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A comparison of soil lateral reaction models for monopile design in clay

Comparaison de modèles de réaction latérale pour le dimensionnement de monopieux dans l'argile

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ABSTRACT: In recent years there has been an increase in the size of offshore wind turbines along with the water depth in which they are being installed. As a result, the loads are increasing and geotechnical engineers need accurate models to deliver safe and cost-effective foundations. Monopile foundations are the most popular foundation type, covering about 80% of the installed offshore wind turbines in Europe to date. Monopiles mobilise lateral soil reactions to withstand the large environmental loads carried by the wind turbine. This paper presents a comparison of the main approaches used in the industry (API, PISA, 3D FE). Their performances are compared for an example of monopile design in a range of homogenous clay profiles. Finally, a refinement of the current industry design approach is proposed to improve accuracy of the modelling.

RÉSUMÉ : Ces dernières années, la taille des éoliennes offshore et la profondeur des eaux dans lesquelles elles sont installées ont augmenté. En conséquence, les charges augmentent et les ingénieurs ont plus que jamais besoin de modèles précis pour dimensionner des fondations sûres et optimisées. Les fondations monopieux sont le type de fondation le plus populaire, couvrant environ 80 % des éoliennes offshore installées en Europe à ce jour. Les monopieux mobilisent les réactions latérales du sol pour résister aux importantes charges environnementales exercées sur l'éolienne. Cet article présente une comparaison des principales approches utilisées dans l'industrie (API, PISA, 3D FE). Leurs performances sont comparées pour le dimensionnement d'un monopieu dans différents sols argileux. Enfin, des pistes d'améliorations sont proposées.

KEYWORDS: offshore, wind turbine, monopile, geotechnical, lateral design.

1 INTRODUCTION.

Monopiles are large diameter open ended steel pipe driven into the seabed. During the first years of the offshore wind energy industry, monopiles were designed with a diameter of about 4 m. As the industry is maturing, diameters up to 10 m are being designed, and up to 12 m are expected in the future. Monopiles are relatively short with slenderness ratios (*L/D*, where *L* is the embedded length and *D* the diameter) historically lower than 6, and now typically around 3. One of the main design drivers is to avoid resonance of the whole structure with the forcing frequencies. Monopiles are typically designed in the soft-stiff domain where the first mode of vibration must fit between the rotor frequency (*1P*) and the blade passing frequency (*3P* for a three bladed turbine) as shown on Figure 1. Excessive conservatism in the design approaches may not only lead to uneconomical design but may also make the foundation stiffer than expected. There is a risk that the natural frequency of the structure will coincide with the blade passing frequency leading to excessive deflections and fatigue. Hence, geotechnical engineers need an accurate design approach to correctly size monopiles supporting offshore wind turbine.

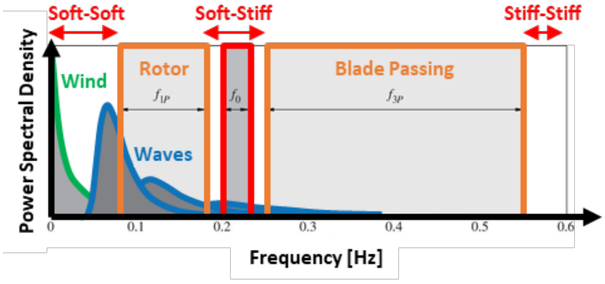

Figure 1. Natural frequency criterion for monopile design (modified after Kallehave et al. 2015).

2 MODELLING SOIL LATERAL REACTIONS

This section briefly presents the main approaches to model soil lateral reactions used in the offshore wind industry for monopile design.

2.1 *Traditional API 'p-y' approach*

The traditional industry approach consists in modelling the embedded part of the monopile using discrete Euler-Bernoulli beam element as shown on Figure 2. Soil lateral reactions are modelled as a series of independent non-linear springs. Theses curves, called 'p-y' curves, give the lateral reaction force *p* pushing against the pile as a result of the pile lateral displacement *y*. This approach is directly taken from the oil and gas industry (API 2014) and is recommended in the main offshore wind standards (DNV 2014).

