Numerical Modelling of Large Diameter Steel Piles under Monotonic and Cyclic Horizontal Loading

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ABSTRACT: To make economic use of offshore wind energy possible, foundation structures with minimum costs, but sufficient stiffness have to be designed. One foundation concept which has recently often been realized in this field is the monopile concept. Here the foundation consists of one steel pile with a large diameter of up to or even more than 5 m. This paper aims to investigate the behaviour of such a monopile under monotonic and cyclic 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. A special numerical scheme was developed to account for the effect of cyclic loading. With that, based on results of cyclic triaxial tests, the increase of pile displacements with the number of loading cycles can be quantified.

1 INTRODUCTION

The offshore wind farms planned in the German part of the North and the Baltic Seas 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 subsoil. For this, monopile foundations can be used. In principle the monopile is an extension of the main tower into the soil under the sea bed (Fig. 1). Monopiles with diameters of up to 5 m have been constructed recently in water depths of up to 15 m. The application is expected to be possible for water depths of up to 25 to 30 m. The monopile diameters in such depths will be between 6 and 8 m.

Since wind energy plants are relatively sensitive to deformations, in particular tilting, it is very important to estimate these as exactly as possible. For the mentioned large-diameters piles, there is no approved design procedure yet. The p-y curve method given in offshore design regulations (API 2000, DNV 2004), which is usually used for the design of offshore piles, was developed for piles with maximum diameters of about 2 m. In this paper, the results of numerical investigations of the load-deformation behaviour of monopiles under static and cyclic loading are presented.

2 NUMERICAL MODELLING OF MONOPILE BEHAVIOUR UNDER STATIC MONOTONIC LOADING

For the investigation of the load-deformation behaviour of monopiles with large diameters, three-dimensional finite element calculations were made. An idealized homogeneous soil typical of North Sea conditions 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 in all calculations presented here.

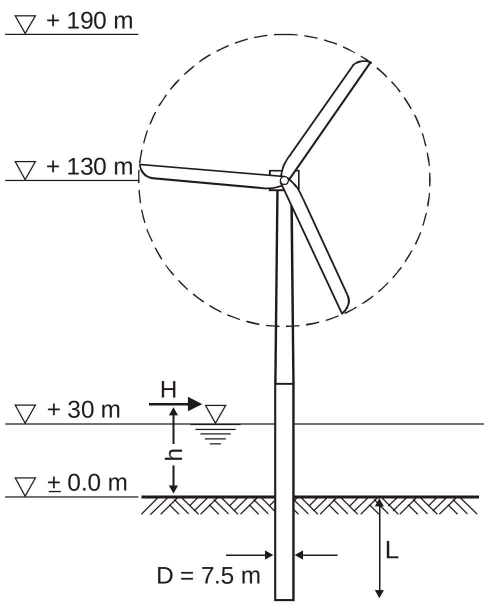

Figure 1. Example of a monopile foundation.

The finite element model is shown in Figure 2. The behaviour of the pile under a vertical load of 10 MN representing the structure's weight and a variable horizontal load H acting at a certain height h above the sea bed level was investigated. By variation of the load application height h, combinations of horizontal force H and bending moment $M = H \times h$ were realized.

Due to the symmetric loading condition only a half-cylinder representing the subsoil 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.

The computations were done with the program system ABAQUS (Abaqus 2005). In order to carry out numerous calculations for varying boundary and loading conditions, an advanced computer system with parallel processor technology was used to minimize the computation time.

For both the soil and 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 $\delta = 0.67\ \varphi'$. 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 interaction problems 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 stiffness modulus with the following equation:

$$E_S = \kappa\, \sigma_{at} \left(\frac{\sigma_m}{\sigma_{at}} \right)^{\lambda} \qquad (1)$$

Here $\sigma_{at} = 100$ kN/m^2 is a reference (atmospheric) stress and σ_m is the current mean principal stress in the considered soil element. The parameter κ determines the soil stiffness in 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. For more details about the numerical modelling reference is made to Abdel-Rahman & Achmus (2005).

