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RELIABILITY BASED EVALUATION OF OFFSHORE DESIGN APPROACHES FOR COMPRESSIVE PILES IN NON-COHESIVE SOIL

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The design of offshore foundation piles regarding the compressive bearing capacity is normally done by applying the well known API "Main Text" method and CPT-based design methods, like the ICP-, UWA-, Fugro- and NGI-method. However, in practice a strong deviation regarding the required embedded pile length within these design methods has to be faced. In this study 50 deterministic pile designs were compared with reliability based designs regarding the corresponding failure probability. Based on a new calibration approach, where the most likely failure probability is determined, quality factors were obtained for each design method. With the so obtained quality factors an enhanced evaluation of each deterministic design method as well as the safety level in terms of the required partial safety factor is possible. Altogether, it can be stated that the UWA-method is the most robust method for the considered boundary conditions in comparison to the other design methods.

Keywords: Quality factors, Partial safety factors, RBD calibration, Offshore pile design.

1 Introduction

In the near future several offshore wind farms are planned to be built in the North Sea. A number of projects will be located in sea areas with relatively high water depths (exceeding 40 m). For such water depths, jacket and tripod support structures with mainly axially loaded foundation piles will probably be employed in most cases.

The axial bearing resistance of such piles is normally estimated acc. to API (2014) by applying the so termed "Main Text" method, in this paper also referred to as API-Method. However, several investigations have shown that the application of the API-method is not reliable and may lead to a significant deviation compared to the real in situ bearing capacity. To enhance the reliability of design methods, 4 additional CPT-based design methods, namely ICP, UWA, Fugro and NGI, were recommended within API (2014). These methods were calibrated on pile field tests, where basically the skin friction of a pile and the end bearing are estimated on basis of the cone resistance measured in a CPT. However, the application of the CPT-based methods leads in practice to a high deviation in the required pile length within a design.

In this study different pile-soil systems were designed deterministically and investigated regarding the failure probability with reliability based approaches. By applying a new calibration approach, quality factors for each design method were derived. Based on these quality factors, an enhanced evaluation of a certain design method in question regarding the reliability of the predicted bearing capacity can be done. Moreover a more reliable as well as more robust deterministic design can be achieved by taking the proposed quality factors into account.

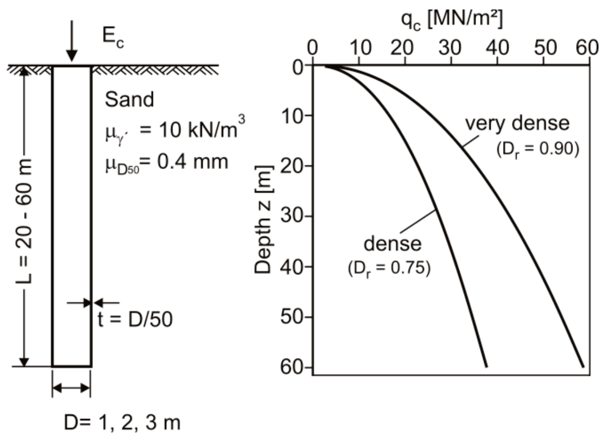

Figure 1. Pile-soil systems under consideration.

2 Pile-Soil system under consideration

For the performed study typical site conditions for the North Sea and typical ranges for the foundation pile properties were assumed. Two idealized CPT profiles for a homogeneous dense (D_r = 0.75) and a very dense (D_r = 0.90) sand were considered, respectively. The mean value of the grain size at 50 % of the grain size distribution was chosen to μ_{D50} = 0.4 mm. The mean value of the effective unit weight was assumed to be $\mu_{\gamma'}$ = 10 kN/m³. The choice of pile properties depends on the type of support structure for the wind turbine, the water depth and the subsoil condition at the desired location. In general it can be said that pile slenderness ratios (embedded length to diameter) between L/D = 15 and L/D = 40 are used for jacket support structures of offshore wind turbines. Thereby the pile outer diameter is varying between D = 1 - 3 m, where the pile embedded length is commonly chosen to be between L = 20 - 60 m. The regular pile wall thickness which is mostly used can approximately be determined by t = D/50. Figure 1 shows the considered pile properties and soil conditions.

