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Reliability Based Investigation on the Choice of Characteristic Soil Properties

Jann-Eike Saathoff, Kirill Alexander Schmoor, Martin Achmus, Mauricio Terceros

Abstract—By using partial factors of safety, uncertainties due to the inherent variability of the soil properties and loads are taken into account in the geotechnical design process. According to the reliability index concept in Eurocode-0 in conjunction with Eurocode-7 a minimum safety level of $\beta = 3.8$ for reliability class RC2 shall be established. The reliability of the system depends heavily on the choice of the prespecified safety factor and the choice of the characteristic soil properties. The safety factors stated in the standards are mainly based on experience. However, no general accepted method for the calculation of a characteristic value within the current design practice exists. In this study, a laterally loaded monopile is investigated and the influence of the chosen quantile values of the deterministic system, calculated with p-y springs, will be presented. Monopiles are the most common foundation concepts for offshore wind energy converters. Based on the calculations for non-cohesive soils, a recommendation for an appropriate quantile value for the necessary safety level according to the standards for a deterministic design is given.

***Keywords*—**Asymptotic sampling, characteristic value, monopile foundation, probabilistic design, quantile values.

I. Introduction

THE concept of using piles for the foundation of structures has been proven over centuries. They are among the most widely used foundation concepts onshore, but also offshore. For the foundation of offshore wind energy converters (OWEC) in water depths up to 40 m large-diameter, monopiles are often used.

Monopiles have to be designed such that failure and inadmissible deformation due to lateral loading are avoided. In the design, uncertainties due to the variability of the soil properties and loads are taken into account by using partial factors of safety. These factors shall establish at least a safety level of $\beta = 3.8$ for an ultimate limit state analysis for offshore structures. The reliability of the system depends heavily on the choice of the safety factor and the choice of the characteristic soil properties. The safety factors stated in the standards are mainly based on experience which was gained in the past through different practical onshore applications. Characteristic values on the other hand are not strictly regulated, but the choice of characteristic values also affects the safety level. In this study, the influence of the chosen quantile value of the deterministic system for a laterally loaded monopile foundation will be investigated for the Ultimate Limit State (ULS). The ULS proof regards the system failure and ensures that the strength of the pile-soil system is not exceeded, whereas the SLS proof focuses on limiting the deformations during operation.

J. E. Saathoff, M. Achmus, and M. Terceros are with the Institute for Geotechnical Engineering, Leibniz University Hannover, Germany (e-mail: saathoff@igth.uni-hannover.de, achmus@igth.uni-hannover.de, terceros@igth.uni-hannover.de).

K. A. Schmoor is with the ACP Prof. Achmus + CRP Planungsgesellschaft für Grundbau mbH Berlin, Germany (e-mail: k.schmoor@acp-grundbau.de).

Every design is based on design values R_d which are derived from characteristic values R_k (1). The partial safety factors γ_k are nationally determined and are soil-independent to adjust the correct safety level, whereas the characteristic values should account for site specific properties.

$$R_d = {R_k(X_k)}/{\gamma_k} \tag{1}$$

X_k denotes the characteristic value of a relevant soil parameter (for instance angle of internal friction). Besides the derivation of X_k, the procedure to derive the partial factors of safety γ_k is not regulated and does not account for site-specific variations. Furthermore, already stated safety factors cannot easily be transferred to new or improved design methods as the influence is solely based on experience [1]. Several authors suggest a lack of theoretical basis in the definition of γ_k .

The values for the calculation of the characteristic value can be derived from site-investigation data. In many cases, there is only a small database; the design values are then estimated based on engineering judgment. The Eurocode-7 [2] states that the characteristic values shall be selected as a cautious estimate of the value affecting the occurrence of the limit state. The wording is a clear indicator to include prior knowledge and engineering judgment additionally to the site-specific data in order to estimate the characteristic value.

The EC-7 also suggests using the 5% quantile value, for which for a normal distribution (2) arises. However, the 5% quantile value should not directly be applied to the overall distribution of available data. Furthermore, a set of mean values along the failure surface in question has to be determined as mentioned above. The characteristic value corresponds to the 5% quantile value of the distribution arising from the collected set of mean values. Consequently, a higher quantile value is demanded by the EC-7 in case of considering all available data. To use the 5% quantile on the one hand includes knowledge about the mean value and standard deviation, but on the other hand it is difficult to calculate these correctly as there is often only a small sample size. However, it is in congruence with the definition in other engineering disciplines [3]. Due to a relatively high variation in soil parameters in comparison to the material parameters of other engineering disciplines, this approach may not directly be applied to a soil material. Furthermore, the effect of stress distribution between weaker and stronger areas causes additional difficulties within the estimation of characteristic soil values.

In the following, a λ-value shall be used for comparison purposes. This value indicates the distance of the characteristic value to the mean value when multiplied with the standard deviation e.g. in (2): λ = 1.645.

$$X_{k,5\%} = X_{mean}(1 - 1.645\, X_{COV}) \tag{2}$$

$$X_{k,50\%} = X_{mean} \tag{3}$$

The direct use of the mean values as the characteristic value (λ = 0) does not require knowledge about the soil variability (3). The assumption of no (or a high) soil variability directly affects the assumed reliability of the whole system.

