A parametric study of different analytical design methods to determine the axial bearing capacity of monopiles

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HIGHLIGHTS

- A sensitivity analysis is carried out to show the influence of different parameters.
- The sensitivity analysis shows a variation between the capacities of 1.7 to 16.4.
- A comparison with five different field tests shows the performance of the methods.
- For some methods, there is good agreement between predicted and measured capacities.
- Other methods show a strong deviation to the measured capacity.

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ABSTRACT

In Europe many offshore wind power parks have already been realized and more are planned. Often monopiles are used as foundation for wind power plants. The determination of the axial pile capacity of monopiles is among the important issues for offshore energy projects. Despite progress made so far in this field, there are still big challenges faced by researchers. Among them is to provide guidance to assist engineers in the selection of an appropriate design method for the soil condition and pile configuration of interest. First, different state of the art methods (API, Fugro-05, ICP-05, NGI-05 and UWA-05) together with two new methods (HKU-12 and a German approach) to determine the axial bearing capacity based on the cone resistance of the soil are presented. Afterwards there is a summary of different approaches to determine the average cone resistance value. Furthermore a sensitivity analysis is carried out to show the influence of the pile diameter and the cone resistance on the predicted axial bearing capacity of the different design methods. In the last part, published field tests are used to compare the presented design methods and to identify their potential for reliable predictions for engineering practice. In addition to existing publications, these comparisons contain the two new design methods to determine the axial bearing capacity.

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1. Introduction

Monopiles (open steel pipe piles in general) are becoming increasingly important in constructing offshore structures, especially as foundation for offshore wind power plants. In order to replace fossil and nuclear energy with renewable energy sources, more and more wind power parks are being constructed. The determination of the axial pile capacity of monopiles is one of the important issues of offshore energy projects. Despite the progress made so far in this field, researchers are still faced with many challenges. Among them is the provision of guidance to assist engineers in the selection of an appropriate design method based on the soil condition and pile configuration of interest. Hence, this paper discusses an important issue related to geomechanics in energy and environment.

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The axial bearing capacity of open steel pipe piles in granular soils depends, among others, on the installation method, the pile diameter, the embedment length of the pile, the interface friction angle between the pile and the soil, and the relative density of the soil. During the installation, a plug may be formed inside the pile. A plug can be visualized as a spatial bracing of soil between the inner surface of the pile, cf. Lüking. Such a plug influences the axial bearing capacity significantly.

In Fig. 1, the principle axial bearing behaviour of an open steel pipe pile is shown. In the case of a pile with no plugging effect, the axial bearing capacity is achieved by an internal and external frictional resistance and a resistance mobilized by the tip pressure of the annulus (a). In the case of a partially plugged pile (b), there is an additional resistance mobilized by the plug inside the pile, though the internal frictional resistance is lowered compared to the pile without a plug. In the case of a fully plugged pile (c), an open pile behaves like a closed pile, with a high resistance mobilized by the plug and no internal frictional resistance. Observations by Jardine et al. show, that an open steel pipe pile is likely to form a plug when its diameter is 1.5 m and below. This means that in the case of large diameter open steel pipe piles, no plugging effect is expected and the bearing behaviour presented in Fig. 1(a) can be anticipated.

A parameter to define the degree of plugging is the Incremental Filling Ratio (IFR) introduced by Brucy et al. It is defined as follows

\[
IFR = \frac{\Delta h}{\Delta L}.
\]  

In real applications, for example in the installation of monopiles, it is hard to determine the IFR because the height of the plug as well as the penetration depth of the pile need to be continuously monitored. For this reason Paik et al. introduced the Plug Length Ratio (PLR). Its definition is similar to the IFR except that the PLR is not an incremental value and is only measured once at the end of the installation process.

\[
PLR = \frac{h}{L}.
\]

In scientific papers, different analytical approaches exist to determine the axial bearing capacity, e.g. Randolph et al. and Paik et al. Nowadays the API and the DIN EN ISO 19902:2014-01 operate as technical standards for the design of monopiles. Both are referring to CPT based design methods. These CPT based methods should lead to a more realistic and economical axial design of the piles because the axial bearing capacity is directly linked to the cone resistance of a CPT. The referred methods are the Fugro-05, ICP-05, NGI-05 and UWA-05. More recent methods to determine the axial bearing capacity are the HKU-12 method and an approach recommended by EA-Pfähle. What all these methods have in common is that they are only valid for impact-driven piles. Lammertz proposed a method to determine the axial bearing capacity of vibratory-driven piles.

The objective of this paper is to perform a comparison among the different analytical axial design methods for monopiles. A sensitivity analysis with regard to the pile diameter and the cone resistance shows the quality of the methods in a parametric study. A comparison with the results from different field tests shows the quality of the methods under real conditions. The result of this paper could serve as a guide to help the engineer select a method that suits his individual case.

