Monopile soil-structure interaction for estimating the dynamic response of an offshore wind turbine

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ABSTRACT: There has been significant reductions in the cost of developing Offshore Wind Turbines (OWTs) over the past few years. In favourable locations - with high wind speeds, shallow water depth <35m and stiff soils – the cost of developing OWTs is now competitive with traditional forms of fossil fuel electricity generation. In order to ensure offshore wind remains competitive at less favourable sites, further research is required to optimise the structural design and lifetime of the structure. The largest uncertainty with respect to modelling the dynamic response of an OWT typically relates to the geotechnical design. Accurate modelling of geotechnical behaviour and soil-structure interaction (SSI) of an OWT is essential to providing an optimised design. This paper compares a state of the art SSI approach utilising advanced 3D Finite Element (FE) modelling with traditional soil spring p-y approach and assess the impact of modelling approach on the foundation stiffness and natural frequency of the OWT structure.

KEY WORDS: Monopiles; Offshore Wind; Soil-Structure Interaction.

1 INTRODUCTION

Offshore Wind Turbines (OWTs) are most commonly supported on Monopile foundations, which account for more than 80% of all OWT substructures installed in Europe to date. Monopiles are single large diameter (up to 10m) open-ended tubular steel piles, usually driven into the sea bed, which rely on the stiffness and strength of the surrounding soil to provide resistance against large environmental loads. OWT's founded on monopiles are typically designed as "soft-stiff" structures, where the fundamental natural frequency of the structure lies between the excitation frequencies corresponding to one full revolution of the turbine (1P) and of each turbine blade passing the tower (3P) [1]. In most practical situations, considerations of the dynamic response and natural frequency of the OWT structure will govern the monopile diameter [2].

The industry standard approach for the geotechnical design of monopiles in Europe are those recommended by Det Norske Veritas - Germanischer Lloyd (DNV-GL) [3], which are based on American Petroleum Institute (API) design guidelines [4] originally intended for oil & gas jacket piles. Both methods use a decoupled Winkler beam approach [5] where the lateral soil reaction is described by non-linear 'p-y' springs. The methods were calibrated using a limited number of pile tests performed on slender jacket piles with diameters less than 1m, and are now recognized as being unsuitable for predicting the response of large diameter monopiles [3]. Kallehave et al. [6] presented Nacelle measurements from OWT's at the Walney offshore wind farm to show the measured fundamental frequencies were significantly higher (≈5 – 7%) than best-estimate predictions using the traditional API/DNV p-y approaches in sand. The difference between the measured and calculated natural frequency was attributed to the DNV method underestimating the soil-stiffness, particularly at low strain levels which are relevant for fatigue loading on large diameter monopiles. In recent years, a number of modifications to the DNV approach have been proposed and are now being used in monopile design practice (Kallehave et al. 2012 [6] , Kirsch et al. 2014 [7] and others). Recent updates to the DNV-GL guidelines [3] have added text suggesting 3D Finite Element analysis should be used to verify the monopile response, although no guidance on how this should be performed is provided.

Recent research ([2], [8], [9] and [10]) has attempted to address some of these issues by validating FE modelling approaches against high quality large scale field testing, and there is significant research effort in this area currently ongoing. This paper compares the dynamic response of an OWT structure where the below ground pile-soil behaviour was modelled using (i) the conventional DNV 'p-y' approach and (ii) an advanced in-situ calibrated 3D FE geotechnical design approach. The difference in stiffness between the two approaches is compared and the outputs were used in separate dynamic wind turbine models to assess the effect on the overall structures fundamental natural frequency.

2 MODELLING

2.1 Site Location, Pile Geometry and Loading Conditions

Soil profiles from an offshore wind development zone in the Netherlands, referred to as Holland Kust (Zuid), was chosen due to the public availability of the soil profile information and geotechnical testing reports. Following a careful review, the HKZ2_BH03 location was deemed to be representative of a typical North Sea medium dense to dense sand site. The Cone Penetration Test (CPT) profile from the site is shown in Figure 1 below. All soil parameters required for both the DNV/API approach and the 3D FE modelling were derived from the CPT profiles following the method defined by [2]. Based on the mean sea level at the location a water depth of 30m was selected for analysis.

For the purposes of modelling the below mudline pile response, an initial approximate monopile geometry for the site was estimated as 7m in diameter, 28m embedment with a constant 70mm wall thickness below mudline. The geometry was chosen based on authors experience of designing of monopiles at similar locations. Above the mudline, the monopile and wind-turbine geometry were developed based on a 5MW reference turbine as defined by the NREL (Jonkman et al. [11]). The tower properties were modified (stiffened) to account for the 30m water depth as compared with the 20m water depth defined in [11]. The above mudline monopile and tower properties are shown in Figure 3.