Reference [4] suggests that if there are more than 10 test results a cautious estimate for the characteristic value may be chosen as 0.5 standard deviations under the mean value of the test results (4) [5].

$$X_{k,Schn} = X_{mean} * (1 - 0.5\, X_{COV}) \tag{4}$$

However, in case of pile foundations, there are already approaches which shall be introduced to the next version of the EC-7 [6]. A simple design equation to calculate the characteristic value (5) dependent on the pile length, and a quality index a is foreseen.

$$X_{k,mod} = X_{mean}\left(1 - \mathrm{a}\, 3 \left(\frac{1}{L_v}\right)^{0.5} X_{COV}\right) \tag{5}$$

with a = 0.75 (for an average quality of the test values) and L_v = pile length. This equation considers the distribution of the resistance as well as an independent calibration factor. More details can be found in [6].

In practical designs of OWEC, the p-y method which is a subgrade reaction approach with nonlinear spring stiffnesses is often used. In the paper, at hand a monopile in a homogenous non-cohesive soil is investigated. The p-y approach was chosen as one recent state of the art approach according to [7]. Even if this approach is not stated in the offshore guidelines, it gives realistic results under arbitrary loading conditions. The failure probability of the pile-soil system is determined by applying the asymptotic sampling technique according to [8]. Furthermore, typical variabilities of basic soil parameters such as the angle of internal friction and the soil unit weight are chosen. The stiffness modulus, shear modulus, and other related soil properties have been derived with empirical correlations based on the chosen angle of internal friction. The result of the reliability-based analysis is compared to the deterministic design. The most influencing parameter besides the soil strength is the choice of the model error. The model error accounts for the difference between measured values on-site and the calculated values with empirical methods. Regarding the model error, the approach for the ULS case according to [9] is applied.

II. Calculation and Design Method

A. p-y Curves

The p-y method is usually used for the design of laterally loaded offshore piles [10]. The monopile is substituted with a beam with a bending stiffness EI and the embedment in soil is considered by several uncoupled springs. The non-linear soil response can be implemented by a non-linear relationship of soil displacement y and soil reaction p (p-y curve, Fig. 1). One of the first approaches for p-y curves in cohesionless soil was proposed by [11]. A modification with a hyperbolic tangent function was later introduced by [12], and this approach was incorporated in the American Petroleum Institute (API) [10]. Thereby, the maximum bedding resistance p_u depends on the depth, the diameter of the pile, the angle of internal friction and the unit weight of the soil. The first methods were based on investigations on flexible piles and have later been extended and modified in order to realistically reproduce the behaviour of larger diameter piles. In the following, the approach according to [7] shall be used, which resembles the behaviour of large-diameter pile much better than the API approach.

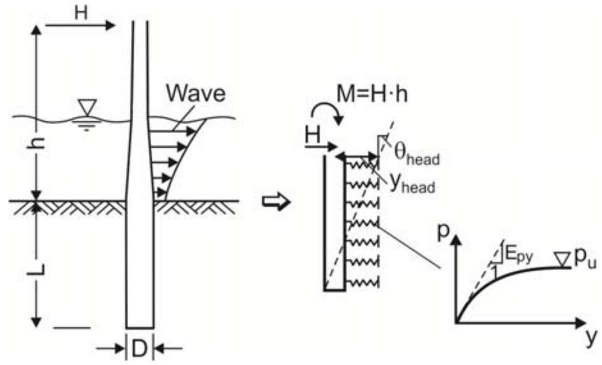
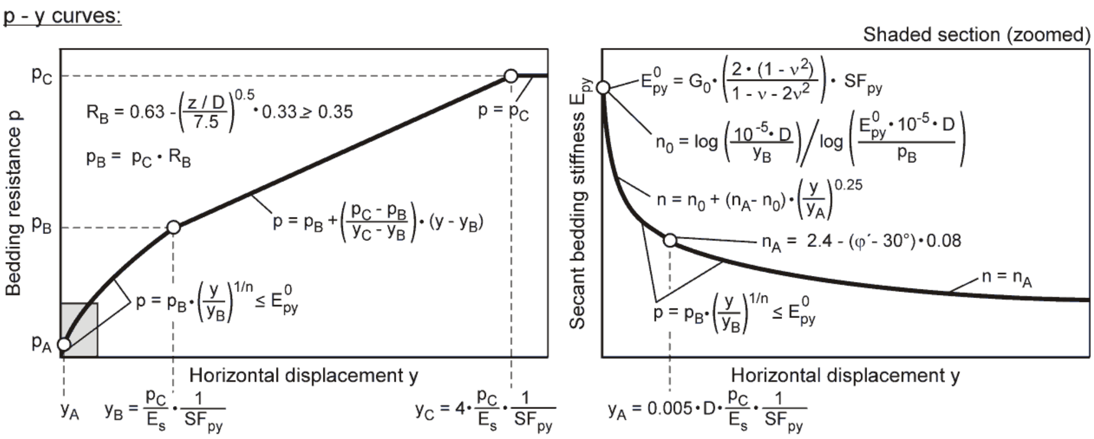


Fig. 1 Concept of p-y curves

The approach is based on a sophisticated numerical model with an advanced material law and a p-y formulation for a constant horizontally displaced pile of infinite length. The resulting p-y curve consists of three sections with an ultimate bedding resistance, a transition point and displacement at which the maximum bedding resistance is fully mobilized (Fig. 2). The value is derived from the spatial earth pressure given in the German standard DIN 4085 [13]. Furthermore, the bedding resistance at shallow depths is corrected (see [7]).

