# Three Dimensional Numerical Modelling of Foundation Systems for Offshore Wind Energy Plants

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*Abstract:* Offshore wind energy plants promise to become an important source of energy in the near future. It is expected that within 10 years, wind parks with a total capacity of thousands of megawatts will be installed in European seas. The foundation of such offshore wind energy plants plays an important role in the stability of these structures. High demands have to be made concerning foundation design and construction methods to find economical and technically optimal solutions. Two foundation concepts which can be used in this field are the monopile and the tripod. Regarding design practice and research in the field of the soil-structure-interaction the behaviour of these foundations could be covered using numerical modeling. A three-dimensional numerical model using the finite element system ABAQUS was developed. In this model the material behaviour of the subsoil is described using an elasto-plastic constitutive model with Mohr-Coulomb failure criterion. This material law was extended in the elastic range by a stress-dependency of the modulus of elasticity. To describe the real behaviour of the structure, the interactions between the foundation system and the surrounding soil are modeled using contact algorithm based on slave-master concept. The results of Finite Element simulations will be presented and evaluated.

*Keywords*: Offshore, Monopile, Tripod, soil-structure-interaction, Abaqus, Mohr-Coulomb, slave-master-concept.

## 1. Introduction

Onshore wind energy has grown enormously over the last decade to the point where it generates more than 10% of all electricity in certain regions (such as Denmark, Schleswig-Holstein in Germany). However, due to noise and visual pollution, further expansion of onshore wind energy is limited.

Offshore installation costs are much higher than onshore, and the integration of the offshore wind farm into the electrical network is much more expensive. Economic construction and design methods are indispensable to make offshore wind energy competitive. At the time being, artificial competitiveness is reached by a law forcing energy suppliers to buy wind current at a fixed price. However, the costs of wind turbines are falling and are expected to continue doing so over the coming decade. Also, experience will be gained in building offshore wind farms, so that the offshore construction industry will likely find other cost-savings. Hence it is hoped that offshore wind energy will become really competitive with time.

## 2. Boundary Conditions

A number of different foundation types can be used for offshore wind energy converters (OWECs). The major types under discussion are the monopole and the tripod foundations. A monopile foundation consists of a large-diameter steel pile, which is simply a prolongation of the tower shaft into the ground. The monopile must be capable to transfer both lateral and axial loads from the structure into the seabed. The steel piles are of simple tubular construction which is inexpensive to produce and provide a low cost fabrication option. The tripod consists of a spatial steel frame transferring the forces from the tower to primarily tension and compression forces in three hollow steel piles driven into the seabed, located in the corners of a triangle (see Figure 1). In contrast to the monopile, the steel piles used are of lower diameters (less than 4.0 m).

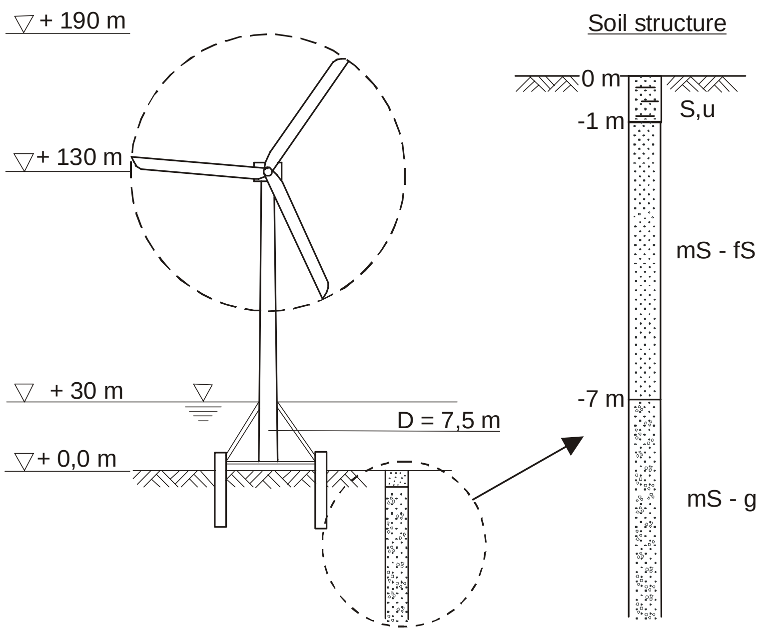


**Figure 1. Example of a tripod foundation**

The soil conditions in the North Sea and the Baltic Sea are characterized by Pleistocene and Holocene sediments. The pleistocene soils were strongly preloaded during ice ages and are thus highly over consolidated (e. g. boulder clays) or in dense state (sands and gravels). They are overlayed by Holocene soils of varying thickness like loose or medium dense silty sands or at some locations peat or mud.

