# Prediction of long-term deformations for monopile foundations of offshore wind power plants

Torsten Wichtmann
Andrzej Niemunis
Theodor Triantafyllidis
*Institute of Soil Mechanics and Rock Mechanics, University of Karlsruhe*

ABSTRACT: The foundations of offshore wind power plants (OWPPs) are subjected to a high-cyclic loading due to wind and waves. No suitable methods for the prediction of the long-term deformations of these foundations do exist so far. The paper discusses the application of a high-cycle accumulation (HCA) model for this purpose. Finite element calculations of OWPP monopile foundations are presented and necessary modifications of the HCA model are discussed.

## 1 INTRODUCTION

A large number of offshore wind parks is planned to be constructed in the vicinity of the German coast of the North Sea during the next years. Due to the large dimensions of these offshore wind power plants (OWPPs) and due to the large horizontal cyclic loading the design of the foundation of these OWPPs is not a trivial problem.

Offshore wind power plants may be founded shallow or on suction caissons. Alternatively, a solution with one single large pile (monopile, diameter usually $>$ 5 m) and structures with three or four smaller piles (tripod, jacket) can be designed. The present paper concentrates on monopiles. For shallow foundations the reader is referred to Sturm et al. [15].

The loading of the foundation of an OWPP is composed of an aerodynamic portion due to wind and a hydromechanical portion due to current and waves. During its lifetime an OWPP is subjected to a few load cycles with large amplitudes (due to strong storms) but also to many (millions or billions of) cycles with small to intermediate amplitudes (so-called high-cyclic or poly-cyclic loading).

Both the large and the small cycles may cause permanent deformations. Even tiny deformations may accumulate if the number of cycles is high. The serviceability of the structure may get lost due to an excessive tilting of the tower. It is also possible that extreme storm events cause large deflections of the monopile accompanied by a local dilatation in the soil. Subsequent small cycles may lead to a recompaction of the soil and in some cases even to a decrease of the lateral deflections of the monopile (so-called ”self-healing” effect).

The cyclic loading may also cause a change of the lateral bedding of the pile due to a horizontal stress relaxation. For a constant load amplitude this would mean larger displacement amplitudes, which could also endanger the serviceability of an OWPP. If the drainage is poor (low permeability, large amplitudes) a temporary increase of the pore water pressure during a cycle or even an accumulation of the pore water pressure may occur reducing the effective stress and thus the stiffness of the soil. This could also lead to larger displacement amplitudes. In the extreme case a "liquefaction" of the soil may occur, i.e. the effective stress and thus the shear strength almost vanishes. In that case the bearing capacity of the OWPP foundation would dissapear.

Engineers have little experience concerning the long-term behaviour of OWPP monopile foundations. Existing offshore wind parks lay usually near the coast in relatively shallow waters (water depth $<$ 10 m). The water depths of the new parks planned in the vicinity of the German coast of the North Sea will be up to 30 m, i.e. the lever arm of the horizontal loads and thus the bending moments at the seabed level will be much larger than for the existing structures. Furthermore, the new OWPPs will have larger dimensions than the present ones due to power generation requirements. This will further increase the loads. Methods for estimating the bearing capacity or the serviceability developed for smaller pile diameters and lower loads cannot be directly adopted to the new OWPPs. In comparison to conventional offshore foundations (e.g. oil rigs) the water depths and the loads of OWPPS are usually smaller but the ratio of the horizontal to the vertical loads is significantly larger. Thus, experiences gained from existing offshore or onshore wind power plants and other offshore structures cannot be directly adapted.

During the past years the authors have developed a high-cycle accumulation (HCA) model (Niemunis et al. [10]). The model predicts permanent deformations or excess pore water pressures in non-cohesive soils due to many cycles ($N > 10^3$) with small to intermediate amplitudes (so-called high- or polycyclic loading). It is based on an extensive laboratory testing program on sand (Wichtmann et al. [19]). The "explicit" strategy for FE calculations (in contrast to the conventional "implicit" one) consists in treating packages of cycles in a single calculation step, cf. Niemunis et al. [10].

