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SCALE EFFECTS IN LATERAL LOAD RESPONSE OF LARGE DIAMETER MONOPILES

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ABSTRACT

Monopile foundations are frequently used for offshore wind energy converters. These piles are highly laterally loaded structures with large horizontal forces and bending moments. Due to the harsh environmental conditions in the southern North Sea diameters of 4 to 8 m are required to maintain serviceability. In common practice smaller laterally loaded pipe piles are designed using the well-known p-y-method, in which the pile-soil stiffness is considered by nonlinear p-y-curves derived from field tests. An alternative design method is the strain wedge method in which the pile response is derived from the stress-strain relationship of the soil assuming a certain failure zone ahead of the pile. In the present paper, the design of a large diameter monopile foundation for typical loading conditions is presented. The pile response in cohesionless soil determined by the p-y method and the strain wedge method is compared with a finite element (FE) analysis with respect to scale effects when extrapolated from commonly used pipe pile diameters to large size monopiles.

INTRODUCTION

Monopiles frequently have been installed as foundations for offshore wind energy converters all over Europe, e. g. at Horns Rev in Denmark or Arklow Bank in Ireland. Beside its simplicity, the major advantage of monopiles is that the loading due to wave, currents and ice can be clearly defined because of the simple shape of the foundation. Another aspect is the limited occupied footprint, which is favourable for the ecological acceptance of an offshore wind farm. The environmental conditions in the German part of the southern North Sea are extremely harsh, characterized by high wave heights and high wind speeds as shown in Table 1.

Table 1. Environmental conditions in the southern North Sea

Wind:	
Hub-height 50-year extreme 10min mean wind	50.0 m/s
Hub-height 50-year extreme 5s gust	60.0 m/s
Water depth:	
Mean water depth	35.0 m
50–year extreme water depth	41.0 m
Wave & currents:	
50–year maximum wave height H_{max}	22.3 m
Related wave period T	14.5 s
50–year tidal current surface velocity	1.71 m/s
50–year storm surge current surface velocity	0.43 m/s

Assuming an offshore wind energy converter with a rated power of 5 MW, a hub-height of about 95 m above still-water-level and a rotor of 125 m diameter these environmental conditions lead to an approximate quasi-static loading at mud line as summarized in Table 2 (Lesny and Wiemann 2005). The resultant loading is dominated by the wave loading, which causes extremely high bending moments controlling the foundation design.

Table 2. Quasi-static loading at mud line for a 5-MW turbine

		North Sea Conditions
Vertical Load V	[MN]	35
Horizontal Load H	[MN]	16
Bending Moment M	[MNm]	562
Torsional Moment M_T	[MNm]	4

These loading conditions require pile diameters of 6 m and more which are beyond common experience in design and installation. Current design procedures often are of semi-empirical nature and only roughly consider the effect of the diameter on the pile behaviour. The p-y method, for example, has been derived from field test results with pile diameters of up to 0.60 m. By the experiences gained over many years this method may be acceptable for piles with diameters of up to 1 or 2 m. Though piles with larger diameters (up to 4.5 m) have been recently designed and installed (Menck, 2006), no experimental data or longterm pile behaviour experience exists. Similarly, the strain wedge (SW) model developed by Norris (1986) has been verified only for conventional pile diameters. Hence, the extrapolation of these methods to large diameter monopiles for offshore wind energy converters requires an evaluation of the scale effects for the lateral load response.

In the following sections, the design procedures according to the p-y method and the SW model are shortly described. The response of laterally loaded piles of different sizes in homogenous, non-cohesive soil from a FE analysis is compared to the results from the two conventional design methods.

P-Y METHOD

The p-y method is the standard procedure for the design of laterally loaded piles as recommended in the relevant guidelines for offshore engineering (e. g. API 2000, DNV 2004). In this approach, the pile-soil system is modeled as a beam on elastic foundation (BEF) with springs acting independently of each other according to Winkler's hypothesis. Hence, the subgrade reaction at the pile in a certain depth is not influenced by the pile displacements at any other depths. Figure 1 shows the Winkler model and the composed and non-steady p-y curves for sand as developed by Reese et al. (1974).

