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The Geotechnical Challenges Facing the Offshore Wind Sector

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ABSTRACT

The offshore wind sector is undergoing rapid expansion across Europe, driven by the demand for renewable energy and the uncertainty regarding fossil fuel supplies. The proposed wind farm developments are creating significant geotechnical challenges, particularly in terms of efficient foundation design. The majority of wind farms constructed to date have been founded in water depths of less than thirty meters. However 70% of the proposed turbines over the next 10 years will be located in water between 30 and 70m, which increases the lateral loads and moments applied to the foundations. This paper outlines the geotechnical design considerations for turbines located in relatively deep water. The empirical design models commonly used for monopiles and jackets are briefly discussed, in order to highlight the limitations in existing design procedures, originally formulated for the oil and gas sector.

INTRODUCTION

In recent years, increasing pressure to reduce carbon emissions and the dependence on fossil fuels has led to rapid expansion of the renewable energy industry across Europe. Nowhere is this more prevalent than in the offshore wind sector. This expansion is marked by a transition into deeper water and more complex ground conditions which results in significant challenges for the geotechnical design of the turbine substructures. The majority of sites planned for development over the next 10 to 15 years are located in water depths ranging from 30 to 70 m and as a result are outside the scope of existing experience, as shown by Figure 1. As the water depth increases, the span between the turbine superstructure and the seabed also increases resulting in larger moments applied to the foundation. This, coupled with more extreme environmental loading from higher magnitude wind and waves, results in larger moments applied to the foundation. In addition, rapidly developing technology and advancements in construction techniques have resulted in larger turbines becoming available to developers. This trend will see existing machines with capacities from 1.5-3.5 MW being replaced by 5 to 7 MW turbines in future developments. While these turbines generate higher power output, they are also significantly heavier and result in a significantly higher wind loading, placing further demand on the supporting foundation. The combined impact of the increased water depths and larger turbine capacity causes a significant challenge for the geotechnical designer, as the existing foundation solutions are at the limits of previous construction experience. This paper outlines the foundation concepts available to the offshore wind industry and describes the underlying design methodology. The

limitations of existing design approaches are discussed in detail, which leads to recommendations for future research. Due to space limitations and the relevance to North Sea soil deposits, the scope of this paper is limited to piles in cohesionless soil.

