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Design of laterally-loaded monopiles in layered soils

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ABSTRACT

This paper describes an implementation of the methodology, developed in Phase 2 the recent PISA (PILE Soil Analysis) joint industry research project, for the design of laterally-loaded monopiles in layered soils. The software PLAXIS MoDeTo and PLAXIS 3D are employed to obtain the soil reaction curves that are required for the method, following the PISA ‘numerical-based’ design approach. A particular design space is selected to define the variation of the geometrical parameters assigned to the three-dimensional (3D) Finite Element (FE) calibration models. The parameters that span the design space are the embedded length (L), the outer pile diameter (D), the pile wall thickness (t) and the height above the mudline (h), where the design load is applied. The soil reaction curves are determined from the 3D FE calibration models for separate homogeneous soil conditions consisting of stiff normally consolidated clay and very dense sand. The calibration set consists of eight 3D FE models, for each homogeneous soil profile. Subsequently, the soil reaction curves are parameterised and used to calibrate a one-dimensional (1D) FE model, formulated by means of Timoshenko beam theory, which allows for fast and robust design calculations. A final design model (DM) is defined and its response is studied considering the two homogeneous profiles and four additional layered soil profiles. The results of each 1D analysis are compared with equivalent 3D FE models and a 1D FE model developed at the University of Oxford (OxPile) as part of the PISA research. The accuracy metric η is used to compare quantitatively the response among the employed models, focusing on large displacements at ground level (about $D/10$). The results indicate a very good match for all considered soil profiles; all computed η values exceed 90%. The research findings support the applicability of the PISA design methodology in both homogeneous and layered soil conditions.

Keywords: PISA, MoDeTo, monopile design, layered soils

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INTRODUCTION

The energy transition from carbon fuels to renewable energy has stimulated several new research initiatives. The PISA (PIle Soil Analysis) joint industry research project (*Byrne et al., 2015; Burd et al., 2017; Byrne et al., 2017; Byrne et al., 2018; Zdravković et al., 2018; Burd et al., 2019; Byrne et al., 2019a,b*) aims at the optimisation and cost reduction of the design of monopiles as foundation elements for offshore wind turbines. The outcome of this research project is a novel design methodology in which a rapid design procedure is followed to obtain an optimized design for a monopile for specific soil and loading conditions. A 1D FE design model is employed, leading to more realistic, more reliable and likely more economic monopile designs. The Phase I of the PISA research project focused on developing a design methodology for homogeneous soil conditions. 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 (*Panagoulis et al., 2018c*) was developed and released in 2018. Meanwhile, the Phase 2 of the PISA research project commenced (*Byrne et al., 2019a*), leading to further developments in this design approach. The focus of the second phase was on layered soil profiles, also considering additional homogeneous soil profiles, namely Bothkennar soft clay, London stiff clay and three additional variations of Dunkirk sand with different relative densities from 45% to 90%. The extended version of the PISA design methodology, which was the outcome of the second phase of the project (*He et al., 2017; Byrne et al., 2019a*), is currently being implemented in the next version of PLAXIS MoDeTo, which will be publicly released in early 2020.

The present research studies the suitability of the PISA design methodology for layered soil profiles. An updated version of the PLAXIS MoDeTo 1D FE model is used, which supports analyses with layered soil profiles. The PLAXIS 3D 2018 version (*Brinkgreve et al., 2018*) is used to support the extraction of the soil reaction curves from a set of 3D FE calibration models for homogeneous soil profiles. Example analyses are presented in which soil properties typically encountered in the North Sea are employed. The PISA design methodology adopted in this study is discussed, while the 3D FE calibration models and soil profiles adopted in the analyses are subsequently presented. The results of the 1D FE calculations are discussed and verified against PLAXIS 3D, and also an implementation of the 1D model – known as OxPile – that was developed at the University of Oxford during the PISA project. Concluding remarks are given in the last section.

THE PISA DESIGN METHODOLOGY

The PISA design methodology envisages two design approaches, namely ‘rule-based’ design (RBD) and ‘numerical-based’ design (NBD). Both approaches are implemented in PLAXIS MoDeTo (*Panagoulis et al., 2018a,c*). The design methodology involves fast and robust design calculations, making use of a 1D FE design framework, modelled by means of Timoshenko beam theory. Soil-structure interaction (SSI) is modelled by non-linear soil reactions applied to the finite elements. Due to the large diameter of the monopiles and the low length-to-diameter ratio that is usually required to comply with the design criteria (*Byrne et al., 2017*), four soil reaction components are employed to model SSI; the lateral soil reaction along the shaft (p), the distributed moment reaction along the shaft (m), the base horizontal force (H_B) and the base moment (M_B). The non-linear functions that relate the soil reaction components (force or moment) to the local pile deformation (displacement, v , or rotation, ψ) are called ‘soil reaction curves’. Figure 1 illustrates schematically the 1D FE model (after *Byrne et al., 2015*), with a design load H applied at height h above the ground level.

The calibration of the four types of soil reaction curves involves normalization and fitting a conic function to each reaction component separately; a procedure known as ‘parameterisation’ (*Panagoulis et al., 2018c*). The selected mathematical function is calibrated by four different parameters. These parameters (four for each of the four soil reaction components) are depth-dependent. The depth-dependency is linear or exponential (depending on the soil reaction component and the soil type: clay or sand) and is described by sixteen continuous depth variation functions (dvf). The latter are derived from the results of a series of 3D FE calculations, i.e. the soil reaction curves are combined all together at various depths before parameterisation. Engineering judgment is needed to define the variation of the geometrical parameters of the 3D calibration models. They should cover an estimated ‘design space’, i.e. a range of monopile diameters (D -range), embedded lengths (L -range), thicknesses (t -range) and load eccentricities (h -range)), within which the actual monopile design is optimised.