3 Design methods

The compression bearing capacity of an axially loaded pile consists basically of the tip bearing resistance and the mobilized friction between the pile outer shaft area and the surrounding subsoil. In case of an open-ended pile two different conditions of the soil within a pile, namely “plugged” or “unplugged”, must be considered. In the unplugged case resistance due to friction between the pile outer shaft area and the inner soil as well as the tip resistance on the pile annulus is assumed. In contrast to that, for the plugged case only the resistance between the outer shaft area and the tip resistance on the entire circular area is taken into account. The calculation of the compressive bearing resistance for all design methods was done according to API (2014). Only for the API-method the inner soil friction was reduced to 0.8, since this reduction is applied in practice due to the use of a driving shoe.

4 Deterministic design

According to Simpson (2012) characteristic values are chosen 0.5 times the standard deviation below the mean value or above, if this is unfavorable, on the safe side. Taking into account an overall COV of 10 % for the unit weight and the grain size at 50 % of the grain size distribution a characteristic value of γ’ = 9.5 kN/m³ and D_{50} = 0.42 mm were chosen for the deterministic design, respectively. Therewith 30 % quantile values were chosen as characteristic design parameter. In contrast the q_c-profiles according to Fig. 1 were considered without reduction in the deterministic design. The internal friction angle was determined by the approach proposed by Kulhawy & Mayne (1990). Characteristic compression loads of E_c = 10, 20, 30 and 40 MN were considered for the pile foundation. The characteristic load corresponds to the 95 % quantile value, which is acc. to Holicky et al. (2007) representative for a 50 year extreme event.

In contrast to onshore pile design, where the load is distinguished in dead and live load, special design load cases are considered for offshore structures. Within these design load cases a combined load is determined for a certain event. Hence, only one harmonized partial safety factor has to be applied for the load. According to the German standard DIN EN 61400-3 (2010) a partial safety factor of γ_E = 1.35 has to be applied for the load, which considers the ultimate limit state proof of the pile foundation. According to DIN 1054 (2010) a partial safety factor of γ_R = 1.40 should be applied for the resistance of a compression pile in case no pile capacity load test has been executed. Thus, a global safety factor of GSF = 1.89 is prescribed by application of the appropriate German standards. On basis of the assumed soil conditions, the considered embedded pile length range and slenderness ratio range in section 2 as well as the chosen characteristic values for the resistance and load parameters and the prescribed GSF above, 5 combinations of a suitable pile diameter and characteristic loading were investigated, where in total 50 deterministic design cases were studied.

Figure 2 (left) depicts the development of the GSF with the embedded pile length for D = 2 m, D_r = 0.75 and E_c = 20 MN. For the prescribed GSF of 1.89 a strong deviation in the required embedded pile length can be noticed. This deviation varies from ΔL = 27 - 41 m for D_r = 0.75 and ΔL = 18 - 34 m for D_r = 0.90 for all investigated design combinations, respectively. Hence, the estimation of the required pile length may result in a difficult task for engineers in practice.

5 Stochastic subsoil model

The stochastic submodel consists of five input variables, namely the cone tip resistance q_c, the unit buoyant weight of the soil γ', the grain size at 50 % of the grain size distribution D_{50}, a transformation error for the estimation of the internal friction angle from a CPT $\varepsilon_{t,\varphi'}$ and the model error ε_R for the design method in question.

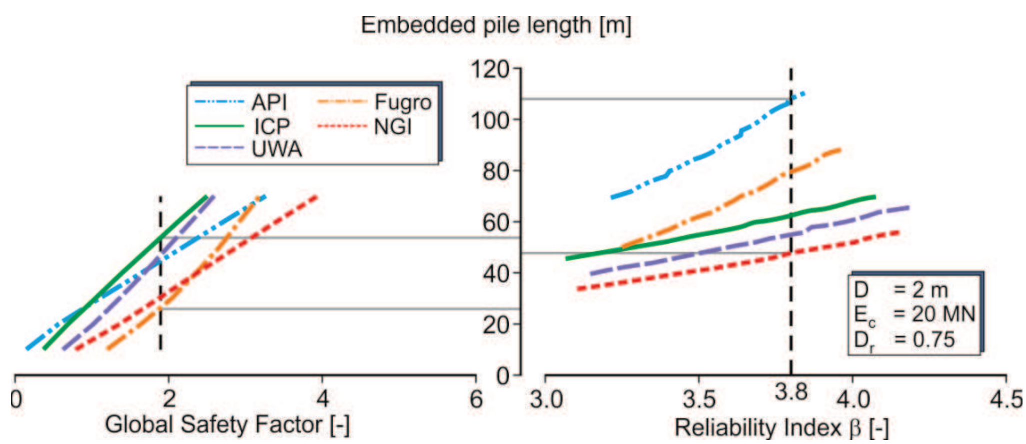

Figure 2. Development of the GSF (left) and Reliability Index (right) with increasing pile length.

