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CONCEPT DESIGN STUDY OF LATERALLY LOADED MONOPILES IN SAND

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ABSTRACT

Single large diameter piles, known as monopiles, are, nowadays, the most common foundation type for offshore wind turbines (OWT). Several research initiatives have been focusing on improving the conventional design practice. The PISA (Pile Soil Analysis) joint industry research project resulted in a proposal for a new design methodology for OWT pile foundations, focusing on laterally loaded monopiles, with length-to-diameter ratios (L/D) between 2 and 6. The proposed methodology defines a one-dimensional (1D) model, based on the use of Timoshenko beam theory, which overcomes certain limitations in current design practice. Soil reaction components can either be generated through mathematical formulations (rule-based design method) or calibrated from 3D Finite Element (FE) models (numerical-based design method). The present paper examines the suitability of the numerical-based PISA design method for analyzing the pile response in sandy soil conditions. A concept design study for monopiles in dense sand is considered. Soil properties are based on realistic conditions encountered in the North Sea. The PISA methodology is compared with the methodology followed by the current design practice. Results are discussed and several conclusions are drawn providing factual insight into the applicability of the PISA design methodology for the design of monopiles in sand.

Keywords: PISA, offshore wind, monopile, concept design, sand, finite element modelling

INTRODUCTION

Monopiles are tubular steel piles with diameter of about 5 m to 10 m and length-to-diameter ratios of 3 to 6. Monopiles constitute the most commonly used in practice foundation type of OWTs in Europe, reaching about 80% (Pisanò, 2019). The usage of increasingly large OWTs in deeper waters indicates the need for monopiles of even larger diameter, assuming that they remain the preferred foundation solution. This is a topic of extensive research nowadays, indicating the need for development of state-of-the-art design standards for the forthcoming offshore wind projects.

The PISA (Pile Soil Analysis) joint industry research project lead to a novel design methodology in which a fast and robust one-dimensional (1D) finite element (FE) model is used to obtain an optimized design for a monopile for site-specific soil and loading conditions.

The PISA project was structured in two phases. In Phase 1 (Byrne et al., 2015; Zdravković et al., 2015) the focus was on homogeneous soil conditions. Field tests on dense to very dense marine sand at Dunkirk (Burd et al., 2017) and stiff over-consolidated glacial clay at Cowden (Byrne et al., 2017) were used as a basis to develop the PISA design methodology for homogeneous soil profiles. Based on this design approach, a first version of a monopile design tool named PLAXIS MoDeTo was developed and released in 2018 (Panagoulias et al., 2018a).

The design approach adopted in this tool has been validated for Cowden clay (Minga and Burd, 2019a) employing the NGI-ADP model (Andresen and Jostad, 1999) and Dunkirk sand (Minga and Burd, 2019b) employing the Hardening Soil model with small-strain stiffness (HSsmall; Benz, 2007).

Phase 2 of the PISA research project (Byrne et al., 2019a) led to further developments. The focus of this second phase was on layered soil profiles while also considering additional homogeneous profiles, including three additional variations of Dunkirk sand with different relative densities, varying from 45% to 90%. The extension to the PISA design methodology, which was the outcome of the second phase of the project, was subsequently implemented in the same software application.

Previous research has discussed the application of the PISA methodology to the concept design of monopile foundations in homogeneous clayey soil conditions (Panagoulias et al., 2018b, 2018c; Kaltekis et al., 2019) and idealized layered soil profiles (He et al., 2017; Panagoulias et al., 2019). The present paper examines its application to the analysis of the pile response in homogeneous sand, based on realistic conditions encountered in the North Sea.

PISA DESIGN METHODOLOGY

The PISA design methodology describes two parallel non-conflicting approaches, 'rule-based design' (RBD) and 'numerical-based design' (NBD). Both approaches make use of a 1D FE design framework, based on the Timoshenko beam theory, where soil-structure interaction (SSI) is modelled by independent non-linear soil reactions applied to the finite elements. To account for the large diameter and low length-to-diameter ratio that are characteristic of monopile foundations, four soil reaction components are employed to model SSI: the lateral soil reaction along the shaft (p), the distributed moment reaction as a result of the vertical shear stress distribution along the shaft (m), the shear force reaction at the base (HB), and the moment reaction as a result of the normal stress distribution at the base (MB). The set of non-linear functions that relate the soil reaction components (forces or moments) to the local pile deformations (displacement, v, or rotation, ψ) is called 'soil reaction curves' (SRC). While in rule-based design the SRC are defined by mathematical formulations, in numerical-based design the SRC are derived from the results of a series of three-dimensional (3D) FE calculations; a procedure known as 'calibration' (Panagoulias et al., 2018a).