The shape of the 'p-y' curves is provided in the standards. For example, in soft clay the API (API 2014) recommends curves based on the work of Matlock (1970) as below:

$$\frac{p}{p_u} = \frac{1}{2}\left(\frac{y}{y_c}\right)^{\frac{1}{3}} \leq 1 \tag{1}$$

Where p_u is the ultimate lateral reaction calculated from the undrained shear strength (s_u), the depth (z), the effective unit weight (γ'), the pile diameter (D) and an empirical dimensionless constant (J) in the range of 0.25 to 0.5 as per:

$$p_u = \left(3s_u + z\gamma'\right)D + Js_u z \leq 9s_u D \tag{2}$$

And y_c is the reference displacement at which 50% of the ultimate capacity is mobilized. It is directly scaled from the strain at one-half of the maximum stress in undrained tri-axial compression test (ε_{50}) as per:

$$y_c = 2.5\varepsilon_{50}D \tag{3}$$

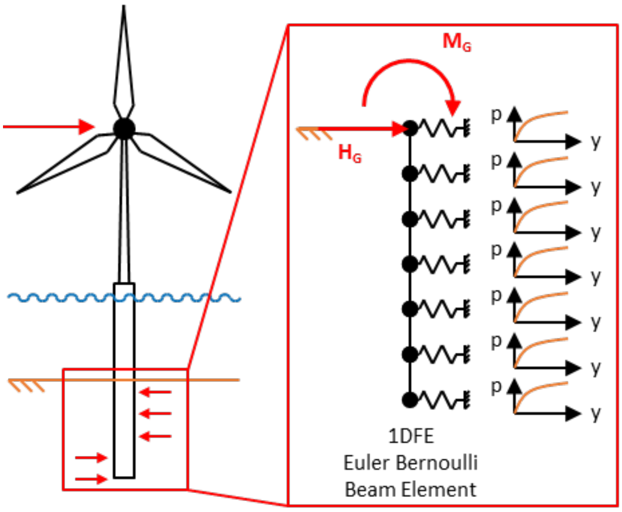

Figure 2. Representation of the API ‘p-y’ approach.

The API ‘p-y’ approach has been successfully used in the oil and gas industry for decades and was used at the early stage of the offshore wind industry. However, it is now widely acknowledged that this approach is unsuitable for monopile design due to fundamental differences between the two industries. It has only been validated against a small database of field tests on long and slender piles with diameter up to about 1 m. In contrast, monopiles are short with diameter larger than 6 m, up to 10 m. DNV-ST-0126 clause 7.6.2.6 now recommends validating the use of p-y curves for monopiles by means of finite element analysis (DNV 2018).

2.2 *State of the art PISA approach*

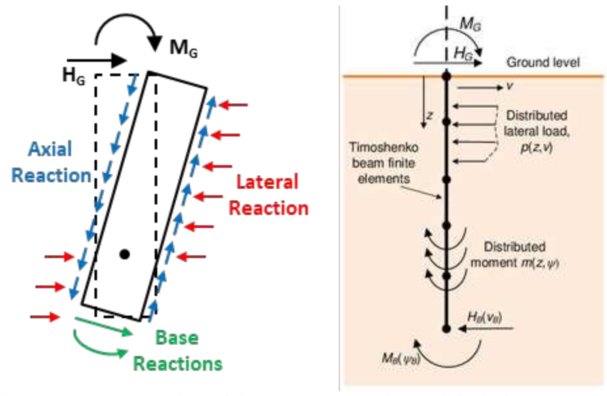

Figure 3. Representation of the PISA approach (modified after Byrne et al. 2019).

Due to the shortcomings of the API ‘p-y’ approach, the recently completed PISA project aimed at developing state-of-the-art design methodology for monopiles. One of the key differences is the addition of other soil reaction components such as distributed moment, base shear and base moment (see Figure 3). Upon lateral loading, monopiles not only mobilise soil lateral reactions but also axial reactions as a result of their rotation due to their large diameter. Also, monopiles typically behave rigidly (due to their low slenderness ratio) and show significant toe displacement mobilising base shear force and base resisting moment. Similarly to API ‘p-y’ approach, these four soil reaction components are integrated into 1D finite element. However, PISA investigators preferred Timoshenko beam element type over Euler-Bernoulli in order to take into account shear deformations. Byrne et al. (2015) compared monopile load-displacement curves at mudline obtained from 3D and 1D finite element modelling. The response considering ‘p-y’ curves only was found to be significantly softer. The principal investigators of the PISA project showed that adding these additional soil reactions makes the response stiffer and in better agreement with 3D FE. It was also shown that their contributions become more significant as pile diameter increases.