The planned plants are to have hub heights of approximately 100 m and rotor diameters of approximately 120 m. The design procedure for OWEC foundations is in Germany given in the Germanische Lloyd rules and regulations (GL 1999). In this regulation concerning the behaviour of piles under horizontal loading reference is made to the regulation code of the American Petroleum Institute (API 2000). The Norwegian guidelines (DNV 2004) refer also to the API code.

## 3. Numerical Modelling of Monopile under Quasi-Static Loading

For the investigation of the load-deformation behaviour of monopiles of large diameters, three-dimensional finite element calculations were accomplished. Piles with a diameter of $D$ = 7.5 m having an embedded lengths under the sea-bed of $L$ = 30 m were investigated.

Different load application heights $h$ of the load above sea-bed and thus combinations of horizontal force $H$ and bending moment $M = H \times h$ were realized. Additionally, a vertical load was applied to take the structure's weight into account.

The computations were done with the program system ABAQUS (Abaqus 2006). Due to the symmetric loading condition only a half-cylinder representing the sub-soil and the monopile could be considered. The discretized model area had a diameter of 90 m, which is twelve times the pile diameter. The bottom boundary of the model was taken 15 m below the base of the monopile. With these model dimensions the calculated behaviour of the pile is not influenced by the boundaries (Fig. 2 left). The numerical computations were done on a super computer with parallel processors technology to reduce the CPU time and enhance the computation efficiency.

For the soil as well as for the pile 8-node continuum elements (C3D6&C3D8) were used. The frictional behaviour in the boundary surface between pile and soil was modelled by contact elements, whereby the wall friction angle was set to $\delta = 0.67\ \varphi'$, where $\varphi'$ is the friction angle for the subsoil (refer to Table 1).

Of crucial importance for the quality of the numerical computation results of soil structure interactions is the modelling of the material behaviour of the soil. The elasto-plastic material law with Mohr-Coulomb failure criterion, provided in the ABAQUS program, was used. This material law was extended in the elastic range by a stress-dependency of the oedometric modulus of elasticity with the following equation:

$$E_S = \kappa\, \sigma_{at} \left( \frac{\sigma_m}{\sigma_{at}} \right)^{\lambda} \qquad \text{Equation 1}$$

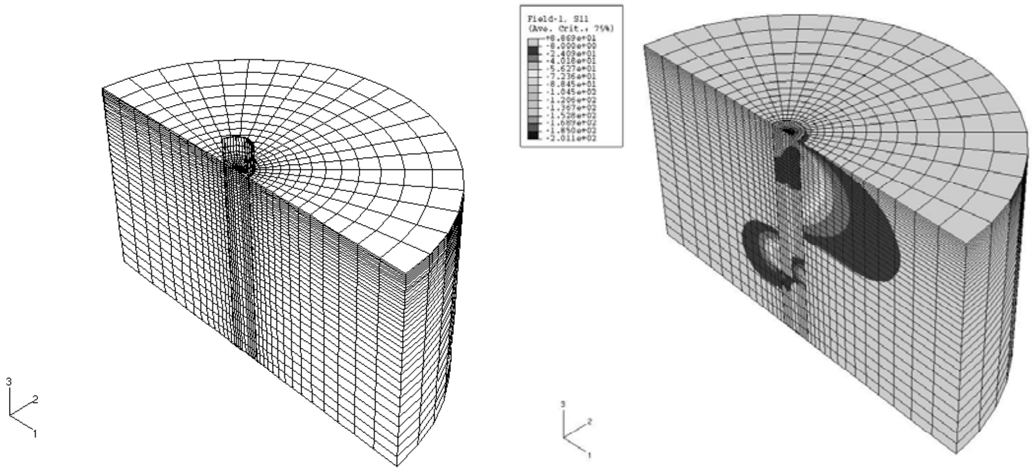

**Figure 2. Finite element mesh and horizontal bedding pressure (in the symmetry axis of the monopile) for D = 7.5 m, L = 30 m in dense sand, H = 8 MN, h/L = 1.**

