Proceedings of the XVII ECSMGE-2019
Geotechnical Engineering foundation of the future
ISBN 978-9935-9436-1-3

doi: 10.32075/17ECSMGE-2019-1072

XVII European Conference
ECSMGE 2019
Reykjavik
on Soil Mechanics
and Geotechnical Engineering

Analysis of axially loaded piles in sand by means of FEM and large-scale tests

Analyse des pieux chargés axialement dans le sable à l'aide de FEM et d'essais à grande échelle

Aligi Foglia, Tulio Quiroz, Severin Spill
Fraunhofer Institute for Wind Energy System IWES, Hanover, Germany

Elmar Wisotzki
TenneT Offshore GmbH, Lehrte, Germany

ABSTRACT: At the Test Center for Support Structures in Hanover (Germany) a new large-scale indoor geotechnical testing facility has been put into operation. The facility consists of a 14 m x 9 m x 10 m sand pit, in which the material is carefully compacted and slowly saturated by means of a base flooding system. Such a testing environment allows a precise experimental validation of soil-structure-interaction models. The study case presented in this paper involves an axially loaded open-ended steel pile for offshore applications which is driven into the prepared test seabed. The pile test is retrospectively simulated by means of an axial symmetric wished-in-place finite element model (FEM). The analysis shows that by using conventional parameters the prediction cannot capture the experimental behaviour. By modifying the soil parameters with existing empirical equations that take pile installation effects into account, a much better model prediction is achieved.

RÉSUMÉ: Dans le centre d'essais pour les structures de soutien de Hanovre, une nouvelle installation d'intérieur d'essais géotechniques à grande échelle a été mise en service. L'installation consiste d'un bac à sable de 14 m x 9 m x 10 m, où le matériel soit soigneusement compacté et lentement saturé avec un système de inondation. Un environnement d'essais de ce type permet d'obtenir une validation précise des modèles d'interaction sol-structure. Le cas d'étude présenté dans un pieu overt en acier chargé axialement, utilisé dans des applications offshore. C'est simulé retrospectivement au moyen d'un modèles d'éléments finis (FEM) d'axe symétrique. L'analyse montre qu'en utilisant des paramètres conventionnels, les prévisions ne peut pas refléter le comportement expérimental. En modifiant les paramètres du sol avec les équations existantes qui prennent en compte les effets de l'installation des pieux, on obtient une prévision du modèle bien meilleure.

Keywords: large-scale tests; FEM; axially loaded piles

1 INTRODUCTION

Modelling of pile foundations under axial load has long been one of the most investigated topics in geotechnical research. Empirical methods such as the alpha and beta methods (Tomlinson and Woodward, 2015) or the CPT methods (Lehane et al., 2005) together with spring models such as the p-y and Q-z curves (Bohn et al., 2016) are still the main design methods used in both onshore and offshore geotechnical engineering. Since a couple of decades pile foundation design can be supported by numerical models to either shed light on complex project situations or loading conditions (Achmus and Thieken, 2010) or to answer very specific questions about the fundamentals of pile behaviour (De Nicola and Randolph, 1993). Interestingly, recent research projects on piled foundations focused on the simulation of the installation behaviour. Three separate effects which are not independent from each other but which can, at least preliminarily, be separately investigated account for the post installation behaviour of axially loaded piles (Dikjstra, 2009; Engin, 2013). These effects are related to the physical properties of the material (i.e particle size), the densification state of the material and the soil stress fields of the geotechnical system. In Henke and Grabe (2008) three different pile installation techniques are numerically simulated. They find that both stresses and sand densification states are evidently dependent on the installation method used. In particular, they find that a vibro-driven pile causes soil densification around its shaft, which in turn diminishes the radial stresses. On the other hand, installing a pile by impact-driving or by jacking tends to dilate the sand around the pile, therefore increasing the radial stresses acting on the pile surface. Similar installation effects have been recently found using a material point method (MPM) model in Phuong et al. (2016). The results of numerical models that take the installation effects into account are usually compared to geotechnical centrifuge tests (Phuong et al., 2016) or to photoelasticity aided small-scale tests (Dikjstra, 2009).