This procedure involves obtaining numerical SRC at different depths for a small number of 3D FE calibration models, which define the geometric boundaries of a 'calibration space' or 'design space', within which the four types of SRC for each soil material are normalized and parameterized. These parameterized SRC can then be used by the 1D FE model to enable rapid calculations for any combination of monopile geometry and loading conditions within the limits of the design space but not necessarily part of the initial calibration set.

Two different material types are supported, clay and sand. Parameterized SRC for a particular (homogeneous) soil material (e.g. Dunkirk sand with relative density of 90%) can also be stored and reused at any other site where the same soil unit is encountered, e.g. at multiple locations within an offshore wind farm. Regardless of whether the actual-site soil profile is homogeneous or layered, the basic assumption of the methodology, stated in He et al. (2017) and developed in Phase 2 of the PISA research project (Byrne et al. 2019a), is that parameterized SRC derived from homogeneous soil profiles can be employed directly in the 1D FE model for the analysis of monopiles in either homogeneous or layered soil conditions. Therefore, the calibration procedure is always performed on (one or more) homogeneous soil profiles. Although each calibration profile is homogeneous, the methodology supports the definition of sub-layers to characterize in detail the variation with depth of material parameters.

For each calibration model and soil material, the four SRC components at different depths are normalized separately, employing different normalization factors depending on the component and soil type (clay or sand), and fitted to a selected mathematical function. Although this process could be applied to a range of arbitrary functions, in the PISA design methodology only conic functions were adopted. The sixteen parameters (four for each of the four components) defining the normalized conic SRC are depth dependent. For a homogeneous calibration profile, the depth-dependency is described by sixteen continuous ‘depth variation functions’ (DVF), which can be linear or exponential, once more depending on the SRC component and the soil type.

After normalized SRC are obtained for each of the calibration models, they are parameterized to generate a set of DVF characteristic to the specific (homogeneous) soil material and geometric boundaries of the design space. Previous research suggests that as few as four 3D FE models per homogeneous soil profile are required to define a design space where a suitable calibration can be achieved (Kaltekis et al., 2019).

In a final step, the DVF defining the parameterized SRC for each soil material are assigned to the corresponding soil layers in the 1D FE model, which enables instantly back-calculating particularized (de-normalized) SRC for any combination of monopile geometry, stratigraphy, and loading conditions considered in the design space. The response of different monopile geometries under different design load cases can then be analyzed in a rapid 1D FE design model, leading to more realistic, reliable, and potentially more efficient monopile designs. Previously published comparative design cases have shown potential reductions in embedded length of up to 35% in clayey soil conditions (Byrne et al., 2019b; Kaltekis et al., 2019). The present concept design study explores such a potential savings in sandy soil conditions.

SOIL CONDITIONS AND CONSTITUTIVE MODEL

The selected soil profile is based on a uniform sandy offshore North Sea location. High quality seafloor seismic cone penetration test (SCPT) data were used for the soil interpretation. The small-strain stiffness profile was evaluated directly from the SCPT data by use of empirical correlations (Rix and Stokoe, 1991). Strength parameters were derived based on Schmertmann (1978), while the soil relative density was assessed in accordance with Baldi et al. (1986).

The main soil parameters of the selected soil profile are presented in Table 1, where γ' is the submerged soil unit weight, φ' the effective friction angle, ψ the dilation angle and G_0 the small-strain stiffness modulus. The layering in the employed soil model is rather fine to accurately capture the depth variation of stiffness and strength. The bottom layer is thicker as none of the studied pile penetration depths reaches below 30.0 m.