For this initial study, a fatigue damage equivalent horizontal load, H, of 4000kN was estimated (based on the authors experience) to act at an eccentricity, e, of 40m above mudline. It should be noted that this load is much lower than the ultimate limit state (ULS) extreme load which may be experienced by the monopile, but is deemed representative of typical operational loads acting over the lifetime of the structure.

2.2 Monopile Finite Element Modelling

The 3D FE modelling was undertaken using a static analysis in Plaxis 3D 2017 software. The pile was modelled in half space to reduce computation time (Figure 2). After a sensitivity analysis to optimize the model geometry, the boundaries were set at ±60m in the y-axis (direction of loading), 0 to +30m in the x-axis and +40m to -50m in the z-axis (with mudline defined at +0m). The soil elements were modelled as ten-node tetrahedral elements. The pile wall was modelled using six-node plate elements with interface elements added to allow a reduction in interface shear strength by a factor, R_{inter}, of 0.7. The pile plates were modelled as linear elastic elements with a Young's Modulus of 210 GPa, Poisson's ratio of 0.3 and a unit weight of 77 kN/m^3. The Hardening Soil with small strain stiffness model (HSS) as defined by Schanz [12] was used to define the soil response. All the required soil parameters for this model were derived from the CPT profile following the procedure defined in [2], which has been validated against a suite of monopile field tests. Once the 3D FE analysis was successfully completed, the soil reactions were extracted using the procedure described in [2]. A summary of the soil input parameters is provided in Table 1.

For comparison with the 3D FE, the DNV 'p-y' approach was also considered (for brevity, details of the formulation of DNV approach will not be discussed in this paper). To summarize the approach taken, the DNV cyclic p-y curves were applied in a Winkler beam model using a static solver developed in Matlab (similar to the commonly used commercial LPile software). Under the applied fatigue loading (H=4000kN), the displacement profile and secant stiffness of each non-linear p-y soil spring was then output from the monopile model for comparison with the 3D FE approach.

2.3 Estimation of natural frequency

For this study, a simplified approach, proposed by Arany et al. [13], was used for estimating the first natural frequency of the OWT structure. The approach is based on the simple cantilever beam formula, and applies modifying coefficients to take account of the foundation flexibility. The first natural frequency of the OWT structure, f_0, can be calculated as:

$$f_0 = C_L C_R C_S f_{FB} \tag{1}$$

where C_L and C_R are the lateral and rotational flexibility coefficients and C_s is the structural flexibility coefficient and f_{FB} is the fixed base cantilever natural frequency of the tower. In order to calculate C_L and C_R it is necessary to derive the pile head stiffness (K) matrix at mudline as shown below:

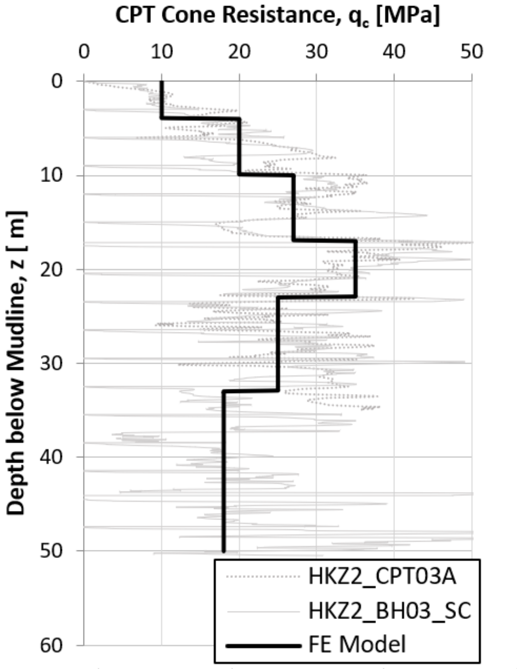

Figure 1: CPT Profile from HKZ2_BH03

Table 1: Summary of Plaxis soil input parameters

Top Depth [m]	$E_{50,ref}$ [kPa]	$E_{oed,ref}$ [kPa]	$E_{ur,ref}$ [kPa]	m [-]	ϕ' [°]	ψ' [°]
0	93246	58253	279738	0.5	43.2	14.0
4	125868	77228	377604	0.5	44.1	15.1
7	146607	91462	439821	0.5	43.2	14.1
10	169413	104912	508239	0.5	43.6	14.5
17	191225	117960	573676	0.5	43.8	14.7
23	223015	145851	669044	0.5	40.8	11.0
33	264797	185994	794390	0.5	37.1	6.3

