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Axial capacity design practice for North European wind-turbine projects

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ABSTRACT: Improving foundation design is central to the offshore wind industry developing deeper water sites. This paper reviews the technical and regulatory difficulties for design of axially loaded piles to German offshore windfarm projects. It is argued that moving towards reliable forward predictive pile design methods and away from 'dynamic proving tests' will be vital to reducing unnecessarily high material and installation costs, installation risks and disturbance to marine mammals. Steps are outlined to implement such a change either in combination with regional or international load and resistance factors.

1 INTRODUCTION

Pile-based multi-footed foundations are used widely for offshore wind turbines and transformer platforms. Developments involving deeper water and larger turbines are likely to found a larger proportion of wind turbines on piled jackets. Piles supporting offshore transformer jacket structures experience comparable loading conditions to oil and gas platforms, where compression loading often dominates design. However, light-weight wind turbine jackets often expose their piles to higher degrees of axial cycling; tension loading cases are also critical.

Offshore piles are commonly designed within the API and ISO frameworks. However, the main text API approach suffers from poor reliability in sands and gives an uncomfortably high Coefficient of Variation (CoV) $\approx$0.70 when assessed against field tests. It also delivers significant biases with respect to Length/Diameter ratio (L/D) and sand state. It is over-conservative for shaft resistance with dense sands and low L/D piles, but potentially non-conservative for large diameter piles' end bearing; Jardine et al. (2005). More accurate CPT methods are now cited by API. Lehane et al. (2005) showed that two of these, the UWA and 'full' ICP methods, reduce predictive CoVs below 0.3. Considerable experience has been gained in applying the ICP in the North Sea and elsewhere since 1995 (Overy 2007).

The consequences of unnecessary conservatism extend beyond additional pile cost. Pile efficiency reduces with L/D and the challenge of driving long piles into dense sand may force designers towards a two pile-per-leg configuration, adding significantly to jacket costs. Other difficulties include increasing installation failure risks and impacts on marine mammals. Moving towards more reliable axial capacity prediction offers environmental and economic benefits, which can only be grasped if accepted by third-party certifying bodies and national agencies.

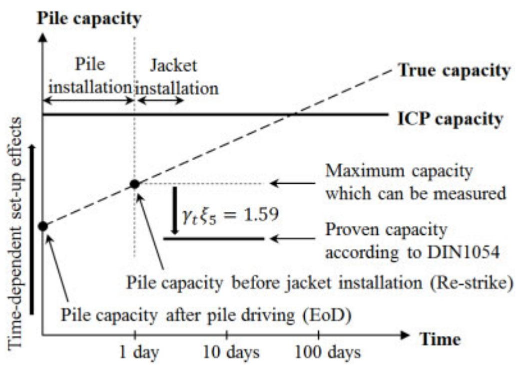

Figure 1. Schematic illustration of the challenge of proving capacity according to DIN 1054:2010-12.

1.1 *Challenge of German regulations*

The German North Sea presents many dense sand sites where the ICP and main text API/ISO approach give widely different predictions. Current local regulations require conformance with the EC7 Framework. Concerns over the limited databases of very large pile tests has led to the German national annex DIN 1054:2010-12 specifying that the capacities of axially loaded piles must be proven by field measurements made after pile installation.

Offshore static pile testing is usually considered excessively costly and it is more common to rely on capacities interpreted from dynamic monitoring

Table 1. Correlation factors from DIN 1054:2010-12 depending on number of piles tested, N.

	$N \geq 2$	$N \geq 5$	$N \geq 10$	$N \geq 15$	$N \geq 20$
$\xi_{0,5}$	1.60	1.50	1.45	1.42	1.40
$\xi_{0,6}$	1.50	1.35	1.30	1.25	1.25

$\xi_5 = (\xi_{0,5} + \Delta\xi) \cdot \eta_D$
$\xi_6 = (\xi_{0,6} + \Delta\xi) \cdot \eta_D$
where $\eta_D = 0.85$ and $\Delta\xi = 0.1$

of driving, or later re-strikes. However, driving measurements offer poor static axial capacity predictions. Likins & Rausche (2008) report a CoV = 0.95 between 'dynamic' and short-term static capacities. As emphasised in Section 2, static capacities grow with time, particularly in sands. However, checking this with long-term re-strike tests is very costly. These uncertainties led DIN to apply a high resistance factor to 'dynamic capacities'. However, this approach poses economic and programme problems for developers. Reliance on dynamic monitoring poses difficulties for developers: the acceptability of any pile can only be verified long after it has been driven; applying unnecessarily stringent LRFD factors leads to added costs, risks and environmental impact.

