Proceedings of the 21st ICSMGE, Vienna, Austria, 14 – 19 June 2026. Pistrol, Adam & Schweiger (eds.)
Published by: ÖGG, Austrian Society for Geomechanics, Salzburg, Austria, ISBN 978-3-9503898-4-5
https://doi.org/10.53243/ICSMGE2026-355

Numerical modelling of composite foundation systems for offshore renewables

Kenneth Gavin
InGEO Consulting Engineers, Delft, Netherlands

Yeganeh Attari
Multiconsult Norge AS, Oslo, Norway; Yeganeh.attari@multiconsult.no

Ozan Alver
InGEO Consulting Engineers, Delft, Netherlands

Gulin Yetginer
Equinor, Stavanger, Norway

ABSTRACT: The offshore renewable industry uses a range of foundation systems originally developed for the oil and gas sector. Innovative foundation measures are required as we move to deeper water depths offshore, to address key differences in scale, loading conditions and the geographical areas in which renewable developments are deployed. The supply chain is struggling with a number of issues such as increased pile and vessel size, uncertainty with regard to the installation location and capacity of anchor systems. Composite foundations where small piles (tubes or sheets) can be arranged in efficient shapes have been investigated by researchers in the past for onshore projects. In the offshore environment, such solutions could be economically advantageous due to the reduction in vessel sizes required for construction work. In this paper, the technical feasibility of such foundation concepts is examined through a series of 3D finite element analyses. In the first part of the paper, onshore field tests of composite foundation systems are modelled to verify the accuracy of the model. In the next section, the geotechnical conditions at a Dutch Offshore Wind Farm site are used to assess the potential for replacing conventional large diameter pile foundations with modular foundation groups.

KEYWORDS: Composite foundations, Offshore piles, Finite Element Modelling.

1 INTRODUCTION

Many countries across the globe have ambitious targets for energy generation from offshore renewable sources. The most advanced technology to date is offshore wind. Fixed bottom and floating offshore wind farms are seen as key to achieving this aim. Siting turbines offshore provides several benefits including availability of high unrestricted wind speeds and the ability to use larger turbines. To date many of the foundation systems deployed have built on experience in the oil and gas sectors, See Figure 1.

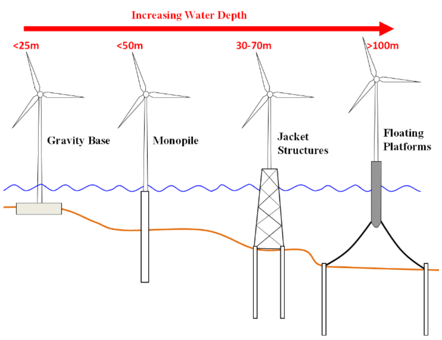

Figure 1. Typical offshore foundation systems

For water depths in the range 30 to 75m that characterize many offshore wind development sites, piled foundation systems give an economical solution. Monopiles and many jacket structures use large diameter open-ended steel piles installed in the sea bed to provide vertical and/or lateral load resistance. In deep-water, floating platforms will be deployed and support systems could include piles, suction anchors, drag anchors etc. Whilst the foundations systems are well-known and have been deployed successfully for many projects key differences in the loading experienced, geographical extent of wind farm developments and global location of resources result in many geotechnical challenges for such developments. In this paper we identify some piling challenges facing the offshore wind sector. Using field test data for validation we explore whether finite element analyses can accurately model the complex soil-structure interaction problem. In the final section we compare the pull-out capacity of a composite pile foundation with one installed using conventional impact hammering in typical North Sea soil.

2 OFFSHORE CHALLENGES FOR PILE FOUNDATION

Open-ended tubular piles are widely used offshore, either to support multiple legs of a jacket structure or a single, monopiles. Piles used for jacket structures that support predominantly vertical loads are typically 1m to 2m in diameter whilst to resist the large horizontal and moment loads, monopile diameters of up to 10m are common. These larger piles require very large installation vessels and have weights of the order of 1800 tonnes. Whilst some initial self-weight penetration of piles placed on the seabed is normal, See Duffy et al. 2025, a number of recent cases of pile run (uncontrolled penetration) have been recorded, See Dyson et al. 2025. In such cases uncontrolled displacement of the pile can result in pile loss and damage to equipment including vessels, cranes etc. Another challenge with the increasingly large dimensions of offshore piles is noise effects during installation. Conventional installation is by dynamic driving. As pile diameters increase, the sound pressure increases (Bellmann et al. 2020) potentially causing harm to marine mammals. A number of parties are looking at ways of reducing these impacts either through novel hammer systems, installation using vibration or the adoption of mitigation effects such as bubble curtains. In such instances, the use of a composite or cell foundations where a number of smaller piles are arranged to work in groups could provide an environmentally friendly and cost-effective solution.

