

Finite Element Modelling of Horizontally Loaded Monopile Foundations for Offshore Wind Energy Converters in Germany

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ABSTRACT: Offshore wind energy offers a huge potential for the expansion of renewable energies in Germany. High demands have to be made concerning foundation design and construction methods to find an economically and technically optimal solution. A promising foundation concept which can be used in this field is the monopile, which in principle is a prolongation of the tower shaft into the ground. The behaviour of large-diameter monopiles under the horizontal loading induced by wind and wave loads is analysed using a three-dimensional finite element model. Non-linear elastoplastic soil behaviour is taken into account. The results of the numerical simulations are presented and compared with the results of the p-y method commonly used for the design of laterally loaded piles.

1 INTRODUCTION

Offshore wind farms 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.

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.

These problems in the development of wind energy can be resolved by the installation of wind energy farms in offshore regions, e.g. in the North Sea and in the Baltic Sea. This solution has the following advantages:

- 1) availability of large continuous areas, suitable for major projects,
- 2) higher wind speeds, which generally increase with distance from the shore,
- 3) less wind turbulence, which allows the turbines to harvest the energy more effectively and reduces the fatigue loads on the turbine.

Of course, also problems arise in moving offshore. Offshore installation costs are much higher than onshore, and the integration of the offshore wind farm into the electricity network is much more expensive. Economic construction and design methods are in-

dispensable to make offshore wind energy competitive. Currently, artificial competitiveness is maintained in Germany 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 in time.

2 FOUNDATION REQUIREMENTS

A number of different foundation types can be used for offshore wind energy converters (OWECs). The major types under discussion are the monopile, the gravity and the tripod foundations.

A monopile foundation consists of a large-diameter steel pile, which is in principle simply a prolongation of the tower shaft into the ground. The monopile must be able 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 gravity foundation, unlike the piled foundation, is designed with the objective of avoiding tensile loads between the support structure and the seabed. This is achieved by providing sufficient dead

loads to stabilise the structure under the overturning moments which result from wind and wave action.

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. In contrast to the monopile, the steel piles used are of smaller diameters (less than 2 m).

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 overlaid by holocene soils of varying thickness like loose or medium dense silty sands or at some locations peat or mud (Lesny et al. 2002).

In the past OWECs established in the North Sea and in the Baltic Sea have rated power outputs of maximum 2 MW and are located at small distances from the coast at moderate water depths of up to about 8 m. Most of these structures are founded on monopiles.

The OWECs now planned in the German offshore areas shall have rated outputs of up to 5 MW and are thus much larger than the converters already installed. The planned plants are to have hub heights of approximately 100 m and rotor diameters of approximately 120 m (Figure 1). Moreover, the wind farms shall be installed in areas far away from the coast. At these locations, water depths from 20 m to 40 m (or even 50 m) are expected.

For the large depths the tripod foundation is the most promising foundation type. But, for depths of up to about 30 m the monopile foundation is thought to be an alternative. This foundation type is simple and elegant and has advantages concerning the damage risks in case of a ship collision. However, for monopiles with diameters in the range of 6 to 8 m, which are necessary for OWEC sizes and water depths described above, no experience exists concerning the load-deformation behaviour.

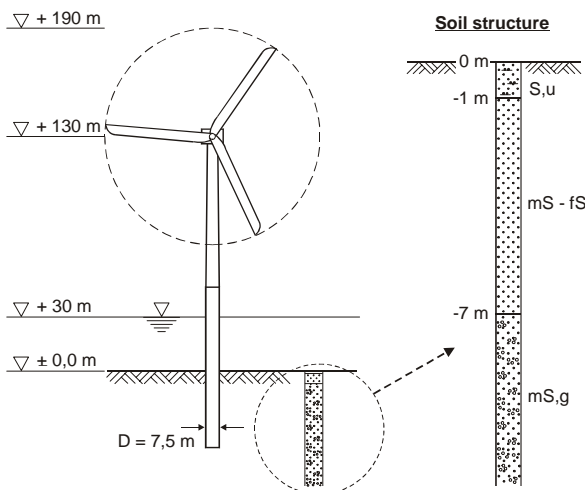


Figure 1. Monopile foundation of a wind energy plant.

The analysis of the behaviour of large monopiles under monotonic horizontal loading is the objective of this paper.

Assuming a water depth of 30 m and a maximum design wave height of 14.5 m, the design horizontal load for a monopile with a diameter of 7.5 m amounts to about 8 MN, the resultant horizontal force acting about 30 m above sea level, i. e. nearly at still water level. Thus, the corresponding bending moment at sea bed level is about 240 MNm.

