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Abstract

This paper provides an overview of the PISA design model recently developed for laterally loaded offshore wind turbine monopiles through a major European joint-industry academic research project, the PISA Project. The focus was on large diameter, relatively rigid piles, with low length to diameter (L/D) ratios, embedded in clay soils of different strength characteristics, sand soils of different densities and in layered soils combining clays and sands. The resulting design model introduces new procedures for site specific calibration of soil reaction curves that can be applied within a one-dimensional (1D), Winkler-type, computational model. This paper summarises the results and key conclusions from PISA, including design methods for (a) stiff glacial clay till (Cowden till), (b) brittle stiff plastic clay (London clay), (c) soft clay (Bothkennar clay), (d) sand of varying densities (Dunkirk), and, (e) layered profiles (combining soils from (a) to (d)). The results indicate that the homogeneous soil reaction curves applied appropriately for layered profiles in the 1D PISA design model provide a very good fit to the three-dimensional finite element (3D FE) calculations, particularly for profiles relevant to current European offshore wind farm sites. Only a small number of cases, involving soft clay, very dense sand and $L/D =$ 2 monopiles, would appear to require more detailed and bespoke analysis.

Introduction

Offshore wind is central to decarbonising the world economy, with many nations actively planning, designing and constructing substantial numbers of offshore wind farms. The UK, for example, is committed to more than 20 GW of offshore wind operational within the next decade. Currently about 2000 offshore wind turbines have been installed, operating with a capacity of about 8 GW, and a further 360 (2.5 GW) are being constructed. It is anticipated that a further 1200 wind turbines (12 GW) will be installed in the next 5 to 10 years, with a likely capital expenditure of between £30B and £45B. The UK's ambitions will grow in time. Similar ambitions exist across Europe, USA, China, Taiwan, making the worldwide offshore wind market very significant. A central element to an offshore wind turbine is

the foundation and substructure, upon which the tower, nacelle and rotors are located. The foundation, including installation, typically comprises 20% to 30% of the capital costs, representing a very significant component of the overall design. A number of different foundation options have been identified, although the main foundation deployed so far across European waters is the monopile foundation. These are large diameter steel tubes, impact driven into the seafloor, currently 8 m to 10 m in diameter, 30 m to 50 m long, often with a wall thickness exceeding 100 mm and weighing up to 1000 tonnes. A schematic of an offshore wind turbine on a monopile is shown in Figure 1.
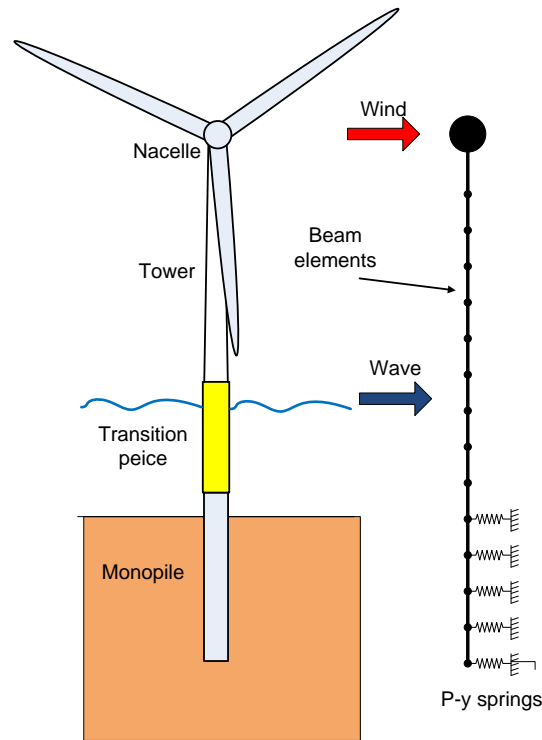

Figure 1: Offshore wind turbine monopile structure and computational model

Offshore wind turbines are typically designed by exploring a very significant number of load cases, to consider the effect of wave conditions, wind conditions, directionality, turbine shut-down, as well as numerous accidental load cases. The number of computations for a single turbine can be large and so efficient ways to capture the wind turbine behaviour, including the foundation-soil interaction, is important, particularly when many turbines must be modelled across a wind farm. The standard method to model the lateral loading of piles involves the use of non-linear springs to represent the lateral soil reaction. A schematic of a simple computational model for an offshore wind turbine is shown on Figure 1, where the turbine tower and foundation is modelled by beam elements and the nacelle/rotor system as a lumped mass and inertia. The soil reactions are defined by springs, with the standard formulation being the non-linear *p-y* models, as defined in API guidance for both sand and clay (API, 2010; see also Matlock, 1970; Reese et al., 1974, 1975; Cox et al., 1974).

Although offshore wind turbine design is concerned with avoiding the ULS condition, it is also critically important that the FLS condition is avoided. In particular the accurate determination of natural frequency for the turbine is needed to assess the fatigue life of the turbine system. For monopile founded turbines, the structural natural frequency is highly dependent on the soil-structure interaction response at the foundation. Measurements of installed turbines around Europe have indicated that measured structural natural frequencies are higher than as designed, indicating the structural and foundation system is far stiffer than assumed, with the greatest amount of uncertainty being the foundation stiffness (see for example Kallehave et al., 2015). Optimisation of the monopile foundation can therefore be

obtained through more accurate characterisation of the soil-structure interaction. To address this design problem the Pile Soil Analysis (PISA) project was established, with scientific work led by University of Oxford in collaboration with Imperial College London and University College Dublin. The close involvement of 10 industry partners, led by Ørsted through the Carbon Trust's Offshore Wind Accelerator, ensured the research focused on industry needs, and has meant a rapid uptake of outputs throughout the project and afterwards. The project has led to the development of the PISA design model which can take account of complex offshore ground conditions to enable site-specific and turbine-specific optimisation. In turn, this leads to lighter weight and more cost-effective structures, along with better quantification of the design risk. This is particularly important considering that an offshore wind farm can span large areas of seabed, in which the geotechnical properties can vary significantly across the site. Rapid calculation approaches that can capture these variations robustly are critical.

Overview of the PISA Project

The PISA design model involves using site-specific three-dimensional finite element (3D FE) calculations to calibrate site-specific one-dimensional finite element models (1D model), as illustrated on Figure 2. The latter are more rapid to compute – necessary for the large number of computations in wind farm design – and can be tuned to capture the fidelity of the 3D FE computation. Two phases of work were completed during the development of the PISA design model, with Phase 1 running from 2013 to 2016 and Phase 2 from 2017 to 2018. Phase 1 involved benchmarking the 3D FE calculations against a series of 28 highly instrumented medium scale field tests at two different sites, to demonstrate the accuracy and robustness of numerical methods in predicting laterally loaded pile behaviour. This gave confidence in the use of the 3D FE method across an extended parameter space, relevant to offshore wind turbine monopiles, to provide information for calibration of the simpler 1D computational model. The 1D model incorporates the standard p-y lateral soil reaction (termed p-v in the PISA design model) but is extended to allow for a distributed moment (down the pile), as well as a horizontal and a moment soil reaction at the pile base. These three additional soil reaction components are found to be important contributors to the overall pile performance, particularly for low L/D ratio monopile foundations. Phase 2 of the PISA project extended the application of the 1D model to layered offshore ground profiles, verified against data from 3D FE analyses of layered profiles.
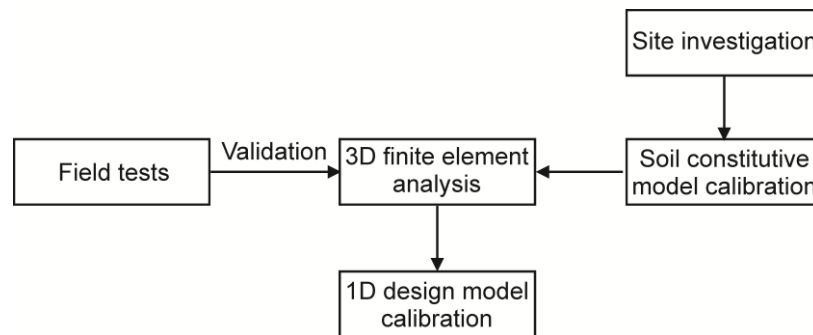

Figure 2: High level strategy adopted during the PISA Project.

Field Testing

A central element to establishing the PISA design method was the execution of high-quality medium scale field pile tests to establish a database of laterally loaded monopile behaviour and to validate the 3D FE numerical models developed for the test sites. Two sites, representative of some offshore North Sea soil conditions, were chosen for the field testing; (a) a site at Dunkirk to represent dense marine sand, and, (b) a site at Cowden to represent stiff glacial clay till. A key feature of both sites was a previous history of site characterisation and pile testing (though principally for axially loaded piles). This enabled rapid progress of the required numerical modelling for the project, given that the calibration of soil

constitutive models could make use of the historical database of in-situ investigation and laboratory element test results. These data were supplemented during the project by an extensive suite of further site investigation including CPTs, seismic CPTs, boreholes and trial pits, as well as a range of advanced laboratory testing, including an extensive triaxial testing campaign (see Zdravković et al., 2019a).

