Design of Monopile Foundations for Offshore Wind Energy Converters

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ABSTRACT: This paper aims to investigate the behaviour of large-diameter monopiles under monotonic and cyclic loading taking the interaction between the pile and the subsoil into account. A three-dimensional numerical model using the finite element method was established. A special numerical approach was developed to account for the effect of one-way cyclic loading. With this approach, the increase of pile displacements with the number of loading cycles can be quantified.

It is found that the p-y method in its present form is not suitable to account for the behaviour of large-diameter piles. The results of parametric studies for monopiles under monotonic design load and under cyclic loading are presented. Based on these results, considerations and recommendations are made concerning the design of large-diameter monopiles.

1. INTRODUCTION

The planned offshore wind farms in the German parts of North Sea and Baltic Sea will be constructed in water depths varying from approximately 15 to 40 m. By means of suitable foundation constructions, the large horizontal forces and bending moments resulting from wind and wave loads, must be economically and safely transferred to the sea soil. Monopile foundations can be used as one of these foundation types. This foundation method was already implemented for offshore wind energy converters (OWECs) in North and Baltic Sea, but only in water depths of less than about 10 m. Its application is expected to be extendable for water depths up to about 25 to 30 m. However, the diameters of such monopiles will vary between 5.0 and 7.5 m.

Since wind energy converters are relatively sensitive to deformations, in particular tilting, it is very important to estimate these as exact as possible. For the mentioned large-diameter piles, till now there is no approved procedure to do this.
2. STATE OF THE ART FOR THE COMPUTATION OF DEFLECTIONS OF HORIZONTALLY LOADED PILES

The design procedure for OWEC foundations in Germany is given in the Germanische Lloyd rules and regulations (GL, 1999). In these regulations, 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. API (2000) describes the construction of p-y-curves for soft and stiff clay as well as for sandy soils. Due to API, p-y-curves for sandy soils can be derived as follows:

- The maximum mobilized soil reaction force per unit length of the pile $p_u$ depends on the regarded depth under sea bed $z$, the submerged unit weight of the soil $\gamma'$, the pile diameter $D$ and on the angle of internal friction $\phi'$ of the sand:

$$p_u = (c_1 z + c_2 D) \gamma' z \quad (1.1)$$

$$p_{ud} = c_3 D \gamma' z \quad (1.2)$$

The first mentioned equation applies to small depths ($p_u$) and the second equation to larger depths ($p_{ud}$), the smaller of both values is to be considered. The influence of the internal friction angle is described by the factors $c_1$, $c_2$ and $c_3$ (see Fig. 1 left).

- The p-y-curve is described by the following equation:

$$p = A p_u \tanh \left( \frac{k z}{A p_u} y \right) \quad (2)$$

with $A = 3.0 - 0.8 z / D \geq 0.9$ for static loading and $A = 0.9$ for cyclic loading.

Here $p$ is the soil resistance per unit length of the pile and $y$ is the horizontal deflection. The parameter $k$ describes the initial modulus of subgrade reaction and is dependent on the relative density $I_D$ and with that on the angle of internal friction (Fig. 1 right).

![Fig. 1. Coefficients $c_1$, $c_2$, $c_3$ and initial modulus $k$ given in API (2000).](image)

The equations (1) and (2) are mainly based on investigations of Reese and Cox (Reese et al. 1974). They tested a 21 m long steel tube pile having a diameter of 61 cm under different loads and then evaluated their results. For cyclic tests, a maximum number of 200 load cycles was realized. The correction factor $A$ according to equation (2) was adjusted based on the measurements done.

The application of this method worked satisfactorily in offshore practice over many years, whereby the collected experiences only refer to piles with diameters up to about 2 m. According to Wiemann et al. (2004) the subgrade modulus for piles of large diameter is overestimated with the API method. They suggested a diameter-dependent correction factor of the initial subgrade modulus $k$. Also the authors of the paper in hand showed that the deflections of large-diameter piles under static loading are underestimated by the API method (Achmus et al., 2007).

Regarding the accumulation of monopile displacements due to cyclic loading to be expected over the lifetime of the foundation structure, the p-y method is not suitable, since the number of load cycles is not taken into account. By the method proposed by Little & Briaud (1988) the number of load cycles is taken into account. However, only
one parameter governs the displacement accumulation, and it is more or less unknown how this parameter is affected by soil, geometry and loading conditions.

In the following, the results of numerical calculations of the load-deformation behaviour of monopiles are presented. First, the behaviour under static loads is considered. Subsequently, a recently developed numerical method to estimate cyclic displacement accumulation of horizontally loaded piles is presented.

