Research Repository UCD

Provided by the author(s) and University College Dublin Library in accordance with publisher policies. Please cite the published version when available.

Title	Design Tools Available For Monopile Engineering
Author(s)	Doherty, Paul; Prendergast, Luke J.; Murphy, Gerry; Gavin, Kenneth
Publication date	2014-03-13
Publication information	Proceedings of the European Wind Energy Association 2014
Conference details	European Wind Energy Association Conference, Barcelona, Spain, 10 - 13 March, 2014
Publisher	European Wind Energy Association
Link to online version	http://www.ewea.org/annual2014/
Item record/more information	http://hdl.handle.net/10197/7289

Downloaded 2017-11-22T13:18:31Z

The UCD community has made this article openly available. Please share how this access benefits you. Your story matters! (@ucd_oa)

Some rights reserved. For more information, please see the item record link above.

DESIGN TOOLS AVAILABLE FOR MONOPILE ENGINEERING

Paul Doherty, Luke Prendergast, Gerry Murphy, Kenneth Gavin

(Geotechnical Research Group, University College Dublin, Ireland)

Abstract: Monopiles have been by far the most common support structure used for offshore turbines, with approximately 75% of existing wind farms founded on these large diameter steel tubes EWEA(2014). However, despite the widespread prevalence of monopiles across the wind sector, the design tools commonly used by industry have typically evolved from those developed by the oil and gas sector, which apply to significantly different design conditions. This paper introduces some of the design approaches available in practice and identifies some of the limitations of current offshore codes. Finite Element Methods (FEM) are suggested as a means of more accurately considering offshore soil behaviour, although the importance of accurate calibration of these models against real data (lab and/or field data) is stressed. Novel means of determining the in-situ frequency response are also discussed and the potential implications for monopile design at different sites. Finally, some design aspects of XL monopiles are considered that suggest monopiles may be pushed into ever increasing water depths.

Keywords: Foundation Design, Modelling, Dynamic Response

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 [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 of 3 to 4 MW being replaced by 5 to 8 MW turbines in future developments. While these turbines generate higher power output, they are also significantly heavier and result in higher wind loads, 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 traditional design of a laterally loaded pile as developed by the oil and gas sector and describes a comparison approach using more recent 3D FEM software that is made feasible by the increases in computing power. Furthermore, the design assumptions for determining the dynamic stability are outlined and an alternative means of calculating the resonant frequency is explored which allows the soil-structure interaction to be considered more accurately. Due to space limitations, the scope of the paper is limited to consideration of granular siliceous sand deposits prevalent in the North Sea.

Monopile Design Considerations

Traditional laterally loaded pile design assumes that the pile behaves as a vertical beam and the soil can be modelled as a series of lateral springs which are decoupled and provide mechanical resistance as an analogue to the soil continuum. This approach is commonly referred to as the Winkler p-y method. In this instance, each spring is defined by the soil reaction, p, (mobilised as a function of the displacement), and the lateral displacement y. Therefore, increased displacement typically mobilises additional resistance. The shape of the p-y curve can vary in complexity from a simple elastic response to elastic-plastic and even non-linear elastic plastic. The API and DNV design codes present a hyperbolic curve to represent the soil reaction (p) at a given depth in terms of the lateral movement (y). The hyperbolic curve is shown in Equation (1) [2].

(3) $$p = Ap_u \tanh\left(\frac{kxy}{Ap_u}y\right)$$

where pu = ultimate lateral soil resistance at depth 'x' below the surface (kN m-1), k = depth-independent initial modulus of subgrade reaction (kN m-3), A = empirical factor accounting for static or cyclic loading conditions, y = lateral deflection (m). It is worth noting that the form of this equation was developed from a series of research studies undertaken by Reese and Matlock (1970) and further developed by Murchison and O'Neill (1984) [3&4]. The basis of this equation was a suite of tests completed at Mustang Island on 760mm diameter piles on laterally loaded piles with minimal moment load contribution. Considering the range of monopile geometries being deployed in practice today it is clear that these design approaches are being extrapolated well outside their original development dataset. The validity of such an approach requires urgent investigation.