At each site 14 pile tests were completed in the configuration shown by Figure 3(a), comprising of piles at 3 different scales to provide evidence of scaling relationships. At the smallest scale were 4 by 0.273 m diameter piles; these allowed the testing and data-logging systems to be commissioned. The main suite of 8 by 0.763 m diameter pile tests, reacting against a central larger pile, focused mainly on monotonic loading, though several of these piles were subjected to cyclic loading to provide data for future research projects. Figure 3(b) shows a schematic of the layout of these medium pile tests, indicating the wide range of instrumentation deployed, such as fibre optic strain gauges, extensometers, inclinometers, load cells, and displacement sensors. It is noteworthy that this project represented the first widespread deployment of fibre optic strain gauging on impact driven piles, with a very high success rate. At the largest scale 2 by 2.0 m diameter pile tests (one reacting against the other) were completed; a photo of this test at Dunkirk is shown in Figure 4. Uniquely for these tests the loads were applied at a height of 10 m above the ground surface reflecting typical loading patterns for offshore wind turbines, to ensure that the pile kinematics under loading was representative. A very significant amount of high quality data were gathered from the testing at both sites and is now reported in Byrne et al. (2019) and McAdam et al. (2019). The data provide the evidence for the development of new design models, such as the PISA design model, but can also be used for validation of other methods (e.g. Page et al., 2018; Zhang et al., 2019).
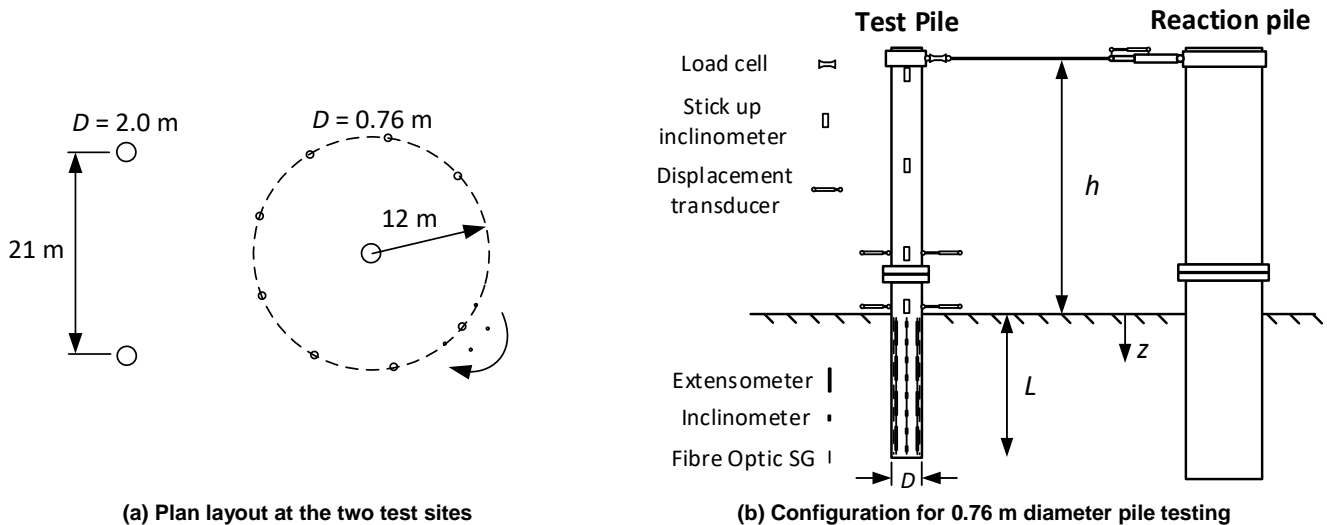

(a) Plan layout at the two test sites   (b) Configuration for 0.76 m diameter pile testing
Figure 3: Schematics of the medium scale field pile testing.

Figure 4: Photo of 2.0 m diameter pile tests at Dunkirk, France.

Numerical Modelling

The inputs for developing the PISA design model were derived from site-specific high-fidelity 3D FE analyses, performed using the FE code ICFEP (Potts & Zdravković, 1999) and following the overarching strategy outlined in Figure 5. Particular attention was placed on integrating laboratory and field data to characterise the soil behaviour and initial ground conditions, in conjunction with the selection of appropriate soil constitutive models capable of reproducing both ultimate and operational states of soil response.
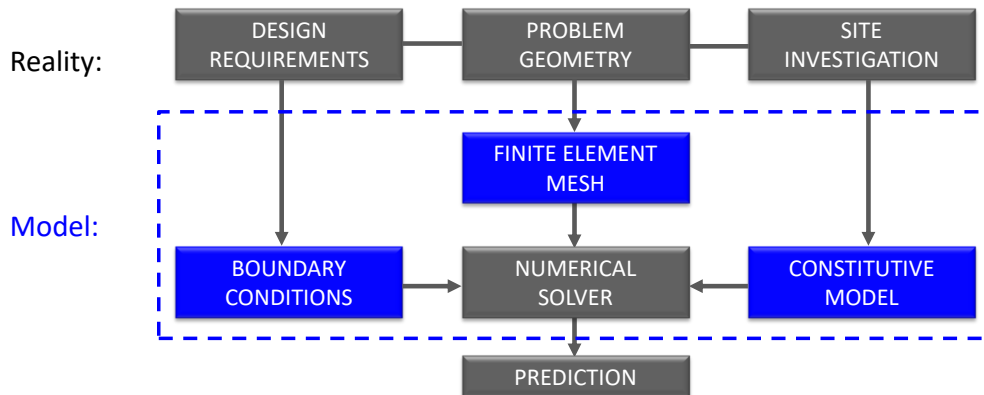

Figure 5: Numerical modelling approach.

For the clays studied in the project, the constitutive model adopts elements of the original modified Cam clay model formulation (Roscoe and Burland, 1968), but is enhanced with (a) a nonlinear Hvorslev-type surface (Tsiampousi et al., 2013) to capture accurately the undrained strength of Cowden till and London clay dry of critical; (b) a generalised shape for the yield and plastic potential surfaces in the deviatoric plane (Van Eekelen, 1980) to account for the effect of the intermediate principal stress, as evidenced by the different strengths in triaxial compression and extension; and (c) a non-linear small strain model for simulating the non-linear variation of the elastic shear modulus with mean effective stress and deformation level (Taborda and Zdravković, 2012; Taborda et al., 2016). In the case of London clay, which exhibits pronounced stiffness anisotropy at small strains, the small strain model of Taborda et al. (2016) was further enhanced by the inclusion of the three-parameter model of Graham and Houlsby (1983), to account for small strain stiffness anisotropy.

For sand the state parameter-based bounding surface plasticity model is used, as described in Taborda et al. (2014), which is itself an evolution of the model originally proposed by Manzari and Dafalias

(1997). For elastic response in sand the model adopts a non-linear elastic Ramberg-Osgood (1943) type stiffness degradation. Careful calibration of the constitutive models was carried out prior to the field test program, and a set of calculations for each test configuration was completed a-priori. For each site a single ground model was established as the basis for all calculations at that site. Variations in soil properties local to each pile were not accounted for in the modelling.

The boundary conditions were established by reference to the field testing configurations, and an appropriate finite element model was developed, with particular attention paid to the modelling of the pile-soil interface. This included special interface elements (Day and Potts, 1994) that can reproduce opening of a gap around a laterally loaded pile, which was evidenced at both test sites. An example comparison of the 3D FE output to the field tests for the load-displacement response at ground level is shown in Figure 6 for the 2.0 m diameter pile test at Cowden. The computation shows exceptionally good agreement at both small displacements and at large displacements, providing reassuring evidence of the robustness of this calculation method for predicting monopile behaviour. Similar comparisons were found for all the field testing completed, including at different scales (diameters of 0.763 m and 2.0 m) providing reassurance on scaling the method to full scale monopiles (e.g. 10 m diameter).
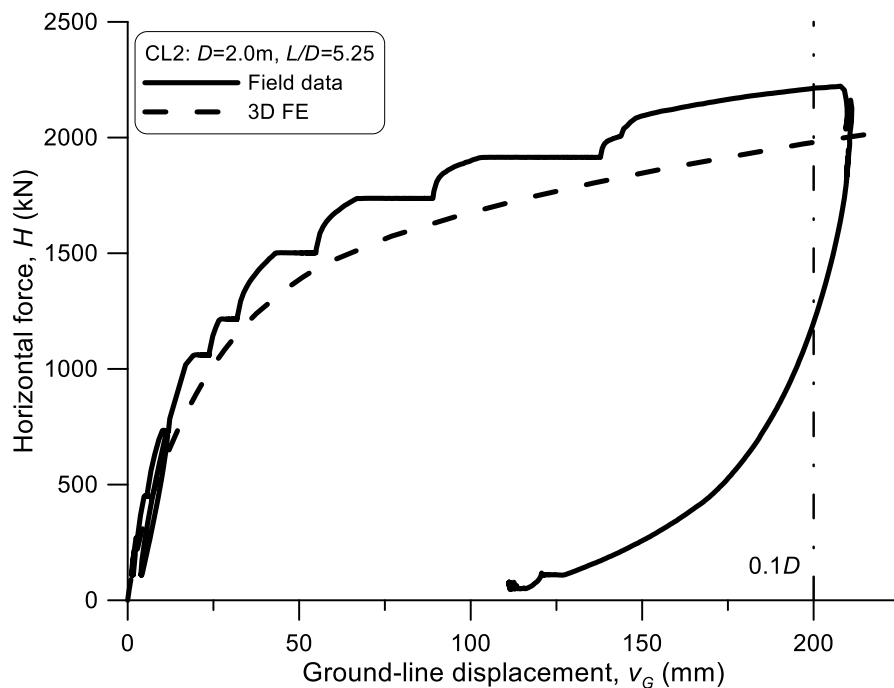

Figure 6: Comparison of finite element results to field test measurements at Cowden.

