3D FINITE ELEMENT ANALYSIS OF MONOPILES AND ITS APPLICATION IN OFFSHORE WIND FARM DESIGN

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ABSTRACT

The PISA Joint Industry Project has led to significant advances in monopile design, with 3D Finite Element (FE) analysis playing a key role in the development of the method, as well as in its application in practice. This paper presents 3D FE analyses carried out as part of the design process for monopile foundations for an offshore wind farm in the North Sea. The paper is divided into three parts. The first part demonstrates the ability of the adopted constitutive models to describe the soil response for the soils encountered, on the basis of high quality ground investigation campaigns for the site. The second part presents the calibration of the constitutive model used in the FE analyses to simulate the sands' response, against the PISA field tests. The third part presents the results of the 3D analyses at different monopile locations across the wind farm, in terms of load displacement curves and structural forces within the monopiles.

Keywords: monopile foundations, finite element analysis, constitutive modelling

INTRODUCTION

Monopile foundation design for offshore Wind Turbine Generators (WTG) requires consideration of many aspects of behaviour. The recently completed PISA Joint Industry Project has led to significant advances in monopile design (Byrne et al. 2019 and Burd et al. 2019), with 3D Finite element (FE) analysis playing a fundamental role in the development of the method and its application into the practical design of monopiles. The development of the PISA method was based on sophisticated 3D FE analyses using the Imperial College finite element code ICFEP (Potts & Zdravkovic, 1999). This included Class A predictions of field tests on monopiles subjected to lateral loading at two sites; i.e. Cowden, a glacial clay till site (Zdravkovic et al. 2019), and Dunkirk, a marine sand site (Taborda et al. 2019).

In the application of the PISA method in design 3D FE analysis are performed for monopiles at key locations across a wind farm site. These analyses are used to extract site specific soil reaction curves which can then be utilised in simpler 1D Winkler-type beam-spring models applied to every WTG position. It is important to highlight that appropriate constitutive models are essential and in order to calibrate the model parameters high quality ground investigation information, combining in-situ and laboratory testing results, is required.

This paper presents 3D FE analyses carried out as part of the design process for monopile foundations for an offshore wind farm in the North Sea. The analyses have been carried out using the FE code ICFEP, which was used in the development of the PISA method. The paper is divided into three parts; the first part demonstrates the ability of the constitutive models adopted in the 3D FE analyses to describe the soil response, for clay and sand dominated materials, encountered in the offshore wind farm site. In the FE analyses presented in this paper a different constitutive model to the one used for the development of the PISA method was adopted for the modelling of the sand deposits. Therefore a verification of this model's ability to replicate the Dunkirk PISA field tests was undertaken, which is presented in the second part of the paper. The third part of the paper presents the results of the 3D FE analyses of monopiles at different locations across the windfarm. These include load displacement curves and structural forces developed within the monopiles.

SOIL CONDITIONS, CONSTITUTIVE MODELS AND ASSOCIATED PARAMETERS

In the offshore wind farm site considered in this paper the soil deposits encountered consist of i) stiff clays of low and high plasticity and ii) sands (in most locations encountered in a dense state).

Clay deposits

The low plasticity clay deposits were modelled in the FE analyses with the same constitutive model adopted in the PISA work for the stiff glacial clay till at Cowden (Zdravkovic et al. 2019). This is an enhanced version of the Modified Cam Clay model featuring a non-linear Hvorslev surface on the dry side, a generalised shape for the yield and plastic potential surfaces in the deviatoric plane and strain dependent non-linear behaviour pre-yield, based on the IC.G3S model by Taborda et al. 2016.

The model was found to reproduce well the response of the low plasticity glacial till deposits encountered at the offshore wind farm site. The parameters of the model were derived on the basis of laboratory tests across the whole site (which included oedometer tests, triaxial CAUc and CAUe tests, bender element tests, resonant columns tests). Figure 1a shows the stiffness variation with strain obtained from undrained triaxial tests with local instrumentation, which allowed the measurement of stiffness from very small strains, for one of these glacial clay tills. Also shown on this figure are the results of bender element (BE) and resonant column (RC) tests. For comparison purposes, these are plotted in terms of normalised $3G/p'_o$ (for an isotropic material E_u=3G). It should be noted that the bender element tests, as well as the stress paths measured in the triaxial tests indicate some stiffness anisotropy. However, the available tests are not sufficient to fully define this aspect of behavior of the material and hence, in line with the PISA work, isotropic stiffness was assumed. In-situ P-S logging tests were also performed in some boreholes. Results of these tests were scattered and significantly biased compared to the laboratory measurements; they were considered less reliable and hence the stiffness strain curve adopted in the FE analyses was based on the laboratory tests (Figure 1a). Figure 1b also shows the results of triaxial and BE tests on this clay from a nearby site. Figure 2 presents the results of an undrained triaxial compression test on this glacial till together with the results of a single element finite element simulation of this test; good comparisons between predictions and measurements can be seen.