The transition between horizontal displacement and actual (partly rotational) pile deformation of a lateral loaded pile is done in an iterative manner with a “stretching factor” SF_{py}. In this procedure, the pile is calculated with the basic p-y curve, and then, the stretching factor is applied dependent on the deflection of the pile and the relative position of the p-y curve to the rotation point. The calculation of the pile deflection is iteratively repeated with the adapted p-y curves until no further significant change of the deflection line is obtained (Fig. 3).



Ultimate bedding resistance p_C:

$$p_C = p_C^{Basic} \cdot \left[3 - 2 \cdot (z / (2.5 \cdot D))^{0.25}\right] \geq p_C^{Basic}$$

$$p_C^{Basic} = \frac{11}{16} \cdot \gamma \cdot z^{1.5} \cdot K_{pgh} \cdot (1 + 2 \cdot \tan\varphi) \cdot \sqrt{D}$$

$$K_{pgh} = \sqrt{\frac{\left(\left(\frac{1 + \sin\varphi'}{1 - \sin\varphi'}\right) \cdot (1 - 0.53 \cdot \delta_p)^{0.26 + 5.96 \cdot \varphi'}\right)^2}{1 + (\tan\delta_p)^2}}$$

$$\delta_p = -2/3 \cdot \varphi'$$

Stretching factor SF_{py}:

$$SF_{py} = SF_{py,bend} + SF_{py,tip}$$

$$SF_{py,tip} = 5 \cdot \left(\frac{z - L}{2 \cdot D} + 1\right)^5 \quad \text{if } z \geq L - 2 \cdot D$$

$$SF_{py,tip} = 5 \cdot \left(\frac{z - L}{2 \cdot D} + 1\right)^5 + 3 \quad \text{if } z \geq L - 0.1 \cdot D$$

Basic curves: $SF_{py} = 1.0$

Fig. 2 Equations of the p-y approach of [7]

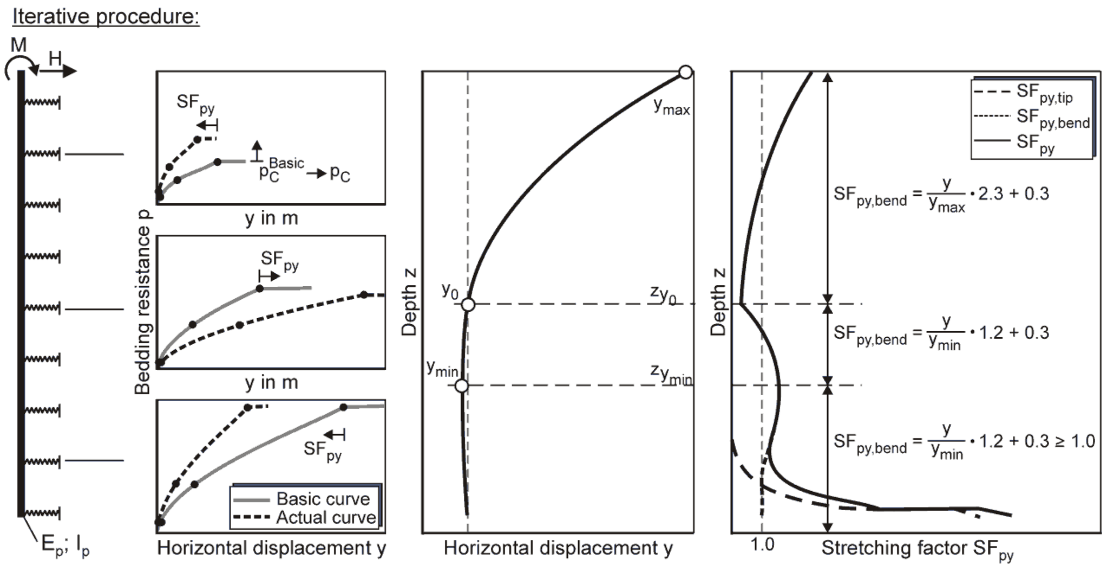


Fig. 3 Conception of the p-y approach of [7]

Compared to the API approach, the p-y curve according to Thieken et al. [7] needs two additional input parameters besides the unit weight γ' and the friction angle φ', namely the dynamic oedometric soil stiffness E_{sd} (the dynamic shear modulus G_0 and the Poisson's ratio ν) and the static oedometric soil stiffness E_S. The stiffness is assumed to be depth-dependent with a reference value at p = 100 kPa and a power m_G for the shear modulus and m_{Es} for the stiffness modulus (compare Table II).

The calculations with different p-y methods presented in this paper were done by using the in-house software IGtHPile [14], in which the Thieken et al. [7] and API [10] approach for cohesionless soils are implemented.

B. DIN 1054 and EC-7 Geo-2 ULS Proof

According to EC-7 the ULS design proof can be a Geo-2 or Geo-3 proof. The entire Geo-2 calculation is carried out by using characteristic values. Only in the last step, when checking the limit state equation, the characteristic effects and resistances are factorized by the partial safety factors. In the Geo-3 method, the design values of the effects and resistances of the subsoil are directly determined with design values of shear parameters. This means that the partial safety factors are applied to the shear parameters before application of any calculation method.