Further details about the numerical modelling and the comparisons with the field testing can be found in Zdravković et al. (2015), Byrne et al. (2017), Burd et al. (2017), Taborda et al. (2019) and Zdravković et al. (2019b).

PISA Design Model

The numerical modelling identified that, for low L/D ratio piles under lateral loading, the failure mode is more complex than assumed with the traditional p-y method. In particular there are further soil reactions, in addition to the lateral reaction. As illustrated in Figure 7, four separate soil reaction components are considered at the soil-pile interface: (i) distributed lateral loads (ii) vertical shear tractions combining to form a distributed moment (iii) a horizontal force at the pile base and (iv) a moment at the pile base. Each of the soil reaction components is related in the model to the local lateral displacement or rotation (i.e. adopting a 'Winkler' approach) by a calibrated parametric function referred to as a 'soil reaction curve'. In the computational implementation (the 1D model illustrated in Figure 7(b)) the pile is

represented as an embedded beam, which is best approximated by Timoshenko beam theory. A distributed lateral load p and a distributed moment m are assumed to act along its length. The distributed moment represents the moment associated with the vertical shear tractions induced at the soil-pile interface. Additionally, a horizontal force H_B and a moment M_B act on the pile base.
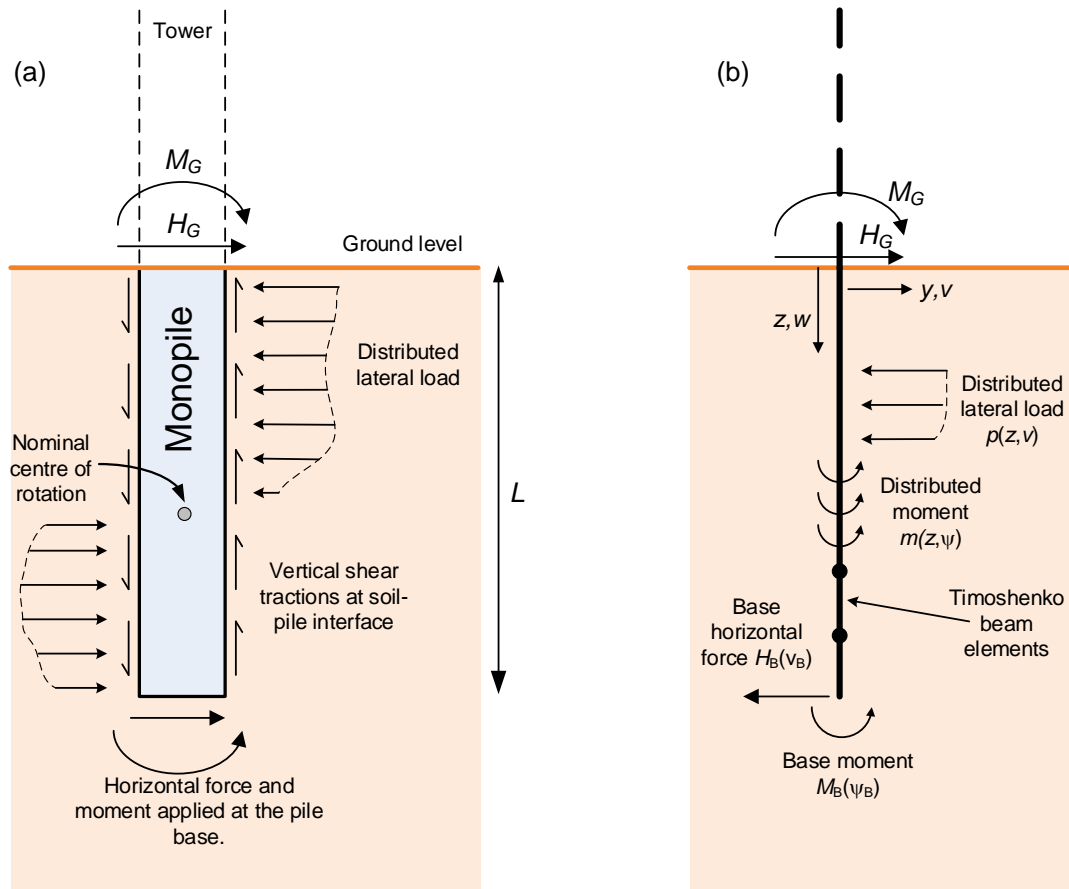

Figure 7: (a) Idealisation of the soil reaction components acting on a monopile (b) 1D finite element implementation of the PISA design model. Note that the reactions are depicted in Figure (a) as acting in the expected direction. In Figure (b) the reactions are shown in directions that are consistent with the coordinate directions shown.

During Phase 1 of the PISA project a method for calibrating these soil reaction curves for site specific conditions was developed. Figure 8 shows a high level overview of the calibration process; the site specific conditions are encoded within the 3D FE model via the calibration of the constitutive model and the problem boundary conditions. It is possible to interrogate the outputs for each 3D FE calculation to derive the detailed stress/strain behaviour at an element level at the soil/pile interface, from which the local soil reaction responses can be defined at any depth, for that calculation. These responses can be normalised, using appropriate dimensionless groups, and then parameterised. The distribution of parameters with depth can then be abstracted for future input into the 1D model.
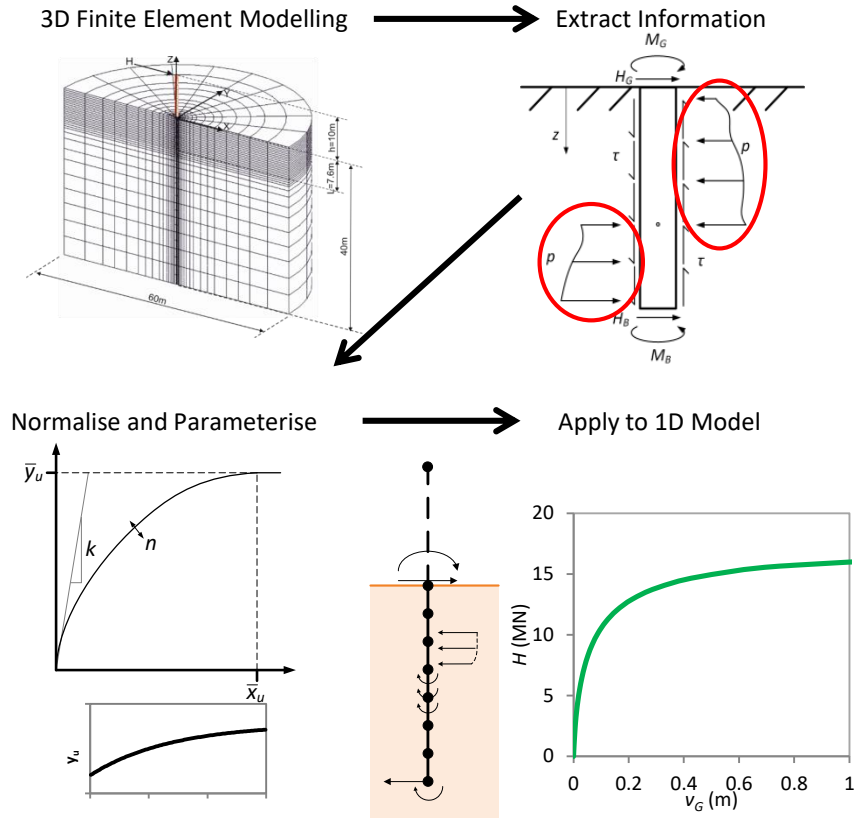
3D Finite Element Modelling → Extract Information

Normalise and Parameterise → Apply to 1D Model

Figure 8: High level calibration process

Figure 9 shows this calibration process in more detail, taking the lateral reaction for clay soil as an example. Firstly, the raw lateral soil reaction curves, designated as *p-v* responses in the PISA design model, are obtained with depth from the 3D FE calculation for a single pile. A normalisation is completed by dividing (for these variables in the clay PISA design model framework) p by $s_u D$ and v by D/I_R, where I_R is the rigidity index given by G/s_u and G is the small strain shear modulus. A base function is then adopted that captures the essential non-linear shape of the abstracted *p-v* curves, and the function is fitted using a least-squares regression process.

