Approaches for the Design of Foundations for Offshore Wind Turbines

A Review Based on Comparisons with HCA-Based Models

Hauke Zachert[1(✉)] and Torsten Wichtmann[2]

[1] Institute and Laboratory of Geotechnics, Technische Universität Darmstadt, Franziska-Braun-Straße 7, 64287 Darmstadt, Germany
zachert@geotechnik.tu-darmstadt.de

[2] Chair for Soil Mechanics, Foundation Engineering and Environmental Geotechnics, Ruhr-University Bochum, Universitätsstraße 150, 44789 Bochum, Germany
torsten.wichtmann@rub.de

Abstract. The design of foundations for offshore wind turbines is still a challenging engineering task. Design rules are not always settled in national standards and especially tasks like the design against cyclic loading remain a field where results of scientific research are immediately implemented in practical design approaches. The PISA project demonstrated, that even the static design of monopiles can be improved by validating new design approaches and implementing them in design tools. This paper compares different approaches for the static and cyclic design of foundations for offshore wind turbines, including new approaches like PISA, SOLDYN and TANDEM, as far as available. Focus is laid on the validation process of a cyclic approach, based on the high cycle accumulation (HCA) model. The main results of this validation on monopiles and shallow foundations in different scales are discussed. The HCA model showed to be very well suited to simulate and predict the cyclic soil structure interaction processes of foundations for offshore wind turbines. Effects like the accumulation of deformations or the redistribution of stresses and internal forces induced by a cyclic loading on monopiles or shallow foundations are well reproduced by the HCA model. This is demonstrated based on a comparison of results from model tests and in situ tests with numerical predictions. Furthermore, the results of these studies on the HCA model are set in relation to the approaches from other research groups. The outcome of the PISA project for the static design of monopiles is shown and reviewed based on the own results of numerical simulations.

1 Introduction

Wind energy has a key role in the worldwide transition from fossil energy sources to renewable energies. While onshore wind energy can be considered a matured industry, the offshore wind market still exhibits a vast growth. By 2018, a total capacity of 23.1 GW was installed offshore worldwide and forecasts predict a growth to about 100 GW

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by 2025 [18]. Europe has the highest installed offshore wind capacity, but Asia is catching up and shows yearly additions of about 5–7 GW [18]. New markets like Taiwan are developing and existing markets like Europe experience a boost by the first auctions which were closed by so called “zero bids”, where the development, construction and operation of the offshore wind farms is offered subsidy free [45]. This encourages the public, like the Dutch government to increase their targets for the offshore wind development.

These achievements were enabled by a huge professionalisation of the whole supply chain but also by a continuous increase of turbine size and efficiency [18]. A turbine capacity of about 8 MW is already reached and the trend leads towards 12 MW [13]. This goes along with an increase in the diameter of wind turbine blades, which already reached 167 m for a modern 8 MW turbine [44].

Increasing rotor diameters will transfer increasingly higher loads to the foundations. The dominant foundation type for offshore wind turbines (OWT) is the monopile: a single pile with about 6 m to 8 m diameter and an embedment length of about 20 m to 30 m below seabed. It represents about 80% of the installed OWT foundations [46]. Beside monopiles, jacket or tripod foundations and gravity base foundations have been applied for the secure transfer of offshore loads to the ground. OWT foundations have to withstand different types of loading: intense horizontal loads due to wind and large single waves, vertical loads caused by the self-weight of the turbine and a high-cyclic loading due to wind and water waves. The latter may cause an accumulation of permanent deformations in the soil. The serviceability of the OWTs may get lost due to an excessive tilting of the tower, considering the strict operational requirements of the turbines (a maximum tilting of 0.25° is often stated, see e.g. [15] or [31]). In addition, the change of the bedding reaction due to an accumulation of deformation or changes of lateral stress along the pile shaft may cause a shift in the eigenfrequencies of the turbine. Experience from conventional offshore foundations (e.g. oil rigs) cannot be easily adapted since the ratio of the horizontal cyclic loads and the self-weight of the structure is significantly larger for the OWTs. Early offshore wind parks had smaller turbines (2 to 3 MW) and were installed in shallow waters up to 20 m [46]. New offshore windfarm areas are predominantly assigned further offshore with increasing water depths up to 40 m [46]. Thus, bending moments in monopile foundations are much larger for the new OWT foundations. Furthermore, the increased rotor diameter and the increased height of the turbines lead to higher loads transferred to the foundations. Practical long-term experience regarding such large pile diameters in combination with high ratios of cyclic horizontal load to the self-weight of the structure is still limited.

Only few design codes like API RP2GEO [2], DIN EN ISO 19901-4 [12], DIN EN ISO 19902 [11] or DNV-GL [15] are considering cyclic loading at all and their approaches are based on in situ tests on flexible piles with high slenderness (length L to diameter D ratio), diameters below 1 m and subjected to only about 100 load cycles. Since the discrepancy to the current industry practice for monopiles applying small slenderness and large pile diameters, and considering high numbers of load cycles is obvious, several research projects on horizontally loaded monopiles were conducted in the past decade.

Besides the few mentioned design standards, some technical recommendations for the design of offshore wind turbine foundations were established based on these research projects to support the safe design of such foundations, like BSH-standard [9], SOLCYP [15] or EA-Pfähle [16]. Still, most of these recommendations are based on small-scale 1 g-model tests or centrifuge tests. Large-scale in situ tests on horizontally loaded monopiles have been performed recently to support the validation of such approaches but are mostly still under evaluation or have only recently been published [7, 16, 33].

The most common approaches for the design of offshore monopiles will be briefly discussed in Sect. 2.

2 Industry Standards for the Design of Foundations for Offshore Wind Turbines

The general design of offshore wind turbine foundations has to comply with the applicable standards and regulations, e.g. defined in the BSH-Standard [9], DIN EN 1997-1 [14] in combination with EA-Pfähle [16], API [2] or DNVGL [15]. However, as stated before, most of these regulations do not refer to the design of cyclically loaded foundations on a validated basis or they have not been validated for the current standard of large diameter monopiles. For monopiles, most designs follow the API-approach [2] with p-y curves defining the load-deflection relationships for the horizontal soil reaction. Cyclic loading is only referred to by a moderate reduction of the stiffness.

Several dominant effects for the safe and economic design of OWT foundations are neglected or not accurately covered by the API-approach. The horizontal bearing behaviour of very stiff monopiles like the modern XXL-monopiles has not been studied in detail yet (see e.g. [13, 21]), the accumulation of deformations due to the high-cyclic loading is not covered accurately, the potential accumulation of excess pore water pressure is often neglected [21] and furthermore, the soil stiffness during dynamic loading is not covered accurately although it heavily affects the dynamic response of the wind turbine [32]. These uncertainties lead to the development of a variety of design approaches on national and international level. One approach for the design of monopiles which is addressing the mentioned inaccuracies is described in [21] and [31], another one is proposed in [4].