Furthermore, the entire p-y response for a given soil strata is derived from a single soil parameter - the soil internal angle of friction (φ). This parameter is used to estimate the C1, C2 and C3 parameters (See Figure 1a), which in turn are used to derive the ultimate resistance pu. The initial modulus (k), is also derived using a similar direct correlation with the friction angle (See Figure 1b).

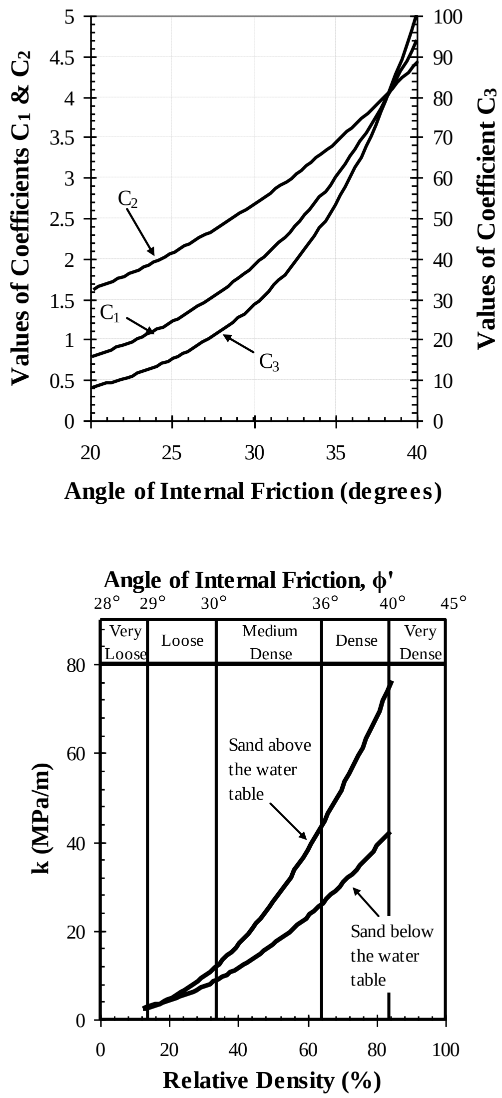

Figure 1: Determination of p-y parameters from soil friction angle (after DNV 2007)

Considering the design approach outlined above, it is clear that the method currently used in the existing design codes gives no consideration to the soil stiffness and employs a single strength parameter to derive the entire lateral response. As a result, soils of different ratios of stiffness to strength as measured using parameters such as the G0/qc ratio, where G0 is the small strain modulus and qc is the cone tip resistance during continuous penetration, will yield identical responses when predicted using API/DNV p-y methods for granular deposits.

Finite Element Methods

As an alternative to employing the API method a number of more sophisticated analysis tools are being used with increasing frequency. Finite Element Modelling methods are gaining increasing traction due to the availability of commercial software and the increased computation efficiency of modern powerful processors. In addition to pure FEM methods, an alternative immediate approach has been proposed where FEM has been used within a parametric study to derive alternative p-y curves that are better suited to the range of geometries and soil conditions mobilised for offshore wind.

This paper presents a simple PLAXIS FEM based on a full space 3D geometry of a 6m diameter monopile, installed in 30m of water depth with a wall thickness of 45mm and a pile penetration of 30m. A freeboard of 15m above mean sea-level was assumed for the interface height (allowing for access platform being above the extreme sea state). The lateral loads were then applied at a lever arm 40m above this point to ensure the ratio of horizontal to moment load was accurately simulated.