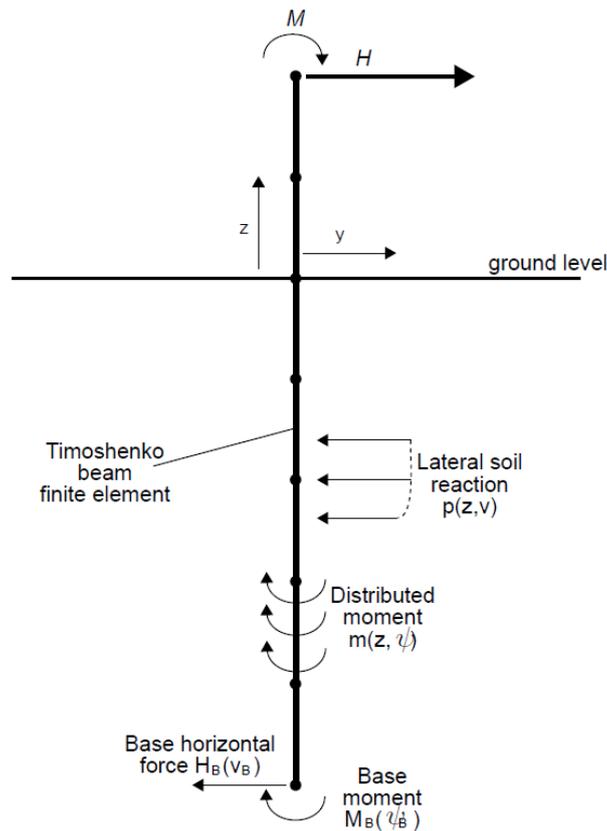


Figure 1. Soil reaction components applied to the 1D FE model (after *Byrne et al., 2015*)

Application to homogeneous soil profiles

The application of the PISA design methodology to homogeneous soil profiles, as implemented in PLAXIS MoDeTo, is discussed by *Panagoulas et al. (2018a,b)* and *Kaltekis et al. (2019)*. Although the soil profile is homogeneous, the design methodology supports the presence of sub-layers to define, as accurately as needed, the variation of the material parameters with depth. Two different soil material types are supported; clay and sand. The soil reaction curves are derived from a series of 3D FE calculations; they are subsequently parameterised to generate the dvf. The dvf parameters are used as input to the 1D FE design model to enable rapid calculations and optimise the monopile geometry for the considered soil and loading conditions, within the assumed design space. The adopted design methodology has been verified for Cowden clay employing the NGI-ADP model (*Minga & Burd, 2019a*) and Dunkirk sand employing the Hardening Soil model with small-strain stiffness (HSsmall; *Minga & Burd, 2019b*).

Application to layered soil profiles

An application of the PISA design methodology to layered soil profiles, as developed in the second phase of the joint industry PISA research project (*Byrne et al., 2019a*) is described below. In this approach, the parameterised soil reaction curves derived from homogeneous soil profiles are employed directly in the 1D FE design model for the analysis and design optimisation of monopiles in layered soil conditions. The approach is demonstrated in the following example.

Consider that a monopile needs to be designed in a layered soil profile consisting of three different soil types; SmatA, SmatB and SmatC, as indicated in Figure 2. Individual hypothetical homogeneous soil profiles are then developed for each of these different soil types, i.e. three in the current example. If the same (or similar) soil type is encountered more than once (at different depths), then a single homogeneous profile for this soil type may be used. In the current example (Figure 2) the three soil types are assumed to be different. Engineering judgment is required to define the variation of the material parameters in the hypothetical homogeneous soil profiles.

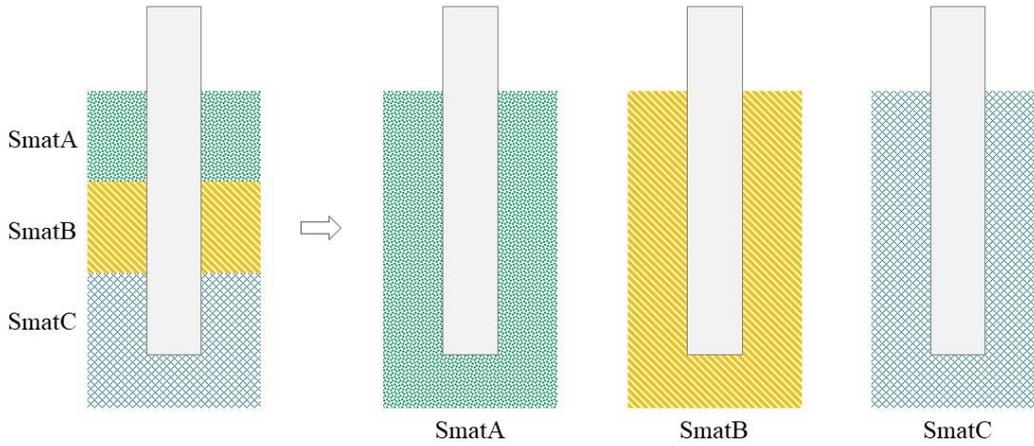


Figure 2. Hypothetical homogeneous soil profiles to represent individual soil types in a layered profile

The soil reaction curves for each soil type are derived from a set of 3D FE calibration models employing the homogeneous soil profiles. A certain design space needs to be defined based on the target layered soil profile. Approximately eight 3D FE models are required per homogeneous soil profile to achieve a suitable calibration of the soil reaction curves (*Panagoulas et al., 2018a,b*). Subsequently, the parameterised soil reaction curves are assigned to the corresponding soil layers in the 1D FE model. The 1D model can then be used to simulate the response of the monopile in layered soil profiles.

Parameterised soil reactions derived from a particular (homogeneous) soil material (e.g. Dunkirk sand with relative density of 90%) can be stored and (re-)used at different sites where the same soil type is encountered, for example, at different locations within a wind farm. Hence, a database of many different sets of parameterised soil reactions (each set linked to a specific soil type) can be created over time. These datasets can be used in any new location where the same (or similar) soil type is encountered. However, in addition to the characteristics of the encountered soils, design engineers should also consider the geometric and loading variations under which the soil reaction curves have been initially generated and parameterised.

DESIGN ASSUMPTIONS

Soil conditions

Two idealized homogenous soil profiles are used to calibrate the 1D FE model in the current example; stiff normally consolidated clay (C) and very dense sand (S). Besides the homogeneous soil profiles, a set of four layered soil profiles is defined; clay over sand (CS), sand over clay (SC), clay over sand over clay (CSC) and sand over clay over sand (SCS). The profiles are illustrated in Figure 3. None of these layered soil profiles correspond to an actual borehole or location. They are based on the authors' judgment as idealised example cases of layered soil profiles that might be encountered on site.