Table 1. Assumed stochastic values for the input variables.

Variable	Char. Value	Mean	COV	Type*	Dimension*	Physical Boundaries
q_c	Acc. to Figure 1		94.01 %**	B	1-D field	0.1 - 100 MPa
γ'	9.5 kN/m³	10 kN/m³	23.25 %**	B	1-D field	7 - 13 kN/m³
D_{50}	0.42 mm	0.40 mm	10 %	B	SV	0.06 - 2 mm
$\varepsilon_{t,\varphi'}$	-	0 °	σ = 2.9 °	N	1-D field	30 - 45 °
$\varepsilon_{R,API}$	-	0.72	59 %	LN	SV	-
$\varepsilon_{R,ICP}$	-	0.87	29 %	LN	SV	-
$\varepsilon_{R,UWA}$	-	0.96	19 %	LN	SV	-
$\varepsilon_{R,Fugro}$	-	1.13	31 %	LN	SV	-
$\varepsilon_{R,NGI}$	-	1.00	25 %	LN	SV	-
E	10, 20, 30, 40 MN	0.6 Ec	35 %	G	SV	-

* B = Beta, N = Normal, LN = Lognormal, G = Gumbel, SV = Single variable
** Point COV; for L = 20 - 60 m corresponding COV's are in range of COV_{qc} = 6.64 - 3.84 % and $COV_{\gamma'}$ = 10.87 - 6.57 %

The cone tip resistance and the unit weight were simulated over depth by the application of a 1-D autocorrelated field with an exponential autocorrelation function and an autocorrelation length of 0.05 m for q_c and 2.5 m for γ', respectively. Hence, for the friction angle also a profile over depth was calculated. Thereby the corresponding transformation uncertainty proposed by Phoon & Kulhawy (1999) was taken into account. The variable D_{50} was modeled as single variable. For each design method also a model error was considered. Thereby boundary conditions for the estimation of the model error similar to the investigated system were considered. Hence, a model error according to Schneider et al. (2008) was applied for the bearing compression capacity. These values for the model error are also in good agreement with findings of Lehane et al. (2005). The model error acc. to Schneider et al. (2008) was derived on basis of 16 compression pile load tests for open-ended circular steel piles with a diameter in between D = 0.324 – 1.22 m and an embedded pile length in between L = 5.3 – 79 m. Since the model error is directly applied to the performance function, ε_R was modeled as a single variable. Table 1 summarizes the used stochastic values for the considered input parameters.

6 Reliability based approach

On basis of the described stochastic subsoil model and loading condition a Monte Carlo Simulation with 6E6 realizations for each design case was executed. Thereby the pile embedded length was varied and the pile length corresponding to the reliability index β = 3.8 required for a ULS design acc. to DIN EN 1990 (2010) was sought. Figure 2 (right) depicts the calculated reliability index over the embedded pile length for D = 2 m, D_r = 0.75 and E_c = 20 MN. Since the real bearing capacity and the variation for the subsoil parameters and therefore the real curve of the reliability index over depth is unknown, Figure 2 (right) should be interpreted that for β = 3.8 the embedded pile length should be in a range between approximately 48 – 108 m. However, since all CPT-based design methods indicate a much smaller required pile length for all investigated design cases, the results for the API-method can be considered as unlikely here. Due to this fact, the reliability based results of the API-method are not taken into account for further evaluation in this paper.

7 Determination of Quality factors

Within the new calibration approach the minimum and maximum embedded pile lengths of all design methods, except the API-method, are considered as the upper and lower boundaries for a corresponding reliability index. The mean value of the required embedded pile lengths is assumed as the most likely outcome within this range. Due to the deterministic design the corresponding GSF can be obtained for that pile length. Further, for each design method, soil's relative density and loading condition quality factors η were obtained, where the global safety factor due to the reliability based outcome (GSF_{RBD}) is related to the prescribed GSF of 1.89 ($\eta = GSF_{RBG} / 1.89$). Thus, an integration of all CPT-based design methods and model errors is executed. In that way an indirect increase of the assumed database for the model error is done. For η = 1 no adaption in the GSF or partial safety factor is required. On the other hand the distance from unity for η indicates the rate for a required adaption of a certain design method by the η-value regarding the assumed most likely outcome. Thereby η higher than unity indicates an overestimation of capacity by the design method and vice versa.

