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Piles for offshore wind turbines: a state-of-the-art review

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The paper considers the current state of the art for estimating the pull-out capacity of driven open-ended piles used to support wind turbine foundations founded on sand. The latest edition of the American Petroleum Institute guidelines for pile design includes a conventional earth pressure approach and four alternative cone penetration test (CPT) methods for estimating pile shaft resistance in sand. A database of open-ended pile tests was used to assess the predictive reliability of the design approaches. While the earth pressure approach was unreliable, exhibiting bias with pile slenderness and sand relative density, the CPT methods were shown to provide improved and relatively consistent estimates of pile capacity. However, the tension loads experienced by wind turbine foundations are significantly higher than those applied to piles in the database. When the CPT methods were used to estimate the pile length required to support a 5 MW turbine installed in typical offshore soil conditions, the CPT methods provided a wide range of predicted pile lengths. The reasons for this divergence are discussed and an alternative framework for considering driven pile shaft resistance is put forward.

Notation

$A_{r,eff}$	effective area ratio
A_s	area of pile shaft
D	external pile diameter
D^*	equivalent pile diameter
D_i	internal pile diameter
D_r	relative density
G	operational shear modulus of soil
h	distance from pile tip
K	earth pressure coefficient
L	pile length
L/D	slenderness ratio
ΔL	change in pile penetration
ΔL_p	change in soil core length
P_{atm}	atmospheric pressure (= 100 kPa)
Q_c	calculated shaft capacity
Q_m	measured shaft capacity
q_c	CPT cone resistance

R	pile radius
R_i	internal pile radius
t	pile wall thickness
Δy	radial displacement during pile loading
α	scalar CPT coefficient = τ_{av}/q_c
β	$K \tan \delta_f = \tau_f/\sigma'_{v0}$
δ_f	interface friction angle at failure
λ	$\sigma'_{rc,min}/q_c$
σ'_r	radial effective stress
σ'_{rc}	equalised radial effective stress
$\sigma'_{rc,min}$	minimum equalised radial effective stress
σ'_{rf}	peak radial effective stress at failure
$\Delta\sigma'_{rd}$	increase in radial effective stress during loading
σ'_{rs}	stationary radial effective stress
σ'_v	vertical effective stress
σ'_{v0}	in situ vertical effective stress
τ_{av}	average external unit shaft friction
τ_f	external unit shaft friction at failure
τ_{max}	limiting maximum shaft resistance

Abbreviations

bgl	below ground level
CPT	cone penetration test
ICP	Imperial College pile
IFR	incremental filling ratio
NGI	Norwegian Geotechnical Institute
UCD	University College Dublin
UWA	University of Western Australia

1. Introduction

In an attempt to reduce carbon emissions, many countries have developed ambitious targets for energy generation from renewable sources. Both the UK and Irish governments aim to provide 30–35% of their electricity generation from renewables by 2020. Onshore wind turbines are a fairly mature technology, but in order to achieve these ambitious targets it will be necessary to exploit offshore wind resources fully. Offshore developments offer several benefits, including

- (a) the availability of high, unrestricted wind speeds
- (b) the ability to use larger turbines
- (c) the ability to develop combined wind and wave/solar energy installations as alternative renewable energy solutions that become economically viable.

The cost of foundations can represent up to 50% of the development cost for an offshore wind farm. The range of foundations that are in common use include gravity bases, monopiles, jacket structures founded on piles or caissons and, more recently, floating turbines tethered to the seabed with tension anchors (see Figure 1). At sites where the soil at or near the bed level has adequate bearing capacity, gravity bases have been used in relatively shallow water depths (< 25 m). Where the near-surface ground conditions are poor and the water depth is less than 35 m, monopiles, which are large-diameter steel tubes with very high moment resistance, have proven to be an efficient foundation option. Because most developments constructed to date have been built in relatively shallow water depths (< 30 m), 95% of

completed offshore wind farm developments have been founded on either gravity base (20%) or monopile foundations (75%). Most of the developments that are currently planned for construction in the next 10–15 years will be located in water depths ranging from 30 to 70 m. While significant research has been carried out on the development of gravity base foundations, research on monopile and suction caisson-type foundations research is ongoing, and it is likely that many of these new, deeper-water developments will be supported by jacket structures founded on driven open-ended steel piles. A jacket structure consists of a three- or four-legged steel lattice frame founded on a single pile placed below each leg. The critical loading condition for a typical four-legged platform occurs when the resultant environmental load acts at 45° to the structure. In this case, one pile will support most of the compression load, and the opposite pile will experience a significant tension load. Offshore pile design methods have been developed by the oil and gas industries, where platforms usually have sufficient self-weight to ensure that even when environmental loading causes uplift, the piles themselves stay in compression. In this condition the axial load is resisted through a combination of shaft resistance and end bearing resistance. Research by Chow (1997), Lehane and Gavin (2001) and Lehane *et al.* (2005) has led to the development of improved design procedures for estimating the base resistance for piles driven in sand. In addition, each leg of an offshore structure is typically supported by several piles. Therefore the critical loading condition for offshore wind turbine foundations (single pile in tension) is significantly different from those for which the current design methods have been developed and calibrated. For this reason, this paper concentrates on the accuracy with which the tension shaft capacity of offshore piles can be estimated using current design methods.