Raw → Normalised → Simplified Shape → Fitted

Soil Reaction Values (One Pile)

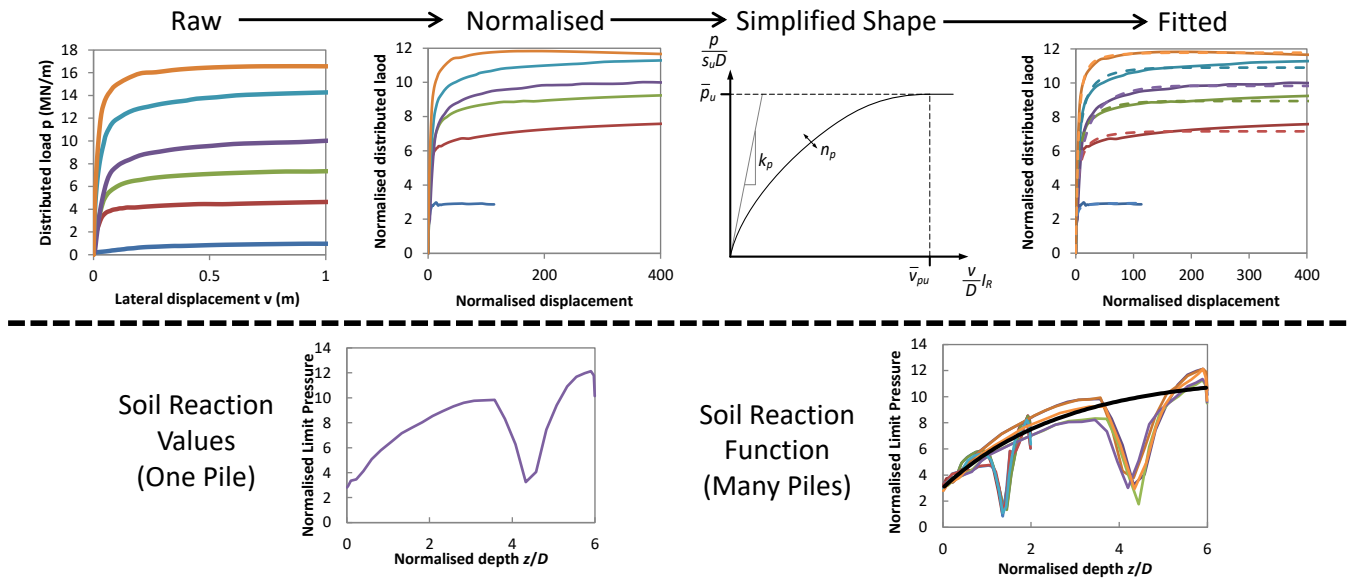Soil Reaction Function (Many Piles)

Figure 9: Detailed calibration, in this instance for the lateral soil reaction in clay.

The base function adopted in PISA is determined by 4 parameters, and so the distribution of these parameters with depth for an individual pile can be found. This is shown in Figure 9 for the normalised limit pressure. The results can then be aggregated across a number of different pile calculations that make up the calibration space, to identify the appropriate parameter variation. For example, Figure 9 shows the normalised limit pressure varying with depth from about 3 at the surface to about 10 at a depth of 6 diameters, similarly to other work on this topic (e.g. Murff, 1993; Jeanjean et al. 2017; see also Byrne et al., 2017, and Byrne et al., 2019). The deduced parametric expressions can then be applied in the 1D PISA design model framework to produce calculations of pile response.

The calibration process for any particular soil profile will depend on the number of 3D FE calculations completed and the parameter space covered, including the variation of soil properties, the pile geometry and loading conditions. The resulting 1D model formulation can then be used to interpolate across the parameter space to make predictions of pile behaviour, though caution must be exercised in extrapolating outside of the calibration space. This process was completed during Phase 1 for soils representing the Cowden and Dunkirk sites, using 3D FE models verified by the field testing process. A total of 30 3D FE calculations were performed for this Phase of the project. Byrne et al. (2019) and Burd et al. (2019) describe the detailed PISA design model formulation for stiff glacial clay till and dense marine sand respectively.

An example calculation using the 1D model for Cowden glacial till is shown in Figure 10, at both small and large displacements. This plot shows the 3D FE result along with the output from the 1D (Parametric) model, which is the PISA design model for Cowden till. Also shown is the 1D (Numerical) result which is a 1D model computation that re-applies the abstracted numerical soil reaction curves from the 3D FE. The (small) difference between 1D (Numerical) curve and the 3D FE results represents the shortcomings in approximating (a) the pile behaviour through a 4 component simplified Winkler model and (b) the monopile as a Timoshenko beam.

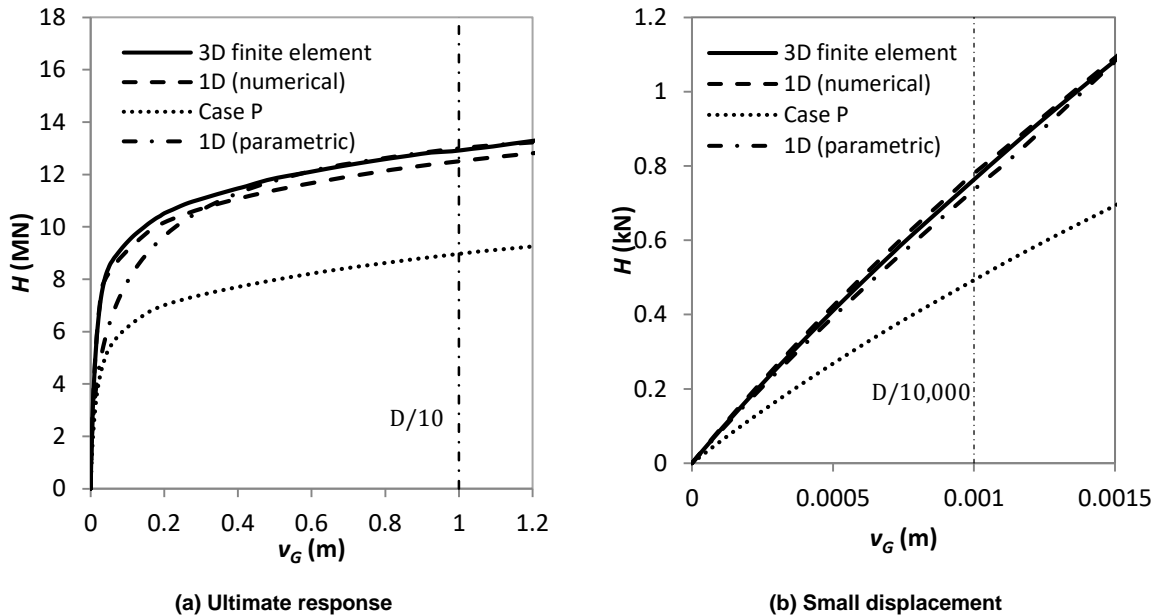(a) Ultimate response   (b) Small displacement

Figure 10: Comparison of load-ground level displacement response determined from the 3D calibration analyses and the corresponding results obtained using the 1D model employing the numerical and parametric soil reaction curves (D = 10m, L = 20m). Case P refers to the case where the distributed lateral soil reactions only are included in the 1D model

Also shown is a calculation that only uses the numerical $p\text{-}v$ soil reaction response (Case P), indicating the importance of including, for this pile geometry, the other 3 soil reactions. The relative importance of these terms depends on the pile geometry and rigidity, with short rigid piles requiring 4

components and long slender piles likely requiring just the *p-v* component for acceptable accuracy. Overall the 1D (Parametric) model demonstrates an excellent agreement with 3D FE calculation at small and large displacements, although in this instance the curvature at intermediate displacements is less well captured. This could be remedied by adopting a different base function for the parameterisation, in which more attention is placed to capturing this transition. It should be noted that different models could be produced by weighting the fitting to different parts of the curve, and whilst PISA focused on capturing the full range of behaviour it may be appropriate in future calibration exercises to only focus on small displacement or on large displacement, depending on the design problem being addressed.