2. Analytical design methods

All the respectively predicted capacities and resistances are ultimate values assuming a settlement of the pile of 10% of the diameter, i.e. \( s_D = 0.1 \). To calculate the total axial bearing capacity, \( Q_{\text{tot}} \), of open steel pipe piles, the following general equation is used for almost all of the presented methods

\[
Q_{\text{tot}} = Q_b + Q_s = 0.25 \pi D^2 q_b + \pi D \int \tau_s(z) \, dz. 
\]

Fig. 2(a) and (b) can be used to determine the qualitative distribution of \( Q_b \) and \( Q_s \) before the relative settlement of the pile reached 0.1D. A summary of the design equations to determine \( q_b \) and \( \tau_s \) of the CPT based design methods can be found in Tables 3 and 4.

2.1. API

The API 2A-WSD is a technical standard that provides guidance in the design of offshore structures. Among other things, a recommendation on the determination of the axial bearing capacity is given. This recommendation can also be found in DIN EN ISO 19902:2014-01. The values of
Table 1
Recommended values according to API\(^7\) respectively DIN EN ISO 19902:2014-01.\(^8\)

<table>
<thead>
<tr>
<th>Relative density</th>
<th>(D_r) [%]</th>
<th>Soil classification</th>
<th>(N_q)</th>
<th>(q_{\text{max}}) [MPa]</th>
<th>(\beta)</th>
<th>(\tau_{\text{max}}) [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose</td>
<td>0–15</td>
<td>Sand</td>
<td>Not applicable</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>15–35</td>
<td>Sand</td>
<td>CPT based methods recommended</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Medium dense</td>
<td>35–65</td>
<td>Silt</td>
<td>12</td>
<td>3</td>
<td>0.29</td>
<td>67</td>
</tr>
<tr>
<td>Dense</td>
<td>65–85</td>
<td>Silt</td>
<td>20</td>
<td>5</td>
<td>0.37</td>
<td>81</td>
</tr>
<tr>
<td>Medium dense</td>
<td>35–65</td>
<td>Sand–silt</td>
<td>40</td>
<td>10</td>
<td>0.46</td>
<td>96</td>
</tr>
<tr>
<td>Dense</td>
<td>65–85</td>
<td>Sand–silt</td>
<td>50</td>
<td>12</td>
<td>0.56</td>
<td>115</td>
</tr>
<tr>
<td>Very dense</td>
<td>85–100</td>
<td>Sand–silt</td>
<td>50</td>
<td>12</td>
<td>0.56</td>
<td>115</td>
</tr>
</tbody>
</table>

\(q_b\) and \(\tau_s\) can be determined by the following equations

\[
q_b = N_q \sigma_{v0}'(z) \leq q_{\text{max}} \tag{4}
\]

\[
\tau_s(z) = \beta \sigma_{v0}'(z) \leq \tau_{\text{max}}. \tag{5}
\]

The values can be determined according to Table 1.

\(q_b\) and \(\tau_s\) are limited due to the fact that the end bearing and shaft friction do not linearly increase with depth and overburden pressure, especially for long piles.

2.2. ICP-05

The ICP-05 method is a CPT based method proposed by Jardine et al.\(^2\) and it intends to predict the axial pile capacity which may be mobilized in slow maintained load tests on previously unfailed piles conducted 10 days after installation. Jardine et al.\(^16\) however, stated that the axial pile capacity predicted by the ICP-05 method is more or less the measured capacity after 100 days of installation.

This method states equations to determine \(q_b\) and \(\tau_s\). It is based on field tests of closed and open ended piles with pile diameters of up to 0.76 m driven into sand and clay. The method distinguishes between a fully plugged and unplugged pile. In the case of an unplugged pile the unit end bearing resistance is directly linked to the average cone resistance and the internal shaft friction is neglected. The external unit shaft friction is determined by using the Coulomb failure criterion.

The radial effective stress after installation and equalization, \(\sigma_{r}'\), is based on the correlation between the local cone resistance, the normalized effective vertical stress and the normalized distance above the tip of the pile. The change in radial stress due to dilation, \(\sigma_{rd}'\), is based on a cylindrical cavity expansion analogy.