This paper presents a brief overview of the development of offshore pile design methods. The reliability of these methods in estimating the tension shaft resistance developed by open-ended piles driven into sand is considered. The reader is referred to the work of Randolph (2003) and Jardine and Chow (2007) for comprehensive reviews of factors controlling the axial resistance of piles installed in sand.

2. Evolution of pile design methods

2.1 Earth pressure approaches

The evolution of offshore pile design practice has been driven by the oil and gas industries, and is reflected directly in updates to the American Petroleum Institute (API) RP2A design code. In the first edition (API, 1969), the local shear resistance (τ_f) was estimated using a conventional earth pressure approach

$$1. \quad \tau_f = K\sigma'_v \tan \delta_f$$

where σ'_v is the vertical effective stress, K is an earth pressure coefficient linking σ'_v to the radial effective stress σ'_r and δ_f is

Figure 1. Foundations used to support offshore wind turbines

the interface friction angle. Empirical K values of 0.7 and 0.5 were adopted for compression and tension loading respectively, and δ_f values were assumed to vary with soil type. These K and δ_f values were best-fit empirical factors, which fitted a database of static load tests.

Although the code has been revised several times, the basic form of Equation 1 has been maintained. A review by Olson and Dennis (1982) led to substantive changes in the 1984 edition of the API code. These included the following.

- (a) The tension loading reduction factor was removed.
- (b) Open-ended piles were assumed to develop 20% lower shaft resistance than closed-ended piles, and K values of 1.0 and 0.8 were introduced for closed and open-ended piles respectively.
- (c) δ values were assumed to vary with soil density and particle size.

The API (1984) values for interface friction angle and limiting maximum shaft resistance ($\tau_{\max}$) are given in Table 1.

The use of limiting maximum shaft resistance values in the API code is an area of ongoing contention. They were included in the original 1969 code, removed in 1972 but reintroduced in the 1984 edition, and they remain in the current version. The values were introduced as a result of observations that the average shaft friction (τ_{av}) developed during installation of driven piles appeared to approach a limiting value at some critical depth.

Lings (1985) performed an assessment of the predictive reliability of the API (1984) method. He compared the calculated and measured shaft capacities (Q_c/Q_m) for a database of piles. Despite the reintroduction of limiting friction values, he found significant scatter when he considered the effect of pile slenderness (ratio of pile length L to diameter D), with the capacity of some long piles being significantly overpredicted. Lings also reported a significant

relative density bias, where the API method overpredicted the capacity of piles in loose sand and significantly underpredicted the capacity of piles in dense sand. Van Weele (1989) noted that the latter finding had serious implications for the economic design of foundations in the heavily overconsolidated, very dense, North Sea sand.

2.2 Cone penetration test methods

The difficulty in estimating the operational earth pressure coefficient K , and the natural variability of sand deposits, which means that techniques that average the shaft resistance or soil properties along the pile shaft are questionable, have resulted in a significant move towards developing correlations between τ_f and in situ test parameters. Because of the similarities between the penetrometer installation in the cone penetration test (CPT) and pile installation, and also the widespread use of CPT testing in the offshore environment, correlations between τ_f and the CPT end resistance (q_c) have been extensively explored. Arguably, the most important advances in our understanding of the mechanisms controlling the development of shaft resistance on displacement piles in sand have been obtained from tests performed with the Imperial College pile (ICP) described by Lehane (1992) and Chow (1997). The ICP is a 102 mm diameter, closed-ended steel pile with multiple levels of sensors along the pile shaft. The location of the instruments is described by their distance from the pile tip, h , normalised by the pile diameter D (see Figure 2). Each instrument cluster was capable of measuring τ_f and radial effective stress (σ'_r) during installation and load testing. Measurements of τ_f during installation of the ICP in loose to medium-dense sand at Labenne in France are shown in Figure 2. The data confirm a strong correlation between τ_f and q_c ; the local shear stress profiles measured at the three instrument locations closely mirror the CPT q_c profile. The effects of friction fatigue (a term used to describe the reduction in shear stress mobilised along the pile shaft as a result of cyclic loading) are clearly evident, with the ratio τ_f/q_c measured at a given depth reducing as the pile tip depth (or h/D) increases. Chow (1997) performed additional tests using the ICP in dense sand at Dunkirk in France. Jardine *et al.* (2005) show that the local shaft resistance developed during loading of the pile at both tests sites can be described using the Mohr–Coulomb failure criterion

