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On the design of monopile foundations with respect to static and quasi-static cyclic loading

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Abstract:

Most of the existing wind energy converters in the North Sea are founded on monopiles, and this foundation concept seems also very promising for the wind farms planned in the German parts of the North and the Baltic Sea with water depths of about 20 to 40 m. For the wind energy converters foreseen, monopile diameters of 6 to 8 m will be necessary.

The design of horizontally loaded offshore piles is commonly done using the p-y-curve method according to the API regulations. In this regulation, p-y-curve approaches for static as well as for cyclic loading are given. But, the admissibility of this method for large-diameter piles is not proved.

A three-dimensional numerical model for the investigation of the monopile behaviour under monotonous loading was established. In this model the non-linear material behaviour of the subsoil is described using an elasto-plastic constitutive model with a stress-dependent stiffness formulation. Results of a parametric study with different pile geometries, soil and loading conditions are presented and compared with results from the API method for sandy soils. The comparison indicates that the API method is not in general suitable for the design of large-diameter piles.

Subsequently, cyclic loading of monopiles is considered. An overview is given on the nature of cyclic loads due to wave loading. The API approach for cyclic loading appears not suitable to cover the loading conditions. Existing methods to deal with such loads are presented and discussed. It is shown that at the time being no reliable method exists to estimate accumulated displacements under cyclic loads of varying amplitudes and directions. Possible ways of dealing with cyclic loading in the design of monopiles are discussed.

On the design of monopile foundations with respect to static and quasi-static cyclic loading

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1 Introduction

The monopile concept (Fig. 1) is an elegant and fast-to-install foundation structure for offshore wind energy converters (OWEC's). Most of the existing wind energy converters in the North Sea – which are located in relatively shallow water – are founded on monopiles.
This foundation concept seems also very promising for the wind farms planned in the German parts of the North and the Baltic Sea with water depths of about 20 to 40 m. For the wind energy converters foreseen, monopile diameters of 6 to 8 m will be necessary, whereas the maximum diameter of installed monopiles so far was about 5 m.

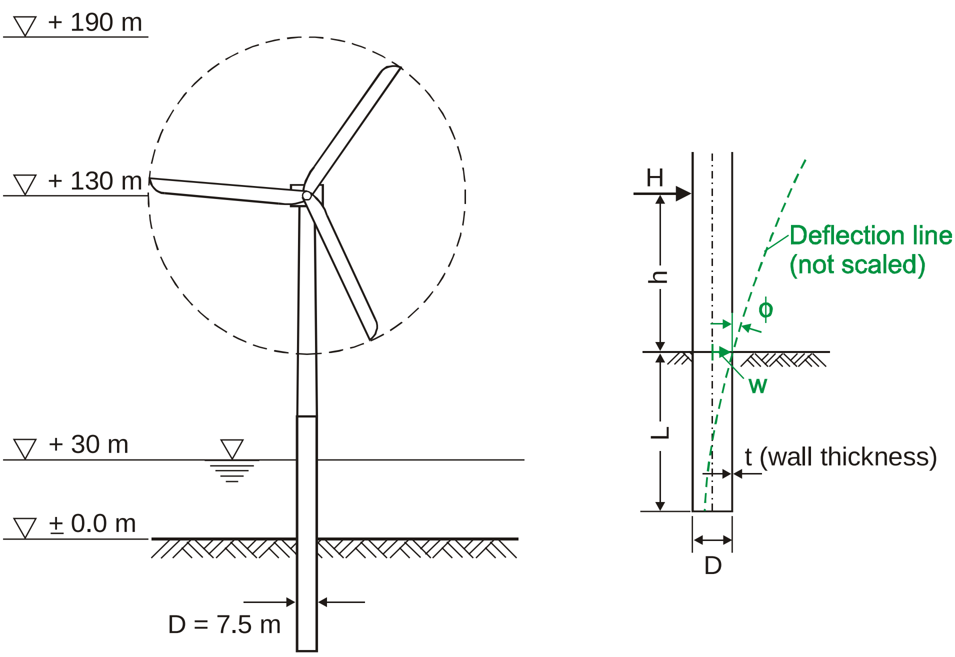


Figure 1. Monopile foundation (schematic).