In the PISA framework, the soil reaction curves are normalised. Figure 4 shows a ‘p-y’ curve as an example. The lateral reaction *p* is normalised over the pile diameter (*D*) and the undrained shear strength (s_u) while the displacement *y* is normalised by the ratio of shear modulus at small strain (G_0) over pile diameter and undrained shear strength. Then, the curves are parameterised according to a conic function (see Eq. 4) with 4 parameters (x_u, k, n, y_u) to be fitted. Each of them relates to a particular aspect of the curve as shown on Figure 4.

$$-n\left(\frac{\bar{y}}{y_u}-\frac{\bar{x}}{x_u}\right)^2+(1-n)\left(\frac{\bar{y}}{y_u}-\frac{\bar{x}k}{y_u}\right)\left(\frac{\bar{y}}{y_u}-1\right)=0 \qquad (4)$$

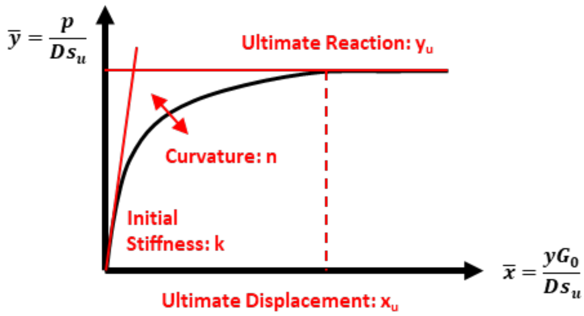

Figure 4. Example of PISA normalisation and parameterisation.

The PISA project developed two approaches: the ‘rule-based’ approach and the ‘numerical-based’ approach. These are discussed in the following sections.

2.2.1 *PISA rule-based approach*

In the PISA rule-based approach, generic depth variation functions give profiles of these 4 parameters with depth for each soil reaction component, giving a total of 16 depth variation functions. Table 1 reports the depth variation functions that were calibrated for the Cowden till for a wide range of pile geometry examined during the PISA project. At the monopile concept design stage, these could be used in any similar clay profile. Only a limited number of soil input parameters are required (s_u and G_0) to de-normalise the soil reaction curves. Monopile mudline response under any loads can then be quickly estimated in a 1D finite element solver for a range of pile geometries.

Table 1. Depth variation functions calibrated in Cowden till (Byrne et al. 2019).

Soil Reaction Component	Parameter	Depth Variation Function
Distributed lateral load, p	x_u	241.4
	k	10.60 - 1.650 * Z/D
	n	0.9390 - 0.03345 * Z/D
	y_u	10.70 - 7.101 * exp(-0.3085 * Z/D)
Distributed moment, m	x_u	Given by y_u/k
	k	1.420 - 0.09643 * Z/D
	n	0
	y_u	0.2899 - 0.04775 * Z/D
Base shear, H_b	x_u	235.7
	k	2.717 - 0.3575 * L/D
	n	0.8793 - 0.03150 * L/D
	y_u	0.4038 + 0.04812 * L/D
Base Moment, M_b	x_u	173.1
	k	0.2146 - 0.002132 * L/D
	n	1.079 - 0.1087 * L/D
	y_u	0.8192 - 0.08588 * L/D

2.2.1 *PISA numerical-based approach*

The PISA framework also offers the possibility to develop site specific soil reaction curves. This approach involves running numerous advanced 3D finite element models for a range of pile geometries. For each geometry, the soil rection curves are extracted, normalised and parameterised, giving site-specific fitting parameters (x_u, k, n, y_u). The 16 depth variation functions are then fitted to the site-specific fitting parameters. Figure 5 shows an example of such fitting for the normalized ultimate lateral distributed load in Cowden till (Byrne et al. 2019). Dots show fitting parameters from different pile geometries while the lines show successive fitting attempts.