The HSsmall constitutive model is employed to simulate the soil response. The constitutive model is calibrated using empirical correlations (Brinkgreve et al., 2018). The model-specific input parameters are presented in Table 2, where the stiffness parameters are given in terms of reference values at a reference pressure level of 100 kPa. $E_{50,ref}$ represents the secant stiffness in standard drained triaxial tests, $E_{oed,ref}$ the tangent stiffness for primary oedometer loading, $E_{ur,ref}$ the elastic un/re-loading stiffness in drained triaxial tests, $\gamma_{0.7}$ the threshold shear strain and m is the power for the stress-level dependency of stiffness. The stiffness depth-variation is formulated as given by Eq. 1:

$$E = E_{ref}\left(\frac{\sigma'}{p_{ref}}\right)^m \qquad [1]$$

Where E = stiffness modulus, E_{ref} = reference value of the stiffness modulus at a reference pressure p_{ref} equal to 100 kPa and σ’ is the effective stress. In case of the stiffness moduli E_{50},

E_{ur} and G_0, σ' is the minor principal effective stress (noted as σ'_3), whereas for the stiffness modulus E_{oed}, σ' is the major principal effective stress (noted as σ'_1).

For each soil layer the stiffness moduli $E_{oed,ref}$ and $E_{ur,ref}$ are estimated based on $E_{50,ref}$. More precisely, $E_{oed,ref}$ is assumed to be approximately equal to $E_{50,ref}$, while $E_{ur,ref}$ is selected to be three times $E_{50,ref}$.

Table 1. Summary of main soil parameters

#Layer	Depth [m]	γ' [kN/m^3]	φ' [deg]	ψ [deg]	G_0 [MPa]
1	0.0-0.8	6.7	40	10	16
2	0.8-1.3	8.8	45	15	35
3	1.3-2.3	9.9	46	16	52
4	2.3-3.5	10.6	45	15	101
5	3.5-5.0	11.1	45	15	134
6	5.0-7.8	11.4	45	15	164
7	7.8-10.5	11.3	43	13	148
8	10.5-11.5	11.0	41	11	146
9	11.5-12.8	10.2	38	8	122
10	12.8-15.0	10.9	41	11	158
11	15.0-17.6	11.6	44	14	204
12	17.6-21.1	11.5	43	13	211
13	21.1-25.5	11.4	42	12	227
14	25.5-29.4	11.5	42	12	237
15	29.4-30.4	10.3	40	10	209
16	30.4-31.4	11.0	42	12	258
17	31.4-32.5	11.0	42	12	250
18	32.5-43.0	10.7	40	10	219

Table 2. Summary of HSsmall input parameters

#Layer	$E_{50,ref}$ [MPa]	$E_{oed,ref}$ [MPa]	$E_{ur,ref}$ [MPa]	$G_{0,ref}$ [MPa]	$\gamma_{0.7}$ [-]	m [-]
1	17	17	52	373	2.0E-05	0.68
2	21	19	64	469	3.0E-05	0.68
3	33	28	100	458	4.0E-05	0.68
4	96	85	287	187	6.0E-05	0.56
5	99	89	296	193	8.0E-05	0.56
6	93	83	279	182	1.0E-04	0.56
7	63	63	190	123	1.7E-04	0.81
8	56	56	167	107	2.0E-04	0.81
9	28	28	83	83	2.6E-04	0.81
10	54	54	161	105	2.2E-04	0.81
11	68	66	205	133	1.9E-04	0.81
12	58	58	173	127	2.1E-04	0.81
13	39	39	116	124	2.3E-04	0.81
14	30	30	89	120	2.5E-04	0.81
15	19	19	57	99	3.1E-04	0.81
16	37	37	111	124	2.5E-04	0.81
17	33	33	99	119	2.7E-04	0.81
18	21	21	64	99	3.1E-04	0.81

FINITE ELEMENT MODELING

In the present study, the PISA design methodology is employed via PLAXIS MoDeTo (Panagoulias et al., 2018a). In each one of the automatically generated 3D FE calibration models (CM), half of the real geometry is considered due to symmetry. The soil domain consists of 10-noded quadrilateral elements while the monopile is modeled via 6-noded shell elements (Brinkgreve et al., 2018) with conventional steel material properties (Young's modulus of 210 GPa, Poisson's ratio of 0.3 and a submerged unit weight of 68.5 kN/m^3). The finite elements domain is extended adequately at all sides of the model to minimize boundary effects. On average about 60,000 finite elements are used per 3D model. Interface elements with reduced strength and stiffness (strength-stiffness reduction factor of 0.67) are used at the outer side and at the bottom of the pile, to allow for slipping and gap opening. Figure 1 illustrates one of the employed calibration models (model CM5 in Table 3). As depicted, the mesh is further refined around the monopile to increase the accuracy of the numerical results.

Drained analysis is used assuming fully drained soil conditions during lateral static loading in sand. The numerical calculations are displacement-controlled, and the piles are pushed laterally to reach a lateral displacement of about 0.2 times the diameter at ground level. This is needed to ensure good calibration quality of the 1D FE model (Panagoulias et al., 2018a).