3. INVESTIGATION OF THE MONOPILE BEHAVIOUR UNDER STATIC LOADING

A three-dimensional (3D) finite element model was established in order to analyse the behaviour of monopiles embedded in sand soil. The computations were carried out using the finite element program system ABAQUS (Abaqus 2006).

The most important issue in geotechnical numerical modelling is the simulation of the soil stress-strain-behaviour. An elasto-plastic material law with Mohr-Coulomb failure criterion was used. The soil stiffness is herein represented by a stiffness modulus for oedometric compression $E_S$ and a Poisson’s ratio $\nu$. To account for the non-linear soil behaviour, a stress dependency of the stiffness modulus was implemented as follows:

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

(3)

Herein $\sigma_{at}=100 \text{kN/m}^2$ is a reference (atmospheric) stress and $\sigma$ is the current mean principal stress in the considered 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. The material parameters used here are given in Table 1. For more details about the numerical modelling reference is made to Abdel-Rahman & Achmus (2005).

<table>
<thead>
<tr>
<th>Table 1. Material parameters used for dense sand.</th>
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<tr>
<td>Unit buoyant weight $\gamma'$</td>
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<tr>
<td>Oedometric stiffness parameter $\kappa$</td>
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<tr>
<td>Oedometric stiffness parameter $\lambda$</td>
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<tr>
<td>Poisson’s ratio $\nu$</td>
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<td>Internal friction angle $\varphi'$</td>
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<td>Dilation angle $\psi$</td>
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<td>Cohesion $c'$</td>
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The stress-dependency of the stiffness modulus given by Equation (3) is often used in soil mechanics. However, no direct experience exists on the magnitude of the two parameters $(\kappa, \lambda)$ to be used in the calculation of horizontally loaded piles. In order to calibrate these parameters in connection with the numerical model, firstly monopiles of smaller diameters were investigated. For such diameters the p-y method is known to give a suitable estimation of pile deflection. Thus the numerical results could be compared with the results of the API p-y method for calibration. The calculations with the API method were carried out by means of the Lpile program (Lpile 2000).

The calculated deformations of a monopile with a diameter $D=2.0$ m with different embedded lengths varying from 20.0 to 40.0 m with a wall thickness of 3.0 cm under monotonic loading are shown in Fig. 2 and compared with the p-y method results. The loading consisted of a horizontal force acting at a height $h$ above the soil surface. The magnitude of the load was varied between 0.5 and 4.5 MN. The results are valid for $\kappa=800$ and $\lambda=0.55$. This parameter combination was found to give the best matching results with respect to the p-y method. This was also verified with similar comparisons for a pile of a diameter $D=1$ m and embedded lengths of 20 to 40 m. Thus, for a homogeneous sand soil with an angle of internal friction of 37.5° dealt with here, this...
parameter combination might be considered as realistic.

Numerical simulations with the calibrated parameters were carried out for large diameter monopiles and the same soil conditions, i.e. homogeneous sand with an angle of internal friction of 37.5°. Fig. 3 shows exemplarily the deflection lines of monopiles embedded 30 m in sand with two different diameters (D=5.0m and D=7.5m) under lateral load H with moment arm h=15 m. The moment arm indicates the height of the loading point above the soil surface, i.e. above seabed in case of offshore structures. The numerical results obtained using the calibrated soil parameters were compared with the standard p-y-method results. For both pile diameters, the results show relatively large deviations (Fig. 3). The p-y method underestimates the pile deflections significantly. This proves the statement given above that the API procedure is not suitable and should normally not be used in the serviceability design of monopiles of large diameters.

4. INVESTIGATION OF THE MONOPILE BEHAVIOUR UNDER CYCLIC LOADING

To investigate the lateral deformation response of a monopile under cyclic loading, a method was developed, which yields the permanent displacements of a pile-soil system by numerical calculations, taking the behaviour of soils under cyclic loading investigated in cyclic triaxial tests into account (Achmus et al., 2007, Kuo, 2008). This method bases on the finite element model presented above and accounts for cyclic loading by a special stiffness degradation approach.

A principle sketch of the results of a stress-controlled cyclic triaxial test under drained conditions is shown in Figure 4. The plastic portion of the axial strain \( \varepsilon_p^a \) increases with the number of load cycles. The increase rate of the plastic strain is mainly dependent on the initial stress state (confining stress) and on the magnitude of the cyclic load portion.
Fig. 4. Degradation of secant modulus under cyclic loading in a drained triaxial test.

