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DESIGN OF MONOPILE FOUNDATIONS FOR OFFSHORE WIND ENERGY PLANTS

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ABSTRACT

The foundation of offshore wind energy plants plays an important role in the stability of these structures. One foundation concept which can be used in this field is the monopile concept. Here the foundation consists of one large-diameter pile (up to 8.0 m). Common design practice, e. g. according to API regulations, does not cover horizontally loaded piles of such dimensions. Thus, the soil-structure-interaction and the behaviour of these foundations has to be covered using numerical modelling. This paper aims to investigate the behaviour of the monopile under monotonous loading taking the interaction between the pile and the subsoil into account. A three-dimensional numerical model using the finite element method was established. In this model the non-linear material behaviour of the subsoil is described using an elasto-plastic constitutive model. The interactions between the monopile and the surrounding soil are modelled using contact elements. A parametric study with different pile geometries, soil and loading conditions has been carried out. The results of the finite element simulations are presented and evaluated. Finally, an overview on the state of knowledge concerning the influence of cyclic loading is presented and open questions are discussed.

Keywords: Constitutive Model, Cyclic loading, Interface Friction, Monopile, Numerical Modelling, Offshore Wind Energy Plants.

1 INTRODUCTION

The planned offshore wind parks in the German part of North and Baltic Sea will be constructed in water depths varying from approximately 15 to 40 m. By means of suitable foundation constructions, the large horizontal forces and bending moments resulting from wind and wave loadings, must be economically and safely transferred to the sea soil. Monopile foundations can be used as one of these foundation types. In principle the monopile is an

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extension of the main tower into the soil under the sea bed (Fig. 1). This foundation method was already implemented in North and Baltic Sea, but only for wind energy plants in water depths of less than about 10 m. Its application is expected to be possible for water depths up to about 25 to 30 m. The diameters of such monopiles vary between 6 and 8 m.

Since wind energy plants are relatively sensitive to deformations, in particular tilting, it is really important to estimate these as exactly as possible. For the mentioned large-diameters piles, till now there is no exact design procedure approved. In this paper, the results of numerical investigations of the load-deformation behaviour of monopiles under static loads are presented.

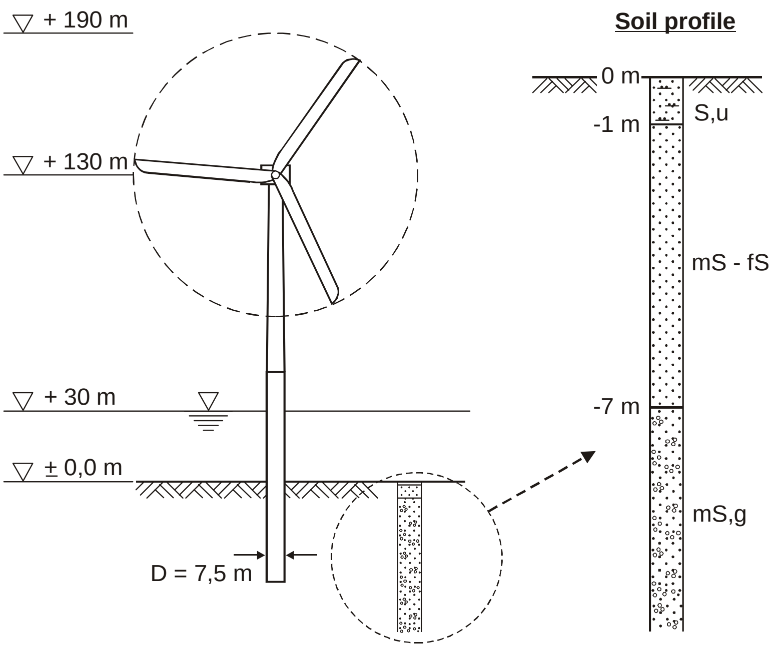

Fig. 1: Example of a monopile foundation.