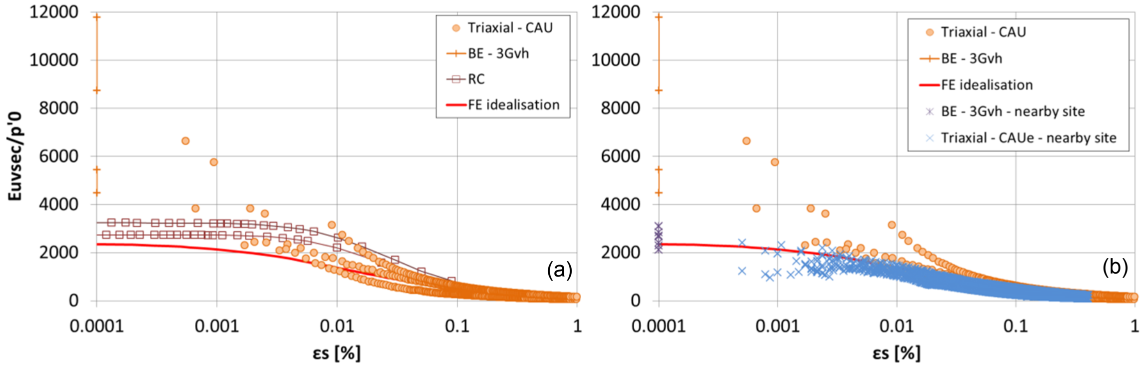


Fig. 1. Stiffness strain response of stiff glacial clay till

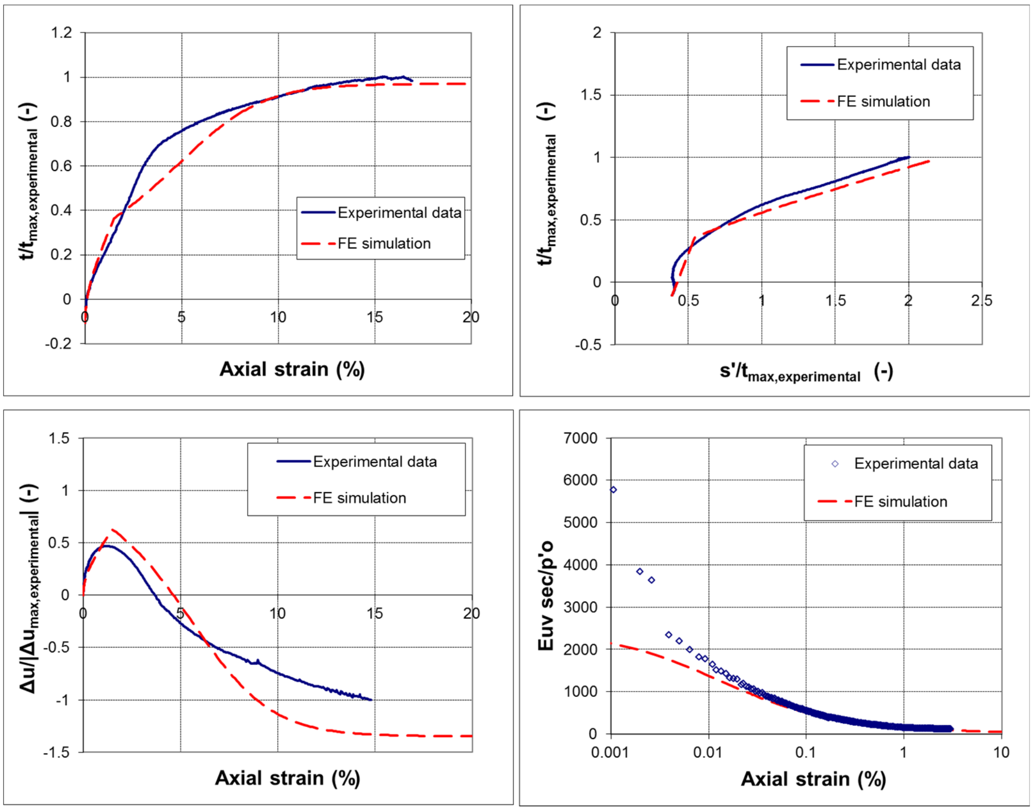


Fig. 2. Comparison of experimental results and simulated response for CAUc test on stiff glacial till clay

However, in another clay deposit, index properties tests indicated significant variability of the material encountered. Figure 3 shows profiles of index properties with depth and the corresponding plasticity chart for this clay deposit. Considerable variability of the plasticity of the clay can be seen even within the same borehole (i.e. BH06). As expected, the response of this clay in undrained triaxial tests was also found to be variable. In some cases the material showed a strain-stress response akin to a low plasticity clay, i.e. the clay dilated when sheared, in a manner similar to the one shown in Figure 2. In other cases, the clay showed a stress-strain response typical of a stiff plastic clay, i.e. when sheared it showed a peak strength followed by a drop to a “post-rupture” strength. Figure 4 shows an example of the latter response. In this clay deposit, the reduction in strength from peak to post-peak varied between around 3% and 18% in the tests which showed strain softening. However, it should be noted that undrained triaxial tests reported by Grammatikopoulou et al. (2017) on a similar clay deposit, of high plasticity, depicted a much higher reduction of strength from peak to post-peak, i.e. in the order of 30%.

As Grammatikopoulou et al. (2017) noted, from CPT tests it is not always clear whether the stiff clays encountered would show a stress-strain response akin to a stiff plastic clay or a low plasticity clay. As such it is important that laboratory tests are undertaken and the measured response is examined carefully before choosing what type of constitutive model is used to best represent the clay deposits encountered. In the absence of location specific laboratory testing a conservative approach would need to be adopted.