The behaviour of monopiles with large diameters is often studied by means of 3D finite element simulations, which is also recommended by DNVGL [15] to overcome the uncertainties regarding the large pile diameters. However, a validation of these numerical approaches can only be achieved by large-scale tests as they are currently performed in several studies, like the PISA [7] or the TANDEM [33] projects.

In the TANDEM project, two series of piles were tested in the "Testzentrum Tragstrukturen Hannover" by Fraunhofer IWES. With reference to a monopile with Diameter $D = 8.0$ m, embedment length $L_{pen} = 32.0$ m and a maximum horizontal loading $H_{ULS} = 25.4$ MN, three different test piles with different scales were tested in sand compacted in layers: D1220 ($D = 1.220$ m, $L_{pen} = 7.50$ m, $H_{ULS} = 217$ kN), D914 ($D = 0.914$ m, $L_{pen} = 6.15$ m, $H_{ULS} = 108$ kN) and D610 ($D = 0.610$ m, $L_{pen} = 4.70$ m, $H_{ULS} = 45$ kN). The deflection curves of a first series of these three test piles in the TANDEM project are shown in Fig. 1, together with the deflection obtained from 3D finite element simulations applying a modified hypoplastic constitutive model for the sand [13]. In principle, the agreement between the test data and the model prediction seems to be very good, but especially the deformation at the pile tip, which

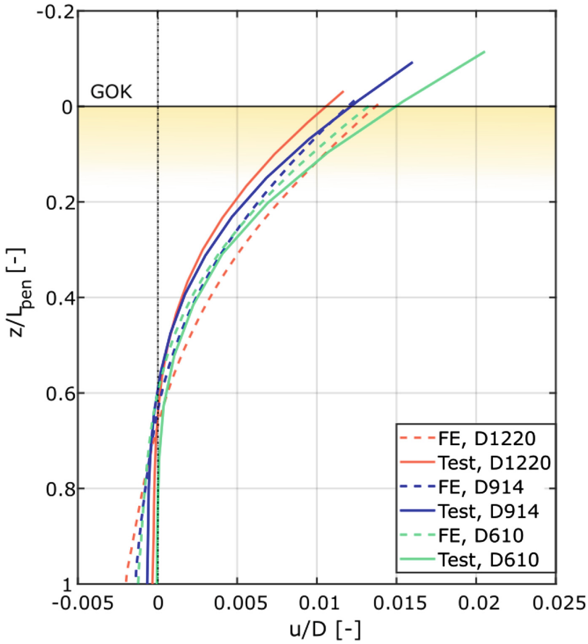

Fig. 1. Pile deflection curve for three tests in the TANDEM project ("Test") and their numerical approximation ("FE"), D1220 (D = 1.220 m, L_{pen} = 7.50 m, H_{ULS} = 217 kN), D914 (D = 0.914 m, L_{pen} = 6.15 m, H_{ULS} = 108 kN) and D610 (D = 0.610 m, L_{pen} = 4.70 m, H_{ULS} = 45 kN) [13].

is critical for the development of the base shear (see explanations below) shows some deviations from the tests.

Especially the PISA project is attracting a lot of attention. In [8] a new approach for the static design of monopiles was proposed which is based on the results of the large-scale monopile tests in [7] and [5], where monopiles were tested in sand and in clay. The monopiles in sand had dimensions of D = 0.273 m (L/D = 5.25, 8 and 10), D = 0.762 m (L/D = 3, 5.25 and 8) and D = 2.00 m (L/D = 4 and 5.25) and were subjected to horizontal static and cyclic loads. In contrast to the majority of the common approaches, it is explicitly considering large-scale monopiles and its almost rigid behaviour. This led to the introduction of a base shear force at the pile tip [6]. While the regular monopile design was requiring an almost vertical tangent at the pile tip or at least very small deflections (5% of the pile head deflections, see [21]), the PISA approach is acknowledging the more rigid pile behaviour by introducing this base shear. This general approach is also followed in the TANDEM project, see [13].

The PISA approach has already been introduced into a commercial PLAXIS tool, called MoDeTo (Monopile Design Tool), which enables designers to apply the PISA approach for their monopile design, see e.g. [27]. Up to now, the PISA approach is only referring to the design of monopiles subjected to static loading. An extension for cyclic loading has not been published yet. The PISA approach will be discussed in more detail in Sect. 4.1.

The cyclic behaviour of piles was the main subject of a French research group which gathered their results in the SOLCYP recommendations [29]. The recommendations concentrate on piles under vertical cyclic loading and are based on results of small-scale model tests, large-scale field tests and centrifuge tests on piles. Horizontally loaded piles have been studied in detail by means of centrifuge model tests in non-cohesive soils [30]. Large-scale tests on horizontally loaded piles were not part of the program. Furthermore, the horizontally loaded piles investigated in the SOLCYP were rather flexible than rigid, which puts the applicability of the results to modern monopiles into question. Besides these limitations, the model tests showed some relevant results for the cyclic pile-soil-interaction. Figure 2 shows the cyclic *P*-*y*-curves for a model pile in dry sand at different depths below ground with *P* defined in [29] as the soil reaction of the pile at a certain depth. It is obvious, that the stiffness increases with depth and that the residual deformations grow with increasing number of cycles, which is usually considered by a decrease of the *p*-*y*-stiffness.

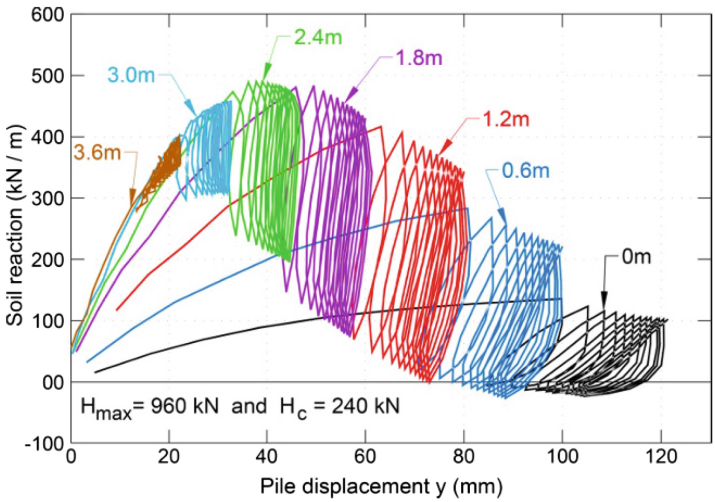

Fig. 2. Example of cyclic *P*-*y*-curves at different depths for a model monopile in dense (I_D = 86%) dry sand tested in a geotechnical centrifuge, representing a pile with L = 12.0 m and D = 0.72 m, [30] according to [29].