The finite element calculation is executed in steps. 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 gradually. 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 great water depths, the horizontal (bedding) stresses acting on the pile in the symmetry plane are shown in Figure 2. A monopile with an embedded length of 30 m in dense sand is considered. The characteristic loading behaviour of the pile with bedding stresses of opposite direction above and below a point of rotation can be seen clearly. For the considered case the point of rotation lies at about 22 m below the sea bed.

In Figure 3, load-displacement curves are given for the same pile dimensions and soil conditions. As a characteristic value, the horizontal displacement at the sea bed level w is plotted versus the horizontal load H. For comparison, the results gained with the method given in API (2000) are also shown. Usually offshore foundation structures are designed with respect to the API regulations. Here the p-y method, which is a special subgrade reaction method, is recommended for the design of laterally loaded piles and p-y curves for different soil conditions are given. Figure 3 shows that the stiffness of large-diameter monopiles with respect to horizontal loading in sand is overestimated with this method.

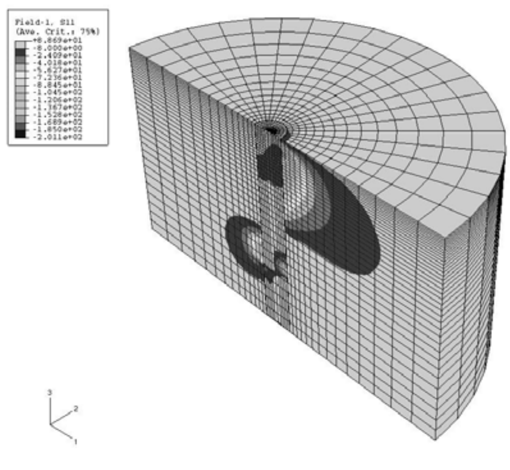

Figure 2. Finite element mesh and horizontal bedding pressure (in the plane of symmetry) for D = 7.5 m, L = 30 m, dense sand, H = 8 MN, h/L = 1.

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

	dense	medium dense
Unit buoyant weight γ'	11 kN/m^3	11 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	7.5°	5°
Cohesion c	0.1 kN/m^2	0.1 kN/m^2

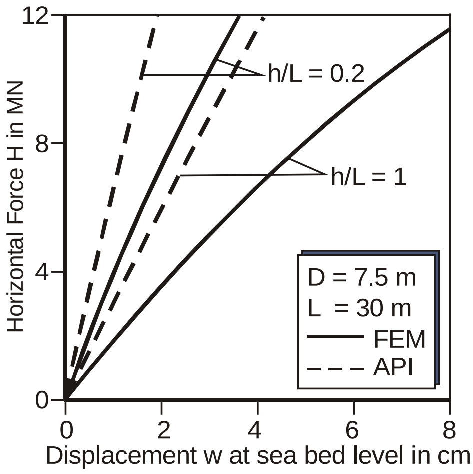

Figure 3. Load-displacement curves for D = 7.5 m, L = 30 m, dense sand.

3 EFFECT OF CYCLIC LOADING

During the lifetime of an offshore wind energy plant 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.

As regards 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. However, for monopiles of large diameter there is no approved method for estimating the permanent displacements due to cyclic loading.

In the API regulations (API 2000) mentioned above cyclic loading is taken into account by a decrease of soil spring stiffnesses up to a certain depth below the seabed. For large-diameter monopiles, this leads to moderate increases of the calculated displacements, which is elucidated by an example calculation in Figure 4.

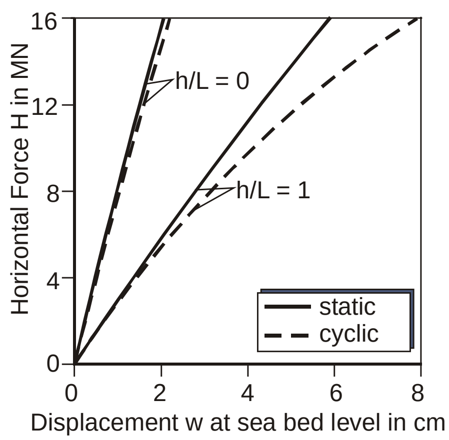

Figure 4. Comparison of load-deformation curves for a monopile D = 7.5 m, L = 30 m in medium dense sand for static and cyclic loading due to API (2000).