Decisive for the design of OWEC foundations in general is the loading by horizontal forces and bending moments induced by wind, current and wave loads and (in the Baltic Sea) also by ice loads. A monopile transfers these forces by way of horizontal earth pressure on the pile into the ground. Especially wind and wave loads are of extremely cyclic nature. The effect of cyclic loading has to be taken into account for the ultimate limit design as well as for the serviceability limit design.
Concerning the design of monopiles, in the current regulations (DNV 2004, GL 1999) reference is made to a special subgrade reaction method (p-y curve method) given in API (2000). However, this method was formerly only used for piles with diameters of up to about 3 m, and questions arise concerning the transfer of the method to very large-diameter piles (e. g. Wiemann & Lesny 2004, Abdel-Rahman & Achmus 2005).
In this paper, results of Finite Element (FE) calculations of the behaviour of monopiles under static loading and considerations on the influence of cyclic loading are presented.

2 Design of monopiles due to current regulations

The design procedure for OWEC foundations is in Germany 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. This method is in principle a subgrade modulus method with non-linear and depth-dependent load-deformation (p-y) characteristics of the soil springs.

As the focus in this paper lies on monopile foundations in sandy soils, the API procedure to construct p-y curves for sandy soils is outlined briefly:
1) The ultimate lateral resistance per unit length p_u is taken as the minimum of two expressions. The first one is valid for shallow depths, whereas the second is valid for greater depths:

$$p_{us} = (c_1 z + c_2 D)\gamma' z \tag{1}$$

$$p_{ud} = c_3 D \gamma' z \tag{2}$$

Herein z is the given depth in metres, D is the pile diameter in metres, γ' is the effective unit weight of soil (kN/m^3). The coefficients c_1, c_2, c_3 are dependent on the friction angle of the soil.

2) The p-y curve is given at a specific depth by the following expression:

$$p = A\, p_u \tanh\left(\frac{k\, z}{A\, p_u}\, y \right) \tag{3}$$

where $A = 3.0 - 0.8\, z / D \geq 0.9$ for static loading and $A = 0.9$ for cyclic loading, p is the soil resistance per unit length, y is the actual lateral deflection and k is the initial modulus of subgrade reaction determined as a function of the friction angle.

Load-displacement and load-rotation curves determined with this method for a monopile D = 7.5 m, L = 30 m embedded in sand (φ’ = 35°) are given in Figure 2 for static as well as for cyclic loading. Even for high design load levels with H = 16 MN and h = L = 30 m the displacements lie in a range which might be admissible concerning the serviceability of the structure. Due to the API method, the effect of cyclic loading leads only to a relatively slight increase of deformations.

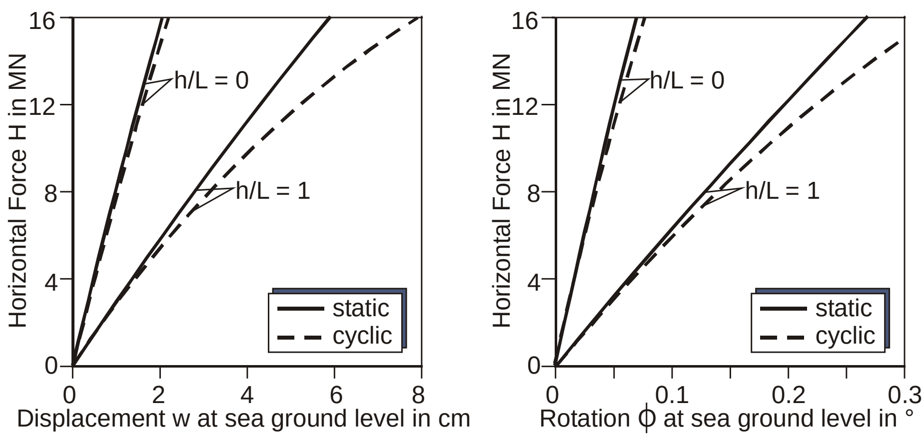


Figure 2. Monopile deformations according to API method (D = 7.5 m, L = 30 m, t = 9 cm, sand φ’ = 35°).

Since the API method is not confirmed by experience for piles of very large diameters, these results have to be checked. The question is to be answered, whether the method can be used also for the design of large-diameter piles. This holds both for static (design) load and for the effect of cyclic loading.