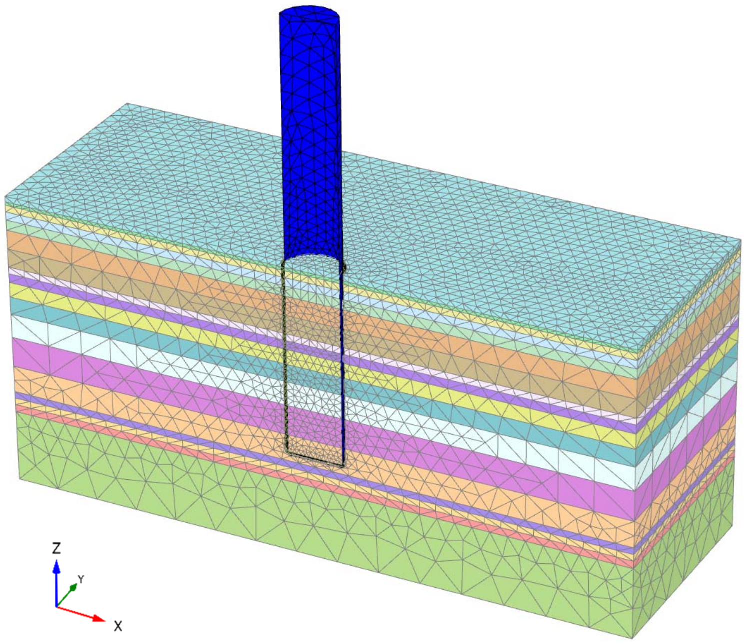

Fig. 1. Calibration model CM5

CONCEPT DESIGN STUDY

A certain variation of geometrical parameters is assumed a priori in order to define the needed calibration or design space. Table 3 summarizes the employed 3D FE calibration models. A minimum aspect ratio *L/D* of 3.0 is examined, whereas the maximum is 4.3. The penetration depth *L* varies between 24.0 m and 30.0 m, the diameter *D* lies between 7.0 m and 8.0 m, while the pile wall thickness *t* is constant, equal to 80 mm. For simplicity, a variation in the monopile thickness is not considered in this study as it is observed that it has a low impact on the results (Byrne et al., 2017). The lever arm above mudline *h* is also held constant and equal to 35.0 m. This is assumed to be the height at which the horizontal resultant force is applied, representing the contemporaneous action of wave and wind loads.

In this concept design study, the lateral monopile response is studied under a static lateral load of 15.0 MN acting at height *h* above mudline, resulting to a moment of 525.0 MNm at mudline. A vertical load of 19.0 MN is applied to the top of the pile representing the self-weight of the structure. In line with previously published results on a homogeneous clay soil profile by Kaltekis et al. (2019), the following design criteria are adopted to assess the appropriate monopile geometrical configuration for the studied soil profile:

- A global safety factored of 1.5 is used on the ultimate limit state (ULS) loading conditions following the Working Stress Design (WSD) approach;
- Under ULS loading conditions, the average lateral displacement at mudline should not exceed 0.1 times the monopile diameter;
- Under serviceability limit state (SLS) loading condition (unfactored load), the average rotation at mudline should not exceed 0.25 deg, conservatively assuming irreversible pile rotation.

Table 3. Set of 3D FE models to calibrate the PISA 1D model

3D model	D [m]	L [m]	h [m]	t [mm]	L/D [-]
CM1	7.0	24.0	35.0	80	3.4
CM2	7.0	27.0	35.0	80	3.9
CM3	7.0	30.0	35.0	80	4.3
CM4	8.0	24.0	35.0	80	3.0
CM5	8.0	27.0	35.0	80	3.4
CM6	8.0	30.0	35.0	80	3.8

To evaluate the applicability of the PISA design methodology in the examined homogeneous sandy soil conditions, use of the current standard design methodology is made, based on the ‘*p-y*’ approach (DNVGL-RP-C212, 2017). The soil parameters included in Table 1 are used to calibrate the ‘*p-y*’ soil reaction curves.

Note that the present study does not consider other design considerations such as installation and scour effects. The pile response is studied under monotonic loading conditions and any effect of cyclic loading on the soil strength and stiffness, but also structural fatigue, is out of the scope of this study. In addition, no safety factor is applied to the vertical load as that would lead to a non-conservative system; higher vertical loads would lead to higher base friction and hence stiffer system response.