2 STATE OF THE ART FOR COMPUTATION OF HORIZONTALLY LOADED MONOPILE FOUNDATIONS

The design procedure for OWEP foundations in Germany is given in the Germanische Lloyd rules and regulations (GL 1999). In this regulation, concerning the behaviour of piles under horizontal loading reference is made to the regulation code of the American Petroleum Institute (API 2000). The Norwegian guidelines (DNV 2004) also refer to the API code. In the API code the p-y method is recommended for the design of horizontally loaded piles.

In principle, the p-y method is a subgrade modulus method with non-linear and depth-dependent load-deformation (p-y) characteristics of the soil springs.

API (2000) describes the construction of p-y-curves for soft and stiff clay as well as for sandy soils. Due to API, p-y-curves for sandy soils can be derived as follows:

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

$$p_{us} = (c_1\, z + c_2\, D)\,\gamma'\, z \qquad (1.1)$$

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

The first mentioned equation applies to small depths (p_{us}) and the second equation for larger depths (p_{ud}), the smaller of both values is to be considered. The influence of the internal friction angle is described by the factors c_1, c_2 and c_3 (see Fig. 2 left).

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

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

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

and $A = 0.9$ for cyclic loading.

Herein p is the soil resistance per unit length of the pile and y is the actual horizontal deflection. The parameter k describes the initial modulus of subgrade reaction and is dependent on the relative density I_D and/or on the angle of internal friction (Fig. 2 right).

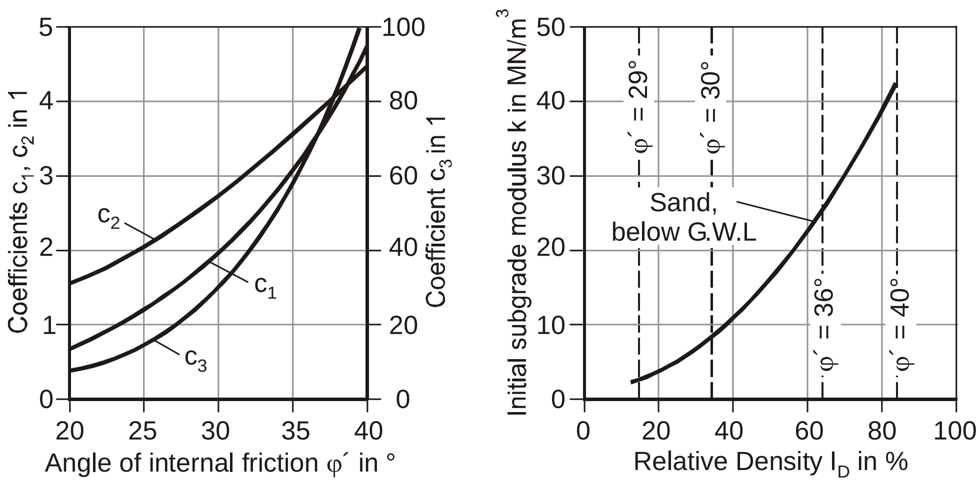

Fig. 2: Coefficients c_1, c_2, c_3 and initial modulus k given in API (2000).

The equations (1) and (2) are 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 loading and then evaluated their results. For cyclic tests, a maximum number of 100 load cycles was realized. The correction factor A according to equation (2) was adjusted based on the measurements done.

The application of this method worked satisfactorily in offshore practice over many years, whereby the collected experiences only refer to piles with diameters up to 2 m. Therefore, the API procedure should not be used directly for monopiles of large diameters (Achmus & Abdel Rahman 2004, Lesny et al. 2002). After Lesny & Wiemann (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.

In the following, the results of numerical calculations of the load-deformation behaviour of monopiles are presented and compared with the results of the API p-y method. The calculations with the API method were carried out by means of the LPILE program (Lpile 2000).

3 NUMERICAL MODELLING OF MONOPILE BEHAVIOUR UNDER STATIC MONOTONOUS LOADING

For the investigation of the load-deformation behaviour of monopiles of large diameters, three-dimensional finite element calculations were accomplished. Piles with a diameter of D = 7.5 m having two different embedded lengths under the sea-bed of L = 20 m and L = 30 m were investigated.
Different load application heights h of the load above sea-bed and thus combinations of horizontal force H and bending moment $M = H \times h$ were realized. Additionally, a vertical load was applied to take the structure's weight into account.