Figures 4 to 6 display the small-strain stiffness (G_0) variation with depth for the six example profiles in Figure 3. In the clay layers, G_0 varies linearly with depth, as described in the following subsection (Equation 3), while in the sand layers a parabolic variation is used, adopting the stress-dependent stiffness formulation of the HSsmall model (*Benz, 2007*), given by Equation 1 (compression is negative). Note that the selected small-strain stiffness of the sand and the clay is of equal order of magnitude.

$$G_0 = G_0^{ref} \left(\frac{c \cos\varphi - \sigma'_3 \sin\varphi}{c \cos\varphi + p_{ref} \sin\varphi} \right)^m \quad (1)$$

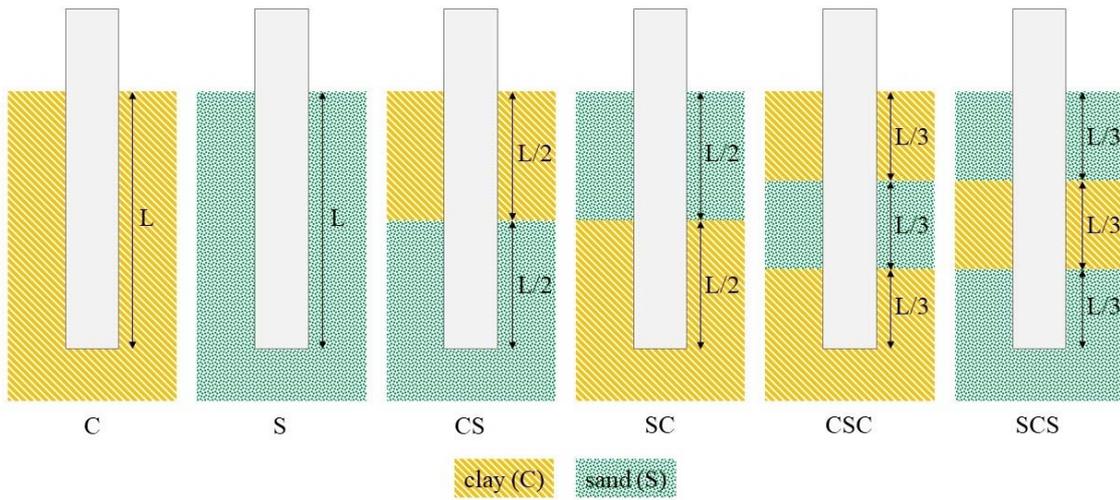


Figure 3. Soil profiles considered in the design example; two homogeneous cases (C, S) and four layered cases (CS, SC, CSC, SCS)

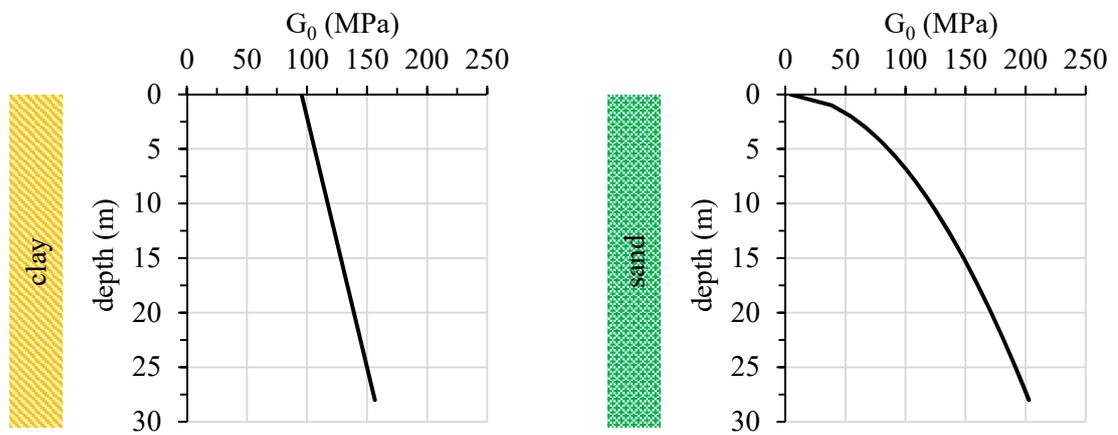


Figure 4. Small-strain stiffness variation with depth of the homogeneous soil profiles (C, S)

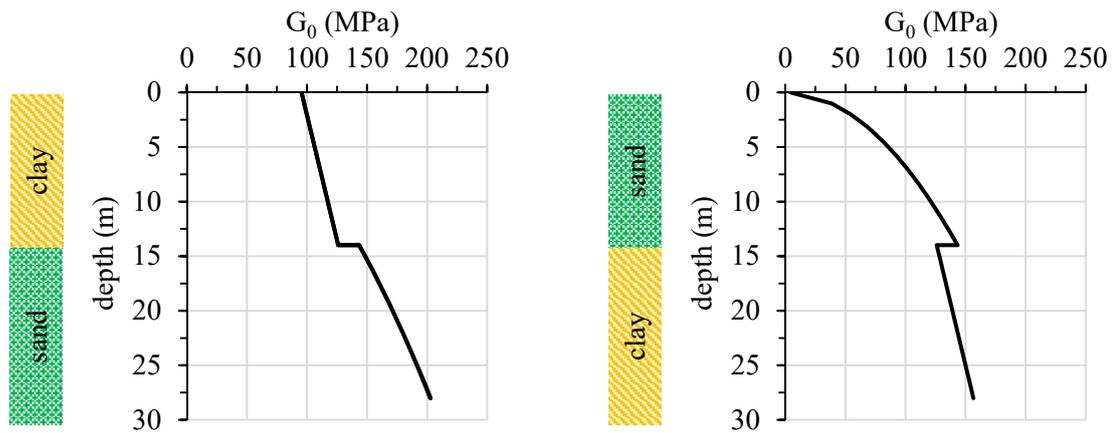


Figure 5. Small-strain stiffness variation with depth of the two-layer soil profiles (CS, SC)

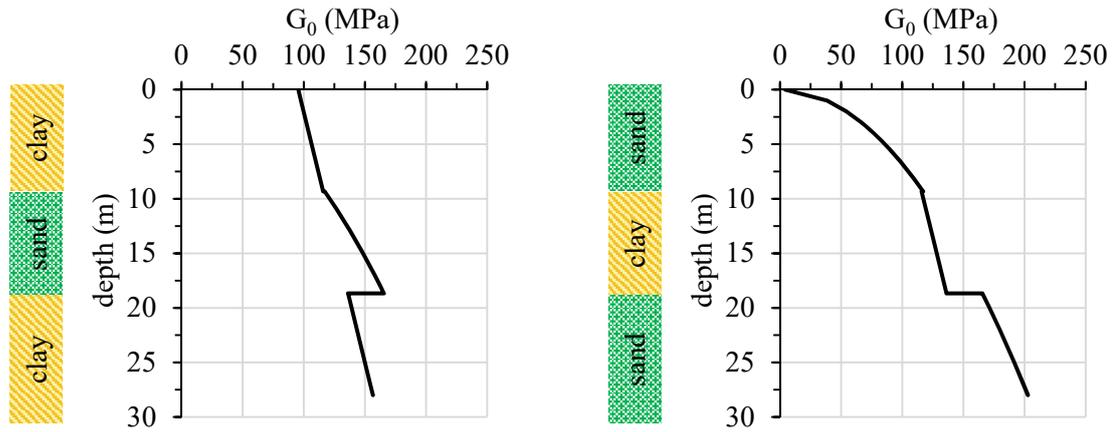


Figure 6. Small-strain stiffness variation with depth of the three-layer soil profiles (CSC, SCS)