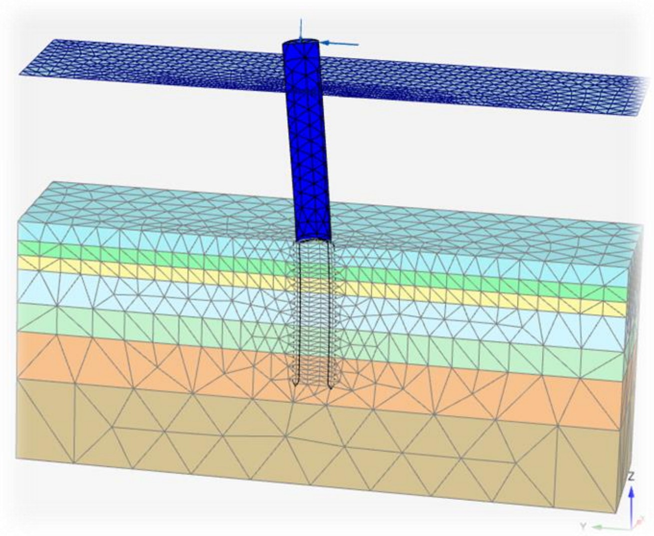

Figure 2: Plaxis 3D model mesh and geometry (deformations scaled x20)

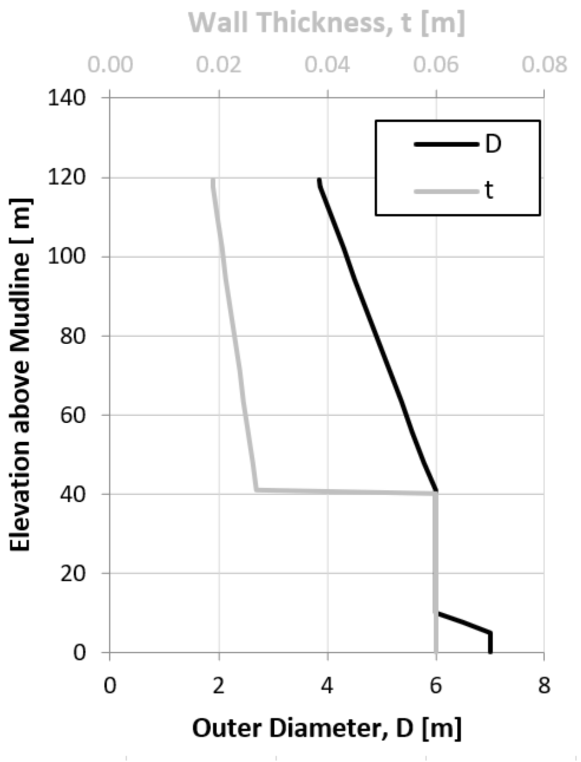

Figure 3: Above mudline pile diameter and wall thickness

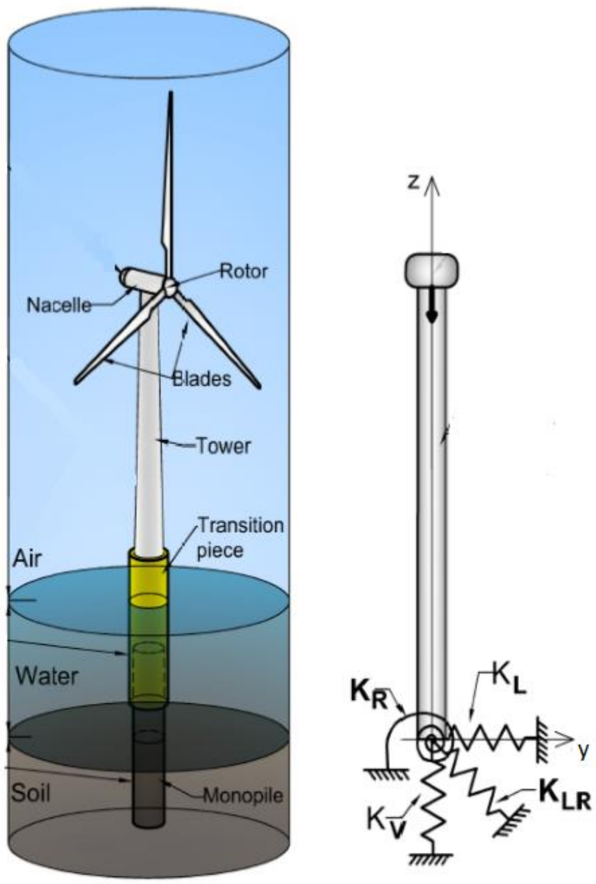

Figure 4: Monopile Foundation Stiffness Coefficients (after Arany et al. [13])

$$\begin{bmatrix} F_y \\ M_x \end{bmatrix} = \begin{bmatrix} K_L & K_{LR} \\ K_{LR} & K_R \end{bmatrix} \begin{bmatrix} \rho \\ \theta \end{bmatrix} \tag{2}$$

where F_y and M_x are the lateral force and overturning moment at mudline, K_L is the lateral spring, K_R is the rotational spring and K_{LR} is the cross coupling spring, ρ is the displacement and θ is the rotation at mudline.

