Design of Axially and Laterally Loaded Piles for the Support of Offshore Wind Energy Converters

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ABSTRACT

A large number of offshore wind farms is being planned in the North Sea and the Baltic Sea in Europe and will be erected in the coming years. Possible foundation structures for water depths of up to 50m are jacket and tripod structures, i.e. structures with three or four mainly axially loaded piles, and for moderate water depths also monopiles, which are mainly horizontally loaded large diameter piles. A special aspect in design is the question how effects induced by cyclic loading of these foundation piles can be considered adequately. For cyclic axially loaded piles degradation of pile capacity might occur, and for cyclic horizontally loaded piles stability has to be proved and an increase of permanent deformation over the lifetime is to be expected. The paper in hand presents calculation approaches for the piles under axial and lateral loading and outlines possible design procedures with consideration of cyclic load effects.

1. INTRODUCTION

In the North Sea and the Baltic Sea in Europe a vast number of offshore wind farms are being planned and several have already been installed in recent years. Up to now, in most cases wind farms were erected in moderate water depths (less than 20m) and monopile foundations have been built as support structures for the wind tower and the turbine. A monopile consists of a single open steel pipe pile of large diameter which is driven into the seabed. Diameters of up to 5m have been realized recently. The tower is connected to the monopile by a transition piece located above the water level (Fig. 1, left). This type of foundation transfers the loads from wind and waves mainly by horizontal stresses into the ground and is believed to be suitable for water depths of up to 25m.

In the German parts of North Sea and Baltic Sea water depths of up to 50m exist. For such large water depths steel frame structures (jackets with four legs or tripods with three legs) can be used, which are supported by four or three piles located in the edges of the construction (Fig. 1, right). Regarding the lengths of these piles, the axial (compressive or tensile) loads induced by wind and waves are design-driving.

Design methods and experience with offshore piles exist mainly from structures built by the oil and gas industry. However, the loading conditions for offshore wind mill foundations are different. The vertical loads are much smaller than for oil or gas platforms, and thus the horizontal loads are of similar magnitude compared to the vertical loads. This means that the extremely cyclic nature of wind and wave forces is much more important than for very heavy structures. Due to that, consideration of cyclic load effects is extremely important.

Regarding monopiles, on one hand the question, whether usual calculation methods (p-y method) can be used for piles of very large diameter, has to be answered. On the other hand it has to be investigated how the system stability under cyclic loads can be proved and how accumulated deformations due to cyclic loading can be predicted. The latter is particularly important, since the requirements regarding the stiffness of such structures are very strict. A maximum rotation of the pile head of 0.5° is usually demanded.

Regarding axially loaded piles an important question is how the axial ultimate pile capacity can be predicted with sufficient accuracy. The  method commonly used in offshore design (e.g. API, 2000) is known to either over- or underestimate pile capacities, dependent on the boundary conditions. Recently, CPT-based methods have been developed as an alternative. Another open question is how
the degradation of skin friction to be expected due to cyclic axial loading can be accounted for in the design.

In the following, calculation approaches for the above mentioned problems are presented and critically assessed. Moreover, problems and possibilities regarding the consideration of cyclic load effects are presented.

Fig. 1: Schematic Sketches of a Monopile (left) and a Jacket Foundation (right)

2. DESIGN OF MONOPILES

For monopiles in sand soils the proof of serviceability under horizontal loading is usually design-driving. A large stiffness under operational load is demanded in order to ensure a natural frequency of the system which is higher than the main excitation frequency. Also, the deflections and rotations at mudline must be small to enable a sound operation of the wind turbine. Moreover, the allowable permanent rotation of the monopile system is rather small.

**p-y Method**

The usual design procedure for foundations of offshore wind energy converters in Germany is given in the Germanische Lloyd rules and regulations (GL, 2005). 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 Scandinavian 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. According 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 \( \varphi' \) of the sand:

  \[
  p_u = (c_1 z + c_2 D) \gamma' z \quad (1a)
  \]

  \[
  p_u = c_3 D \gamma' z \quad (1b)
  \]

- The first mentioned equation applies to small depths \( (p_u) \) and the second equation to larger depths \( (p_u) \), 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 \( c_3 \), which are given in API (2000) dependent on the angle of internal friction of the sand.