This is not a straight-forward process as most 3D FE software will not provide soil reaction curves directly. This approach is implemented in the commercially available Plaxis Monopile Designer software (Panagoulias et al. 2021), which can automatically extract the soil reaction curves and fit the depth variation functions.

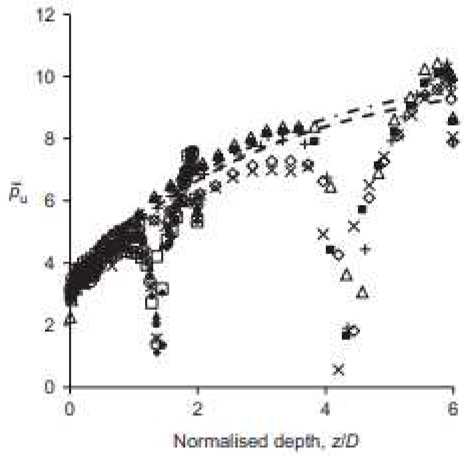

Figure 5. Example of depth variation function fitted to normalised ultimate lateral distributed load in Cowden till (Byrne et al. 2019).

2.2 *Reference 3D finite element modelling*

A full 3D finite element modelling comes at expensive computation cost but is a more rigorous approach to model the pile-soil interaction. The accuracy of the 3D FE analysis is dependent on choice of constitutive soil model and the availability of high-quality ground investigation data to define the required input parameters (as it is with the PISA numerical approach). The constitutive model should be able to capture the nonlinear stress-strain behaviour of the soil.

Figure 6 shows a typical monopile model using the commercial finite element package Plaxis 3D. The monopile is modelled using shell elements with elastic properties of structural steel (i.e. $E = 210$ GPa, $\nu = 0.3$). Interface elements are introduced between pile and soil to allow for differential displacements. To obtain the required overturning moment M at mudline, the lateral force H is applied at an eccentricity e=M/H above mudline. Taking advantage of the plan of symmetry, the model is set up in half-space in order to reduce computation time.

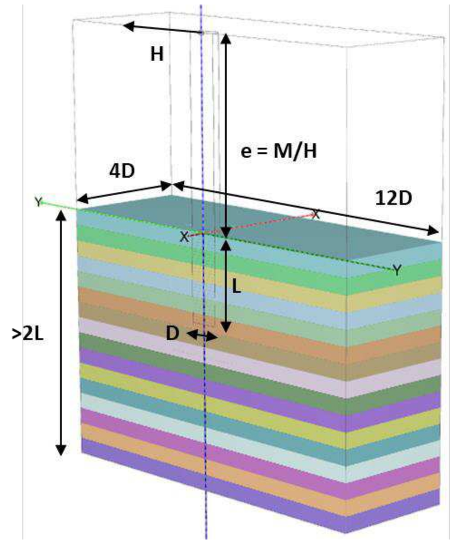

Figure 6. Model set up in Plaxis 3D.

3 APPLICATION TO MONOPILE DESIGN

In this section, the different soil lateral reaction models are compared for an example monopile design.

3.1 *Inputs for monopile design*

For simplicity, a number of input parameters for this monopile design exercise are assumed based on experience and engineering judgement (see Figure 7). A Serviceability Limit State (SLS) lateral load of 10 MN applied 60 m above the seafloor is deemed representative of a large capacity offshore wind turbine installed in typical water depth. The pile diameter, *D*, is assumed at 9 m with a constant wall thickness, *t*, of 90 mm ($D/t = 100$). This optimised geometry should be based on natural frequency assessment and to avoid fatigue or resonance issues but these assessments are omitted here for the sake of conciseness. Only the pile embedded length is considered for optimisation. A range of synthetic homogeneous clay profiles are considered. Soil effective unit weight and coefficient of earth pressure are kept constant with depth at value of 10 kN/m^3 and 1, respectively. A total of three profiles are considered here with constant undrained shear strength, s_u, with depth as shown in Figure 8. Although a constant s_u profile with depth is unrealistic, this is chosen for simplicity, as the aim of this study is to compare different methods. The stiffness to strength ratios (G_0/s_u) are taken as 800 for all three profiles.