In the FE analyses presented here this clay was modelled using a Mohr-Coulomb type model, in which $\varphi' = 0$ and c' equal to the undrained strength were assumed, i.e. like a Tresca model (similar to Grammatikopoulou et al. 2017), but combined with a non-linear model which uses the same hyperbolic expression for the tangent shear modulus as the one used in the variant of the MCC model discussed above. Figure 4 presents the results of a single element finite element simulation. As discussed in Grammatikopoulou et al. (2017) the Tresca model provides a conservative representation of the stress-strain response, although it cannot reproduce the measured stress path and pore pressure response. It should be noted that the original PISA study considered the modelling of the glacial till deposit of low plasticity at Cowden, as discussed above. The modelling of stiff plastic clays has subsequently been addressed by the PISA2 JIP, which used a more sophisticated model than Tresca.

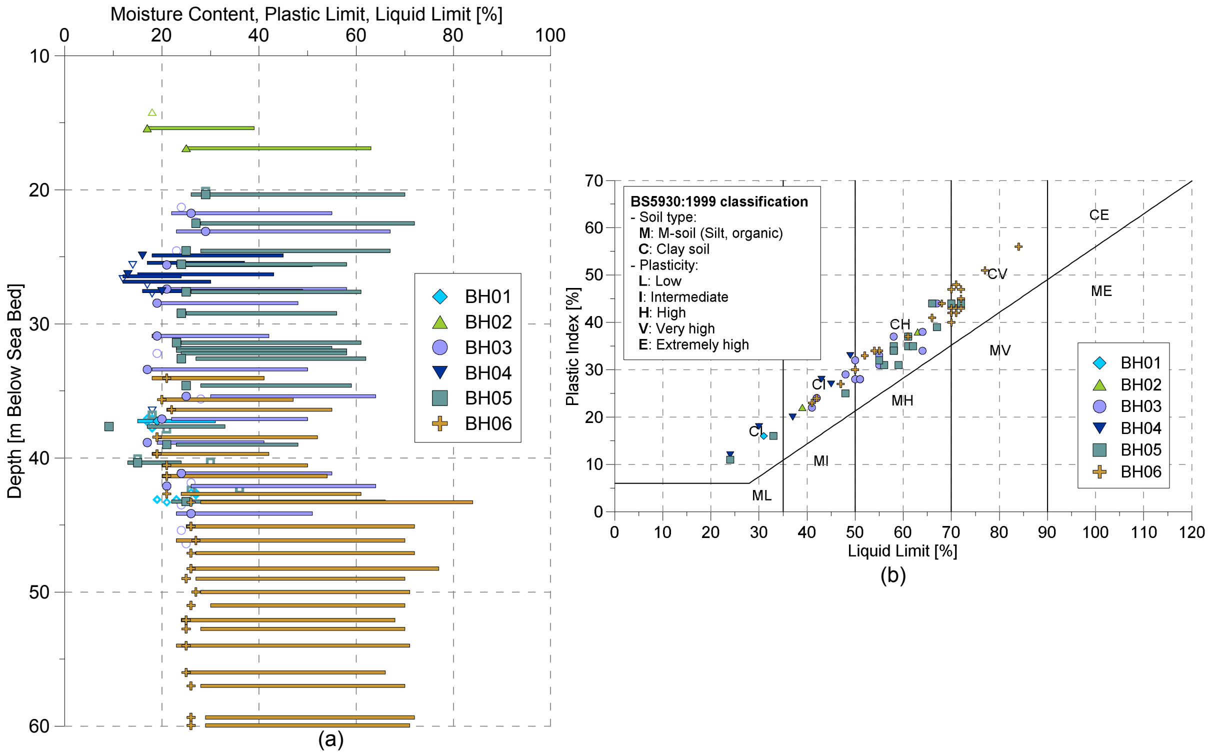


Fig.3. a) Profiles of index properties with depth b) Plasticity chart

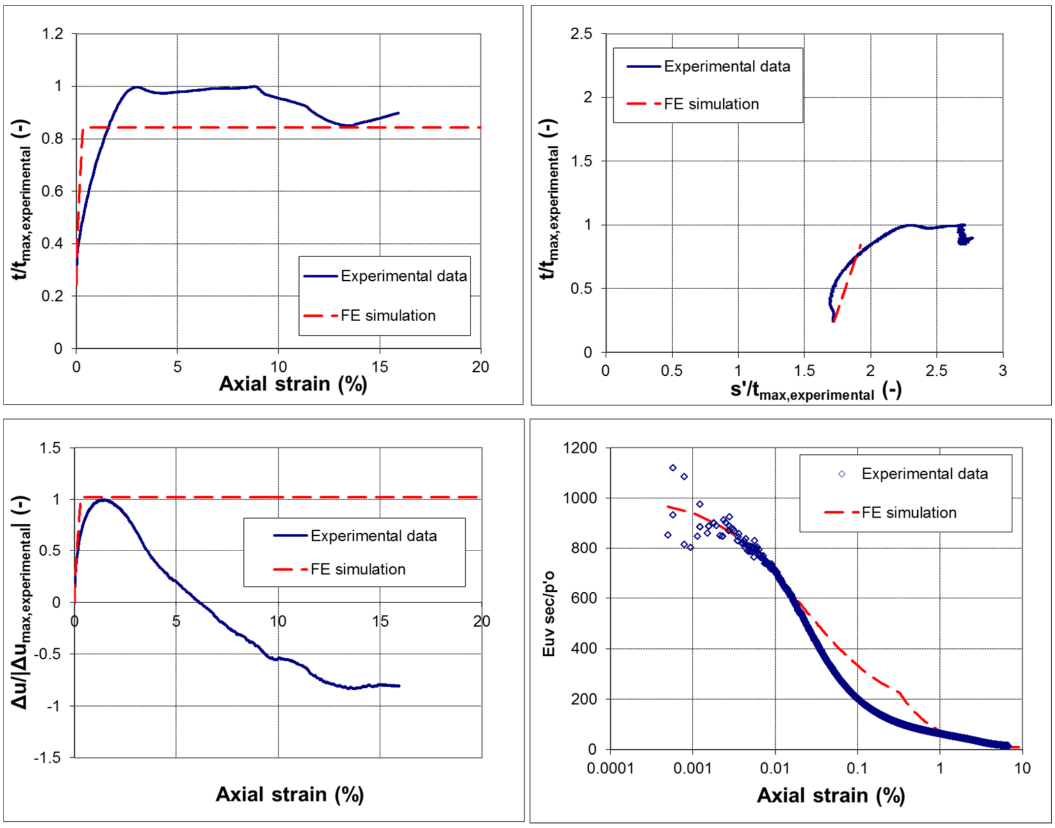
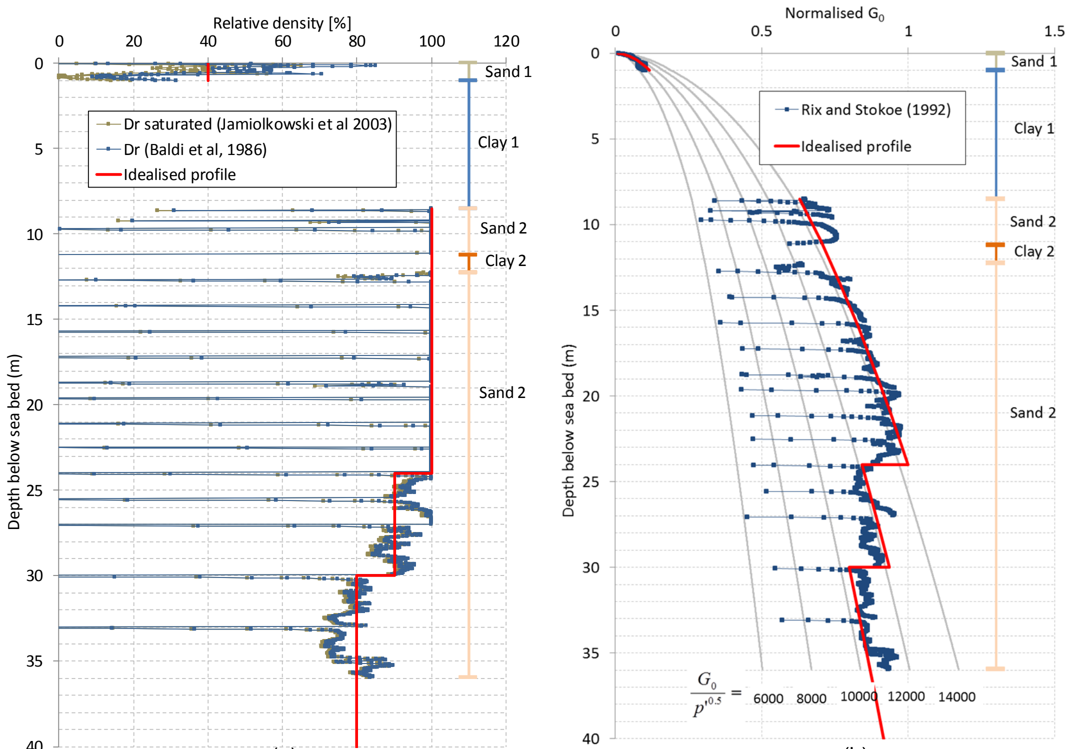


Fig. 4. Comparison of experimental and simulated response for CAUc test on stiff plastic clay

Sand deposits

In the PISA work the sands were modelled with an advanced bounding surface plasticity model which can account for the effects of both stress level and void ratio, as well as a number of other features of sand behavior, as described in Taborda et al. (2019). In the FE analyses presented here the response of the sand deposits was modelled with a strain softening variation of the Mohr-Coulomb model, in which the angles of shearing resistance, dilation and cohesion intercept (if necessary) are allowed to vary with strains, as described by Potts and Zdravkovic (1999) and discussed in Grammatikopoulou et al. (2017). It should be noted that the peak strength and dilation of a sand will depend on the stress state and density of the material. Hence, when using this model in the FE analyses the input parameters will have to vary to account for different in situ densities; unlike the bounding surface model used in the PISA work, which is capable of reproducing the sand response under a wide range of states using a single set of parameters.