The foundation stiffness matrix is calculated from the 1D Winkler beam model of the foundation. The K_L term is calculated by fixing the pile head rotation, $\theta = 0$, and applying the lateral force. K_L can then be calculated as:

$$K_L = \frac{F_y}{\rho} \tag{3}$$

The K_{LR} cross coupling term can then be calculated by diving the moment required to fix the pile head rotation, $M_{\theta=0}$, by the displacement.

$$K_{LR} = \frac{-M_{\theta=0}}{\rho} \tag{4}$$

The K_R term is calculated by fixing the pile head deflection, $\rho = 0$, and applying the overturning moment. K_R can then be calculated as:

$$K_R = \frac{M_x}{\theta} \tag{5}$$

Once the K matrix is derived, the flexibility coefficients C_L, C_R and C_S can be calculated based on the geometry and mass of the structure. The exact details of the calculation procedure can be found in Arany et al. [13].

3 RESULTS

The displacement and linearized stiffness profiles, output from the 3D FE and DNV p-y static loading analyses are shown in Figure 5 and Figure 6. It is evident that the 3D FE provides a stiffer pile response, with the 3D FE analysis shows 30% lower displacement under the applied load of 4000 kN. The secant stiffness values from the 3D FE is significantly higher than the DNV model near the pile toe, due in part to the 3D FE models ability to capture the pile toe shear response.

The pile head stiffness matrix components derived from the separate analyses are provided in Table 2. It is evident that the lateral spring K_L is 64% stiffer and the rotational spring K_R is 30% stiffer from the 3D FE analysis compared with the DNV p-y approach.

Table 2. Pile Head Stiffness Matrix Components.

Spring	3D FE	DNV	% Diff
K_L	4.57E+06	2.79E+06	64%
K_{LR}	-2.99E+07	-2.10E+07	42%
K_R	3.64E+08	2.80E+08	30%

Calculations of the natural frequency of the OWT system using the above stiffness matrix components suggest a 4.4% increase in first natural frequency using values from the 3D FE approach (0.226 Hz) when compared with the DNV p-y approach (0.217 Hz).