$$2. \quad \tau_f = (\sigma'_{rc} + \Delta\sigma'_{rd}) \tan \delta_f$$

where σ'_{rc} is the radial effective stress acting on the pile shaft prior to the pile load test, and $\Delta\sigma'_{rd}$ is a component derived by dilation during loading. Equation 2 forms the basis of the well-known ICP-05 design method for displacement piles. Chow (1997) extended correlations originally devised by Lehane (1992) to estimate σ'_{rc} . Although σ'_{rc} values measured at Dunkirk were approximately 300% higher than those at Labenne, she found that σ'_{rc} values at both sites were almost directly proportional to the q_c value at that level and the normalised distance h/D from the pile tip to the instrument location. Jardine *et al.* (2005) proposed

Density	Soil description	δ : degrees	$\tau_{\max}$: kPa
Very loose	Sand	15	48
Loose	Sandy silt		
Medium	Silt		
Loose	Sand	20	67.2
Medium	Sandy silt		
Dense	Silt		
Medium	Sand	25	81.6
Dense	Sandy silt		
Dense	Sand	30	96
Very dense	Sandy silt		
Dense	Gravel	35	115.2
Very dense	Sand		

Table 1. API (1984)

Figure 2. Data from ICP installation at Labenne

the following equation, which is a best-fit correlation with the σ'_{rc} values at the two test sites

$$3. \quad \sigma'_{rc} = \frac{q_c}{34} \left(\frac{h}{R} \right)^{-0.38} \left(\frac{\sigma'_{v0}}{P_{atm}} \right)^{0.13}$$

where R is the pile radius, and P_{atm} is the atmospheric pressure (which can be taken as 100 kPa). In this expression, the constant in the first term describes the constant ratio of radial effective stress to the q_c value mobilised near the toe of the pile, the second accounts for the effects of friction fatigue (a minimum h/R value of 8 should be used), and the third term suggests a weak stress dependence in the correlation. Lehane (1992) used simple elastic cavity expansion theory to estimate the dilational component of radial stress, $\Delta\sigma'_{rd}$, developed during pile loading, as

$$4. \quad \Delta\sigma'_{rd} = \frac{4G}{D} \Delta y$$

where G is the operational shear modulus of the soil, and Δy is the radial displacement during pile loading. Since $\Delta\sigma'_{rd}$ is inversely proportional to the pile diameter, its effects are likely to be relatively small for offshore piles. Jardine *et al.* (2005) recommend that the δ_f value used in Equation 2 should be obtained from simple laboratory interface ring shear tests.

Several reviews, including Chow (1997) and Gavin (1998), have shown that the ICP-05 method is significantly more reliable than

earth pressure approaches such as API. Through consideration of soil state (using q_c) and friction fatigue in their formulation, they removed the significant bias with pile slenderness and relative density exhibited by the API method. However, these advances were achieved from tests that directly measured the radial effective stress response at the pile/soil interface on the ICP. Various model- and prototype-scale, instrumented, open-ended pile tests have been reported (e.g. Lehane and Gavin, 2001; Lee *et al.*, 2003). These tests suggest that the degree of plugging experienced during pile installation affects the shaft resistance developed by a pile. However, none of these tests included horizontal stress measurements, which would allow similar insights into the radial effective stress regime around an open-ended pile. In the absence of such data, Chow (1997) considered methods to correct the value of σ'_{rc} derived from Equation 3 to account for the reduced levels of soil displacement and lower stress level changes caused during the installation of an open-ended pile. Her approach, based largely on observations from tests performed on 324 mm open-ended piles installed in Dunkirk and reported by Brucy *et al.* (1991), was to assume that the σ'_{rc} values developed near the base of open- and closed-ended piles were equal, and that the rate of reduction of shear stress with h/D (friction fatigue term) was increased for open-ended piles. This was achieved by substituting R^* for R in Equation 3 to give

$$5. \quad R^* = \sqrt{R^2 - R_i^2}$$

where R_i is the internal diameter of the pile. This reduction technique, which implies that no plugging takes place during pile installation, was adopted in the ICP-05 design method.