Herein $\sigma_{at}$ = 100 kN/m$^2$ is a reference stress and $\sigma_m$ is the current mean principle stress in the regarded soil element. The parameter $\kappa$ determines the soil stiffness at the reference stress state and the parameter $\lambda$ rules the stress dependency of the soil stiffness. In the context of the computations presented here, the material parameters used with reference to EAU (1996) are shown in Table 1. The material behaviour of the monopile was assumed linear elastic with the parameters $E = 2.1{\cdot}10^5$ MN/m$^2$ (Young's modulus) and $\nu = 0.2$ (Poisson's ratio) for steel.

**Table 1: Material parameters used in the numerical computations**

| Material | Unit weight $\gamma'$ in kN/m$^3$ | Stiffness | | Poisson's ratio $\nu$ in 1 | Shear parameters | | |
|---|---|---|---|---|---|---|---|
| | | $\kappa$ in 1 | $\lambda$ in 1 | | $\varphi'$ in ° | c' in kN/m$^2$ | $\psi$ in ° |
| Sand, medium dense | 11 | 400 | 0.60 | 0.25 | 35 | 0.1 | 5 |
| Sand, dense | 11 | 600 | 0.55 | 0.25 | 37.5 | 0.1 | 7.5 |

The finite element calculation is executed stepwise. At first, for the generation of the initial stress state the whole model area is discretized using soil elements only. Subsequently, the monopile is generated by replacing the soil elements located at the pile position by steel elements and activating the contact conditions between pile and soil.

Then the vertical load is applied, and finally the horizontal load is applied and increased step by step. The monopile elements were extended above the ground surface of the model in order to realize different load combinations (horizontal forces and bending moments).

For a resultant horizontal load of 8 MN and a bending moment at sea-bed level of 240 MNm, which is in the order of a possible design load for the considered large water depths, the horizontal (bedding) stresses acting on the pile in the symmetry plane are shown in Figure 2, right. The characteristic loading behaviour of the pile with bedding stresses of opposite sign above and below a point of rotation can be seen clearly. For the considered case the point of rotation lies about 22 m below sea bed.

For a specific design problem force-head displacement and force-head rotation curves can be helpful, because especially the limitation of head rotation is of importance for the serviceability of the wind energy plant. As an example, such curves are given for D = 7.5 m, L = 30 m, dense sand in Figure 3.
In the API code the p-y method is recommended for the design of horizontally loaded piles. In principle, the p-y method is a subgrade modulus method with non-linear and depth-dependent load-deformation (p-y) characteristics of the soil springs. The comparison with API results (Lpile 2000) verifies that this method gives lower deformation than finite element calculations.

In case of the horizontal displacement (w) the deviation ranges between 50% and 100%. Similar results are obvious for the rotation ($\phi$) having a deviation between 20% till 40%.

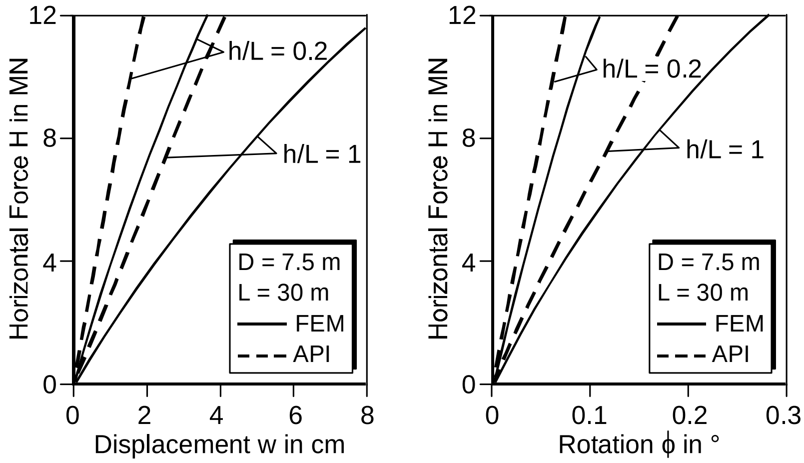


**Figure 3. Exemplary comparison of the pile deflection according to API method and numerical simulation for monopiles in dense sand (h = M/H)**

Of course, also the FE results do not necessarily represent exactly the true pile behaviour because assumptions have to be made concerning initial stress state and material behaviour and have thus to be checked. But, the findings give rise to the conclusion that the API method for large-diameter piles should be used with great care (refer to Achmus & Abdel-Rahman 2005).

