Behaviour of Monopile and Suction Bucket Foundation Systems for Offshore Wind Energy Plants

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ABSTRACT

The foundation of offshore wind energy plants plays an important role in the stability of these structures. One foundation concept which can be used in this field is the monopile concept. Here the foundation consists of one large-diameter pile (up to 8.0 m). Common design practice, e. g. according to API regulations, does not cover horizontally loaded piles of such dimensions. Thus, the soil-structure-interaction and the behaviour of these foundations have to be covered using numerical modelling. Suction bucket can be considered as an innovative foundation concept. This paper aims to investigate the behaviour of the monopile and the suction bucket under monotonous loading taking the interaction between the foundation system and the subsoil into account. A three-dimensional numerical model using the finite element method was developed. In this model the non-linear material behaviour of the subsoil is described using an elasto-plastic constitutive model. The results of the finite element simulations are presented and evaluated.

Keywords: Constitutive Model, Monopile, Suction Bucket, Numerical Modelling, Offshore Wind Energy Plants.

1 INTRODUCTION

The planned offshore wind parks in the German part of North and Baltic Sea will be constructed in water depths varying from approximately 15 to 40 m. By means of suitable foundation constructions, the large horizontal forces and bending moments resulting from wind and wave loading, must be economically and safely transferred to the sea ground. Monopile foundations can be used as one of these foundation types. In principle the monopile is an extension of the main tower into the soil under the sea bed (Fig. 1-left). This foundation method was already implemented in North and Baltic Sea, but only for wind energy plants in water depths of less than about 10 m. Its application is expected to be possible for water depths up to about 25 to 30 m. The diameters of such monopiles will vary between 6 and 8 m.

A suction bucket foundation consists of an upside down cylinder, which is pressed into the subsoil (Fig. 1 right). The bucket penetrates into the seabed partly by self-weight and partly by applied suction.

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Since wind energy plants are relatively sensitive to deformations, in particular tilting, it is really important to estimate these as exactly as possible. In this paper, the results of numerical investigations of the load-deformation behaviour of monopiles and suction buckets under static loads are presented.

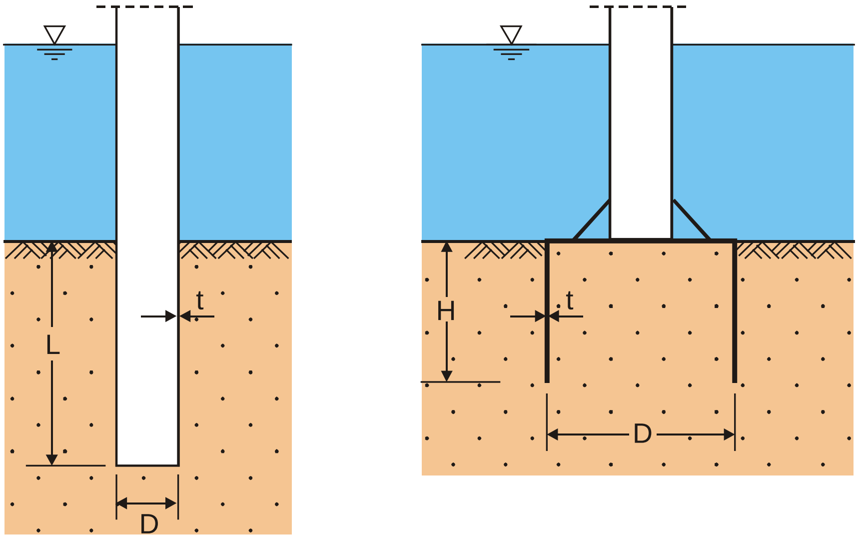

Fig. 1: Schematic view of monopile (left) and suction bucket foundation (right).