2.3. UWA-05

The UWA-05 method is a CPT based method proposed by Lehane et al.\(^11\) which is based on findings by Xu et al.\(^17\) Lehane et al.\(^18\) published an updated version of the UWA-05 method with an extrapolation to large diameters. The UWA-05 method states equations for \(q_b\) and \(\tau_s\). It also divides the unit end bearing resistance into the bearing resistance of the annulus, \(q_{b,\text{ann}}\), and the plug, \(q_{b,\text{plug}}\). The UWA-05 method is based on a database of 13 full-scale static load tests done by driving open ended piles in siliceous sand at seven different sites. For each site, the cone resistance profiles of a CPT are available. The diameter of most piles varied from 0.324 to 0.914 m. One pile had a diameter of 2.0 m. The embedment lengths of the piles varied from 5.3 to 79.1 m. The IFR was partially measured during the installation of the piles. If the IFR could not be measured, the PLR was measured at the end of the installation process.
2.4. **HKU-12**

The HKU-12 method is a CPT based method proposed by Yu and Yang.\(^2\)\(^1\)\(^2\)\(^1\) This method uses the same approach as the UWA-05 method, dividing the unit end bearing resistance into a resistance of the annulus and the plug inside the pile.

The proposed equation for the resistance of the annulus is based on centrifuge tests performed by De Nicola and Randolph.\(^9\) The centrifuge study investigates the effect of plugging inside open steel pipe piles driven into sand. The piles had prototype embedment lengths of 15–18 m, diameter of 1.6 m and wall thickness of 0.055 m. The tests were performed in loose, medium dense and very dense sand. With these results, Yu and Yang derived an equation which is directly dependent on the slenderness of the pile. There is no direct dependency on the relative density of the soil what is reasoned with the direct correlation with the cone resistance of the CPT.

The proposed equation for the resistance of the plug is based on tests by De Nicola and Randolph,\(^9\) Lehane and Gavin,\(^2\)\(^0\) and Lee et al.\(^2\)\(^1\) Lehane and Gavin performed small scale tests with open and closed ended steel pipe piles jacked in loose sand. The piles had diameters of 40 mm and 114 mm respectively and their embedment lengths varied from 1 to 1.55 m. Lee et al. performed small scale tests with open steel pipe piles driven into loose, medium dense and very dense sand. The piles had a diameter of 42.7 mm and embedment lengths of 0.25–0.76 m. The derived equation is based on an investigation of the plug resistance dependence on the PLR.

To calculate the external shaft friction, the HKU-12 method assumes a relative depth in the soil equal to \(\xi L\), where \(0 \leq \xi \leq 1\) and \(\xi = 1\) describes the co-ordinate at the pile toe, whereas \(\xi = 0\) describes the co-ordinate at the ground level. Utilizing this approach, the formula of the shaft resistance \(Q_s\) of Eq. (3) has to be modified as follows

\[
Q_s = \pi DL \int_0^1 \tau_s (\xi) \, d\xi. \tag{6}
\]

2.5. **Fugro-05**

The Fugro-05 method, proposed by Kolk et al.,\(^9\) is based on a database of 45 pile load tests at ten different sites. The piles, open and closed ended, were all installed in siliceous sand with diameters varying from 0.356 to 0.763 m and pile slenderness, \(L/D\), varying from 5 to 100.

2.6. **NGI-05**

The NGI-05 method, proposed by Clausen et al.,\(^1\) is based on a database of 85 pile tests in sand at 35 different sites. The piles, open and closed ended, were driven into loose to very dense sand with embedment lengths of up to 90 m. Contrary to the other CPT based design methods, this method utilizes the relative density, \(D_r\), instead of the pile diameter to determine the unit end bearing resistance.

The unit end bearing capacity distinguishes between plugged and coring behaviour, although there is no specific criterion for distinction. The unit end bearing resistance is taken as the smallest of the coring and the plugged resistance.

2.7. **German approach—EA-Pfähle**

The German approach, proposed by Lüking and Becker,\(^2\)\(^2\)\(^2\) is based on the EA-Pfähle\(^1\)\(^4\) and EAU\(^2\)\(^3\) recommendations and is documented in Moormann and Kempfert.\(^2\)\(^4\) The method tries to capture the strengths of both technical recommendations and combines them into one method. Both EA-Pfähle (“Recommendations on Piling”), and EAU (“Recommendations on waterfront structures, harbours and waterways”) were elaborated by the German Geotechnical Society (DGGT) and are widely used in Germany and partly in other European countries.

The proposed method is applicable for open steel piles with various diameters. Basically, the method distinguishes among three cases:

1. Fully plugged with \(D \leq 0.5\) m.
2. Full coring with \(D \geq 1.5\) m.
3. Partially plugged with \(0.5\) m \(< D < 1.5\) m.
The determination of the internal shaft friction as well as friction. This reduction is justified by the uncertainties in the inner shaft friction is reduced to 50% of the outer shaft upper 20% of the embedded part of the pile. Furthermore, the additional resistance of the plug is dropped and the resistance of the annulus is directly linked to the cone frictional resistance of the outer shaft, the resistance of the CPT of the soil.