This strain-softening Mohr-Coulomb elasto-plastic model was combined in the FE analyses with the non-linear elastic model IC.G3S (Taborda et al. 2016). The latter allows the simulation of a variety of features of sand response, including non-linearity from early stages of loading, as well as dependence of stiffness on stress state and void ratio, which is important for sands. It is generally accepted that the stiffness at very small strains (G_0) depends on the mean effective stress (p') to the power of n, where n = 0.5 is reasonably representative for sands. The formulation of the IC.G3S model allows the modelling of the dependence of G_0 on $\sqrt{p'}$, as well as on the sand density. The parameters of the IC.G3S model which control the stiffness G_0 at small strains were chosen for each sand deposit, in combination with the in situ relative density, D_r, so as to reproduce the Rix & Stokoe (1992) G_0 interpretation of the CPT profiles for all monopile locations considered. Figure 5 shows an example from a monopile location. Figure 5a shows the idealised relative density D_r assumed for that location and Figure 5b the corresponding G_0 modelled by the IC.G3S model for the chosen model parameters (in this figure G_0 is normalized by the maximum value in the idealised profile). Figure 6 shows a simulation of a drained triaxial compression test on a sample from this dense sand (tested at p'_{in}=140kPa, Dr~100%), using the strain softening Mohr Coulomb model in combination with the non-linear IC.G3S model.



(a) (b)

Fig. 5. CPT interpretation of a) D_r and b) G_0 and idealised response for sand deposits

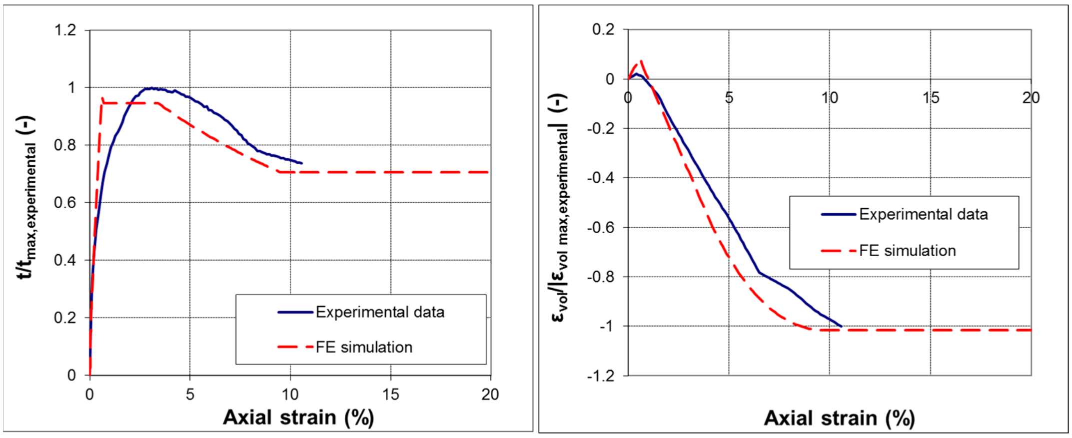


Fig. 6. Comparison of experimental results and simulated response for CID test on dense sand

REPRODUCTION OF DUNKIRK FIELD TESTS

In order to further confirm the capability of the chosen strain softening Mohr-Coulomb/IC.G3S model to reproduce the response of sands, the model was calibrated for the Dunkirk Sand encountered at the PISA field test site and then used in 3D FE analyses of two test piles (DM4 and DL2). Figure 7 shows comparisons of laboratory results from drained triaxial tests on samples of Dunkirk Sand (from Taborda et al. 2019) and single element simulations using the Mohr-Coulomb/IC.G3S model described above. It should be noted that two different variations of the angle of shearing resistance and dilation were assumed in these single element simulations, in order to represent the two initial void ratios of the samples tested. Figures 8 and 9 show comparisons of the field measured response and the results of the 3D FE modelling using the Mohr-Coulomb/IC.G3S model adopted in this work for test piles DM4 (with diameter D=0.762m and embedded length L=4m) and DL2 (D=2.0m and L=10.5m) respectively. For comparison, also shown on the same figures are the results of the "3D FE PISA" analyses (from Taborda et al. 2019). Figures 8 and 9 show that there is a good agreement of the overall load-response between predictions and field test data. It is worth noting that the model adopted in this work results in a slightly softer response at large mudline displacements (in the order of 10% difference in horizontal force for mudline displacements of approximately 10%D) than the prediction of the final 3D PISA analyses, suggesting conservative results. (The load corresponding to horizontal displacement of 10% of the monopile diameter is a value frequently considered as the geotechnical capacity of the monopile).