2 NUMERICAL MODELLING OF MONOPILE BEHAVIOUR UNDER STATIC MONOTONOUS LOADING

For the investigation of the behaviour of laterally loaded monopiles with large diameters, a three-dimensional (3-D) numerical model was developed. The computations were done using the finite element program system ABAQUS (Abaqus 2005). In order to carry out many calculations for varying boundary and loading conditions, a special computer system with parallel processor technology was used to minimize the computation time.
The aim of the investigation was to analyse the behaviour of a large monopile in principle and to check whether the API method can be used for such large piles. For that, an idealized homogeneous soil consisting of medium dense or dense sand was considered. A monopile diameter of D = 7.5 m and a wall thickness of 9 cm was assumed.
The elasto-plastic material law with Mohr-Coulomb failure criterion, provided in the ABAQUS program, was used. This material law was extended in the elastic range by a stress-dependency of the oedometric modulus of elasticity with the following equation:

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

Herein σ_{at} = 100 kN/m^2 is a reference stress and σ_m is the current mean principle stress in the regarded soil element. The parameter κ determines the soil stiffness at the reference stress state and the parameter λ rules the stress dependency of the soil stiffness (EAU 2004).
Table 1 shows the material parameters used to describe the behaviour of dense and medium dense sand. Concerning more details about the numerical modelling reference is made to Abdel-Rahman & Achmus (2005).

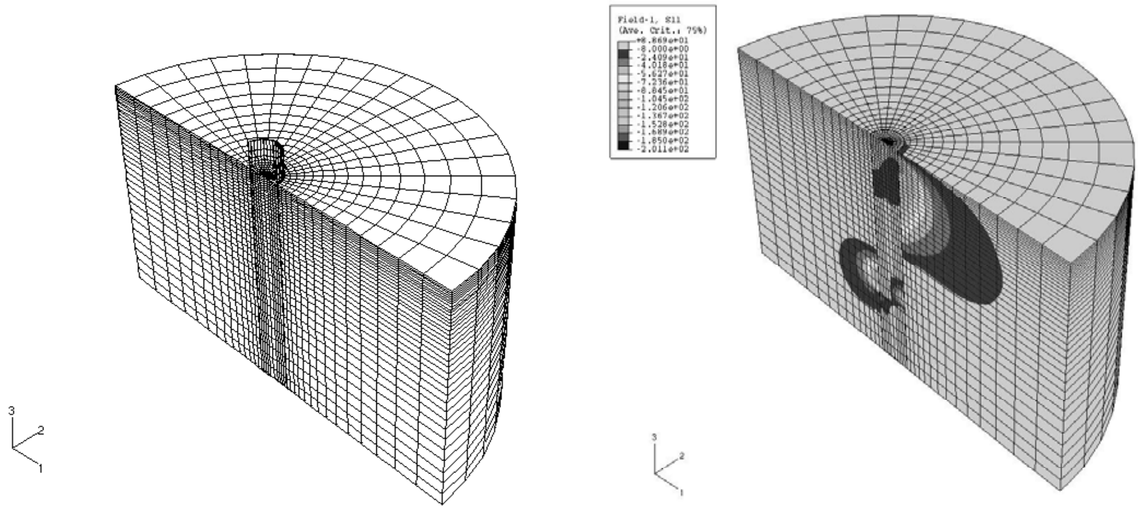

Fig. 2: Finite element mesh and horizontal bedding pressure (in the symmetry axis of the monopile) for D = 7.5 m, L = 30 m in dense sand, H = 8 MN, M = 240 MNm.

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

	dense	medium dense
Unit buoyant weight γ'	11.0 kN/m^3	11.0 kN/m^3
Oedometric stiffness parameter κ	600	400
Oedometric stiffness parameter λ	0.55	0.60
Poisson's ratio ν	0.25	0.25
Internal friction angle φ'	37.5°	35°
Dilation angle ψ	7.0°	5°
Cohesion c'	0.1 kN/m^2	0.1 kN/m^2

For a resultant horizontal load of 8 MN and a bending moment at sea-bed level of 240 MNm, which is in the order of a possible design load for the considered large water depths, the horizontal (bedding) stresses acting on the pile in the symmetry plane are shown in Fig. 2, right. The characteristic loading behaviour of the pile with bedding stresses of opposite sign above and below a point of rotation can be seen clearly. For the considered case the point of rotation lies about 22 m below sea bed.