In this contribution, the comparison between the results of a numerical model and of a large-scale model test for an impact-driven single pile is presented. Alternatively to the scientific work quoted above, the installation effect is not numerically modelled. Instead, the material parameters are derived from site investigations and are modified in accordance to existing empirical approaches that take the pile installation effects into account.

The innovative aspect of this preliminary research work lies in the unique test environment featured by the Test Center for Support Structures in Hanover, which allows large-scale foundations

Figure 1. Sand pit of the Test Center for Support Structures in Hanover during the sand preparation works (Wisotzki et al., 2018)

to be tested in a uniform sand bed. This enables a confident knowledge of the test environment which leads to accurate and reliable foundation analysis.

2 PHYSICAL AND NUMERICAL MODEL

The objective of this work is the investigation of a single pile in sand subjected to an axial compressive load. In the following two sections the physical model testing and the numerical modelling technique used for the comparison are described.

2.1 Physical model

The test results for this study were obtained from an experimental campaign conducted by Fraunhofer IWES investigating pile group effects for offshore wind substations. The experimental work as well as the test data post processing was financed by TenneT Offshore GmbH (Wisotzki et al., 2018). The tests were carried out in the sand pit of the Test Center for Support Structures of the Leibniz University of Hannover. The sand pit is 10 m deep, 14 m long and 9 m wide. A picture of the sand pit during the sand preparation work is shown in Figure 1. The sand sample is prepared by distributing and compacting the sand over the pit surface in layers of around 25 to 30 cm. The water table was very slowly raised at the end of the sand preparation up to 0.5 m below the sand surface. The test sand is a uniformly graded quarz sand and has the following physical properties: particle size by 50% passing material $d_{50} = 0.36$ mm, uniformity coefficient $C_u = 1.82$, coefficient of curvature $C_c = 0.96$, minimum and maximum soil porosity $n_{\min} = 0.31$ and $n_{\max} = 0.46$. To characterise the sand sample, core samples were taken from each layer and cone penetration tests (CPTs) were performed after completion of the sand preparation. The interpretation of the core samples revealed a homogeneously and densely compacted soil sample with a relative density $D_r = 0.7$ to which a coefficient of variation over the entire depth equal to ±2.9% is associated. To estimate some of the essential mechanical properties of the sand, a series of consolidated drained triaxial tests on ten samples with two different relative densities were executed. The critical friction angle, $\phi'_{\mathrm{cr,tx}}$, was found to be 28.4°.

The test specimen consisted of an open-ended pile with diameter $D = 254$ mm, wall thickness $t = 8$ mm and embedded length $L = 5500$ mm. The pile was installed with a double acting hammer SB-120 from MENCK GmbH. The verticality of the pile was ensured through a pile guide fixed to the sand surface. A photo of the pile installation can be seen in Figure 2. The plug length ratio at the end of the installation process was 0.87. A further CPT series was executed after the pile installation to capture some of the installation effects described in Section 1. This CPT profile was used to determine the soil parameters for the numerical modelling. After a set-up period of 23 days was allowed for, the pile was loaded in compression. As expected for this pile geometry, the measurement of the plug before and after the static compression test confirmed that the pile failed in a plugged manner. The CPT profiles together with some more detailed information of the test campaign can be found in Wisotzki et al. (2018).

Figure 2. Installation of the open-ended steel pile in the sand pit of the Test Center for Support Structures

2.2 Numerical modelling

The finite element software package ABAQUS 6.14 was used for the simulations. The foundation geometry and the load applied allow an axial symmetric model to be used. The model elements are of the type CAX4R. The pile was modelled as a wished-in-place full cross section specimen. It should be noted that this choice is justified by the fact that the pile in the physical experiment failed in a fully plugged fashion. Equivalent unit weight and equivalent Young's modulus for the pile were calculated as a function of sand and steel material properties (above and below the water table) and of the plug length ratio. To exclude any interference between boundaries and model elements, the soil adjacent to the pile shaft was extended for 10 pile diameters while the soil below the pile toe was extended for 6 pile diameters. A constraint in the vertical direction was assigned to the lateral boundary whereas constraints in