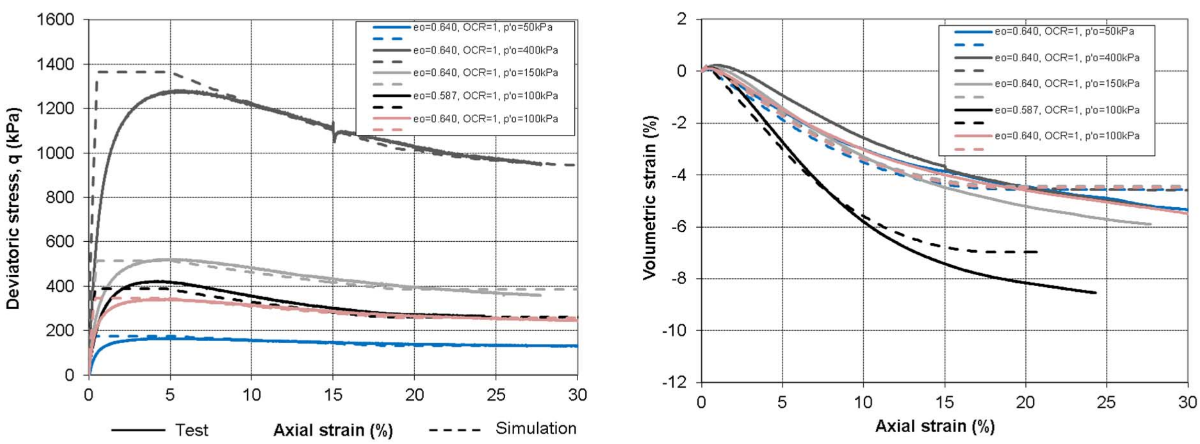


Fig. 7. Comparison of experimental results (from Taborda et al. 2019) and simulated response in triaxial tests of Dunkirk Sand

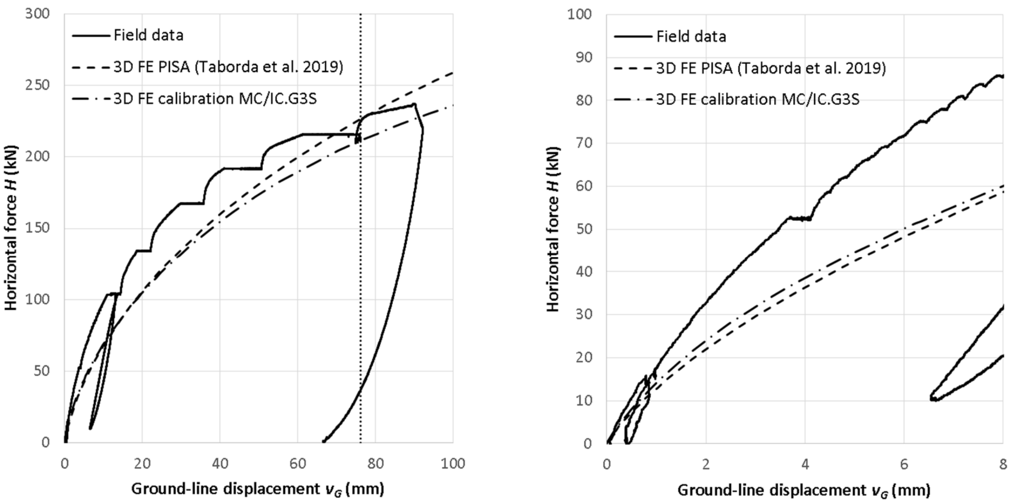


Fig. 8. Comparison of PISA field measured response and 3D FE modelling using Mohr-Coulomb/IC.G3S model for monopile DM4

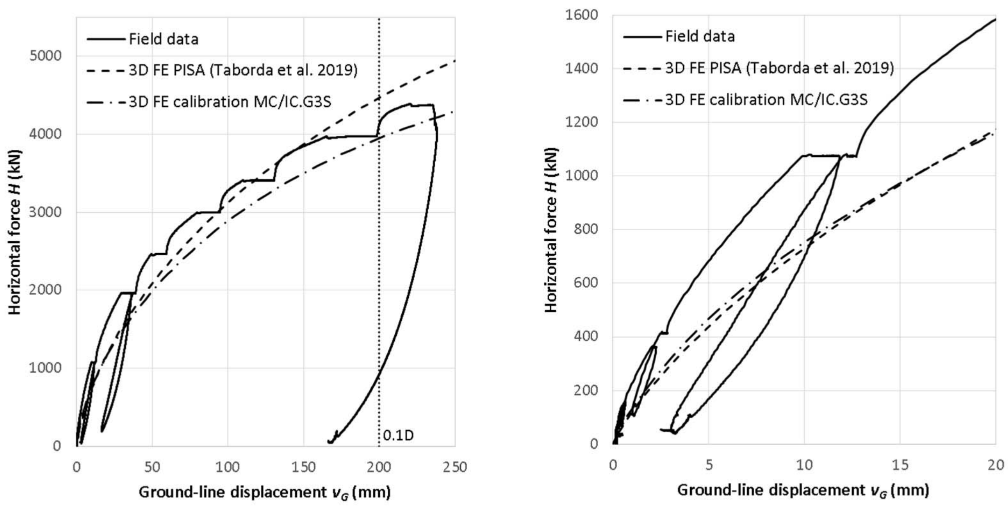


Fig. 9. Comparison of PISA field measured response and 3D FE modelling using Mohr-Coulomb/IC.G3S model for monopile DL2

RESULTS OF 3D FE ANALYSES OF MONOPILES

Figure 10 presents horizontal load-displacement curves at design seabed level obtained from the 3D FE analyses of monopiles at eight locations across the offshore wind farm. These locations were chosen in order to cover different stratigraphies encountered across the site, consisting of alternating sand and clay layers of varying thickness, density and plasticity (with the exception of WTG2 where only sand layers were encountered). In Figure 10 the horizontal load is normalized by the load corresponding to horizontal displacement of 10% of the monopile diameter for location WTG6 (i.e. the location with the smaller ultimate load). Figure 10a shows the overall response and Figure 10b shows the initial response up to a lateral displacement of 2%D. Figure 10a shows that the ultimate capacity of the monopiles is predicted to vary by a factor of about 2. This is not surprising as the locations have different stratigraphies and monopile geometries. It is interesting to note that in many cases the load increases due to increases in lateral displacements from around 10%D to around 20%D. In most cases this increase is generally smaller than 10%, but in locations WTG2 and WTG7 it is between 10% and 20%. Figure 10b shows that it is not only the ultimate response that is different but also the initial response, which is more relevant to operational conditions.