For a specific design problem force-head displacement and force-head rotation curves can be helpful, because especially the limitation of head rotation is of importance for the serviceability of the wind energy plant. As an example, such curves are given for D = 7.5 m, L = 30 m, dense sand in Figure 3. The comparison with API results (Lpile 2000) verifies that this method gives lower deformation than finite element calculations.
In case of the horizontal displacement (w) the deviation range between 50% and 100%. Similar results are obvious for the rotation (ϕ) having a deviation between 20% till 40%.
Of course, also the FE results do not necessarily represent exactly the true pile behaviour because assumptions have to be made concerning initial stress state and material behaviour and have thus to be checked. But, the findings give rise to the conclusion that the API method for large-diameter piles should be used with great care (refer to Achmus & Abdel-Rahman 2005).

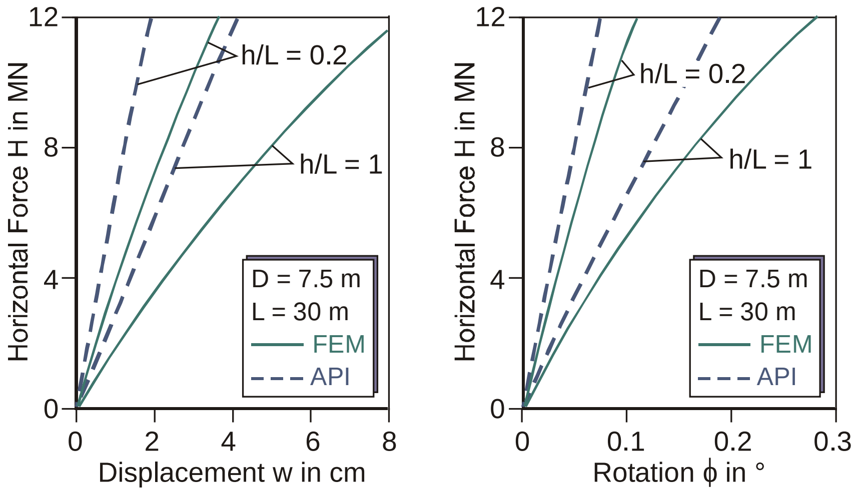

Fig. 3: Exemplary comparison of the pile deflection according to API method and numerical simulation for monopiles in dense sand ($h = M/H$).

Regarding the parametric studies, different pile diameters *D*, different pile lengths *L* and different heights of point of load application *h* were numerically simulated. Diagrams representing the pile head displacement *w* and the pile head rotation ϕ at sea-bed level as a function of the horizontal load were determined.
It is evident from Figure 3 that the load-deflection relationships are only slightly curved. Thus, at least for a certain load range, the behaviour can be described by the following parameters, which may be interpreted as integral stiffness parameters:

$$C_w = \frac{H}{w} \tag{2}$$

$$C_\phi = \frac{H}{\phi} \tag{3}$$

$C_w(h)$ and $C_\phi(h)$ diagrams can be derived by evaluation of a number of numerical calculations and can give a good overview on the behaviour of monopiles with different diameters and lengths.
In Figure 4 such diagrams are given for the case of monopiles in dense and medium dense sand. The integral stiffness values were calculated for a load of H = 8 MN.
The results given indicate that an increase of the embedded pile length from 20 to 30 m significantly increases the integral stiffnesses nearly by the factor 2. Thus, the deformations for a given load are nearly halved by lengthening the pile by 10 m.
An increase of the pile diameter from 5 to 7.5 m has a similar effect. In this case, the integral stiffnesses are at least doubled, i. e. the pile deformations at sea bed level are more than halved.
For a specific site similar or comparable soil profile, a pre-dimensioning of a monopile foundation for static load design can be carried out on the basis of these diagrams (Fig. 4).