Calibration models and final design model

A set of eight 3D FE calibration models (CM1 to CM8) is used for the parameterisation of the soil reaction curves for the homogeneous soil profiles of clay and sand respectively, i.e. sixteen 3D FE models in total. The design space needs to be defined considering the target layered soil profiles. However, a simplified approach is adopted in this study and the assumed geometrical variations are simply derived from the current design practice. The geometrical characteristics of the calibration monopiles are presented in Table 1; h is the height at which the design load is applied above mudline (load eccentricity), L is the embedded length, D is the outer diameter and t is the pile wall thickness.

Table 1. Geometrical properties of the eight calibration models (CM) and the final design model (DM)

| 3D FE model | h (m) | L (m) | D (m) | t (m) | L/D (-) | h/L (-) | D/t (-) |
|-------------|---------|---------|---------|---------|-----------|-----------|-----------|
| CM1 | 20.0 | 20.0 | 5.0 | 0.05 | 4.0 | 1.0 | 100.0 |
| CM2 | 75.0 | 25.0 | 5.0 | 0.05 | 5.0 | 3.0 | 100.0 |
| CM3 | 75.0 | 25.0 | 8.0 | 0.08 | 3.125 | 3.0 | 100.0 |
| CM4 | 35.0 | 35.0 | 8.0 | 0.08 | 4.375 | 1.0 | 100.0 |
| CM5 | 60.0 | 20.0 | 5.0 | 0.05 | 4.0 | 1.0 | 100.0 |
| CM6 | 25.0 | 25.0 | 5.0 | 0.05 | 5.0 | 3.0 | 100.0 |
| CM7 | 25.0 | 25.0 | 8.0 | 0.08 | 3.125 | 3.0 | 100.0 |
| CM8 | 105.0 | 35.0 | 8.0 | 0.08 | 4.375 | 1.0 | 100.0 |
| DM | 56.0 | 28.0 | 7.0 | 0.07 | 4.0 | 2.0 | 100.0 |

The geometry of the assumed final design model (DM) is also presented in Table 1. Note that this model is not included in the calibration procedure but used as an indicative design case, given the presumed soil and loading conditions. The purpose of this paper is to evaluate the performance of the PISA design methodology for layered soil profiles rather than to demonstrate the procedure required to define an optimised design for the considered soil profiles. This design process is discussed by Panagoulas *et al.* (2018b) for a clayey homogeneous soil profile; a similar design philosophy applies to the layered soil profiles. As indicated in the schematic representation of the design space, Figure 7 (left), the geometrical characteristics of the DM fall within the range of geometrical configurations adopted for the calibration models.

Figure 7 (right) depicts the PLAXIS 3D model used for the analyses of the DM under homogeneous soil conditions (sand and clay). Half the monopile is modelled, considering the symmetry of the problem. The finite

element mesh consists of approximately 9500 10-noded quadratic tetrahedral finite elements (Brinkgreve *et al.*, 2018), further refined at the vicinity of the pile. The pile is modelled with 5-noded shell elements according to Mindlin's theory (Bathe, 2014). Weightless shell elements are used focusing on the lateral response of the monopile and neglecting the influence of weight loads. Conventional steel material properties are employed; Young's modulus equal to 210 GPa and Poisson's ratio equal to 0.3. Interface elements are placed on the exterior and below the monopile to allow for the possibility of soil-pile slipping and gapping.

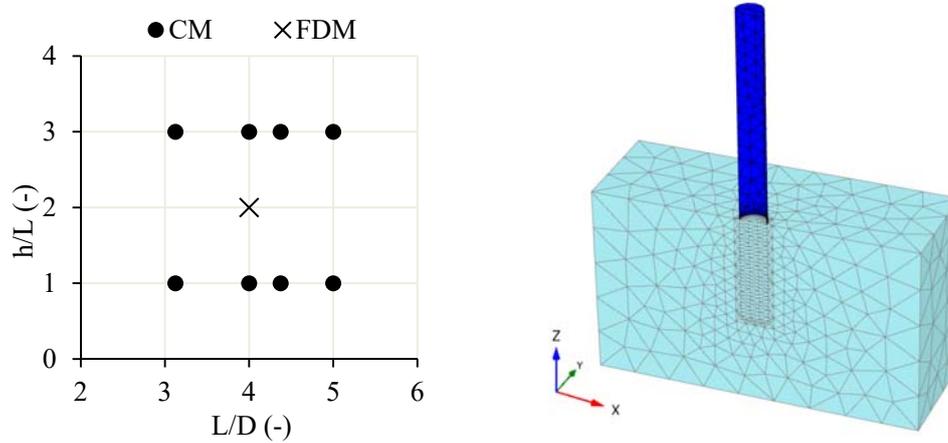


Figure 7. Design space (left) and the DM in PLAXIS 3D for homogeneous soil profiles (right)

Soil constitutive models

The stiff clay layers are modelled by with the NGI-ADP constitutive model (Andersen & Jostad, 1999). Undrained soil conditions are assumed. Three input parameters are used to define the undrained shear strength for three different stress paths, namely active (s_u^A), passive (s_u^P) and direct simple shear (s_u^{DSS}). The active undrained shear strength s_u^A increases linearly with depth, as given by Equation 2. The small-strain shear modulus also varies linearly with depth, defined as a fixed ratio of the active undrained shear strength s_u^A (Equation 3). The other parameters employ empirical correlations (Panagoulas *et al.*, 2018c). The selected constitutive parameters are presented in Table 2. A reduction of 65% is assumed on the strength of the clayey material used at the interface (Palix *et al.*, 2017).