The present paper discusses only an application of the HCA model to predict long-term deformations of OWPP monopile foundations. It presents some first finite element calculations (Section 3) and addresses necessary modifications / extensions of the HCA model (Section 4).

## 2 LITERATURE REVIEW

OWPPs may be designed using the general geotechnical standard codes (e.g. DIN 1054 [3]), special guidelines (e.g. Germanischer Lloyd [8], DIBT [5]) or rules for conventional offshore structures (e.g. API [4], DNV [17]). However, recommendations on the serviceability of piles under lateral cyclic loading are hardly to be found in these documents.

For the estimation of deformations of piles under lateral monotonous loading DIN 1054 [3] recommends test loadings which are difficult to perform offshore. Recommendations how to consider a lateral cyclic loading are missing in DIN 1054 [3]. An application of the "observation method", i.e. a combination of predictions and in-situ measurements (at all OWPPs of a wind park) would be technically and economically laborious and the fast application of counter-measures cannot be guaranteed (Richwien [14]).

The guidelines of "Germanischer Lloyd" [8] demand an investigation of the effect of cyclic loading. Both the short-term and the long-term soil-structure interaction should be examined. However, the methods and the extent of such investigations are not further specified. Concerning the long-term behaviour, it is stated that there exist neither a theory nor established investigation methods. It is recommended to utilize the experience from similar projects in the past, which is missing in the case of the new OWPPs. As a possible approach the adherence of given maximum deflections under a static equivalent load is mentioned but judged as possibly nonconservative. In the guidelines of [8] it is further mentioned that no established method exists for an estimation if the soil around an OWPP monopile will liquefy or not.

The recommendations of the American Petroleum Institute (API) [4] use the model of an elastically supported beam with non-linear springs for the lateral load-deflection. The relationship between the lateral stress $p$ and the lateral displacement $y$ is addressed as "$p$-$y$-curve".

In [4] functions $p(y)$ are given in dependence on depth, soil density, friction angle, pile diameter and type of loading (static or cyclic). A cyclic loading is considered by reducing $p$ by a constant factor 0.9 (according to Reese et al. [12]). The applicability to the large diameters of the new OWPPs is not confirmed yet. The consideration of a cyclic loading by the factor 0.9, independent of amplitude, number of cycles and soil conditions, seems to be oversimplified.

The Offshore Standard DNV-OS-J101 (Det Norske Veritas [17]) also proposes $p$-$y$-curves. The cyclic loading should be considered by ”suitable” $p$-reduction factors, which are not further specified. Concerning the serviceability limit state a calculation of the cumulative deformations in the soil in a ”suitable manner” is demanded. However, no such method is specified.

Some extensions of $p$-$y$-curves by the number of cycles $N$ were proposed in the literature (Swinianski & Sawicki [16], Welch & Reese [18]). The latter ones e.g. recommended the relationship $y(N) = y_s + y_{50}\ C\ \log(N)$ for a dry, stiff clay. Therein $y_s$ and $y_{50}$ are the deformation at 100 % or 50 % of the bearing capacity, respectively, and the factor $C$ considers the amplitude. However, these modified $p$-$y$-curves were not developed and are thus not suitable for the large pile diameters and loading conditions of the new OWPPs.

Little & Briaud [7] proposed an estimation of the lateral deformations of piles in sand due to cyclic lateral loading by means of $p$-$y$-curves developed on the basis of pressuremeter tests with monotonous and cyclic loading. From a cyclic test (Figure 1) the ”secant shear stiffness” $G$ is obtained as a function of the number of cycles. The degradation of $G$ with $N$ is described by $G(N) = G(N = 1)\ N^{-a}$. The lateral deformations of the pile after $N$ cycles can be estimated by using a modification of the $p$-$y$-curve from the monotonous test: $y(N) = y(N = 1)\ N^a$. The method could be approved by re-calculations of model tests. However, its application to large pile diameters is not confirmed yet and the extrapolation of the pressuremeter data to large $N$-values seems doubtful.