3 Numerical modelling of monopile behaviour due to static loading

For the investigation of the behaviour of laterally loaded monopiles with large diameters, a three-dimensional (3-D) numerical model was established. The computations were done using the finite element program system ABAQUS (Abaqus 2004). In order to carry out many calculations for varying boundary and loading conditions, a large computer system with parallel processor technology was used to minimize the time effort.
The aim of the investigation was to analyse the behaviour of a large monopile in principle and to check whether the API method can be used for such large piles. For that, an idealized homogeneous soil consisting of medium dense or dense sand was considered. A monopile diameter of D = 7.5 m and a wall thickness of 9 cm was assumed.
The most important item of geotechnical numerical modelling is the simulation of the soil's stress-strain-behaviour. An elasto-plastic material law with Mohr-Coulomb failure criterion was used. The soil stiffness is herein represented by a stiffness modulus for oedometric compression E_S and a Poisson's ratio ν. To account for the non-linear soil behaviour, a stress dependency of the stiffness modulus was implemented as follows:

$$E_S = \kappa \, \sigma_{at} \left(\frac{\sigma}{\sigma_{at}} \right)^{\lambda} \tag{4}$$

Herein σ_{at} = 100 kN/m^2 is a reference (atmospheric) stress and σ is the current mean principal stress in the considered 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.
The material parameters used in the calculations are given in Table 1. Concerning more details about the numerical modelling reference is made to Abdel-Rahman & Achmus (2005).

Table 1. Material parameters used for dense sand / medium dense sand.

	dense	medium dense
Unit buoyant weight γ'	11.0 kN/m^3	11.0 kN/m^3
Oedometric stiffness parameter κ	600	400
Oedometric stiffness parameter λ	0.55	0.60
Poisson's ratio ν	0.25	0.25
Internal friction angle φ'	37.5°	35°
Dilation angle ψ	7.0°	5°
Cohesion c'	0.1 kN/m^2	0.1 kN/m^2

For an example (D=7.5 m, L=20 m, H = 8 MN, dense sand) calculated deflection lines are shown in Figure 3 and compared with API method results. For a dense sand, the choice of an angle of internal friction of φ' = 35° seems suitable. This corresponds with an initial bedding modulus of k = 22 MN/m^3. From Figure 3 (left) it is evident that this yields too low deflections for the practically relevant cases of h/L > 0. Better agreement regarding head deflections is obtained for setting φ' = 32.5° (k = 14 MN/m^3) with the API method, see Figure

3 right. However, the overall deflection lines remain different. Of course, also the FE results do not represent the truth and have thus to be checked. But, the findings give rise to the conclusion that the API method for large-diameter piles should be used with great care, especially concerning the choice of the k-value.

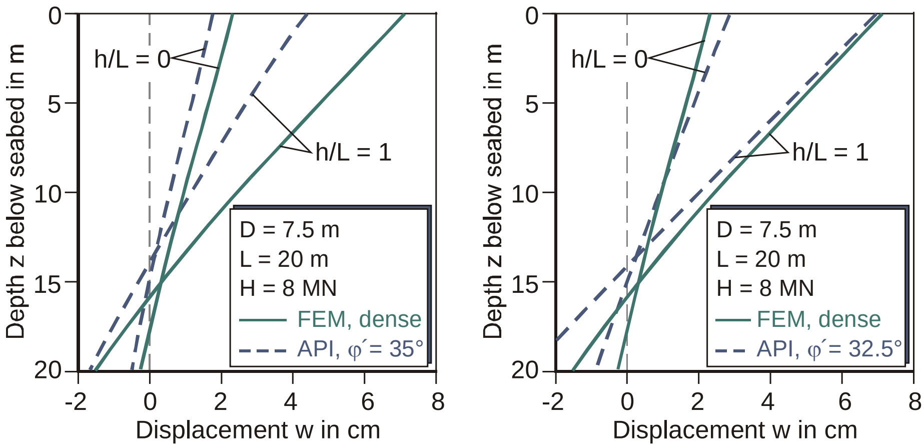


Figure 3. Comparison of deflection lines determined by FEM and by the API method (D = 7.5 m, L = 20 m, dense sand, H = 8 MN).