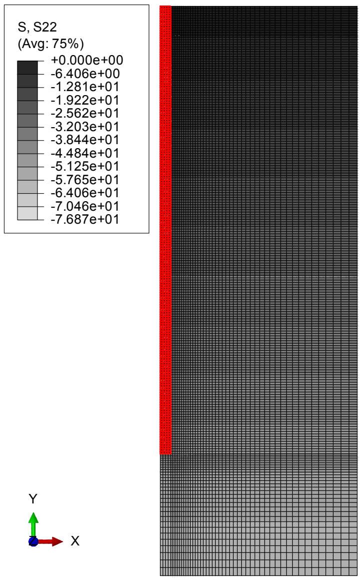

Figure 3. Axial symmetric finite element model used for the simulations. Stage between Step 1 and Step 2. The contour plot refers to the distribution of the vertical effective stresses in kPa. The pile is marked in red

both directions were given to the bottom boundary. The reacting soil around the pile was divided along the model depth into four horizontal layers based on the CPT results. The soil parameters were independently assigned to each sand layer. To ensure a realistic modelling, a refinement of the element mesh was provided in the regions close to the pile shaft and below the pile toe. Prior to the start of the simulations a mesh study was carried out. This revealed an optimal mesh density of 686 elements/m^2. The well-established Mohr-Coulomb elasto plastic material model was used for the simulations. The linear perfectly plastic in-built penalty friction formulation was selected for the contact modelling in the interface between the pile shaft and the adjacent soil and between the pile toe and the soil directly below. The elastic slip was set to 1 mm for all presented simulations while the friction coefficient was set equal to the steel-soil interface angle, δ, according to different assumptions (see Section 2.2.1).

The simulations were run in three steps. In Step 1 only the soil was present and the geostatic stresses were calculated. In Step 2 the pile replaced the portion of soil on the upper left part of the model and was subjected to its own weight. In Step 3 the pile was compressed in a displacement controlled fashion by assigning a vertical displacement to the pile head equal to one tenth of the pile diameter (i.e. 25.4 mm). An image of the model after the geostatic stage (i.e. between Step 1 and Step 2) is shown in Figure 3. In the figure, the contour plot shows the distribution of the vertical effective stresses *S22* in kPa.

2.2.1 Selection of the soil parameters

A comprehensive parameter study was performed in which a number of parameter sets were retrieved from both element tests and CPTs. In this paper, the results of two relevant sets of parameters will be shown. Throughout the article, these parameter sets will be addressed as Set-1 and Set-2. The only common material parameter between the two sets is the Poisson's ratio. This was assigned equal to 0.3 for the soil elements as well as for the pile elements.

Set-1 represents a potential parameter set that does not account for any parameter change as a consequence of the pile installation process. The friction angle and the Young's modulus were calculated on the basis of the CPT results according to Kulhawy and Mayne (1990) and Lunne and Christopherson (1983), respectively. The dilation angle was conventionally calculated as $\psi' = \phi' - 30°$. The lateral earth pressure coefficient, K, was taken as the lateral earth pressure coefficient at rest for normally consolidated soils, $K_0 = 1 - \sin(\phi')$. The steel-soil interface angle, δ, was conventionally taken as two thirds of the sand friction angle.

The parameters of Set-2 embody an attempt to take the effects induced by the impact-driving into account. The friction angle was taken equal to critical friction angle, $\phi'_{cr,tx}$. The Young's Modulus of the sand was calculated according to the CPT-based approach of Robertson and Cabal (2015). The critical state framework would impose a dilation angle equal to zero when the critical friction angle is used. However, to avoid potential numerical instabilities, ψ' was set to 0.1°. Following the indication of Yang et al. (2015), δ was taken equal to 29°. According to Randolph (2003) the lateral earth pressure coefficient has been calculated as follows:

$$K = K_{min} + (K_{max} - K_{min})e^{-\mu h/D} \quad (1)$$

where μ is an empirical parameter which may be taken equal to 0.05 and h is the distance from the pile toe to a given point along the pile shaft. K_{min} was calculated following the Rankine theory for active soil pressure:

$$K_{min} = K_a = \frac{1-sin(\phi')}{1+sin(\phi')} \quad (2)$$

Table 1. Soil parameters of Set-1 for each sand layer

Layer Nr. (depth [m])	*E* [MPa]	*ϕ'* [deg]	*ψ'* [deg]	*δ* [deg]	*K* [-]
Layer 1 (0 - 1)	10.6	42.8	12.8	28.5	0.32
Layer 2 (1 - 2.5)	24.1	44.4	14.4	29.6	0.30
Layer 3 (2.5 - 4.5)	31.4	44.2	14.2	29.5	0.30
Layer 4 (4.5 - 7)	38.5	44.7	14.7	29.8	0.30