1.2 *Proving pile capacity set-up*

Cost-effective offshore pile, jacket and top-sides installation involves minimizing vessel mobilization and day-rate costs. Cost and practical issues typically make dynamic pile loading difficult to achieve after installation. Field ageing checks are usually limited to re-strikes a few days after driving. EC7 and DIN 1054:2010-12 require that a resistance factor is applied to the measured pile capacities to account for uncertainties in: a) dynamic pile load testing and b) extrapolating results to neighbouring piles. The 'proven measured pile capacity' is calculated from the measurements $R_{c,m}$ made in N tests by:

$$R_d = \frac{1}{\gamma_t} \cdot \min\left(\frac{(R_{c,m})_{mean}}{\xi_5}; \frac{(R_{c,m})_{min}}{\xi_6}\right) \qquad (1)$$

where $y_t = 1.1$ for compression and $y_t = 1.15$ for tension. ξ_5 and ξ_6 are the factors listed in Table 1.

The total resistance factor for a 4-legged jacket where all piles are 'tested' via pile driving monitoring is $\gamma_t \cdot \xi_6 = 1.590$ and only 62% of the 'measured' pile capacity is utilizable. Additional loading factors are applied in design and DIN 1054:2010-12 effectively requires an equivalent Working Stress Design (WSD) total safety factor of 2.1-2.2 to be applied to the End of Driving (EoD) capacity. As discussed below, shaft capacities grow with time and safety factors of 3.0-3.5 are likely to apply in service life. Rather than ensuring safe foundation design, predicting the dynamic driving resistances becomes the main driver affecting pile lengths, diameters and jacket configuration.

1.3 *Scope for reducing factors by adopting more reliable static predictive methods*

Current offshore German practice implicitly assumes that the static capacities may be predicted more reliably from dynamic measurements than from forward predictions based on modern site investigations and capacity models. As noted earlier, systematic checking shows this to be untrue. The CoV associated with dynamic interpretation ($\approx$0.95 according to Likins & Rausche 2008) is greater than that applying to the API main text method (CoV $\approx$ 0.7) and far above those of modern CPT based design methods ($\approx$0.3 for the ICP and UWA-05 methods). The far lower CoVs and biases of the latter approaches allow fully rational assessments of the resistance factors required to achieve the target reliability levels. Jardine et al. (2005) outline such an approach and report the use of an overall resistance reduction factor of 0.75 in compression and 0.65 in tension for Shell's N. Sea manned platforms, with a single lower factor (0.85) applying to unmanned cases. Additional factors are applied to deal with loading uncertainty. Overy (2007) reports on the fully successful performance of a broad range of significant North Sea projects whose piles were designed by combining the ICP predictive methods with the specified load and resistance factor design (LRFD) factors. The latter approach leads to far more economic outcomes than current German offshore practice.

2 TIME-EFFECTS AND ASSESSMENTS OF AXIAL CAPACITY

It is well known that the axial capacities of piles driven in sands and clays generally increase over time after driving. Changes continue after full pore pressure equalization that may relate to a range of processes including: a) radial effective stresses increasing steadily due to creep processes relaxing circumferential arching around the shaft, b) increased shaft dilatancy developing on loading and c) physiochemical and biological activity that may disrupt shear surfaces and block reduced strength interface shearing. The medium term effects are most pronounced in sensitive low YSR clays; see for example Jardine et al. (2005) or Karlsrud et al. (2014). Base capacity is thought to change less significantly over time, although end resistances maybe smaller during driving than under subsequent static testing. Tension axial load tests performed to failure at different times after driving 19 m long 456 mm OD steel piles in dense North Sea sands at Dunkirk, northern France, are reported by Jardine et al. (2006). The multiple 'first-time' loading tests demonstrated marked increases in capacity over the months following installation, and multiple re-tests revealed that a brittle failure mode applied to the aged piles. Figure 2 shows the increase in pile shaft capacity over time, normalized by the piles' ICP capacities, from the 'first-time' tests at Dunkirk along with recent tests on piles of the same type and scale in Ireland