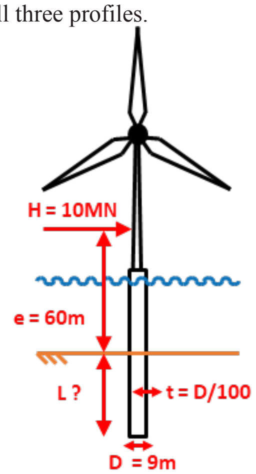

Figure 7. Inputs for example monopile design.

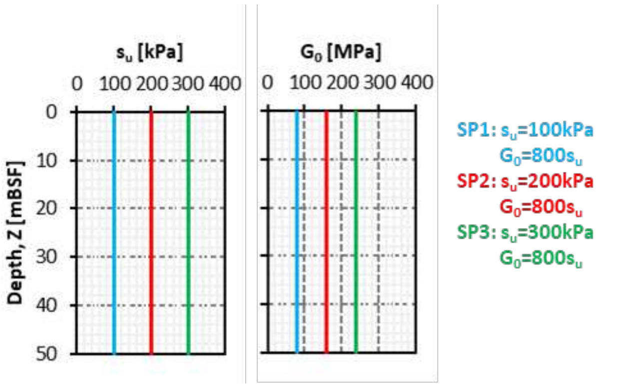

Figure 8. Range of undrained shear strength and small strain shear modulus profiles considered.

3.2 *Implementation of the different approaches*

The API 'p-y' approach and the PISA rule-based approach are implemented in a MATLAB 1D finite element solver. For the API approach, 'p-y' curves are computed as per API (2014) and the work of Matlock (1970) for soft clay (s_u < 96 kPa) and as per Reese et al. (1975) for stiffer clay. For the PISA rule-based approach, the depth variation functions calibrated in Cowden till (Byrne et al. 2019) are considered.

The implementation of the PISA numerical-based approach was undertaken in PLAXIS Monopile Designer V21. For each soil profile, the depth variation functions are fitted based on eight calibrating 3D FE models where the pile length range from 20 m to 55 m. Pile diameter, wall thickness and load eccentricity are kept constant as assumed in the previous section.

For the 3D finite element modelling, the soil is modelled using the NGI-ADP model for clay in Plaxis 3D CE V21. The missing parameters are correlated with s_u and G_0 according to the recommendations made by Panagoulias et al. (2021). These correlations and the use of NGI-ADP to model monopiles in clay was validated against PISA field tests in Cowden till (Minga & Burd 2019).

3.3 *Design pile embedded length*

Based on each approach considered, the monopile rotation at sea floor under the assumed SLS load was calculated for a range pile embedded length (see Figure 9 for an example in SP1). The SLS-GEO lateral check typically requires limiting the permanent accumulated rotation at mudline due to cyclic loading to 0.25 degree (DNV 2018). For the sake of simplicity, the 0.25 degree criterion is directly applied to the static rotation here. This is deemed acceptable here because the aim is not to actually design the monopile but to compare the different approaches. The 3D FE analysis case is considered to be the reference case, as all other cases require simplifications (i.e. Winkler approach) or empiricism (e.g. API method) to be introduced. 3D FE leads to required pile embedded length of about 37 m in this case. The API 'p-y' approach results in a significantly larger pile penetration with about 52 m. The PISA approaches are closer to 3D FE with 35 m for the rule-based approach and 33 m for the numerical-based approach. Surprisingly, the numerical based approach which use the same 3D FE analyses to extract site-specific set of depth variation functions are seen to perform worse than the generic rule-based functions.

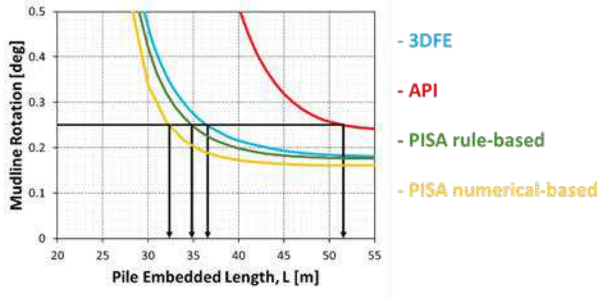

Figure 9. Example of pile embedded length design in SP1.