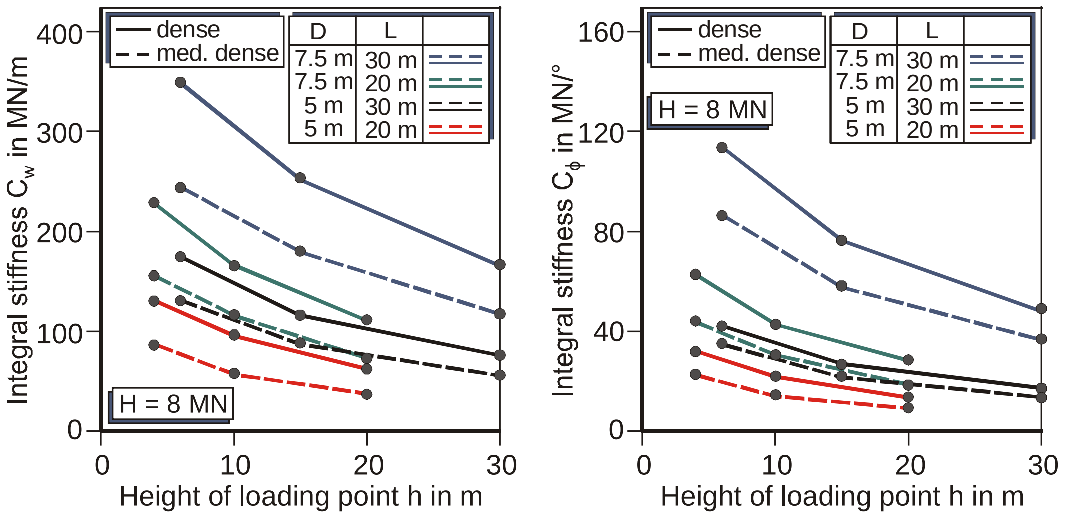

Figure 4: Integral stiffness diagrams for monopiles in dense and medium dense sand.

3 SUCTION BUCKET FOUNDATION

The suction bucket was developed from the suction caisson foundation already used in the offshore technology (Ibsen et al. 2004). In principle its behaviour can be considered as a combination of a gravity base and pile foundation systems. For installation an underpressure is applied in the cavity between the top plate and the seabed (Fig. 5).

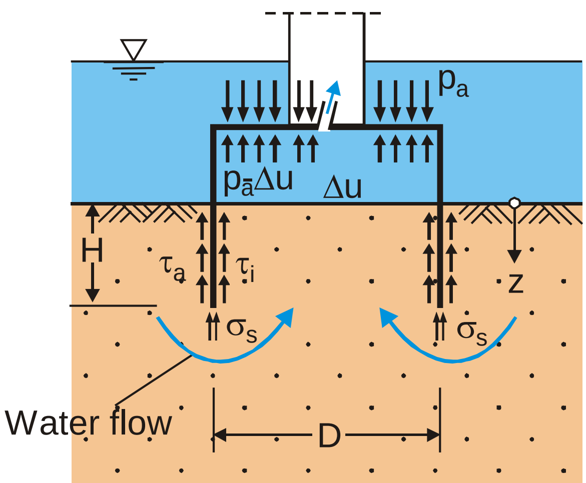

Figure 5: Suction bucket during installation

The mobilized penetration force results from the actual negative pressure multiplied with the internal cross-section area of the bucket plus the dead weight of the bucket foundation. The penetration resistance results from the skin friction mobilised along the outside and inside surface of the bucket as well as the base resistance at the bucket sleeves' toe.

For sandy soils the soil resistances can be calculated as for vertically loaded piles in accordance with API [3]. Feld [6] suggests a set of correction factors to compensate the effect of the water flow produced by the negative pressure during the penetration procedure of the bucket into the seabed.