In Germany, according to [15], a Geo-2 design is required for laterally loaded piles. Generally, the action E_d has to be smaller than the reaction R_d (6).

$$E_d \leq R_d \tag{6}$$

According to the German standard DIN 1054, the rotation point and the bedding resistance are calculated by using characteristic loads and soil parameters. The resultant action is the integration of the bedding reaction and the resultant resistance is the integration of the ultimate resistance p_u from mudline to the point of rotation (Fig. 4). The partial factors of safety are applied to the characteristic action and resistance. The factor for persistent load is $\gamma_R = 1.4$ and for the effect $\gamma_E = 1.35$. The shortcomings of this design proof are the load dependent position of the rotation point and the potential negligence of capacity reserves below the point of rotation [16]. A modification is the Geo-2 design in which the lateral failure load $H_{k,ult}$ is compared to the actual acting load H_k (Fig. 4, right). This approach is comparable with a GEO-3 design [16] and is used here.

The proof ensures that the characteristic load is smaller than the failure load. The safety level is introduced by the respective safety factors. The ultimate load $H_{k,ult}$ is calculated by steadily increasing the acting load until failure occurs, i.e. the soil resistances p_u are fully mobilized along the whole pile length. The design proof is then given by (7):

$$H_k \, \gamma_E = {H_{k,ult}}/{\gamma_R} \tag{7}$$

III. Pile-soil System under Consideration

The investigated monopile has a diameter of 6 m and an embedded length of approximately 30 m for a lateral characteristic load of 7 MN with a fixed load eccentricity of 30 m. The wall thickness is assumed to be 6.6 cm, and a homogenous sand layer is considered. For the 50 year load event, a Gumbel distribution with a COV = 0.35 is taken into account [17] (Table I).

A model error shall be taken into account. A high model error is necessary if the design method lacks several physical fundamentals and is unable to calculate arbitrary model conditions. The overall influence of the model error is very high, as it closes the gap between model assumption and real soil-structure behaviour. It may also include statistical uncertainties and method errors. However, measurement errors are mainly considered in the total COV of the soil property. The implied error depends on many specific factors such as model assumptions, quality of measuring equipment and characteristic of inherent soil variability. The model error shall be incorporated in this analysis with a lognormal distribution with $\mu_{ME} = 1.19$ and $COV_{ME} = 0.43$ [18]. These values are according to [11] for drained conditions.

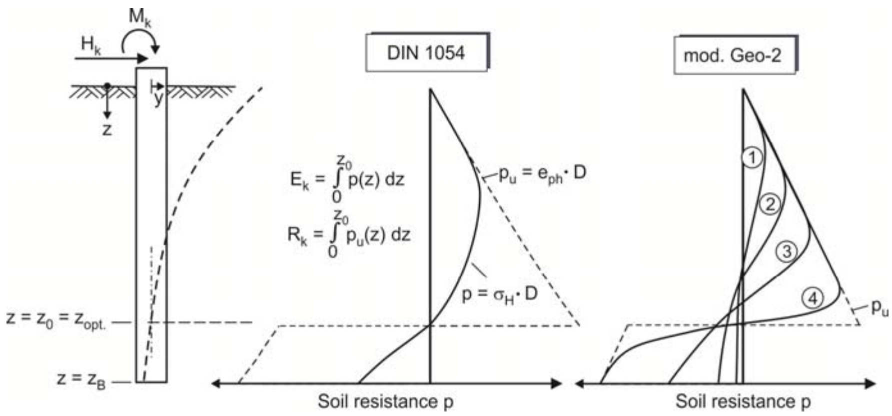


Fig. 4 ULS proof acc. to DIN 1054 and modified Geo-2 proof

Regarding the soil variability, it is generally difficult to depict the overall material variability. In the following, literature values shall be used; for the unit weight the inherent variability is assumed with a $COV_{w\text{-}\gamma'} = 0.1$ and a measurement error $COV_e = 0.01$, which concludes to a total $COV_{\gamma'} = 0.1$ (8). For the angle of internal friction a $COV_{w\text{-}\varphi'} = 0.1$ and a measurement error $COV_e = 0.1$ have been used which lead to a total $COV_{\varphi'} = 0.14$ (e.g. [18]).

$$COV = \sqrt{COV_w{}^2 + COV_e{}^2} \tag{8}$$

The soil properties for the investigated pile with their appropriate distributions are shown in Table I. The properties of the pile itself have not been stochastically distributed.

As the particular COV is only valid for a spatially limited point consideration, the related 1D pile length may be taken into account by using a vertical autocorrelation length θ of 3 m [18]. For a 30 m pile, the σ^2_{total} may be reduced to 42.4% with L/θ = 10 (9).

$$\sigma^2_{total} = \sigma^2 \left(2\left(\frac{L}{\theta} - 1 + \exp\left(\frac{-L}{\theta}\right)\right)\left(\frac{\theta}{L}\right)^2 \right) \tag{9}$$

TABLE I
PARAMETERS FOR REFERENCE SYSTEM WITH DISTRIBUTION

Parameter	Distribution	μ	σ	COV
Buoyant unit weight, γ' [kN/m³]	Normal	10	1.0	0.1
Angle of internal friction, φ' [°]	Lognormal	35	3.5	0.1
Lateral load, H [MN]	Gumbel	4.2	1.47	0.35
Modelling error	Lognormal	1.19	0.51	0.43

A further reduction applied to the soil resistance distribution is possible and used in the following. Reference [19] describes the positive spatial effect for lateral loaded monopile foundations, which allows reducing the total standard deviation of the soil resistance distribution to 80% due to the consideration of additional horizontal autocorrelation in-situ.