The computations were done with the program system ABAQUS (Abaqus 2004). In order to carry out a lot of calculations for varying boundary and loading conditions, a large computer system with parallel processor technology was used to minimize the time effort.
Due to the symmetric loading condition only a half-cylinder representing the sub-soil and the monopile could be considered. The discretized model area had a diameter of 90 m, which is twelve times the pile diameter. The bottom boundary of the model was taken 15 m below the base of the monopile. With these model dimensions the calculated behaviour of the pile is not influenced by the boundaries (Fig. 3 left).
For the soil as well as for the pile 8-node continuum elements were used. The frictional behaviour in the boundary surface between pile and soil was modelled by contact elements, whereby the wall friction angle was set to δ = 0.67 φ'. The material behaviour of the monopile was assumed linear elastic with the parameters $E = 2.1 \cdot 10^5$ MN/m^2 (Young's modulus) and $\nu = 0.2$ (Poisson's ratio) for steel.
Of crucial importance for the quality of the numerical computation results of soil structure interactions is the modelling of the material behaviour of the soil.
The elasto-plastic material law with Mohr-Coulomb failure criterion, provided in the ABAQUS program, was used. This material law was extended in the elastic range by a stress-dependency of the oedometric modulus of elasticity with the following equation:

$$E_S = \kappa\, \sigma_{at} \left(\frac{\sigma_m}{\sigma_{at}} \right)^{\lambda} \tag{3}$$

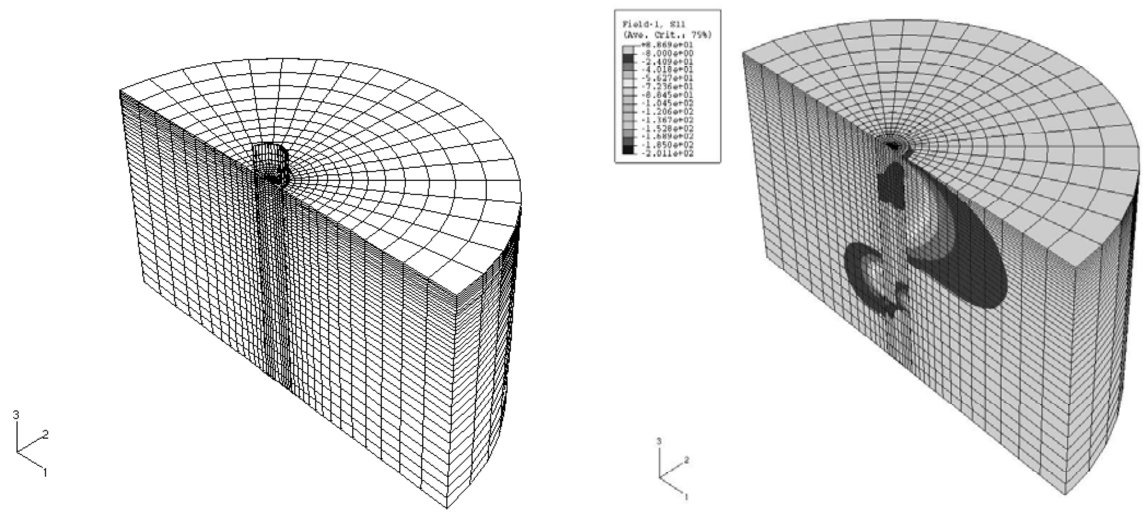

Fig. 3: Finite element mesh and horizontal bedding pressure (in the symmetry axis of the monopile) for D = 7.5 m, L = 30 m in dense sand, H = 8 MN, h/L = 1.

Herein σ_{at} = 100 kN/m^2 is a reference stress and σ_m is the current mean principle stress in the regarded soil element. The parameter κ determines the soil stiffness at the reference stress state and the parameter λ rules the stress dependency of the soil stiffness.
This material law has the advantage that it can be generally used for both cohesive and non-cohesive soils. In the context of the computations presented here, the material parameters used with reference to EAU (1996) are shown in Table 1.