An alternative CPT design method known as Fugro-05 was developed specifically for offshore open-ended piles by Kolk *et al.* (2005b). The authors compiled a database of large-scale instrumented load tests, which included tests performed for the EURIPIDES project (Kolk *et al.*, 2005a), tests at Ras Tanajib (Kolk *et al.*, 2005b) and at Jamuna bridge (Jardine *et al.* 2006) and a small number of additional load tests. They reasoned that since offshore piles usually have large diameters, the effects of dilation could be ignored. They assumed that the interface friction angle was constant at 29°, and that the local shear stress could be calculated using the following expressions.

For compression loading where $h/R^* > 4$

$$6a. \quad \tau_f = \left[0.08 q_c \left(\frac{\sigma'_{v0}}{P_{atm}} \right)^{0.05} \left(\frac{h}{R^*} \right)^{-0.9} \right]$$

For compression loading where $h/R^* < 4$

$$6b. \quad \tau_f = \left[0.08 q_c \left(\frac{\sigma'_{v0}}{P_{atm}} \right)^{0.05} (4)^{-0.9} \left(\frac{h}{4R^*} \right) \right]$$

For tension loading

$$6c. \quad \tau_f = \left[0.045 q_c \left(\frac{\sigma'_{v0}}{P_{atm}} \right)^{0.15} \max \left(\frac{h}{R^*}, 4 \right)^{-0.85} \right]$$

Although Equation 6 maintains the same basic form as Equation 3, wherein the constants were adjusted to fit the author's database, the distribution of shear stress predicted using the Fugro-05 expressions differs significantly from those predicted using ICP-05. Equation 6 predicts the mobilisation of relatively large shear stress at a distance $h/R^* = 4$ from the pile base. Friction fatigue effects are assumed to be much greater, with stresses reducing quickly as h/R increases, and in the vicinity of the pile tip (as h/R reduces to zero).

A study of design methods for piles in sand completed by the Norwegian Geotechnical Institute (NGI) by Clausen *et al.* (2005) resulted in the formulation of a design method known as NGI-05. Although this is ostensibly a CPT method, the cone resistance is used in a correlation to obtain the sand's relative density. Friction fatigue is incorporated using a sliding triangle approach (similar to that suggested by Toolan *et al.*, 1990) and the shaft resistance mobilised by open-ended piles is assumed to be ~38% lower than that for closed-ended piles.

$$7a. \quad \tau_f = \frac{z}{L p_{ref} F_{Dr} F_{sig} F_{tip} F_{load} F_{mat}} \geq \tau_{min}$$

$$7b. \quad F_{Dr} = 2.1(D_r - 0.1)^{1.7}$$

$$7c. \quad D_r = 0.4 \ln \left(\frac{q_{c1N}}{22} \right)$$

$$7d. \quad q_{c1N} = \frac{q_c / P_{atm}}{(\sigma'_{v0} / P_{atm})^{0.5}}$$

where $F_{sig} = (\sigma'_{v0} / p_{ref})^{-0.25}$; $F_{tip} = 1.0$ for driven open-ended and 1.6 for driven closed-ended piles; $F_{load} = 1.0$ for tension and 1.3 for compression; $F_{mat} = 1.0$ for steel and 1.2 for concrete; and $\tau_{min} = 0.1\sigma'_{v0}$.

Lehane *et al.* (2005) performed a review of a proposed updated version of the 21st edition of the API code (API-00) and the three CPT methods (ICP-05, Fugro-05 and NGI-05). They compiled a database of static load tests performed on instrumented piles in sand with which to check the reliability of the existing design approaches (Schneider *et al.*, 2008). As a result of this review, they proposed an alternative CPT design method known as UWA-05, where the equalised radial effective radial effective stress developed by a displacement pile in sand was given by

$$8. \quad \sigma'_{rc} = \frac{q_c}{33} \left(\frac{h}{D} \right)^{-0.5}$$

The UWA-05 design is similar in many respects to ICP-05. The local shear stress is calculated using Equation 2, which includes a dilational component for radial effective stress and a reduction factor for tension loading. The principal difference between the UWA-05 and ICP-05 design guidelines is in their treatment of the effect of plugging on the shaft resistance developed by open-ended piles.