The FEM analysis was conducted using a 3D mesh containing approximately 36,500 elements. The soil elements were modelled using 10-noded tetrahedral elements and the monopiles were modelled as cylinders of 6-noded plate elements. The side boundaries were placed 100m from the centre of the monopiles and were fixed in the horizontal directions but were free to displace in the vertical direction. The bottom boundary was 100m below the sea bed level and fixed in all directions as recommended in the PLAXIS 3D Manual [5]. Interface elements were used around the pile with an interface strength reduction factor of 0.7. Interface elements are special 12-noded elements that are used to allow proper modelling of soil-structure interaction. The interfaces are applied to simulate a thin zone of intensely shearing material at the contact of the pile plate and the surrounding soil (slippage) or to simulate the structure coming away from the adjacent soil (collapse).

It is worth noting that a range of material models can be deployed to determine the pile response which allow for increasingly complex aspects of pile behaviour such as anisotropy, drainage permeability, cohesion, non-linearity strain softening/hardening, dilation and overconsolidation ratios to name a few. Due to space limitations the paper considers the Mohr-coulomb model with only fundamental inputs, however the authors strongly suggest the use of a more encompassing constitutive model such as the small strain hardening model for detailed engineering with the Mohr-Coulomb model constrained to preliminary design and feasibility studies.

The moment-rotation response of the 6m diameter pile is presented in Figure 2 where it is compared to the API/DNV response determined by implementing the standard p-y curves within a finite difference solver. This figure shows a comparable initial response between the two sets of analysis which suggests that the existing p-y method provides reasonable predictions of the piles behaviour. Furthermore, Figure 3 shows the PLAXIS predictions for a range of different friction angles (assuming the soil stiffness value, E_{50} is 120MPa). The increase in pile rotational stiffness is roughly in-line with the API/DNV response suggesting that the changing soil friction angle is well considered in the p-y method.

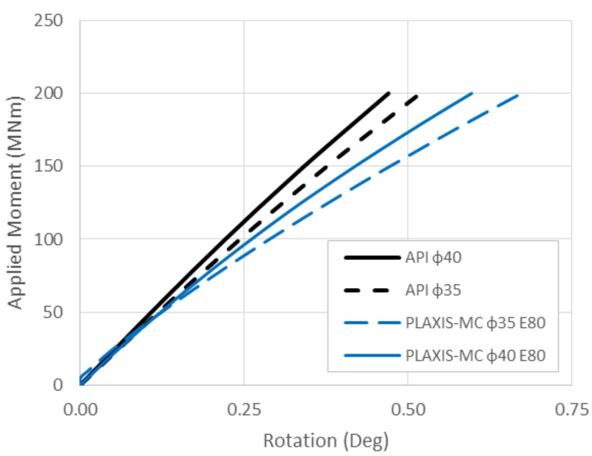

Figure 2: Moment Rotation Response of 6m Monopile using PLAXIS 3D and LPile

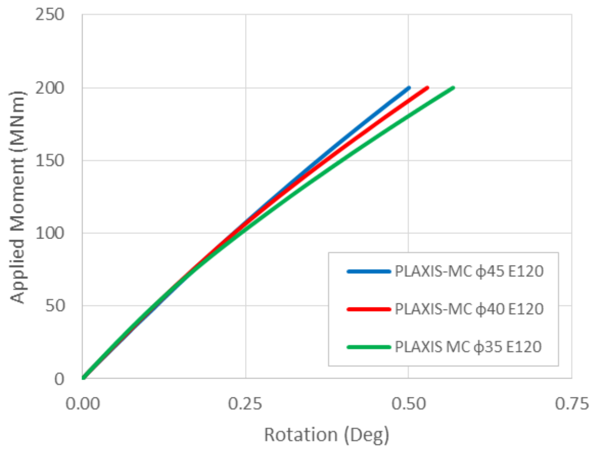

Figure 3: Moment Rotation Response of 6m Monopile at varying friction angles using PLAXIS 3D

However, the reasonable comparison of the initial pile response seen between the derived API/DNV methods and the PLAXIS analysis may be slightly coincidental when considering the range of alternative parameters that will yield different FEM responses but are not considered within the p-y framework. Figure 4 below shows the impact of varying the soil stiffness over a relatively small range, where the moment-rotational stiffness is seen also to vary. This spread of rotations cannot be accounted for using the existing standards, where soils of different E/φ values will yield the same response regardless of the initial soil stiffness (provided the failure strength is the same). Therefore, it is recommended that FE models have much better scope to accurately consider the range of soil properties likely to be encountered offshore. However to ensure these conditions are accurately represented in the FE model, a detailed calibration is required.