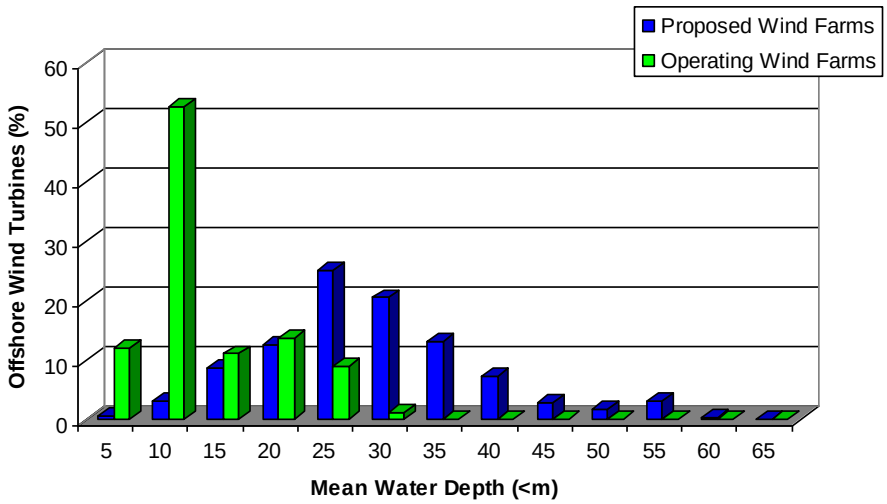

Figure 1: Comparison of Water Depths for existing and proposed Wind Turbines

FOUNDATION OPTIONS

A number of sub-structure options are currently available offshore. These include gravity bases, monopiles, jackets/tripods and more recently, floating turbines tethered to the seabed with tension anchors. These foundation solutions are illustrated schematically in Figure 2. In shallow water, and where the ground conditions below sea-bed have adequate bearing capacity, gravity bases have proved successful. These concrete foundations are similar in principal to onshore turbine foundations and resist the applied load through the bearing resistance of the underlying soil strata and the dead weight of the concrete base. A competent bearing stratum at seabed level is therefore a prerequisite for this foundation option. Where the ground conditions allow relatively easy driving of piles, monopiles - comprising a single large diameter steel tube driven into the sea bed to a specified penetration - have proven to be an efficient and elegant alternative in water depths up to 35m. The DNV (2007) design code suggests monopiles are suitable to water depths up to approximately 25 m, however recent wind farm construction has extended this limit to 35m. These piles resist the lateral wave loading (and resulting moments) through cantilever action, which generates horizontal earth pressures in the direction opposing the applied loads. From 35m to 60m water depths, jacket structures have been used to support the wind turbine super structure. The jacket consists of a steel lattice frame founded on piles under the legs of the structure. The lateral applied loads are transferred through the jacket structure into the foundation piles which resist the loads in a push-pull action through the axial skin friction developed along the pile shaft. In a similar manner, tripod sub-structures rely on the axial pull out resistance of three supporting pile elements. A recent pilot project off the Norway coast has demonstrated the

possibility of using deep water floating turbines. However, the commercial viability of floating designs still warrants significant investigation and there remains considerable scope for improved efficiency. The breakdown of current foundation practice for the operational offshore wind sector is given in Figure 3, which highlights the prevalence of gravity bases (~20%) and monopile (~75%) foundations.