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

  \[
  p = A p_u \tanh\left( \frac{k z}{A p_u} \gamma \right) \quad (2)
  \]

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 \( k \) also given in API (2000) 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.

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.

In a similar manner, also p-y curve approaches for cohesive soils are given in API (2000) or in the literature. Here, the undrained shear strength and a strain value determined in a UU triaxial test are used as the central parameters describing the soil behaviour. Static and cyclic loading is also considered by different factors.

The application of these methods worked satisfactorily in offshore practice over many years, whereby the collected experiences only refer to piles with diameters up to about 2 or 2.5m. 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 author 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, Abdel-Rahman & Achmus, 2005). Recently, Soerensen et al. (2010) proposed an approach to decrease the p-y curve stiffnesses with respect to the pile diameter.

Regarding stability under cyclic loads and 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. As mentioned above, the cyclic load approach was found by execution of at maximum 200 – and in most cases much less – load cycles.

**Numerical Modeling**

A three-dimensional (3D) finite element model was established in order to analyze the behavior of monopiles. The computations were carried out using the finite element program system ABAQUS.

The most important issue in geotechnical numerical modeling is the simulation of the soil stress-strain-behaviour. In the case of monotonic loading, 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 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.

A typical finite element mesh is shown in Fig. 2. 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.

**Effect of Monopile Diameter**

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 (see also Achmus et al. 2008). For diameters of up to 2.5m 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 p-y method were carried out by means of the LPILE program.

Fig. 3 left shows the deflection lines for monopiles with different diameters in homogeneous dense sand, derived once by the p-y method and once by the numerical model. To ensure a similar pile behaviour, different pile lengths were examined, and typical service loads for the different pile geometries were applied.

The stiffness parameters of the numerical model were calibrated by comparison with the results of the p-y method for the pile with a diameter of 1.5m. Thus, the deflection lines of both methods are almost identical for this case. The results show that for larger diameters the p-y method underestimates the pile deflections. For a pile with a diameter of 4m, the deviation is 27% with respect to pile head deflection. For the pile with a diameter of 7.5m, the respective deviation is 38%.

Fig. 3 right compares the numerically obtained pile deflection lines for the \( D = 7.5 \)m pile to the results obtained by the above mentioned approaches of Wiemann et al. (2004) and Soerensen et al. (2010). The results obtained here are in good agreement with the Wiemann approach, whereas the approach of Soerensen et al. predicts an even larger diameter effect.
Effect of Cyclic Loading

It is known from different experimental investigations that the deflections of a horizontally loaded pile increase under cyclic loading. As an example, model test results of Hettler (1981) for flexible piles in homogeneous sand are shown in Fig. 4.

In general, the increase of head deflection can be described by the following equation:

\[ y_N = y_1 f_N(N) \]  

(4)

Here \( y_N \) and \( y_1 \) are the horizontal pile head deflections after \( N \) load cycles and after 1 load cycle (static loading), respectively. \( f_N(N) \) is a function which describes the increase of deflections. As long as the cyclic load amplitude is well below the ultimate pile capacity, sediment behaviour can be expected, which means that the deflection rate decreases with increasing number of load cycles. The most common functions of displacement of structures under cyclic loading that are found in literature are of the exponential type such as Equation 5 (e.g. Little & Briaud 1988) and of logarithmic type such as Equation 6 (e.g. Hettler 1981):

\[ f_N = N^m \]  

(5)

\[ f_N = 1 + t \times \ln N \]  

(6)

Here \( m \) and \( t \) are empirical degradation parameters. Assuming that these parameters \( (m \) and \( t \)) are constants, Equations 5 and 6 imply that the function of load cycle number is independent of the load amplitude. Peralta & Achmus (2010) found, based on model tests, that the exponential function of displacement increase with respect to number of load cycles better fits the cyclic displacement curves of almost rigid piles while the logarithmic function better fits the displacement curves of flexible piles.