TABLE II
PARAMETERS FOR REFERENCE SYSTEM AND FOR SYSTEMS WITH VARIED SOIL PROPERTIES WITH VERY DENSE (VD), DENSE (D), MEDIUM DENSE (MD), LOOSE (L) AND VERY LOOSE (VL) SOIL

Description	VD	D	MD	L	VL
Unit weight, γ'[kN/m³]	10.31	10	9.76	9.41	9.1
Friction angle, φ' [°]	40	37.5	35	32.5	30
Poisson's ratio, ν [1]	0.2	0.225	0.25	0.275	0.3
Stiffness parameter, m_G [1]	0.5	0.5	0.5	0.5	0.5
Stiffness parameter, m_{Es} [1]	0.5	0.55	0.6	0.65	0.7
Stiffness parameter, $E_{s,ref}$ [kN/m²]	700	500	400	325	250
Dynamic stiffness ratio, E_{dyn}/E_s [1]	4.9	5.5	5.9	6.36	6.95

IV. RELIABILITY BASED CALCULATION

For OWEC, the target safety level in the normal safety class is according to the standards a nominal annual failure probability of $p_f = 10^{-4}$ for unmanned structures during a high loading event [20]. The value represents reliability class 2. A minimum safety level of $\beta = 3.8$ for reliability class RC2 according to the reliability index concept in Eurocode-0 [21] in conjunction with Eurocode-7 shall be established. The reliability index β is related to p_f according to (10):

$$\beta = \Phi^{-1}(1-p_f) \tag{10}$$

Herein, Φ^{-1} is the inverse cumulative distribution function of the standardised Gaussian distribution. The simplest and most comprehensive manner to carry out a reliability- based calculation is a plain Monte Carlo simulation (MCS). The idea is to calculate many different systems with randomly chosen properties from the corresponding density functions. Therefore, to achieve a variation of 5% with a $\beta = 3.8$ approximately $5.5 \cdot 10^6$ plain MCS are required [22]:

$$n_{req} = \frac{1-p_f}{p_f \ \mathrm{var}_{pf}^{\ 2}} \tag{11}$$

A sampling method is used in order to reduce the number of required calculations. This method increases the number of calculations in the failure zone and tries to converge to the particular result for a theoretical high number of simulations.

A. Asymptotic Sampling Technique

The asymptotic sampling technique presented in [8] is based on the asymptotic behaviour of the failure probability as the failure probability converges to zero. The safety index can be given for $f = 1/\sigma$ as:

$$\beta(f) = f\,\beta(1) \tag{12}$$

The value for β(f) can be derived for different values of f and hence the standard deviation σ in a MCS and then extrapolated to β(1). The chosen function is:

$$\beta(f) = A\,f + \frac{B}{f} \tag{13}$$

The general procedure starts by choosing a start value for f ($f \leq 1$) and a number of required failure samples No (here taken with No = 5). The reduced number of Monte Carlo calculations Cn can be estimated with a short preliminary calculation. In the course of the calculation, the number of failures in each run will increase with decreasing f. If there are sufficient failures in one run, the β-f-pair will be saved and the calculation will continue, with a chosen step size df, until K runs have been reached. The step size and df have also been estimated in preliminary simulations. Examples of this procedure are given in Fig. 5 [23].

B. Calibration of Sampling Technique

The results of the asymptotic sampling technique have been compared to the results of a plain MCS with at least 10^6 simulations ($\beta_{MCS} = 3.72$, dashed line in Fig. 5). Fig. 5 shows the results for the reference system. Table III shows varied parameter by means of preliminary analyses in order to get sufficiently accurate results, but decrease with calculation time. Thereby, four parameters have been varied and already presented in the last section. The range of these values is similar to the values found by the authors for other soil properties and pile dimensions.

TABLE III
VARIED PARAMETER LIMITS FOR ASYMPTOTIC SAMPLING

K	3	5
Cn	1E3	1E6
Df	0.05	0.1
f_{start}	0.95	0.8

The diamond markers in Fig. 5 (Cn = 1000, K = 5) cover a wide range, but overestimate the safety level, because of the small sample size; the cross markers (Cn = 10000, K = 3) have a higher number of calculations, but are not well distributed, hence the safety level is overestimated within the regression. The simulations with 1000 calculations in each step do not lead to sufficiently accurate results with a sufficiently small variation and the simulations with $1 \cdot 10^5$ calculations are computationally very expensive. The best estimation could be achieved by using five points with 10000 calculations with an f_{start}-value of 0.9 and a df of 0.1. For the reference system with an embedded length of 23 m and a pile diameter D = 6 m, a mean value of the safety level $\beta_\mu = 3.87$ with $\beta_\sigma = 0.037$ was established.