Table 2. Soil parameters of Set-2 for each sand layer

Layer Nr. (depth [m])	*E* [MPa]	*ϕ'* [deg]	*ψ'* [deg]	*δ* [deg]	*K* [-]
Layer 1 (0 - 1)	7.3	28.4	0.1	29	2.65
Layer 2 (1 - 2.5)	21.0	28.4	0.1	29	2.1
Layer 3 (2.5 - 4.5)	34.0	28.4	0.1	29	2.25
Layer 4 (4.5 - 7)	60.0	28.4	0.1	29	2.72

K_{max} was taken as proposed by Randolph (2003):

$$K_{max} = 0.01\frac{q_c}{\sigma'_{v0}} \quad (3)$$

where q_c and σ'_{v0} are respectively the CPT cone resistance and the initial effective vertical stress at a given depth. The parameters of the two sets are listed in Table 1 and Table 2.

3 PRESENTATION OF THE RESULTS

Selected results from the numerical analysis and the physical model tests are presented in the following two sections. These will be compared in Section 4.

3.1 Experimental results

The model pile was tested in a load controlled manner following the German Recommendations on Piling (EA Pfähle, 2013).

The test was executed in two phases. In the first phase the force applied reached approximately one third of the expected ultimate capacity. The pile was then unloaded and reloaded until a head displacement of around 60 mm (> 0.2·D) was achieved. The time series of the applied force and the related displacement are illustrated in Figure 4.

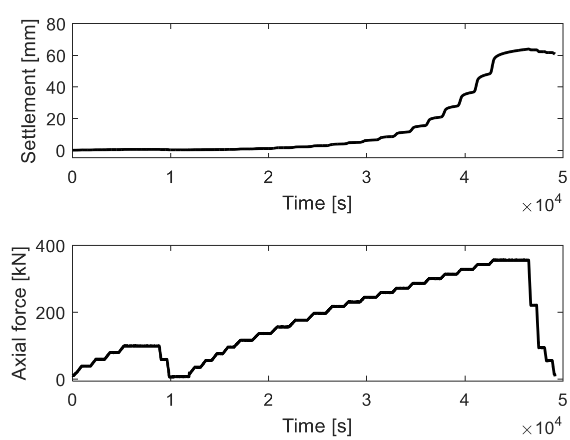

Figure 4. Time series of axial force and relative displacement of the test pile (Wisotzki et al., 2018)

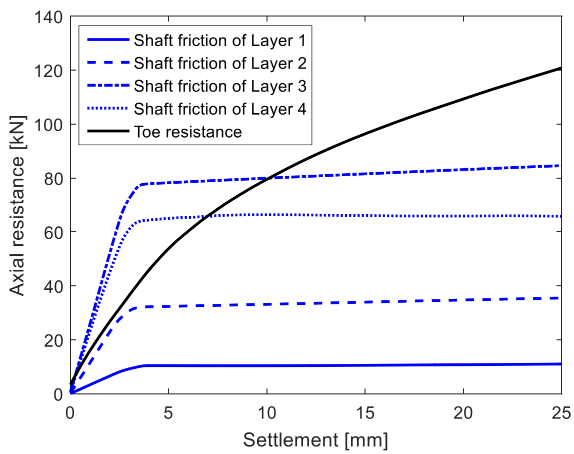

Figure 5. Contribution of shaft friction and toe resistance from Step 2 to Step 3 with Set-2

It is evident that a substantial creep of the geotechnical system occurred only in the second phase of the test. For the comparison with the numerical results a backbone load-displacement curve was considered.