Table 1: Material parameters used in the numerical computations.

Material	Unit weight γ' in kN/m^3	Stiffness		Poisson's ratio ν in 1	Shear parameters		
		κ in 1	λ in 1		φ' in °	c' in kN/m^2	ψ in °
Sand, medium dense	11	400	0.60	0.25	35	0.1	5
Sand, dense	11	600	0.55	0.25	37.5	0.1	7.5
Boulder clay, semi-firm	11	40	0.90	0.25	32.5	15	2.5
Mud sand	10	E_s = 24 MN/m^2		0.25	27.5	1	0

The finite element calculation is executed stepwise. At first, for the generation of the initial stress state the whole model area is discretized using soil elements only. Subsequently, the monopile is generated by replacing the soil elements located at the pile position by steel elements and activating the contact conditions between pile and soil.
Then the vertical load is applied, and finally the horizontal load is applied and increased step by step. The monopile elements were extended above the ground surface of the model in order to realize different load combinations (horizontal forces and bending moments).

For a resultant horizontal load of 8 MN and a bending moment at sea-bed level of 240 MNm, which is in the order of a possible design load for the considered large water depths, the horizontal (bedding) stresses acting on the pile in the symmetry plane are shown in Fig. 3, right. The characteristic loading behaviour of the pile with bedding stresses of opposite sign above and below a point of rotation can be seen clearly. For the considered case the point of rotation lies about 22 m below sea bed.

In Fig. 4, the numerically determined deflection lines for piles of two diameters (D = 2 m, D = 7.5 m) in homogeneous dense sand are plotted against the results computed according to the API procedure. For the pile with 2 m diameter, results for a horizontal force of 3 MN agree quite good in the case of high moment load (h/L = 1), whereas the deformations obtained by the API method are clearly smaller with h/L = 0. For the pile with 7.5 m diameter, results show in both cases relatively large deviations.
Of course, the finite element results do not necessarily reflect the reality, because assumptions have to be made concerning initial stress state and material behaviour. Using "calibrated" material parameters, good agreement may be obtained for both pile diameters in the case h/L = 0. But, in that case the pile deformations for h/L = 1 will be significantly underestimated by the API method.
The main reason for the deviations is probably the overestimation of the soil stiffness in large depths by the API method. Considering the initial stiffness, the API method predicts for medium dense sand stiffness values in the order of 500 MN/m^2 in depths of around 20 m. This is of particular influence on the results for high load levels, i. e. in the example given in Fig. 4 for h/L = 1 much more than for h/L = 0. Moreover, for a large-diameter pile the shearing resistance in the pile tip area may play a more important role compared to a small-diameter piles.
It is thus concluded that in general the use of the API method for the design of large-diameter monopiles is not suitable.