Additionally a vertical load of 10 MN representing the own weight of the turbine, the blades and the tower was assumed. Such loads have to be considered analyzing the behaviour of monopiles.

3 COMMON DESIGN OF HORIZONTALLY LOADED PILES

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) also refer to the API code. 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. In the API code, the following procedure is given to construct p-y curves for sandy soils:

1) The ultimate lateral resistance per unit length p_u is taken as the minimum of two expressions. The first one is valid for shallow depths, whereas the second is valid for greater depths:

$$p_{us} = (c_1 z + c_2 D) \gamma' z \quad (1)$$

$$p_{ud} = c_3 D \gamma' z \quad (2)$$

Herein z is the given depth in metres, D is the average pile diameter in metres, γ' is the effective unit weight of soil (kN/m^3). The coefficients c_1 , c_2 , c_3 are dependent on the friction angle of the soil.

2) The p-y curve is given at a specific depth by the following expression:

$$p = A p_u \tanh\left(\frac{k z}{A p_u} y\right) \quad (3)$$

where $A = 3.0 - 0.8 z / D \geq 0.9$ for static loading and $A = 0.9$ for cyclic loading, p is the soil resistance

per unit length, y is the actual lateral deflection and k is the initial modulus of subgrade reaction determined as a function of the friction angle.

This method is verified only for piles with diameters of up to about 2 m. The question is to be answered, whether the method can be used also for the design of large-diameter piles.

In the following, results of numerical calculations of the load-deformation behaviour of monopiles are presented and compared with the results of the API p - y method. The calculations with the API method were carried out by means of the LPILE program (Lpile 2000).

4 FINITE ELEMENT MODELLING

4.1 Model features

For the investigation of the behaviour of laterally loaded monopiles with large diameters, a three-dimensional (3-D) numerical model was established.

The computations were done using the finite element program system ABAQUS (Abaqus 2004). In order to carry out many calculations for varying boundary and loading conditions, a large computer system with parallel processor technology was used to minimize the time effort.

The aim of the investigation was to analyse the behaviour of a large monopile in principle and to check whether the API method can be used for such large piles. For that, an idealized homogeneous soil consisting of dense sand was considered. A monopile diameter of $D = 7.5$ m and a wall thickness of 9 cm was assumed. The loading consists of a resultant horizontal force acting at a given height h above the sea bed level. The bending moment at sea level is thus $M = H \cdot h$. By variation of H and h any water depths or load combinations can be considered. Additionally, a vertical load was applied to take the structure's weight into account.

Due to the symmetric loading condition only a half-cylinder representing the sub-soil and the monopile was considered. The discretized model 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 lengths the calculated behaviour of the pile is not influenced by the boundaries. A view of the discretized model area is given in Figure 2.

For the soil as well as for the pile 8-node continuum elements were used. The interaction behaviour between the monopile and the sand soil is simulated using contact elements. The maximum shear stress in the contact area is determined by a friction coefficient. For the calculations presented herein this coefficient was set to $\mu = 0.4$.

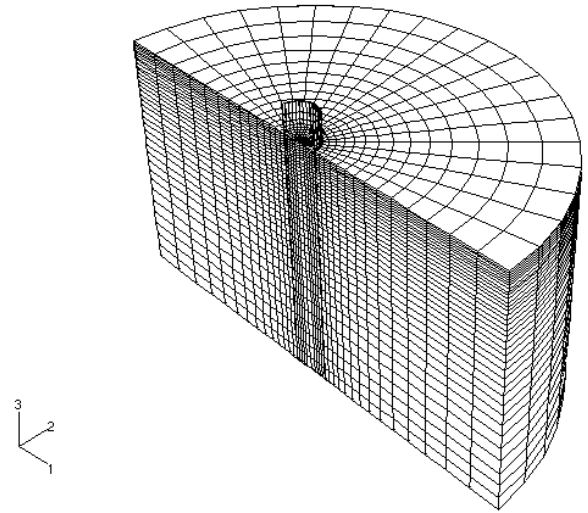


Figure 2. Finite element mesh.

The material behaviour of the monopile was assumed to be linear elastic with the parameters $E = 2.1 \cdot 10^5$ MN/m² (Young's modulus) and $\nu = 0.2$ (Poisson's ratio) for steel.