Regarding the parametric studies, different pile diameters $D$, different pile lengths $L$ and different heights of point of load application $h$ were numerically simulated. Diagrams representing the pile head displacement $w$ and the pile head rotation $\phi$ at sea-bed level as a function of the horizontal load were determined.

For each combination of pile geometry and soil profile, two diagrams were derived (*H-w* and *H-$\phi$* curves) as a result. Figure 4 represents the results obtained for a monopile with diameter equal to 7.50 m and an embedded length of 30.0m.

For a similar specific site or a comparable soil profiles, a pre-dimensioning of a monopile foundation for static load design can be carried out on the basis of these diagrams.

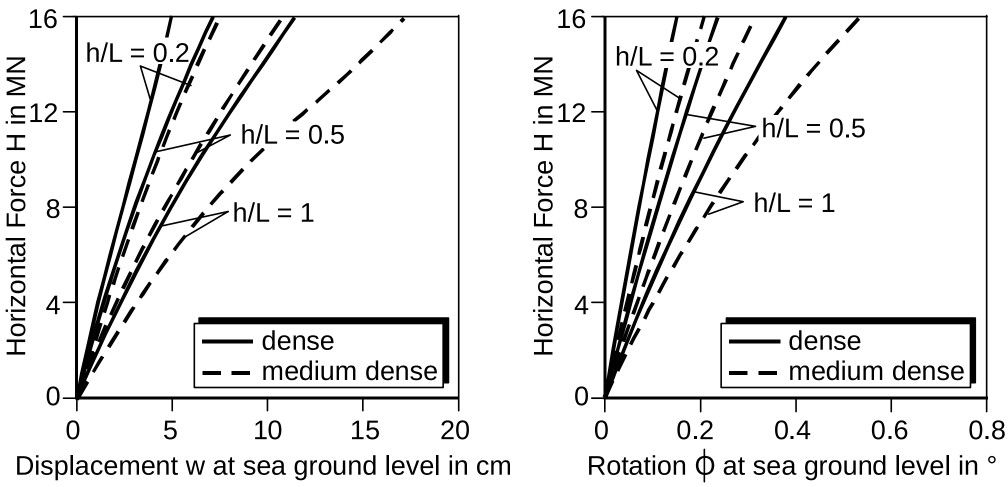


**Figure 4. Load-deformation curves for monopile D = 7.5 m, L = 30 m in medium dense and dense sands**

The expected influence of the soil profile and the height of the point of application of the horizontal load is evident from the Figure 4. In homogeneous sandy soil the deformations (both the displacement and the rotation) due for *h/L* = 1 are approximately double of the deformations for *h/L* = 0.2.

## 4. Numerical Modelling of Tripod under Quasi-Static Loading

For the investigation of the behaviour of a laterally loaded tripod, a three dimensional (3-D) numerical model was established. The computations were done with the same program system ABAQUS (Abaqus 2006).

For the main tower a diameter of D = 7.5 m and a wall thickness of 9.0 cm was assumed. This main tower was braced by three diagonal members at 45.0° having a diameter D = 2.0 m and a wall thickness of 4.0 cm and another three horizontal members having the same dimensions as the diagonal bracing transferring the loading to the legs (the piles).

The piles are braced together with three members having a diameter of 1.50 m and a wall thickness of 3.0 cm (Figure 5). The diameter of the supporting piles was chosen to be 3.50 m with a thickness of 6.0 cm. The embedded length was chosen to be 20.0 m.