Furthermore, an increase of the bending moment of the pile was reported as shown in Fig. 3. This increase demonstrates the complex soil-structure-interaction occurring during cyclic loading. Such a change in the bending moment should not be neglected in a monopile design. Similar evolutions of bending moments have been reported in [47].

For the design of monopiles, most of the described shortcomings could also be overcome by a numerical model which is capable to capture all effects of a cyclic loading, in particular the accumulation of deformations or excess pore water pressures and changes of stresses in the soil and the structure, and the scale effects. For that purpose, the authors of the present paper propose the application of the High Cycle Accumulation model (HCA) developed at the Institute of Soil Mechanics and Rock Mechanics (IBF) in Karlsruhe. The huge advantage of such a numerical approach is the flexibility with regard to the foundation type and size investigated. Furthermore, it allows insights into the effects of cyclic soil-structure-interaction which are difficult to identify in model tests. The HCA model was validated by several comparisons with boundary value problems with high-cyclic loading [48]. Section 3 will show some examples for this validation.

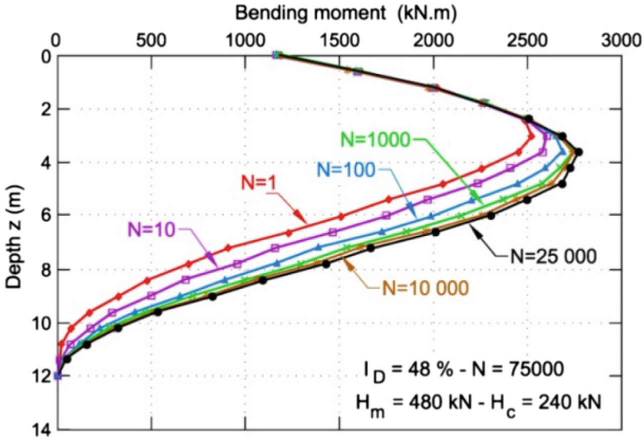

Fig. 3. Example of the evolution of bending moments for a flexible model pile (representing a pile with L = 12 m, D = 0.72 m) tested in a geotechnical centrifuge [30] according to [29].

3 Application of HCA Based Models for the Design of Offshore Wind Turbine Foundations

3.1 HCA Model

The procedure for the proof of serviceability for offshore wind turbine foundations is based on FE calculations with the HCA model. The HCA model of Niemunis et al. [25] is only applicable to a high-cyclic loading. Monotonic loading and unloading phases and a few number of cycles, especially at the beginning of a load package, have to be calculated with conventional soil models. Here, Hypoplasticity with Intergranular Strain [26] is used for these so-called *implicit* steps. Steps 1–5 in Fig. 4 represent for example the application of the self-weight and of the first two load cycles. Step 4 is called *irregular cycle*, because it includes the first loading to the maximum load and can produce a much higher residual strain than the following (*regular*) cycles. The strain path during the first regular cycle (step 5) is used to evaluate the strain amplitude ε^{ampl}. One basic assumption of the HCA model is, that strain and stress paths resulting from cyclic loading can be divided into an oscillating part and a trend. The oscillating part is captured by ε^{ampl}, which is assumed to remain constant over a certain number of cycles with constant load amplitude. ε^{ampl} is one of the main input parameters of the HCA model, which is used for the calculation of the accumulation trend, i.e. the development of permanent deformations with increasing number of cycles, see step 6 in Fig. 4. This trend due to an increment ΔN of the number of cycles is calculated in analogy to a creep process within the so-called *explicit* step. During the explicit parts of the calculation the external loads are kept constant on their average values. The strain amplitude can be updated in a so-called control cycle calculated implicitly (step 7 in Fig. 4).

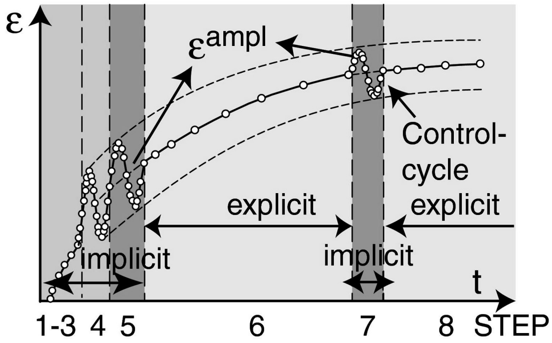

Fig. 4. Implicit and explicit steps in FE calculations with the HCA model.

The prediction of the development of permanent deformations with the HCA model (step 8 in Fig. 4) is then continued with this updated value of ε^{ampl}. The constitutive equations are described in detail by Niemunis et al. [25]. The HCA model captures the main influencing parameters on the accumulated strain, such as the strain amplitude ε^{ampl}, the number of cycles N, the void ratio e, the average mean pressure p^{av}, the average stress ratio η^{av} and polarization changes. The equations of the HCA model are based on numerous cyclic triaxial tests on different sands [39].

3.2 Validation of the HCA – Based Approach for Monopiles

Developed in a theoretical framework and calibrated with numerous cyclic triaxial tests on sand, the HCA model has also been applied in several simulations of offshore wind turbine foundations subjected to cyclic loading (e.g. [43, 48, 50]).

The validation of the HCA model for the application on monopiles was based on small-scale 1 g-model tests in two different scales, see e.g. [50] and [43]. In the following mainly the small-scale model tests performed by the first author at the Institute of Soil Mechanics and Rock Mechanics (IBF) at Karlsruhe Institute of Technology (KIT) will be discussed.

The test setup, applied for the model tests on monopiles consists of a steel container (diameter: 95 cm, depth: 145 cm) which was filled with Karlsruhe fine sand (clean quartz sand, mean grain size $d_{50} = 0.14$ mm, uniformity coefficient $Cu = d_{60}/$

d_{10} = 1.5) by pluviation. The model pile was then driven by impact into the dry sand until it reached the target embedment length L which was chosen between 40 cm and 100 cm. Two separate frames, the measurement frame and the loading frame were mounted on top of the steel container. The test setup is presented in Fig. 5 together with a scheme showing the positions of sensors for displacement and strain measurements at the model pile.

