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Geotechnical Robustness and Deformation Recovery Effects of Offshore Wind Turbine Foundations

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The serviceability limit of offshore wind turbines is reached at a certain permanent inclination of the structure. This may be caused by a strong storm event with a small number of high water waves inducing large cyclic loads. However, there is a possibility to at least partly revert the permanent inclination due to a rather regular exposure to wind and waves with smaller amplitudes.
This paper focuses on monopiles and on shallow foundations on dense to medium dense sands. Under certain conditions, both foundation types show a recovery of accumulated rotations. Presented are first experimental investigations of this phenomenon in small scale model tests for monopiles and for a circular gravity plate in dry sand under typical cyclic loads after an initial extreme loading.
On the basis of our tests we demonstrate how the behavior of the foundation significantly depends on the load type and the load intensity. Finite element models were developed to analyze the observed effects numerically. First results are presented as well as methods to determine the required parameters for the used so called “high-cycle-accumulation soil model”.

Cyclic Accumulation Model

For the FE calculations the hypoplastic model with intergranular strain [1] and the high-cycle accumulation (HCA) model proposed by Niemunis et al. [2] are used. For both models, the material constants for the fine sand were determined in laboratory tests. The calibration of the HCA model is briefly addressed in the following. The basic equations of the HCA model read:

$$\dot{\boldsymbol{\sigma}}' = \mathsf{E} : (\dot{\boldsymbol{\varepsilon}} - \dot{\boldsymbol{\varepsilon}}^{\mathrm{acc}} - \dot{\boldsymbol{\varepsilon}}^{\mathrm{pl}}) \qquad \text{with} \qquad \dot{\boldsymbol{\varepsilon}}^{\mathrm{acc}} = \dot{\varepsilon}^{\mathrm{acc}} \mathbf{m} \tag{1}$$

wherein $\dot{\boldsymbol{\sigma}}'$ is the rate of the effective stress $\boldsymbol{\sigma}'$, $\dot{\boldsymbol{\varepsilon}}$ is the strain rate, $\dot{\boldsymbol{\varepsilon}}^{\mathrm{acc}}$ the given accumulation rate, $\dot{\boldsymbol{\varepsilon}}^{\mathrm{pl}}$ a plastic strain rate (for stress paths touching the yield surface) and E an isotropic elastic stiffness. The accumulation rate $\dot{\boldsymbol{\varepsilon}}^{\mathrm{acc}}$ is prescribed as the product of the scalar *intensity* of accumulation $\dot{\varepsilon}^{\mathrm{acc}}$ and the tensorial *direction* of accumulation **m**.

Two elastic constants are sufficient to quantify the elastic stiffness E, for example bulk modulus K and Poisson’s ratio ν. The bulk modulus K can be determined from a comparison of drained and undrained cyclic triaxial tests with similar initial conditions [4,6]. K is calculated as the ratio of the rate of volumetric strain accumulation in the drained test and the rate of pore pressure accumulation in the undrained test. The test series on fine sand revealed that the pressure-dependence of K is well described by $K = A\ (p_{\mathrm{atm}})^{1-n}\ p^n$ with $p_{\mathrm{atm}} = 100$ kPa, $A = 467$ and $n = 0.46$. Poisson’s ratio ν can be chosen based on the average effective stress path in a strain-controlled undrained cyclic test with anisotropic consolidation stresses (Figure 1, a value of $\nu = 0.32$ was found to be appropriate for the fine sand [5,6]).
The critical friction angle $\varphi_c = 33.1°$ used in the equations for the direction of accumulation **m** was obtained as the inclination of a pluviated cone of sand. The seven material constants C_{ampl}, C_e, C_p, C_Y, C_{N1}, C_{N2} and C_{N3} describing the

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dependence of the accumulation intensity on strain amplitude, void ratio, average stress and cyclic preloading can be determined from drained cyclic triaxial tests. As a simplified procedure, the parameters can be estimated based on granulometric properties. The correlations of the HCA parameters with mean grain size d_{50} and the coefficient of uniformity $C_u = d_{60}/d_{10}$ proposed by Wichtmann et al. [7] are based on more than 350 drained cyclic triaxial tests on 22 quartz sands with different grain size distribution curves. For the fine sand used in the model tests, the parameters C_{ampl} = 1.70, C_p = 0.47 and C_Y = 2.16 were estimated from these correlations while C_e = 0.57, C_{N1}= 2.18 * 10^{-4}, C_{N2} = 0.31 and C_{N3} = 2.24 * 10^{-5} were derived from drained cyclic triaxial tests with different initial relative densities I_{D0} (Figure 2).