Grabe et al. [6] studied changes of the pore water pressure in the soil near a monopile foun-

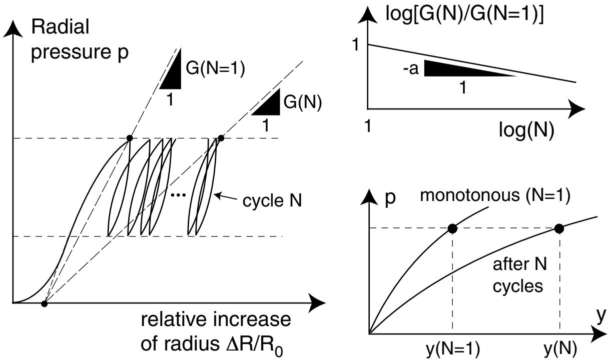


Figure 1. *Modified $p$-$y$-curve obtained from pressuremeter tests as proposed by Little & Briaud [7]*

dation. In model tests (scale 1:100) with a fine to medium coarse sand (medium dense to dense) no accumulation of pore water pressure was measured during 12 cycles with a relatively large displacement amplitude applied with a frequency of 1 Hz. However, since the coefficient of permeability was not scaled, increased pore pressures around the pile could be underestimated. In 3-D coupled FE calculations (20 cycles) using the hypoplastic constitutive model with intergranular strain (Niemunis & Herle, [9]), only for low permeabilities of the sand ($k = 10^{-5}$ m/s) an accumulation of pore water pressure was observed. Grabe et al. [6] state that their implicit calculations should not be extrapolated to larger numbers of cycles and that the problem of the long-term deformations is not solved yet.

Richwien et al. [13] propose a two-step concept for the design of OWPPs. First, both the bearing capacity and the serviceability are proven for an extreme storm event (equivalent static load). The use of $p$-$y$-curves for large pile diameters based on elasto-plastic FE calculations is recommended. Next, a possible accumulation of pore water pressure $u$ during the operation or due to a storm has to be checked by calculating a small number of cycles implicitly using a coupled FE analysis. If no accumulation of u takes place the permanent deformations are the main concern. It has to be proven that the ”cyclic stress ratio” $CLR = 2H^{\text{ampl}}/Q_s$ with the bearing capacity $Q_s$ does not exceed a critical value $CLR_{\text{crit}}$. It can be determined e.g. from cyclic laboratory tests with a step-wise increased amplitude. In the case $CLR < CLR_{\text{crit}}$ the deformations during the lifetime of the OWPP have to be determined. A suitable method for this prediction has not been proposed by Richwien et al. [13], however.

## 3 FE CALCULATION

For the numerical calculation the geometry, loading and soil conditions of a real OWPP project have been supplied by ”Germanischer Lloyd Wind Energie GmbH”. The inner and outer diameter of the steel pipe (pile) are $d_i$ = 5.00 m and $d_a$ = 5.09 m, respectively. The depth of embedding is 32.65 m. An erosion of the upper 3 m of the soil is likely to occur, thus only the lower 29.65 m were considered.

The FE calculations were performed with the program ABAQUS. The 3D-FE discretisation is shown in Figure 2. Changes of the polarization of the loading have not been considered yet, i.e. the symmetry could be utilized and only one half of the problem was modelled. In order to choose an appropriate type of elements for the pile, preliminary comparative calculations of the pure steel pipe subjected to a bending moment were performed. Since calculations with continuum elements C3D8 and shell elements CC8R delivered similar deformations (no shear locking), continuum elements were used for modelling the pile. Elements with reduced integration C3D8R were used for the whole model in order to reduce the accumulation of unintentional self-stresses in the elements (Niemunis et al. [10]). Ten layers of elements were chosen along the depth of embedding and two elements were used for the thickness of the pipe. The soil was modelled within the radius of 20 m around the pile shaft and up to a depth of 20 m below the pile tip. The pile was modelled up to 1 m above the seabed. The Young's modulus of the steel pipe was $E = 2.1 \cdot 10^8$ kPa except the upper 1 m where it was chosen as $E = 10^{10}$ kPa in order to distribute the concentrated loads applied to the head of the pile. A Mohr-Coulomb frictional contact with $\mu$ = 0.3 was used at the inner and outer side of the steel pipe.