Yetginer et al. (2003) describe a series of load tests on piles jacked-in place using the silent piling method developed by

Giken (www.giken.eu). The performance of the cell foundation shape shown in Figure 2 (left) was compared to a single pile. The piles were installed at the Takasu Research Centre in Kochi, Japan at a site where the Cone Penetration Test, CPT end resistance, q_c is shown in Figure 2 (right) and soil stratigraphy is summarised in Table 1. The piles with an external diameter of 101.6mm and wall thickness of 5.7 mm were jacked to a final penetration depth of 5.85m. The final installation force for each pile was recorded using a load cell.

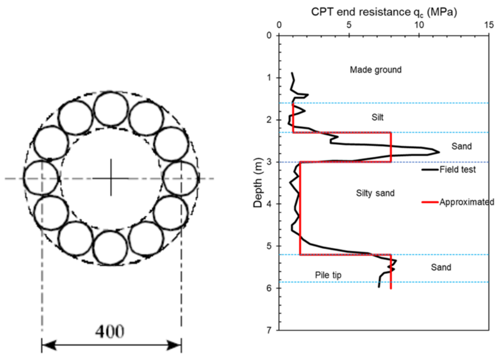

Figure 2. Left) Dimension of the composite foundations tested by Yetginer et al. 2003 (dimensions in mm), Right) Cone Penetration Test end resistance, qc

Table 1. Summary of the soil type

Depth (m)	Soil type
0 to 1.6	Fill Sand
1.6 to 2.3	Silt
2.3 to 3	Sand and Gravel
3 to 5.2	Silty Sand
5.2 to 9.4	Sand
0 to 1.6	Fill Sand

3 3D FINITE ELEMENT ANALYSIS FOR PILE CELL

3.1 *Introduction*

Analyses were conducted in PLAXIS 3D to simulate the behaviour of both single piles and pile groups. The model dimensions were 6 m × 6 m × 10 m in the x, y, and z directions, respectively. The lateral boundaries were fixed in the normal direction, while the bottom boundary was fully constrained. Due to the closely spaced piles, the fine mesh option was selected, along with an additional global coarseness refinement (set to 0.3, compared to the default value of 1.0). A total of 46,603 solid elements and 76,529 nodes were generated for the soil domain. The PLAXIS 3D model is illustrated in Figure 3.

3.1 *Soil model*

The steel tubular piles were modelled using shell elements with linear elastic material behavior. The pile diameter was geometrically represented as a 101.6 mm circle, while the shell thickness was assigned to simulate the bending stiffness. The soil domain was modelled using solid elements. Hardening Soil (HS) model was adopted as the soil constitutive model for the soil, with parameters estimated from the CPT-based correlations. The soil profile mainly consists of sands and silts, and the top made ground layer was replaced with sand according to Yetginer et al., 2003. For this layer, an average cone resistance (qc) of 5 MPa was assumed, corresponding to medium dense sand with a relative density of 61 %. In this study, all soil layers were assumed to be cohesionless, and relative densities were estimated based on CPT results (Kulhawy and Mayne, 1990). The deformation modulus and strength parameters were then derived using correlations proposed by Kulhawy and Mayne (1990) and Bolton (1996), respectively. The soil parameters used in the numerical analysis for HS model are summarized in Table 2.