Model pile tests reported by Gavin and Lehane (2003) suggested a direct link between τ_f , q_c and the degree of soil plugging experienced during pile installation. White *et al.* (2005) argued that a logical basis for extending correlations developed for closed-ended piles to open-ended piles would involve applying a modification to the q_c term in Equation 8 that would result in the prediction of lower shaft friction near the pile base. They argued that the degree of plugging experienced by the pile should be considered explicitly in these correlations. Plugging is best considered using the incremental filling ratio (IFR)

$$9. \quad \text{IFR} = \frac{\Delta L_p}{\Delta L}$$

where ΔL_p is the change in the soil core length during an increase in pile penetration ΔL . Using cavity expansion theory, White *et al.* (2005) introduced a scalar reduction factor in the form of an effective area ratio $A_{r,\text{eff}}$, given by

$$10. \quad A_{r,\text{eff}} = 1 - \text{IFR} \left(\frac{D_i}{D} \right)^2$$

where $A_{r,\text{eff}}$ expressed as a power law is used to modify the UWA design methods for closed-ended piles to account for open-ended conditions

$$11. \quad \sigma'_{rc} = \frac{q_c}{33} A_{r,\text{eff}}^{0.3} \left(\frac{h}{D} \right)^{-0.5}$$

Following the review by Lehane and his co-workers, a new edition of the API RP2A (API 2007) was issued. The main text version of this included an updated conventional earth pressure approach to estimate shaft resistance in sand

$$12. \quad \tau_f = \beta \sigma'_{v0}$$

where, for unplugged piles, β (a dimensionless factor) varies from 0.29 for medium-dense sand to 0.56 for very dense sand (see Table 2). Four CPT methods are included in the commentary of the API (2007). These are the Fugro-05, NGI-05, ICP-05 and UWA-5 approaches, with special offshore versions of both the ICP and UWA also being included, although Jardine and Chow (2007) questioned the validity of these offshore variants. The main text suggests that these CPT methods are more reliable than Equation 12, but it also notes that they are largely unproven in the offshore environment, and that therefore Equation 12 should be used for routine design. The β values given in Table 2 apply to unplugged, open-ended piles. For closed-ended (or plugged)

Density	Soil description	β	τ_{max} : kPa
Loose	Sand	N/A	N/A
Medium dense	Sand-silt	0.29	67
Medium dense	Sand	0.37	81
Dense	Sand	0.46	96
Very dense	Sand	0.56	115

Table 2. API (2007)

piles, β values can be increased by 25%. In recognition that previous editions of the code proved unconservative for piles in loose sand, the method is deemed unsuitable for these materials, and users are instructed to use one of the alternative CPT methods.

The current state of the art for designing offshore pile in sand has been discussed in the context of the developments of the API guidelines for offshore structures, which were developed principally for the offshore oil and gas industry. The only design code developed specifically for offshore wind turbine design, the Danish national code (DNV-07; DNV, 2007), contains recommendations for pile design that are identical to the API (1984) recommendations. In the following section, a brief, independent review of the reliability of these various design methods is presented.

3. Reliability of existing design methods

3.1 Comparison of design methods using database piles

A brief review of the reliability of methods contained in the API (2007) and DNV (2007) guidelines is presented in this section. Comprehensive database studies have been reported by Chow (1997) and Schneider *et al.* (2008), but the purpose here is to provide an independent assessment of the approaches for estimating the tension shaft resistance developed by open-ended piles in sand, which is critical for jacket structures designed to support offshore wind turbines. In total, 17 load tests were available from 11 sites (See Table 3). The piles considered in the database study presented here were selected based on the following criteria.

- Only piles with diameters larger than 300 mm and lengths greater than 5 m were considered.
- All piles were installed by driving into primarily siliceous sand deposits where CPT q_c profiles were available.
- Where thin clay layers were present in a given soil profile, the shaft resistance in these layers was estimated using the procedure proposed by Schneider *et al.* (2008).

The shaft capacities calculated using the design methods (Q_c) are compared with values measured in static load tests (Q_m). Summary statistics from the evaluation are given in Table 4, and the ratio Q_c/Q_m is plotted against slenderness ratio L/D and relative density D_r in Figure 3. Points of interest include the following.

- The DNV-07 and API-07 methods provided conservative estimates of the pile capacities (i.e. they underestimated the actual mobilised capacity), with a mean Q_c/Q_m in the range 0.75–0.8, and exhibited the greatest scatter, with a coefficient of variance, COV (= standard deviation/mean) of ~0.49. Since the methods are very similar, only the DNV-07 results are shown. Much of the scatter is seen to result from the methods' bias, where it underestimates the shaft resistance