Soil sampling at the location of the OWPP shows that the sand layers reach up to a depth of $z$ = 27.6 m. It is a fine to medium coarse sand with different fractions of coarse sand and fine gravel. In the FE calculations the material con-

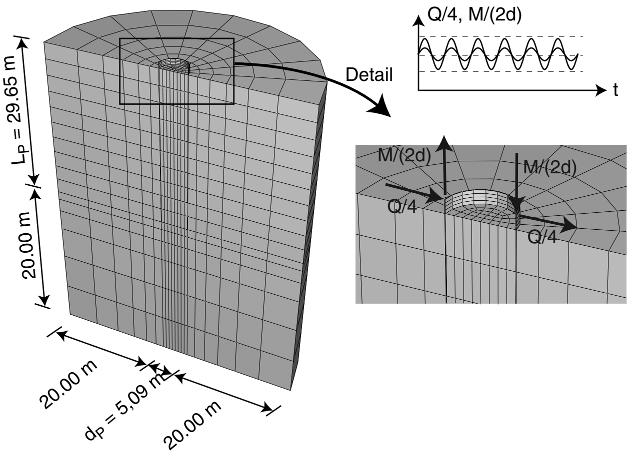


Figure 2. *FE discretization of the monopile foundation*

stants for a similar fine sand (sand ”CFS” in Tables 7.1 and 7.2 given by Wichtmann [19]) were considered for the HCA model and for the hypoplastic model which is used for the implicit calculation of the first two cycles. The sand layers are underlain by a stiff over-consolidated clay layer. Since a HCA model for clay is still under development, for the time being the clay layer has been modelled with the constants for the fine sand.

A cone penetration test provided the tip resistance $q_c(z)$. Using the relationship $I_D = -0.33 + 0.73\ \lg(q_c[\text{MPa}])$ from DIN 4094 [1, 2] the profile $I_D(z)$ given in Figure 3a was obtained (actually this correlation $I_D(q_c)$ is only applicable to sands above the ground water table; however, no correlations for saturated soils are given in DIN 4094). $I_D$ increases approximately linearly from 0.5 at the seabed to 0.9 in a depth $z$ = 30 m. $I_D$-values obtained from an analysis of available SPT data using the correlation $I_D = 0.18 + 0.370\ \lg(N_{\text{SPT}})$ (DIN 4094, applicable also to sands below the ground water table) were similar to the CPT-based values. For the FE calculation the profile $I_D(z)$ was approximated as shown in Figure 3b. Using $e_{\min}$ = 0.575 and $e_{\max}$ = 0.908 as determined for the fine sand in the laboratory, the profile of the initial void ratio $e_0(z)$ given in Figure 3b was obtained. The initial lateral stress $\sigma_{h0}$ was calculated from the vertical one assuming $\sigma_{h0}$ = $K_0\ \sigma_{v0}$ with a coefficient $K_0$ = 0.5.