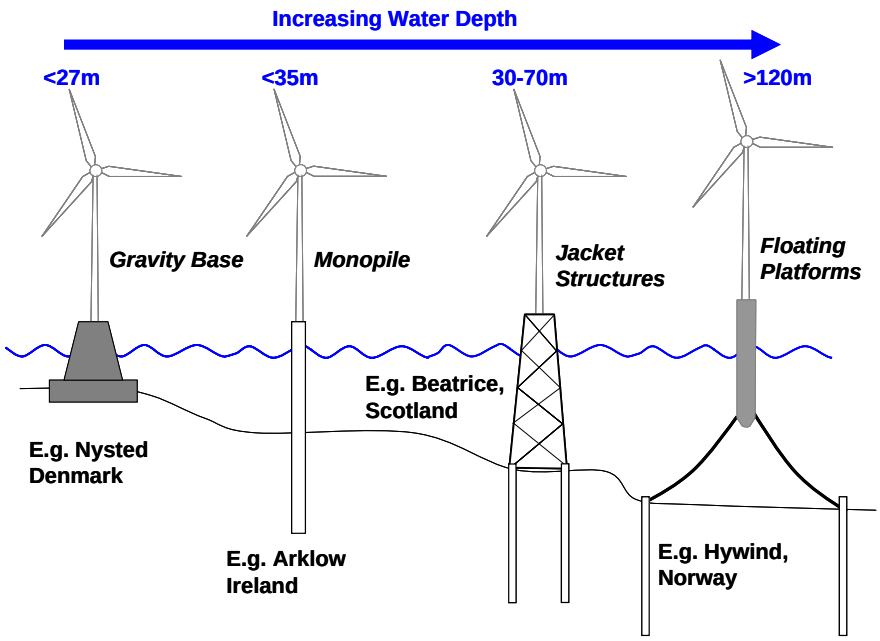

Figure 2: Foundation dependence on water depth

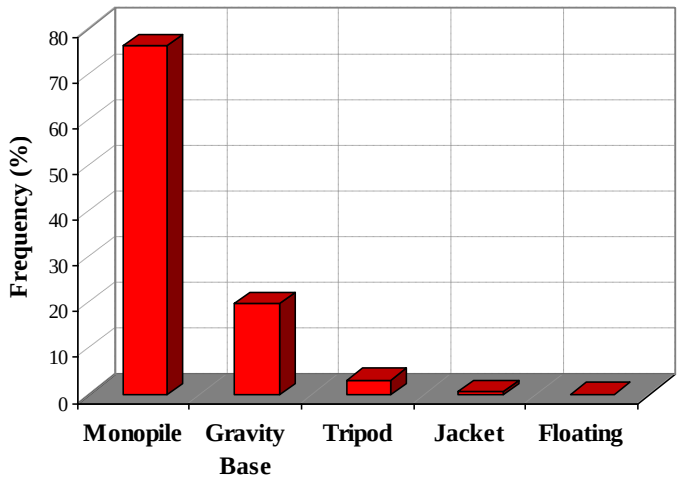

Figure 3: The Foundation Breakdown of turbines currently generating power

LATERAL LOADING

Current Design Standards

The monopile design is controlled by the lateral loads. The most popular method of analyzing piles subjected to lateral loads is the p-y approach, which models the pile as a beam of known stiffness and the soil as a series of uncoupled independent nonlinear springs. This method is the recommended design approach in both the offshore design codes for the petroleum sector (API, 2007) and the standards for the offshore wind sector (DNV, 2007). The solution of the pile stresses and movements to a lateral load applied at the pile head can be obtained from the solution of the differential equations given by Equation 1.

$$EI\frac{d^4y}{dx^4}+Q_A\frac{d^2y}{dx^2}-p(y)+q=0 \qquad (1)$$

Where EI is the flexural rigidity of the pile, x denotes the position from the pile head, y is the lateral deflection, Q_A is the axial load and q is a distributed load along the pile. The governing differential equation (Eqn 1) is readily solved by an iterative finite difference solution.

The DNV (2007) offshore standards for wind turbine design recommend a separate p-y curves for piles in clay and sand. The maximum theoretical soil resistance, p_c, is obtained from the minimum of Equations 3 and 4. These equations represent two separate modes of failure, specifically a wedge failure at shallow depths (giving p_{cs}) and a lateral flow type failure at deeper levels (giving p_{cd}). The transition depth where the deeper failure mode applies is denoted Z_T and occurs where Equation 3 exceeds Equation 4. The ultimate soil resistance, p_u, is then determined by multiplying the theoretical value by a depth dependent empirical adjustment factor, A, which is given by Equation 6 for static loading and is a constant value of 0.9 for cyclic loading conditions. The factors C_1, C_2 and C_3 are dependent on the friction angle of the soil and are determined from Figure 4a.

$$p_c=\min\{p_{cs},p_{cd}\} \qquad (2)$$

$$p_{cs}=(C_1z+C_2D)\gamma' z \qquad (3)$$

$$p_{cd}=C_3D\gamma' z \qquad (4)$$

$$p_u=Ap_c \qquad (5)$$

$$A=(3-0.8\frac{z}{D})\geq 0.9 \qquad (6)$$

In addition z is the depth below ground level, D is the pile diameter and γ' is the effective unit weight of the soil. The p-y curve is generated according to the hyperbolic transfer function in Equation 7, where k is the initial modulus of subgrade reaction obtained from Figure 4b.