For case 1, the fully plugged pile, the equation of the total axial capacity is

\[ Q_{tot, case \ 1} = \eta_{b, plug} q_{b, plug} A_{plug} + q_{b, an} A_{an} \]

\[ + \sum_j \eta_{s} \tau_{s,j} A_{s,j} \]

(7)

where \( \eta_{b, plug} = 2.52 \ e^{-1.85D} \) and \( \eta_{s} = 1.53 \ e^{-1.85D} \) both with \( D \) in [m].

For case 2, the full coring pile, the equation of the total axial capacity is

\[ Q_{tot, case \ 2} = q_{b, an} A_{an} + \sum_j \eta_{s} \tau_{s,j} A_{s,j} \]

\[ + \sum_j \tau_{s,j} A_{s,j} \]

(8)

where the index, \( i \), is linked to the inner part of the pile.

For case 3, the partially plugged pile, the equation of the total axial capacity is

\[ Q_{tot, case \ 3} = \psi \ Q_{tot, case \ 1} + \chi \ Q_{tot, case \ 2} \]

(9)

where \( \psi \) and \( \chi \) are calculation factors which can be determined according to Fig. 4.

The missing values of Eqs. (7) and (8) can be determined according to Table 2. For each parameter there are two values. The lower bound describes the 10% quantile, whereas the upper bound describes the 50% quantile. Furthermore, the proposed method is only valid for a cone resistance of \( q_{c} \leq 25 \) MPa.

### 3. Determination of \( q_{c, \ avg} \)

The determination of \( q_{c, \ avg} \) plays a significant role in all the CPT based design methods. DIN EN ISO 19902:2014-01 suggests taking the average 1.5\( D \) above and below the pile tip

\[ q_{c, \ avg, 1.5D} = \frac{1}{3D} \int_{-1.5D}^{1.5D} q_{c}(z) \ dz \]

(10)

Another method is the Dutch method by Schmertmann. The Dutch method assumes a zone of 0.7\( D \) to 4.0\( D \) below and 8\( D \) above the pile tip. This method takes the value 1 as an average below the pile tip of all values of \( q_{c}(z) \) until \( \min q_{c}(z) \). Afterwards, the average of the value 1 and \( \min q_{c}(z) \) is taken as \( q_{c,1} \). The average above the pile tip \( q_{c,2} \) is taken as the average of all minima in the range of 8\( D \). The average considered is then obtained by the following equation

\[ q_{c, \ avg, \ Dutch} = \frac{q_{c,1} + q_{c,2}}{2} \]

(11)

A more recent approach was proposed by Yu and Yang within the HKU-12 method. Assuming \( h_{ij} \) is the depth of
### Table 3
Summary of the design equations for the unit end bearing resistance of the CPT based methods for open ended steel piles impact driven into siliceous sand.

<table>
<thead>
<tr>
<th>Method</th>
<th>Unit end bearing resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fugro-05</td>
<td>$q_b = 8.5\ p_u \ \left(\frac{q_u}{p_u}\right)^{0.5} A_1^{0.25}$</td>
</tr>
<tr>
<td>HKU-12</td>
<td>$q_b = q_{bu} + q_{plag} \cdot \left(1 - A_1\right)$</td>
</tr>
<tr>
<td></td>
<td>$q_{bu} = q_{c,\ avg} \cdot \min\ \left[1.063 - 0.045 \ \left(\frac{d}{D}\right), 0.46\right]$</td>
</tr>
<tr>
<td></td>
<td>$q_{plag} = 1.063 \ q_{c,\ avg} \ \exp\ (-1.933 \ \text{PLR})$</td>
</tr>
<tr>
<td>ICP-05</td>
<td>$q_b = q_{b,\ plag} \cdot \left[\frac{d}{D_{\text{CPT}}} \leq 0.30\right] \ \left(\frac{\sigma_{c,\ plag}}{\sigma_{c,\ avg}} \ \text{max}\ \left[0.5 - 0.25 \ \log\ \left(\frac{D}{D_{\text{CPT}}}\right), 0.15\right] A_1\right)$</td>
</tr>
<tr>
<td>NGI-05</td>
<td>$q_b = \min\ \left(q_{c,\ avg}, q_{b,\ plag}\right)$</td>
</tr>
<tr>
<td></td>
<td>$q_{plag} = 12 \ \frac{\pi}{2} \ q_{c,\ avg} \ \frac{1}{4} \ q_{c,\ avg} \ \frac{1}{2}$</td>
</tr>
<tr>
<td>UWA-05</td>
<td>$q_b = \left(0.15 + 0.45 \ A_1^2\right) \ q_{c,\ avg}$</td>
</tr>
<tr>
<td></td>
<td>$\text{Remark: } q_{c,\ avg} \ \text{has to be evaluated with } D^* = D A_1^{0.5}$</td>
</tr>
</tbody>
</table>

### Table 4
Summary of the design equations for the unit shaft friction of the CPT based methods for open ended steel piles impact driven into siliceous sand.