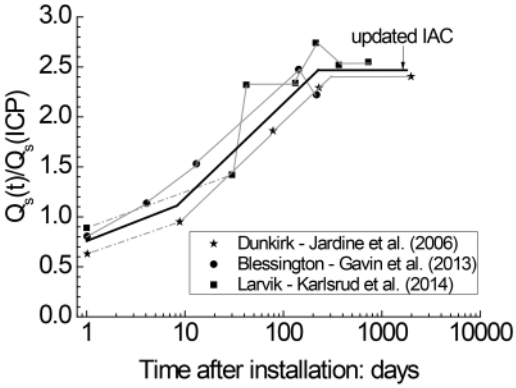

Figure 2. Static tension tests on ≈450mm diameter steel tubular piles at sand sites, capacities Q_s normalized by ICP Q_s predictions; Rimoy & Jardine (2015).

(Gavin et al. 2013) and Norway (Karlsrud et al. 2014). All follow practically the same trends.

Rimoy & Jardine (2015) describe how early age capacities were derived for each case, noting that the shaft capacities measured at the end of driving, by either static or dynamic means, usually fall significantly below the ICP capacities. Ten or more days are usually required for the ICP capacities to be achieved; Figure 2 shows that the latter offer representative predictions for the medium term field capacities. Far larger shaft capacities can be expected over the subsequent months and years in service.

Dynamic test interpretation applies PDA or stress wave inverse analysis procedures that assume highly simplified soil behaviour. The analyst has to assume damping and quake values; this process can be subjective and can lead to non-unique, operator dependnet solutions. Dynamic capacity assessments are inevitably subject to greater scatter, potential bias and poorer reliability than well conducted static testing: Likins & Rausche (2008), Karlsrud et al. (2014), Rimoy & Jardine (2015). Any dynamic checking needs to recognize: a) dynamic base resistances tend to fall well below seen those in static tests, b) the potentially strong effects of ageing and set-up and c) their inherent subjectivity and lower reliability.

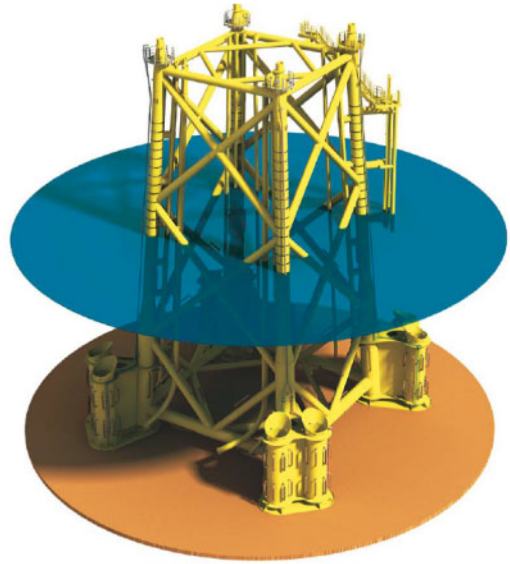

Figure 3. General scheme of eight-pile Borkum Riffgrund Transformer Substation jacket structure.

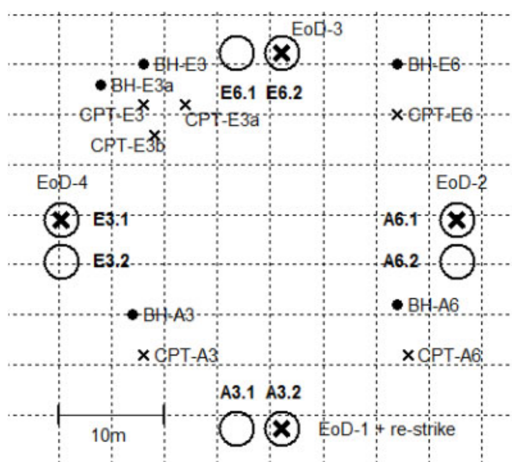

Figure 4. Pile and site investigation layout for Borkum Riffgrund Substation.