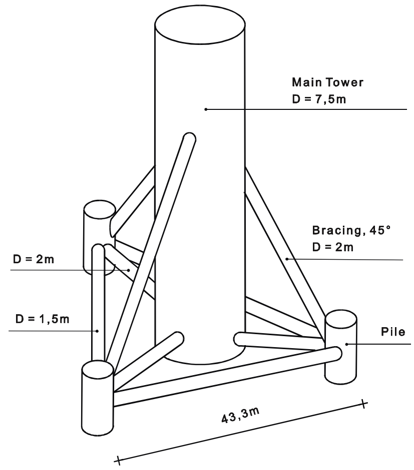


**Figure 5. Dimensions of the proposed tripod**

The loading consists of a resultant horizontal force acting at about 30.0 m above the sea bed level. Additionally, a vertical load was applied to take the structure's weight including the turbine and the rotor into account (the same vertical load being used for the monopile foundation). For the sake of simplicity by the numerical modelling, the soil regions surrounding the piles were discretized separately. The numerical model consists of three cylinders of a radius of 20 m, which is about six times the pile diameter. The bottom boundary of the model was taken 15 m below the base of the piles. With these model dimensions the calculated behaviour of the piles is not influenced by the boundaries. A view of the discretized model is given in Figure 6.

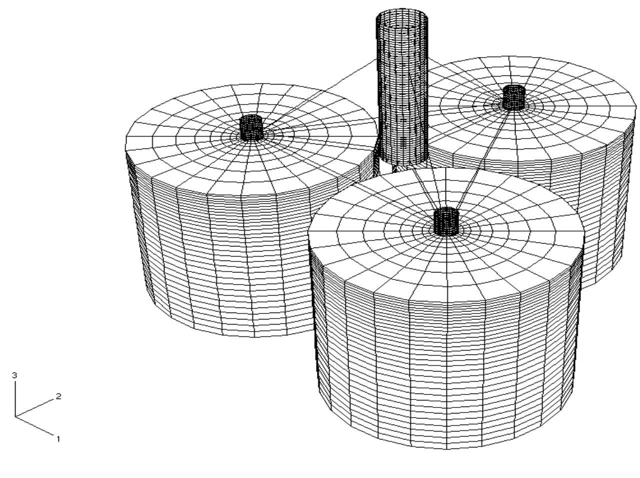

**Figure 6. Discretized finite element model**

For the soil as well as for the pile 8-node continuum elements (C3D6&C3D8) were used. For the bracing members a 3-nodes beam element (B32) were used. The interaction behaviour between the piles and the sandy soil is simulated using contact elements between both of them. The maximum shear stress in the contact area is determined by a friction coefficient. For the calculations presented herein this coefficient was set to $\mu = 0.67\ \varphi'$.

The material behaviour of sandy soil in general is very complex. In the case of monotonic loading considered here, an elasto-plastic material law with Mohr-Coulomb failure criterion was used with the same parameters as for the monopile modelling (refer to the previous part).

The material behaviour of the piles and the bracing was assumed linear elastic with the parameters $E = 2.1 \cdot 10^5$ MN/m$^2$ (Young's modulus) and $\nu = 0.2$ (Poisson's ratio) for steel.

The finite element calculation is executed stepwise in a similar way like by the monopile modelling. Taking into account the bracing members to be activated together with the pile and the tower in the second step. In the last step, the horizontal load is applied directly on the main tower on 30.0 m height above the sea level and increased incrementally.

Under a design load of H = 8 MN (the same horizontal force used in the monopole-case) acting in the direction of the y-axis (axis of symmetry), – annotated as 2-axis in Figure 6 - the horizontal displacement of the main tower and the piles are shown in Figure 7, the tower displacement at sea water level amounts to be about w = 4.50 cm and the tower rotation is about 0.11°.