Fig. 4: Model Test Results of Hettler (1981)

These empirical equations take the number of load cycles explicitly 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 common practice, substitute design requirements regarding the pile behavior under monotonic (static) extreme load are used. To limit the deformations due to cyclic loading, in many projects rigid clamping of the pile in the subsoil under static extreme loads is demanded. This means that the deflection line of the pile must have two zero deflection points, i.e. no or negative pile toe deflection (zero-toe-kick criterion), or it must at least have a vertical tangent (vertical-tangent criterion). The background of these requirements is the more or less intuitive idea that a pile which is clamped in the soil under extreme load would hardly be significantly loosened by cyclic load actions. However, for monopiles with very large diameters and thus large bending stiffnesses, in particular the zero-toe-kick criterion, but also the vertical-tangent criterion lead to very long embedded pile lengths. Thus, the suitability of these criteria has to be proved.

Stiffness Degradation Method

The stiffness degradation method (SDM) developed by the author and his co-workers is a method based on a combination of a finite element simulation of the pile-soil interaction and an evaluation of drained cyclic triaxial tests.

In cyclic triaxial tests, the accumulation of plastic strains with the number of cycles under different loading conditions can be observed. This increase of plastic strain can be interpreted as a decrease in soil secant stiffness. Assessing the stress conditions in the distinct elements and introducing the stiffness degradation obtained by comparison with the cyclic test results in the finite element model yields the accumulated deformations of the pile-soil system. This is the basic concept of this model.

The numerical model of a monopile foundation under monotonic lateral load presented above is used as a basis for cyclic analysis. The degradation stiffness approach to account for cyclic loading effects is elucidated in Fig. 5. In a cyclic triaxial test, an increase of the plastic axial strain can be observed. Assuming the elastic strain to be negligible, the degradation rate of secant stiffness after first cycle \( E_{s1} \) and \( N \)th cycle \( E_{sN} \) can be presented by the plastic axial strains after first cycle \( \varepsilon_{ep,N=1} \) and after \( N \)th cycle \( \varepsilon_{ep,N} \) according to the following equation:

\[ E_{sN} = E_{s1} \frac{\varepsilon_{ep,N=1}}{\varepsilon_{ep,N}} \]  

(7)

Fig. 5: Degradation of Secant Modulus Under Cyclic Loading in the Pile-soil Model (Schematic)
The accumulation of plastic strains in a cyclic triaxial test can be estimated from a semi-empirical approach of Huurman (1996). With that, the degradation of stiffness can be described using two material parameters \( b_1 \) and \( b_2 \) as follows:

\[
\frac{E_N}{E_s} = \frac{\varepsilon_{\sigma_p,N=1}^N}{\varepsilon_{\sigma_p,N}^{\text{st}}} = N^{-b_1(x)^{b_2}} \tag{8}
\]

Here \( N \) is the number of load cycles and \( X \) is the cyclic stress ratio defined by Huurman (1996) for cohesionless material as follows:

\[
X = \frac{\sigma_{1,cyc}}{\sigma_{1,\text{sf}}} \tag{9}
\]

where \( \sigma_{1,\text{sf}} \) is the major principal stress at static failure state and \( \sigma_{1,cyc} \) is the major principal stress for the cyclic stress state under consideration. The cyclic stress ratio is thus dependent on the confining pressure and on the cyclic stress level.

From cyclic triaxial test results documented in the literature, typical regression parameters \( b_1 \) and \( b_2 \) were found for dense sand to be \( b_1 = 0.20, b_2 = 5.76 \) and for medium dense sand \( b_1 = 0.16, b_2 = 0.38 \) (Kuo 2008).

A problem to be dealt with is that the Equations (7) and (8) are valid for triaxial test conditions with isotropic initial stress conditions and a constant confining pressure \( \sigma_3 \) during cyclic loading. In the pile-soil system, the initial stress conditions (before application of the horizontal load) are anisotropic and the minor principal stress in the elements as well as the direction of the principal stress axes in general change with the application of the load. To overcome this problem, a characteristic cyclic stress ratio \( X_c \) is defined here as

\[
X_c = \frac{X^{(i)} - X^{(0)}}{1 - X^{(0)}} \tag{10}
\]

Here the index \( ^{(i)} \) means the cyclic stress ratio at loading phase and the index \( ^{(0)} \) means at unloading phase (cf Fig. 6). At the initial (and unloading) phase, only the vertical load \( V \) due to the tower weight is considered, and the lateral load \( H \) is applied subsequently in the loading phase. The characteristic cyclic stress ratio is derived from the difference between the stress ratios in the loading and the unloading phase. Due to the denominator in Equation (10) this value varies from 0 to 1. The accumulation of plastic strain and the degradation of stiffness of the soil element can be obtained from Eq. (8) by replacing \( X \) by \( X_c \).