The API approach was derived by means of loading tests in which in most cases less than 100 load cycles were applied. In fact, strain accumulation does not stop on 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:

$$w_N = w_1 \left(1 + C_N \ln N\right) \tag{2}$$

Here w_1 is the lateral pile head displacement for static loading and C_N is a factor which lies in the range of 0.20 for sand.

Long & Vanneste (1994) proposed a subgrade reaction method with linear increasing subgrade modulus with depth, in which the spring stiffnesses decrease with the number of load cycles according to the following equation (see Fig. 5 top):

$$k_s(z) = N^{-t}\, n_{h,1}\, z \tag{3}$$

Here 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 Figure 5. It was found that after 5000 cycles the displacement at the pile head is more than 3 times the static displacement.

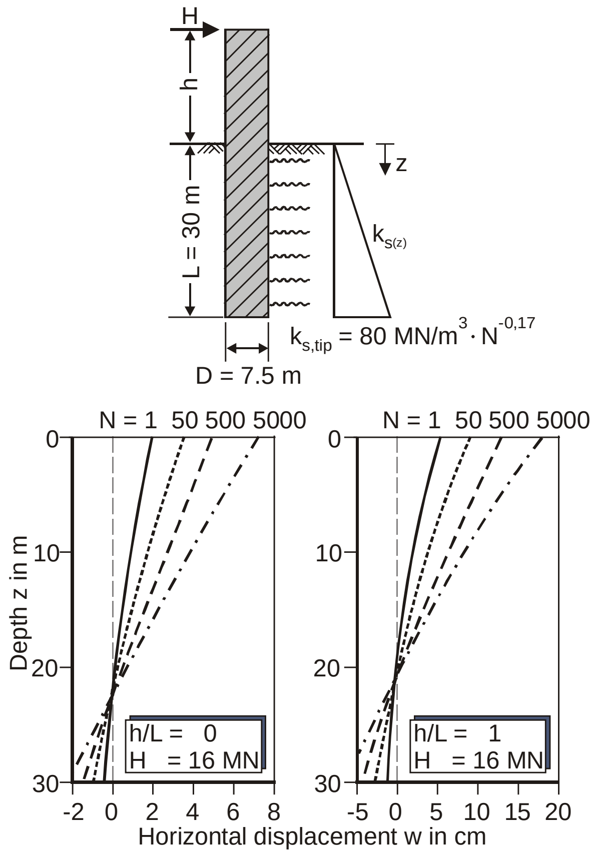

Figure 5. Influence of cyclic one-way loading according to the method from Long & Vanneste (1994) for a monopile D = 7.5 m, L = 30 m in medium dense sand.

However, this method has not yet been approved, and in particular for large-diameter piles under high bending moments. A method of analysis is needed in which the development of displacements for general boundary conditions can be calculated, based on the results of laboratory tests with the soil to be considered. A German standard for soil investigations for offshore wind energy converters (BSH 2003) suggests cyclic laboratory tests for the assessment of the behaviour under cyclic loads. Such a method is outlined in the following.

4 STIFFNESS DEGRADATION MODEL FOR THE SIMULATION OF DISPLACEMENT ACCUMULATION

In the following, a practical method for the simulation of displacement accumulation is presented. The method is based on finite element calculations, in which the stiffnesses of the elements are adopted with respect to the number of cycles, to the stress state and to material parameters determined in cyclic triaxial tests.

4.1 *Stiffness degradation in cyclic triaxial tests*

A schematic sketch of the results of a stress-controlled cyclic triaxial test under drained conditions is shown in Figure 6. The plastic portion of the axial strain ε_p^a increases with the number of load cycles. The quantity of the increase is mainly dependent on the initial stress state (confining stress) and on the magnitude of the cyclic load portion.