To determine the effectiveness of the PISA design model two different metrics were defined (see Figure 11). The first accuracy metric is defined as $\eta = \frac{A_{ref} - A_{diff}}{A_{ref}}$, where A_{ref} is the area below the reference load-displacement curve (the 3D FE calculation) and A_{diff} is the area of the region delimited by the reference and the simulated curve (the 1D model output). A second metric is simply the ratio of loads of the 1D model to the 3D FE analysis at a given displacement (the ratio metric, ρ), where small displacement is defined as a ground level pile displacement of D/10,000 and ultimate displacement as a ground level pile displacement of D/10. Accuracy metrics for the Cowden glacial till and Dunkirk $D_R =$ 75% sand are shown in Table 1, demonstrating that the 1D model approximates the 3D FE calibration data very well.

Calculation Class	N	η_{sd}	η_{ult}	ρ_{sd}	ρ_{ult}
PISA Phase 1 (Homogeneous)					
Cowden	14	0.97	0.95	1.00	1.03
Dunkirk D_R=75%	13	0.96	0.96	0.99	0.97
PISA Phase 2 (Homogeneous)					
Bothkennar	12	0.94	0.99	1.04	0.98
London clay	9	0.95	0.95	0.98	1.06
Dunkirk D_R=45%	12	0.96	0.96	1.01	1.04
Dunkirk D_R=60%	11	0.96	0.97	0.99	1.01
Dunkirk D_R=90%	12	0.96	0.97	0.97	1.00
PISA Phase 2 Layered					
Layered, Cases A0	4	0.97	0.96	1.00	1.00
Layered, Cases A	4	0.97	0.95	1.02	0.96
Layered, Cases B	9	0.96	0.93	1.04	1.02
Layered, Cases C	9	0.95	0.94	1.06	0.96
Layered, Cases D	2	0.97	0.91	1.01	1.13
Layered, Cases E	3	0.96	0.98	1.01	0.98
Layered, Cases F	2	0.99	0.96	0.99	1.06
Layered, Cases PC	6	0.96	0.86	0.97	1.08
Homogeneous, All Sand	51	0.96	0.96	0.99	1.00
Homogeneous, All Clay	35	0.96	0.96	1.01	1.02
Layered	39	0.96	0.93	1.02	1.01
All Calculations	125	0.96	0.95	1.01	1.01
CoV (All Calculations)				4.7%	7.1%

Table 1: Accuracy metrics for the different calculations completed during the PISA Project.

(a) Ultimate response   (b) Small displacement response

Figure 11: Definition of accuracy metrics
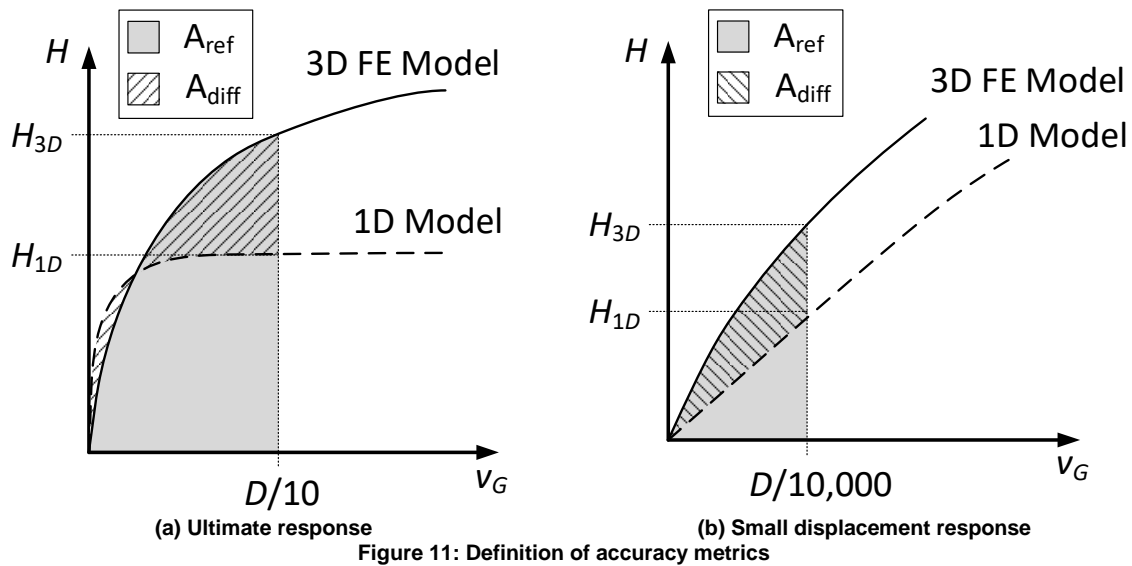
A further test of the model is to apply it to a design case within the parameter space, but not included in the calibration set, and this is shown in Figure 12. Subsequently a 3D FE calculation is computed to demonstrate the goodness of fit of the 1D model. Again an excellent result is obtained, providing reassurance of the robustness of the modelling approach for prediction of pile performance.
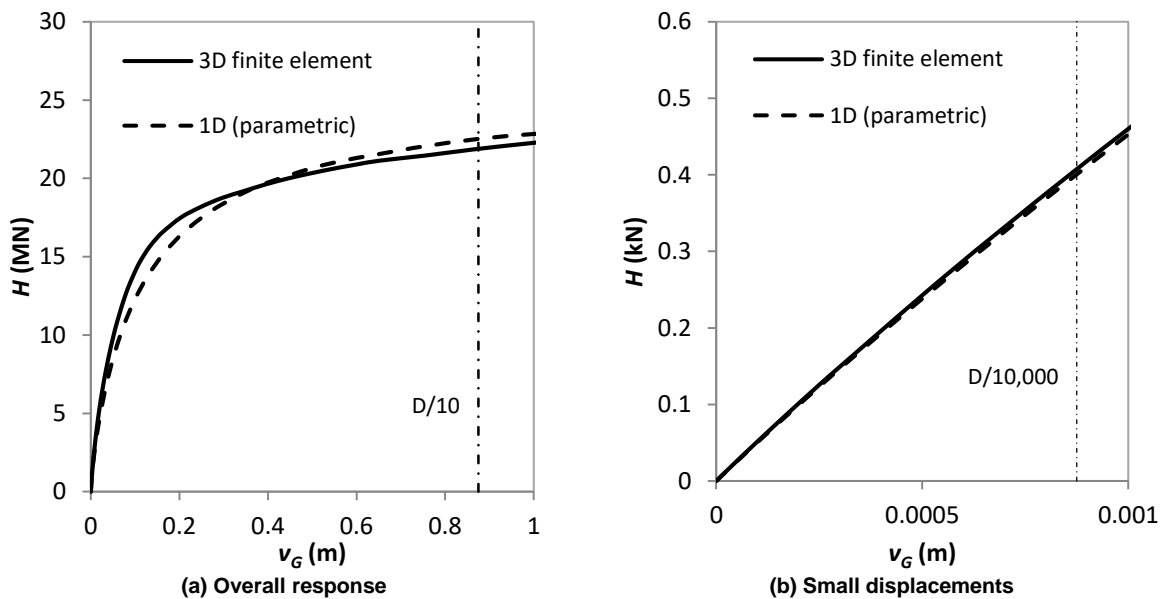

(a) Overall response   (b) Small displacements

Figure 12: Comparison of load-displacement response at ground level from the 3D finite element calibration calculations and the 1D model simulations using calibrated soil reactions for a design case (D = 8.75 m, L/D = 4).

Generalisation to a Design Method

The evidence obtained during Phase 1 of the PISA project has led to a more advanced one-dimensional (1D) pile analysis method for optimised design, involving the definition of case-specific soil reaction curves. During the PISA project these reaction curves were defined through finite element analyses, though they could, in principle, be defined from analytical approaches or taken from existing formulations. Figure 13 sets out two alternative processes that may be followed to define the sets of soil reaction curves.
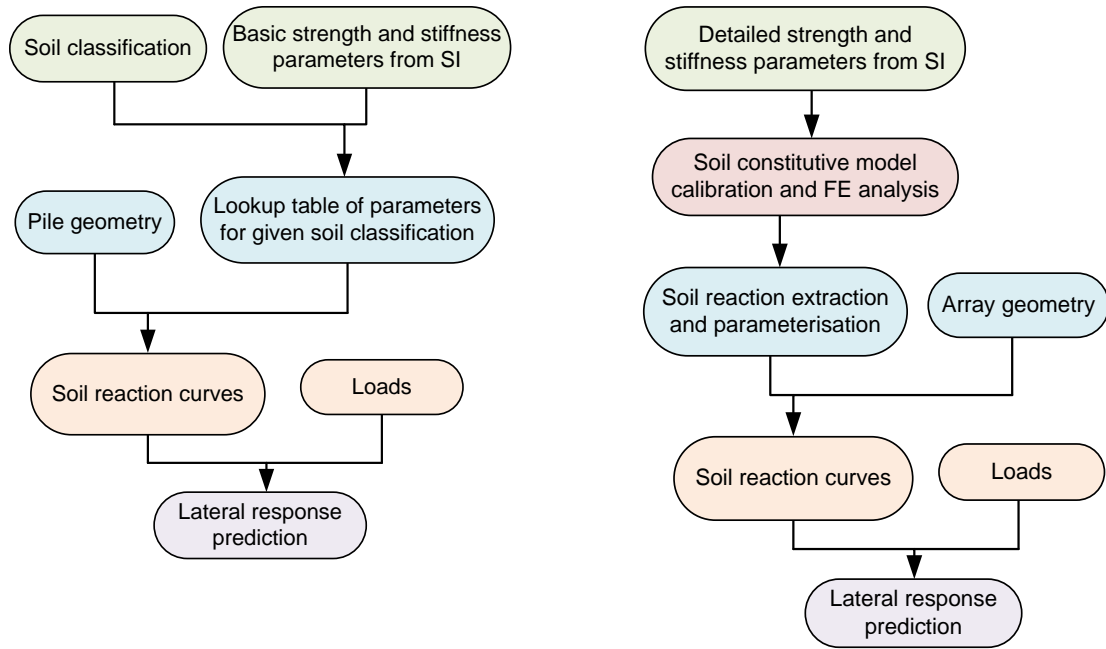
Figure 13: Application modes for the design method: (a) rule-based method, and (b) numerical-based method.