$$s_u^A = 70.0 + 1.6 z \quad (2)$$

$$G_0/s_u^A = 1363.6 \quad (3)$$

Table 2. Material parameters for the NGI-ADP model

| <i>Material parameter</i> | Symbol | Unit | Value |
|--|-------------------|-------------------|--------------|
| <i>Submerged unit weight</i> | γ' | kN/m ³ | 8.0 |
| <i>Small-strain shear modulus over active shear strength</i> | G_0/s_u^A | - | 1364.0 |
| <i>Shear strain at failure in triaxial compression</i> | γ_f^C | % | 4.4 |
| <i>Shear strain at failure in triaxial extension</i> | γ_f^E | % | 8.8 |
| <i>Shear strain at failure in direct simple shear</i> | γ_f^{DSS} | % | 6.6 |
| <i>Passive over active shear strength</i> | s_u^P/s_u^A | - | 0.5 |
| <i>Direct simple shear over active shear strength</i> | s_u^{DSS}/s_u^A | - | 0.75 |
| <i>Initial mobilization of shear stress</i> | τ_0/s_u^A | - | 0.0 |
| <i>Lateral earth pressure coefficient at rest</i> | K_0 | - | 1.0 |

The sand layers are modelled with the HSsmall model (Benz, 2007). The model is appropriate to capture the soil behaviour at small strains, accounting for stress and strain stiffness dependency. Drained soil conditions are employed. A very dense sand material is assumed (relative density of about 90%); the empirical relationships proposed by Brinkgreve *et al.* (2010) are employed to derive the material parameters. The selected values are presented in Table 3. For the soil-pile interface, strength reduction is assumed by adopting an effective friction angle of 29.0 degrees and zero dilation angle (Jardine *et al.*, 2005).

Table 3. Material parameters for the HSsmall model

| <i>Material parameter</i> | Symbol | Unit | Value |
|---|-------------------------------|-------------------|----------------------|
| <i>Submerged unit weight</i> | γ' | kN/m ³ | 10.0 |
| <i>Secant stiffness in standard drained triaxial test</i> | E_{50}^{ref} | kN/m ² | $54.0 \cdot 10^3$ |
| <i>Tangent stiffness for primary oedometer loading</i> | $E_{\text{ocd}}^{\text{ref}}$ | kN/m ² | $54.0 \cdot 10^3$ |
| <i>Un/reloading stiffness from drained triaxial test</i> | $E_{\text{ur}}^{\text{ref}}$ | kN/m ² | $162.0 \cdot 10^3$ |
| <i>Power for stress-level dependency of stiffness</i> | m | - | 0.5 |
| <i>Reference shear modulus at very small strains</i> | G_0^{ref} | kN/m ² | $121.2 \cdot 10^3$ |
| <i>Threshold shear strain</i> | $\gamma_{0.722}$ | - | $0.11 \cdot 10^{-3}$ |
| <i>Effective cohesion</i> | c' | kN/m ² | 0.1 |
| <i>Effective friction angle</i> | ϕ' | deg | 39.0 |
| <i>Dilation angle</i> | ψ | deg | 9.0 |
| <i>Poisson's ratio for un/reloading</i> | ν_{ur} | - | 0.2 |
| <i>Reference stress</i> | p_{ref} | kN/m ² | 100.0 |
| <i>Failure stress ratio</i> | R_f | - | 0.8875 |
| <i>Lateral earth pressure coefficient at rest</i> | K_0 | - | 1.0 |
| <i>Lateral earth pressure coefficient at rest (NC)</i> | K_0^{nc} | - | 0.37 |

RESULTS

Having parameterised the soil reaction curves for the two homogeneous soil profiles, the focus of this section is on the DM (Table 1). The results of the PLAXIS MoDeTo 1D FE model are discussed, considering the two homogenous and the four layered soil profiles (Figure 3). Note that the DM was not part of the calibration procedure, in which only the CM1 to CM8 models were used (Table 1). As a first step, the results are verified against the 1D FE model – referred to as ‘OxPile’ – developed at the University of Oxford and employing the numerical formulations developed during the PISA project (Byrne *et al.*, 2019b; Burd *et al.*, 2019). Subsequently, the results are verified against equivalent 3D FE models in PLAXIS 3D.

Verification against OxPile

A single verification example with OxPile, relating to the two-layer soil profile CS, is presented below. Similar patterns emerged from the comparison between OxPile and PLAXIS MoDeTo 1D FE model for the other profiles. Figure 8 depicts the comparison between the two 1D FE models, for a horizontal reaction force H equal to 18.0 MN and eccentricity h equal to 56.0 m. The results are presented in terms of horizontal reaction force vs. displacement at mudline, reaction moment at mudline vs. rotation at mudline, lateral deflection (v), bending moment in the pile (M), shear force in the pile (Q), lateral soil reaction along the pile (p) and distributed moment reaction along the pile (m). A very good match is obtained between the two models. The boundary between the two soil layers, at depth equal to 14.0 m, is apparent in the Q , p and m plots. In the same

plots the minor differences stem from the fact that the results of the PLAXIS MoDeTo 1D FE model are given at the finite element nodes, whereas the OxPile results are given at the stress points. For the given loading conditions, the pile rotation point is located approximately at 21.5 m depth (0.77% of L). At that depth p changes sign indicating the change in direction of the lateral soil reaction. In addition, m reaches a local minimum as the shear stress acting at the shaft is of very low value. In general, the soil reaction components, p and m , are of higher values in sand, about an order of magnitude difference compared to clay, as the assumed sandy material is of higher strength than the clayey material.