It is critical to ensure any FE model is representative of the in-situ conditions. To overcome the issue of selecting appropriate modelling parameters for design, a detailed calibration process is proposed. Firstly, a reasonable material model should be selected. In this instance, we will assume that the Hardening model available in PLAXIS provides the required level of complexity to realistically simulate the in-situ conditions. In the first instance, the laboratory triaxial and oedometer tests are simulated within the FE package to develop reasonable "first-pass" parameters for various stiffness values and soil strength. The second step of the calibration involves simulating the offshore CPT tests using cavity expansion methods, which will allow the soil limit pressure to be established and a simulated CPT curve to be determined.

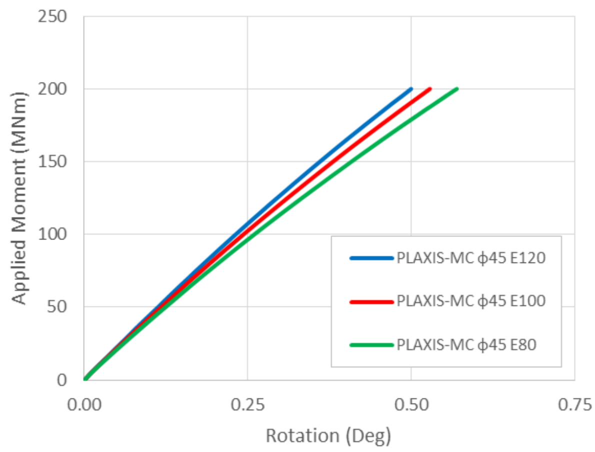

Figure 4: Moment Rotation Response of 6m Monopile at varying E_{50} using PLAXIS 3D

Proposed Calibration Methodology

More realistic modelling of the non-linear stiffness response of sand can be achieved using the Hardening Soil (HS) model is PLAXIS. The drawbacks are that the high quality laboratory data required to calibrate the model are not often available and computation time is increased significantly when dealing in particular with complex 3D problems [6].

For example, specific laboratory data is not widely (publically) available in the Irish Sea, however at the locations of the Arklow wind farms the authors have access to Cone Penetration Tests (See Figure 5). Typically, high quality CPT data is available for offshore wind developments, and this should be utilised where possible. To make the analyses as accurate as possible the soil properties used in the finite element analyses should be tested to ensure that the model is representative of the likely average conditions of real Irish Sea soils. For this process a method of performing CPT tests using the cavity expansion analogue, See Xue (2007), Lehane and Xue (2008) and

Gavin and Tolooiyan (2011) is recommended [7][8][9].

By considering the vertical equilibrium of stresses at the tip of an advancing pile Randolph et al. (1994) suggested that by expanding a cavity of finite radius, a limiting pressure (p_{limit}) is achieved which is related to the cone tip stress qc [10]:

[2] $qc = p_{limit} \times (1 + \tan \phi \tan \theta)$

where θ is the cone angle (i.e. 60o) and ϕ is the friction angle of the soil.

Xu employed the linear-elastic perfectly plastic Mohr-Coulomb (MC) and the non-linear Hardening Soil (HS) models in PLAXIS to predict the CPT end resistance and found that both soil models gave results which were closely comparable to results obtained with a closed-form solution proposed by Yu and Houlsby (1991) [11].