A. *Rule-Based Method*: This approach employs pre-defined curves that are either in the wider literature or may be derived through analytical methods. Examples of such curves include existing published formulations for the *p-y* method (e.g. Jeanjean et al., 2017; Zhang et al., 2019) or new bespoke sets of all four soil reaction curves for glacial clay (Byrne et al., 2019) and marine dense sand (Burd et al., 2019) that are linked to soil parameters acquired through appropriate site investigations.

B. *Numerical-Based Method*: This approach adopts finite element calculations to generate and calibrate site-specific or regional soil type reaction curves that can be used in the 1D model calculations. This requires:

(i) A characterisation of the site ground model and the properties of the soils present that is sufficiently thorough to inform accurate non-linear modelling of their operational and ultimate behaviour. This usually requires detailed knowledge of undrained strength in clays, relative density and CPT cone resistance in sands and reliable information on soil stiffness.

(ii) Specification of the site investigation and laboratory testing program to provide the data required for (i).

(iii) Development of an appropriate numerical model, comprising the selection and calibration of a suitable soil constitutive model and the characterisation of the initial and boundary conditions, used for the finite element calibration analyses, based on the information obtained in (ii)

(iv) Specification of the design parameter space for the finite element analyses, within which the soil reaction curves will be calibrated (e.g. pile geometry, loading conditions, displacement range), and over which the calibrated 1D model is valid.

(v) A choice of the functional form of the non-linear curves used as the base function for the soil reaction curves, calibrated via data abstracted from the finite element analyses.

(vi) A procedure for abstracting the soil reaction curve data from the finite element analyses, and providing an optimisation of fit of the soil reaction curve base function against the abstracted data from the finite element analyses.

(vii) Development of a scaling procedure that allows the soil properties from (i) to be input to (v) and (vi) so that the derived soil reaction curves can be more widely applied in the simplified 1D design model across the proposed design space.

Option A may be adopted for design where site investigation data are limited, or at an early stage of the design process. Option B may be incorporated as part of the detailed design procedure, either using bespoke software (e.g. by extracting soil reaction curves from FE calculations such as from ICFEP or Abaqus) or through commercial software (e.g. PLAXIS MoDeTo). Implementation of Option B at different sites will increase, over time, the spread of cases for which calibrated soil reaction curves are available to apply in Option A. Further details of the method can be found in Byrne et al. (2015), Byrne et al. (2017), Burd et al. (2017), Byrne et al. (2019) and Burd et al. (2019).

The PISA project applied this process specifically to offshore wind turbine monopiles. However, the process can be readily applied more generally to other problems. In particular it could be applied to oil and gas pile design, to suction caisson design (e.g. Suryasentana, 2018) and to other soil-structure interaction problems whether offshore or onshore related.

Application of Generalised Design Method to Homogeneous Soils

Phase 1 of the PISA project was concerned with the development of 1D model parameters for two materials, approximating monopile behaviour in stiff glacial clay at Cowden and dense marine sand at Dunkirk. Phase 2 of the work applied the generalised PISA design model to additional homogeneous soil profiles including Bothkennar clay (an example of a soft clay), London clay (a brittle stiff plastic clay) and a marine sand, based on the Dunkirk sand, for a further three different values of relative density. This work involved a detailed calibration and validation exercise based on existing information from the literature and previous research, but without any further field testing or additional laboratory element testing. Following the calibration of the constitutive models, an investigation across an appropriate parameter space of pile behaviour in the homogeneous soils was completed, including both calibration calculations (defining the models) and design calculations (testing the models). This involved a total of 66 new 3D FE calculations. The geometry space targeted in the study followed that from the PISA Phase 1 work, to enable a robust set of data to be developed, further enhancing the assembled database.

Soil reaction curves for the 1D model were successfully extracted from the 3D FE calculations following the established procedures, allowing similarities to be drawn across different soil conditions, as well as identifying differences. The new sand models (for different relative densities) compared well with that originally developed in PISA Phase 1 and formed the basis for developing a general Dunkirk sand model for arbitrary relative densities. This model allows pile behaviour to be predicted for sand relative density varying from 45% to 90%, covering a wide range of possible offshore soil conditions. Throughout the process different optimisation techniques were explored to identify the best set of soil reaction curves for different soils, including for the general Dunkirk sand model. Both sand and clay results indicate that a number of the 1D soil parameters are similar between soil profiles, giving confidence that a more general approach to determining the soil reaction curves may be possible in the future. The identified soil reaction curves were tested against new design scenario calculations, with considerable success.

Application of PISA Design Models to Layered Soils

During Phase 1 of the PISA project the scope was deliberately restricted to two homogeneous soil systems that represented available, well-characterised, sites for the field testing necessary for method validation. Therefore, the work focused on stiff over-consolidated ductile glacial clay found at Cowden and dense to very dense sand found at Dunkirk. Although these materials are similar to soils at some offshore wind farm sites, they are not representative of all sites. Importantly very few wind farm sites consist of homogeneous soils, with many showing complex layering systems, of varying strengths and stiffness.

The Phase 2 work for the PISA project explored application of the 1D model, calibrated using the homogeneous soil profiles, to analyse monopiles embedded in layered soils. A total of 39 by 3D FE calculations were completed, targeting a wide range of layered soil conditions. The layering stratigraphy explored was identified in consultation with the PISA industry partners and was intended to represent realistic soil profiles found at offshore wind farm sites. Figure 14 gives an indication of the different soil profiles targeted during this study. Profile A and C are two-layer systems, representing the simplest possible layering structures. Profile A has the pile $L/2$ into the bottom layer, and explored sand over clay, clay over sand, soft over stiff clay, whilst Profile C explores the effect of the pile just tipping into a stiffer bottom layer. Profile B focuses on a thin layer which exists at $L/3$ below the ground surface. An initial study identified that a thin layer above the pile rotation point (typically is located at a depth of $2L/3$ to $0.7L$) would have most effect on the pile behaviour, particularly if the layer was very stiff or soft by comparison with the other material. Profile D is a combination of Profile B and C, whilst Profile E and F represent more realistic multi-layering structures. The main pile geometry targeted for these calculations had $L/D = 4$ to be within the calibration space used for defining homogeneous soil reactions ($2 < L/D < 6$), with a small number of calculations completed for L/D of 2 and 6. A final set of "Proof of Concept" (defined as PC) calculations targeted multi-layer profiles, of extreme variation of strength and stiffness, and $L/D = 2$, representing a significant challenge to the modelling.
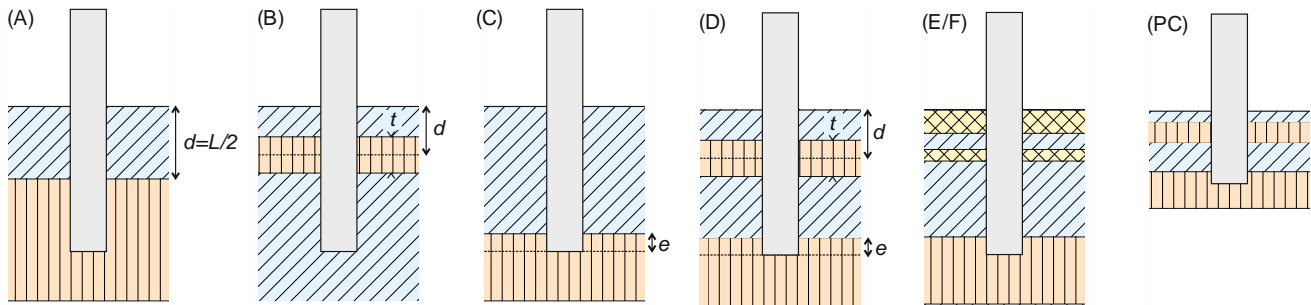

Figure 14: The layered soil configurations: (A) two layer system (B) thin layer system (C) stiff base layer system (D) conceptual multi-layer system (E/F) practical multi-layer system (PC) proof of concept system

Layered Soils Hypothesis

The central hypothesis behind the PISA layered soil work is that *soil reaction curves calibrated using homogeneous soil profiles can be employed, directly, to conduct 1D analyses of monopiles embedded in a layered soil*. The work was structured to explore this hypothesis through a detailed comparison of 3D FE calculations and 1D model calculations for appropriate layering configurations. In assessing the ability of the 1D model to reproduce the behaviour of a monopile computed using 3D FE, consideration was also given to the below-ground bending moments (which are relevant to fatigue calculations as well as pile wall yield ULS checks) as well as the overall H-v_G (load - ground level pile displacement) performance. The consideration of the match between the 1D model and 3D FE embedded bending moments represented an extension on the PISA Phase 1 work.