3.2 *Numerical results*

Figure 5 shows the different contributions of resistance that the pile offers to the applied vertical displacement for Set-2. It can be noted that Layer 3, beeing the thickest layer in contact with the pile shaft, shows the largest resistance. The results seem to be realistic as the shaft friction contribution tends to be mobilised very soon while the toe resistance needs more displacement to be activated. By superimposing the shaft friction contributions shown in Figure 5, it is easy to verify that the geotechnical system turns out to be a skin friction pile foundation (axial resistance mainly offered by the shaft friction). While this is true for Set-2, it is conversely not the case when simulating with the parameter of Set-1.

The contour plots of the displacement fields around the pile toe at the end of Step 3 are depicted for Set-2 in Figure 6. As expected, the largest displacements occur very close to the pile.

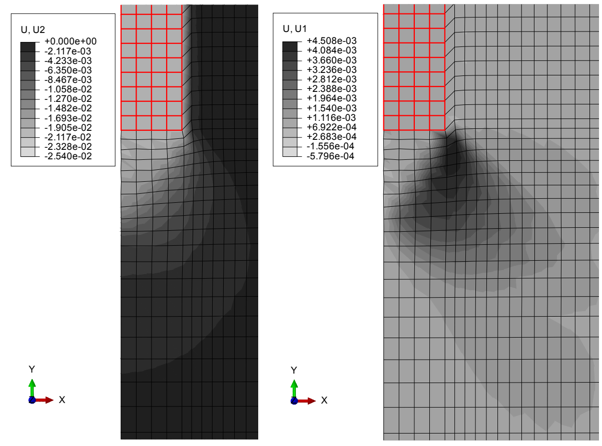

Figure 6. Contour plot of horizontal and vertical displacements at the pile toe at the end of Step 3 for Set-2. a) Vertical displacement U2 (+) up (-) down. b) Horizontal displacement U1 (+) to the right (-) to the left. The pile is marked in red

Both images reveal an influence area between 3 and 4 times the pile diameter.

4 COMPARISON OF NUMERICAL AND PHYSICAL MODEL

Figure 7 gives a comparison between the experimental curve and the numerical predictions for axial resistance. The curve simulated with Set-1 provides a very poor estimation of the geotechnical system. It underestimates considerably the ultimate capacity as well as the stiffness both initially and towards failure. The prediction obtained by accounting for the installation effects (i.e. Set-2) is able to predict very well the ultimate capacity and gives an acceptable representation of the initial stiffness. As already mentioned in Section 1, the effect of installation seems to be once more essential to a realistic modelling of piled foundations. The authors are aware that the parameters of Set-2 are rather unusual. Nevertheless their derivation is based on highly regarded literature and it cannot be excluded that these parameters allow a plausible reproduction of the

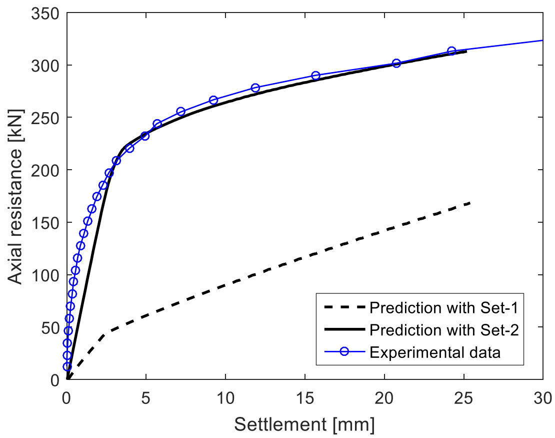

Figure 7. Load-displacement curves. Experimental data against prediction

physical mechanisms that dominate the pile behaviour. This approach should now be further investigated by testing other piles, other pile geometries and other loading conditions (e.g. axial tensile load). Most importantly, pile tests instrumented with strain gauges should be considered so as to be able to discern between shaft friction and toe resistance and obtain thereby an experimental proof of whether the pile is skin friction dominated or not. Furthermore, the numerical model of the pile could also be improved by using a more complex material model or by defining the contact interface between pile and soil more realistically. Finally, an even more appropriate way to model the complex soil-pile-interaction after the pile installation remains the complete modelling of the installation process as, for instance, attempted in Heinrich et al. (2018) for a jacked pile.