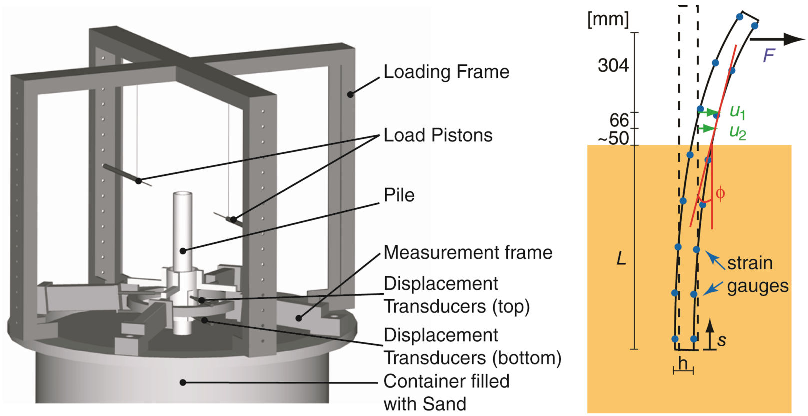

Fig. 5. Small-scale model tests on monopiles as a basis for the validation of the HCA-model approach.

The model pile consists of a PVC pipe (outer diameter: 61 mm, wall thickness: 3 mm) with an elastic modulus E = 3650 N/mm^2 leading to a bending stiffness EI = 0.8412 kNm. EI has been confirmed in calibration tests with static loading on a statically determined system. The pile was equipped with eight layers of strain gauges, equally distributed over the pile length. From the strain difference $\Delta\varepsilon$ measured for each layer the bending moments and the whole deflection curve of the model pile can be derived for every load cycle. This enables a deeper insight into the cyclic soil-pile interaction compared to the common practice where only the pile displacements are measured close above the sand surface, e.g. in [17] or [22]. For details of the model tests at KIT the interested reader is referred to [48, 50] and [47].

The model tests formed the basis for numerical simulations applying the HCA-based approach as described in Sect. 3.1. Static load steps were calculated with Hypoplasticity and intergranular strain while the high-cyclic loading was captured by the HCA model. All FE simulations were performed using the commercial FE software Abaqus Standard and applying material parameters derived independently from laboratory tests. These material parameters for the Karlsruhe fine sand (KFS) are shown in Tables 1 and 2.

All material parameters were completely obtained from element or index tests in the laboratory and were not fitted afterwards to reach a better prediction of the model test results. This independent determination of the parameters of the constitutive models used in the FE simulations represents an important requirement in order to validate the

Table 1. Material parameters of hypoplasticity and intergranular strain for KFS

φ_c	e_{d0}	e_{c0}	e_{i0}	h_s	n	α	β	R	m_R	m_T	β_R	χ
33.1	0.677	1.054	1.212	2000	0.32	0.12	2.0	10^{-4}	2.6	1.3	0.1	6.0

φ_c in [°] and h_s in [MPa]; all other parameters dimensionless

Table 2. Material parameters of the HCA model for KFS (all parameters dimensionless)

C_{ampl}	C_e	C_p	C_Y	C_{N1}	C_{N2}	C_{N3}
1.33	0.60	0.23	1.68	$2.95 \cdot 10^{-4}$	0.41	$1.90 \cdot 10^{-5}$

numerical approach. The FE model of the model tests is shown in Fig. 6. It utilizes the symmetry of the system by representing only one half of it. The pile installation process was not considered in the numerical simulations, that means the pile was modelled *wished in place*. Further details on the FE model can be found in [48] and [47].

As a first step of the validation, the pile behaviour under static loading in the model test and the FE simulations was compared, as shown in Fig. 6. The distribution of the bending moment along depth for different magnitudes of the horizontal load F applied at the pile head was very well captured by the corresponding numerical simulations. The same applied for the static deformations (see [48] for detailed results). The accurate simulation of the static load steps (*implicit steps*) is essential for a successful prediction with the HCA model since the strain amplitude ε^{ampl} is recorded in these steps, which is one of the decisive input parameters of the HCA model.

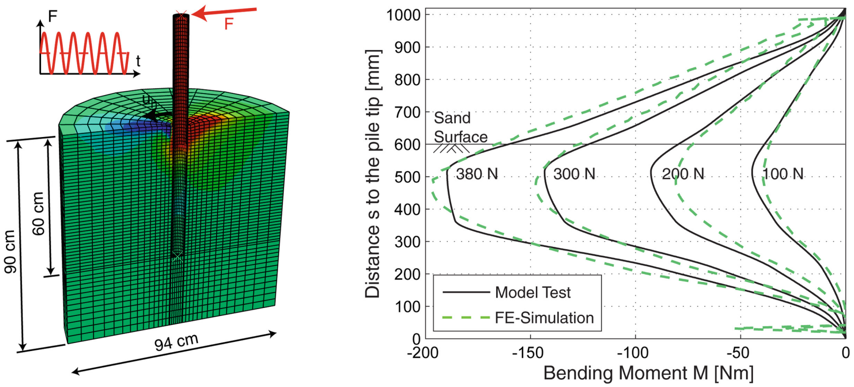

Fig. 6. FE model using C3D8R elements to simulate the model tests on monopiles and distributions of bending moments along depth measured in the model tests in comparison with the results from the numerical simulations for different loading stages.

Figure 7 presents the total horizontal displacements measured slightly above the sand surface in four tests with different constant load amplitudes $F^{ampl} = F^{av}$. Each test was conducted in a freshly pluviated sand. Only the average deformation u^{av} is displayed in Fig. 7. The measured total deformations (static and accumulated part, solid curve in Fig. 7) show again a very good agreement with the FE predictions (dashed curves in Fig. 7). However, the test results for F^{ampl} = 7.5 N reveal a certain scatter which is inevitable in geotechnical model tests, see e.g. [10].

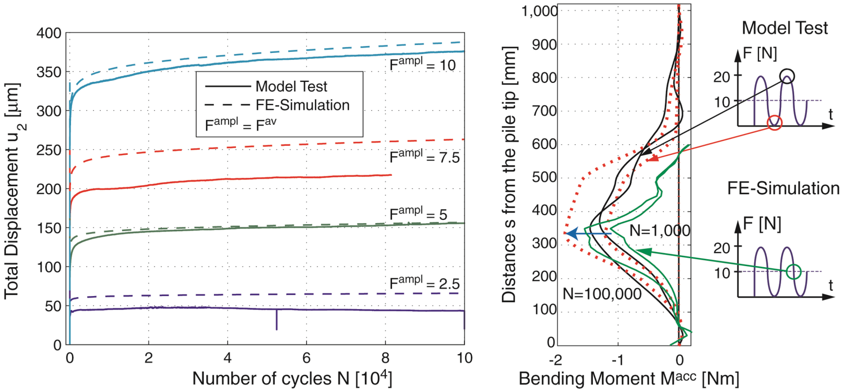

Fig. 7. Total horizontal displacement measured at the model piles in tests with different load levels and with about 100,000 cycles compared to the numerical prediction and change of the bending moment due to the cyclic loading in the model test and the numerical simulation.