<table>
<thead>
<tr>
<th>Method</th>
<th>Unit shaft friction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fugro-05</td>
<td>$\tau_r(z) = 0.08 \ q_u(z) \ \left(\frac{\sigma_{c,\ plag}(z)}{p_u}\right)^{0.05} \ \left(\frac{L}{D}\right)^{-0.5} \ \text{for } \frac{L}{D} \geq 4$</td>
</tr>
<tr>
<td></td>
<td>$\tau_r(z) = 0.08 \ q_u(z) \ \left(\frac{\sigma_{c,\ plag}(z)}{p_u}\right)^{0.05} \ \left(\frac{L}{D}\right)^{4-0.5} \ \text{for } \frac{L}{D} &lt; 4$</td>
</tr>
<tr>
<td>HKU-12</td>
<td>$\tau_r(\xi) = \left(\sigma_{c,\ plag}(\xi) + \Delta \sigma_{plag}(\xi)\right) \ \tan\ \delta_v\</td>
</tr>
<tr>
<td></td>
<td>$\sigma_{c,\ plag}(\xi) = \rho_0^3 \ q_u(\xi) \ \min\ \left[0.03 \ \left(\frac{L}{D}\right)^{-0.5}, 0.021\right]$</td>
</tr>
<tr>
<td></td>
<td>$\Delta \sigma_{plag}(\xi) = 4 \ G(\xi) \ \frac{\rho}{2}$</td>
</tr>
<tr>
<td>ICP-05</td>
<td>$\tau_r(z) = \max\ \left[\frac{\pi}{2} \ p_u \ F_{\text{av}}, F_{\text{cool}}, F_{\eta}, F_{\text{tot}}\right]$</td>
</tr>
<tr>
<td></td>
<td>$F_{\text{av}} = 2.1 \ (D - 0.10)^{0.17}$</td>
</tr>
<tr>
<td></td>
<td>$F_{\text{cool}} = 1.3 \ \text{for compression}$</td>
</tr>
<tr>
<td></td>
<td>$F_{\eta} = 1.0 \ \text{for an open ended pile}$</td>
</tr>
<tr>
<td></td>
<td>$F_{\text{tot}} = 1.0 \ \text{for steel}$</td>
</tr>
<tr>
<td></td>
<td>$F_{\text{ug}} = \left(\frac{\eta(z)}{r}\right)^{-0.5}$</td>
</tr>
<tr>
<td>HKU-05</td>
<td>$\tau_r(z) = \left(\sigma_{c,\ plag}(z) + \Delta \sigma_{plag}(z)\right) \ \tan\ \delta_v\</td>
</tr>
<tr>
<td></td>
<td>$\sigma_{c,\ plag}(z) = 0.029 \ q_u(z) \ \left(\frac{\sigma_{c,\ plag}(z)}{p_u}\right)^{0.13} \ \left[\max\ \left(\frac{1}{4}, 8\right)\right]^{-0.38}$</td>
</tr>
<tr>
<td></td>
<td>$\Delta \sigma_{plag}(z) = 4 \ G(\xi) \ \frac{\rho}{2}$</td>
</tr>
</tbody>
</table>

### 4. Sensitivity analysis

To gain a first impression about the relative performances of the methods presented in Section 2, a sensitivity study was carried out. An open steel pipe pile driven into a sand layer is assumed. The pile has a wall thickness of 40 mm and an embedment length of 20 m. For the purpose of this study, $q_c$ and $D$ have been varied. Three different $q_c$ profiles are assumed: 7.5, 15 and 25 MPa. For the sake of simplicity and to preclude potential error sources, $q_c$, is distributed as a constant over the whole height of the sand layer. The external diameter is varied from 0.5 to 5 m, although most methods were verified against field tests with pile diameters ≤ 1.5 m. In Figs. 5–7, the total calculated axial capacity, $Q_{\text{tot}}$, with regard to the external pile diameter is shown.

With increasing $q_c$, the predicted capacity increases. Furthermore, with increasing $q_c$, the gradient of the curve of the NGI-05 method increases up to a diameter of 2 m. Afterwards, the gradient slightly decreases and then remains constant. Comparing the gradient of the curves of the other methods, it was noted that the gradient of the curves of the Fugro-05, HKU-12 and UWA-05 methods increases with the pile diameter.