3 BORKUM CASE HISTORY

We illustrate our arguments by considering the axial capacities of eight 2.134 m diameter steel tubular piles, with toe wall thicknesses of 45 mm, driven to support a Transformer Substation 37 km off the NW German coast for the Borkum Riffgrund 1 offshore wind project. The jacket, illustrated in Figure 3 was installed in 2013 in a water depth of 24.2 m.

3.1 *Ground conditions at Borkum Riffgrund*

High quality site investigations were carried out for the substation by Fugro in late 2010 including deep boreholes and down-hole PCPT profiles: see Figure 4. The general succession of strata encountered is as outlined below, covering five main sand dominated units and their depths below sea bed:

0 to 12 m Medium to dense SAND
12 to 13 m Interbedded CLAY and SAND
13 to 22 m Dense SAND
22 to 24 m Interbedded CLAY and SAND
24 to 60 m Very dense SAND

High capacity cones were deployed that showed high PCPT q_c values rising from generally 10 to 30 MPa in the first two sand layers to reach a broadly 40 to 110 MPa range in the principally very dense

Table 2. Summary of CAPWAP and ICP axial capacity assessments for Substation piles.

Pile	CPT	Case	Shaft, MN	Base, MN
A3.2	–	CAPWAP EoD	39.3	2.4
	–	CAPWAP re-strike	57.0	4.4
	A3	ICP	36.2	10.3
	A6	ICP	40.0	13.3
A6.1	–	CAPWAP EoD	34.3	3.5
	E6	ICP	36.3	8.9
	A6	ICP	40.0	13.3
E3.1	–	CAPWAP EoD	36.1	3.0
	A3	ICP	36.2	10.3
	E3	ICP	45.5	16.2
E6.2	–	CAPWAP EoD	37.7	2.7
	E3	ICP	45.5	16.2
	E6	ICP	36.3	8.9

sand layer. The piezocones also picked out the typically 1 to 2 m thick clay layers and some locally looser or more silty/clayey sub-layers within the main sand units. Sampling confirmed the conditions summarized above. Site-specific interface ring shear testing was not performed, but Merritt et al. (2012) report data from the nearby Borkum West II site.

3.2 *Pile driving and monitoring*

The Substation piles were driven to 38.5 m penetrations in 2013 with an IHC-S800 hammer. The four piles instrumented with pairs of strain gauges and accelerometers were driven in the sequence A3.2 > A6.1 > E6.2 > E3.1 and their signals recorded fully during driving. All drove with 60–80 blows per meter initially, increasing to 120–160 blows per meter towards the final penetrations. A re-strike test was performed on pile A3.2 six days after its installation. GeoDrive carried out CAPWAP stress wave analyses of the EoD and the re-strike field data. Their interpretation was designed to meet the DIN 1054:2010-12 requirements. The signal matching quality was good, leading to the results and ICP pile capacity estimates given in Table 2 and developed as outlined below.

The Authors' ICP axial capacity assessments adopted submerged unit weights (γ') of 10.5 to 11 kN/m^3 in the main layers and $s_u = 125$ kPa in the two thin clay layers present. Pile-specific analyses were undertaken with the layering tailored to match each local PCPT log. The CPT q_c profiles were discretized for shaft resistance at 0.5 to 1 m intervals for capacity assessment, based on a moderately cautious interpretation of average values and an upper bound of 100 MPa, leading to the example shaft-design qc-depth profile shown in Figure 5 for CPT A3.