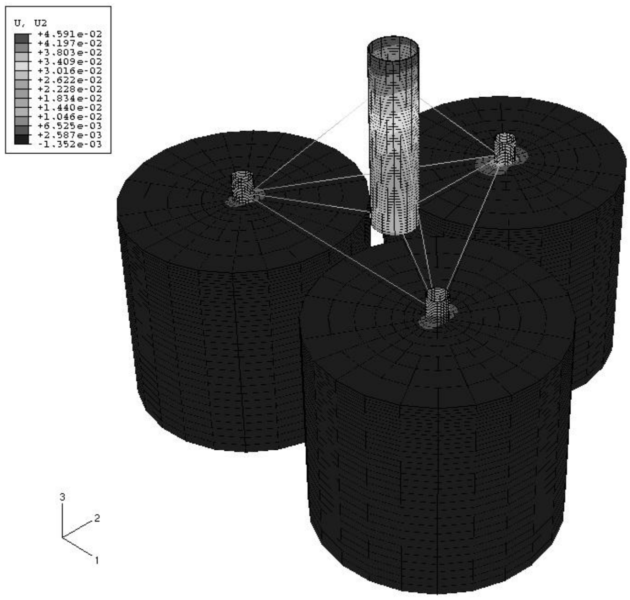


**Figure 7. Horizontal displacement of the main tower and the piles (medium-dense sand)**

Comparing these results with the monopile foundation system (Diameter=7.50m and embedded length 30.0m) under the same loading conditions, the tower displacement at sea water level was about 25.0 cm and the tower rotation was about 0.35° (Achmus & Abdel-Rahman 2005). This means that the bracing between the piles and the main tower are enhancing the performance of the foundation. The characteristic behaviour of the tripod including the bracing and the piles can be seen clearly in Figure 8. From this diagram, it is clear that the bracings are dominating the behaviour of the tripod and controlling the displacement of the piles.

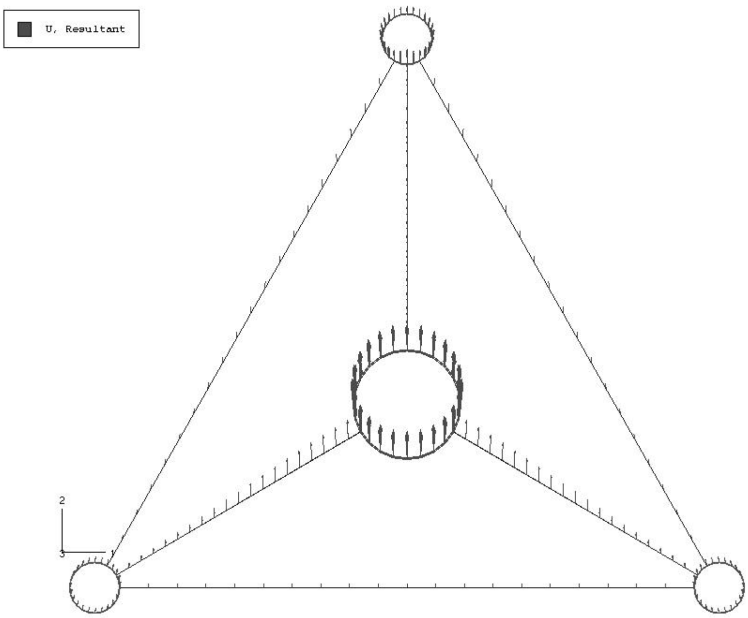


**Figure 8. Displacement field of the main tower and the carrying piles as vector-plot.**

A detailed behaviour of the tripod can be discussed concerning the loads carried by each of the three foundation piles (Table 2) under the considered loading case (neglecting the own weight of the structure):

**Table 2: Loads transferred to the vertical piles**

| Pile Nr. | $F_2$ [kN] | $U_2$ [cm] | $F_3$ [kN] | $U_3$ [cm] |
|---|---|---|---|---|
| 1 | 1800.0 | 0.0885 | -360.0 | -0.041 |
| 2 | 1800.0 | 0.0885 | -360.0 | -0.041 |
| 3 | 4400.0 | 1.867 | -9280.0 | -1.114 |

According to these results, it is obvious that the integrated modelling of the tripod with the surrounding soil is highly recommended to predict the actual behaviour of the tripod structure.

## 5. Conclusions

Thus, at the time being the execution of numerical investigations is recommended for the design of the foundation tripod systems for OWECs planned in the German offshore areas. Of course, such investigations are complex and time-consuming. Further investigations of the authors will be the verification of the numerical results for different soil profiles typically for the North Sea and the Baltic Sea and also for varying pile diameters and embedded lengths. Moreover, the behaviour of these foundation systems under cyclic loading must be and will be a subject of future research.

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