For the fatigue analysis of the tower the irregular cyclic loading is grouped into packages

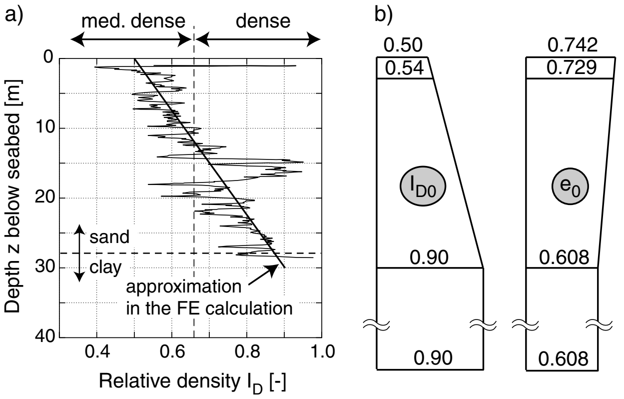


Figure 3. *Profi le $I_D(z)$ estimated from cone penetration test data*

of cycles each with a constant amplitude and a constant average value. Figure 4 presents such a (simplified) loading scheme with 19 packages ordered with increasing amplitudes. Figure 4 shows the bending moment $M$ in the pile in the height of the seabed. A similar loading scheme is available for the shear force $Q$. The number of cycles usually decreases with increasing amplitude of the packages. Due to its preloading variable $g^A$ (Niemunis et al. [10]), the HCA model can handle subsequent packages of cycles with different amplitudes almost independently of their sequence (Miner's rule). Generally, it has to be checked if the loading proposed for the fatigue analysis of the tower is useful for studies of the long-term deformations of the OWPP foundations.

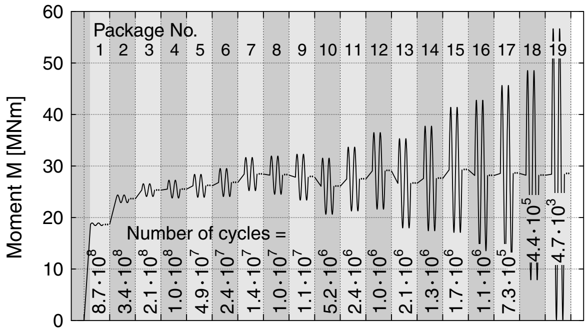


Figure 4. *Loading scheme: Bending moment in the monopile in the height of the seabed*

For the present preliminary FE study an application of the loading scheme in Figure 4 was not thought to be meaningful due to two different reasons. First, the HCA model is based on tests with $N \leq 2 \cdot 10^6$ cycles and has still to be confirmed for larger numbers of cycles (Section 4). For many of the packages in Figure 4, $N \gg 2 \cdot 10^6$ holds. Second, a numerical problem related to very large numbers of cycles (unintentional accumulation of self-stresses, Section 4) has still to be resolved. Thus, the calculations were restricted to $N \leq 2 \cdot 10^6$ cycles and to a constant amplitude.

In the FE model the bending moment was applied as a pair of vertical forces, Figure 2. The horizontal load acting on the pile was also equally distributed to these two nodes.

In the FE calculation first the self weight of the soil with the geostatic initial stress was applied (without generating deformations). In a second step the vertical force $V$ = 9247 kN due to the weight of the tower was applied to the head of the pile in form of a distributed load. In the third step the average values of the moment ($M^{\text{av}}$) and the shear force ($Q^{\text{av}}$) were applied. In step four an implicit calculation of the first cycle was performed. The loading was applied sinusoidal with the amplitudes $M^{\text{ampl}}$ and $Q^{\text{ampl}}$. The fifth step consisted of the calculation of the second cycle. It was equal to the fourth step except for the strain path that was recorded in each integration point. At the beginning of the sixth step the strain amplitude (an input parameter for the HCA model) was calculated from the recorded strain path followed by an explicit calculation of the subsequent cycles of the package.

Figure 5 presents the results of calculations with $N = 10^6$ cycles and different amplitudes $10 \leq M^{\text{ampl}} \leq 20$ MNm and different average values $20 \leq M^{\text{av}} \leq 40$ MNm of the bending moment. The ratio $Q/M$ = 0.027 1/m was kept constant. Obviously, the lateral displacement and thus the tilting of the pile increases with increasing number of cycles. The accumulated displacements increase with increasing amplitude $M^{\text{ampl}}$ (compare Figure 5a-c) and with increasing average value $M^{\text{av}}$ (compare Figure 5b,d,e).