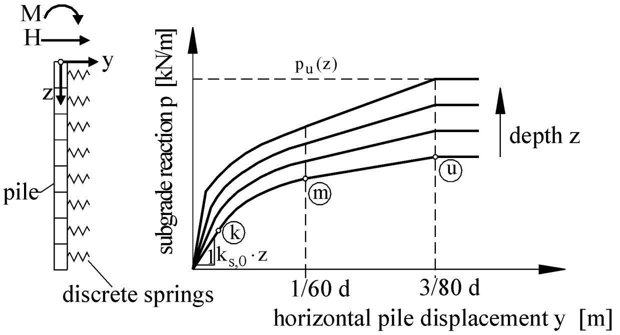


Figure 1. BEF model and p-y-curves for sand according to Reese et al. (1974).

API (2000) adopted the closed hyperbolic formula suggested by Murchinson and O'Neill (1984) substituting the formulation of Reese et al. (1974):

$$p(y,z) = A \cdot p_u(z) \cdot \tanh\left(\frac{k_{s,0} \cdot z}{A \cdot p_u(z)} \cdot y\right) \qquad (1)$$

In Equation 1 $k_{s,0}$ represents the initial modulus of subgrade reaction (see Fig. 1) depending on the angle of internal friction or the relative density of the soil. The quantity $p_u(z)$ is the maximum subgrade reaction defined in the following way:

$$p_u(z) = \min\begin{cases}(C_1 \cdot z + C_2 \cdot d) \cdot \gamma \cdot z \\ C_3 \cdot d \cdot \gamma \cdot z\end{cases} \qquad (2)$$

The coefficients C_1, C_2 and C_3 are functions of the angle of internal friction and can be determined according to API (2000). The parameter A fits the theoretical value of $p_u(z)$ in Equation 2 to field test results (Reese et al. 1974):

$$A = \begin{cases}\left(3 - 0.8\frac{z}{d}\right) \geq 0.9 & \text{for static loading} \\ A = 0.9 & \text{for cyclic loading}\end{cases} \qquad (3)$$

A thorough description of the p-y-method and an overview of different p-y-curves and new developments is given in Reese and van Impe (2001).

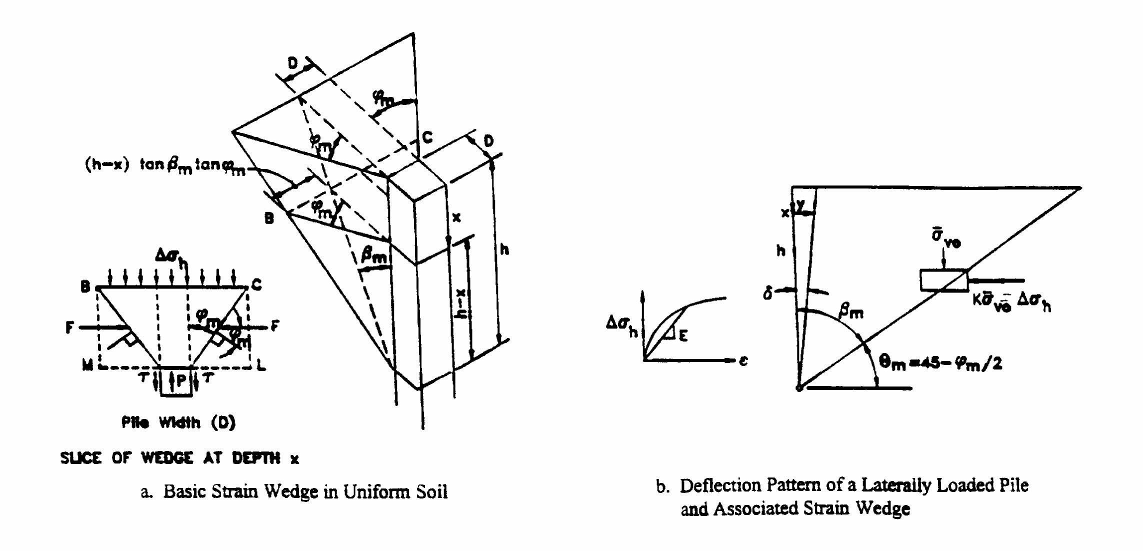
STRAIN WEDGE METHOD

In the strain wedge (SW) model approach (Norris 1986), the aforementioned traditional one-dimensional BEF pile response parameters can be characterized in terms of three-dimensional soil-pile interaction behavior. The SW model parameters are related to an envisioned three-dimensional passive wedge of soil developing in front of the pile as shown in Figure 2. The SW model provides a theoretical link between the more complex three-dimensional soil-pile interaction and the simpler one-dimensional BEF characterization, and allows the appropriate selection of BEF parameters to solve the fourth-order ordinary BEF differential equation:

$$EI\left(\frac{d^4y}{dx^4}\right)+k_s(x)\cdot y=0 \qquad (4)$$

where: EI = flexural rigidity of the pile
k_s = modulus of subgrade reaction associated with BEF characterization, $k_s = p/y$