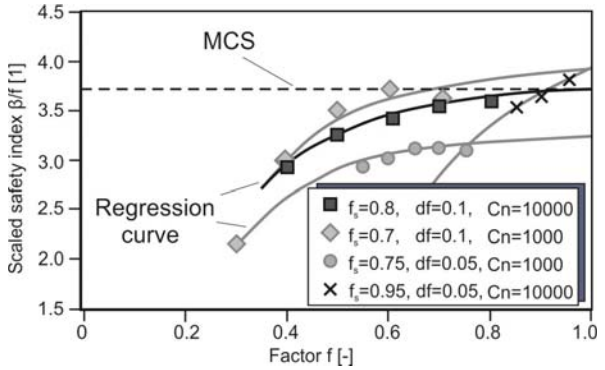


Fig. 5 Concept of asymptotic sampling method with exemplary results of preliminary analyses [23]

V. Comparison of Different Modeling Methods

A probabilistic analysis for lateral loaded piles in sand can be carried out in different ways. Fig. 6 shows results for the API approach and results for the approach according to Thieken et al. in which the angle of internal friction and the unit weight have been changed in order to show the influences of the modeling approaches. As the API approach does not realistically reproduce the pile behaviour for larger diameter piles, the approach according to [7] was subsequently used. In this stage only the two parameters mentioned beforehand are stochastically considered with constant stiffness parameters. However, from field and laboratory tests it can be shown that not only the angle of internal friction, but also the stiffness parameters are mutually affected due to the inherent soil variability. Correlations have been used to derive related parameters from the angle of internal friction, which was seen as an indicator for the relative density. In the course of the simulation, the angle of internal friction was varied according to a lognormal distribution, and all other parameters have been related to this change by using non-linear equations. Table II shows soil properties for different relative densities. The exponent m was varied linearly and the stiffness of the soil with a second degree polynomial related to the angle of internal friction as an first indicator for the relative density. The shear modulus was derived from the stiffness modulus and back-calculated with a bi-potency function according to the diagram depicted in the [24]. The two approaches show a different sensitivity to the variation of input parameters by means of the bedding resistance -independent of which approach may be more realistic. The friction angle is taken into account with factors c_1, c_2, and c_3 in the case of the API approach and in Thieken et al. in p_{cbasic}, in the earth pressure coefficient and the friction interaction angle (see [7], [10] and [25]).

With the reference values of Table II and the asymptotic sampling technique an investigation of the safety level and utilization factor for different pile lengths with an autocorrelation length of 3 m was made. Due to the usage of the modified Geo-2 proof, the proof is independent on the position of the rotation point contrary to the DIN 1054 proof. $H_{k,ult}$ results to 13.4 MN. The model error can hence directly be applied to the soil resistance. By doing so the soil variability is kept constant.

Fig. 6 (a) shows calculated β-values for pile lengths between 20 m and 27 m and on the right λ, the related distance of the standard deviation subtracted from the mean value for a utilization factor of 1. The vertical lines on the right side represent the 5% quantile and λ according to (5). For both approaches separate deterministic calculations have to be carried out for which a reliability based design may be done. Thereby, the API approach results in a $\lambda = 1.35$ and the Thieken et al. approach for a constant stiffness parameter $\lambda = 1.4$. For a complete variation of the input values $\lambda = 1.44$ arises. The differentiation between the two Thieken et al. approaches results in only a minor difference. However, the variation of all parameters seems to reproduce the reality more realistically and is hence used in the following.

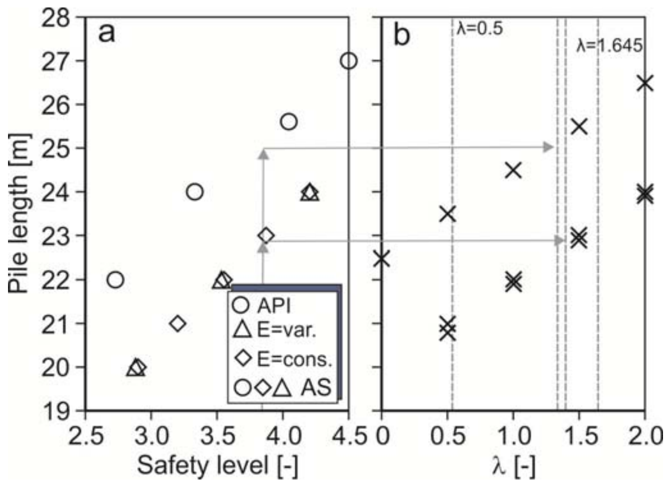


Fig. 6 Safety level (a) and λ-value (b) for the reference system for the ULS according to [7] and [10]

VI. Choice of Quantile Values

Fig. 7 shows three probability density functions: the acting lateral load H and the soil resistances R_1 and R_2. The deterministic load is assumed to be at 7 MN and represents the 95% quantile of the Gumbel distribution. From the lateral load H_k, the necessary resistance R_k can be calculated with the global safety level of 1.89 to 13.2 MN (cf. (7)). The position of R_k in the distribution depends on the COV of the resistance and the choice of the input values of the soil strength and stiffness.

The soil variation is the same for both resistance distributions. However, R_2 establishes a higher β value than R_1. To consider more broad or narrow distributions the choice of the characteristic values shifts the distribution in a way that the particular safety level can be guaranteed. The required quantile value R_k for R_2 is different from R_1.

The difference in λ_1 and λ_2 is related to the choice of the input values. In current practice, there is no influence of the COV related to the characteristic value to account for the site-specific inherent variability of the soil resistance. This example does not consider different variability. For a higher variation coefficient, an even higher distance to R_k may be necessary.