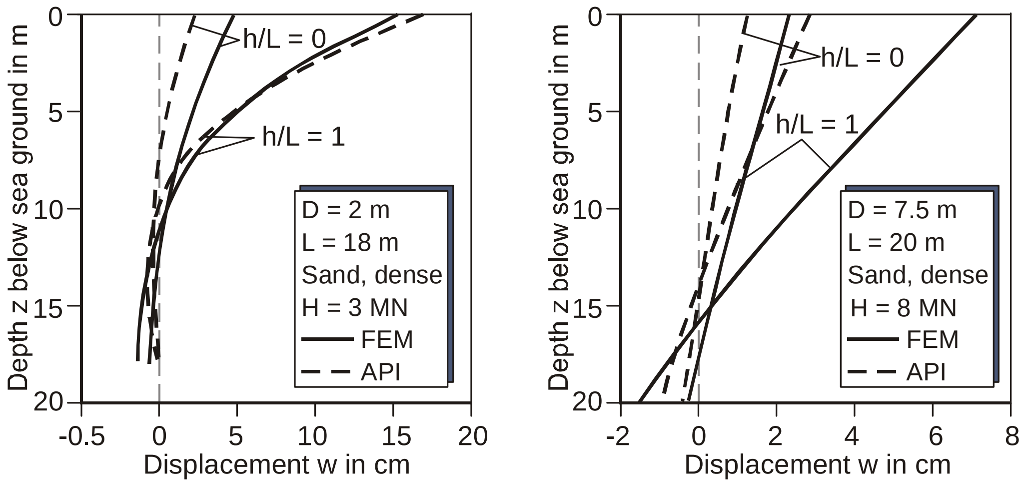

Fig. 4: Exemplary comparison of the pile deflection according to API method and numerical simulation for monopiles in dense sand.

Regarding the parametric studies, different pile diameters *D*, different pile lengths *L* and different heights of point of load application *h* were numerically simulated. Diagrams representing the pile head displacement *w* and the pile head rotation ϕ at sea-bed level as a function of the horizontal load were determined. Thereby three soil profiles given in Fig. 5 were considered to idealize different soil profiles in the North Sea.

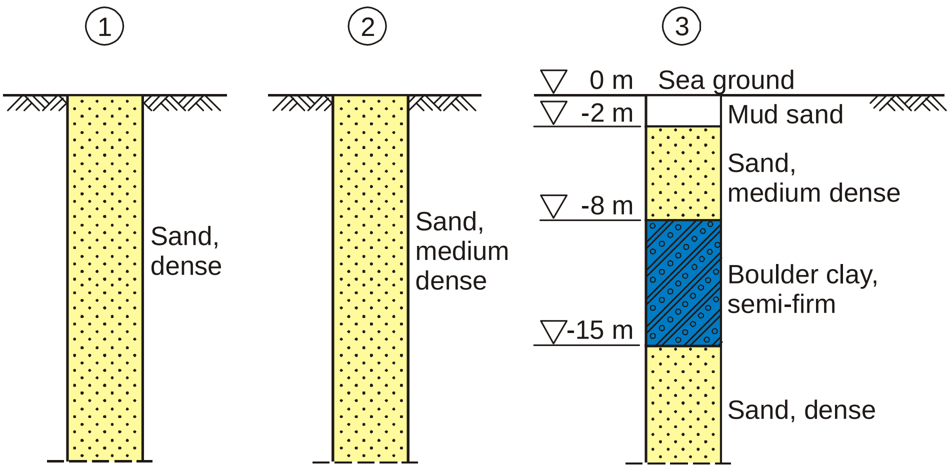

Fig. 5: Idealised soil profiles considered in the numerical computations.

For each combination of pile geometry and soil profile, two diagrams were derived (*H-w* and *H-ϕ* curves) as a result. These diagrams are represented in Figures 6, 7 and 8. If at a specific site similar or comparable soil profiles exist, a pre-dimensioning of a monopile foundation for static load design can be carried out on the basis of these diagrams.

The expected influence of the pile length and the height of the point of application of the horizontal load is evident from the Figures given below. In homogeneous sand the deformations for *h/L* = 1 are approximately double of the deformations for *h/L* = 0.2. For the layered soil profile the ratio is even higher, which is mainly due to the relatively small stiffness of the boulder clay layer.