Beside the total and accumulated deformations, the development of bending moments during cyclic loading has a governing impact on the design of monopiles. From the measurements in the model tests an increase of the bending moments with increasing number of cycles could be concluded. Similar changes of bending moments have been observed in centrifuge model tests on cyclically loaded monopiles ([30], according to [29]). The change of bending moment M^{acc} is calculated as the difference between the bending moment at a certain cycle and its initial value after three cycles. M^{acc} is shown for one exemplary model test and the corresponding simulation in Fig. 7. Again, the numerical results show a satisfying accordance with the test data. This demonstrates the advantage of a FE model capturing all effects of a cyclic loading at once without relying on phenomenological descriptive relations between deflections and the number of cycles as in some early approaches.

Beside the model tests with a single package of cycles of constant load amplitude, tests with several such packages of different amplitudes applied in succession have been performed. They were also simulated with the FE model. Figure 8 demonstrates the good agreement between horizontal displacements measured in such a model test with 13 different load packages and the corresponding numerical prediction, applying the HCA approach.

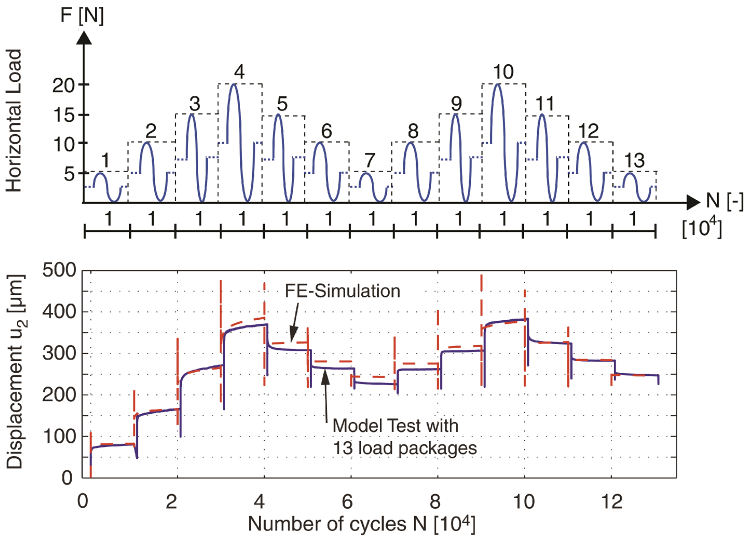

Fig. 8. Total horizontal displacement of a monopile subjected to 13 load packages with different amplitudes and 10,000 cycles each. Comparison of the data from the model test with the corresponding numerical prediction.

Beside these small-scale model tests performed at the KIT, [48] and [43] demonstrate the application of the HCA-based approach to model tests with a larger pile diameter, performed at TU Berlin [35]. These simulations showed as well a satisfying agreement with the model tests.

Several experimental studies on the horizontal bearing capacity of large-scale monopiles and on the effect of cyclic loading on monopiles are still in execution or have only recently been published. Some of them apply industry dimensions like [24], others use a scale of 1:3 to 1:10, which still results in test pile diameters of 1 to 2 m like in [33] and [6]. These tests have not been simulated with the HCA-based approach yet. Successful recalculations of these tests would mean a validation of the HCA-based approach for monopiles in a larger scale and would thus further improve the confidence in the approach.

3.3 Validation of the HCA – Based Approach for Gravity-Based Foundations

Some of the first offshore windfarms were founded on shallow foundations. In the meanwhile, monopiles represent about 80% of all installed offshore wind foundations [46]. With the aim to provide a new foundation structure for OWT, the Ed. Züblin AG developed a gravity-based foundation, designed for water depths up to 40 m. In order to demonstrate the bearing capacity and serviceability of this innovative foundation under static and cyclic loading, a large-scale test was performed on a prototype foundation. Details of this test are presented in [19] and [20]. The foundation consists of a concrete box girder with a shaft and four precast concrete plates (A–D). The foundation is filled with a sand ballast and the excavation pit was flooded in order to simulate offshore conditions [19]. The subsoil consists mainly of medium dense to dense sand which is similar to the offshore in situ conditions. The foundation was subjected to an unidirectional high-cyclic loading with about 1.5 million cycles in order to simulate the loading from wind and water waves. The foundation and the test loading arrangement are shown in Fig. 9.

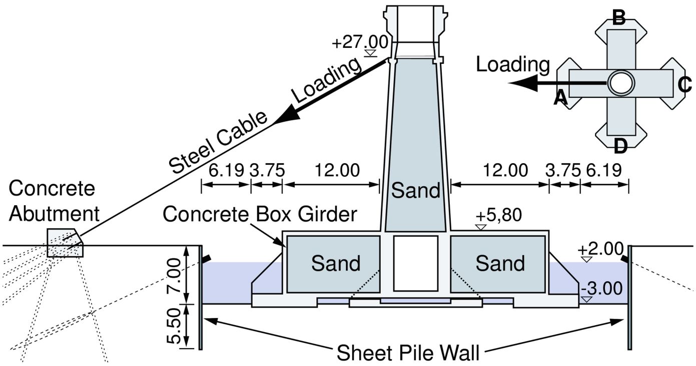

Fig. 9. Test foundation subjected to 1.5 million load cycles, after [19].

Not only the test foundation but also the subsoil was extensively instrumented in order to capture for example the settlements of the foundation itself (e.g. vertical extensometers V1 and V2 in Fig. 10) or in different depths below the plates. Furthermore, pore water pressures and total pressures (soil pressures) were measured beneath each of the plates at different locations and depths.

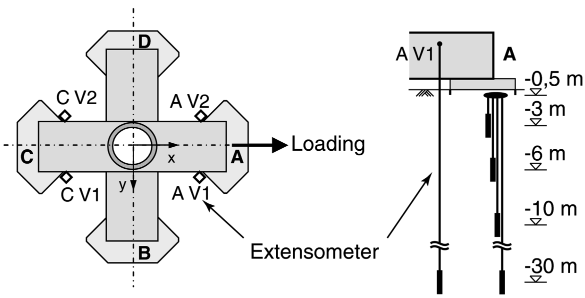

Fig. 10. Top view on the plates of the test foundation and the installed extensometers below and aside the plates, after [19].

One of the main concerns in terms of the application of new design approaches is the missing validation on prototype tests. This still holds true for monopiles. The shown test foundation however can serve as a basis for the evaluation of numerical models for shallow foundations for OWT. The test foundation is a unique example of an in-situ test, with a huge amount of measurement equipment and thoroughly performed. It was utilized in [48–50] to validate the HCA-approach for OWT shallow foundations. The finite element mesh applied for the numerical simulations is shown in Fig. 11.