In the last step of the simulation (model C in Fig. 6), the deformation response of the system is analyzed using the degradation stiffnesses obtained from evaluation of models A and B. Poisson’s ratio is assumed to remain constant in the three discrete finite element models.
The stiffness degradation method was applied, using the typical parameters $b_1$ and $b_2$ for dense sand stated above. In Fig. 7 bottom the relative increases (with regard to the static deflection) of pile head displacement are shown. These curves can be interpreted as a measure of the cyclic performance of a pile. The longer flexible pile ($L=21m$) performs better than the shorter and almost rigid pile ($L=15m$). Thus, the function $f_\alpha(N)$ (cf. Equations 4 to 6) is obviously also dependent on the pile-soil system, i.e. the relative stiffness of the pile.

The calculated performance curves can be well described by the exponential equation type given in Equation (5). The parameter $m$ is $m=0.145$ for the shorter pile and $m=0.123$ for the longer pile. These values are plausible. Little & Briaud (1988) reported measured $m$-values for long and flexible piles between 0.04 and 0.09, whereas from results of Long & Vanneste (1994) $m$-values for rigid piles between 0.10 and 0.25 can be derived. For a driven pile in dense sand, the Long & Vanneste-approach for rigid piles between 0.10 and 0.25 can be derived. For a driven pile in dense sand, the Long & Vanneste-approach yields $m=0.136$.

Consideration of Variable Load Amplitudes

The empirical prediction methods and also the presented numerical method demand the definition of a cyclic swell (one-way) load with a corresponding number of load cycles. Actually, the horizontal loads to be considered for offshore foundations, one decisive question is the static ultimate pile capacity, and another is the amount of decrease of pile capacity with cyclic loading.

Since the required embedded depth of driven piles for offshore wind turbines depends in most cases on the tension loading case, only tensile pile capacities are considered in the following.

Static Tensile Pile Capacity

In general the tensile bearing capacity of piles consists of the pile’s weight and outer and inner skin friction. In the case of plugging, the latter is limited to the total weight of the soil plug inside the pile.

$$R_t = f_{ct} \cdot A_o + G' + Min[G'_p, f_{ct} \cdot A_i]$$

where $f_{ct}$ = outer unit skin friction for tension; $f_{ct}$ = inner unit skin friction for tension; $A_o$ = outer pile shaft area; $A_i$ = inner pile shaft area; $G'_t$ = effective steel weight of pile; and $G'_p$ = effective weight of soil plug inside the pile.

The common method of calculating tensile capacities of offshore piles in sand is the $\beta$-method given in the API RP 2A regulation (API 2000). Here, skin friction values are given dependent on the relative density of the soil and on the vertical effective overburden stress at the considered depth. In API (2000) skin friction values for tension and compression load are not distinguished. However, according to regulations of the Germanische Lloyd (GL 2005) usually applied for structures in the German North Sea, the skin friction under tension load should be assumed to be considerably smaller than the skin friction under compression load with $f_c = 2/3 f_c$. Taking this into account, the skin friction under tension load can be formulated as given in Equation (13).

$$f_{ti}(z) = \frac{2}{3} \beta \cdot \sigma_c(z) \leq f_{t,max}$$

where $\sigma_c$ = effective overburden stress; $f_{t,max}$ = limiting value of shaft friction to be taken from Table 1; and $\beta$ = shaft friction factor to be taken from Table 1.

This method was used in offshore engineering for a long time. However, it is now known to possibly either underestimate or overestimate the actual pile capacity.

Toolan et al. (1990) published the results of pile tests which showed an overestimation of pile capacity in the case of long piles in loose to medium dense sand. Lehane et al. (2005a) reported that the $\bar{a}$-method is conservative for relatively short offshore piles ($< 45m$ embedded length) in dense to very dense sands but may overestimate the pile capacity in all other conditions.
The use of cone penetration test (CPT) results potentially allows a more precise reflection of soil density, compressibility and stress level than the consideration of the subsoil only with regard to relative density in the \( \beta \)-method. In 2007 the API published an Errata and Supplement 3 to the guideline API RP 2A, including new “CPT-based methods” (API 2007). These approaches consider all the influencing factors given above and should thus allow a more accurate calculation of the pile capacity for a wide range of non-cohesive soils. However, offshore experience with the application of these CPT methods is still limited and therefore more experience is needed before they can be recommended for routine design, to replace the API \( \beta \)-method.