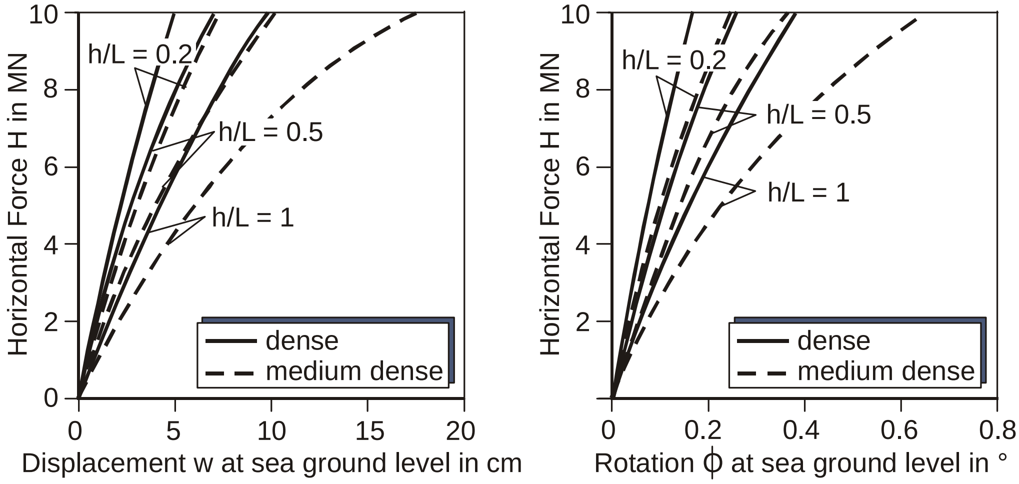

Fig. 6: Load-deformation curves for monopile D = 7.5 m, L = 20 m in medium dense and dense sands (Soil profiles 1 and 2 with regard to Fig. 5).

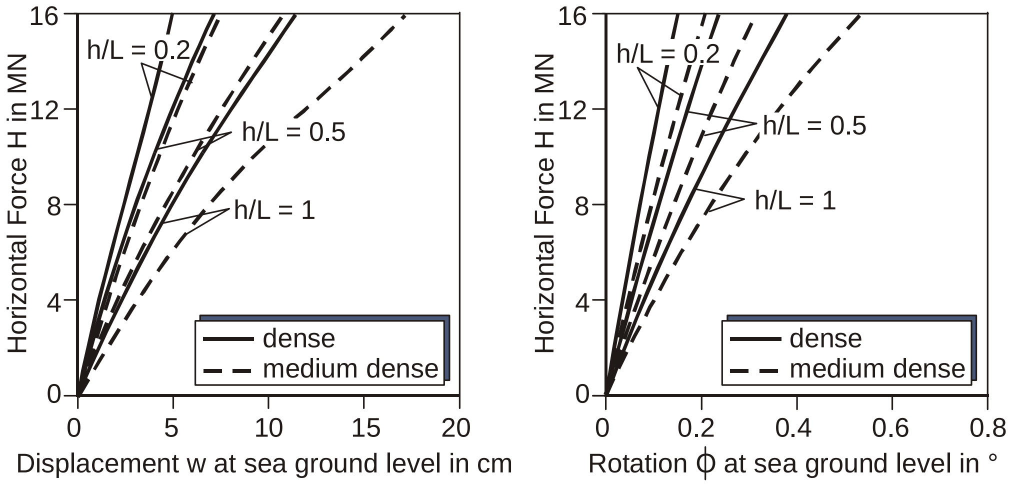

Fig. 7: Load-deformation curves for monopile D = 7.5 m, L = 30 m in medium dense and dense sands (Soil profiles 1 and 2 with regard to Fig. 5).

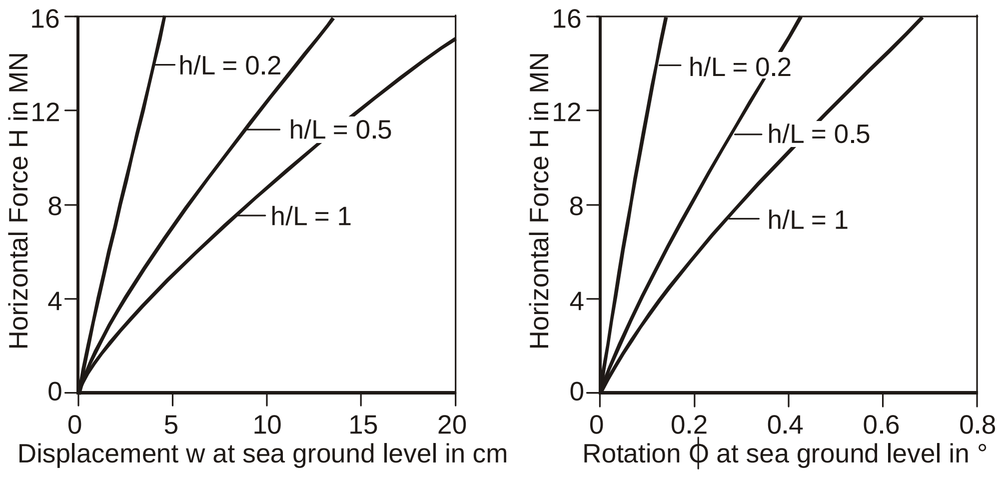

Fig. 8: Load-deformation curves for monopile D = 7.5 m, L = 30 m in layered soil (Soil profile 3 with regard to Fig. 5).