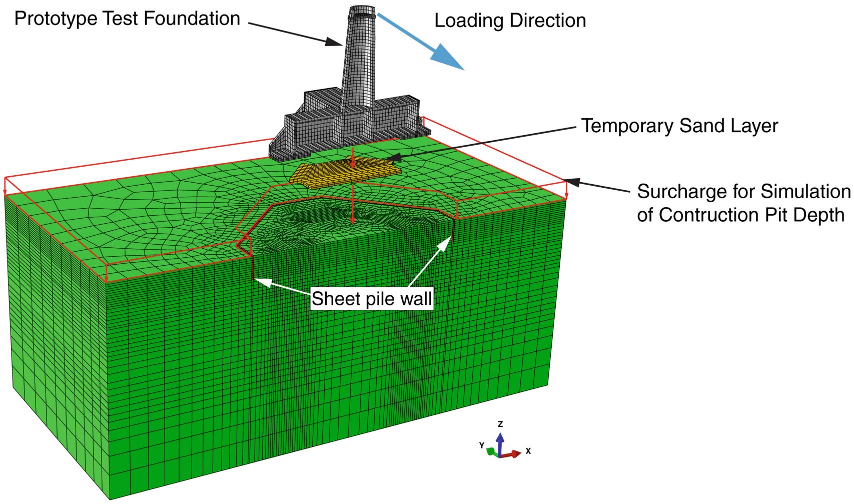

Fig. 11. FE mesh for the simulation of the test foundation.

The subsoil conditions were evaluated based on CPT profiles determined prior to the construction of the foundation and on a set of static and cyclic laboratory tests on the fine and medium sands gained from liner drillings exactly below the plates of the foundation. Based on these laboratory tests, three different approaches according to [40] and [41] were applied to determine the material parameters of the constitutive models. The approaches apply different numbers of cyclic triaxial tests and thus can reach different levels of accuracy regarding the HCA model parameters.

Figure 12 shows the comparison between measured and simulated settlements in the first tested storm event, with a satisfying agreement of both curves, in particular for foundation plate C. The simulation has been performed with the best estimate material parameters based on all available cyclic laboratory tests and with the most accurate evaluation of the in-situ conditions with respect to the relative density. It is important to note that these simulations were treated like a prediction, not a back-calculation. Thus, no results of the test itself were used to calibrate the material parameters in [48]. Applying different parameter sets, it was confirmed that a more detailed determination of material parameters also leads to a higher level of accuracy with respect to the accumulation of foundation settlements under cyclic loading. However, already the approach which purely relies on correlations and does not require any cyclic triaxial test [41] was giving a reasonably good agreement with the measurements at the prototype foundation, cf. [49].

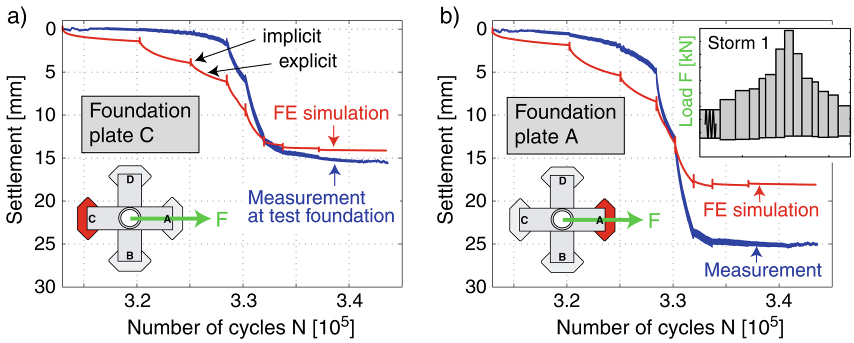

Fig. 12. Measured and calculated accumulated settlements in the first simulated storm.

While the number of cyclic triaxial tests used for the determination of the material parameters had a limited impact on the numerical results in this case, it was proven in [49] that the accurate determination of the relative density I_D in the field has a huge impact on the predicted accumulation of deformation. However, a sufficiently accurate prediction of the settlement accumulation can be achieved by applying mean values of I_D across soil layers. A more detailed consideration of density fluctuations, e.g. by means of sublayers (e.g. 20 cm layers, each with a constant I_D) or random void ratio fluctuations seems not necessary, at least not in the simulations of the prototype foundation.

Not only the settlements were compared between simulation and test, also the excess pore water pressures and the contact pressures between the plates and the subsoil were assessed in detail, cf. [50] and [43]. The contact pressures were exhibiting a strong evolution caused by the cyclic loading. The pressures below the mainly loaded plates (Fig. 10: plates A and C) were decreasing with every storm event while the load plates perpendicular to the loading direction experienced a significant increase of contact pressures. This redistribution of stresses was well predicted by the numerical models applying the HCA-based approach. Detailed results can be found in [48] and [50].

Considering the overall good reproduction of the behaviour of the test foundation by the HCA-based approach and the good agreement on small-scale model tests and on other applications [43], the HCA-based approach can be regarded as validated for cyclically loaded structure in non-cohesive soils. Section 4 will compare the results of HCA-based simulations with the findings from other research groups working on the behaviour of OWT foundations under static and cyclic loading, with a focus on monopiles.

4 Comparison of HCA-Based Simulations with Other Standards

4.1 PISA Approach for Static Loading of Monopiles

The geotechnical design of monopiles is governed by static loading. The PISA approach has gained a lot of attention in the last years and is starting to be implemented into current wind farm designs. Thus, it will be shown more in detail in this Section and will be set into relation to numerical simulations using a hypoplastic model.

The PISA model relies on the data of field tests on scaled monopiles with diameters D between 0.27 m and 2.00 m and *L*/*D*-ratios between 3 and 10 (majority at L/D = 5.25) cf. [5] and [7]. Based on the test results with static loading, a new approach for the design of monopiles was developed [8]. In principal it is following the well-established approach of defining the bedding reaction of the pile as function of the deflections by *p*-*y*-curves (horizontal deformation *y* is referred to as *v* in the PISA approach). Instead of relying on the *p*-*v* curves only, the PISA approach introduces further relations for additional soil reactions applicable to less slender monopiles [8], see Fig. 13:

- Distributed lateral load *p* vs. horizontal deformation *v*
- Distributed bending moment of the pile *m* vs. local cross section rotation ψ
- Base shear force at pile tip H_B vs. horizontal displacement at pile tip v_B
- Base moment M_B vs. rotation of the pile tip ψ_B

All these additional reaction forces were determined in 3D finite element simulations and are simplifications of the shear forces which mainly occur at the inside and the outside of the large diameter piles. The base shear H_B is a result of the stiff pile behaviour which enables the so-called toe kick and a reaction force in the soil-soil