The closed form solution of equation 4 has been obtained by Matlock and Reese (1961) for the case of uniform soil. The governing analytical formulations should be related to the passive wedge in front of the pile, the soil's stress-strain relationship, and the related soil-pile interaction.



a. Basic Strain Wedge in Uniform Soil

b. Deflection Pattern of a Laterally Loaded Pile and Associated Strain Wedge

Figure 2. Configuration of the strain wedge model (Ashour and Norris 2000)

The geometry of the mobilized passive wedge in front of the pile is shown in Figure 2, in which $\beta_m = 45° + \varphi_m/2$ is the base angle, h is the passive wedge depth, ω_m is the mobilized friction angle, $\Delta\sigma_h$ is the horizontal stress change at the wedge face, and τ is the side shear. One of the main assumptions of the SW model is that the deflection pattern of the pile is taken to be linear over the controlling depth of the soil near the pile top, resulting in a linearized deflection angle, δ, as seen in Figure 2, Norris (1986), Ashour and Norris (2000).

ANALYSIS OF THE MONOPILE BEHAVIOUR

The behaviour of large diameter monopiles has been thoroughly analyzed by finite-element modeling using the FEM code ABAQUS. For details of this study see Lesny and Wiemann (2005, 2006). The present paper focuses on the pile-soil interaction under serviceability conditions.

In the FE analysis the steel monopile was modeled as a half pipe pile using symmetry conditions. Linear elastic material behaviour was assumed for the pile and the pile-soil contact was modeled by the Coulomb friction law. Tension stresses between pile and soil were excluded. Table 3 shows the cross-sectional parameters of two selected piles of 1 m and 6 m diameter, respectively.

Table 3. Cross-sectional parameters of the analysed pipe piles

Diameter d [m]	Pile wall thickness t [m]	Cross-section A [m²]	Moment of inertia I [m^4]
1	0.02	0.062	0.0074
6	0.07	1.304	5.7330

A homogenous non-cohesive soil profile with an elasto-plastic material behaviour has been assumed. Thereby, the oedometric modulus increased parabolically with depth z:

$$E_s(z) = \left(\frac{z}{L}\right)^{0.5} \cdot E_{s,max} \qquad (5)$$

The basic soil parameters are summarized in Table 4. More details of the numerical model are given in Wiemann and Lesny (2004).

Table 4. Soil parameters for Essen Sand

Relative density I_D	0.55
Void ratio e	0.629
Angle of internal friction φ´	40.5°
Oedometric modulus E_S (mean stress range)	50-80 MN/m²
Weight/submerged weight γ/γ´	17/10 kN/m^3
Initial modulus of subgrade reaction $k_{S,0}$	19,000 kN/m^3

For the FE analysis the two piles have been pre-designed using the p-y method (API, 2000) and the parameters of Tables 3 and 4. In the calculations only a bending moment has been considered as this is the dominant load component (see Table 2). The p-y method provides the basis for the later comparison of the three design methods.

Two design criteria have been used in the pre-design: 1) the pile length should be sufficient for a rigid fixation and, 2) the pile head rotation was limited to $\alpha = 0.7°$. This value represents the upper limit for undisturbed operation of a typical 5-MW turbine. The critical embedded pile length required to ensure a rigid fixation of the pile to be determined according to (Titze, 1977) for a linear increasing modulus of subgrade reaction k_s with depth z:

$$L_c = \lambda \cdot L_0 \text{ with } L_0 = \sqrt[5]{\frac{EJ \cdot z}{d \cdot k_s(z)}} \qquad (6)$$

The factor λ usually varies between 4 and 5. Table 5 summarizes the pile lengths, the resulting bending moments and pile head displacements.