The material behaviour of sand and soil in general is very complex. In the case of monotonic loading considered here, essential requirements on the material law are the consideration of the non-linear, stress-dependent soil stiffness and the consideration of possible shear failure. An elasto-plastic material law with Mohr-Coulomb failure criterion was used. The soil stiffness is represented by a stiffness modulus for oedometric compression E_s and a Poisson's ratio ν . A stress dependency of the stiffness modulus was accounted as follows:

$$E_s = \kappa \sigma_{at} \left(\frac{\sigma}{\sigma_{at}} \right)^\lambda \quad (4)$$

Herein $\sigma_{at} = 100$ kN/m² is a reference (atmospheric) stress and σ is the current mean principal stress in the considered soil element. The parameter κ determines the soil stiffness at the reference stress state and the parameter λ rules the stress dependency of the soil stiffness.

The advantage of the material law used is that it can be generally used for non-cohesive as well as for cohesive soils. The parameters used for the calculations presented here are typical for a dense sand (EAU 1996) and are given in Table 1.

Table 1. Material parameters used for dense sand.

Unit buoyant weight γ'	11.0 kN/m ³
Oedometric stiffness parameter κ	600
Oedometric stiffness parameter λ	0.55
Poisson's ratio ν	0.25
Internal friction angle φ'	35.0°
Dilation angle ψ'	5.0°
Cohesion c'	0.0 kN/m ²

The finite element calculation is executed step-wise. 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).

4.2 Model results

A monopile with a diameter of $D = 7.5$ m and an embedded length below sea bed of $L = 30$ m is taken as the basic case. With an assumed water depth of 30 m the resultant design wave load acts at $h \approx 30$ m above sea bed level, i. e. $h/L \approx 1.0$.

For a resultant horizontal load of 8 MN, which is a possible design load for the considered water depth, the horizontal (bedding) stresses acting on the pile in the symmetry plane are shown in Fig. 3. 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 the horizontal force of $H = 8$ MN and the bending moment at sea bed level of $M = 240$ MNm the pile displacement at sea bed level amounts to about $w = 4$ cm and the angle of rotation to about $\phi = 0.17^\circ$.

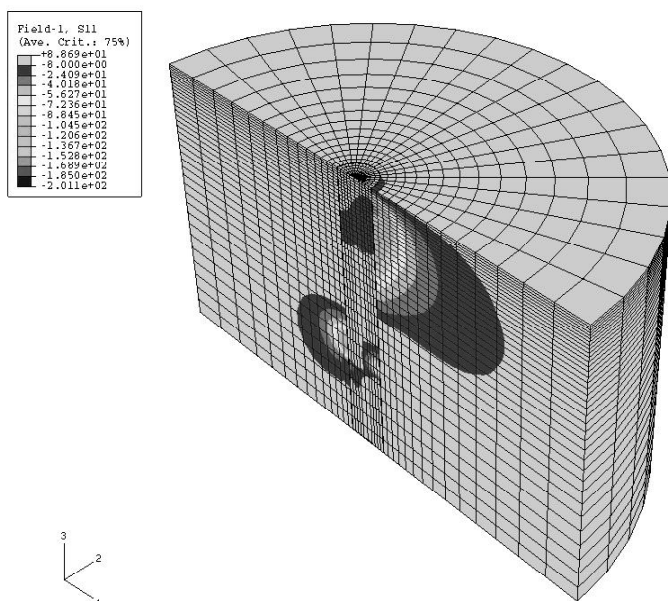


Figure 3. Mobilised bedding pressure for a monopile with $D = 7.5$ m, $L = 30$ m in dense sand, $H = 8$ MN, $M = H \cdot h = 240$ MNm.

From a practical point of view, the design procedure for a monopile foundation depends mainly on the calculation of the horizontal displacement and the pile rotation with respect to the applied loading conditions.

In Figures 4 and 5 the calculated force-displacement and the force-rotation relationships for the basic case considered are given. Also the results obtained with the API method are shown for comparison. With this method, the pile deformations are smaller and may thus be underestimated for large horizontal forces. However, for horizontal forces less than about 6 MN the results of the API method and the numerical calculations are in fairly good agreement.