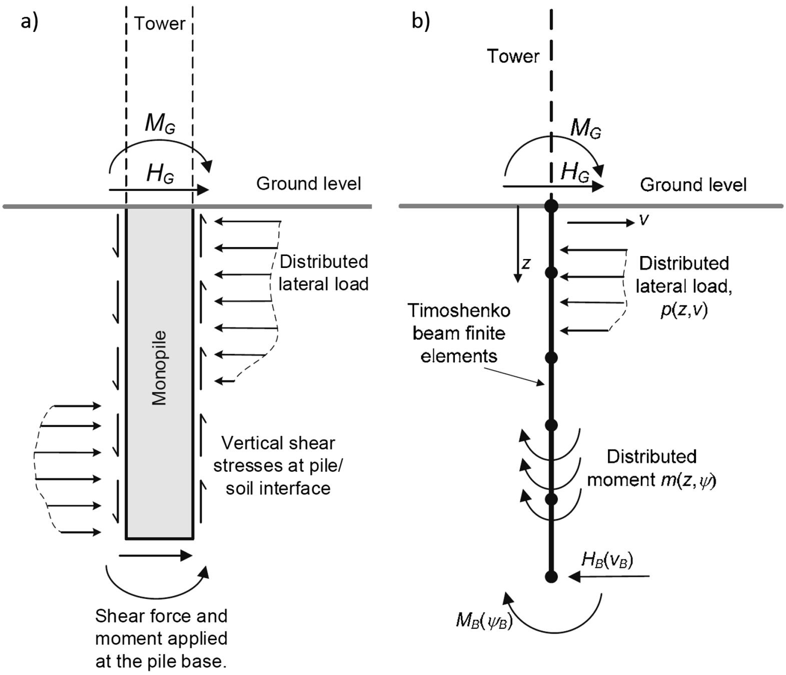

Fig. 13. (a) Soil reactions implemented in the PISA design model and (b) their simplification as Timoshenko beam for the 1D FE approach [8].

interface at the pile tip. These general forces are applied in the design approach [13] based on the pile tests in the TANDEM project.

Two approaches for the design of monopiles are proposed in [8], the so-called "Rule-Based Method" and the "Numerical-Based Method", see Fig. 14. Both methods aim to predict the lateral response of the monopiles based on soil reaction curves. The numerical-based approach combines FE simulations on a 3D model with simplified FE simulations of a 1D Timoshenko beam representing the monopile and capturing the soil reaction curves. 3D simulations have to be performed to determine the normalized soil reaction curves for a first draft pile geometry and the exact soil conditions, represented by a suitable soil model. [8] applied the bounding surface model after [23] modified by [34] for simulations in sand. These calibrated soil reaction curves are then applied in a 1D FE simulation covering the Timoshenko beam shown in Fig. 13. This 1D FE model is applied to allow efficient parameter studies on the pile geometry which finally lead to the optimized pile design. One of the ambitions of the PISA project was to develop design tools which are as simple as possible but consider the relevant soil-pile interaction mechanisms and the relevant soil behaviour. This shall enable a wide group of design engineers to apply these techniques in their offshore design practice.

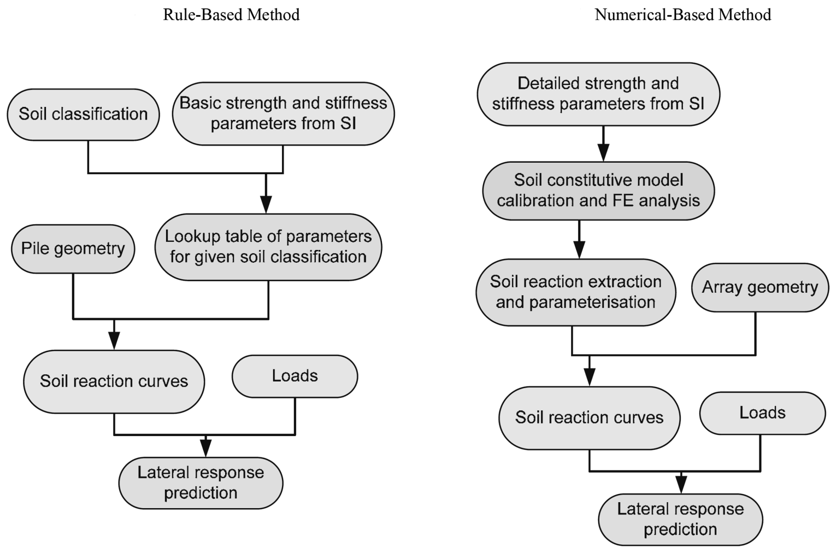

Fig. 14. Rule-Based and Numerical-Based Method for the PISA approach [8].

The application of the PISA model in engineering practice is even further improved by a specific Plaxis design tool: MoDeTo (Monopile Design Tool), which enables the user to perform the 3D and 1D FE simulations automized and even more important, to extract the soil reaction curves fully automatized from the 3D model and transfer them into the 1D simulations [27].

At the TU Darmstadt, studies are ongoing on detailed simulations with the PISA model as well based on an own developed program covering the 1D approach and the soil reaction curves but also using the Plaxis MoDeTo environment. The results of these simulations are set in relation to 3D FE simulations with Hypoplasticity and Intergranular Strain performed in Abaqus. Monopile dimensions and soil properties are chosen in line with the structure presented in [38]. The monopile with a diameter D = 5 m, embedment length L = 30 m and a wall thickness of t = 6 cm is loaded at a height of 20 m above ground level with different horizontal loads (0.5 MN, 1.5 MN, 2.5 MN). Soil conditions are chosen to be a homogenous fine quartz sand with three different relative densities.

Figure 15 shows the monopile deformations and the bending moments obtained from the simulations with the three different models: the PISA approach following the numerical-based method in the MoDeTo environment, the p-y curves according to API and a 3D FE model using Hypoplasticity and Intergranular Strain in Abaqus. The latter simulations are referred to has HIA (Hypoplasticity, Intergranular Strain, Accumulation Model), because generally they could be extended by an explicit phase using the HCA model (which has not been done here). The soil parameters for API and HIA are given in [37]. Material parameters for the PISA approach were determined according to the recommendations of the MoDeTo user manual [28]. However, these PISA parameters are still under investigation since they preferably should be determined from laboratory tests to reach the same level of accuracy as the HIA parameters.