A possible oscillation or accumulation of pore water pressure has not been considered yet. A coupled analysis with special consolidation elements should be performed for this purpose.

## 4 NECESSARY IMPROVEMENTS OF THE HCA MODEL

The HCA model is based on tests with $N \leq 2 \cdot 10^6$ cycles. OWPPS are subjected to much

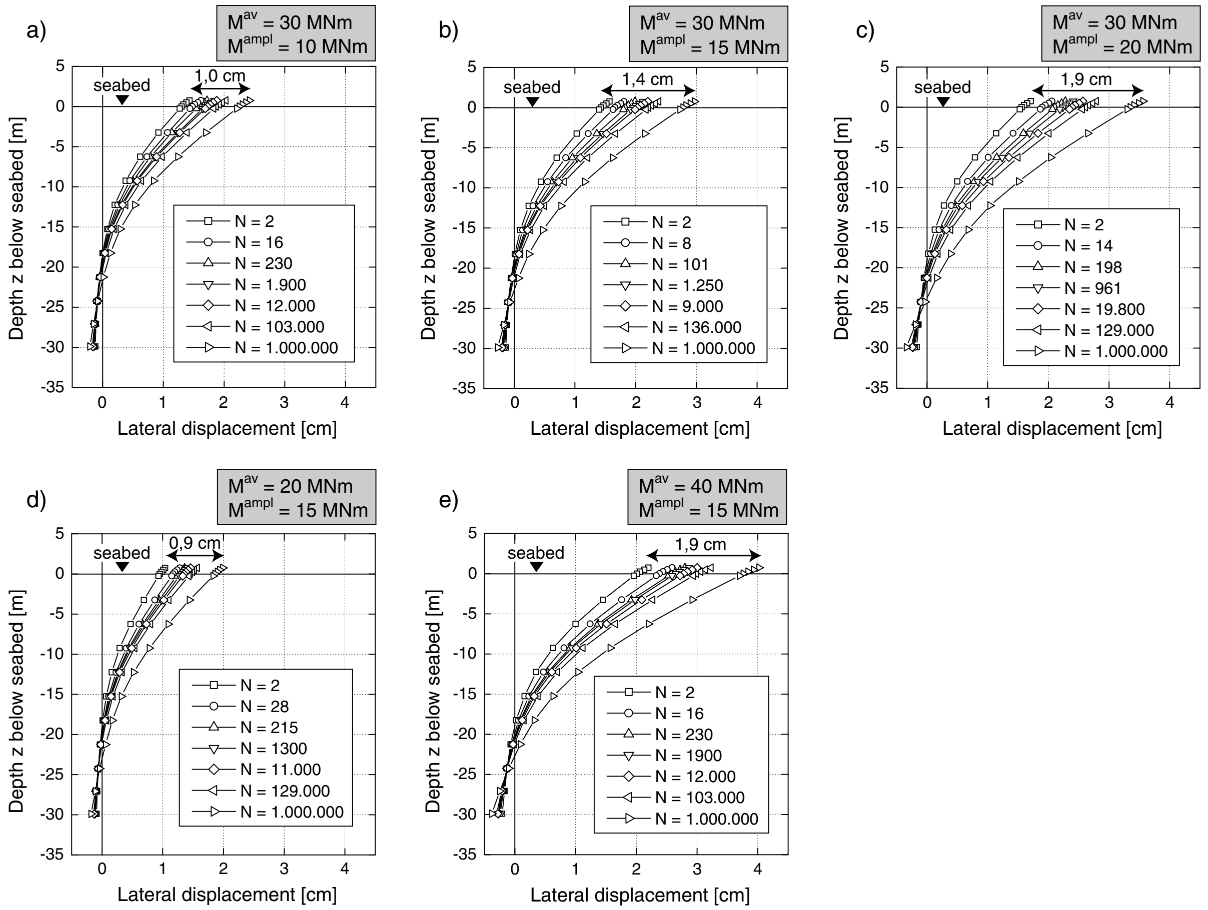


Figure 5. *Results of the FE calculations: Horizontal displacement of the monopile as a function of depth below seabed, for different amplitudes and average values of the bending moment*