The strain increase can be interpreted as a decrease of the secant stiffness modulus. When the elastic strain is negligible, the degradation of the secant modulus \( E_{sN} \) can be formulated in the following way dependent on the plastic strain in the first cycle \( \varepsilon_{p,N}^{a} \) and in the \( N^{th} \) cycle \( \varepsilon_{p,N}^{a} \):

\[
\frac{E_{sN}}{E_{s1}} = \frac{\varepsilon_{p,N}^{a}}{\varepsilon_{p,N}^{a}} = N^{-b_{1}(X)^p}
\]  

(4)

The degradation of secant stiffness in a triaxial cyclic test with isotropic initial stress condition can be determined from the plastic strains measured with a regression equation. Such equations were presented, for instance, from Huurm an (1996), Gotschol (2002) and Werkmeister (2004). Due to the approach of Huurm an used here, the increase of deformation or the decrease of stiffness, respectively, can be described by the following equation:

\[
\frac{E_{sN}}{E_{s1}} = \frac{\varepsilon_{p,N}^{a}}{\varepsilon_{p,N}^{a}} = N^{-b_{1}(X)^p}
\]  

(5)

Here \( N \) is the number of cycles, \( X \) is a stress-dependent variable (cyclic stress ratio), and \( b_{1}, b_{2} \) are regression parameters to be determined in triaxial tests.

The cyclic stress ratio is defined as

\[
X = \frac{\sigma_{1,cyclic}}{\sigma_{1,f}}
\]

(6)

wherein \( \sigma_{1,f} \) is the main principal stress at failure in a monotonic test. Thus, the stress ratio is dependent on the initial stress state (confining stress) and on the cyclic load level.

In the pile-soil model, the soil element is subjected to an anisotropic initial stress condition and the principal stress orientation rotates with changing minor principal stress. To overcome this problem, a modified cyclic stress ratio \( X \) is calculated as elucidated in Figure 5. The initial stress condition of each soil element is obtained from the finite element model under the vertical load in a first step. In a second step, the horizontal load is considered. For both the initial stress state (Index \( 0 \)) and the state under action of the (cyclic) horizontal load (Index \( 1 \)) a stress ratio \( X_{a} \) is calculated:

\[
X_{a}^{(0)} = \frac{\sigma_{1}^{(0)}}{\sigma_{1,f}^{(0)}}, \quad X_{a}^{(1)} = \frac{\sigma_{1}^{(1)}}{\sigma_{1,f}^{(1)}}
\]  

(7)

The modified cyclic stress ratio \( X \) is then calculated by

\[
X = \frac{X_{a}^{(1)} - X_{a}^{(0)}}{1 - X_{a}^{(0)}}
\]  

(8)

This parameter \( X \) characterizes the increase of the stress level in each element under cyclic load and can thus be used for the determination of the stiffness decrease with equation (5). Values \( X < 0 \), which may arise due to a decrease of deviatoric stress from the initial to the loaded state, are not taken into account, i.e. in such cases the soil stiffness remains unchanged. The degradation parameters \( b_{1} \) and \( b_{2} \) in Equation (5) have to be determined from a series of cyclic triaxial tests. If different soil layers exist, this can be easily accounted for by different sets of the parameters \( b_{1} \) and \( b_{2} \).

Figure 6 shows the lateral deflection of monopiles with different embedded lengths (L=20.0m and 40.0m) and with a wall thickness \( t_{p} \) of 9cm calculated using the stiffness degradation method (Kuo, 2008).

The long pile shows a better cyclic performance than the shorter one, which
could be explained by the different loading levels \((H/H_u)\). The ultimate horizontal loading \((H_u)\) was determined here using the hyperbolic method (Manoliu et al., 1984).

Figure 6. Lateral pile deflection obtained from degradation stiffness model (Kuo, 2008).

From Figure 7 it is evident that the accumulation rate for the pile with longer embedded length and with that a lower cyclic loading level is smaller than for the shorter pile with a higher cyclic loading ratio. This means that the accumulation rate is a function of the cyclic loading level \((H/H_u)\). This seems obvious, but with existing approaches an effect of the loading level is not reflected. The new degradation stiffness model is capable to take soil, geometry and loading conditions into account.

**On the rigid clamping criterion**

Up to now, in the design of horizontally loaded offshore piles “rigid clamping” was usually required. This means that for the loading under consideration, the pile deflection line should exhibit two zero deflection points or at least a vertical tangent at a certain depth below the point of rotation. This requirement stems from the idea that a pile clamped in the soil in this way will be relatively insensitive to cyclic loading. For offshore piles of usual diameters up to 2 or 2.5m, this requirement has proved to be reasonable. However, for very large diameters and thus very stiff monopiles this requirement leads to very large embedded lengths. As can be seen in Fig. 6, even the monopile with \(L=40m\) does not fulfil the rigid clamping criterion.