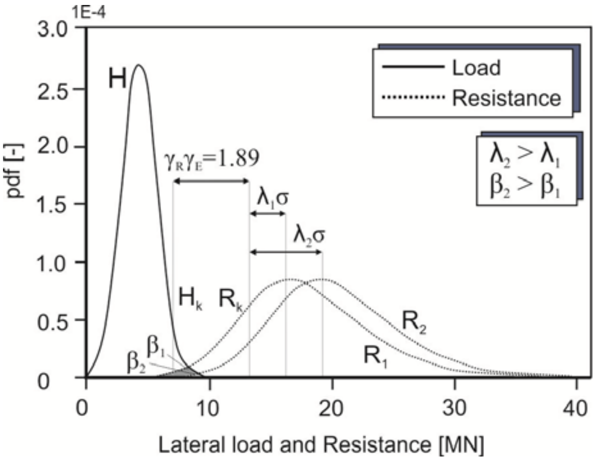


Fig. 7 Load and resistance distribution with safety factor

VII. Results

A. Dependency on Load Conditions and Pile Length

Fig. 8 (a) shows calculated β-values for pile lengths between 18 m and 25 m and on the right λ, the related distance of the standard deviation subtracted from the mean value for a utilization factor of 1. The reference system was investigated and additionally lateral loads of 5 MN (diamond marker) and 9 MN (triangle marker) were applied in order to investigate the load level dependency.

From the required β-value the λ-value can be derived and compared to the equations already given. In Fig. 8, there are three cluster trends, as the pile length increases for a higher load. The results for the asymptotic sampling and results for all three loading conditions for a plain Monte-Carlo simulation are presented. The very good agreement between the results can be seen as an indicator for the general applicability of the sampling method.

For a modified lateral load the pile length changes accordingly. A required β-value of 3.8 can be achieved for $\lambda = 1.44$ for a lateral load of 7 MN. For a reduced lateral load of 5 MN, the value is reduced to $\lambda = 1.38$ and for a higher lateral load of 9 MN it is derived to $\lambda = 1.43$.

The resulting λ-values are more in the range of the 5% quantile ($\lambda = 1.645$) compared to the in practice often applied $\lambda = 0.5$ (4). However, the 5% quantile may be appropriate for structural but not for geotechnical engineering, and hence, (2) and (3) give no realistic values. The general influence of the input parameter and a comparison without any model factors is also shown in the last paragraph.

B. Dependency on Pile Diameter

As the system behaviour is highly nonlinear, the diameter of the system was changed to D = 4 m and D = 8 m. The acting load is kept as H = 7 MN, because an independence of the results to the load level is assumed.

Fig. 9 shows the results of the calculations. The λ-value results in both cases to 1.4 for the required safety level and for an utilization ratio of the pile in the deterministic proof of 1.

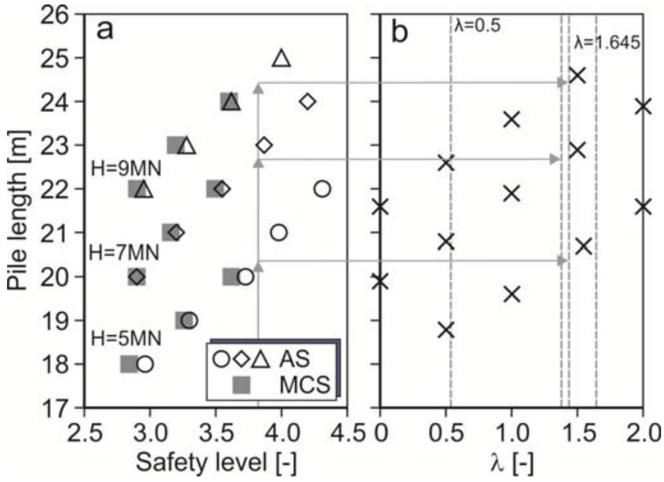


Fig. 8 Safety level (a) and λ-value (b) for ULS for H = 5 MN, H = 7 MN and H = 9 MN and comparison with MCS results

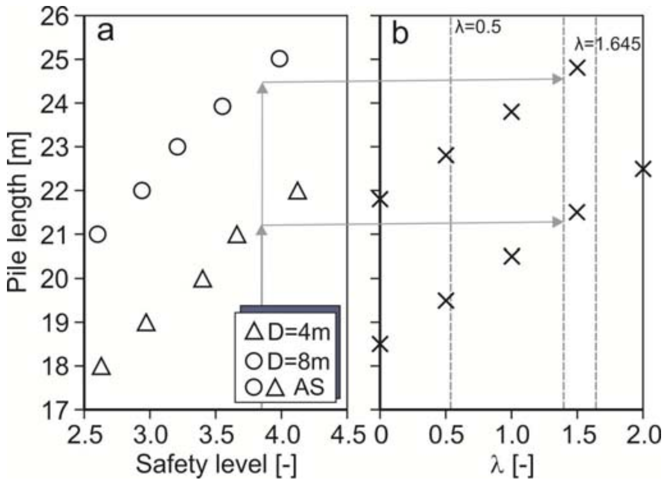


Fig. 9 Safety level (a) and λ-value (b) for ULS with D = 4 m and D = 8 m for H = 7 MN

C. Dependency on Ground Condition

In order to investigate the system response to different soil properties, the mean angle of internal friction is once decreased to 32.5° and once increased to 37.5° in the reference pile system. The variation coefficient of sand is assumed to be still realistic and hence not varied. The pile dimensions and load situation are not adapted and kept constant, because an influence of the diameter and load level has already been investigated and the change in the results is not very pronounced. The λ-values of the soil properties results to $\lambda = 1.45$ and $\lambda = 1.3$ (Fig. 10) for a de- and increase of the mean value, respectively. This is the same range as the other λ-values.