The CPT-based methods which were introduced in the API (2007) are simplified versions of the full versions published by different research groups (Jardine et al. 2005, Kolk and Baaijens 2005, Lehane et al. 2005b, Clausen et al. 2005). These simplified methods can yield slightly different results than the full versions of these methods, but for the case of offshore piles these differences are assumed to be small. The CPT-method results discussed in this paper were derived from the simplified versions given in API (2007) and are termed in the following ICP, FUGRO, UWA and NGI, respectively.

To determine skin friction according to the first three methods, the following general formula can be used (API 2007).

\[
 f_i = u \cdot q_{c,z} \left( \frac{\sigma'_{vo}}{p_a} \right) \left( A_i \right)^{\frac{c}{v}} \left[ \max \left( \frac{L-z}{D_i} ; v \right) \right]^{\frac{a}{v}} \tan \delta_{c,z} \quad (14)
\]

where \( q_{c,z} \) = CPT cone tip resistance at depth \( z \); \( \sigma'_{vo} \) = effective vertical in-situ soil stress; \( p_a \) = atmospheric pressure = 100 kPa; \( A_i \) = effective area ratio \( A_i = 1 - (D_i / D_o)^{c} \); \( D_o \) = pile outer diameter; \( D_i \) = pile inner diameter; \( L \) = embedded pile length; \( \delta_{c,z} \) = critical interface friction angle; \( a, b, c, d, u \) and \( v \) = empirical parameters to be taken from Table 2 (given for tension loading).

The fourth method for estimating the skin friction is the NGI approach (Eqs. 15 and 16).

\[
f_i = \frac{z}{L} \cdot p_a \cdot \sigma'_{vo} \cdot 0.25 \cdot \left[ \max \left( D_i ; 0.1 \right) \right]^{-0.1} \cdot \left( 1 - \frac{D_i}{D_o} \right)^{-0.7} \cdot \left( \frac{D_i}{D_o} \right)^{0.25} \cdot \left\{ \left[ \left( D_i ; 0.1 \right) - 0.1 \right] \right\}^{7} \quad (15)
\]

\[
D_r = 0.4 \cdot \ln \left( \frac{q_{c,z}}{22 \sqrt{\sigma'_{vo} \cdot p_a}} \right) \quad (16)
\]

where \( D_r \) = relative density of the soil.

The FUGRO and NGI approaches both apply a constant friction coefficient, whereas in the ICP and UWA approaches a dependence on the sand coarseness is considered, with a maximum value of \( \tan \delta_{c} = 0.55 \) for driven piles.

The CPT-based methods described above are all semi-empirical approaches, which were calibrated against a database of pile test results. Although the different databases had a large number of tests in common, they were in general different. Tests in very different soils and with different pile systems (open-ended and closed-ended steel piles, rectangular concrete piles) are included. Most of the tested piles had diameters smaller than 1.0m.

The subsoil in the German North Sea typically consists mainly of sandy soils, which are at least in a medium dense and often dense to very dense state. Intermediate cohesive layers occur, but normally with limited thicknesses. The piles to be used for tripod or jacket foundations are usually open-ended steel pipe piles with diameters between 1.5 and 3m and slenderness ratios (embedded length to diameter) between \( L/D = 10 \) and \( L/D = 40 \). Pile tests which are relevant to these conditions are very scarce in the databases.