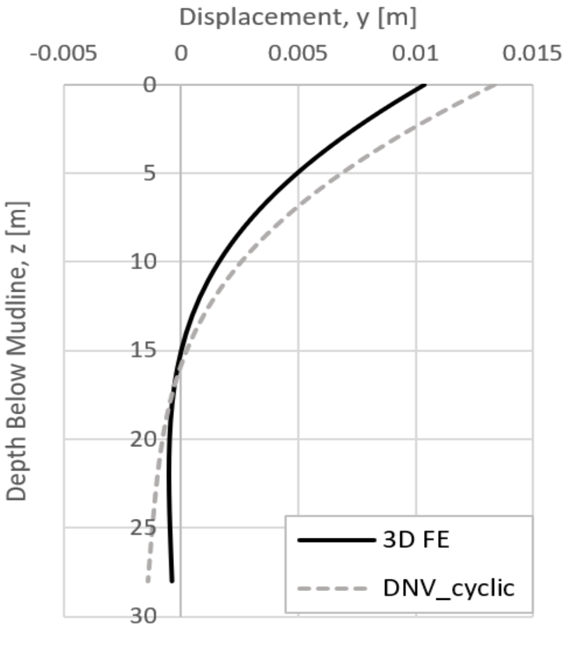

Figure 5: Displacement Profile from Static Monopile Models

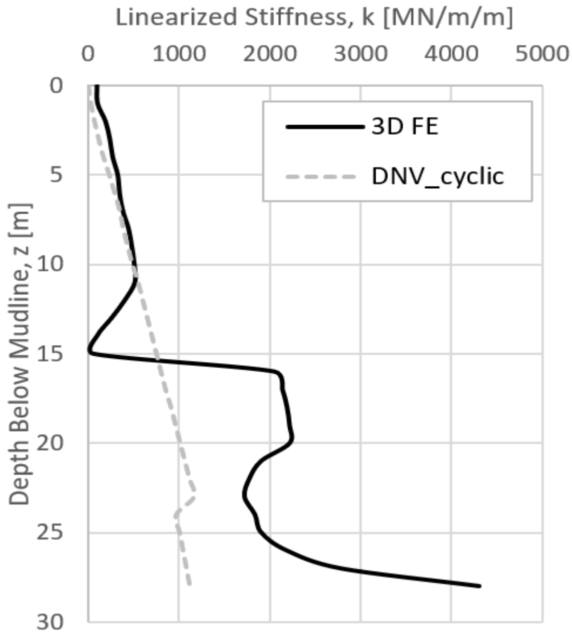

Figure 6: Linearized Spring Stiffness from Static Monopile Models

4 CONCLUSIONS / FUTURE WORK

The results of this study show that using a state of the art 3D FE approach to model the soil-structure interaction of a monopile can provide a notably stiffer response when compared with a traditional DNV p-y model approach. The 3D FE analysis demonstrated reduced displacements and a higher natural frequency response of the OWT structure. Even a small increase in natural frequency can result in significantly reduced structural fatigue over the lifetime of the structure. Therefore it is imperative to accurately model the monopile response in order to optimise the foundation and tower for the OWT.

Further research is ongoing at the Trinity College Dublin offshore research group to optimise the foundation and tower design of OWTs. Work is currently underway to improve the accuracy of the natural frequency calculations using a dynamic modelling approach where the wind turbine structure will be modelled using a multibody approach following Kane's method [14].

REFERENCES

[1] M. Arshad and B. C. O'Kelly, "Analysis and design of monopile foundations for offshore wind-turbine structures," *Mar. Georesources Geotechnol.*, vol. 34, pp. 504–525, 2016.

[2] G. Murphy, D. Igoe, P. Doherty, and K. G. Gavin, "3D FEM approach for laterally loaded monopile design," *Comput. Geotech.*, vol. 100, pp. 76–83, 2018.

[3] DNV-GL, "DNVGL-ST-0126 : Support structures for wind turbines," 2016.

[4] API, "Recommended Practice for Planning , Designing and Constructing Fixed Offshore Platforms — Working Stress Design," *Api Recomm. Pract.*, vol. 24–WSD, no. December 2000, p. 242, 2007.

[5] E. Winkler, “Theory of elasticity and strength,” in *Dominicus Prague*, 1867.
[6] D. Kallehave, C. LeBlanc, and M. Liingaard, “Modification of the API Py Formulation of Initial Stiffness of Sand,” in *Offshore Site Investigation and Geotechnics: Integrated Technologies - Present and Future*, 2012, pp. 465–472.
[7] F. Kirsch, T. Richter, and M. Coronel, “Geotechnische Aspekte bei der Gründungsbemessung von Offshore-Windenergieanlagen auf Monopfählen mit sehr großen Durchmessern,” *Stahlbau Spez. 2014 – Erneuerbare Energien*, vol. 83, no. SUPPL. 2, pp. 61–67, 2014.
[8] B. W. Byrne *et al.*, “New design methods for large diameter piles under lateral loading for offshore wind applications,” in *Frontiers in Offshore Geotechnics III - Proceedings of the 3rd International Symposium on Frontiers in Offshore Geotechnics, ISFOG 2015*, 2015.
[9] B. W. Byrne *et al.*, “Field testing of large diameter piles under lateral loading for offshore wind applications,” in *Geotechnical Engineering for Infrastructure and Development - Proceedings of the XVI European Conference on Soil Mechanics and Geotechnical Engineering, ECSMGE 2015*, 2015, vol. 3.
[10] B. W. Byrne *et al.*, “PISA: New Design Methods for Offshore Wind Turbine Monopiles (Keynote),” in *International Conference on Offshore Site Investigations and Geotechnics*, 2017.
[11] J. Jonkman, S. Butterfield, W. Musial, and G. Scott, “Definition of a 5-MW Reference Wind Turbine for Offshore System Development,” no. February, 2009.
[12] T. Schanz, a Vermeer, and P. Bonnier, “The hardening soil model: formulation and verification,” *Beyond 2000 Comput. Geotech. 10 years PLAXIS Int. Proc. Int. Symp. beyond 2000 Comput. Geotech. Amsterdam Netherlands 1820 March 1999*, p. 281, 1999.
[13] L. Arany, S. Bhattacharya, J. Macdonald, and S. J. Hogan, “Design of monopiles for offshore wind turbines in 10 steps,” *Soil Dyn. Earthq. Eng.*, 2017.
[14] Kane, Thomas R., and David A. Levinson. Dynamics: Theory and Applications. New York: McGraw-Hill, 1985.