PREDICTION OF MONOPILE DEFORMATION UNDER HIGH CYCLIC LATERAL LOADING

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1 Summary

The design of monopile foundations for offshore wind energy converters is characterized by difficult environmental conditions. Beside extreme load events like a storm in combination with the design wave the permanently acting forces due to smaller waves during operational conditions have to be considered. A high number of load cycles influences the structural behaviour resulting in increasing deformations and pile section forces moving to deeper regions. These effects have to be included within the calculation of the deformations of the monopile over the whole lifetime. Computations using the Strain-Wedge method show these phenomena and provide an engineering approach to predict the monopile behaviour considering cyclic loading. Another important component of the approach is a sufficient testing of the soil to analyse the element behaviour under cyclic loading.

2 Introduction

Foundations for offshore wind energy converters are designed for a lifetime of about 50 years. During this time different loads act on the converter, which all have a stochastic character. Beside extreme load events phases may occur with a relatively small load amplitude but acting over a very large period. Over the whole lifetime of the structure this can add up to 150 Mio. of cycles. In engineering practice a large number of cycles is called high cyclic loading. The loads are transferred by the structure to the subsoil, which shows a nonlinear stress-strain behaviour. Cyclic loading may provoke a change in the soil behaviour and, with that, in the structural behaviour resulting in an unallowable inclination or even a loss of structural stability. Hence, the current design philosophy follows a design on the safe side, but from the economic point of view the structures need to be optimised. The present paper proposes a possible engineering approach taking the special features of the structure-soil interaction into account.

3 Loading of the Monopile

The loading of the monopile foundation of an offshore wind energy converter includes the weight of the structure, which is the only dead load and wind, waves and currents which act in horizontal direction. Due to their stochastic character it is not obvious to quantify their magnitude separately. Furthermore, loads like for example

-ice loads (mainly in the Baltic Sea)
-ship impact

must be taken into account, but they are not object of this paper.

The stochastic forces and moments vary in their magnitude and directions over the time. For an engineering analysis stochastic loads, here in particular the wave loads, can be summarized as cyclic load collectives. They are characterized by a certain number of cycles and the related load amplitude. Experiments on model piles and soil specimens show a significant influence on the behaviour of the system depending on the applied cyclic loading [1]. The reason for the changing soil response under cyclic loading is the nonlinear stress-strain-behaviour of the soil.

The cyclic loading causes a deformation behaviour of the pile-soil-system which can be classified into three specific modes according to Fig. 1. If the cyclic load amplitude exceeds a certain threshold value the pile deflections continuously increase leading to a progressive failure of the system. The deflections are progressively accumulated because of an accompanying softening of the soil which causes the strain rate to increase. If a smaller cyclic load amplitude acts on the system the deformation behaviour may be different. Due to the lower stress level the soil may be able to sustain the load and the rate of plastic deformations becomes continuously smaller resulting in a stabilization of the pile. Depending on the number of cycles the plastic deformations may exceed an allowable maximum and affect the serviceability of the structure. In the third case a cyclic shakedown may be observed. Here, the strain rate decrease until a steady state is reached in which almost only elastic strains are induced.

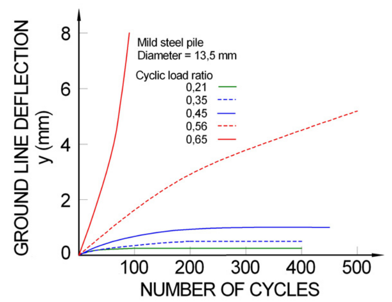

Fig. 1: Development of the pile head deflection under cyclic loading [1]

4 Monopile-Soil Interaction

4.1 Soil Behaviour – Element Testing

Before the monopile-soil interaction is analysed in detail, the soil behaviour under cyclic loading must be known. For this purpose it is reasonable to conduct cyclic triaxial tests. To adjust the stress level in the test, the real stress level around the pile must be taken into account. If stress measurements in the near field of a monopile are not available, it is possible to simulate the interaction by finite-element-analysis. For example, the stress distribution over time in three different depths in the vicinity of a monopile is shown in Fig. 2. In a sufficient depth the influence of the cyclic loading is only marginal.

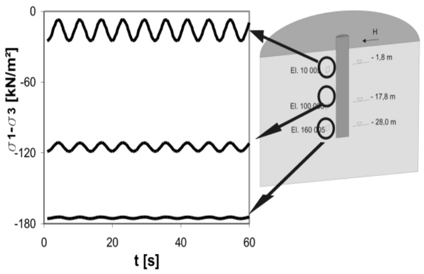

Fig.2: Cyclic stress distribution in the subsoil in the near field of a monopile [2]

For a unified evaluation of the triaxial tests (Fig. 3) a cyclic load ratio CLR is defined depending on the applied stresses by:

$$\mathbf{CLR} = \frac{(\sigma_1 - \sigma_3)_{\mathbf{cycl.}}}{(\sigma_1 - \sigma_3)_{\mathbf{max,\ stat.}}} \qquad (1)$$

with: CLR…cyclic load ratio
$(\sigma_1\text{-}\sigma_3)_{cycl}$ …cyclic deviatoric stress
$(\sigma_1\text{-}\sigma_3)_{max,stat}$…maximum deviatoric stress in static triaxial test