From the sensitivity analysis, it can be seen that with increasing pile diameter, the predicted capacities vary by factors of $1.84$ ($q_c = 7.5$ MPa), $1.75$ ($q_c = 15$ MPa), and
Table 5

<table>
<thead>
<tr>
<th>Case</th>
<th>Soil condition</th>
<th>Range above pile tip</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
</tr>
<tr>
<td>$h_d &lt; 8D$</td>
<td>Extreme variation in $q_c$</td>
<td>$8D$</td>
</tr>
<tr>
<td>$h_d &lt; 8D$</td>
<td>Other situations</td>
<td>$1.5D$</td>
</tr>
<tr>
<td>$h_d \geq 8D$</td>
<td>Embedded in sand of</td>
<td>Low compressibility</td>
</tr>
<tr>
<td></td>
<td></td>
<td>High compressibility</td>
</tr>
</tbody>
</table>

Fig. 5. Total predicted axial bearing capacity with an assumed $q_c$ of 7.5 MPa.

Fig. 6. Total predicted axial bearing capacity with an assumed $q_c$ of 15 MPa.

1.70 ($q_c = 25$ MPa) for $D = 0.5$ m up to factors of 16.40, 12.56, and 13.10 for $D = 5.0$ m. For a $D$ of 1.5 m, the factors of variation are 6.40, 5.09, and 5.47. This strong variation with increasing diameter indicates already the limits of the different design methods. None of them were validated against pile load tests with a pile diameter larger than 2.0 m.

In comparison to offshore conditions, comparable low $q_c$ values were chosen for this sensitivity analysis. All of the presented methods in Section 2 were developed for different applications. Some of them were developed for application in the offshore environment, while some others were developed for other applications but are used in the offshore environment. To qualitatively compare the presented design methods, the range of $q_c$ values was chosen in a way that all methods could be used in their validated range of site conditions.

5. Comparison with published field test results

To show the performance of the proposed methods under real conditions, they were compared against the results of published field tests. These well-documented field tests, containing the measurement of the total axial capacity of a driven pile in non-cohesive soil, only investigated piles with diameters less than 2 m. Thus, the maximum investigated pile diameter used was 2 m. In order to perform this comparison, four pile load tests documented in Yang et al. were used. Instead of a simplification with a constant cone resistance profile as in Section 4, the real measured cone resistance profiles of the different sites were utilized (cf. Fig. 8). The four field tests are the following:
Table 6
Average $q_c$ at the pile tip (MPa).

<table>
<thead>
<tr>
<th>Location</th>
<th>$q_c$, avg, 1.5D</th>
<th>$q_c$, avg, Dutch</th>
<th>$q_c$, avg, HKU</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mobile Bay</td>
<td>10.0</td>
<td>9.7</td>
<td>5.6</td>
</tr>
<tr>
<td>EURIPIDES</td>
<td>49.0</td>
<td>50.7</td>
<td>52.0</td>
</tr>
<tr>
<td>Hound Point</td>
<td>21.5</td>
<td>16.8</td>
<td>21.2</td>
</tr>
<tr>
<td>Tokyo Bay</td>
<td>28.5</td>
<td>9.3</td>
<td>14.8</td>
</tr>
</tbody>
</table>

(a) Mobile Bay, USA: A pile with an external diameter of 0.324 m and an embedment length of 15.2 m. There was no set-up time before the pile load test documented. The soil consists of silty and fine sand. The $q_c$ profile linearly increases up to a maximum of 40 MPa, cf. Fig. 8(a).

(b) EURIPIDES, Netherlands: A pile with an external diameter of 0.763 m and an embedment length of 38.7 m. The set-up time before the pile load test was 2 days. The first 20 m of the soil consists of fine sand with partial clay inclusions and a soft clay layer, followed by the bearing layer. The bearing layer consists of medium to very dense silty sand. The $q_c$ profile linearly increases in the bearing layer up to a maximum of 60 MPa, cf. Fig. 8(b).

(c) Hound Point, Scotland: A pile with an external diameter of 1.22 m and an embedment length of 26 m. The set-up time before the pile load test was 21 days. The soil consists of a 17 m thick clay layer followed by a sandy gravel layer which acts as the bearing layer. The $q_c$ profile shows large variations, oscillating from 5 to 40 MPa, cf. Fig. 8(c).

(d) Tokyo Bay, Japan: A pile with an external diameter of 2.0 m and an embedment length of 30.6 m. The set-up time before the pile load test was 52 days. The soil consists of a 4 m thick loose alluvial sand layer followed by a dense sand containing thin layers of sandy clay. The $q_c$ profile oscillates between 5 and 60 MPa, cf. Fig. 8(d).

The average cone resistance at the pile tip calculated with the methods presented in Section 3 is shown in Table 6. To ensure a better comparability of the design methods, $q_c$, avg, 1.5D was utilized for the calculations presented in this section. The predicted capacities of the EA-Pfähle approach were calculated by applying the 50% value. Furthermore, the design values given in Table 2 were linearly extrapolated up to the cone resistance of the investigated sites, even though the design method is not validated against such conditions. The same was done with the other design methods; for example, the UWA-05 method was validated against one pile load test with a pile diameter of 2.0 m, whereas the ICP-05 method was only validated against a maximum pile diameter of 0.76 m.