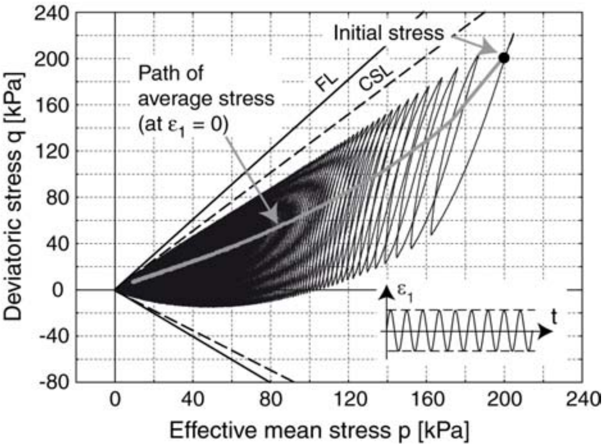

Figure 1: Effective stress path in a strain-controlled undrained cyclic triaxial test on fine sand [5]

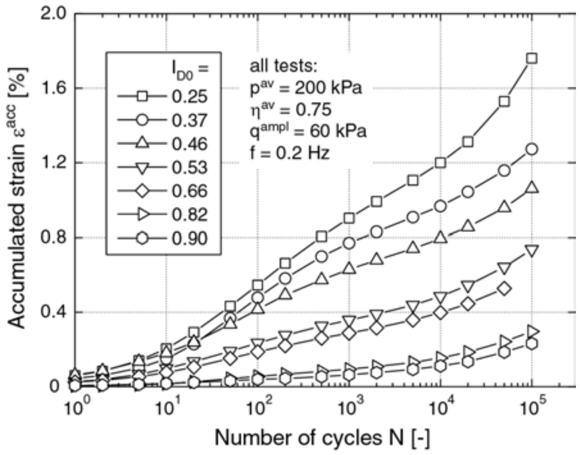

Figure 2: Accumulated strain in drained cyclic triaxial tests with different initial relative densities on fine sand

Model tests

Model tests on monopiles were performed with a setup consisting of a steel container (0.97 m diameter, 1.40 m height) filled with sand. The sand is brought in dry and in 15 vibro-compacted layers, which results in relative densities of about 90%. The test piles were thin-walled aluminium and PVC tubes, driven with a pneumatic hammer. With embedment lengths of 0,8 to 1,0 m and diameters of 30 to 63 mm, they represent monopiles scaled 1:50. After the pile was in place, the measurement and loading frames were placed on top. The loading device works pneumatically (force controlled) in two dimensions (Figure 3). Displacements were measured as horizontal pile deflections in two different heights. The measurement device works synchronised to the loading device in order to reduce the data amount during long term experiments.

Typical loading sequences comprised 500 symmetric pre-cycles and three consecutive series each consisting of an ULS load H^{max} (representing extreme storms), an unloading to H^{av} and 10000 SLS load cycles with amplitude H^{ampl} (representing normal turbine operation). The corresponding displacements u^{max}, u^{av} and the accumulated ones u^{acc} due to the cyclic load H^{ampl} were recorded. H^{max} and H^{ampl} were varied in the tests (Figure 4). As expected, Δu^{max} and Δu^{av} are increased with larger preload H^{max}. The cyclic regime leads to a reduction of the accumulated deformation Δu^{acc} and the pile rotates back against the previous H^{max} and Δu^{av} direction. We define the back rotation $\Theta = \Delta u^{acc}/\Delta u^{av}$ as a measure for the observed "self healing effect". Θ is in the order of 5-15 % in the first cyclic load package and increases to 35-75 % in the third package. There is no clear dependence on H^{max}. However, Θ increases with H^{ampl} and can reach almost 100 %. Accumulations are obvious during the first hundred cycles already, but they continue steadily beyond 10^4 or 10^5 cycles.