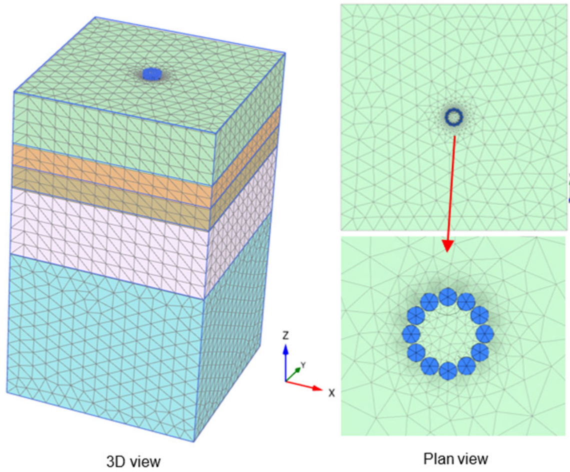

Figure 3. Plaxis model for the circular pile group

Table 2. Soil parameters in HS model

Soil type	Z (m)	qc (MPa)	DR (%)	$E_{oed,ref}$ (MPa)	$E_{50,ref}$ (MPa)	$E_{ur,ref}$ (MPa)	φ (°)	ψ (°)
Sand	0 / 1.6	5	61	56	71	213	41	10
Silt	1.6 / 2.3	1	22	14	15	46	31	0
Sand	2.3 / 3.0	8	58	53	66	198	40	9
Silty sand	3.0 / 5.2	1.5	22	15	16	47	31	0
Sand	5.2 / 10.0	8	48	43	52	156	39	7

3.2 *Comparison of Numerical Model and Field Test*

The initial stage of the modelling process involved establishing the geostatic stress conditions (K_0 state). The tubular pile was then activated as wished in place, with a rigid massless pile cap used to apply the pile head load. Since the small-diameter piles were reported to be fully plugged (Yetginer et al., 2003), an additional rigid cap was placed at the pile tip in the numerical model. The installation process was simulated by pushing the piles into the ground to a depth equal to 10 % of the pile diameter (10 mm). This was achieved by applying a pile head load until the target displacement was reached.

The second stage of the analysis involved unloading, during which the pile head load was reduced to zero. However, the displacement did not return to its original position, indicating a residual load at the pile tip. In the final stage, a pile head loading was applied to the top cap, and the resulting load-displacement result response was compared with the experimental results, as shown in Figure 4. The comparison indicated that both initial stiffness and ultimate load were well-captured in the numerical analysis. This outcome suggests that the residual stresses generated during the installation stage were realistically represented in the model.

The installation procedure described above was repeated for each pile in the group, and the resulting installation load-settlement curves at the pile heads are presented in Figure 5. The figure shows that both the stiffness and ultimate installation load increase with the presence of adjacent piles. The residual load at the pile base during installation contributes to higher installation loads, as expected.

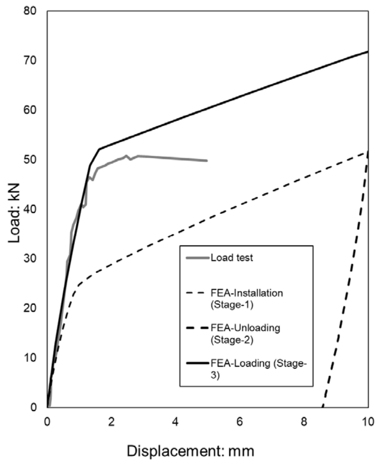

Figure 4. Comparison of single pile load test with the FEA results along with the pile installation and unloading curves

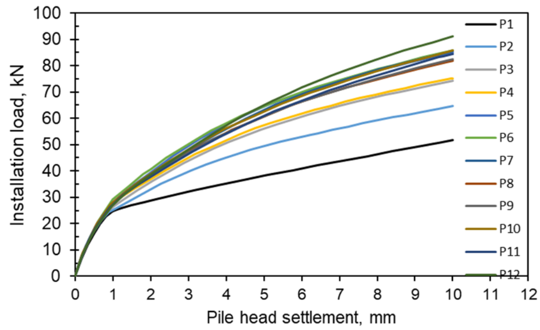

Figure 5. Pile head load-settlement curves during the installation stage in FEA

Figure 6 represents a comparison between the final pile installation loads obtained from the numerical analysis and those from the field test. The general trend was well captured by the numerical model, particularly up to pile number 7. However, for piles 8-12 the field test loads are substantially higher than those predicted by the numerical model. This discrepancy may be attributed to non-homogenous soil conditions in the field or potential stiffening effect from prior construction activities.