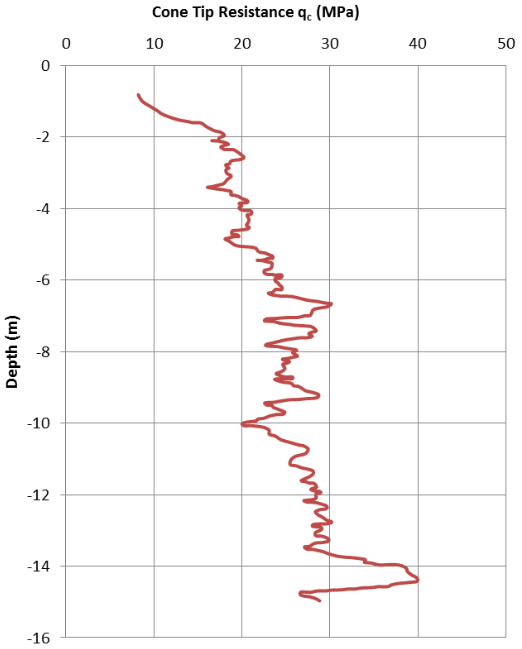

Figure 5: Example of CPT end resistance q_c profile at Arklow Bank

The MC model assumes a constant elastic stiffness (E) to represent soil displacements for stresses up to the yield stress. Salgado et al. (1997) noted that since the cavity expansion limit pressure is significantly affected by soil non-linearity, the MC model is of limited practical significance [12]. The HS model is an advanced, hyperbolic soil model formulated in the framework of hardening plasticity. The non-linear stiffness is defined by using three input stiffness parameters, E_{50} which represents the stiffness measured in a triaxial compression test when the shear stress (τ) is 50% of the maximum shear stress (τ_{max}), the triaxial unloading stiffness (E_{ur}) and a modulus derived from an oedometer test, (E_{oed}). Estimates of HS model parameters for Irish Sea sand were therefore derived to demonstrate this method of calibration.

Table 1: Parameters for Irish Sea Calibration

Dense Marine Sand	
Material Type	Drained, PLAXIS Hardening Soil Model
Unit weight, γ_{sat}	20 kN/m^3
E_{50}^{ref} *	100,000 kN/m^2
E_{oed}^{ref} *	100,000 kN/m^2
E_{ur}^{ref} *	300,000 kN/m^2
Poisson's Ratio, v	0.25
Cohesion, c'	0 kN/m^2
Constant Volume Friction Angle**, φ'_{cv}	36^o
Dilatancy angle, ψ'	15^o at sea bed level, decreasing linearly to 0^o at 50 m below the sea bed
Peak Friction Angle, φ'_{peak}	46.2^o at 5 m below the sea bed 45.2^o at 10 m below the sea bed 43.0^o at 20 m below the sea bed 40.7^o at 30 m below the sea bed 38.4^o at 40 m below the sea bed 36.0^o at 50 m below the sea bed
Lateral earth pressure coefficient, K_0	1.0
* **NOTE**: See text for definitions of these stiffness moduli.	
** **NOTE**: The constant volume friction angle, φ'_{cv}, is not an input parameter in Plaxis. Rather, φ'_{cv} is determined from the peak friction angle, φ'_{peak}, and the dilatancy angle, ψ, using the following formula: $\sin \varphi'_{cv} = \frac{\sin \varphi'_{peak} - \sin \psi}{1 - \sin \varphi'_{peak} \sin \psi}$	

Spherical cavity expansion analyses were performed to estimate p_{limit} and hence estimate q_c using Eq. 2. Axisymmetric analyses were performed using the mesh shown in Figure 6.

The left-hand boundary was the axis of symmetry, the vertical and horizontal boundaries were fixed at the base, and horizontal displacements were restrained at the right-hand boundary. The mesh was 10 m wide and 20 m high. Rather than perform cavity expansion analyses at a number of depths within the soil mesh, significant numerical efficiencies were achieved by placing a surcharge load at the top of the mesh to represent the soil above it. By varying the surcharge, uniform stress conditions in the soil sample were achieved.