Any increase of the pile length beyond the static requirement according to API or DNV has no effect. An increase in pile diameter with unchanged bending stiffness, however, accelerates the back rotation. The soil on windward side is loosened more during H^{max} and less after unloading to H^{av}. Because the windward side experiences stress concentration during unloading, also the relaxation is higher during the subsequent small cycles. A pile with an undeformed toe rotates back, driven by its own elastic stiffness.

Further tests on shallow foundations were performed with the same setup described above. The foundation model was placed on the sand surface prior to the measurement and loading frames. As a foundation model, we used a round plate of 0.26 m diameter without skirts, representing a shallow foundation scaled 1:100. The plate was designed for a maximum horizontal force of about 20 N and a moment of about 8 Nm, in order to obtain stability against overturning.

The foundation plate is equipped with a 0.41 m high flagpole for the force application. For the measurement of displacements and rotations a high resolution (0.1 µm) 3D displacement measuring device was developed.
In first tests, octagonal pre-cycles were performed, followed by three sequences of a single ULS-load H^{max} and 10000 SLS-cycles H^{ampl}. During the pre-cycles the foundation plate rotates in a certain direction (ca. 45°). This direction is somehow random as it is caused by small void ratio inhomogeneities of the soil under the foundation plate. The ULS-loads (2nd and 3rd) partly revert this inclination. During the SLS-cycles the plate rotates back against the direction taken in the pre-cycles.

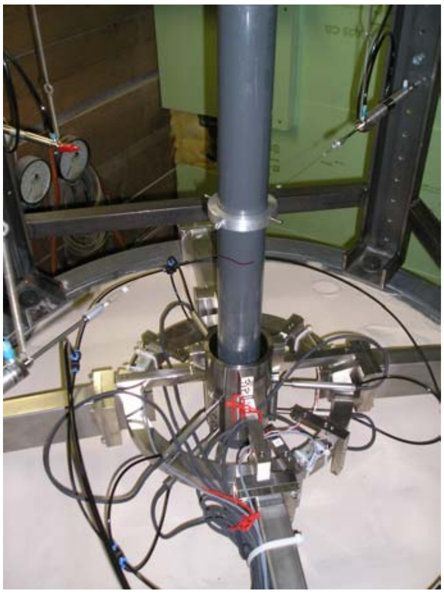

Figure 3:Ttest pile with loading and measurement device

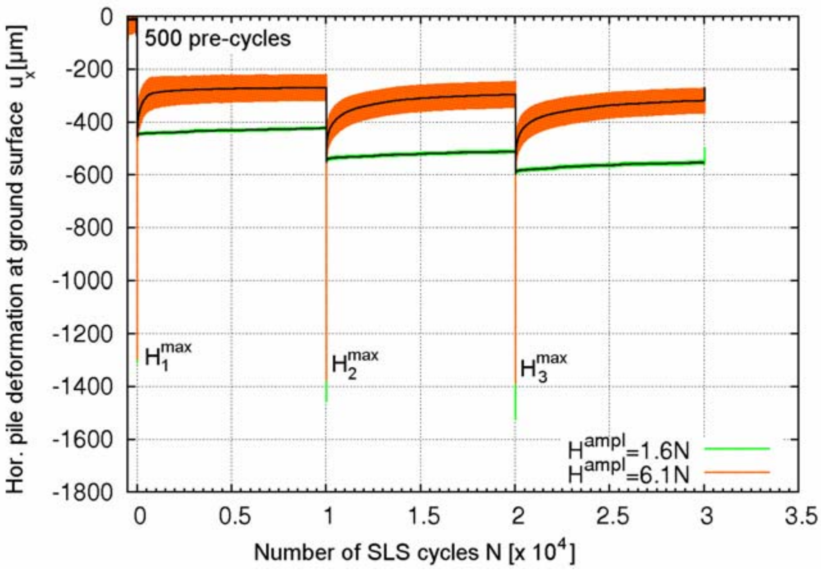

Figure 4: Accumulating horizontal pile deformation at ground surface for two H^{ampl} vs. number of cycles with three interspersed H^{max} loads (60 N)