3.4 *Comparison of the different approaches*

Extending the analysis to all the 3 clay profiles, we observe the same trend. The API 'p-y' approach, shown in red in Figure 10 largely overestimates the pile length from 3D FE in blue. For the three clay profiles considered here, the required pile lengths were 26% to 41% larger. The PISA approaches in green for rule-based and yellow for numerical based are a significant improvement with much closer prediction (from -4% to -12%). However, the PISA numerical-based approach was expected to better match with the 3D FE result than PISA rule-based but it is not the case here.

Foundation design at the scale of an entire offshore wind farm cannot solely rely on 3D finite element modelling. Indeed, 3D FE can be very accurate once properly calibrated but are computationally expensive. Fast design approaches are required for early design stage but need to remain accurate to enable optimisation at later stage of design which may require many thousands of different analysis cases. Even a few metres of pile penetration change can significantly impact fabrication, transportation and installation costs at the scale of the entire offshore wind farm.

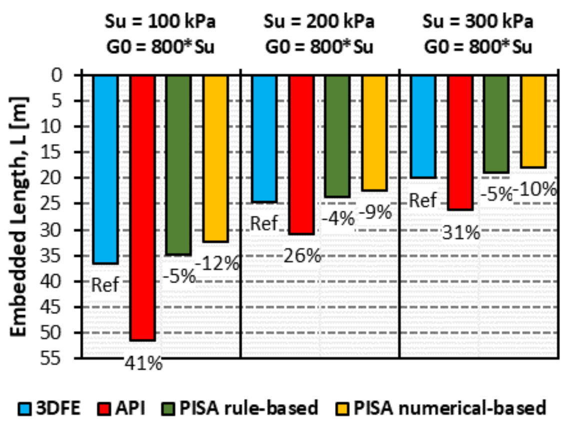

Figure 10. Comparison of design pile embedded lengths.

4 REFINEMENT OF PISA NUMERICAL-BASED APPROACH

4.1 *Current shortcomings*

For all three homogeneous clay profiles considered here, the PISA numerical-based approach (implemented in Plaxis Monopile Designer V21) estimated embedded lengths which were marginally unconservative (up to 12%) in comparison with the reference 3D FE models. Design pile lengths estimated with the PISA rule-based approach (generic depth variation functions calibrated in Cowden till as reported in Table 1) were closer to the 3D FE than the numerical-based approach using depth variation functions calibrated with PLAXIS Monopile Designer V21. Potential causes are listed below:

- The addition of distributed moment, base shear and base moment is a significant improvement of the API 'p-y' only framework. However, the implementation in 1D FE remains an approximation and simplification of the 3D pile-soil interactions.
- Each parameter of each soil reaction component is fitted independently of the other. There is no guarantee that the fittings of the 16 depth variation functions are fully compatible.
- Parameters fitted from soil reaction curves form a scattered point cloud which is hard to fit with depth variation functions. This is due to different soil responses for different pile geometries, a lack of soil reaction close to point of rotation and interactions between distributed and base soil reactions close to the pile toe among other errors. Plaxis Monopile Designer CE V21 does not show the points cloud and hence the user cannot assess the quality of the fit for the depth variation functions. **Error! Reference source not found.** shows an example of a point cloud extracted from 3D FE using the Python interface of Plaxis for SP1.
- The goodness of the match between 1D FE and 3D FE is assessed based on two accuracy metrics comparing the integral of difference over integral of 3D FE load displacement curves. One metric is computed for very small displacements (lower than D/10000) and the other one for large displacements (up to D/10). These metrics are not meaningful a geotechnical engineer as they do not report if 1D FE is overestimating or underestimating the 3D FE response. Also, it is hard to define what is a good accuracy metric as they are all typically high.