RESULTS

Due to the fact the PISA methodology is used by means of PLAXIS MoDeTo the result of the PISA calibrated 1D FE model is denoted as ‘1D PLAXIS MoDeTo’, while the result of the 1D FE model which makes use of the standard ‘p-y’ curves is denoted as ‘1D p-y’. The results coming directly from the PLAXIS 3D models will be denoted as ‘3D PLAXIS’.

The pile diameter is determined via an estimation of the total mass of the support structure assuming various penetration depths and constant wall thickness. Preliminary assessment of the fatigue loads acting on the structure for the various combinations of *D* and *L*, indicated that a diameter of 8.0 m is possibly the most economically feasible for the studied soil and loading conditions. Thus, all the analyses discussed in the present section are based on piles with *D* = 8 m.

Figure 2 (left) illustrates the average monopile rotation at mudline versus penetration depth for the SLS loading conditions. The ‘1D PLAXIS MoDeTo’ results appear to be comparable to the ‘1D p-y’ results up to the threshold of 0.25 deg, suggesting a minimum penetration depth of about 24 m and 25 m respectively. The resulting ‘1D PLAXIS MoDeTo’ value is at the margin of the defined design space (Table 3) but assumed to be acceptable for the purpose of this case study.

Figure 2 (right) also depicts the ultimate load capacity of the structure versus penetration depth. As observed, to withstand the ULS load of 22.5 MN, '1D PLAXIS MoDeTo' suggests a penetration depth of approximately 17 m, whereas '1D p-y' reads about 20 m, i.e. roughly 18% longer pile. In both cases the resulting penetration depth is lower than the depth resulting from the SLS design criterion. Thus, the SLS loading conditions are critical for the present case study, governing the minimum penetration depth.

To study the monopile penetration depths which could potentially satisfy the ULS design criterion, values outside the employed design space (Table 3) were used. As a result, the obtained ultimate loads from '1D PLAXIS MoDeTo' cannot be accurate and should be used with caution, merely as indicative values. However, the '1D p-y' results give confidence to the fact that the SLS loading conditions are driving the monopile design in the present study.

The obtained penetration depths from the employed design criteria discussed above, especially from the ULS, seem to be low comparing to the current design practice, which suggests penetration depths greater than approximately 25 m. This is mainly because cyclic loading effects are not considered in the present study. Such effects would most probably increase the required penetration depth.

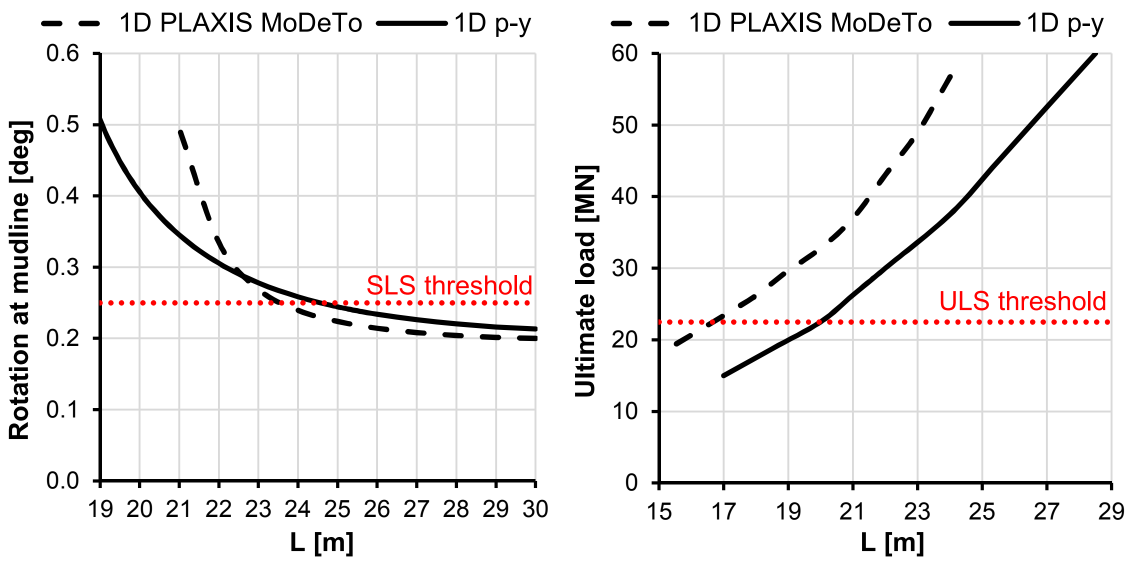

Fig. 2. Penetration depth analysis for the SLS (left) and ULS (right) design criteria