For a concrete design task force-head displacement and force-head rotation curves can be helpful, because especially the limitation of head rotation ϕ (cf. Fig. 1) is of importance for the serviceability of the wind energy converter. As an example, such curves are given for D = 7.5 m, L = 30 m, dense sand in Figure 4. The comparison with API results verifies the finding stated above that this method gives much lower deformations.

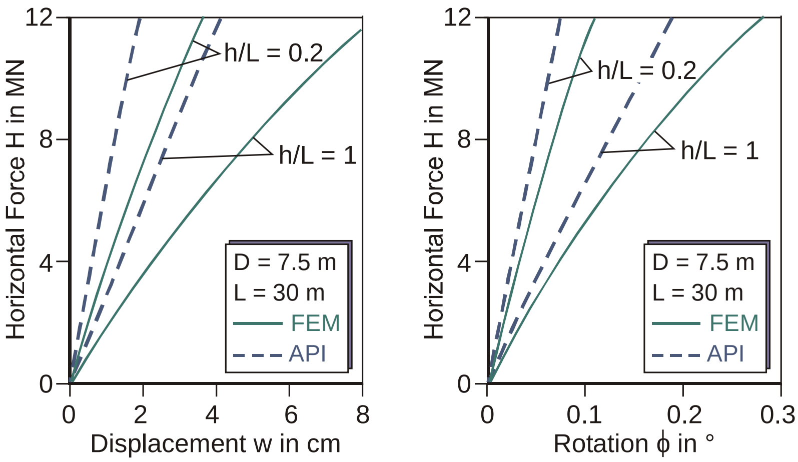


Figure 4. Force-displacement and force-rotation curves determined by FEM and by the API method (D = 7.5 m, L = 30 m, dense sand, API: k = 22 MN/m^3).

It is evident from Figure 4 that the load-deflection relationships are only slightly curved. Thus, at least for a certain load range, the behaviour can be described by the following parameters, which may be interpreted as integral stiffness parameters:

$$C_w = \frac{H}{w} \tag{5}$$

$$C_{\phi} = \frac{H}{\phi} \tag{6}$$

$C_w(h)$ and $C_\phi(h)$ diagrams can be derived by evaluation of a number of numerical calculations and can give a good overview on the behaviour of monopiles with different diameters and lengths.
In Figure 5 such diagrams are given for the case of monopiles in dense and medium dense sand. The integral stiffness values were calculated for a load of H = 8 MN.
The results given indicate that an increase of the embedded pile length from 20 to 30 m significantly increases the integral stiffnesses nearly by the factor 2. Thus, the deformations for a given load are nearly halved by lengthening the pile by 10 m.
An increase of the pile diameter from 5 to 7.5 m has a similar effect. By this measure, the integral stiffnesses are at least doubled, i. e. the pile deformations at sea bed level are more than halved.

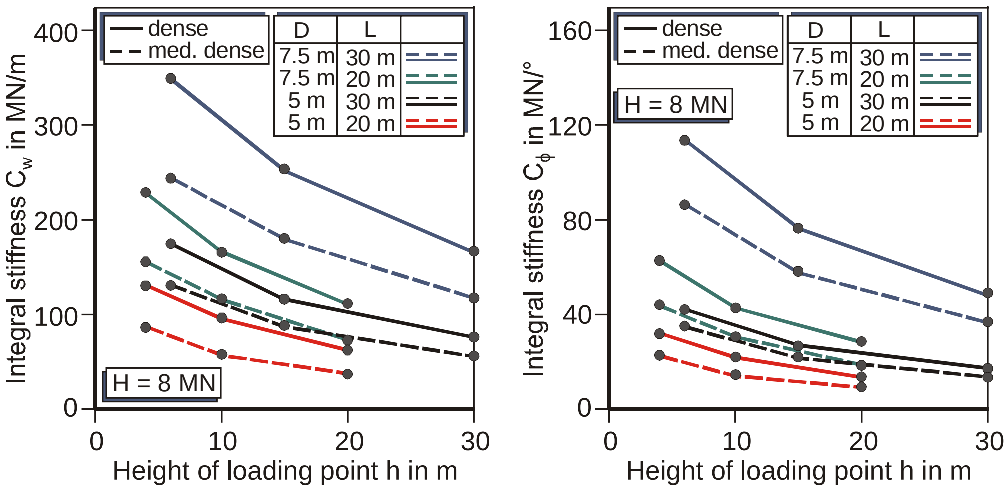


Figure 5. Integral stiffness diagrams for monopiles in dense and medium dense sand.