Results

An example multi-layer calculation (Case F1) is shown in Figure 15, which involves a wide range of layering along the embedded length of the pile. This includes a wide range of strength and stiffness variations. The computed response from the 1D model, following the layered soil hypothesis, agrees exceptionally well with the response from the 3D FE calculation. The computed accuracy metrics are high for both the small displacement response and the ultimate capacity. There is a very slight divergence at large displacement, but this is within the accuracy of the homogeneous soils reaction curve fitting process. There is an excellent match of the bending moments down the pile, providing additional confidence in the calculation methodology. The bending moments are defined by reference to a load taken from the 3D FE results at a ground level displacement of $D/10$. This load is referred to as H_{ult}.

Case F1

Pile D2: D=8.75 m, L/D=4, h=87.5 m, t=91 mm

Accuracy of 1D(parametric) fit:

$$\rho_{sd.} = 1.00, \rho_{ult.} = 1.05; \eta_{sd.} = 0.99, \eta_{ult.} = 0.97$$
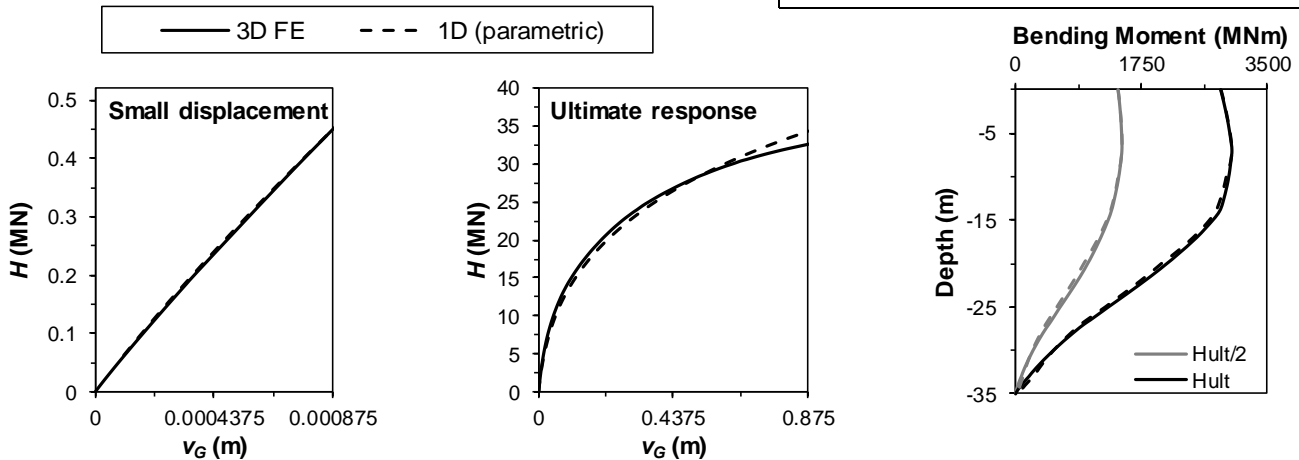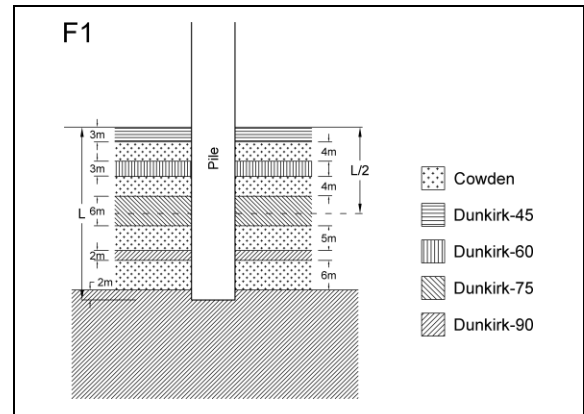

Figure 15: Comparison of 3D FE and 1D (parametric) response for case F1, a realistic multilayer system.

The 1D model results are assessed by comparison to results from the 3D FE calculations, with considerable success. Accuracy metrics for all of the 1D calculations conducted during the PISA study are shown in Figure 16 and 17, ranked from high to low, and identified for L/D ratio as well as homogeneous (H) or layered (L) profiles. These charts plot the loss of accuracy inherent in adopting the 1D model calculation, calibrated by the homogeneous soil reaction curves, as opposed to completing a full 3D FE calculation. For ultimate capacity, defined at a ground level displacement of $D/10$, the match between the 1D model and the 3D FE is reassuringly high, indicating limited loss of accuracy for most calculations. The calculations that are towards the tail involve layered soils with materials of high contrast in strength and stiffness. Only one case represents an extreme outlier: Case PC-D involved a layer of very dense Dunkirk sand within a Bothkennar clay matrix. This indicated that additional soil deformation modes exist associated with the layering that are represented in the 3D FE results but not by the 1D model. In this case, further improvement of the homogeneous soil reaction models are unlikely to give significant improvements in the calculation. Interrogation of the 3D analysis of the soil reactions in this case indicate that the stronger material is mobilised less effectively when surrounded by weaker material, due to 3D effects. It is likely that bespoke calibration of the 1D model will be required in this instance to ensure a conservative calculation.
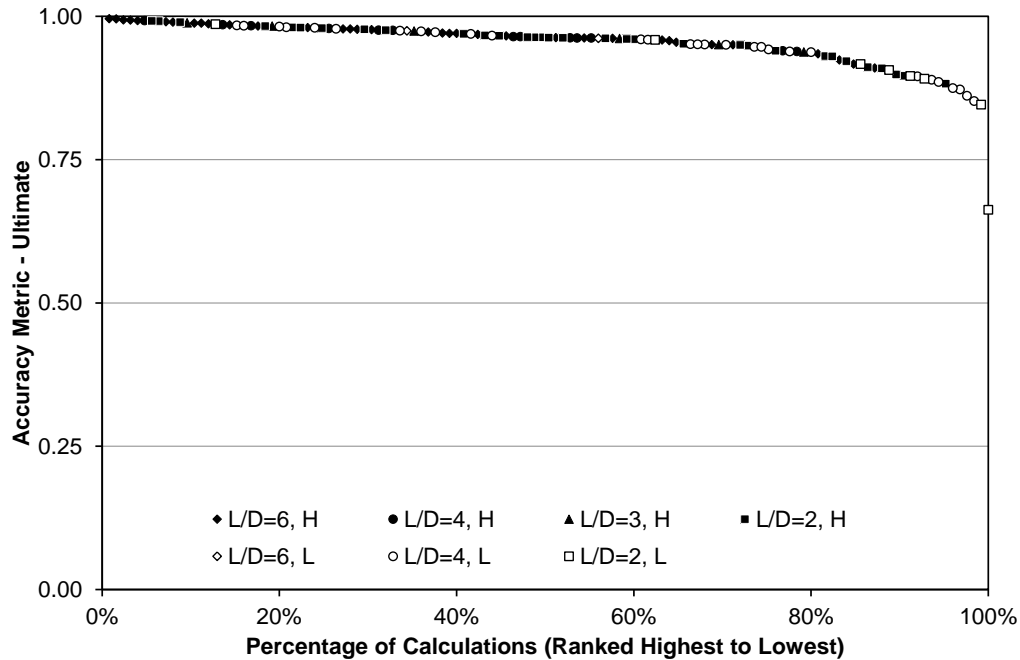
Figure 16: Accuracy metric at ultimate displacement (*D*/10).