Table 5. Pre-design of the piles according to the p-y-method

Diameter d [m]	Length L [m]	Moment M [MNm]	Displacements y_{head} [m]
1	10.6	3.98	0.031
6	38.9	855.0	0.109

The moments in Table 5 represent the maximum moments which can be applied to the system (safety factors were not considered). With the information of Table 2 it may be concluded, that a pile diameter of about 6 m is indeed required for a 5 MW offshore wind energy converter.

The pre-design results of the two piles with the p-y method are compared with the results obtained from the FE analysis and the SW model (using SWM6.0) in Figure 3. The resulting deflection lines of the 1 m pile suggest that the SW model shows a much stiffer pile response than the p-y method with considerably less pile head deflections. The pile length determined with the p-y method seems to be overestimated according to the SW model. On the other hand, the FE analysis results show a softer pile response compared to the p-y method with some greater pile head deflections. A rigid fixation of the pile is not fully achieved as there are still some very small tip displacements. Zero pile tip displacements as theoretically required for a rigid fixation may be too restrictive and, hence, very conservative. From an economic point of view a pile length, which leads to a vertical tangent on the deflection line near the pile tip, is usually sufficient as the pile head deflections hardly decrease for greater pile lengths.

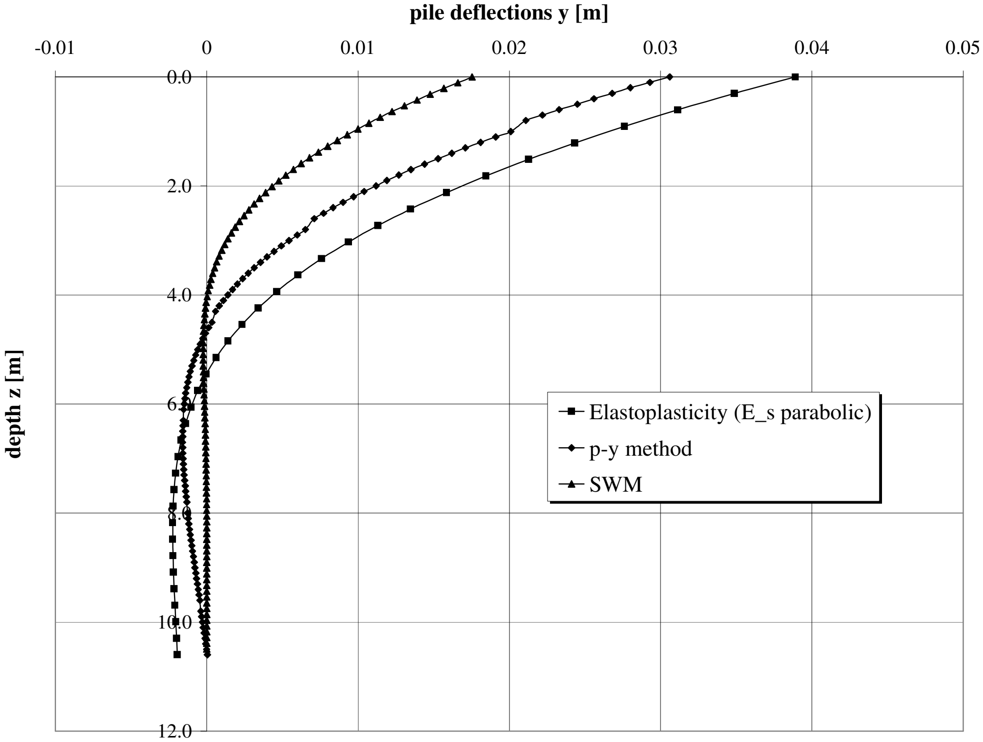


Figure 3. Deflection lines of the 1 m pile according to the p-y-method and the SWM compared to FE results.

Based on Figure 3 it may be concluded that design procedures as the p-y method and the SW model reflect the pile response sufficiently well especially regarding the design pile length. However, there may be some uncertainties regarding the deflection line and the resultant pile head deflection.

Extrapolating these methods to the 6 m diameter pile leads to the deflection lines depicted in Figure 4. The p-y method and the SW model apparently result in a similar pile response as for the 1 m pile. Again, the pile behaviour according to the SW model is stiffer with deflections around zero in the lower half portion of the piles. By contrast, the FE results show significant pile tip displacements, hence the pile does not gain a rigid fixation in the soil. Consequently, the pile head displacements are greater and the pile response is softer. Significant pile tip displacements can induce shear stresses along the pile cross section which may no longer be neglected in the pile design considering the size of the monopile. In contrast, the BEF model assumed within the p-y method and the SW model is based on a rigid fixation of the pile and does not consider such stress conditions.