According to the water flow from outside into the bucket it could come to large hydraulic gradients which cause a hydraulic shear failure of the soil in the bucket. This would lead to a loosening of the subsoil and to uncontrolled penetration of the bucket. Therefore a permissible underpressure, which is dependent on the current penetration depth, may not be exceeded during installation.

According to our computations it results that in homogeneous medium dense sandy soil for a bucket diameter of 15 m a penetration depth of ca. 10 to 13 m and for a bucket diameter of 20 m of ca. 13 to 17 m can be realized (Fig. 6). Since so far no experiences are present, a penetration depth of 8.0 m was used for bucket diameter of 15.0 m and a penetration depth of 10.0 m was used in numerical calculations for bucket diameter of 20.0 m.

For suction buckets with these dimensions in a sandy soil the load-deformation behaviour was modelled in the same way as for the monopiles with finite element computations.

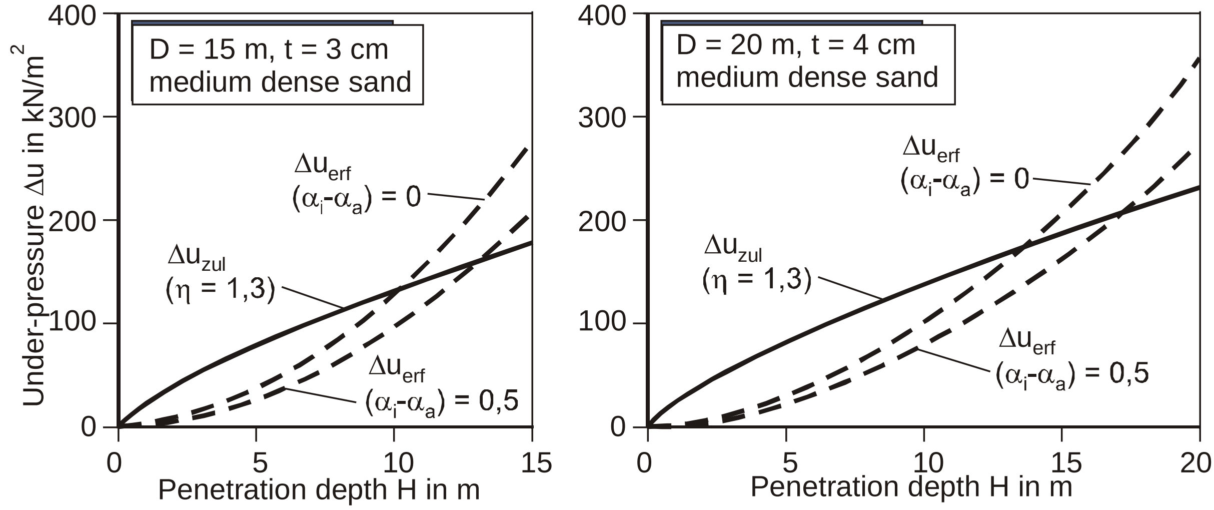

Figure 6: Relation between required and admissible suction pressure and the embedded length of the suction bucket

4 NUMERICAL MODELLING OF SUCTION BUCKET BEHAVIOUR UNDER STATIC MONOTONOUS LOADING

For the investigation of the behavior of suction buckets a three-dimensional finite element model using the program system ABAQUS (abaqus 2005) was developed. Both the bucket and the soil were modelled with volume elements. Due to the symmetric loading condition only a half-cylinder representing the sub-soil and the bucket could be considered. The discretized model area had a radius of at least three times the bucket diameter. The bottom boundary of the model was extended twice the bucket diameter below the toe of the bucket. With these model dimensions the calculated behaviour of the bucket is not significantly influenced by the boundaries (Fig. 7).

For reason of comparison, the same material model and its parameters used for the monopile calculation were used for the bucket computations (s. Table 1).

The following models were examined and modelled:

i) a bucket with an outside diameter of D = 15.0 m, a wall thickness of t = 3 cm and an embedded length in homogeneous, medium dense sand of H = 8 m.

ii) a bucket with an outside diameter of D = 20.0 m, a wall thickness of t = 4 cm and an embedded length in homogeneous, medium dense sand of H = 10 m.

