tip observed by White (2002) during drained penetration.

6 CONCLUSIONS

Based on the methods and data used in this study for large diameter piles in sand, the annulus end bearing observed during self-weight penetration (SWP) was between q_u/q_c = 0.4-0.6. During driving, the ratio was much lower due to undrained (or partially drained) conditions with the data pointing to q_u/q_c<0.2 by signal matching and q_u/q_c~0.13 (for the most reliable results) from global back analysis of the driving logs.

Differences between the drained SWP results and the driving back analysis results are due to the undrained or partially drained conditions operating during a single blow, and the shear strength dependence on a) pore pressure cavitation, and b) hydraulic fracture in dynamic conditions near the pile toe.

Further work is required to evaluate these effects and to test theoretical concepts on databases such as the one used in this paper.

REFERENCES

Alm, T., Hamre, L., (2001). Soil model for pile driveability predictions based on CPT interpretations. Int. Conf. on Soil Mechanics and Geotechnical Engineering. pp. 1297–1302.

Byrne, T., Gavin, K., Prendergast,L.J. Cachim. P, Doherty, P, Pulukul, S.C (2018). Performance of CPT-based methods to assess monopile driveability in North Sea sands, Ocean Engineering, Vol.166, pp 76-91. https://doi.org/10.1016/j.oceaneng.2018.08.010.

Carotenuto. P et al (2023) Study of partial drainage effects in a natural silty deposit using variable rate CPTU testing, Proc. 9th Int. SUT Offshore Site Investigation and Geotechnics Conf, London.

Cathie, D. Jaeck, C. Ozsu, E. and Raymackers, S (2020) Insights into the driveability of large diameter piles, Int. Symp. Frontiers in Offshore Geotechnics, Austin, Texas.

Cathie, D. Jardine, R, Silvano, R. Kontoe, S. and Schroeder, F. (2023). Axial capacity ageing trends of large diameter tubular piles driven in sand, Soils and Foundations, Vol. 53, 6, https://doi.org/10.1016/j.sandf.2023.101401.

House, A. R., Randolph, M., & Watson, P. (2001). In-situ assessment of shear strength and consolidation characteristics of soft sediments. In C. Aubeny, & J-L. Briaud (Eds.), Proc. OTRC 2001 Int. Conf., Houston, USA, pp. 52-63. Offshore Technology Research Centre.

Hölscher, P., van Tol, A.F., Huy, N.Q., 2012. Rapid pile load tests in the geotechnical centrifuge. 9th Int. Conf. on Testing and Design Methods for Deep Foundations 52, 1102–1117.

Jones, L. Rattley, M. and Mancea, S (2020) A CPT-based soil resistance to driving (SRD) method for offshore pile driveability analysis, Proc. Piling 2020 Conf., ICE Publishing, pp 207-216.

Krogh, L. Quinteros, S. Engin, H.K. and Lunne, T (2022) Revisiting interpretation of relative density from shallow depth CPTs in sand, Can. Geotech. J. 59: 808–826, https://dx.doi.org/10.1139/cgj-2021-0200.

Lehane, B. Igoe, D. Gavin, K. Bittar, E. (2022) Application of the Unified CPT method to assess the driving resistance of pipe piles in sand. 11th Int Stress Wave Conf., Rotterdam.

Maynard, A.W., Hamre, L., Butterworth, D., Davison, F., 2019. Improved pile installation predictions for monopiles. 10th Int. Conf. on Stress Wave Theory and Testing Methods for Deep Foundations. ASTM Int.pp.426–449, http://dx.doi.org/10.1520/STP161120170164.

Puech, A., Foray, P., 2002. Refined Model for Interpreting Shallow Penetration CPTs in Sands. Presented at the Offshore Technology Conference, Houston, Texas.

Schneider, J.A., Harmon, I.A., 2010. Analyzing drivability of open-ended piles in very dense sands. J. Deep Found. Inst. 4, 32–44.

Stergiou, T. Perikleous, G. and Meissl, S. (2023). MonoDrive: A novel SRD methodology for offshore wind monopile foundations developed based on an extensive driveability database, Proc. 9th Int. SUT Offshore Site Investigation and Geotechnics Conf, London.

Vergote, T and Raymackers, S (2022). Building a framework for probabilistic assessment accounting for soil, spatial, operational and model uncertainty, applied to pile driveability, Ocean Engineering, Vol.266, part 5.

Vantomme, F. Versteele, H. and Cathie, D (2019) Driving energy losses for constant diameter and tapered submerged monopiles, Applied Ocean Research, 82, pp337-345.

White, D.J., 2002. An investigation into the behaviour of pressed-in piles, PhD Thesis, University of Cambridge.

White, D.J., O'Loughlin, C.D., Stark, N and Chow, S.H. Free fall penetrometer tests in sand: Determining the equivalent static resistance, Cone Penetration Testing 2018, pp695-701.

Yenigul, N. Yan, Y. Braakenburg, L.C.H. and Thumann, V.M. (2023) Evaluation of pile driveability predictions in sand, Proc. 9th Int. SUT Offshore Site Investigation and Geotechnics Conf, London.