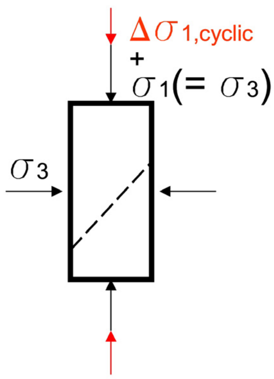

Fig 3: Stress conditions in a soil specimen

From equation (1) it can be seen that a static reference test is necessary in which the maximum deviatoric stress is determined. In Fig. 4 the result of a cyclic triaxial test with the number of cycles on the abscissa and the accumulated plastic strain on the ordinate (black curve) is exemplarily shown. The procedure performed in this test is called multistage test, in which the specimen is subsequently loaded by an increasing CLR. Each load collective with a certain CLR was been run for 10.000 cycles. The test result in Fig. 4 indicates, that the soil specimen is able to stabilise even after the 5th cyclic load ratio. However, once the 6th cyclic load ratio is applied the soil specimen is overloaded and a progressive failure occurs. The red curve shows the derivation of the plastic strains, hence the strain increments. The plastic strain increments become zero if the soil stabilises.

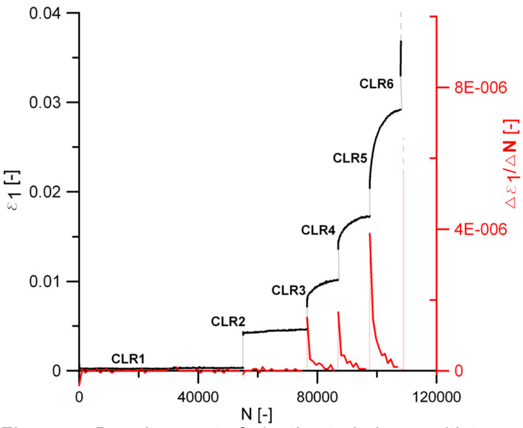

Fig. 4: Development of plastic strain in a multistage cyclic triaxial test

With the test results the soil behaviour under cyclic loading can be evaluated in view of:

1. the stress level, from which plastic strains start to accumulate (lower bound),
2. the stress level, from which progressive failure occurs (upper bound) and
3. the distribution of the plastic strain between the upper and the lower bound.

The so called Ideal-Young's modulus derived from the tests shows a similar distribution as the plastic strains. It is inversely proportional to the accumulated strain and may be defined according to equation 2:

$$\mathbf{E_{ideal}(CLR,N)} = \frac{\Delta(\sigma_3 - \sigma_1)}{\varepsilon_{1,pl}} \qquad (2)$$

This definition follows the traditional form of the Young's modulus. However, in (2) elastic strains have been substituted by plastic strains as shown in Fig. 5.

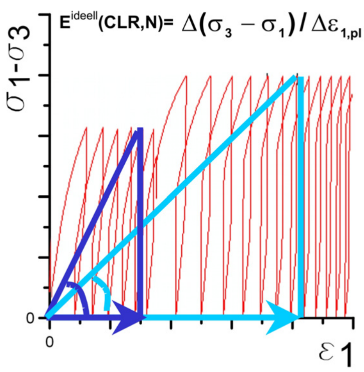

Fig. 5: Definition of the "Ideal-Young's modulus"

4.2 Structural Behaviour

Only a limited region in the vicinity of the monopile is influenced during cyclic loading (Fig. 6). This region shows plastic deformations, hence the cyclic load ratio reaches the critical value of the lower bound. This zone reaches up to a soil depth which is approximately equal to the pile diameter. This assumption is supported by the fact that the highest contact pressure between pile and soil due to horizontal loading is located here [3].

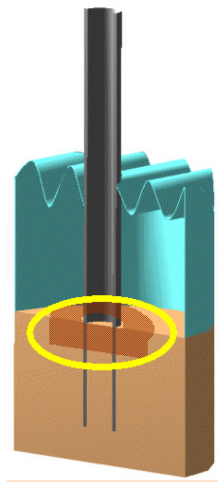

Fig 6: Influence zone in the vicinity of the pile

The dimensions of this influence zone may vary depending on the dimensions and the material of the pile in relation to the stiffness of the soil.

Another effect is the generation of excess pore water pressures due to a compression of the soil matrix which accompanies the accumulation of plastic strains currently. This phenomenon may result in decreasing effective stresses and forces the soil degradation. However, more detailed investigations are conducted.