larger numbers of cycles (Figure 4). It is likely that the constant portion of the function which describes the influence of the cyclic preloading in the HCA model (function $\dot{f}_N$ ) overestimates the accumulation for $N > 2 \cdot 10^6$. Thus, the HCA model, in particular its function $\dot{f}_N$ or its material constants have to be confirmed or modified for very large numbers of cycles.

For a realistic prediction of the long-term deformations of OWPPs the changes of the polarization of the cycles due to the variation of the direction of wind and wave loading has to be considered. Results from multiaxial direct shear tests (Wichtmann [19]) show that a sudden change of the polarization leads to a temporary increase of the accumulation rate. This effect is captured by the function $f_\pi$ of the HCA model. However, it has still some deficiencies (Niemunis et al. [11]). Based on the test results the function $f_\pi$ of the HCA model will be further developed.

Significant changes of stress due to the cyclic loading may be expected in the vicinity of the monopile foundation. In the FE calculation the stress relaxation depends on the elastic stiffness E in the basic equation $\dot{\boldsymbol{\sigma}} = \mathsf{E} : (\dot{\boldsymbol{\varepsilon}} - \dot{\boldsymbol{\varepsilon}}^{\text{acc}} - \dot{\boldsymbol{\varepsilon}}^{\text{pl}})$ of the HCA model. In the FE calculations presented in Section 3, E was chosen according to the recommendations given by Wichtmann et al. [20]. As outlined by Wichtmann et al. [20] further tests on E are necessary.

For the purpose of scour protection stones or ballast will be placed around the pile near the seabed. In order to consider the scour protection in the FE model the constants of the HCA model for such coarse materials have to be determined in large-scale cyclic triaxial tests.

## 5 SUMMARY, CONCLUSIONS AND OUTLOOK

The paper presents FE calculations of a monopile foundation of an offshore wind power plant. The permanent deformations after $10^6$ cycles are predicted using a high-cycle accumulation (HCA) model. The lateral deflections of the monopile increase with increasing amplitude and with increasing average value of the cyclic loading. Necessary modifications of the HCA model are discussed in Section 4.

An open question is also the determination of the cyclic preloading $g^A$ of the in-situ soil (Wichtmann [19]).

Furthermore, a numerical problem has to be solved. Due to an insufficient number of degrees of freedom per element in comparison to the number of stress or strain components prescribed in the integration points, unintended self-stresses may accumulate. The problem may be alleviated by using elements with reduced integration. However, methods for a more efficient reduction of these self-stresses are desirable.

Having improved the HCA model, the FE predictions have to be confirmed by recalculation of model tests and in-situ measurements. FE parametric studies will follow. Based on the parametric studies a simplified procedure for the estimation of the long-term deformations of OWPPs will be developed.

## 6 ACKNOWLEDGEMENTS

This work has been done in the framework of the project ”Geotechnical robustness and self-healing of foundations of offshore wind power plants” founded by the German Federal Ministry for the Environment, Nature Conservation and Nuclear Savety (BMU). The authors are grateful to BMU for the financial support. The provision of data for real offshore structures by ”Germanischer Lloyd” is appreciated.

## REFERENCES

[1] *DIN 4094:1990. Baugrund; Erkundung durch Sondierungen*, 1990.

[2] *DIN 4094:1990. Beiblatt 1. Baugrund; Erkundung durch Sondierungen; Anwendungshilfen, Erklärungen*, 1990.

[3] *DIN 1054:2005. Baugrund; Sicherheitsnachweise im Erd und Grundbau*, 2005.

[4] API American Petroleum Institute. Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms - Working Stress Design. API RP 2A - WSD, 20. Auflage, Dallas, 1993.