Noting that local variations in strata impact more significantly on pile base capacities than shaft resistance (which reflect the integrated profile), a lower bound approach was taken for end bearing assessment which assumed that the piles might tip into the lowest

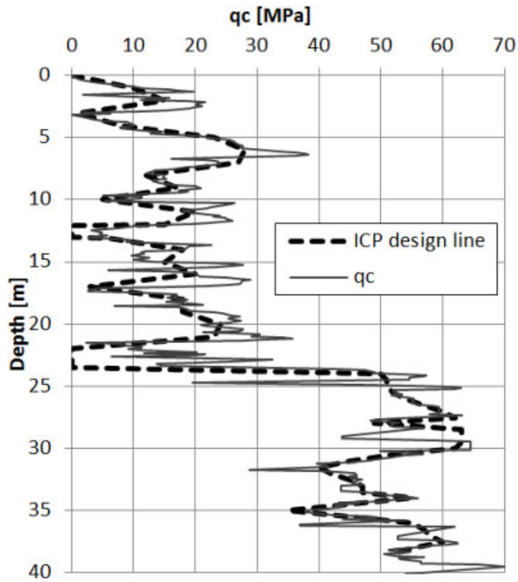

Figure 5. Design idealization of PCPT q_c – depth profiles for ICP sand shaft capacity calculations for CPT A3. Note $q_c = 0$ sections signify clay.

resistance layer found within 2D (4.3 m) of the design penetration depth. The interface shear angles δ were selected cautiously, taking $\delta = 27.5°$ for the sands and 12° for the two minor clay layers present, as no local data was available. Following Tomlinson & Woodward (2008), 1 m of clay down-drag was assumed to have taken place beneath the clay layers into the underlying sands. The ICP approach for clays calls for the Yield Stress Ratios (YSRs) and Sensitivities (S_t) of the clay layers to be specified. The YSRs were assessed from the computed s_u/σ'_v ratios and S_t was taken conservatively as 4.0 in the clays. The compression capacity results are set out in Table 2. Note that the ICP procedure leads to ≈30% lower shaft resistances under tension loading compared to compression in sands (Jardine et al 2005). It also treats any internal skin friction as being relatively small and carried through the base to contribute a minor part of the overall base capacity of large diameter, semi-coring, piles.

3.3 *API and ICP static capacity estimates*

Applying the main text API approach to the generic Borkum Riffgrund profile led to the pile designers making the following capacity estimates:

Shaft outside shaft capacity = 17.81 MN
Shaft inside capacity = 16.03 MN
Total shaft capacity = 33.84 MN
Annular base capacity = 3.61 MN
Inner + outer shaft + annular base = 37.45 MN
Fully plugged base capacity = 42.76 MN

The total API shaft capacities fall 7 to 26% below the 36.3–45.5 MN (external only) range given by the

ICP if fully unplugged coring conditions apply and fall below half of the ICP values if the pile plugs fully. The API base capacity falls below the ICP estimate if an unplugged annular is assumed, but far above it if the pile is assumed to plug. The overall unplugged compressive API capacities fall 17 to 39% below the ICP estimates. Noting that foundation stiffness is usually dominated by external shaft resistance and that any internal shaft resistance relies on mobilizing the relatively soft response of soil beneath the pile toe, the API results also imply a substantially softer response and quite different dynamic behavior during storms to the ICP assessment.

3.4 *Outcomes from Borkum Riffgrund analysis*

The key points from the moderately conservative ICP calculation outcomes and Geodrive CAPWAP assessments listed in Table 2 are:

(1) The CAPWAP EoD shaft capacities are 36.9 MN $\pm 7\%$ and base resistances 2.9 MN $\pm 21\%$.
(2) The mean ratio of the EoD CAPWAP shaft assessments to the ICP predictions is 0.93, which is marginally higher than the early age trend expected from Fig. 2. The ICP calculations adopted conservative interface shear angles. A more optimistic combination of $\delta = 29°$ and $20°$ in the sand and clay layers reduces the CAPWAP/ICP ratio to 0.85.
(3) The EoD-to-static ratio found applying the API method is 1.09, far above that seen in field tests.
(4) The base capacities show far more significant mismatches. The mean CAPWAP EoD/ICP ratio is 0.35, even though a lower bound q_c selection was made. The API 'annular' estimates are closer to the dynamic estimates.
(5) The re-strike indicated pile A3.2's shaft resistance increased by 45% over 6 days, rising more sharply than the research tests plotted in Fig. 2. The restrike CAPWAP/ICP ratio ≈ 1.50 and reduces to 1.35 if the more optimistic δ parameter set is adopted. The re-strike shaft capacity exceeds the API unplugged 'coring' estimate by 69%.
(6) In this case, the base capacity shows a remarkably set-up ratio (1.83) over six days. However, the re-struck value still falls well below the ICP prediction. We recall that the latter is intended to predict the base resistance available after a pile head settlement of D/10 (213 mm) which is orders of magnitude greater than the set developed on re-striking.