4 On the influence of quasi-static cyclic loading

Quasi-static cyclic loading means repeated loading which changes so slowly that inertia effects or development of excess pore pressure do not occur. It is a well-known fact that piles under horizontal cyclic loading experience an accumulation of deformations with increasing load cycles.

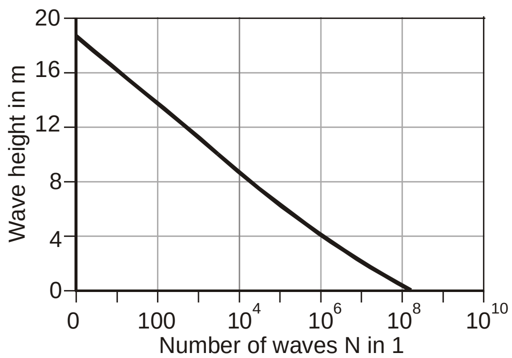
In the API method described in section 2, cyclic loading is accounted for by setting the factor A(z) to 0.9 (see Eq. 3). This approach has been developed by means of in situ tests, in which different load levels were applied and these loads were repeated not more than 100, mostly less than 50 times (Reese et al. 1974, see also Long & Vanneste 1994). It was believed that with subsequent load cycles no significant further displacement accumulation occurs.

In fact, cyclic accumulation does not end after 100 cycles. The accumulation rate decreases with the number of cycles (shakedown behaviour), but it does not get zero. There are several approaches to consider this behaviour, e. g. Hettler (1981), Long & Vanneste (1994). Lin & Liao (1999) give the following equation for the accumulation of the pile head displacement:

$$w_N = w_1 \left(1 + t \ln N\right) \tag{6}$$

Herein w is the displacement for static loading, N is the number of load cycles and w_N is the displacement after N cycles. t is a degradation parameter, which is besides others a function of soil properties and loading type (one-way or two-way loading). For one-way loading t is a value of the order of 0.20. This means for instance, that 1000 load cycles induce an increase of the pile head displacement of about 140%.

Loading of offshore monopiles is extremely cyclic, but it is of course not a one-way loading. The nature of cyclic loading shall be elucidated here for wave loading only. For a location with about 30 m water depth in the German North Sea Mittendorf et al. (2004) determined the wave heights and wave numbers occurring over a time-period of 12 years. The resulting wave sum curve and the wave directions are given in Figure 6. More than 100 million waves coming from all directions are acting on a structure. The maximum wave height is 18.5 m.



Direction and frequency of significant wave heights H_S

	Number of waves H_S=0-3m	H_S=3-6m	H_S=6->9m	Percentage
0-90°	13579	2000	67	45.1
90-180°	5035	74	10	14.7
180-270°	5574	49	1	16.2
270-360°	7977	347	10	24.0

Figure 6. Wave sum curve and wave directions for a location in the German North Sea, water depth 30 m (Mittendorf et al. 2004).

By means of the Morison formula waveloads on a monopile can be determined dependent on the wave height, the wave period and the pile diameter. This has been done for the wave sum curve given, differentiating 7 wave height classes (see Achmus et al. 2005). The results are shown in Figure 7.