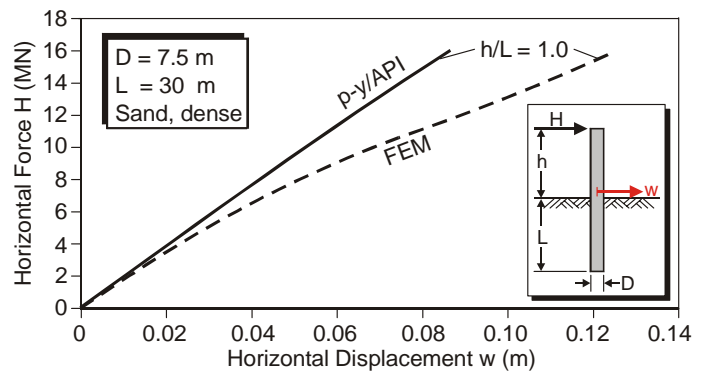


Figure 4. Comparison between finite element results and API method results regarding the horizontal displacement at sea bed level ($D = 7.5$ m, $h = L = 30.0$ m).

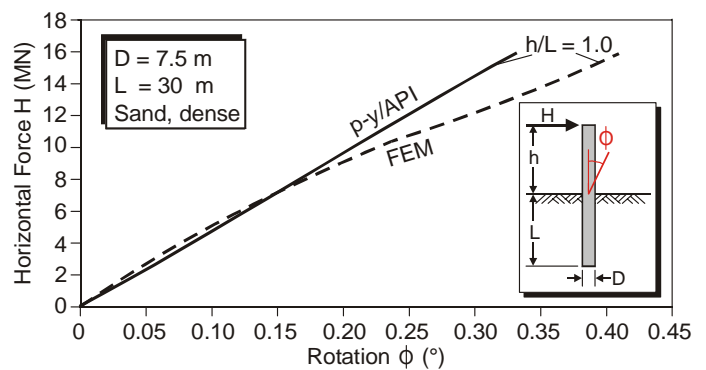


Figure 5. Comparison between finite element results and API method results regarding the rotation angle at sea bed level ($D = 7.5$ m, $h = L = 30.0$ m).

4.3 Variation of pile length and height of loading point

In the scope of a parametric study the height of the horizontal load above sea bed – and with that the bending moment – was varied in order to take different water depths or load combinations into consideration. Additionally, a pile with a smaller embedded length of $L = 20$ m was analysed.

The results are shown in Figures 6 and 7 as load-deformation and load-rotation curves, respectively.

The expected influence of the pile length and the height of the horizontal load is evident. Regarding this aspect it should be considered that with lower h/L -values also the design load is generally lower, because shallower water normally corresponds with lower design wave heights.

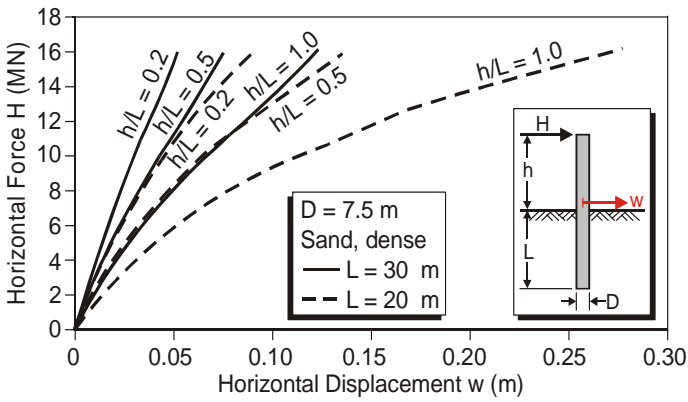


Figure 6. Force-displacement curves for a monopile $D = 7.5$ m, $L = 30$ m / $L = 20$ m embedded in dense sand.

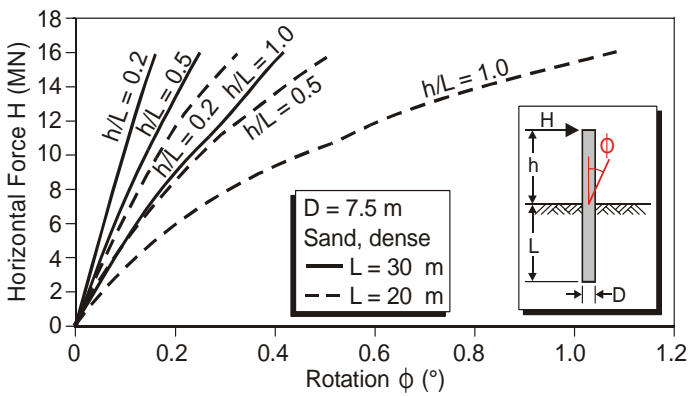


Figure 7. Force-rotation curves for a monopile $D = 7.5$ m, $L = 30$ m / $L = 20$ m embedded in dense sand.