$$p=p_u\tanh(\frac{kz}{p_u}y) \qquad (7)$$

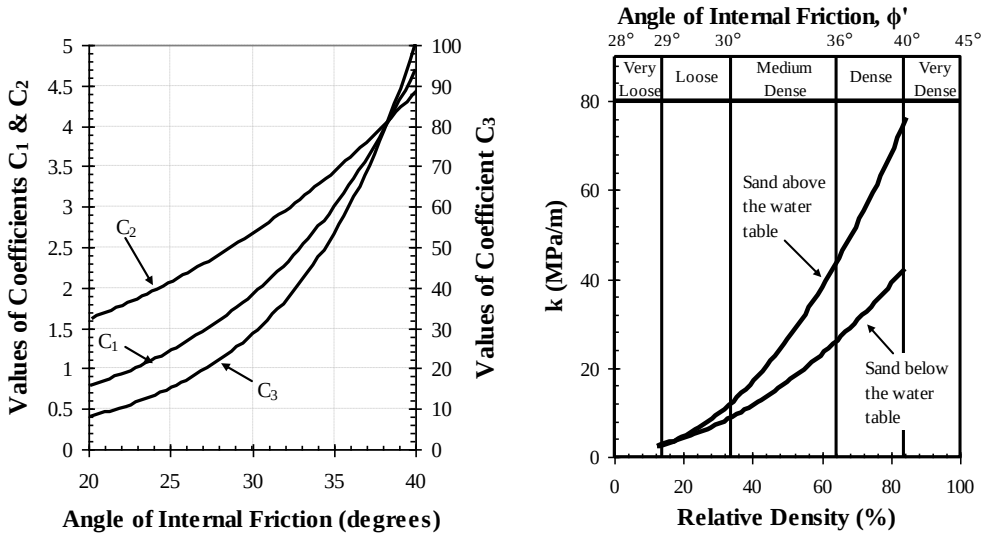

Figure 4: (a) Coefficients in terms of friction Angle (b) Relationship between initial modulus of subgrade reaction, k and angle of internal soil friction (after DNV, 2007)

Limitations of existing standards

1. The main limitation of the existing standards largely arises from the empirical nature of the approach. Monopiles with diameters exceeding 4m have been used in recent wind farm constructions and in subsequent years it is expected this will increase further to 5 or 6m. In contrast the recommended 'p-y' design procedures developed from relatively small diameter load tests. For example, the coefficient A in Equation 6 is an empirical parameter derived from lateral load tests performed by Reese et al (1974) on 610mm diameter piles at Mustang Island. The applicability of this method to large diameter piles is questionable.
2. In addition the slenderness ratio (L/D) of the Mustang Island test was 34.4 which are considerably larger than those commonly adopted by monopiles with slenderness ratios of 4-6. The slenderness ratio can lead to substantial differences in pile behavior. It is generally accepted that monopiles behaved rigidly whereas the Mustang Island tests involved flexible piles.
3. The p-y approach adopts a 2 dimensional response and as such neglects the three dimensional nature of the lateral loading problem. The pile response is illustrated in Figure 5, which shows the lateral reaction as a combination of frontal resistance developed by compression of the resisting soil and shear resistance adjacent to the pile walls. The relative relationship between these components is poorly understood for different geometries and is an additional source of uncertainty in extrapolating the current design procedures to the monopile diameters used offshore.
4. The current method assumes the initial pile-soil stiffness (k) is a direct function of the relative density but is independent of the pile diameter. However, there remains conflicting opinions over the validity of this

assumption. This relationship is particularly important for calculating the deflections of large diameter piles under relatively small loads.