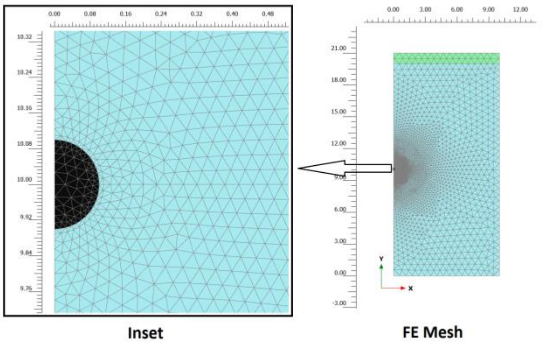

Figure 6: FEM Mesh (10 m wide and 20 m high) used in PLAXIS

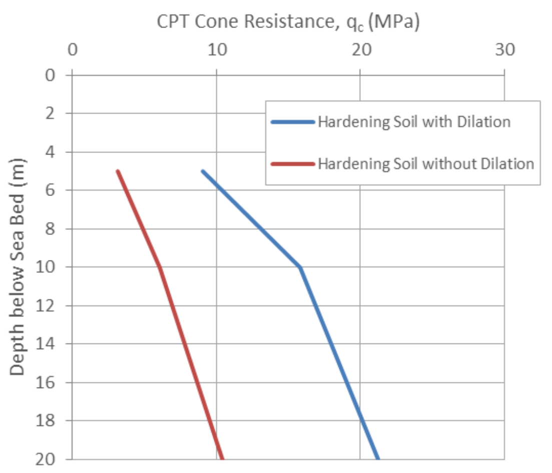

Figure 7: CPT parameters derived from PLAXIS

The importance of accurate calibration is demonstrated by considering Figure 7 above, where the impact of considering a non-linear variation in dilation with depth is required to accurately capture the medium dense to dense nature of the Irish Sea CPT profiles, whereas ignoring dilation in these simulations would significantly underestimate the soil parameters and thereby underestimate the overall moment/lateral pile stiffness. As dilation was set to zero in this basic analysis the comparison between the FE and p-y results in Figure 2 and Figure 9 are suggested to be viewed with extreme caution and also serve to illustrate that the API disregards two significant factors in stiffness and dilation.

Dynamic Analysis

While Finite Element packages such as PLAXIS are not particularly well suited to considering the soil-structure interaction of offshore monopiles under dynamic loading conditions (Due to the time domain nature of the PLAXIS analysis), there are a number of alternative methods that can be considered for dynamic design of the turbine, where excitation forces from the rotor motion and rotor-tower interaction yield the avoidance frequencies (known as the 1P and 3P frequencies respectively) for a three-bladed turbine system. The systems natural frequency is governed by the structural properties of the turbine tower and the nacelle weight, combined with the stiffness of the soil-foundation elements. An accurate estimate of the soil stiffness and the soil-structure interaction is essential to realistically model the turbine behaviour

By employing the same p-y spring methodology as described previously, the dynamic response is controlled by the initial stiffness of the API/DNV p-y curves:

(4) $$\left.\frac{\mathrm{d}\,p}{\mathrm{d}\,y}\right|_{y=0} = Ap_u \left.\frac{\dfrac{kx}{Ap_u}}{\cosh^2\left(\dfrac{kxy}{Ap_u}\right)}\right|_{y=0} = kx$$

Spring stiffness profiles are generated according to classifications given in the API design code for loose, medium dense and dense sand profiles. These correspond to relative density (Dr) values of 30%, 50% and 80%. For these Dr values, the resulting natural frequencies are shown in Table 2. However, an alternative approach is also employed which derives a CPT resistance and small strain stiffness (G0) directly from the relative density using correlations provided by Lunne

(1997) [13]. These values are then implemented within a beam coupling equation to determine a non-linear variation of stiffness with depth. These stiffness profiles are then simulated in the same dynamic beam-spring model as the API p-y stiffness to assess the impact on the natural frequency. Comparison of the results in Table 2, shows that all of the analyses considered in this study fell above a 0.22Hz P1 frequency threshold, however some difference between the API and CPT approaches were observed. The medium dense showed the closest comparison however approximately a 4% difference in frequency was observed between the API and CPT methods in lose sands relative to the threshold value, suggesting that significant cost savings in steel may be achievable in the loose sand case. Although the difference between the modelling approaches and the resulting degree of conservatism varied significantly as the soil state changed. For the dense sand case, the CPT method yielded lower frequency values that suggest a degree of non-conservatism in the dense sand condition.