The finite element calculations were executed stepwise. At first, for the generation of the initial stress state the whole model area is discretized using soil elements only. Subsequently, the bucket is generated by replacing the soil elements located at the bucket position by steel elements and activating the contact conditions between both of them.
Then the vertical load representing the own weight of the tower and the turbine is applied. Finally the horizontal load resulting from wind and wave loads is applied incrementally.

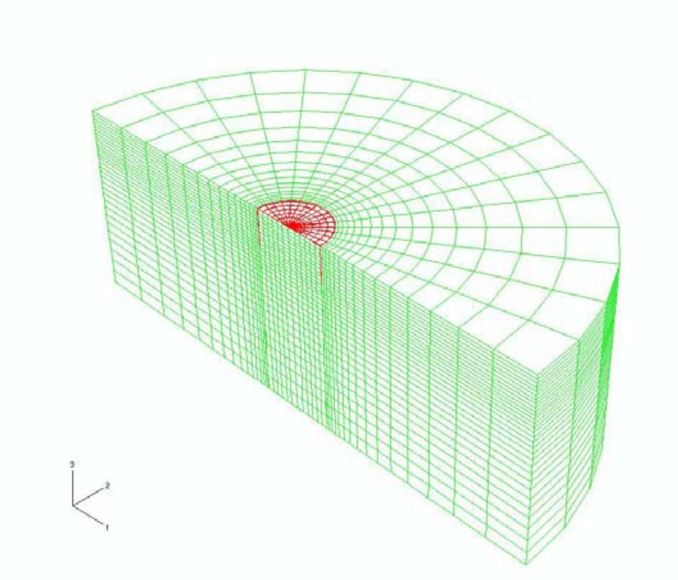

Figure 7: Finite Element Mesh for the suction bucket foundation.

The deformations and in particular the rotation of the foundation construction and thus the tower are of special importance for the design of wind power plants, since a trouble free operation is only secured under relatively small rotation. Therefore the main computation results are presented as the horizontal displacement (w) and the rotation of the bucket (ϕ) at the connection with the main tower.
The upper bucket plate was modelled as rigid to take the stiffening plates connecting the bucket to the tower into account. Therefore the presented results are the displacement and the rotation of the top bucket plate.
The Figure 8 shows the horizontal displacement and the rotation of the bucket construction at seabed level as a function of the horizontal loading applied to the bucket. For comparison the results for monopiles, similarly computed with Abaqus 6.5, are also depicted.
It is evident that suction bucket foundations behave under smaller load levels as stiff or even stiffer than alternatively usable monopiles. However, under higher loads the deformations increase strongly, what can be explained with the much smaller embedded length of the bucket compared to the monopile. The bearing capacity regarding the horizontal displacement

and rotation of this foundation under horizontal loads is smaller than for monopile foundation systems.
According to these results, suction buckets surely represent an alternative solution to monopiles. Under higher loads, i.e. for example in case of large water depths, monopiles are to be preferred in order to achieve the required limitation of deformations of the foundation construction.