![Simulation with degradation stiffness model](image)

**Figure 7. Accumulated displacement of a pile under cyclic loading \((D=7.5m, h=20m, H=15MN)\) (Kuo, 2008).**

Figure 8 compares the lateral deflections for two different piles \((D=7.5m\) and \(2.5m\)) under the same loading conditions. The monopile with larger diameter exhibits a lower rate of accumulated lateral displacement than the pile with smaller diameter, but this pile does not demonstrate the rigid clamping behaviour. On the contrary, a monopile with a smaller diameter has a larger lateral displacement accumulation and thus a worse cyclic performance, but it exhibits a rigid clamping effect. Hence the suitability of using clamping length for a monopile design under long term cyclic loading needs to be reviewed.

**Loads with different amplitudes**
In the stiffness degradation approach, only one load combination is considered, i.e. a constant load amplitude. In fact, wind and wave loads acting on an OWEC of course vary.

Figure 8. Comparison of clamping effect in piles with different diameters (Kuo, 2008).

Regarding the application of the stiffness degradation in practice, the following approach can be used: Amplitude spectra of both moment and horizontal force at mudline should be provided. From these spectra a reference load (the maximum load) has to be defined and the equivalent number of cycles has to be determined. From the spectra distinct load classes (characterized by \( H_k \), \( h_k \) and respective \( N_k \) for each class) must be derived. Since the load classes are defined, the equivalent number of load cycles \( N_k^* \) of the reference load can be calculated by an approach proposed from Lin & Liao (1999):

\[
N_k^* = e^{\left(\frac{\ln 1 + \ln N_k}{\ln w_{1,k}}\right)}
\]

(9)

Herein \( w_{1,1} \) is the static displacement under reference load, \( w_{1,k} \) is the static displacement under the load of different amplitude and \( N_k \) is the number of cycles of this load. The parameter \( t \) is a degradation parameter, which is besides others a function of soil properties and loading type (one-way or two-way loading). For one-way loading \( t \) is a value of the order of 0.20 according to Lin & Liao (1999). The static displacements for a given load \( H \) and height of loading point \( h \) can be determined using the force-displacement curves obtained for static loading.

Summing over all loads of different amplitudes (load classes), the resulting equivalent number of load cycles of the reference load is

\[
N^* = N_1 + \sum_{k=2}^{n} N_k^*
\]

(10)

This method has exemplarily been applied for a monopile \( D = 7.5 \) m, \( L = 30 \) m embedded in dense sand under specific wave loading conditions. A wave spectrum derived by Mittendorf et al. (2004) for a 12-year time period at a specific North Sea location was taken and the loads were determined using the Morison equation. Seven load classes were defined. The loads, the number of load cycles and the resulting equivalent load cycles with respect to the maximum load are shown in Figure 9. The results indicate that over the considered 12-year period an accumulated deformation is to be expected, which is about 43% larger than the static displacement under the maximum load.

The results also indicate that the influence of low wave heights is neglectable, which seems rather plausible. Of course, this method is not yet experimentally verified and yields thus only an estimation of accumulated displacements. However, no other approved method to predict accumulated displacements with respect to specific loading and soil conditions yet exists.

Figure 9. Exemplary application of the method of Lin & Liao (1999) for a monopile \( D = 7.5 \) m, \( L = 30 \) m in dense sand (\( t = 0.17 \)).
CONCLUSIONS

For monopiles with very large diameters of 5m or more the p-y method according to API (2000) usually used in the design of horizontally loaded piles is not suitable. It has been shown that in sand soils the pile deflection under static design load is underestimated. Regarding accumulated displacements to be expected over the structure’s lifetime, the API approach also should not be used, since neither loading conditions nor the number of load cycles are accounted for.

A numerical model is presented, which enables a monopile design for static and also cyclic loads. For the latter, a new approach called stiffness degradation model is introduced. Herein, loading conditions as well as site-specific pile and soil conditions are taken into account. Although the model still needs verification by comparison with field measurements, it seems to be a very promising approach.

References

Little, R. L., Briaud, J-L. (1988). Full scale cyclic lateral load tests on six single piles in sand, Miscellaneous paper GL-88-27, Geotechnical Division, Texas A&M University, College Station, Texas.