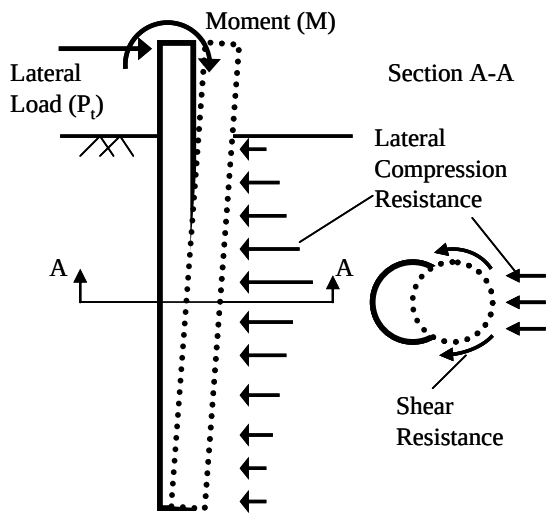

Figure 5: Load transfer of applied lateral load into soil mass

5. As mentioned previously the DNV (2007) code developed from field tests reported by Reese et al (1974). Following this, an inherent assumption of the p-y formulation is that the in-situ horizontal stress is a constant function of the vertical stress, assuming K_0 is 0.4. Fan and Long (2005) showed both the initial stiffness and ultimate soil resistance to be dependent on K_0.
6. An obvious limitation of the p-y approach is the discrete nature of the soil. In reality, the soil will behave as a continuum with the soil reaction at any depth slightly dependent on the adjacent soil response. This includes shearing of adjacent soil particles (or shearing of adjacent spring elements) as the soil reaction is mobilized.
7. The current design for cyclic loading is restricted to the ultimate lateral capacity and fails to consider the accumulation of displacements.
8. The existing approach requires the soil friction angle as a direct input into the analysis, however obtaining a representative friction angle can be difficult. The most common offshore SI includes CPT measurements; however correlations between the CPT tip resistance (q_c) and the friction angel additional uncertainty to the overall design process.
9. The initial modulus of subgrade reaction, k, is only depicted for relative densities up to 80% and therefore significant uncertainties arise in selecting a k value for very dense soils.

AXIAL LOADING

Current Design Standards

The API method remains the most popular approach for designing offshore piles subjected to axial loading. The API method calculates the unit shaft friction, τ_f, using a conventional earth pressure approach, as seen in Equation 8.

$$\tau_f = K\,\sigma'_{v0}\tan\delta < \tau_{max} \qquad [8]$$

Where K is the coefficient of horizontal earth pressure, σ'_{v0} is the in-situ vertical effective stress and δ is the interface friction angle. The API guidelines recommend a K coefficient of 1.0 for displacement piles (closed or plugged) and 0.8 for non-displacement (open, coring) piles. The additional parameters required for API (2002) are reproduced in Table 1. The DNV (2007) code mimics the API main text recommending the same earth pressure approach for the design of wind turbine foundations.

Table 1: API guidelines (1984)

Density	Soil Description	δ (degrees)	τ_{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		

The most recent version of the API code released in 2007 considers four new CPT methods included in the commentary to the main text and states that these methods are preferred to the conventional earth pressure approach. Two of these methods will be briefly considered here, namely the IC-05 (Jardine et al, 2005) and UWA-05 (Lehane et al, 2005) methods. IC-05 relates the radial stresses at failure to the in-situ soil state by incorporating the CPT tip resistance, q_c, into Equation 9.

$$\tau_f = (\sigma'_{rc} + \Delta\sigma'_{rd})\tan\delta \qquad [9a]$$

$$\sigma'_{rc} = \left[0.029 \cdot q_c \left[\max\left(\frac{h}{R^*}, 8\right)\right]^{-0.38} \left(\frac{\sigma'_{v0}}{p_{ref}}\right)^{0.13}\right] \qquad [9b]$$

$\Delta\sigma'_{rd}$ refers to the increase in radial effective stress during loading and is considered negligible for large diameter offshore piles. δ refers to the interface friction angle, h is the distance from the pile tip and R* is the radius of an equivalent closed ended pile with the same solid cross sectional area. One of the most significant advancements of IC-05 is the inclusion of the h/R or friction fatigue term, which accounts for the reduction in radial effective stress at a given location as the pile tip is advanced to deeper levels. The h/R* term used for open ended piles amalgamates the impacts of

friction fatigue and end condition, suggesting more extreme degradation of the radial effective stresses during open ended penetration than closed ended penetration.

The UWA-05 adopts a similar format but differs in the way in which it deals with open-ended piles. The h/R^* term which is used in the IC-05 method is replaced with an h/D term to account for friction fatigue and a separate term to account for the lower radial stresses mobilized by open-ended piles. UWA-05 can be expressed as:

$$\sigma'_{rc} = \left[0.03 \cdot q_c \left[\max\left(\frac{h}{D}, 2 \right) \right]^{-0.5} A_{r,eff}^{\;0.3} \right] \qquad [10a]$$

$$A_{r,eff} = 1 - IFR\,(D_i / D)^2 \qquad [10b]$$

Where the IFR is the incremental change in internal soil plug length for a given pile penetration. Di is the internal pile diameter.