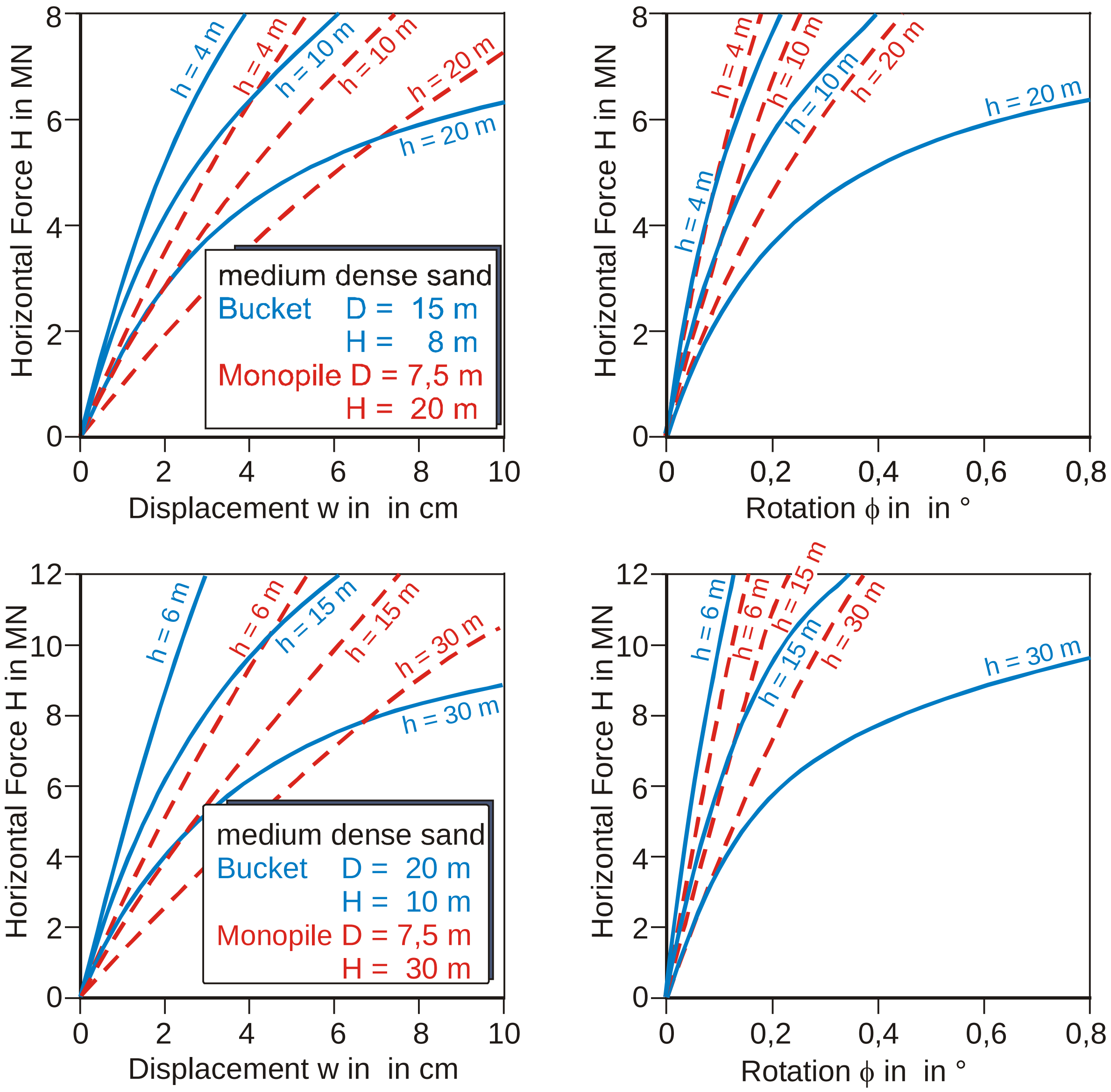

Figure 8: Load-deformation curves for suction bucket foundations in medium dense sand compared to monopile foundations.

5 CONCLUSIONS

The use of the API method for the computation of the deformations of large-diameter monopile foundations for offshore wind energy plants cannot be generally recommended. For static loads, numerical investigations are recommended at present, as they were presented in this paper. Of course, such investigations are complex and time-consuming. For preliminary design steps diagrams can be helpful, which allow a simple determination of the approximate pile deformations to be expected for a specific case.

Suction buckets behave under smaller loads stiffer than monopiles. However, under higher loads, as they are to be expected with wind energy plants in large water depths, the deformations increase strongly and are clearly larger than for monopile foundations.

Suction buckets might thus be applicable rather for moderate water depths. For larger water depths like 30 m and more other offshore foundation constructions such as monopiles or tripods and jackets seem more suitable.

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