Considering Figs. 6 and 7, the influence of the relative density of non-cohesive soils is evident. Thus, a soil exploration with thorough determination of relative densities is to be recommended for the design of monopile foundations.
Finally, comparing Fig. 6 and 7 the influence of the monopile lengths is elucidated. For a monopile with 20 m length, the deformations with H = 10 MN are in the same order as with H = 16 MN for a monopile with 30 m length. Thus, a 50% prolongation of the embedded pile length leads in the considered case to an about 60% higher admissible load with regard to serviceability of the foundation.

4 EFFECT OF CYCLIC LOADING

During the lifetime of an OWEP billions of loading cases induced by wave and wind actions apply to the structure. Thus, the loading has to be classified as intensely cyclic and fatigue design is of great importance.
Concerning the foundation structure, cyclic loading leads to an accumulation of permanent displacements. According to the German offshore regulation (GL 1999), the effect of cyclic loading of the foundation structure has to be taken into account. Unfortunately, concerning monopiles no approved method exists to estimate the permanent displacements due to cyclic loading. In the following, a short summary of the state of knowledge regarding this matter is given.
Due to the API method, the factor A in equation (2) shall be set to 0.9 for cyclic loads. This leads to moderate increases of the calculated displacements, which is elucidated by an example calculation in Fig. 9.

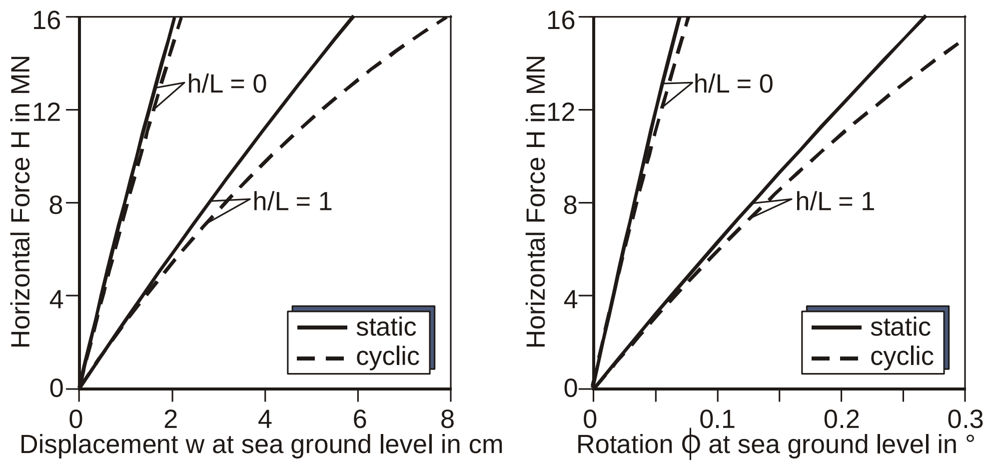

Fig. 9: Comparison of load-deformation curves for a monopile D = 7.5 m, L = 30 m in medium dense sand for static and cyclic loading due to API (2000).

The API approach was derived by means of loading tests in which mostly less than 100 load cycles were applied. In fact, strain accumulation does not stop reaching 100 cycles. Hettler (1981) proposed the following equation for the displacement of a pile in sand loaded by N cycles of the same horizontal load:

$$w_N = w_1 \left(1 + C_N \ln N\right) \tag{4}$$

Herein w_1 is the displacement for static loading and C_N is a factor which for sand lies in the range of 0.20.