All in all, only 6 test results included in the different databases, mainly stemming from the CLAROM, GOPAL, HOOGZAND and EURIP-DES test series, are relevant to open-ended steel pipe piles in sandy soil. For these tests the pile capacities were determined with the different approaches, and the results are shown in Fig. 8 in terms of ratio of calculated to measured pile capacity \( Q/Q_m \) over slenderness ratio \( L/D \). For application of the API \( \beta \)-method, the relative densities of the soil layers were derived from the CPT diagrams given using the method proposed by Jamiołkowski et al. (2001). Due to the limited number of tests, a final assessment of the approaches based on them is difficult. It can however be stated that the API \( \beta \)-approach largely underestimates the axial pile capacity for piles with slenderness ratios of

\[
\begin{array}{ccccccc}
\text{Method} & \text{a} & \text{b} & \text{c} & \text{d} & \text{u} & \text{v} \\
1 \text{(ICP)} & 0.1 & 0.2 & 0.1 & 0.4 & 1 & 0.016 & 4\sqrt{A_i} \\
2 \text{(UWA)} & 0.0 & 0.3 & 0.5 & 1 & 0.022 & 2 \\
3 \text{(FUGRO)} & 0.15 & 0.42 & 0.85 & 0 & 0.025 & 2\sqrt{A_i} \\
\end{array}
\]
less than 20. In order to compare the quality of the different approaches with respect to the test results used, the mean values and the standard deviations (coefficient of variation, COV) of the $Q_c/Q_m$-values were calculated, as shown in Table 3.

![Diagram](image)

**Fig. 8:** Comparison of Calculated and Measured Tensile Pile Capacities with Respect to Pile Slenderness Ratio $L/D$

**Table 3:** Mean Value and Standard Deviation of $Q_c/Q_m$

<table>
<thead>
<tr>
<th>Method</th>
<th>$Q_c/Q_m$ mean</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>API</td>
<td>0.60</td>
<td>0.29</td>
</tr>
<tr>
<td>ICP</td>
<td>0.88</td>
<td>0.15</td>
</tr>
<tr>
<td>UWA</td>
<td>0.82</td>
<td>0.14</td>
</tr>
<tr>
<td>FUGRO</td>
<td>1.03</td>
<td>0.28</td>
</tr>
<tr>
<td>NGI</td>
<td>1.15</td>
<td>0.21</td>
</tr>
</tbody>
</table>

The mean value of the FUGRO approach gives the best results. However, the relatively large standard deviation also shows a large scatter, i.e. large deviations with either overprediction or underprediction of pile capacity are possible. The NGI method tends to overpredict the axial pile capacity under tension loads for the conditions considered. The ICP and UWA approaches seem to be most suitable here. On average, a slight underprediction of pile capacity is to be expected. Moreover, due to the relatively small standard deviations, the scatter is limited and the prediction thus relatively certain.

However, none of the existing design methods can be classed as sufficiently reliable for the special conditions considered. In particular, no experimental evidence exists regarding the relatively large capacities predicted by the CPT-based methods for piles with large diameters of 2 or 3 m. A sensitivity analysis carried out in a parametric study by Achmus & Mueller (2010) clearly showed that the relative density of the sand subsoil and the absolute pile diameter strongly affect the results of the different methods. More experience and experimental pile tests are necessary to improve the accuracy of pile capacity prediction.

**Consideration of Cyclic Effects**

Under cyclic axial loads, a degradation of ultimate skin friction and with that a decrease of the pile capacity is to be expected. The main reason for that is a compaction of the soil beneath the pile shaft due to cyclic shearing, which leads to a reduction of the normal contact stresses acting between pile and soil. According to Randolph (2009), who evaluated cyclic interface tests on sand, the compaction occurs mainly within a small shear band beneath the pile. In a model proposed by Richter & Kirsch (2010), compaction occurs up to a distance at which a threshold value of cyclic shear stress in the soil is reached.

No pile capacity reduction is to be expected if a certain magnitude of the cyclic load portion is not exceeded. The threshold value is termed the critical level of repeated loading (CLRL):

$$CLRL = \frac{E_{cycl}}{R_k}$$  \hspace{1cm} (17)

Here $E_{cycl}$ is the cyclic load amplitude and $R_k$ is the static pile capacity.

From an evaluation of literature results, Schwarz (2002) reported the CLRL-values given in Table 3. The table shows clearly that piles in non-cohesive soils are more sensitive to cyclic axial loading than piles in cohesive soils.