Site	Pile	L : m	D : m	t : m	$A_{r,eff}$	D_r	Time: days	Q_s : MN	Reference
Dunkirk	CS	11	0.324	0.019	0.57	0.76	191	0.40	Chow (1997)
Dunkirk	CL	11	0.324	0.0127	0.53	0.76	179	0.46	Chow (1997)
Dunkirk zdh	C1	10.0	0.457	0.0135	0.31	0.90	69	0.82	Jardine <i>et al.</i> (2006)
Dunkirk zdh	R1	19.3	0.457	0.0135	0.31	0.78	9	1.45	Jardine <i>et al.</i> (2006)
Euripides	la	30.5	0.763	0.0356	0.21	0.59	7	3.00	Kolk <i>et al.</i> (2005b)
Euripides	lb	38.7	0.763	0.0356	0.22	0.72	2	9.75	Kolk <i>et al.</i> (2005b)
Euripides	lc	47.0	0.763	0.0356	0.22	0.76	11	13.75	Kolk <i>et al.</i> (2005b)
Euripides	2a	46.7	0.763	0.0356	0.23	0.77	7	11.00	Kolk <i>et al.</i> (2005b)
Hoogzand	I	7.0	0.356	0.016	0.45	0.94	37	0.82	Beringen <i>et al.</i> (1979)
Hoogzand	III	5.3	0.356	0.02	0.41	0.87	19	0.53	Beringen <i>et al.</i> (1979)
Hound Point	p	34.0	1.22	0.0242	0.12	0.95	11	3.45	Williams <i>et al.</i> (1997)
Hound Point	p	41.0	1.22	0.0242	0.12	0.87	4	3.40	Williams <i>et al.</i> (1997)
Padre Island	A	14.6	0.508	0.0127	0.21	0.69	2	0.48	McClelland (1974)
Padre Island	A	17.1	0.508	0.0127	0.28	0.66	2	0.65	McClelland (1974)
I-880	2-P	12.3	0.61	0.0191	0.28	0.85	28	2.00	Olson and Shantz (2004)
Los Coyotes	5	14.9	0.356	0.0112	0.35	0.47	2	0.87	Olson and Shantz (2004)
SFOBB	E31R	13.3	0.61	0.0125	0.24	0.63	25	1.34	Olson and Shantz (2004)

Table 3. Database of tension tests on open-ended piles

	API-07	DNV-07	NGI-05	Fugro-05	ICP-05	UWA-05
Mean	0.75	0.80	1.18	0.86	0.95	1.01
COV	0.49	0.49	0.41	0.32	0.26	0.27

Table 4. Results of database assessment

- developed by short piles (with $L/D < 20$) and piles in dense sand (with $D_r > 0.75$).
- (b) The NGI-05 method was unconservative, with a mean Q_c/Q_m of 1.18, and exhibited large scatter, with a COV of 0.41, but it did not exhibit any bias with either slenderness or relative density.
  - (c) The Fugro-05 method was conservative, with a mean Q_c/Q_m of 0.86 and a COV of 0.32. Although the dataset for loose and medium-dense sand was limited, the method appeared to exhibit a tendency to provide better predictions for dense sands.
  - (d) The ICP-05 and UWA-05 methods provided the most reliable predictions, with mean Q_c/Q_m values of 0.95 and 1.01 respectively, and comparable COVs (0.26 and 0.27). Neither method exhibited any tendency to bias.

The brief review presented above is encouraging, in that the newer CPT methods predicted, on average, tension pile capacities within 18% of those measured. When coupled with the typical safety factors used in conventional design practice, such predictions seem acceptable. However, comparing the predicted and measured pile capacity with loads applied to the new generation

of 5 MW wind turbines in Figure 4, it is clear that the operational loads experienced by these wind turbines are significantly outside the range of pile capacities measured in the existing database, and this can have unforeseen consequences (Schneider, 2009). It is therefore of interest to compare the performance of the methods when used to predict the pile length required to support a wind turbine in typical offshore soil conditions.

3.2 Comparison of predicted pile lengths in dense sand

In the example, a typical North Sea soil profile was assumed. The very dense sand was assumed to have a constant q_c value of 50 MPa and an effective unit weight of 10 kN/m³. The first pile considered was a 2.5 m diameter pipe pile with a wall thickness $t = 30$ mm, and the design load required was 30 MN (this is representative of the load applied to a 5 MW turbine, and includes a factor of safety of 2). An interface friction of 27° is assumed. The pile depth required to mobilise this design load is shown in Figure 5(a). The four CPT methods provided broadly comparable predictions, with pile lengths ranging from 24 m (ICP-05) to 32 m (UWA-05 and Fugro-05). This is to be expected, given the broadly similar summary statistics obtained in the database study. The DNV-07 method provided the most

Figure 3. Results of database assessment: (a) DNV-07; (b) NGI-05; (c) Fugro-05; (d) ICP-05; (e) UWA-05

Figure 4. Comparison of SWL for wind turbine with load test database

conservative estimate, with a predicted pile length of 44 m. This is again somewhat unsurprising, given that the database studies found the method to be conservative for dense sands.