Table 2: Results of Analysis on a 6m Diameter monopile in 30m water depth

Case	Analysis Type	Frequency [Hz]	% Above P1 Frequency
1	CPT Loose	0.241	9.727
2	API Loose	0.233	6.091
3	CPT Med Dense	0.245	11.182
4	API Med Dense	0.245	11.136
5	CPT Dense	0.248	12.636
6	API Dense	0.251	14.227

XL Monopile Design Feasibility?

In recent years, the trend for monopiles to be used in ever increasing water trend has suggested that these foundation solutions will be deployed for years to come. To assess the feasibility of deploying a monopile in 40m of water, a serviceability, dynamic, and fatigue limit state analysis was undertaken. Due to space restrictions, this paper will only deal with the first two failure scenarios.

The Ultimate Limit State was modelled using the 3D PLAXIS analysis described previously. The geometry was changed to consider an 8m diameter monopile with a wall thickness of 55mm, penetrating 40m into the seabed at a site with 40m water depth. This analysis was undertaken using the Mohr-coulomb constitutive model. The parameters employed were those of dense sand, with friction angle of 40 degrees and young's modulus, E_{50}, of 100MPa. The model geometry is illustrated in Figure 8 below and the moment-rotation is illustrated schematically in Figure 9 below, where the relatively stiff response is evident.

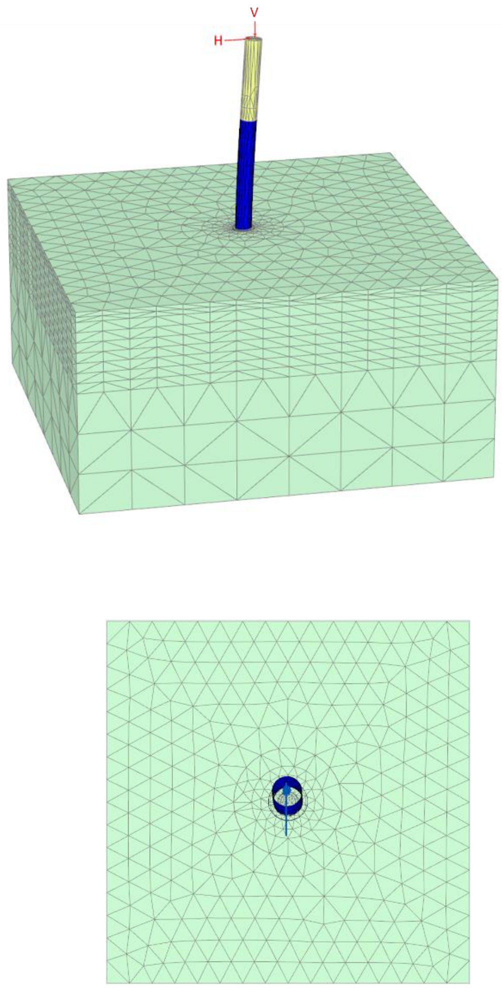

Figure 8: Model Geometry for XL Monopile Analysis

The moment-rotation response indicates that at 0.25 degrees rotation, the pile will be capable of withstanding moments up to 150 MNm and lateral loads of 3.75MN at interface level, which are in-line with the current range of turbines being considered for 40m water depth. Furthermore, while the XL monopile is shown to be feasible from a ULS perspective, it is necessary to assess the response under dynamic conditions. The dynamic model described previously was used to assess the frequency response and from this it is clear that the pile is significantly stiffer than needed to avoid the resonance 1P frequency band. This suggests that a smaller diameter pile is most likely feasible in the proposed 40m water depth or alternatively the proposed XL geometry can be pushed into even deeper waters.