4.3 Method of Calculation

The calculation of pile deformations and section forces requires an engineering concept considering the above mentioned factors. Beside the finite-element-method (FEM) the beam on elastic foundation approach (BEF) is a well known and established analysis procedure. Together with the BEF the p-y procedure describing the nonlinear pile-soil-interaction is recommended by the American Petroleum Institute [4]. Empirical factors are used to define the p-y-curves which have been developed from pile load tests. The general validity of the p-y-curves related to monopiles with large diameters has already been evaluated, e.g. Wiemann [5] or Juirnarongrit & Ashford [6]. P-y-curves are also available for cyclic loading, but they cannot predict the pile behaviour over a service life of several years. The existing methods were usually developed for a very small number of cycles. Only in some cases more than 500 cycles have been analysed [7]. For this reason information from the cyclic triaxial tests should be used to predict the long term behaviour of the foundation. Hence, a procedure allowing the direct utilisation of the results from the cyclic tests has been developed. In this concept the Strain-Wedge-Model (SWM) is applied. The model has been presented by Norris and Ashour [8, 9] and relates the element behaviour to the behaviour of the pile-soil system, see also Tab. 1. Within this model the pile deflections are derived from a passive earth pressure wedge mobilised due to a strain which corresponds to the soil behaviour in a triaxial element test, see Fig. 7. If the results from a cyclic triaxial test are used, the computation gives the pile displacements depending on the number of cycles.

Tab. 1: Relations in the Strain-Wedge Model

Element behaviour **Soil**		**System behaviour** **Monopile**
Strain ε_1		Displacement y
Young's modulus E		Subgrade modulus k
Stress σ_1		Pile resistance p

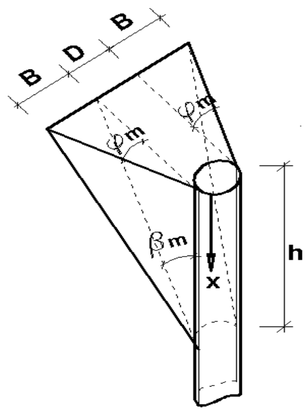

Fig. 7: Geometry of the Strain-Wedge-Model presented by Norris and Ashour [8, 9]

After calibrating the strain wedge model with a finite element analysis for the first load cycle a case study has been performed. In the process the degraded soil properties have been taken into account under utilisation of the results from the cyclic triaxial tests. Depending on the specific case this procedure can be used to assess the behaviour of the monopile of an offshore wind energy converter at the end of its lifetime.

4.4 Application of the Proposed Method

The exemplary application of the SWM under consideration of the soil degradation shows an increased pile head deflection, see Fig. 8 (left). Depending on the soil properties this effect can be more or less pronounced. Furthermore, due to the degradation in the upper soil layer the pile resistance and the subgrade modulus become smaller. To satisfy the conditions of equilibrium the pile resistance is mobilised in larger depths as indicated in Fig. 8 (right). The pile resistance p which is defined by force per pile length decreases in the area of the degraded soil and increases in the zone, where the cyclic load amplitude is nearly negligible.

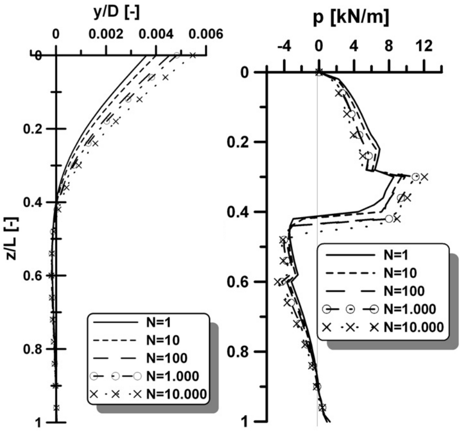

Fig. 8: Pile head deflection and pile resistance

Analysing the pile bending moments shows a movement of the maximum moment to a deeper area. Furthermore, due to the increased lever arm of the horizontal force the value of the maximum moment increases, see Fig. 9.

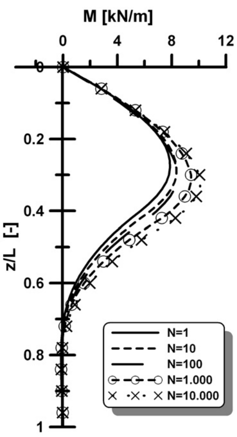

Fig. 9: Bending moment

5 Conclusions

An approach to predict the monopile deflection under high cyclic lateral loading based on cyclic triaxial tests has been presented. Within this approach the interrelation between the element behaviour of the soil and the structural behaviour described by the pile-soil interaction is modelled by the SWM. The accumulated plastic deformations due to cyclic loading have been determined taking the accumulated plastic strains from the cyclic triaxial tests into account. With that, the accumulated pile head deflections after a certain number of cycles can be determined. The results show that the resisting forces and moments move to deeper pile sections. Nevertheless comparative calculations still show small differences, see Fig. 10. As soon as measurements from offshore wind energy converters are available, the procedure must be verified to achieve an optimisation of the foundation.

6 Acknowledgement

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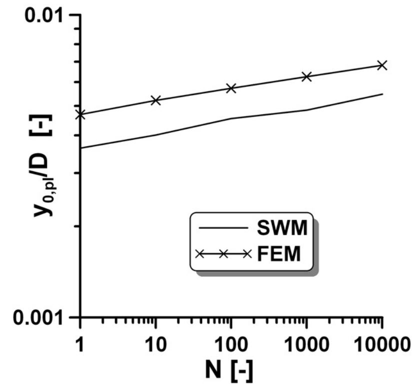

Fig. 10: Comparison between FEM and SWM for the pile head deflection

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