**Table 4:** Critical Level of Repeated Loading According to Schwarz (2002)

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>CLRL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>0.1 – 0.4</td>
</tr>
<tr>
<td>Silt</td>
<td>0.4 – 0.6</td>
</tr>
<tr>
<td>Clay, normally consolidated</td>
<td>0.35 – 0.55</td>
</tr>
<tr>
<td>Clay, overconsolidated</td>
<td>0.85 – 1.0</td>
</tr>
</tbody>
</table>

Poulos (1988) presented a cyclic stability diagram for axially loaded piles (Fig. 9). Based on several model and field tests, the load cycle numbers leading to pile failure are given dependent on the normalized mean load $E_0$ and the cyclic load amplitude. A “stable” region is defined here with $E_{cycl}/R_k \leq 0.2$ (i.e. $CLRL = 0.2$) for $E_0 \leq 0.6 R_k$. From the diagram, a critical cyclic load amplitude (leading to failure) can be obtained dependent on the mean axial load and the number of load cycles. Mittag & Richter (2005) derived the following calculation approach based on the stability diagram of Poulos:

$$\frac{E_{cycl}}{R_k} \leq \kappa \cdot \left(1 - \left(\frac{E_0}{R_k}\right)^2\right)$$  \hspace{1cm} (18)

Here, the factor $\kappa$ accounts for the number of load cycles by $\kappa = 0.5 - 0.05 \log_{10} N$.

The stability curves corresponding to this approach are also shown in Fig. 9.

Kempfert (2009) evaluated pile test results in cohesive and non-cohesive soils separately and proposed the following approach to determine the critical cyclic load amplitude:
\[
\frac{E_{\text{cyc}}}{R_k} \leq \kappa^{*} \left( 1 - \left( \frac{E_0}{R_k} + 0.65 - \kappa^{*} \right)^4 \right)
\]  

Fig. 9: Stability Diagram of Poulos (1988) and Approach of Mittag & Richter (2005)

The factor \( \kappa^{*} \) is dependent on the number of load cycles and the type of soil and is given in Table 5.

Table 5: \( \kappa^{*} \)-Values Dependent on Number of Load Cycles and Soil Type According to Kempfert (2009)

<table>
<thead>
<tr>
<th>Number of Load Cycles N</th>
<th>10^4</th>
<th>10^5</th>
<th>10^6</th>
<th>10^7</th>
<th>10^8</th>
<th>10^9</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \kappa^{*} ) Non-cohesive</td>
<td>0.43</td>
<td>0.38</td>
<td>0.33</td>
<td>0.28</td>
<td>0.23</td>
<td>0.18</td>
</tr>
<tr>
<td>( \kappa^{*} ) Cohesive</td>
<td>0.48</td>
<td>0.43</td>
<td>0.38</td>
<td>0.33</td>
<td>0.28</td>
<td>0.23</td>
</tr>
</tbody>
</table>

Although some calculation approaches to deal with cyclic axial loading of piles exist, the respective knowledge is still rather limited. Another question with particular importance for piles of offshore wind energy foundations is how loads with different amplitudes can be considered. For a first approximation, an approach of Seed et al. (1975) developed for earthquake loading can be used to determine an equivalent substitute load with constant amplitude. However, urgent need for further research must be stated.

4. CONCLUSIONS

Several special questions have to be answered with regard to the design of piles for offshore wind energy foundations. On one hand, new foundation types are used like piles with very large diameters (monopiles), for which no experience exists. Also, in order to save costs for the large number of piles necessary for one wind farm, most accurate pile capacity predictions are demanded. On the other hand, effects of cyclic loading must be assessed and taken into account in the design. This is not a new question, but it is of particular importance here, since the static loads of offshore wind energy converters are small compared to usual offshore structures and thus cyclic loading due to wind and wave actions becomes much more important.

The paper in hand presents calculation approaches for monopiles. In particular, a new method to determine the accumulation of pile deflections with the number of load cycles is described. This method makes it possible to assess the cyclic performance of horizontally loaded piles, taking the pile geometry and the soil conditions (e.g. layered soil) into account.

Regarding mainly axially loaded piles for offshore wind energy foundations, different methods to determine the static tensile pile capacity are presented and compared and approaches to deal with the capacity degradation under cyclic axial load are discussed.

Summarizing the results, it must be stated that the knowledge regarding the behaviour of piles under either axial or lateral cyclic loads is limited. The existing calculation approaches are more or less of an approximate nature. Thus, there is an urgent need of research with regard to cyclically loaded piles.

REFERENCES