Finite element models

The commercial finite element program ABAQUS was used to analyze the observed experimental effects. The explicit description of some 10000 quasistatic load steps alone would not be possible due to munerical errors. Beside the high calculation time, these numerical errors could exceed the material effects. Hypoplasticity with intergranular strain [1] is therefore combined with the high cycle accumulation model proposed by Niemunis et al. [2]. These constitutive models were implemented via a user subroutine (UMAT) for the description of the material's mechanical behavior.
Like the model tests, calculations for shallow foundations were performed under drained conditions using a 3D model with around 60000 elements. The small stress level under the circular model plate (1 to 5 kPa) was expected to cause numerical problems as well as uncertainty due to the applicability of the used material parameters for this small stress level. The material parameters for the above mentioned constitutive models were gained in the range of 50 to 200 kPa [3]. Thus, the first analysis was performed with models scaled up to prototype scale. So far, only the pre-cycles, the first ULS load H^{max} = 10 N and the first sequence of 10000 load cycles H^{ampl} = 3 N (peak-peak) could be simulated. Figure 5 depicts the settlements of some pre-model tests with an octagonal model plate, where only the pre-cycles before the first ULS load were varied. The results show a considerable scatter and compared to the experiments, the calculated settlements are much too small. This might be a shortcoming to describe the small stresses in the model tests since [3] showed that the accumulation intensity increases with smaller stresses. Besides the model tests, also the numerical calculations show some back-rotation effects under certain circumstances, at least qualitatively.

A numerical 3D model with 10320 elements has also been employed for the monopile. The quasistatic H^{max} loading can already be described quite well using a hypoplastic constitutive law, while the calculated accumulated deformation for H^{ampl} using the high-cycle-accumulation model presented here shows to be too small. As for the shallow foundation, this is attributed to the fact that the complete parameter set was not yet determined, especially not for the small pressure range of the model tests.

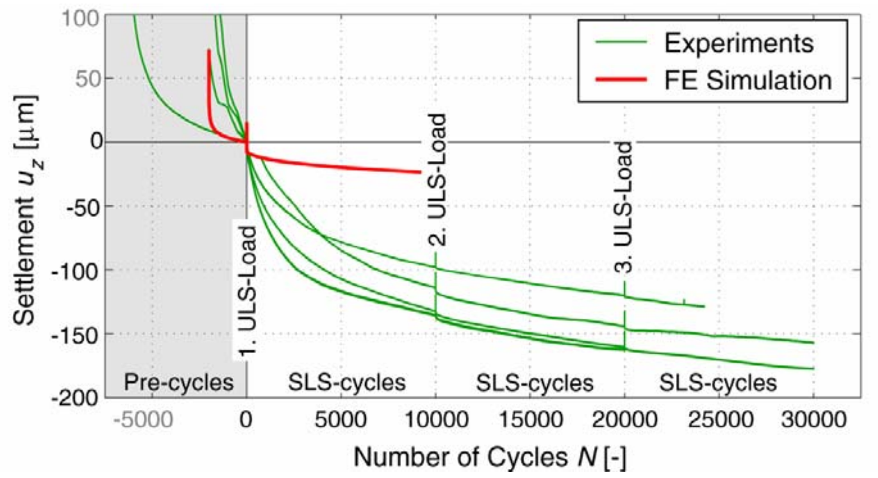

Figure 5. Comparison of the settlements in the pre-model tests and the numerical simulation

Settlements of two adjacent strip foundations have been calculated for cyclic and monotonic loading respectively with a randomly generated soil density distribution. After $N = 10^5$ load cycles with a small amplitude, the calculated differential settlement is up to 4.5 times larger than in the static case. A strong dependence of average stress σ^{av} on the differential settlements is observed, but neither a dependence on amplitude stress σ^{ampl} nor on the correlation length was obtained.

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