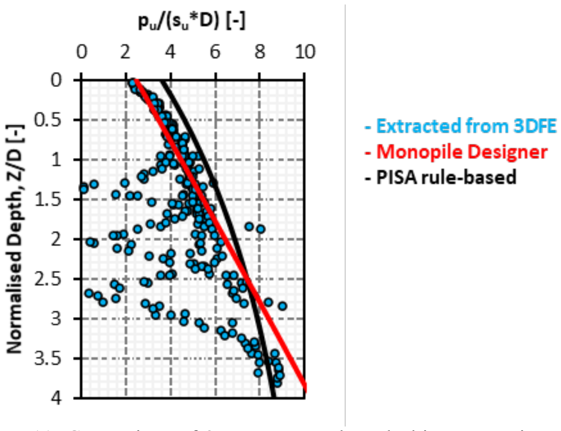

Figure 11. Comparison of 3D FE extract lateral ultimate reaction and depth variation functions fitted by Plaxis Monopile Designer V21 and calibrated in Cowden tillB

4.2 *Proposed solutions*

The PISA numerical-based approach as implemented in Plaxis Monopile Designer V21 corresponds to the first stage calibration reported in Byrne et al. (2019). Site-specific depth variation functions are fitted based on a range of 3D FE models with different pile geometries but keeping the same soil profile. Each depth variation function is fitted independently to offer the best match with the scattered point cloud. During the calibration exercise in Cowden till, Byrne et al. (2019) reported accuracy metrics in the range of 77% to 92% for small displacement and 90% to 98% for large displacement. Depth variation functions were then adjusted during a second stage calibration. The aim was no longer to fit each scattered point cloud individually, but instead to offer the highest accuracy metrics. Metrics in the range of 95% to 99% for small displacement and 90% to 96% for large displacement have been reported in this way.

The first point of improvement to be suggested is to implement the second stage calibration into the PISA numerical-based approach. This second stage calibration seems to be a key aspect of the PISA project and is lacking in Plaxis Monopile Designer CE V21. Recently, a calibration of PISA depth variation functions using in-situ measurements from an instrumented offshore wind turbine founded on monopile was reported (Jurado 2021). The authors reported the use of a Bayesian optimization approach to improve the match between the predicted and measured bending moment profile by applying scaling factor to the initial stiffness (k) and ultimate reaction (y_u) parameters in the PISA approach. This is deemed very interesting as it offers a clear optimization framework with limited number of parameters to account for.

The second point of improvement concern the metrics used to compare 1D FE and 3D FE responses. Accuracy metrics defined within the PISA framework, while useful to gauge model performance, are arguably not ideally suited for monopile geotechnical design. Arguably the three most important criteria for the design of a monopile are:

1. Pile ultimate capacity to ensure minimum factor of safety against failure under extreme loads. There is often no clear plateau in the load-displacement curve and ultimate capacity can be hard to define. It is proposed here to define the ultimate capacity as the lateral load at which pile displacement at sea floor reach 10% of the pile diameter.
2. Pile sea floor rotation under maximum operational loads to verify serviceability limit state. From experience, typical large diameter monopile design are found to have a high margin of safety against failure. Considering a load partial factor of 1.35, material partial factor of 1.25 and a typical ULS utilisation (ratio of design load to design resistance) of 50 – 70%, it is proposed here to estimate operational loads as 1/3 of 3D FE ultimate capacity.
3. Pile stiffness to assess the natural frequency of the structure. It is proposed here to define the small-strain stiffness as the secant stiffness in the load-displacement curve at seafloor under loads equivalent of 2% of the 3D FE ultimate capacity.