Figure 17 focuses on the small displacement response, defined at a ground level displacement of *D*/10,000. Here there appears to be no substantive loss of accuracy in the 1D model when layering is introduced, with a very good fit for the different combinations of materials. This indicates, perhaps unsurprisingly, that for stiffness the effect of layering is less significant for the performance of the 1D model, with the accuracy being dependent mainly on the quality of fit of the homogeneous soil reaction curves. These results are reinforced by the ratio metric shown in Figure 18, with the results at both small and large displacements appearing normally distributed. At small displacement the mean of this ratio was 1.006 with a standard deviation of 0.0468 (giving a coefficient of variation of 4.7%), whilst at large displacement the mean was 1.01 with a standard deviation of 0.0716 (giving a coefficient of variation of 7.1%). There is only one outlier: case PC-D as described before. These accuracy results are also reported for each category of analyses in Table 1. Caution should be exercised in assessing the results for specific categories, given the number completed for each case is very small and the analyses were targeted on particular challenging combinations of soils. However, aggregated together, the results demonstrate that the PISA design model holds great promise for design optimisation.
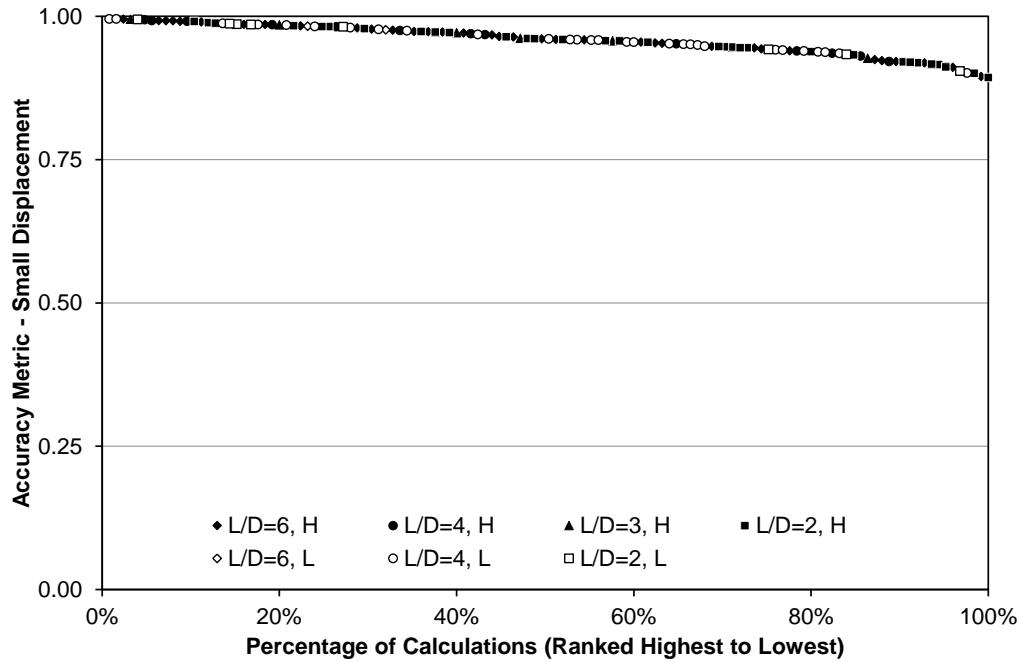
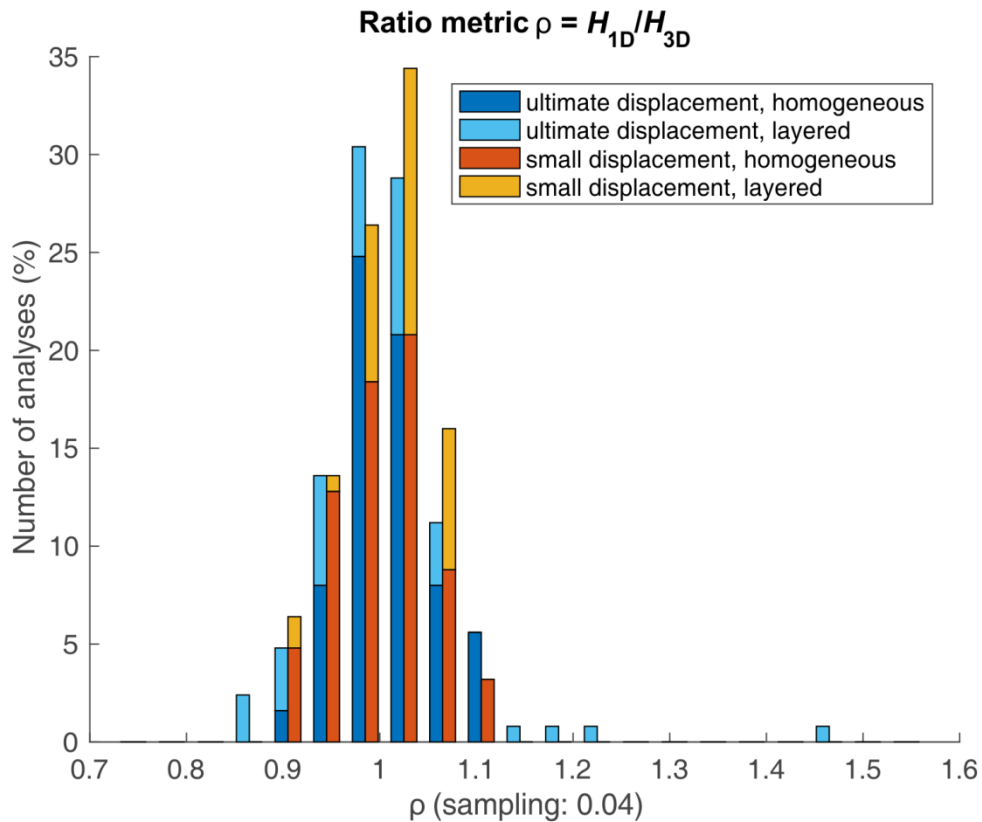**Figure 17: Accuracy metric for small displacement (*D*/10,000).**

Figure 18: Ratio metric for small displacement (*D*/10,000) and ultimate displacement (*D*/10).

Limitations

There are limitations to the application of the design approaches reported here:

(i) The work only considers monotonic loading with an equal focus on the initial stiffness and the ultimate capacity. Different models, with different weightings could, in principle, be developed.

No consideration of cyclic loading has been attempted, though future extensions for cyclic loading are feasible based on the computed monotonic backbone curve.

(ii) The work considers specific soil profiles, as representative soil profiles for offshore wind farm sites around the UK. The profiles are chosen to span a range of likely profiles encountered, although the emphasis of the Phase 2 work is the interrogation of the layered soil hypothesis. Application to other soil profiles and conditions, of the specified "rule-based" methods, will require further analyses and engineering judgment to be applied. Additional work is needed to further develop the "numerical-based" methods for other soil profiles to further expand the data base of "rule-based" methods for application in design.

(iii) The effects of installation on the pile response are neglected, as this was beyond the scope of the project. However, very good agreement between 3D FE predictions and measured pile responses from the available pile tests at Cowden (PISA tests), Dunkirk (PISA tests) and Bothkennar (existing case record), demonstrated that installation effects may not be as dominant a factor in the response of laterally loaded piles, as they are for axially loaded piles.

(iv) The models developed should be used only within the parameter range for which they have been calibrated; for example the models are calibrated for particular geometry ($L/D = 2$ to 6, $D = 5$ to 10 m) and loading conditions ($h/D = 5$ to 15), which must be respected in their application.

Conclusions

The paper describes the different elements of the PISA project including (a) field testing, (b) 3D FE calculations, (c) 1D model formulation, (d) calibration of the 1D model for different soil types, and, (e) application of the 1D model to layered soil configurations. During the course of the project a total of 125 3D FE calculations were completed, including 86 calculations for homogeneous soils, and a further 39 calculations for layered soils of varying complexity. The study likely represents the largest systematic investigation of monopile behaviour in homogeneous soils and layered materials, providing a benchmark for future calculation procedures for layered soils. The homogeneous soil calculations provided the basis for developing 1D design models for 3 different clay soils (Cowden till, Bothkennar clay, London clay), and Dunkirk sand at 4 different relative densities (45%, 60%, 75%, 90%). The outputs for Dunkirk sand allowed development of a general Dunkirk sand model for arbitrary relative densities (45% to 90%).

For layering the work demonstrated that homogeneous soil reaction curves applied appropriately to the 1D PISA model provide a very good fit to the 3D FE calculations, particularly for profiles that appear relevant to current offshore wind farm sites. The stratification adopted in the layered soil study was informed by discussion with and feedback from the PISA industry partners to ensure relevance to industry design. As the work developed, emphasis turned to numerous multi-layer profiles and shorter L/D piles. The modelling approach was severely tested by combinations involving soft clay and very dense sand; it performed well for most of the profiles. Only one profile, of the many tested, indicated that more detailed analysis may be required for when there are very significant strength/stiffness differences between soil layers. The accuracy metrics developed during the PISA project indicated almost no systematic bias and a very low coefficient of variation (much less than 10%). The 3D effect of layering was only evident in the large displacement response, whereas discrepancies at small displacement were derived principally by a lack of fit at small displacement for the homogeneous cases.

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