Based on the adopted design criteria and the findings above, an optimized design is determined for the monopile: D = 8.0 m, L = 24.0 m and t = 80 mm. For the selected geometrical configuration, a '3D PLAXIS' finite element model is generated to verify the response of the calibrated '1D PLAXIS MoDeTo' model. Figure 3 presents the comparison between the two FE models in terms of horizontal load versus lateral displacement at mudline. In the same plot the result of the corresponding '1D p-y' model is also plotted. The calibrated '1D FE PLAXIS MoDeTo' very well matches the '3D PLAXIS' response, both in initial stiffness (small displacements zone) and ultimate capacity (large displacement zone). The response of the '1D PLAXIS MoDeTo' model and the '3D PLAXIS' model is compared quantitatively via the accuracy metric eta (η) (Byrne et al., 2019a), given by Eq. 2. The obtained accuracy, considering lateral displacement of 0.8 m, is about 95%.

$$\eta = \frac{(A_{ref} - A_{diff})}{A_{ref}} \qquad [2]$$

Where A_{ref} is the area below the reference load-displacement curve (3D FE model) and A_{diff} is the area bounded by the 3D FE curve and the curve which corresponds to the 1D FE model.

As observed in Fig. 3, the '1D p-y' method results in a lower ultimate capacity of the pile. This is in line with the results presented in Fig. 2 (right) where the same method indicates that longer piles are needed in order to achieve the defined ULS loading conditions. More precisely, to achieve a hypothetical ultimate capacity of approximately 55 MN to 60 MN, a penetration depth of about 28 m would be needed, which is roughly 17% longer than the 24 m suggested by '1D PLAXIS MoDeTo'. A reason for this discrepancy could be that the static ultimate lateral capacity of the '1D p-y' method is given as a function of the effective friction angle φ', with a maximum of 40 deg (DNVGL-RP-C212, 2017). The soil profile employed in the present study is rather competent, with φ' exceeding 40 deg at various depths (Table 1). Consequently, the ultimate capacity reached by the '1D p-y' model is affected by the bounded values of the effective friction angle.

Looking into the moment versus rotation at mudline plotted in Fig. 4, the '1D PLAXIS MoDeTo' and the '1D p-y' models give similar results. The '1D p-y' model gives slightly softer response which is consistent with the observations in Fig. 2 (left), where the '1D PLAXIS MoDeTo' and the '1D p-y' models resulted in similar penetration depths in order to satisfy the SLS design criterion, but the latter indicated a longer required penetration depth of about 1 m. However, this difference may be characterized as negligible comparing to the various simplifications accounted for in this study.

As Figs. 3 and 4 suggest, the initial stiffness is comparable for all FE models. This observation may advocate that all three models could be adequately used in the parts of the design process that focus on the small loads regime, such as the SLS or the fatigue limit state (FLS) analyses. Nonetheless, the designers should look carefully into the differences in a project and location-specific basis and evaluate the influence of the initial stiffness to the design variables. Such a detailed evaluation is beyond the scope of the present study.

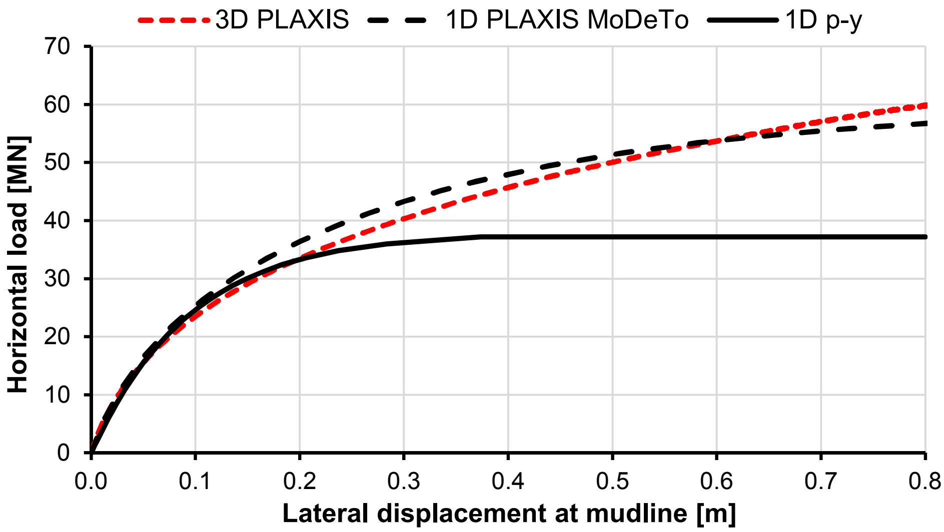

Fig. 3. Horizontal load versus latera displacement at mudline for the design model