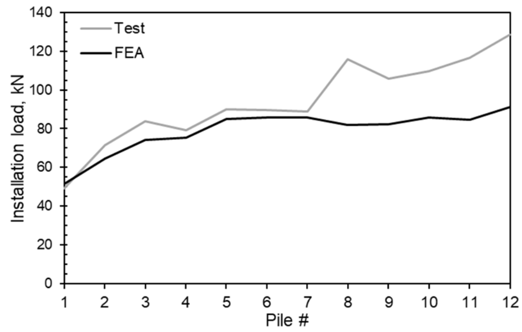

Figure 6. Comparison of pile installation loads of the field test and FEA simulations

3.3 *Pile group (cell) response*

After the installation of 12 piles, a load test performed on the pile group mobilized a maximum load of 600 kN. Figure 7 compares the FEA results with the field test data. The comparison shows that both the initial stiffness and ultimate load of the pile group response were underestimated in the numerical analysis. This discrepancy can be attributed to the lower installation resistances of singles observed in the model (as shown in Figure 4). The same factors that contributed to the lower installation resistance of single piles likely led to a softer overall pile group response.

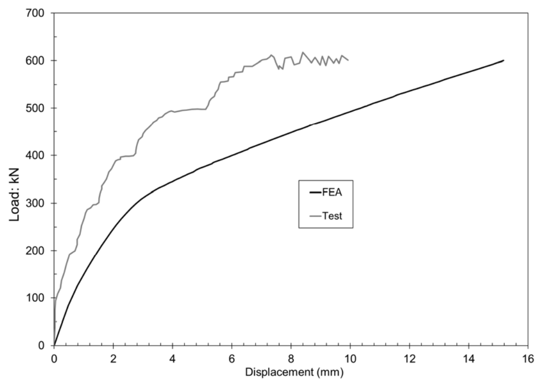

Figure 7. Comparison of group load test field vs FE prediction

Notably, Yetginer et al. (2003) reported that reaction frames used in the pile load tests were positioned 600 mm (approximately 6D) away from the nearest test piles. While this spacing is adequate for isolating single pile behavior, the 12-pile group acts as a composite unit with an effective diameter of about 400 mm, reducing the spacing between the group and the reaction piles to approximately 2D. The proximity of the reaction piles may have stiffened the surrounding soil, enhancing the field-measured response of the pile group. This effect, which was not considered in the numerical model, could explain the underestimation of both stiffness and strength in the pile group response.

The load distribution measured in the FE analyses of the single pile is shown in Figure 8. This allows the distribution of shaft resistance with depth to be derived. The normalized shaft resistance, $\tau_f/q_c = 0.012$ was constant with depth. This value is in keeping with those measured on screw-displacement piles, See Duffy and Gavin 2025 who found because of the absence of friction fatigue during installation such piles could develop relatively high shaft friction.

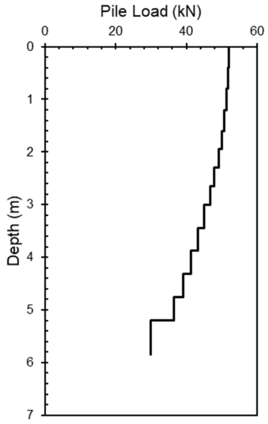

Figure 8. Load distribution with depth for single pile

4 CASE STUDY: NORTH SEA

In order to compare the behavior of a pile installed using conventional hammer driving versus a cell pile, a site typical of the Dutch sector North Sea Sand was considered. At the site the CPT q_c resistance was assumed to be constant with depth, q_c = 50 MPa. The ultimate pull-out capacity of the pile with depth increasing to 50m below sea bed level are compared assuming a pile diameter of 3.5m. The capacity of the pile installed using conventional driving was calculated using the ISO 2025 design method described by Lehane et al (2020). The pull-out capacity of the cell pile is superior to the conventionally driven pile over the entire depth of penetration. The benefit of the cell pile increases with increased penetration depth because of the influence of friction fatigue from conventional impact driving. At 10m below sea bed level the cell pile has double the capacity of the conventional pile, while at 50m below sea bed level the cell pile capacity is 3.5 times higher than the conventional pile. In practice the penetration depth of the cell pile would be much lower than the pile installed using conventional impact hammers.