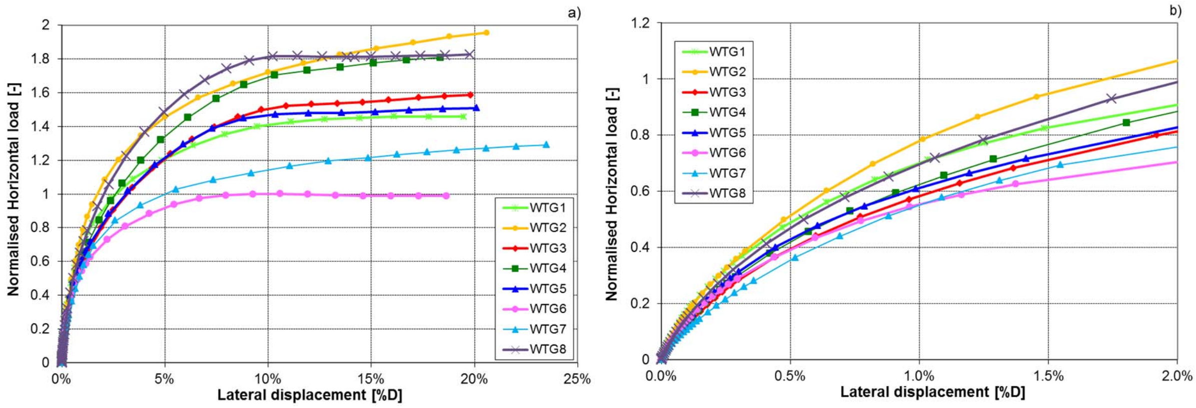
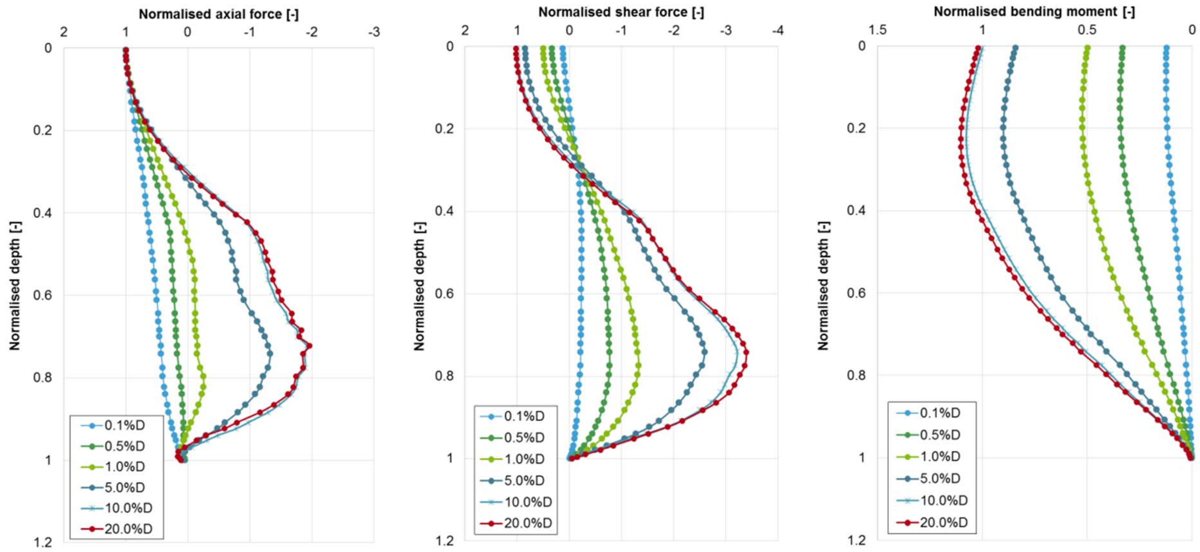
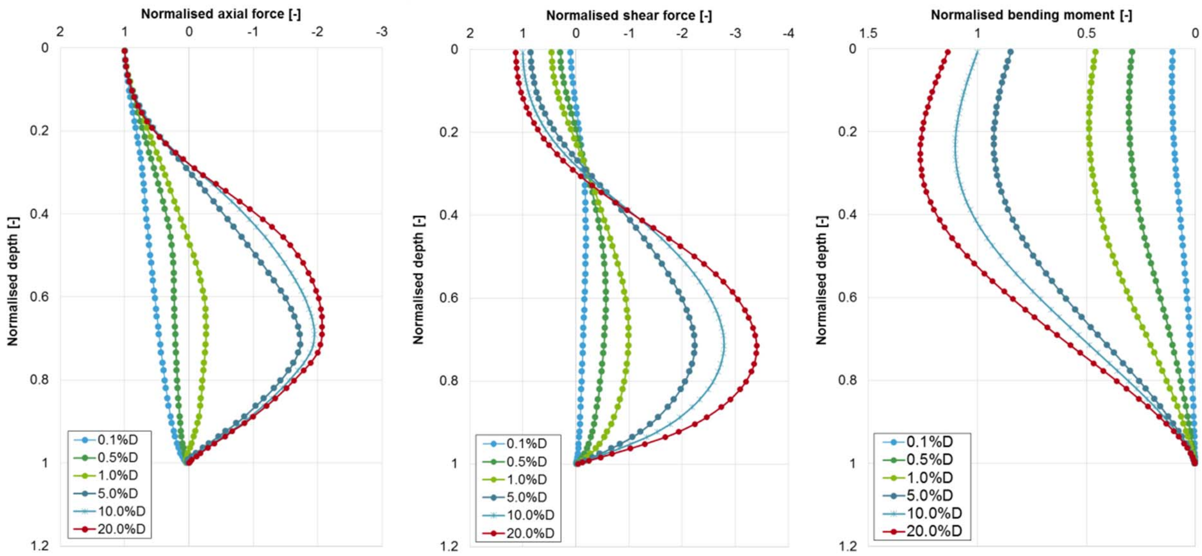