If, in order to provide a simple analysis of the bias effects associated with pile slenderness, a pile of 1.5 m diameter was considered and the calculations were repeated (see Figure 5(b)), a somewhat surprising result would be obtained, with the four CPT methods now producing widely divergent results. The ICP-05 method predicted the shortest pile length of 50 m (together with NGI-05). The UWA-05 method predicted a pile length of 95 m, and a 110 m long pile was predicted by Fugro-05. The additional pile length predicted using the UWA-05 design method arose because of the effect of the scalar reduction factor predicting much lower shaft friction near the pile base than in the ICP-05 method. The extreme pile length predicted using Fugro-05 was a result of the method assuming a very high concentration of shear stresses mobilised near the pile tip, thus predicting the shortest length for a 2.5 m diameter pile but the longest length for the 1.5 m diameter pile. As a result of the very large friction fatigue

Figure 5. Estimates of pile length required to support a 5 MW wind turbine: (a) $D = 2.5$ m; (b) $D = 1.5$ m

factor, doubling the pile length produced just a 33% increase in the predicted shaft load, which would seem unrealistic. The DNV-07 method, which previously appeared to be much more conservative in dense sand (see Figure 3(a)) than the CPT methods, predicted a much lower required pile penetration than either UWA-05 or Fugro-05. It is clear from this simple example that extrapolation of design methods to either geometries or loading conditions that have not been considered in their formulation may not be straightforward.

In the review of the reliability of pile design methods it was noted that the formulation of the two most reliable CPT methods

(ICP-05 and UWA-05) relied heavily on measurements of radial effective stresses developed on the closed-ended ICP. It is of considerable interest to compare the shear stress profiles estimated using these methods for closed-ended and open-ended piles installed in identical soil deposits (see Figure 6). In this example, the τ_f profiles predicted using Equation 3 and Equation 8 are first compared. For the purpose of illustration, a 1.5 m diameter pile was installed 60 m into the sand deposit (with the same soil properties assumed in the previous example). The effects of dilation were ignored, and the interface friction angle was again considered to be 27° . The results show the following.

- For a closed-ended pile, the UWA-05 design method predicted τ_f values near the pile tip that were higher than those predicted using the ICP-05 method, but the higher rate of friction fatigue assumed in the UWA-05 method resulted in τ_f values predicted using this method being slightly lower than those predicted using ICP-05 along most of the pile shaft. The predicted values of τ_{av} (τ_f averaged over the entire pile shaft) of 214 kPa (UWA-05) and 231 kPa (ICP-05) resulted in a modest 8% difference in pile capacity being predicted using the two methods.
- When an open-ended pile was considered, the shear stress distributions predicted by the UWA-05 and ICP-05 methods differed significantly. The ICP-05 method predicted much higher τ_f values near the pile base, and although for this example the relative effects of friction fatigue were higher for the ICP-05 method, the τ_f values predicted using this method

Figure 6. Comparison of shear stress distributions from ICP-05 and UWA-05

remained higher than those predicted using UWA-05 along the entire pile shaft. τ_{av} values predicted using ICP-05 were 157 kPa, compared with a value of only 105 kPa using UWA-05 (a difference of 49%).

4. Comparison of radial stresses on closed- and open-ended piles

4.1 Response during installation

Iggoe and Gavin (2011) presented field measurements of radial effective measurements developed during the installation of closed- and open-ended model piles. The closed-ended pile (CE1) and one open-ended pile were jacked in place, and a second open-ended pile (OE2) was driven to 4.1 m bgl. and thereafter jacked to a final penetration depth of 5.6 m bgl. The σ'_{rc} values measured during installation are shown in Figures 7(a) to 7(c), and the IFR values recorded during the installation of the open-ended piles are illustrated in Figure 7(d). The following points are noteworthy.

- (a) The σ'_{rc} values mobilised on the closed-ended pile (CE1) mirrored the CPT q_c profile, and exhibited friction fatigue.
- (b) The σ'_{rc} values mobilised by the open-ended pile that was jacked into place (OE1) also exhibited friction fatigue. The σ'_{rc} values were initially much lower than those measured on pile CE1. However, as the pile became plugged, they increased significantly.
- (c) During the initial stages of installation, the σ'_{rc} values mobilised during driving of the open-ended pile (OE2) were significantly lower than those measured on either CE1 or OE1. As the IFR values decreased, the σ'_{rc} values mobilised

near the pile base (at $h/D = 1.5$) increased slightly, but σ'_{rc} values along the remainder of the pile shaft remained relatively constant. When the mode of installation was switched to jacking (at 4.1 m bgl), the IFR value reduced to zero, and the σ'_{rc} values measured near the pile base increased rapidly.