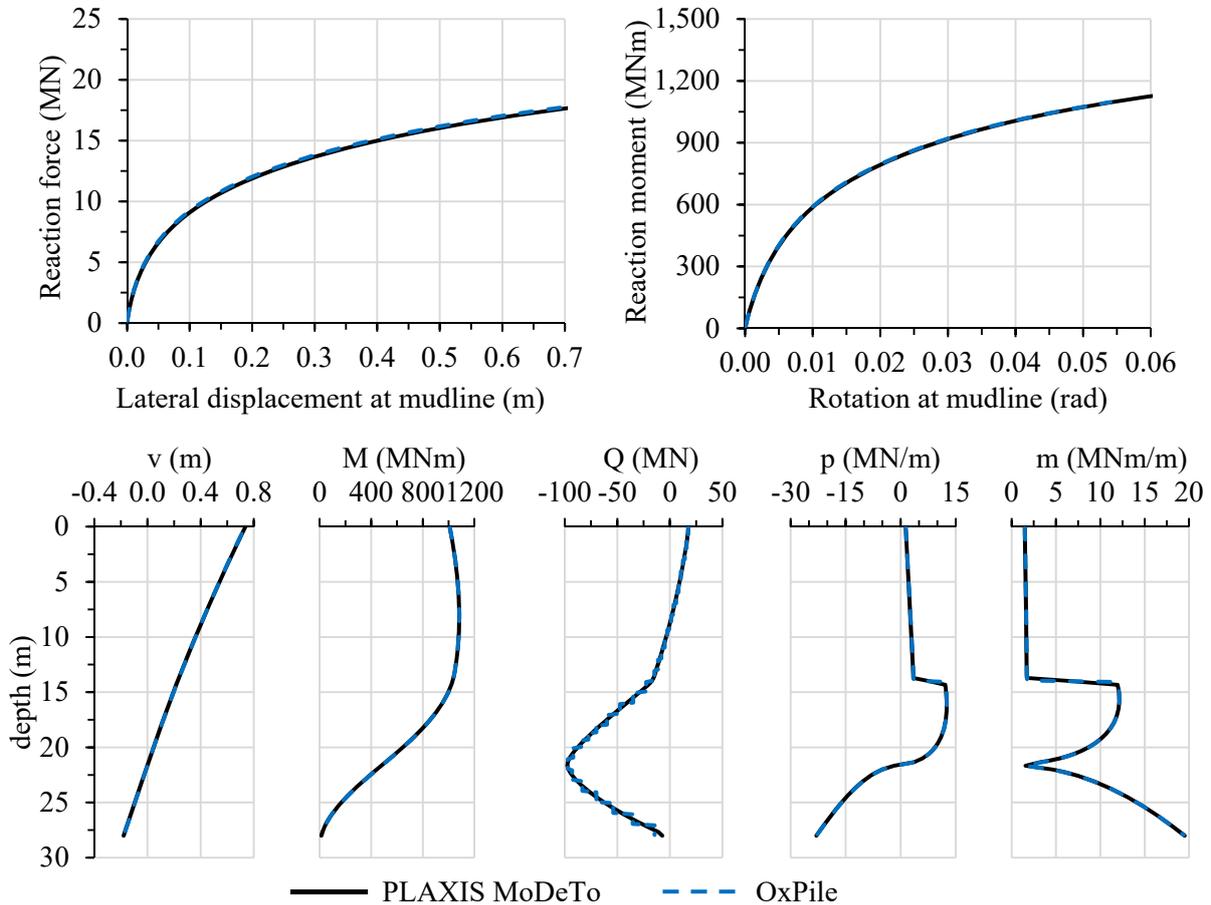


Figure 8. Verification of the PLAXIS MoDeTo 1D model against OxPile for the DM at the soil profile CS

Verification against PLAXIS 3D

The PLAXIS MoDeTo 1D FE results are also verified against equivalent 3D finite element analysis conducted using PLAXIS 3D. Figure 9 presents the comparison between the models in terms of horizontal reaction force vs. lateral displacement at mudline, for all six considered soil profiles. The focus of these calculations is on the large displacement response; the lateral pile displacement at ground level has therefore been taken to $D/10$. The overall response of the 1D FE model is very close to the 3D FE model, not only for the homogeneous soil profiles (C, S), but also for the two-layer (CS, SC) and the three-layer soil profiles (CSC, SCS). The match is very good for the initial stiffness as well as for the ultimate capacity. The accuracy metrics presented in the next section give further insight to the comparison.

Accuracy metrics

The accuracy metric η (η) (Byrne *et al.*, 2019a) is used to compare quantitatively the response of the 1D and 3D models, given by Equation 4. A_{ref} is the area below the reference load-displacement curve (3D FE model) and A_{diff} is the area bounded by the reference curve and the curve which represents the 1D model response.