Table 3: Results of Analysis on an 8m diameter monopile in 40m water depth

Case	Analysis Type	Frequency [Hz]	% Above P1 Frequency
1	CPT Loose	0.301	36.682
2	API Loose	0.293	33.227
3	CPT Med Dense	0.304	38.182
4	API Med Dense	0.304	38.318
5	CPT Dense	0.307	39.727
6	API Dense	0.312	41.727

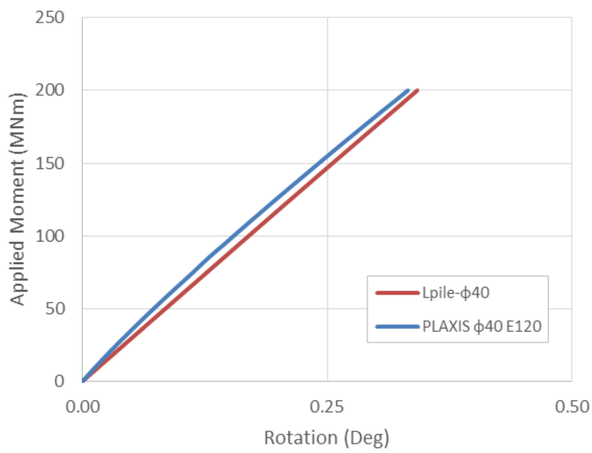

Figure 9: Moment-Rotation for XL Monopile Case

Conclusions

There are a range of design tools available to consider monopile response and the soil-structure interaction between the pile and the seabed conditions. However, this paper highlights that while employing more accurate advanced models can lead to significant cost savings in design, calibration of these models is essential. A suite of 3D finite element analysis stressed the importance of soil calibration. A CPT based calibration framework was identified as being suitable to improve the accuracy of FE based simulations. Similarly, a CPT based approach for dynamic analysis of the frequency behaviour was also proposed instead of the traditional API/DNV p-y stiffness assumptions. In both cases the analysis showed that the soil-structure response varied with soil state and the parameters employed in the analysis. In the final stage of the paper a ULS and Dynamic design check were completed for an 8m diameter monopile and the results suggested that such XL geometries may allow monopiles to be deployed in increasing water depths.

References

[1] EWEA. (2014). The European offshore wind industry - key 2013 trends and statistics - 2013, European Wind Energy Association. Europe.

[2] API RP-2A WSD, 2007

[3] Matlock, H (1970) Corellations for design f laterall loaded piles in soft clay. *Proceedings of the Offshore technology Conference, Houstom, TX* (pp.577-594)

[4] Murchinson, J., & O'Neill, M. (1984). Evaluation of p-y relationships in cohesionless soil: analysis and design of pile foundations. In Proceedings of Symposium in Conjunction with the ASCE National Convention, San Francisco, CA (pp. 174–191). ASCE Technical Council on Codes and Standards, New York.

[5] Plaxis 3D 2012 Manual, Brinkgreve R. B. J., Engin E. and Swolfs W. M. (edt), 2012.

[6] Tolooiyan, A., and Gavin, K., 2012. Numerical Analyses of the base resistance of deep footings in sand, ICE Geotechnical Engineering

[7] Xu, X. (2007). Investigation of the end bearing performance of displacement piles in sand. PhD thesis, The University of Western Australia.

[8] Xu, X. & Lehane, B. M. (2008). Pile and penetrometer end bearing resistance in two-

layered soil profiles. Geotechnique, 58, No. 3, p.187–197

[9] Tolooiyan, A., & Gavin, K. (2010). Finite element analysis of the CPT for design of bored piles. In 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA, USA, May 2010.

[10] Randolph, M. F., Dolwin, J. & Beck, R. (1994). Gebtechnique 44, No. 3, 427-448

[11] Yu, H S & Houlsby, G. T. (1991). Geotechnique41, No.2, 173-183.

[12] Salgado, R., Mitchell, J. K., and Jamiolkowski, M. (1997a). "Cavity Expansion and Penetration Resistance in Sand." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 123(4), 344-354.

[13] Lunne, T., Robertson, P. K., & Powell, J. J. M. (1997). Cone penetration testing in geotechnical practice. Test. Taylor & Francis