4.2 Effect of cyclic loading

Cyclic load tests were performed on the piles after installation. When the number of load cycles applied exceeded 50, the σ'_{rc} values reached a constant minimum value, once they were beyond a distance of approximately five to eight pile diameters from the base. These minimum values were similar on both open- and closed-ended piles, and were unaffected by h/D or IFR. Closer to the pile base the residual σ'_{rc} values were affected by both h/D and IFR. However, the cyclic loading regime applied in these tests was compressive. Gavin and O'Kelly (2007) reported tension load tests performed on piles installed in dense Blessington sand. They found that, during tension load, the residual base stresses set up during pile installation reduced. This was accompanied by a large reduction in σ'_{rc} values measured near the pile toe, and resulted in the removal of the h/D effect. The authors proposed a model to predict σ'_{rc} values for open-ended piles

$$13a. \quad \sigma'_{rc} = q_c \left[0.025 - 0.0025 \left(\frac{h}{D} \right) \right] A_{r,eff} > \sigma'_{rc,min}$$

$$13b. \quad \sigma'_{rc,min} = \lambda q_c$$

Figure 7. Comparison of radial stress measurements on open- and closed-ended piles: (a) closed end; (b) OE1; (c) OE2; (d) incremental filling ratio

where λ is a scalar reduction factor that accounts for friction fatigue. Tentative values of $\lambda = 0.003$ (for loose sand) and 0.006 (for dense sand) were proposed, with higher values possible in very dense, overconsolidated deposits. For large-diameter, open-ended piles, which would typically be fully coring during driving, the method suggests a constant shear stress profile relative to the CPT q_c . This differs significantly from the strong friction fatigue effect suggested in both the ICP and UWA methods for open-ended piles (as shown in Figure 6). The new method would be notably more conservative than the ICP or UWA methods for short piles ($L/D < 15$), but would predict larger capacities for slender piles where L/D exceeds 40.

In the tests described by Igoe and Gavin (2011), the cyclic loading regime was one-way compressive loading. White and Lehane (2004) and other workers have found that two-way loading, where the load changes from compression to tension, causes higher rates of friction fatigue. The loading direction may also affect the distribution of radial stress on the pile. Gavin and O'Kelly (2007) reported tension load tests performed on piles installed in dense Blessington sand. They found that, during tension load, the residual base stresses set up during pile installation reduced. This was accompanied by a large reduction in σ'_{rc} values measured near the pile toe, and resulted in the removal of the h/D effect.

The tendency for the model piles to achieve residual constant radial stress and therefore shear stress values suggests that reductions in pile shaft capacity due to cyclic loading effects would be large on piles that experienced a small number of load cycles during installation. Driven piles, however, experience a large number of installation load cycles, and would be expected to have already mobilised their residual shear stress value. Although their axial capacity is unlikely to be adversely affected by in-service cyclic loading, the accumulation of cyclic displacement should be considered by pile designers.

5. Conclusions

Ambitious plans to develop offshore wind farms present significant challenges for geotechnical engineers. The paper described the development of design methods used to calculate the shaft resistance developed by open-ended piles in sand. These foundations will be used extensively to support offshore wind turbines. It was shown that the CPT methods developed for existing piles supporting oil and gas platforms provided accurate and consistent predictions for a database of pile tests. The vertical tension loads applied to the foundations of wind turbines are much higher than those considered in the calibration of these offshore design methods. While the best CPT design methods were fundamentally based on careful and reliable experimental measurements, empirical factors were introduced to account for differences between the geometry of the closed-ended instrumented piles and the open-ended piles used in the offshore environment. When these design techniques were applied to estimate the pile length required to support typical wind turbine loads, conflicting results were obtained.

Some recent measurements of radial effective stresses measured on instrumented open-ended piles were presented. These suggest that the semi-empirical factors do not properly account for differences in the radial stress regime around closed- and open-ended piles. They revealed that piles installed in loose sand experienced much higher rates of radial stress degradation during cyclic loading than piles installed in dense sand. Semi-empirical design methods based on CPT test results and friction fatigue factors that depended only on pile geometry overestimated the radial stress distribution of a pile in loose sand, and underestimated the profile in dense sand. These results suggest that significant savings could be obtained for piles installed in the dense sands typically encountered in the offshore environment.

A new framework was suggested in which friction fatigue effects are controlled by the sand state. It is essential that full-scale measurements of the radial effective stresses developed during the installation and life cycle of open-ended piles are made, in order to properly calibrate either existing design models or this new framework, before they can be used with confidence in the design of piles for offshore wind turbines. While piles installed in loose sand exhibited much higher rates of friction fatigue when they experienced cyclic loading, it is likely that a combination of pile ageing effects and enhanced dilation during static loading could lead to some recovery of pile shaft resistance.

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