Limitations of existing standards

The above methods are often used arbitrarily by designers to predict the pile capacity at a given site. However the differing methodologies and underlying assumptions result in wide variations in the calculated capacities. To demonstrate this a large diameter (2.5 m) offshore pile was designed to support a 30MN load in dense sand typical of the North Sea (q_c=50MPa). The above design methods predict a wide range in pile lengths from 21m for the IC-05 method up to 44 m depth for the API method (see Figure 6). In general the API/DNV method is shown to be conservative, with the more recent approaches predicting shorter piles to reflect the high in-situ density reflected in the q_c measurements. However even the two CPT approaches differ by a significant margin, reflecting the differing assumptions regarding friction fatigue and the pile end condition. The difference between the API and the CPT methods are explored further for a range of geometries in Figure 7. The API approach is shown to predict grossly conservative capacities with respect to both the IC-05 and UWA-05 approaches for short piles and in particular for short small diameter piles. As the pile slenderness ratio (L/D) increases the CPT approaches tend toward the API method. The UWA method even becomes more conservative than the API approach for long large diameter piles (see Figure 7b). A clear impact of pile diameter is evident in Figure 7 with the CPT approaches providing increasingly closer approximations to the API approach as the pile diameter increases for a given slenderness ratio. This suggests that the differences in pile capacities between the API and CPT approaches

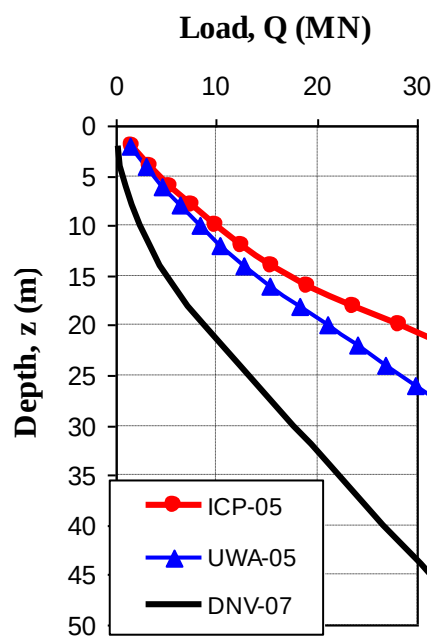

Figure 6: Pile Predictions

is minimized by using long large diameter piles, however even a small difference in resistance can have serious economic implications for the cost of driving piles.