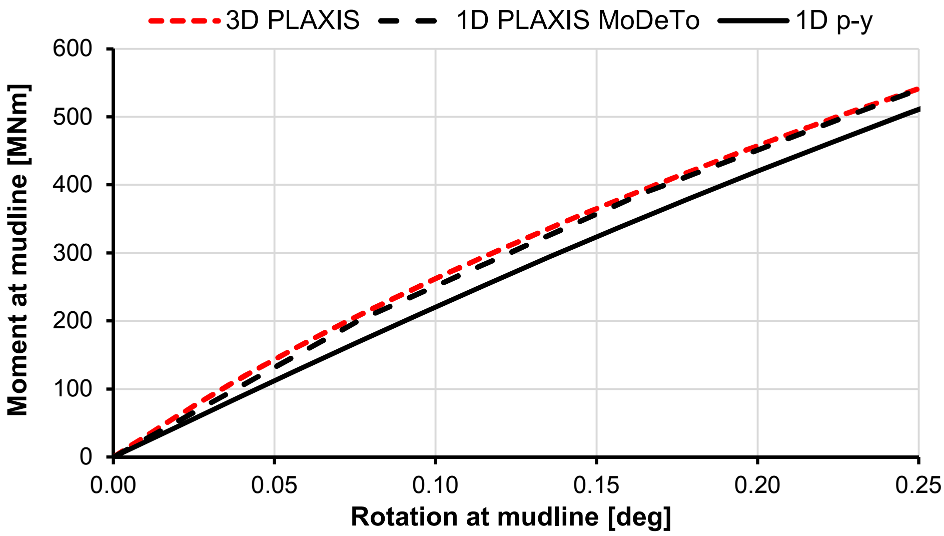

Fig. 4. Moment versus rotation at mudline for the design model

CONCLUSIONS

The paper presented a concept study of the behavior of monopiles in sand during static lateral loading. The applicability of the PISA design methodology was the focus of the study, comparing to the current 'p-y' design practice. A soil profile indicative of an offshore sandy location in the North Sea was employed, and realistic loading conditions were adopted.

The study indicated that the employed SLS design criterion was the decisive condition for the considered soil profile and loading assumptions, suggesting a penetration depth of about 24.0 m for a pile with a diameter of 8.0 m (aspect ratio of 3). The '1D PLAXIS MoDeTo' model, incorporating the PISA design methodology, gave a very good match with the corresponding '3D PLAXIS' model and similar results to the '1D p-y' model with respect to the SLS design criterion. Regarding the ULS design criterion, the '1D p-y' model suggested a 17% longer pile than the '1D PLAXIS MoDeTo' model. From this observation a conclusion could be drawn that as long as the SLS design criterion is governing, then the 'p-y' methodology may give satisfactory results for the design penetration depth in sands. However, in case that the ULS criterion prevails or the tolerance of SLS design criterion increases (i.e. a permanent rotation at mudline higher than 0.25 deg is allowed), then, based on the soil conditions and design assumptions of the present study, the 'p-y' method suggests a lower value of the monopile capacity comparing to the '1D PLAXIS MoDeTo' and '3D PLAXIS' models, resulting in longer required penetration depths.

In the authors' view the present study can be used as indicative for the design of monopiles in sandy soil conditions, and assists the current practice. The absence of advanced laboratory soil test data for the calibration of the employed constitutive model, inherent limitations of the used numerical methods, and adopted simplifications in the design procedure, may also have an influence on the presented results. Further investigation of the monopile response in sand, including comparison between numerical simulations and centrifuge or full-scale experimental data, is required to draw concrete conclusions. Important geotechnical aspects such as cyclic loading conditions, installation and loading-rate effects should be investigated as well in order to establish a state-of-the-art design methodology of monopiles in sand.

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