In Fig. 9, the normalized predicted capacities are presented, i.e. the predicted capacity, $Q_{tot}$, is divided by the measured capacity, $Q_m$, at a settlement of 0.1D. In Fig. 10, the proportion of the end bearing, $Q_b$, to the total capacity, $Q_{tot}$, is shown. In Fig. 11, the proportion of the shaft resistance, $Q_s$, to the total capacity, $Q_{tot}$, is shown. From the figures, the following points can be extracted:

Fig. 8. Cone resistance profiles of (a) Mobile Bay. (b) EURIPIDES. (c) Hound Point and (d) Tokyo Bay according to Yang et al.27

Fig. 9. Predicted axial capacities in relation to the measured axial capacity at a settlement of 0.1 D.
(a) With increasing diameter, the range of total predicted axial capacities increases. For the Mobile Bay and the EURIPIDES site, the range is rather small in comparison to the Hound Point and Tokyo Bay site. The only exception is the result of the EA-Pfähle approach at the Mobile Bay site. This observation is in accordance with the findings in Section 4.

(b) The division of the total resistance into end bearing resistance and shaft friction shows a significant variation among the methods. As in (a), a dependency on the diameter visible. For the Mobile Bay and EURIPIDES sites, the ratio $Q_b/Q_{tot}$ varies from 0.25 to 0.85 and 0.12 to 0.75 respectively. At the Hound Point site, most of the methods predicted a ratio of 0.75. The exceptions are the ICP-05 method and the EA-Pfähle approach with a ratio of 0.4. The Tokyo Bay site shows a similar result. Most methods show a ratio of 0.5, while the ICP-05 predicts a ratio of 0.2 and the API a ratio of 0.7. Even though the scatter at the Mobile Bay and EURIPIDES sites is large, a trend of a decreasing portion of the end bearing resistance onto the total capacity with increasing diameter is visible. This is in accordance with the explanation given in Section 1 that with increasing pile diameter, the probability of a plug being formed decreases and thus more and more load is carried by the shaft resistance of the pile.

(c) It is evident that the predicted capacities have a certain scatter. The Mobile Bay field test gives the lowest scatter, whereas the Hound Point field test shows a bandwidth from 0.27 to 1.55. This may be because of the fact that the true axial bearing capacity of driven piles increases over time. This effect is the set-up effect, cf. Gavin et al. This set-up effect comes along with two potential error sources. As a first source, the time of the pile load test can be identified. It will make a difference if this test is carried out immediately after, a few days after or even 100 or more days after pile installation. The second source is partially combined with the first source. All of the design methods are based on pile load tests of different field tests. This means that all of these methods contain an error relative to the set-up effect. Jardine et al. stated that the predicted capacity of the ICP-05 method is more or less the capacity of the piles after 100 days. This could be one reason for the high range among all of the different predicted capacities.

(d) By further taking into account the set-up effect, it should be noted that the lowest scatter of the predicted capacities was observed at the EURIPIDES site and they are all relatively close to the measured capacity. This is astonishing as the pile load test was carried out only two days after pile installation and thus, the axial pile capacity measured after 100 days should be much larger. It should be further noted that with increasing time of the pile load test after the pile installation, the predicted capacities show an increasing scatter.

Comparing the individual values of $Q_{tot}/Q_m$, shown in Table 7, and the mean values together with the standard deviation and COV, shown in Table 8, of the different design methods against each other, gives rise to the following noteworthy points:
(a) In the mean, the UWA-05, API and NGI-05 methods showed the best agreement with the measured pile capacities. The worst agreement was achieved with the EA-Pfähle approach.

(b) By taking into account the COV of the different design methods, the new HKU-12 method showed the lowest scatter followed by the UWA-05 and ICP-05 methods. The largest scatter was again achieved with the EA-Pfähle approach.

(c) As in Section 4, the Fugro-05 method tends to predict the highest capacities while the EA-Pfähle approach tends to predict the lowest capacities.

Schneider et al. and Gavin et al. conducted similar investigations. Among other methods, they compared the Fugro-05, ICP-05, NGI-05 and UWA-05 methods against each other and demonstrated their relative performance. As shown in Table 8, they also determined a mean value and COV. The results of Lehane et al. are in good agreement with the results obtained here. The values of the mean and COV are comparable. The results of Gavin et al. differ from the results in Table 8, especially regarding the COV where they obtained larger values, e.g. 0.41 for the NGI-05 method. This may be due to the fact that Gavin et al. were investigating the tension shaft resistance of open steel pipe piles to assess the relative performance of different design methods.