5 CONCLUSIONS

The test pit of the Test Center for Support Structures in Hanover affords the unique possibility to perform large-scale geotechnical tests which can be used to accurately validate geotechnical models. In this work, this approach was applied to a FEM simulation of an impact-driven piled foundation subjected to compressive loading. A simple axial symmetric finite element model of a wished-in-place single pile was used for the comparison between the experimental curve and the prediction. Conventional soil parameters implemented in an elastoplastic framework seem to lead to a significant underestimation of the behaviour shown in the experiment. By modifying the lateral earth pressure coefficient with an empirical method which takes the installation effects into account, and by setting the friction angle equal to the critical friction angle, the prediction delivers a much better fit. This preliminary work should not be regarded as a recommendation on how to model pile behaviour with a numerical model, but rather as a further confirmation that pile installation effects play a crucial role and their consideration, whether empirical or numerical, is essential to the design of piled foundations.

6 REFERENCES

Achmus, M., Thieken, K. 2010. On the behaviour of piles in non-cohesive soil under combined horizontal and vertical loading. *Acta Geotechnica* **5**, 199-210.

Bohn C., dos Santos A.L., Frank R. 2016. Development of Axial Pile Load Transfer Curves Based on Instrumented Load Tests. *Journal of Geotechnical and Geoenvironmental Engineering* **143**(1).

De Nicola, A., Randolph, M.F. 1993. Tensile and compressive shaft capacity of piles in sand. *Journal of Geotechnical Engineering*, **119**(12), 1952-1973.

Dijkstra, J. 2009. On the Modelling of Pile Installation. Dissertation, Technical University of Delft.

Deutsche Gesellschaft für Geotechnik 2013. *Recommendations on Piling (EA-Pfähle)*, Ernst & Sohn.

Engin, H.K. 2013. Modelling Pile Installation Effects - A Numerical Approach. Dissertation, Technical University of Delft.

Heinrich, D., Quiroz, T., Schenk, A. 2018. Validation of Coupled Eulerian-Lagrangian (CEL) method by means of medium and large scale foundation testing. Proceedings of the 9th European Conference on Numerical Methods in Geotechnical Engineering in Porto, Portugal.

Henke, S., Grabe, J. 2008. Numerical investigation of soil plugging inside open-ended piles with respect to the installation method. *Acta Geotechnica* **3***,* 215-223.

Kulhawy, F.H., Mayne, P.W. 1990. Manual on Estimating Soil Properties for Foundation Design, Report EPRI EL-6800, Electric Power Research Institute, Palo Alto, California.

Lehane, B.M., Schneider, J.A., Xu, X. 2005. A review of design methods for offshore driven piles in siliceous sand. UWA Report-GEO05358.

Lunne, T., Christoffersen, H.P. 1983. Interpretation of Cone Penetrometer Data for Offshore Sands. Offshore Technology Conference OTC-4464-MS.

Phuong N.T.V., van Tol, A.F., Elkadi A.S.K., Rohe, A. 2016. Numerical investigation of pile installation effects in sand using material point method. *Computers and Geotechnics* **73**, 58-71.

Randolph, M.F. 2003. Science and empiricism in pile foundation design. *Géotechnique* **52**(10), 847-875.

Robertson, P.K., Cabal, K.L. 2015. *Guide to Cone Penetration Testing for Geotechnical Engineering: 6th Edition*. Gregg Drilling & Testing, Inc.

Tomlinson, M., Woodward, J. 2014. *Pile Desing and Construction Practice: Sixth Edition*. CRC Press.

Yang, Z., Jardine, R., Guo, W., Chow F. 2015. A Comprehensive Database of Tests on Axially Loaded Piles Driven in Sand. Zhejiang University Press, Elsevier.

Wisotzki, E., Foglia, A., Schallert, M., Quiroz, T. 2018. Ermittlung des Pfahlgruppeneffektes einer Zweierpfahlgruppe offener Stahlrohrpfähle in Sand. *Geotechnik* **41**(3), 174-185.