4 ENGINEERING RECOMMENDATIONS

The experience gained at Borkum Riffgrund and other sites led to ten recommendations as to how developers can move, with caution, away from 'proving' driven pile axial capacities from dynamic testing and towards more accurate forward prediction methods, supported by dynamic monitoring.

(1) Recognizing that dynamic driving and re-strike tests do not measure medium or long term static capacity. The EoD shaft resistances, after correction for dynamic effects, are subject to substantial scatter and their means are likely to fall 10 to 30% below the medium-term static capacities; more marked reductions ($<70\%$) apply to end bearing.
(2) Calculations based on higher reliability CPT and effective stress based approaches should provide more reliable forward predictions of field capacity. CPT design profiles should be established through a mildly conservative interpretation of high resolution information that is entered at high resolution into shaft capacity calculations. All 'high spikes' covering depth intervals less than 0.4 m should be eliminated and all credible 'low troughs' (excluding starts of pushes from inevitable borehole bases) included.
(3) Continuous high quality measurements are required and caution should be given to q_c values exceeding 90 MPa. An absolute upper limit of 100 kPa is suggested until more experience is obtained.
(4) Allowance must be made in any missing section of PCPT profiles for potentially soft layers, based on the full data set, including geophysics and lab testing. Piezocones and sample descriptions can be key to identifying any sand or silt sub-layers.
(5) End bearing calculations should apply a lower bound CPT profile. The probability that a pile tip will inadvertently terminate in a soft layer is often moderately high and has to be addressed in design.
(6) Site specific interface ring-shear tests should be carried out wherever feasible. Such tests can be highly cost-effective as even modest changes can affect capacity significantly. Conservative assumptions should be made if the data are unavailable.
(7) Allowance should be made for clay being dragged down 1 m below any sand/clay interface, reducing the δ angle applied over the top m of any underlying sand layer to that of the overlying clay.
(8) Allowance should be made for the effects of lateral and axial load cycling, as described by Merritt et al. (2012) and Jardine et al (2012). These factors are likely to reduce operational axial capacity. It is necessary to characterize the distributions of cyclic forces developed at the head of each pile develop in the design storm. Use may then be made of cyclic loading tests (Jardine and Standing (2012) and the pile's lateral displacement-depth profile under extreme conditions to estimate how axial and lateral cycling degrades shaft capacity.
(9) Pile ageing should be addressed when planning and interpreting SRD or re-strike data; see Jardine et al. (2006) and Rimoy & Jardine (2015). Any analysis of the test data-bases should differentiate very clearly between 'first-time and 'multiply-tested' cases – which can show quite different trends.

(10) A ground model approach should be used to interpolate the layering in cases where there is less than one high quality borehole or CPT profile per pile location. Calculations should be run in such cases from the least favorable nearby profile.

5 SUMMARY AND CONCLUSIONS

(1) The current German offshore pile capacity verification requirements for dynamic testing poses three main problems for windfarm developers: a) short term driving measurements are both relatively unreliable and overly conservative due to pile capacities setting-up strongly over time, b) long-term restrike programmes may be difficult or even impossible to achieve and c) applying high resistance reduction factors to the 'measured capacities' results in larger piles and increased driving noise emission.
(2) This paper has argued that moving towards more reliable forward predictive pile design methods and away from 'dynamic proving tests' can lead to better economy. It has also outlined the steps that may be taken to implement such a change in combination with the local EC7 load and resistance factors, or those applied in international offshore practice.
(3) Field experience has been illustrated by reporting driving and pile design data from 2.143 m open ended piles driven at the Borkum Riffgrund 1 site, along with a re-strike test performed six days later.
(4) Stress wave matches show the main text API procedures under-estimating short-term shaft capacity. Better agreement is found with the ICP approach.
(5) The case history gives confidence that the recommended steps will minimize the risks in moving to a more reliable and economic forward prediction based pile design methodology that should help reduce material and installation costs, installation risks and disturbance to marine mammals.

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