The strain increase can be interpreted as a decrease of the secant stiffness modulus. When the elastic strain is negligible and the initial deviatoric stress is much lower than the cyclic deviatoric stress, the degradation of the secant modulus E_{sN} can be formulated in the following way dependent on the plastic strain in the first cycle $\varepsilon_{p,N=1}^a$ and in the Nth cycle $\varepsilon_{p,N}^a$:

$$\frac{E_{sN}}{E_{s1}} \cong \frac{\varepsilon_{p,N=1}^a}{\varepsilon_{p,N}^a} \tag{4}$$

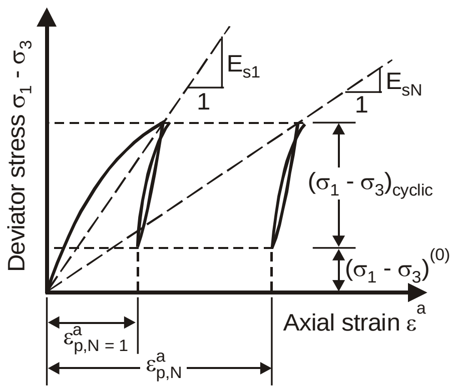

Figure 6. Degradation of secant modulus under cyclic loading in a drained triaxial test.

Equations to describe the development of plastic strains in the cyclic triaxial tests were proposed, for instance, by Huurman (1996), Gotschol (2002) and Werkmeister (2004). According to Huurman's approach used here, the increase of deformation or the decrease of stiffness, respectively, can be described by the following equation:

$$\frac{E_{sN}}{E_{s1}} = \frac{\varepsilon_{p,N=1}^a}{\varepsilon_{p,N}^a} = N^{-b_1(X)^{b_2}} \tag{5}$$

Here N is the number of cycles, X is a stress-dependent variable (cyclic stress ratio), and b_1, b_2 are regression parameters to be determined in triaxial tests.

The cyclic stress ratio is defined as

$$X = \frac{\sigma_{1,cyclic}}{\sigma_{1,f}}, \tag{6}$$

wherein $\sigma_{1,f}$ is the main principal stress at failure in a monotonic test. Thus, the stress ratio is dependent on the initial stress state (confining stress) and on the cyclic load level.

4.2 *Application of the stiffness degradation model for the pile-soil system*

The procedure for the consideration of the stiffness reduction for the pile-soil system is illustrated in Figure 7. In a first step, the loading of the system by the weight of the tower structure and the soil is analyzed. With that, the initial stress state for each element of the system is gained.

In the second step the system behaviour under horizontal load is analyzed, using the stress-dependent soil stiffnesses valid for monotonic loading. Thus, the first and the second step are identical to the calculation of the behaviour of the pile under monotonic loading.

In the third calculation step, the horizontal load is applied again, but the stiffnesses of the soil elements are adopted dependent on their stress history with respect to Equation 5. With that, for any number of load cycles N the system behaviour can be calculated by a re-calculation of the second step using appropriate stiffness values.

A problem in the application of the stiffness degradation model for the pile-soil system is a reasonable definition of the initial stress state and the cyclic load level for each element of the discretized system. On one hand, the initial stress state is not isotropic, and on the other hand the principal stress orientation changes during loading and the lowest principal stress does in general not remain constant.

To overcome these problems, the following scheme was chosen to derive a characteristic cyclic stress ratio. For the initial stress state (Index $^{(0)}$) as well as for the state under action of the cyclic load (Index $^{(1)}$) a stress ratio X_a is calculated:

$$X_a^{(0)} = \frac{\sigma_1^{(0)}}{\sigma_{1,f}^{(0)}} \quad , \quad X_a^{(1)} = \frac{\sigma_1^{(1)}}{\sigma_{1,f}^{(1)}} \tag{7}$$

The cyclic stress ratio X is then calculated by

$$X = \frac{X_a^{(1)} - X_a^{(0)}}{1 - X_a^{(0)}} \tag{8}$$

This parameter X characterizes the increase of the stress level in each element under cyclic load and can thus be used for the determination of the stiffness decrease with Equation 5. Values $X < 0$, which may arise due to a decrease of deviatoric stress from the initial to the loaded state are not taken into account, i.e. in such cases the soil stiffness remains unchanged.

The degradation parameters b_1 and b_2 in Equation 5 have to be determined in a series of cyclic triaxial tests. If different soil layers exist, this can easily be accounted for by different sets of the parameters b_1 and b_2.