D. Dependency on Model Factor

The main influencing parameter in the overall simulation is the model error. The chosen factor in the presented analysis is based on the values according to [11]. However, the model factor may vary more in the design of piles for cohesionless soil than for cohesive soil [26]. According to the literature, the standard deviation may also be smaller [9]. The mean value of the model error of roughly 1.2 seems to fit very well with the stated values.

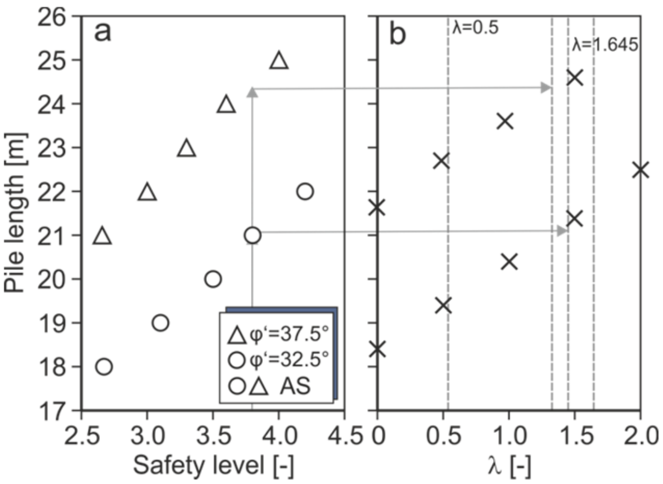


Fig. 10 Safety level (a) and λ-value (b) for ULS with D = 6 m and φ' = 32.5° and 35° for H = 7 MN

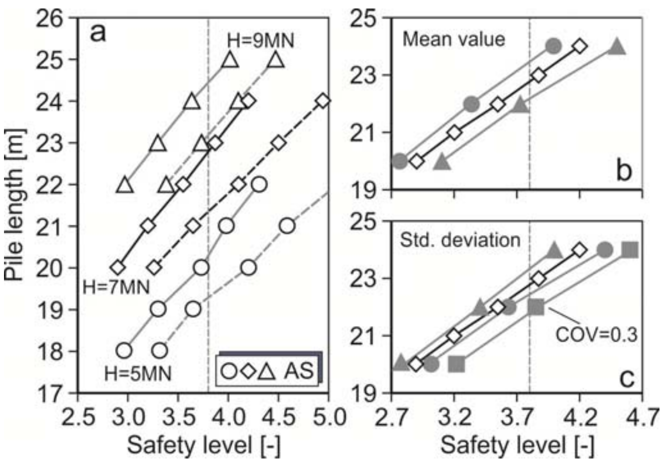


Fig. 11 Safety level for reference model with negligence of model error (a) and varied mean value (b) and standard deviation (c) of model error

Fig. 11 shows the results of a sensitivity study. The mean value and the standard deviation were varied by ±5% respectively. For the neglection of this aspect a value λ = 0.8 instead of λ = 1.44 is derived. A comparison of the safety level with and without (dashed lines) model error is depicted in Fig. 11 (a). The reason for the deviation is the shift of the mean value of the resistance due to the distribution of the model error. For no or high model errors, the mean value of the soil resistance is not shifted or slightly shifted onto the safe side, whereas for model errors with a large standard deviation or mean values smaller than unity, the shift results to the unsafe side and λ increases.

The sensitivity to ±5% is depicted in Fig. 11 (right). The influence can clearly be seen in the change of the λ-value, which is larger for a variation of the mean value than for the standard deviation (compare with Fig. 8). For an increase of the mean value, λ results to 1.21 and for a decrease to 1.76. If the standard deviation is increased, a higher λ-value of 1.72 results and for a decrease λ equals 1.34. The overall λ-value can be reduced up to 15%. For COV_{ME} = 0.3 (Fig. 11 (c)) even a value of 0.95 arises which gives a reduction of 35%. However, in order to account for a correct approximation, reliable model errors need to be established which then in turn will reduce the calculated λ-values in a more appropriate region.

VIII. Conclusion

The design safety depends on how design values are derived. The general safety level is undermined by the non-regulated characteristic design on which the partial factors of safety are applied [27]. The presented investigation deals with the question to what extent the mean parameter of a soil parameter should be reduced to determine the characteristic soil parameter leading to the desired safety level. Here, reliability analyses for a monopile system in homogenous sand were conducted in order to derive suitable characteristic values for the angle of friction of sand. The characteristic acting load herein was set to the 95% quantile value of a Gumbel distribution. Also a model error with a lognormal distribution was considered. Besides the comparison of different p-y methods, a comparison of two model strategies was carried out. In the approach according to [7] the friction angle was varied and the influence of changed stiffness parameters investigated. This approach was subsequently used in the analysis for non-cohesive soils. The influence of the load level, the soil properties and the pile dimensions have been evaluated. In all cases similar results could be achieved. The parametric study showed reductions of the mean value λ = 1.3 to λ = 1.4. The reduction is higher than considered in (5), which is mainly attributed with the governing influence of the model factor. The values are in the region of the 5% value (λ = 1.645) quantile, which would however be overly conservative. A recent approach stated in (5), in which the suitable reduction is formulated depending on the COV of the soil property, the pile length and a quality index, seems very promising. In a last step, the model error was separately investigated as this value seems to produce the very high λ-values. For a further verification, reliable model errors need to be established which will reduce the calculated λ-values to a more appropriate region. Finally, an equation similar to (5) should be derived. It should include all necessary influencing parameters and lead overall to a more consistent choice of characteristic values in engineering practice.

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