As indicated in Section 3, the determination of \( q_c, \text{avg} \) plays a significant role in the calculation of the axial capacity. Table 6 gives an overview and impression of possible differences. It can be seen that the values show differences of up to 50% for different sites, especially if the measured \( q_c \) profile oscillates strongly with a large scatter. The different methods to determine \( q_c, \text{avg} \) show different results. For the four different sites used in this comparison, \( q_c, \text{avg}, 1.5D \) often shows the highest value. Besides the Fugro-05 method, the end bearing resistance of all of the CPT based methods depends linearly on \( q_c, \text{avg} \). The end bearing resistance calculated with the Fugro-05 method has a dependency of \( (q_c, \text{avg})^{0.6} \). As indicated at the beginning of this section, \( q_c, \text{avg}, 1.5D \) was initially applied to all of the design methods. By taking the different values of \( q_c, \text{avg} \) into account when interpreting the calculated pile capacities, the following points are noted:

(a) At the Hound Point site, the calculated pile capacities would decrease by utilizing \( q_c, \text{avg}, \text{Dutch} \) or \( q_c, \text{avg}, \text{HKU} \) and thus would lead to worse results.

(b) At the EURIPIDES site, the influence on the calculated pile capacities would be rather small. The values of \( q_c, \text{avg} \) vary only in a range of 49–52 MPa. Furthermore, the ratio of \( Q_b / Q_{tot} \) of most design methods is less than 50%.

(c) At the Hound Point site, the influence on the calculated pile capacities could be high: On one hand, the variation between the different values of \( q_c, \text{avg} \) is not that high; on the other hand, the ratio of \( Q_b / Q_{tot} \) for most of the methods is 75%. Furthermore, there is a large scatter in the total predicted capacities, i.e. the Fugro-05, NGI-05 and UWA-05 methods would give better results when utilizing \( q_c, \text{avg}, \text{Dutch} \). Nevertheless, the ICP-05 and HKU-12 methods would predict worse results by utilizing a lower value of \( q_c, \text{avg} \).

(d) At the Tokyo Bay site, the Fugro-05 method would benefit from using a lower value of \( q_c, \text{avg} \). However, the other CPT based design methods show good agreement by the utilization of \( q_c, \text{avg}, 1.5D \).

6. Conclusion and outlook

This paper presents a study of the state of the art methods used to determine the axial bearing capacity of open steel pipe piles. In addition to existing publications comparing axial pile design methods against field tests, e.g. Schneider et al. and Gavin et al. this study contains two new design methods: the HKU-12 method and the EA-Pfähle approach. In the sensitivity analysis, the dependency of the predicted bearing capacity on the cone resistance \( q_c \) and the diameter of the pile was shown. The Fugro-05, HKU-12 and UWA-05 methods demonstrated a higher dependency on the pile diameter than the other design methods. For these three methods, the predicted bearing capacity increases in a nonlinear manner with increasing pile diameter.

The comparison with different field tests shows the quality of the design methods under field conditions. The CPT based methods showed that they can give a realistic prediction of the real bearing capacity. The new HKU-12 method especially, predicts results that are in good agreement to the measured pile resistance and a low COV. Moreover, the UWA-05 and HKU-12 methods showed very good overall agreement. The German approach, EA-Pfähle, showed a less suitable overall agreement with the worst mean and COV values.

Furthermore, the dependency of \( q_c, \text{avg} \) on the total axial bearing capacity could be demonstrated. It was observed that the determined value of \( q_c, \text{avg} \) can vary a lot for different \( q_c \) profiles. Also, it was shown that fairly good agreement can be achieved when utilizing \( q_c, \text{avg}, 1.5D \).

This paper has shown, under the study conditions assumed, that the difference range of the predicted results is large. While one design method can predict just 25%, another method predicts almost 150% of the real capacity. The range of the predicted capacities increases with the pile diameter. This indicates that the design methods which we presented work less efficiently, if they are used outside their calibrated environment. To choose an appropriate design method for a specific project, care should be taken to ensure that the method has already been validated against projects with comparable site conditions. However, especially in the case of monopiles, there is a lack of published high quality field tests and the range of predicted capacities may be high. To overcome the lack of documented field tests dealing with the bearing capacity of monopiles, numerical simulations in combination with the experience from design practice of comparable projects can be utilized.

To improve the current design methods for open steel pipe piles with a focus on larger diameters, the lack of published field tests for large diameter piles has to be overcome. This can be done by partially replacing the field tests with calibrated and validated numerical simulations. In future research, a focus will be put on the numerical simulation of the installation process of open steel pipe
piles with regard to different installation methods. With these simulations, comprehensive model- and field tests and the experience from design practice of comparable projects, the axial and lateral bearing behaviour can be investigated and monopiles can be designed in a more economical way.

References