[5] Berlin Deutsches Institut für Bautechnik (DIBT). Richtlinie fr Windenergieanlagen, Einwirkungen und Standsicherheitsnachweise fr Turm und Grndung. Reihe B, Heft8, 2004.

[6] J. Grabe, J. Dührkop, and K.-P. Mahutka. Monopilegründungen von Offshore-Windenergieanlagen - Zur Bildung von Porenwasserüberdrücken aus zyklischer Belastung. *Der Bauingenieur*, 79:418–423, 2004.

[7] R.L. Little and J.-L. Briaud. A pressuremeter method for single piles subjected to cyclic lateral loads in sand. Technical Report GL-88-14, US Army Corps of Engineers, 1988.

[8] Germanischer Lloyd. Rules and Guidelines, IV Industrial Services, 2 Guideline for the Certification of Offshore Wind Turbines, 6 Structures, 2005.

[9] A. Niemunis and I. Herle. Hypoplastic model for cohesionless soils with elastic strain range. *Mechanics of Cohesive-Frictional Materials*, 2:279–299, 1997.

[10] A. Niemunis, T. Wichtmann, and T. Triantafyllidis. A high-cycle accumulation model for sand. *Computers and Geotechnics*, 32(4):245–263, 2005.

[11] A. Niemunis, T. Wichtmann, and Th. Triantafyllidis. On the definition of the fatigue loading for sand. In *International Workshop on Constitutive Modelling - Development, Implementation, Evaluation, and Application, 12-13 January 2007, Hong Kong*, 2007.

[12] L.C. Reese, W.R. Cox, and F.D. Koop. Analysis of laterally loaded piles in sand. In *Proceedings of the 6th Annual Offshore Technology Conference, Houston, Texas, OTC 2080*, pages 473–458, 1974.

[13] W. Richwien, K. Lesny, and P. Hinz. Validierung bautechnischer Bemessungsmethoden für Offshore-Windenergieanlagen anhand der Messdaten der Messplattformen FINO1 und FINO2 (Gigawind plus), Jahresbericht 2004, Kapitel 3: Tragstruktur-Gründung, März 2004.

[14] W. Richwien, K. Lesny, and J. Wiemann. Bau- und umwelttechnische Aspekte von Off-shore Windenergieanlagen (Gigawind), Jahresbericht 2001, Kapitel 6: Tragstruktur-Gründung, April 2002.

[15] H. Sturm, O. Solf, and P. Kudella. Self-healing effects of shallow foundations for off-shore wind turbine structures. In *11th Baltic Sea Geotechnical Conference: "Geotechnics in Maritime Engineering", Gdańsk, Poland, 15-18 September 2008*, 2008.

[16] J. Swinianski and A. Sawicki. A model of soil pile interaction owing to cyclic loading. *Canadian Geotechnical Journal*, 28(1):11–19, 1991.

[17] Det Norske Veritas. Offshore Standard DNV-OS-J101: Design of Offshore Wind Turbine Structures, 2004.

[18] R.C. Welch and L.C. Reese. Laterally loaded behavior of drilled shafts. Technical Report 3-5-65-89, Center for Highway Research, University of Texas, Austin, 1972.

[19] T. Wichtmann. Explicit accumulation model for non-cohesive soils under cyclic loading. Dissertation, Schriftenreihe des Institutes für Grundbau und Bodenmechanik der Ruhr-Universität Bochum, Heft 38, available from www.rz.uni-karlsruhe.de/∼gn97/, 2005.

[20] T. Wichtmann, A. Niemunis, and Th. Triantafyllidis. Recent advances in constitutive modelling of compaction of granular materials under cyclic loading. In N. Bazeos, D.C. Karabalis, D. Polyzos, D.E. Beskos, and J.T. Katsikadelis, editors, *Proc. of 8th HSTAM International Congress on Mechanics, Patras, Greece 12 14 July*, volume 1, pages 121–136, 2007.