$$\eta = (A_{ref} - A_{diff})/A_{ref} \quad (4)$$

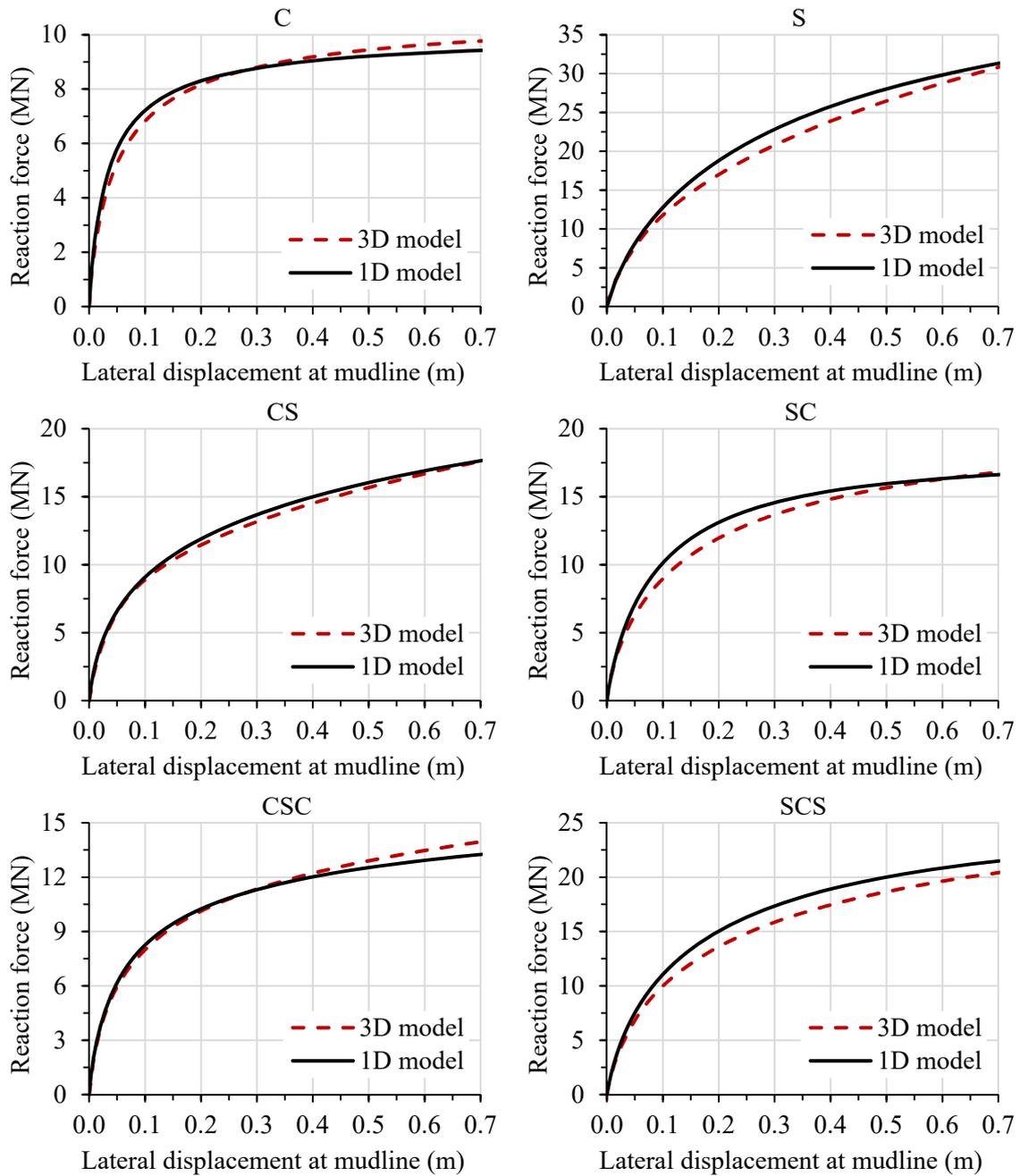


Figure 9. Comparison between the PLAXIS MoDeTo 1D FE model and equivalent PLAXIS 3D models, in terms of the horizontal reaction force vs. lateral displacement at mudline, for the homogeneous (C, S), the two-layer (CS, SC) and the three-layer soil profiles (CSC, SCS)

Considering large displacements at ground level (about $D/10$), Figure 10 (left) presents the results from the comparison between the final design 1D FE model and equivalent 3D FE models, for each case. For comparison, the η values that relate to the 3D calibration models (CM1 to CM8) and the equivalent 1D models, for the homogeneous sand and clay profiles, are shown in Figure 10 (right). For the calibration models (Figure 10, right), all η values exceed 90%, implying a successful calibration in both cases. The average η in sand is about 93%, somewhat lower than the average η in clay, which is about 95%. Overall, a better calibration was achieved in clay as indicated by the values of η for all individual calibration models (CM).

Consistent with these observations, the η value of the DM for the homogeneous clay profile (C) is about 97%, higher than the homogenous sand profile case (S), which is about 93%, as depicted in Figure 10 (left). The η values of the DM for the layered soil profiles are also above 90%; i.e. as good as the homogenous cases. The lowest value is computed for the case SCS (about 92%), as also can be seen in the corresponding load-displacement curve in Figure 9 (1D to 3D load ratio of about 5% at ground displacement equal to $D/10$).

However, the SCS η value is higher than the lowest observed η value, which is obtained during calibration of the CM4 in sand, and equals about 90% (Figure 10, right).

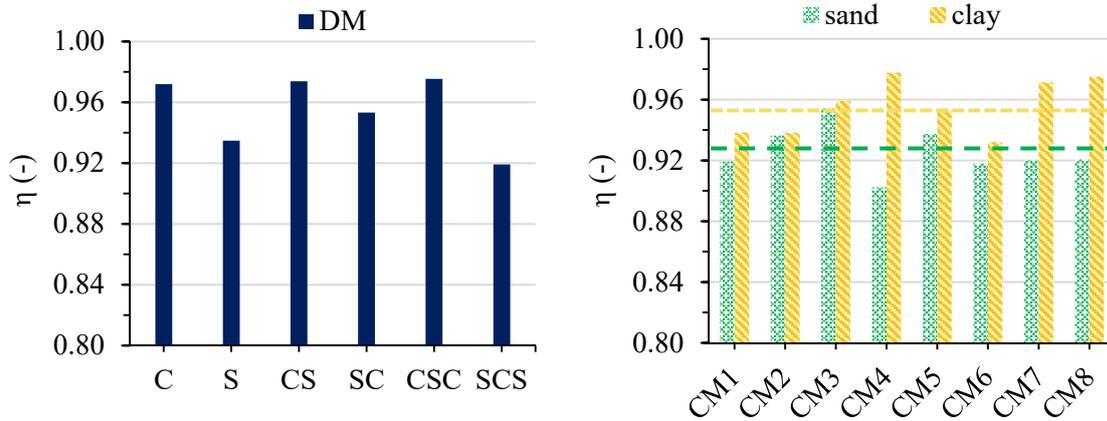


Figure 10. Eta (η) values of the DM in homogeneous and layered soil profiles (left), and the calibration models (CM) in sand and clay (right), considering large displacements ($\sim D/10$)

CONCLUDING REMARKS

In this paper the design methodology based on recent published outcomes of the PISA research project, for laterally-loaded monopile foundations for offshore wind turbines, in layered soil conditions, is discussed. Following the NBD approach, a set of eight 3D FE models is used to parameterise the soil reaction curves for two different homogeneous soil profiles; stiff normally consolidated clay and very dense sand. The fidelity of the calibrated 1D model is checked against the PLAXIS 3D calibration models (CM) for both homogeneous soil profiles. The high values of the accuracy metric η obtained from this process indicate a successful calibration. A particular geometry, not included in the calibration analyses, is adopted for the final design model. The parameterised soil reaction curves are subsequently used to simulate the final design model response in the two homogeneous and four layered soil profiles. The results of the PLAXIS MoDeTo 1D FE are verified against OxPile and PLAXIS 3D. The results indicate a very good match with high values of the accuracy metric η for the PLAXIS 3D comparisons.

These results demonstrate the application of the PISA design approach to layered soils. They indicate that parameterised soil reaction curves determined from homogeneous soil profiles may be combined individually to develop layered soil profiles. The resulting 1D model is able to provide high accuracy simulations of piles embedded in the layered profiles, for monotonic lateral loading. By applying the PISA design methodology in this way, a design process can be developed such that, once parameterised soil reaction curves from various homogeneous soil profiles are available, design engineers can combine them to model layered soil profiles. The resulting model can then be used to conduct design optimisation in a quick and robust manner. The results of the layered soil profiles considered here, despite being limited in number, suggest that a generic design framework could be developed in the future for the design of monopiles in homogeneous and layered soil conditions. However, more cases need to be analysed, considering bigger variety of soil materials and profiles, to develop confidence in the proposed approach.

A known limitation of the presented method, as discussed by *Byrne et al., 2019a*, is related to soil conditions in which very soft soil and very stiff soil layers are combined. In such cases the stronger material is mobilised to a lesser extent than is predicted on the basis of the approach presented here. Additional 3D effects develop in these cases that are not captured by the 1D FE model. However, none of the cases examined in this paper complies with this limitation. Other shortcomings of the suggested design methodology, at present, are related to the modelling of dynamic loading conditions and installation effects, phenomena that are currently being worked on.