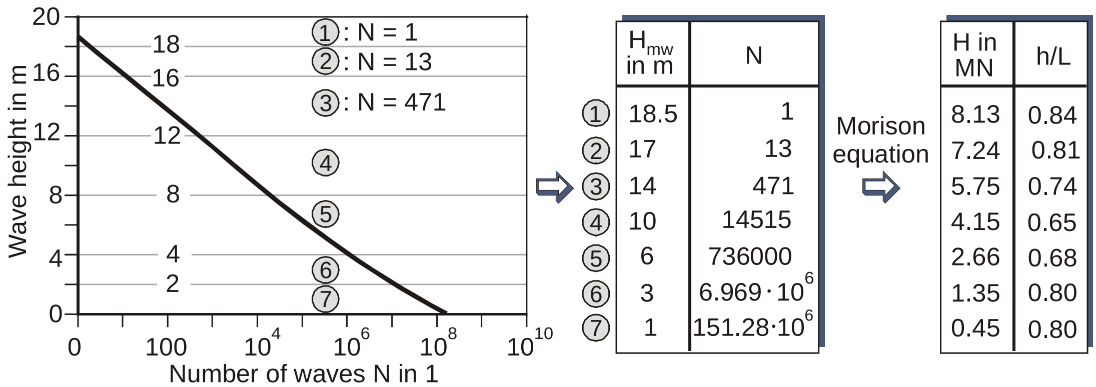


Figure 7. Waveloads on a monopile D = 7.5 m for the wave sum curve given in Fig. 6 (cf. Achmus et al. 2005).

Eq. (6) is valid only for a cyclic loading with constant load amplitude. The problem to be solved is to derive an equivalent number of design load cycles, which has the same effect as the actual loads with varying amplitudes.
Lin & Liao (1999) proposed the following equation to calculate equivalent load numbers N^*:

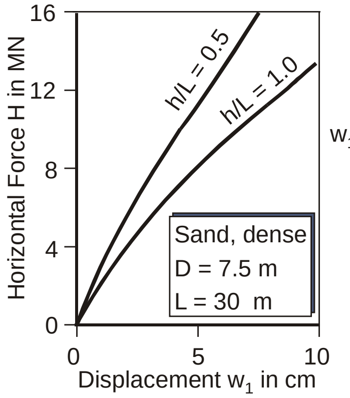
$$N_k^* = e^{\frac{1}{t}\left(\frac{w_{1,k}}{w_{1,1}}(1+t\ln N_k)-1\right)} \tag{7}$$

Herein $w_{1,1}$ is the static displacement under design load, $w_{1,k}$ is the static displacement under the load of different amplitude and N_k is the number of these loads. Summing over all loads of different amplitudes, the resulting cyclic displacement is

$$w_{Ges} = w_{1,1}\left[1+t\ln\left(N_1+\sum_{k=2}^{n} N_k^*\right)\right] \tag{8}$$

The static displacements for a given load H and height of loading point h can be determined using the force-displacement curves presented in section 3.

This method has been applied for the waveloads depicted in Fig. 7 and a monopile D = 7.5 m, L = 30 m embedded in dense sand. The results shown in Figure 8 indicate an increase of the static displacement for the maximum design load of about 43 %.



w_1 = f (H, h/L)

H in MN	h/L	w_1 in cm	N	$N^*_{(t=0.17)}$
8.13	0.84	4.53	1	1
7.24	0.81	3.86	13	3.73
5.75	0.74	2.85	471	5.42
4.15	0.65	1.97	14515	2.32
2.66	0.68	1.10	736000	0.31
1.35	0.80	0.52	$6.969 \cdot 10^6$	0.03
0.45	0.80	0.20	$151.28 \cdot 10^6$	0.01

$\Sigma N^* = 12.82$

$w_{Ges} = 4.53 \cdot (1 + 0.17 \cdot \ln 12.82)$
$= 1.43 \cdot 4.53 = 6.48$ cm

Figure 8. Exemplary application of Eq. (7) and (8) for a monopile D = 7.5 m, L = 30 m in dense sand (t = 0.17).

Of course, this method is not experimentally verified. For the example, all the loads of the 12 year-period acting in different directions were assumed to act in one direction. Thus, only a qualitative insight in the displacement accumulation process is obtained.
However, the results indicate that the influence of the lower wave heights is neglectable. Considering only the 485 waves with heights greater than 10 m, a displacement increase of 40% is obtained. The contribution of loads which induce a static displacement less than about 25% of the static displacement due to design load is of minor importance.

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 load design, numerical investigations are recommended, 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. However, also the numerical calculations need verification. Thus, the observational method should be used for the large-diameter monopiles to be erected.

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. At the time being no reliable method exists for the estimation of accumulated displacements under cyclic loads. The analysis procedures presented need verification and have to be extended to deal with loads acting in different directions. However, the results obtained indicate that only larger forces, which occur relatively seldom, have to be considered regarding the displacement accumulation.

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