Fig. 10. Load displacement curves at design seabed level a) overall response b) initial response



(a) (b) (c)

Fig. 11. Monopile internal forces for WTG 1 a) axial forces b) shear forces c) bending moments



(a) (b) (c)

Fig. 12. Monopile internal forces for WTG 2 a) axial forces b) shear forces c) bending moments

The 3D FE analyses undertaken were used to investigate a number of aspects of monopile design, including the extraction of soil reaction curves, in accordance with the PISA numerical-based method, as well as structural forces developed within the monopile, which are presented herein. Figures 11 and 12 present normalised graphs of axial force, shear force and bending moment developed within the monopile at locations WTG 1 and WTG2 respectively. It can be seen that the structural forces increase gradually up to horizontal displacements of 10%D. For horizontal displacements between 10%D and 20%D a very small increase in axial force is observed for WTG1 which indicates that the failure mechanism is practically mobilised. The same applies to the shear force and bending moment for this location. For location WTG2 the axial forces for displacements between 10% and 20% still increase, at depths below approximately 10m, indicating that the failure mechanism is still not mobilized; this is consistent with the load-displacement response shown in Figure 10a.

SUMMARY AND CONCLUSIONS

The recently completed PISA Joint Industry Project have led to significant advances in monopile design, with 3D Finite Element (FE) analysis playing a fundamental role in the development of the method and its application in the practical design of monopiles. This paper presents 3D FE analyses carried out as part of the design process for monopile foundations for an offshore wind farm in the North Sea, using the finite element code ICFEP, which was used in the development of the PISA method.

The paper is divided into three parts. The first part demonstrates the ability of the adopted constitutive models to describe the soil response for the soils encountered, on the basis of advanced ground investigation campaigns for the site. The soils encountered consist of clays of varying plasticity and (predominantly dense) sands. The clays of low plasticity were modelled with the same constitutive model used in the PISA work to model the glacial till at Cowden. This model was shown to reproduce well the response of the low plasticity glacial tills encountered at the offshore wind farm site, through comparisons of experimental data and single element finite element simulations.

Careful consideration of index properties tests for another clay deposit encountered within the offshore wind farm site indicated material of variable plasticity, even within the same borehole. Variable response was also shown in triaxial tests, with the stress-strain response in some cases akin to a low plasticity clay and in some cases akin to a high plasticity clay exhibiting strain softening when sheared. This once again highlighted the importance of conducting laboratory testing in order to establish the response of the material encountered and to choose the appropriate constitutive model. In the FE analyses this material was simulated with a simple model, acknowledging its limitations. The modelling of stiff plastic clays has subsequently been studied, with more advanced models, in the PISA2 project.

The sands were modelled in the FE analyses with a strain-softening Mohr Coulomb model combined with the non-linear elastic IC.G3S model (Taborda et al. 2016). The latter allows the simulation of a variety of features of sand response, including non-linearity from early stages of loading, as well as dependence of stiffness on stress state and void ratio, which is important for sands. The model was shown to reproduce well the dependence of G_0 on stress state and density and the overall response in drained triaxial tests. However, when using this model in FE analyses the input parameters will have to vary to account for different stress states and densities, unlike the bounding surface model adopted in the PISA study, which can reproduce the sand response for a wide range of states using a single set of parameters.

The second part of the paper presents the verification of the strain-softening Mohr Coulomb/IC.G3S model which was used in the FE analyses, against the PISA field tests at

Dunkirk. The model was initially calibrated for the Dunkirk Sand encountered at the PISA field test site and then used in 3D FE analyses of two test piles (DM4 and DL2). A good agreement was found of the overall load-response between field test data and predictions using this model. When compared with the predictions of the bounding surface model used in the PISA study, the model adopted in this work results in a slightly softer response at large mudline displacements, in the order of 10% difference in horizontal force for mudline displacements of approximately 10%D (a value frequently considered as the geotechnical capacity of the monopile).

The third part of the paper presents the results of the 3D analyses at eight different monopile locations across the wind farm, in terms of load displacement curves and structural forces within the monopiles. The ultimate load was found to vary by a factor of 2 between different locations; reflecting the differing stratigraphies and monopile geometries. Differences were also observed in the initial load-displacement response up to 2%D, which is more applicable to operational conditions. In many cases the horizontal load was found to still increase after horizontal displacements of 10% and in those cases so did the structural forces.

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