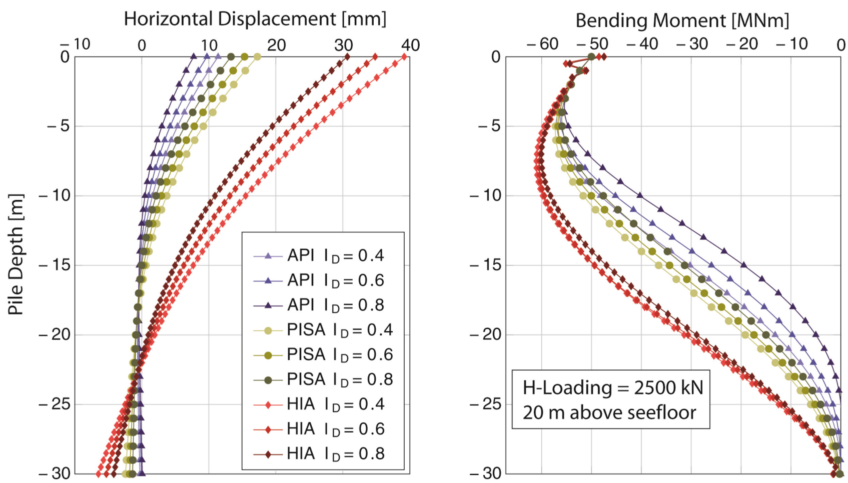

Fig. 15. Pile displacement and bending moment for three relative densities (I_D = 0.4, 0.6 and 0.8) calculated with the PISA model following the numerical-based method using the MoDeTo environment in comparison to results applying the API – approach and using Hypoplasticity with Intergranular Strain in an Abaqus model.

The comparison in Fig. 15 shows a good agreement between API and PISA with the main difference in the pile tip displacement. The HIA simulations were giving much higher displacements and the influence of relative density is more pronounced. The larger deformations go along with higher bending moments in HIA compared to the other approaches. Further studies at TU Darmstadt are currently dealing with the comparison between the analytical approach of PISA in comparison to the MoDeTo as well as testing the sensitivity of the model. Since these studies are still ongoing, results can only be shared in future publications.

Preliminary conclusions applying the PISA model are the following:

- Although the theoretical framework was intended to be easily adoptable for individual applications, the coding of the model turned out to be very sophisticated and not straight forward at some points.
- The material parameters for the PISA model need to be chosen carefully (as for every other model as well).
- Extracting the soil reaction curves, bending moment m, base shear H_B and base moment m_B is a very sophisticated task if not performed via the MoDeTo environment.
- Working in the MoDeTo environment leads to quick results without requiring intense preparation. It somehow has the structure of a black box since some steps are not easy to trace back. However, the easy handling includes the chance that also unexperienced users can successfully apply this environment.
- MoDeTo has some further features which efficiently support the design of monopiles and open the system for future developments
  - The efficient combination of a 3D model with the 1D approach enables fast optimization while still considering the relevant soil-pile interaction effects.
  - Any other soil model can be implemented in the 3D FE environment and MoDeTo will still be able to extract the soil reaction curves determined with this user-defined soil model.
  - MoDeTo offers the option to perform a set of 3D FE simulations with different geometries to define the parameter space in which the mean soil reaction curves will be defined. First evaluations on this showed only a small impact of different parameter spaces on the pile deformations.

The main limitations of the PISA model have also been listed by [8] as:

- The test piles had a maximum diameter of 2 m, which is still a factor 5 to upcoming monopiles with 10 m diameter. The L/D ratio of real monopiles can still differ from validated ratios. The inclusion of the 3D Model will help to reduce/overcome this limitation.
- The PISA model is only applicable to static loading. The implementation of cyclic loading is currently a matter of further research within the PISA consortium. Accumulated excess pore water pressure can not be accounted for in the current version.
- The approach is only considered to be validated for two soil types, sand and clay. All other types of soil have to be dealt with engineering judgement.
- Layered soils can not be described by the PISA model for the time being.
- Pile installation effects (impact driven or vibrated) are not covered.

Future studies and the application in commercial offshore projects will reveal if the PISA approach can overcome the limitations and be established as a new standard. In principal, the approach can be seen as a very successful step towards the improvement of the monopile design and will contribute to the cost reduction for the production of wind energy.

4.2 Comparison with Models for Cyclic Monopile Design

As noted before, the PISA approach is not applicable yet on the design for cyclic loading. Also the SOLDYN [29] studies did not conclude with a new approach for horizontally loaded monopiles. A detailed study on the comparison of different approaches for the design of cyclically loaded monopiles was performed in [37] and [38]. By comparing the horizontal deformations of a monopile subjected to 100,000 load cycles the notable differences in the assessed approaches were shown. The following approaches have been studied: the *p-y* model from API [3], the stiffness degradation model according to Achmus [1], the enhanced strain wedge model according to Taşan [36], the modified *p-y* approach after Dührkop [17] and the HCA based approach HIA. The results for different relative densities are shown in Fig. 16. Beside considerable differences in the total displacements between the various approaches, also the sensitivity on changes in the void ratio or the load regime were found model-dependent.

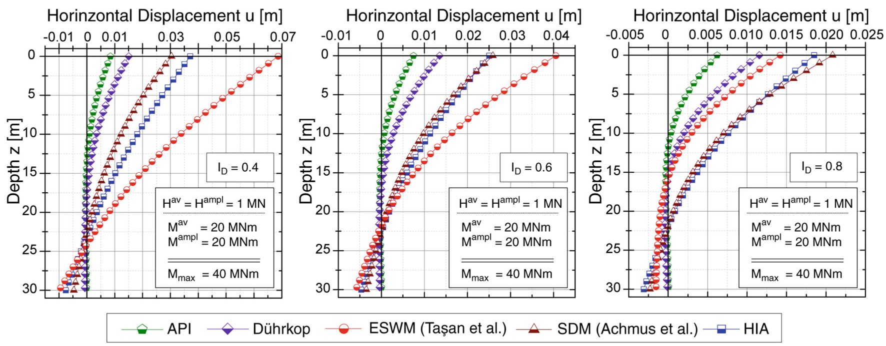

Fig. 16. Pile displacement after 100,000 load cycles obtained with different approaches covering the cyclic monopile design, for three different soil densities and the same loading.

These huge differences in the results demonstrated the need for validated and flexible approaches for the design of foundations for offshore wind turbines. By applying the HCA-based approach to the large-scale test foundation results and to the model tests on monopiles, such a validation was aimed for. Given the satisfying results for the HCA approach, the development of HCA-based engineering models for monopiles and shallow foundations was started [42]. However, the new tests on monopiles ([24, 32, 33]) with a more realistic *L*/*D* ratio give room for further validation not only with respect to the cyclic loading phase but also to static loading.

Summary
The design of foundations for offshore wind turbines remains a challenging task, especially because of the rapid changes and developments in turbine sizes and thus foundation sizes. The variety of studies on static and cyclic loading on monopiles was shown and set in relation to the HCA-based approach. The HCA-based approach showed good results while being applied to small-scale model tests on monopiles and to the prototype test of the gravity based Züblin foundation. Finally, the new PISA approach for the design of monopiles was discussed. First results of a comparison with established soil models like Hypoplasticity with Intergranular Strain showed some differences to the PISA approach which need further investigation. In principal, a combination of HCA-based engineering approaches like those proposed in [42] with popular approaches for the static design, like PISA, should be aimed for to further improve the design of foundations for offshore wind turbines.

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