Figure 3 depicts the regression lines of the quality factors for each design method, pile diameter and soil relative density as a function of the load. As it can be seen the UWA-method is the most robust one. On the other hand a strong deviation can be noticed for the API- and NGI-methods. For the ICP-method almost no difference regarding the soil relative density can be seen, whereas a strong deviation regarding the diameters can be noticed. The Fugro-method has almost the same deviation in comparison to the ICP-method. Though, the Fugro-method tends to a stronger overestimation of the capacity. For the UWA-, ICP- and Fugro-method it also can be seen that the impact of the load is almost negligible.

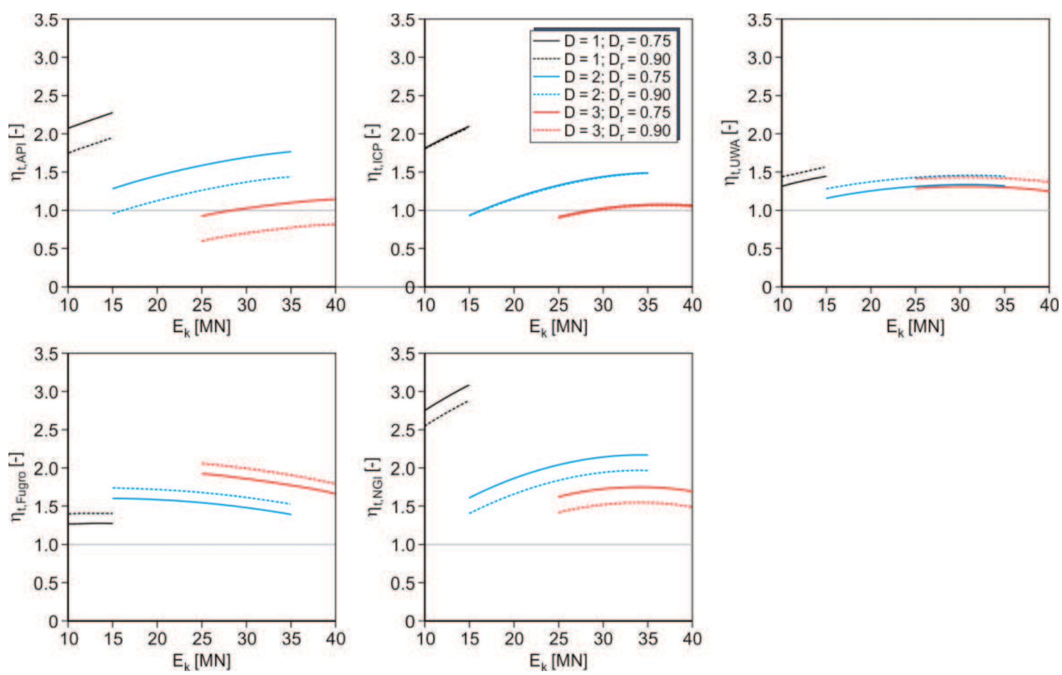

Figure 3. Overview of the quality factors for each design method, assumed density and pile diameter as a function of the load.

Altogether it can be stated that on basis of the investigated pile-soil systems and the assumed model error the UWA-method is the most robust one. Nevertheless, an application of a quality factor of approximately $\eta_{UWA} = 1.35$ to the GSF or partial safety factor of the resistance should be considered within a deterministic design, respectively. However, it should be emphasized that for all design methods the application of the corresponding quality factor for an equal deterministic design case leads to the same embedded pile length, since the quality factors compensate an overestimation or underestimation of the pile capacity within the design methods.

8 Conclusions

In this study typical offshore soil and foundation pile properties for offshore wind converters in the North Sea were assumed. Applying 5 introduced design methods for the determination of the axial compressive resistance, namely the API-, ICP-, UWA-, Fugro- and NGI-methods, in total 50 deterministic designs were conducted. By taking into account typical variability for the assumed soil condition as well as model errors for the investigated design methods, a reliability based determination of the failure probability was executed. Further, a new calibration approach regarding the required GSF for the prescribed failure probability was introduced. Thereby the results due to the reliability based calculations of all design methods were taken into account. Furthermore, quality factors were derived for each design method as function of the load, pile diameter and soil density. On basis of these quality factors a more sophisticated evaluation of both, the considered design method as well as the safety level in terms of the required GSF (or partial safety factor) is possible. Based on the investigated pile-soil systems and the assumed model error the calculated quality factors indicate that the UWA-method is the most robust design method in comparison to the other design methods. By applying the UWA-method an increase in the GSF or the resistance partial safety factor of approximately 1.35 with respect to the prescribed GSF of 1.89 should be considered.

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