Long & Vanneste (1994) proposed a subgrade reaction method with linear increasing subgrade modulus with depth, in which the moduli decrease with the number of load cycles due to the following equation (see Fig. 10 left):

$$k_s(z) = N^{-t}\, n_{h,1}\, z \tag{5}$$

Herein t is a factor dependent on the pile installation method, the load characteristic (one- or two-way loading) and on the relative density of the sand. For a driven pile with one-way loading in medium dense sand t = 0.17 is recommended. With this value an example was calculated. The results are given in Fig. 10. It was found that after 5000 cycles the displacement at the pile top is more than 3 times the static displacement.

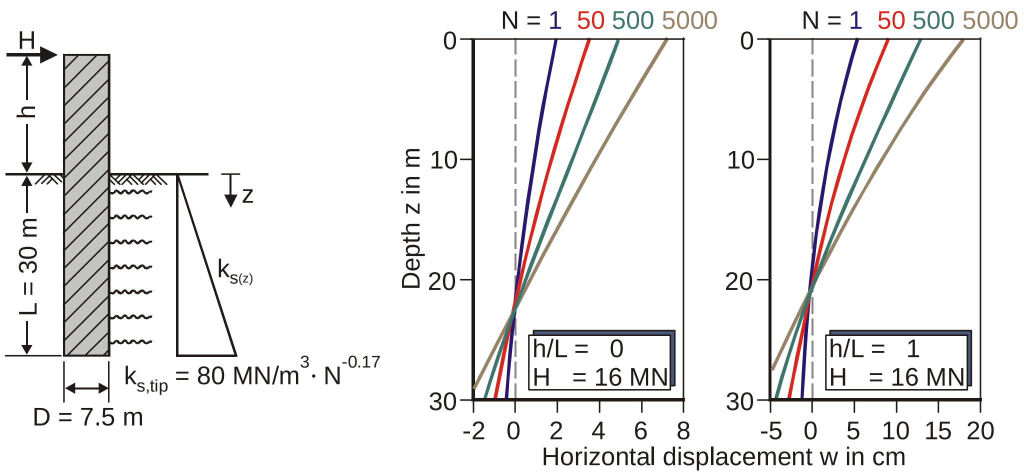

Fig. 10: Influence of cyclic one-way loading due to the method from Long & Vanneste (1994) for a monopile D = 7.5 m, L = 30 m in medium dense sand.

In reality, the amplitude of the load is varying with the wave heights and the wind velocities. For loads of varying amplitudes Lin & Liao (1999) proposed a strain superposition method to determine the resultant permanent displacement. For that the following equations apply:

$$w_{Ges} = w_{1,1}\left[1 + t \ln\left(N_1 + \sum_{k=2}^{n} N_k^*\right)\right] \tag{6.1}$$

$$\text{with} \qquad N_k^* = e^{\frac{1}{t}\left(\frac{w_{1,k}}{w_{1,1}}(1 + t \ln N_k) - 1\right)} \tag{6.2}$$

Herein $w_{1,i}$ is the static displacement due to a load B_i, which can be obtained by the calculation of the static load-displacement curves presented in section 3.

However, due to Lin & Liao the displacements obtained with this method overestimate the real displacements for large numbers of cycles. Additionally, also the directions of the loads are varying. For two-way cyclic loading it is known that the cyclic displacements are significantly smaller than for one-way loading due to densification effects in the soil (see Hettler 1981, Long & Vanneste 1994). But at least the pile behaviour under general variation of loading directions and amplitudes is an open question, which will be object of further research.

5 CONCLUSIONS

The use of the API method for the computation of the deformations of large-diameter monopile foundations for offshore wind energy plants cannot be generally recommended. This applies to the design for static loads and particularly to the estimation of the influence of cyclic loading.

For static loads, numerical investigations are recommended at present, as they were presented in this paper. Of course, such investigations are complex and time-consuming. For preliminary design steps diagrams can be helpful, which allow a simple determination of the approximate pile deformations to be expected for a specific case.

The behaviour of monopile foundations under cyclic loading of varying amplitude and direction is up to now not well understood and must be a subject of future research.

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