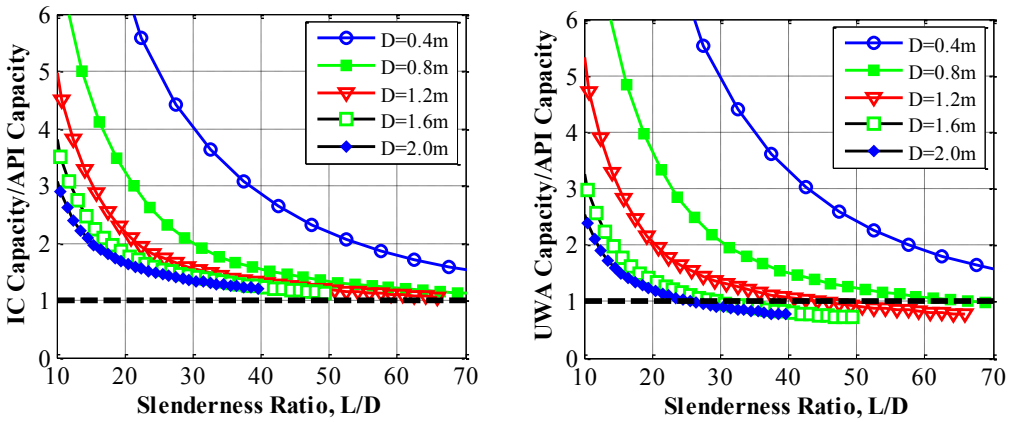

Figure 7: Comparison of IC-05 and UWA-05 with API Main Text

The differences between the UWA-05 and IC-05 methods are compared directly in Figure 8 as a function of the pile geometry. At low slenderness ratios the UWA capacity is greater than the IC-05 capacity, with the ratio of these design methods reducing as the slenderness ratio increases. This reflects the higher friction fatigue effect incorporated into the UWA-05 approach. A strong impact of diameter is also present with the UWA-05 method decreasing with respect to the IC-05 as the diameter increases for a specific slenderness ratio. Large diameter piles (>1.2m) designed using the UWA-05 method always predict a smaller resistance than the IC-05 approach resulting in longer pile lengths when using the UWA-05 method than IC-05, for these geometries. These CPT based design methods are a welcome advancement on the previous API approach; however these methods are being extrapolated well outside the database from which they were formulated and there remains considerable uncertainty regarding the impact of pile geometry on factors such as friction fatigue. As a result it is difficult to say which of the new methods is the most appropriate as they remain largely untested on offshore scale piles. This problem is exacerbated for the offshore wind sector where larger pile geometries may be used than those employed by the oil/gas industry. This highlights the need for additional field testing to validate the relative reliability of these methods and confirm the underlying assumptions (particularly regarding friction fatigue).

There has been extensive axial loading research over the past thirty years, culminating in new design procedures. Unfortunately this has not been mimicked for lateral loading, with the design procedures remaining largely unchanged. This is largely driven by the governing axial loads that control offshore oil/gas platforms. In contrast the next round of wind turbine installations will be controlled by the horizontal loading requirements and as a result highly instrumented field tests are required to develop new reliable CPT based design procedures for lateral loading.

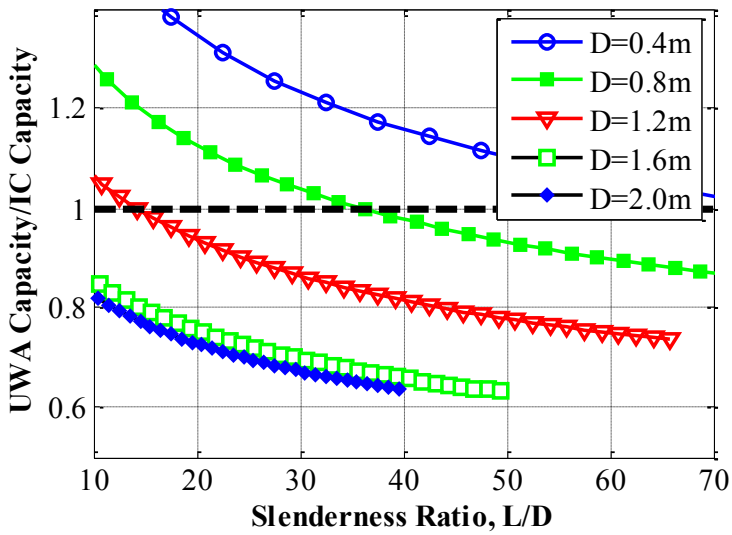

Figure 8: Comparison of IC-05 and UWA-05 for a range of geometries

CONCLUSIONS

The loading conditions and foundation solutions adopted by the offshore wind sector are considerably different from the offshore oil and gas industry. Large diameter monopiles with low slenderness ratios are commonly used in intermediate water depths and jacket/tripod structures are used in deeper waters. Despite the difference in geometries offshore turbine foundations are designed using procedures developed for the oil and gas sector. The limitations of these designs were discussed both for monopile foundations subjected to lateral loading and for jacket piles subjected to axial forces. Additional instrumented large scale field tests would help clarify some of the outstanding sources of uncertainty.

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