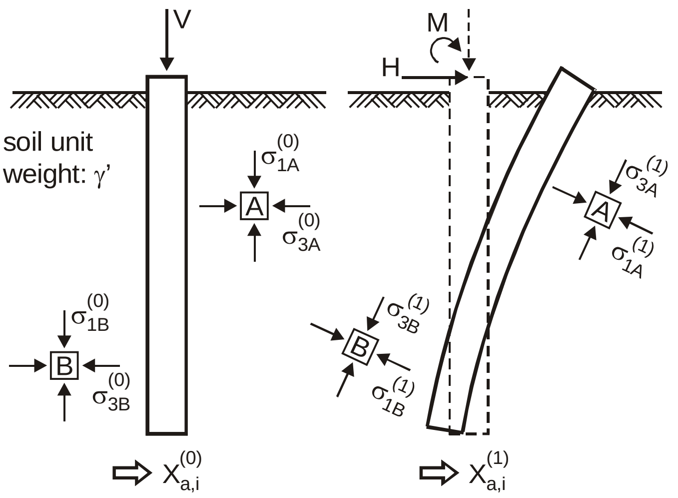

Figure 7. Initial stress state and stress state under cyclic load for the pile-soil system.

4.3 *Simulation results*

In Figures 8 and 9 results obtained with the stiffness degradation model are presented. Again a monopile with a diameter of 7.5 m and an embedded length of 30 m in dense sand (Fig. 8) and medium dense sand (Fig. 9) was considered.

The regression parameters b_1 and b_2 were determined based on several triaxial tests with sand samples reported in the literature (Morgan 1966, Timmerman & Wu 1969, Gaskin et al. 1979, Addo-Abedi 1980, Thiel 1988, Huurman 1996, Gotschol 2002 and Wichtmann 2005). From that, possible parameter combinations were found to be:

- Dense sand: $b_1 = 0.20$, $b_2 = 5.76$;
- Medium dense sand: $b_1 = 0.16$, $b_2 = 0.38$.

A significant increase in both head and toe displacement with cyclic loading of the monopile is found. With increasing displacements, also a slight downward movement of the rotation point (zero deflection point) of the pile occurs.

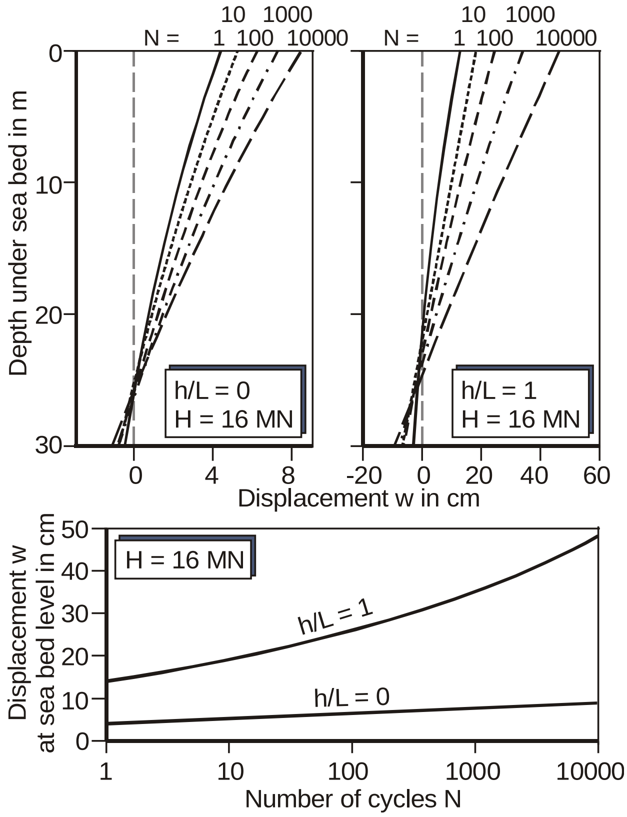

Figure 8. Results for displacement accumulation of a pile in dense sand.