Hence, 3 new metrics based are proposed for calculating relative error for pile ultimate capacity (Eq. 5), pile sea floor rotation under operational loads (Eq. 6) and pile small-strain strain stiffness (Eq. 7). A metric value of zero is equivalent to a perfect match, a negative value implies the 1D FE is underestimating compared to the 3D FE and a positive value implies the 1D FE is overestimating compared to the 3D FE. Overestimation of ultimate capacity or underestimation of rotation under operational loads leads to unconservative design. For the stiffness, both may lead to unconservative design as discussion in the introduction.

$$\delta_{ULS} = \frac{H_{ult}^{1DFE} - H_{ult}^{3DFE}}{H_{ult}^{3DFE}} \quad (5)$$

$$\delta_{SLS} = \frac{\theta_{ult/3}^{1DFE} - \theta_{ult/3}^{3DFE}}{\theta_{ult/3}^{3DFE}} \quad (6)$$

$$\delta_{FLS} = \frac{K_{ult/50}^{1DFE} - K_{ult/50}^{3DFE}}{K_{ult/50}^{3DFE}} \quad (7)$$

4.3 *Future works*

Plaxis Monopile Designer is a commercial software and users do not have access to all functions and variables required to implement new functionalities. Hence, current works consist in developing a new tool to implement the proposed solution as presented in Figure 12. Python was selected as the programming language as it enables interface with Plaxis 3D to automate the setup of 3D FE models and the extraction of soil reaction curves. Future works will consist in transferring the current Matlab 1D finite element solver used in this paper to Python in order to streamline the workflow. An optimisation procedure will be introduced in order to update the depth variation functions in a second stage calibration phase. The aim is to ensure the best match with 3D FE mudline response. Although the three new metrics need to be reported to understand the performance of 1D FE with respect to 3D FE, the optimisation procedure needs the three metrics to be assembled in a cost function. The optimisation procedure will aim at minimizing the cost function C below where n is the number of 3D FE models used for the calibration of the depth variation functions:

$$C = \sqrt{\frac{1}{n}\sum_{i=1}^{n} \delta_{ULS,i}^{\;2}} + \sqrt{\frac{1}{n}\sum_{i=1}^{n} \delta_{SLS,i}^{\;2}} + \sqrt{\frac{1}{n}\sum_{i=1}^{n} \delta_{FLS,i}^{\;2}} \quad (8)$$

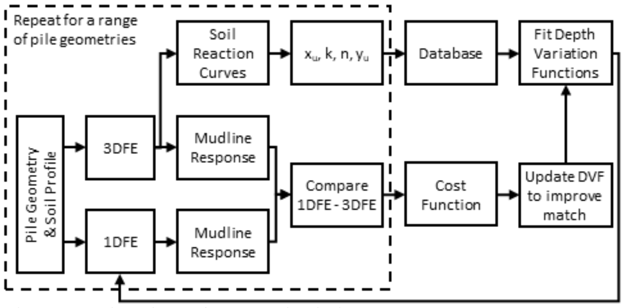

Figure 12. Flowchart of proposed solutions.

Once fully implemented, this tool will be used to populate a database of depth variation functions in a range of representative clay profiles, which may then use without the need for new 3D FE analysis (i.e. as per PISA rule-based approach).

5 CONCLUSIONS

This paper presented the main industry approaches used for the geotechnical design of monopile supporting offshore wind turbines. The traditional API 'p-y' approach, the state-of-the-art PISA rule-based and numerical-based approaches and 3D finite element modelling were considered for a typical monopile design in clays. It was shown that:

- API largely overestimated the required pile penetration in comparison to 3D FE and would lead to an uneconomical design. While this might be conservative for ultimate capacity check or serviceability limit check, it might be unconservative for the natural frequency check.
- The PISA approaches are a significant improvement although they were found slightly unconservative in this case.
- The PISA numerical-based approach as implemented in Plaxis Monopile Designer CE V21 is not performing as expected. The fitted depth variations functions led to a worse match with 3D FE than the generic functions calibrated in Cowden till. A number of limitations of Plaxis Monopile Designer CE V21 were discussed. The main limitation appears to be the lack of a second stage calibration of the depth variation functions to ensure maximum match with 3D FE mudline response.

Hence, current and future works consist in the implementation of a Python code able to automate the setting up of Plaxis 3D models, the extraction, normalisation and fitting of soil reaction curves and the two-stage calibration of depth variation functions. Also, new accuracy metrics are considered in order to better assess the performance of the PISA numerical-based approach. These new metrics are based on meaningful criterion for the geotechnical design of monopile. Once fully implemented, this tool will be used to populate a database of depth variations functions in a range of representative clay profiles.

6 ACKNOWLEDGEMENTS

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