5 CONCLUSIONS

According to the results of the numerical calculations carried out for monopiles of large diameter for high design loads, the p-y curve method given in API (2000) underestimates pile deformations. The main reason for these results is probably an overestimation of the initial soil stiffness in large depths by the API method. Moreover, for a large-diameter pile the shearing resistance in the pile tip area may play an important role compared to a small-diameter piles. Since this method is not verified by measurements at large-diameter piles, it should in general not be used for the design of monopile foundations.

Thus, for the time being the execution of numerical investigations is recommended for the design of the large monopiles planned in the German offshore areas. Of course it must be mentioned that also the

validity of numerical results are limited with respect to the accuracy of the material law and the parameters assumed. Thus, such results also need verification. However, in order to investigate the influence of the monopile diameter, the embedded length and the type of loading, the numerical model presented seems to be more reliable than the API method.

For preliminary design steps diagrams can be helpful, which allow a simple determination of the approximate pile deformations to be expected for a specific case.

As integral stiffness values of a monopile foundation two spring stiffnesses can be defined as follows:

$$\text{Lateral stiffness: } C_h = \frac{H}{w} \quad (5)$$

$$\text{Rotational stiffness: } C_\phi = \frac{M}{\phi}$$

Herein H and $M = H \cdot h$ are the load values, and w and ϕ are the pile displacement and rotation at sea bed level.

From the results of numerical calculations diagrams can be developed, from which these integral stiffness values can be determined. For the basic case considered herein such diagrams are presented in the Figures 8 and 9. For comparison also the respective curves obtained with the API method are given.

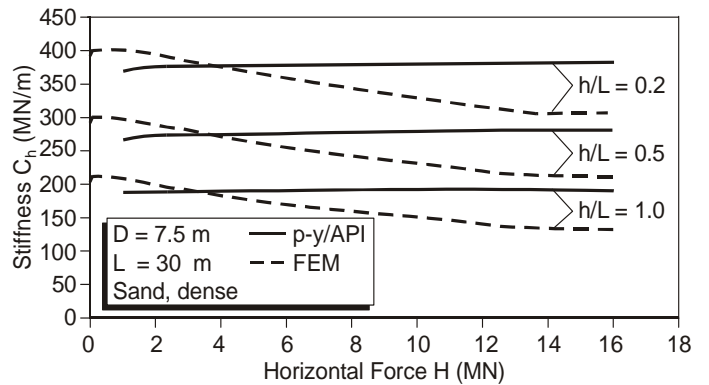


Figure 8. Equivalent lateral stiffness curves for a monopile $D = 7.5$ m, $L = 30$ m embedded in dense sand.

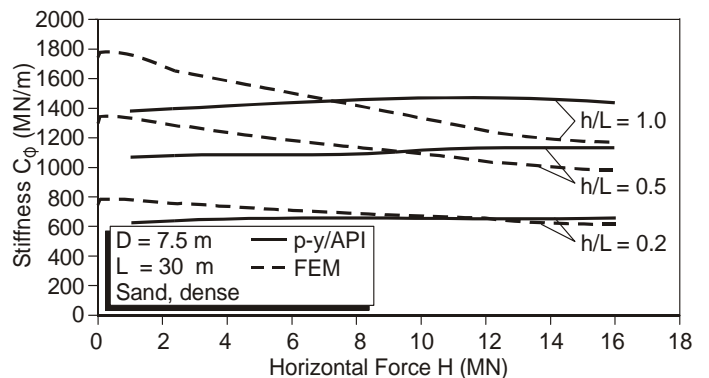


Figure 9. Equivalent rotational stiffness curves for a monopile $D = 7.5$ m, $L = 30$ m embedded in dense sand.

With the API method nearly constant stiffness values are obtained, which means that the monopile foundation stiffness is almost independent of the loading level. In contrast, with the numerical calculations a decreasing stiffness is obtained with increasing load level. The curves in Figures 8 and 9 show, that the load-deformation relationship obtained by the API method is nearly linear. Nonlinearities, which are to be expected for high load levels, are thus not covered by the method. The reason for this is that with the small displacements in relation to the diameter of the monopile only the nearly linear beginning portion of the API p-y curve becomes relevant (see also Achmus & Abdel-Rahman 2003).

Further investigations of the authors will be the derivation of diagrams for preliminary design of monopiles for other soil profiles typically for the North Sea and the Baltic Sea and also for varying pile diameters and embedded lengths. Moreover, the behaviour of monopile foundations under cyclic loading must be and will be a subject of future research.

6 ACKNOWLEDGEMENTS

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