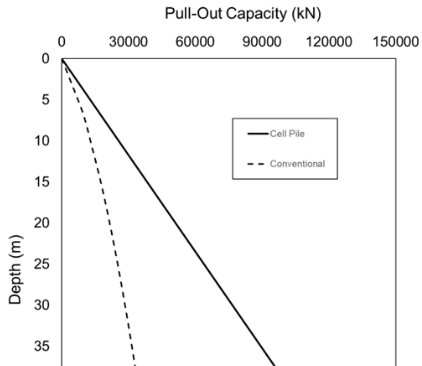

Figure 9. Pile penetration required using conventional hammer installation versus a cell pile

5 CONCLUSIONS

Offshore wind offers an environmentally friendly solution for our future energy needs and governments across the globe have ambitious development targets. In order to achieve these various technical and logistical challenges must be overcome. In this paper the use of modular or cell foundation systems pushed-into the sea bed are considered that address the following aspects, namely: (i) Decreasing pile sizes through the use of modular systems, (ii) Reducing vessel size through the use of remotely operated sea bed pile installation equipment, (iii) Reducing noise and vibration by avoiding pile hammering and (iv) Eliminating pile run problems as the piles are both much lighter and connected to the piling rig.

As full-scale offshore tests of novel foundation systems are in most projects prohibitively expensive, a first-step in proof of concept studies being undertaken by the authors is to perform finite element analyses. Given the complexity of the soil-structure interaction problem considered we used existing onshore field tests of a cell foundation structure to validate the capability of the FE model. The FE model allowed the normalized shaft resistance to be determined for a pile installed using push-in technology. In the final part of the paper the axial pull-out capacity of a cell pile installed using push-in technology was compared to a pile installed using a conventional impact hammer. The comparison suggested that significant saving on pile length could be achieved. This has significant implications for cost (including reduced material usage and installation time, smaller vessels) and reduces the environmental impact (C02 and noise).

6 ACKNOWLEDGEMENTS

The first and third authors acknowledge funding received from the European Union Project BEEYONDERS, Grant agreement ID: 101058548.

7 REFERENCES

Bellmann, Michael A., et al. (2020) "Underwater noise during percussive pile driving: Influencing factors on pile-driving noise and technical possibilities to comply with noise mitigation values." ERa Report: Experience report on piling-driving noise with and without technical noise mitigation measures.

Bolton, M. D. (1986). The strength and dilatancy of sands. Geotechnique, 36(1), 65-78.

Duffy KJ and Gavin KG (2025) CPT-based axial design of screw displacement piles in granular soils. Proceedings of the Institution of Civil Engineers – Geotechnical Engineering, https://doi.org/10.1680/jgeen.24.00441

Duffy, K. Reale, C. and Gavin. K.G. (2025). Analytical approach to predicting pile self-weight penetration, considering penetration rate effects, Ocean Engineering, Volume 338.

Dyson, A.P., Tolooiyan, A., Gavin, K.G., 2025. Evaluating clay stiffness effects on offshore pile running with the coupled eulerian lagrangian method. Computers and Geotechnics. 183,

ISO (2025). ISO 19901-4:2025: Oil and gas industries including lower carbon energy – Specific requirements for offshore structures – Part 4: Geotechnical design considerations. International Organization for Standardization.

Kulhawy, F. H., & Mayne, P. W. (1990). Manual on estimating soil properties for foundation de-sign (No. EPRI-EL-6800). Electric Power Research Inst., Palo Alto, CA (USA); Cornell Univ., Itha-ca, NY (USA). Geotechnical Engineering Group.

Lehane, B.M., Liu, Z., Bittar, E., Nadim, F., Lacasse, S., Jardine, R., Carotenuto, P., Rattley, M., Gavin, K.G., & others. (2020). A new 'Unified' CPT-based axial pile capacity design method for driven piles in sand. Proceedings of the 4th International Symposium on Frontiers in Offshore Geotechnics, (Z. Westgate (ed)), Austin, Texas, USA, pp. 462-477.

Yetginer, A. G., White, D. J. & Bolton, M. D. (2003). Press-in piling: field testing of cell foundations. Proceedings of the BGA international conference on foundations, Dundee, pp. 963–973.

Yetginer, A. G., D. J. White, and M. D. Bolton. "Field measurements of the stiffness of jacked piles and pile groups." Geotechnique 56.5 (2006): 349-354.