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REFERENCES

- Andresen, L., & Jostad, H.P. (1999). Application of an anisotropic hardening model for undrained response of saturated clay. In Proc. Numerical Models in Geomechanics - NUMOG VII. Graz, Austria.
- Bathe, K.J. (2014). Finite element procedures, 2nd edition. Watertown, Massachusetts, United States of America.
- Benz, T. (2007). Small-strain stiffness of soils and its numerical consequences. PhD thesis. Univ. Stuttgart, Inst. f. Geotechnik. Stuttgart, Germany.
- Brinkgreve, R.B.J., Engin, E. & Engin, H.K. (2010). Validation of empirical formulas to derive model parameters for sands. In Proc. 7th NUMGE, Trondheim, Norway.
- Brinkgreve R.B.J., Kumarswamy S. & Swolfs W.M. (2018). PLAXIS manual 2018. Plaxis bv, Delft, the Netherlands.
- Burd H.J., Byrne B.W., McAdam R.A., Houlsby G.T., Martin C.M., Beuckelaers W.J.A.P., Zdravković L., Taborda D.M.G., Potts D.M., Jardine R.J., Gavin K., Doherty P., Igoe D., Skov Gretlund J., Pacheco Andrade J.M., Muir Wood A (2017). Design aspects for monopile foundations. In Proc. TC209 Workshop, 19th International Conference on Soil Mechanics and Geotechnical Engineering (ICSMGE), pp. 35-44, Seoul, Korea.
- Burd H.J., Taborda D.M.G., Zdravković L., Abadie C.N., Byrne B.W., Houlsby G.T., Gavin K., Igoe D., Jardine R.J., Martin C.M., McAdam R.A., Pedro A.M.G., Potts D.M. (2019). PISA Design Model for Monopiles for Offshore Wind Turbines: Application to a Marine Sand. (submitted)
- Byrne B.W., McAdam R., Burd H.J., Houlsby G., Martin C., Zdravković L., Taborda D.M.G., Potts D.M., Jardine R.J., Sideri M., Schroeder F.C. (2015). New design methods for large diameter piles under lateral loading for offshore wind applications. In Proc. 3rd International Symposium on Frontiers in Offshore Geotechnics (ISFOG 2015), Oslo, Norway.
- Byrne B.W., McAdam R.A., Burd H.J., Houlsby G.T., Martin C.M., Beuckelaers W.J.A.P., Zdravković L., Taborda D.M.G., Potts D.M., Jardine R.J., Ushev E., Liu T., Abadias D., Gavin K., Igoe D., Doherty P., Skov Gretlund J., Pacheco Andrade M., Muir Wood A., Schroeder F.C., Turner S., Plummer M.A.L. (2017). PISA: new design methods for offshore wind turbine monopiles. In Proc. Offshore Site Investigation and Geotechnics, SUT, vol. 1, pp. 142-161, London, United Kingdom.
- Byrne B.W., Burd H.J., Kenneth G., Houlsby G.T., Jardine R.J., McAdam R.A., Martin C.M., Potts D.M., Taborda D.M.G., Zdravković L. (2018). PISA: Recent Developments in Offshore Wind Turbine Monopile Design. In Proc. Vietnam Symposium on Advances in Offshore Engineering, Hanoi, Vietnam.
- Byrne B.W., Burd H.J., Zdravković L., Abadie C.N., Houlsby G.T., Jardine R.J., Martin C.M., McAdam R.A., Pacheco Andrade M., Pedro A.M.G., Potts D.M., Taborda D.M.G. (2019a). PISA Design Methods for Offshore Wind Turbine Monopiles. In Proc. Offshore Technology Conference, Houston, Texas, United States of America.
- Byrne B.W., Houlsby G.T., Burd H.J., Gavin K., Igoe D., Jardine R.J., Martin C.M., McAdam R.A., Potts D.M., Taborda D.M.G., Zdravković, L. (2019b). PISA Design Model for Monopiles for Offshore Wind Turbines: Application to a Stiff Glacial Clay Till. (submitted)
- He Y., Byrne B.W., Burd H.J. (2017). Application of a numerical-based design method for laterally loaded monopiles in layered soils. In Proc. Offshore Site Investigation and Geotechnics, SUT, vol. 2, pp. 1116-1123, London, United Kingdom.
- Jardine R.J, Chow F., Overy R. (2005). ICP design methods for driven piles in sands and clays. Thomas Telford, London, United Kingdom.
- Kaltekis K., Panagoulas S., van Dijk B.F.J., Brinkgreve R.B.J., da Silva M.R. (2019). Comparative Concept Design Study of Laterally Loaded Monopiles. WindEurope 2019 Conference & Exhibition, Bilbao, Spain.
- Minga E., Burd H.J. (2019a). Validation of PLAXIS MoDeTo based on the Cowden till PISA field tests. University of Oxford, Oxford, United Kingdom.
- Minga E., Burd H.J. (2019b). Validation of the PLAXIS MoDeTo 1D model for dense sand. University of Oxford, Oxford, United Kingdom.

- Palix E., Willems T., Kay S. (2010). Caisson capacity in clay: VHM resistance envelope – part 1: 3D FEM numerical study. In Proc. 2nd International Symposium on Frontiers in Offshore Geotechnics (ISFOG 2011), pp. 753-758, Perth, Australia.
- Panagoulas S., Hosseini S., Brinkgreve R.B.J. (2018a). An innovative design methodology for offshore wind monopile foundations. In Proc. 26th European Young Geotechnical Engineers Conference, Graz, Austria.
- Panagoulas S., Brinkgreve R.B.J., Minga E., Burd H.J., McAdam R.A. (2018b). Application of the PISA framework to the design of offshore wind turbine monopile foundations. WindEurope 2018 conference at the Global Wind Summit, Hamburg, Germany.
- Panagoulas S., Brinkgreve R.B.J., Zampich L. (2018c). PLAXIS MoDeTo manual 2018. Plaxis bv, Delft, the Netherlands.
- Zdravković L., Jardine R.J., Taborda D.M.G., Abadias D., Burd H.J., Byrne B.W., Gavin K., Houlsby G.T., Igoe D., Liu T., Martin C.M., McAdam R.A., Muir Wood A., Potts D.M., Skov Gretlund J., Ushev E. (2018). Ground characterisation for PISA pile testing and analysis. Géotechnique, in press.