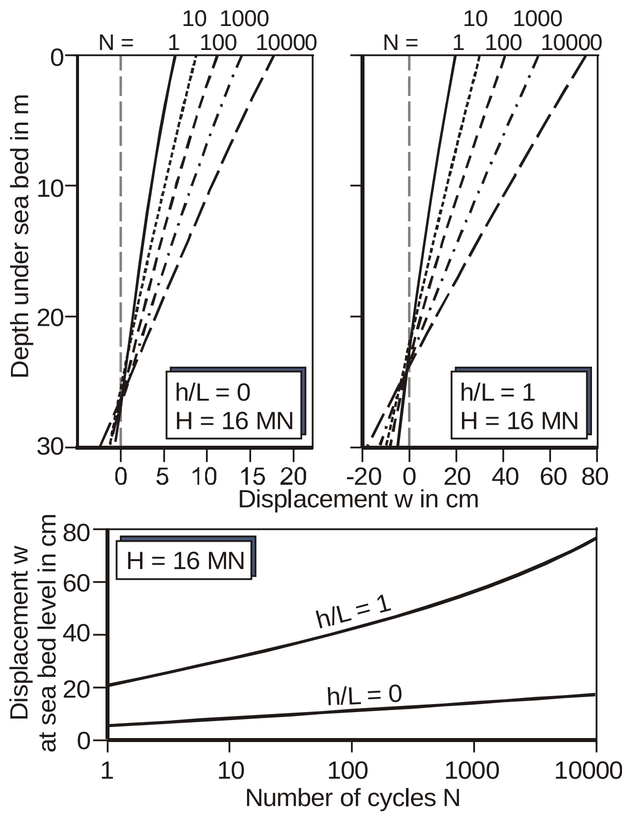

Figure 9. Results for displacement accumulation of a pile in medium dense sand.

In Figure 10 the relative increase w_N/w_1 is given for the four systems considered and compared with Hettler's approach given in Equation 2. Tendentially the Hettler approach is confirmed. However, the model results show that the rate of deformation increase is not constant, but dependent on the system.

For medium dense sand the rate is higher than for dense sand and for *h/L* = 1 it is higher than for *h/L* = 0. Thus it seems that the rate of accumulation is dependent on the relative load level, i.e. the ratio of the actual to the ultimate load of the system.

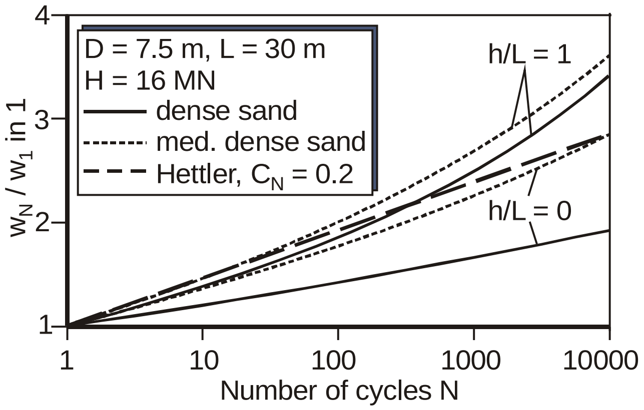

Figure 10. Results for the relative increase of the lateral pile displacement at sea bed level.

5 CONCLUSIONS

The accumulation of displacements of monopiles under cyclic loading needs to be taken into account in the design. The proposed method is based on numerical simulations in combination with an evaluation of cyclic triaxial tests. It is in principle also applicable for cohesive soil or layered subsoil and thus has the potential to carry out a site-specific design. Although the numerical results of course need further verification, the first results gained are promising. The model can at least be used to carry out parametric studies to identify important parameters affecting the system behaviour under cyclic loads.

A remaining question is the number of cycles of the design load to be considered in the design. In reality, the amplitude and the direction of the load varies with the wave heights and the wind velocities, and the maximum load is to be expected only once during the lifetime of the structure. For loads of varying amplitudes Lin & Liao (1999) proposed a strain superposition method to determine the substitute number of design load cycles characterizing the effect of the whole load history. For that, the number of cycles and the static pile displacement at sea bed level for each load amplitude has to be determined, which can be done with the numerical model presented. However, the effect of the varying load directions on the monopile behaviour is an open question, which will be the subject of further research.

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