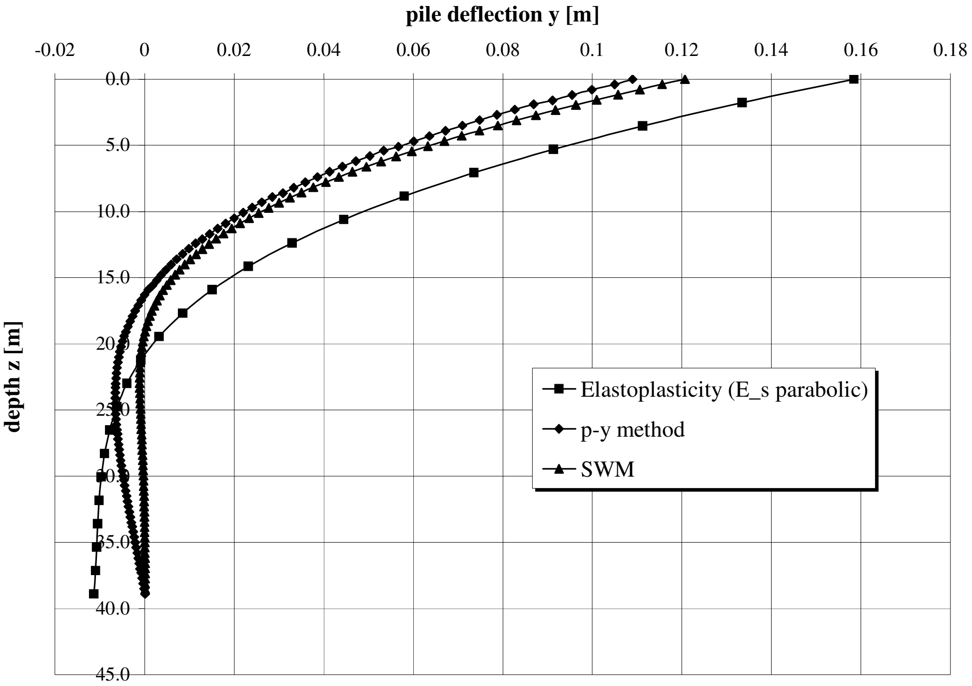


Figure 4. Deflection lines of the 6 m pile according to the p-y-method and the SWM compared to FE results.

As a result, the pile length determined by these methods is apparently not sufficient to achieve a full fixation of the pile in the soil. This is due to the stiffness relations implied in the p-y-method as well as in the SW model. Both methods assume a linear variation of the soil stiffness with depth (see Lesny and Wiemann 2006, Norris 1986). Monopiles of greater diameter, however, require a greater pile length for rigid fixation. The assumption of a linear increasing soil stiffness with depth, leads to an overestimation of the oedometric modulus of the soil at these great depths. For example, Lesny and Wiemann (2006) backcalculated the soil stiffness implied in the p-y method and obtained values at the pile tip of $E_{s,max} \approx 120\,MN/m^2$ for the 1 m pile but $E_{s,max} \approx 512\,MN/m^2$ for the 6 m pile. Whereas the magnitude of the soil stiffness for the 1 m pile is acceptable, the value for the 6 m pile is far too high.

If these stiffness values are considered in the FE calculations, the resulting deflection lines show a vertical tangent near the pile tip, hence, a rigid fixation of the pile as well (Figure 5). The pile head displacements for a linear distribution of the oedometric modulus (as assumed by the p-y and the SW methods) using the FE-analysis, are greater than the displacements resulting from the p-y and SW methods. Only a parabolic distribution of the stiffness can reflect the pile response according to the p-y method and the SWM. The reason for these differences may be found in the pile-soil-stiffness relations for the small pile diameters which were the basis for the development of both methods. Piles of larger diameter, however, with a greater critical length show a different pile-soil-stiffness relation, which is not accounted for by the p-y or the SW methods. As a result, the use of these methods should not be directly extrapolated to conditions outside their diameter range of application. Lesny and Wiemann (2006) suggested a simple modification of the standard p-y method depending on the pile diameter. This modification allows one to account for the variation in the stiffness condition of large diameter open pipe piles.

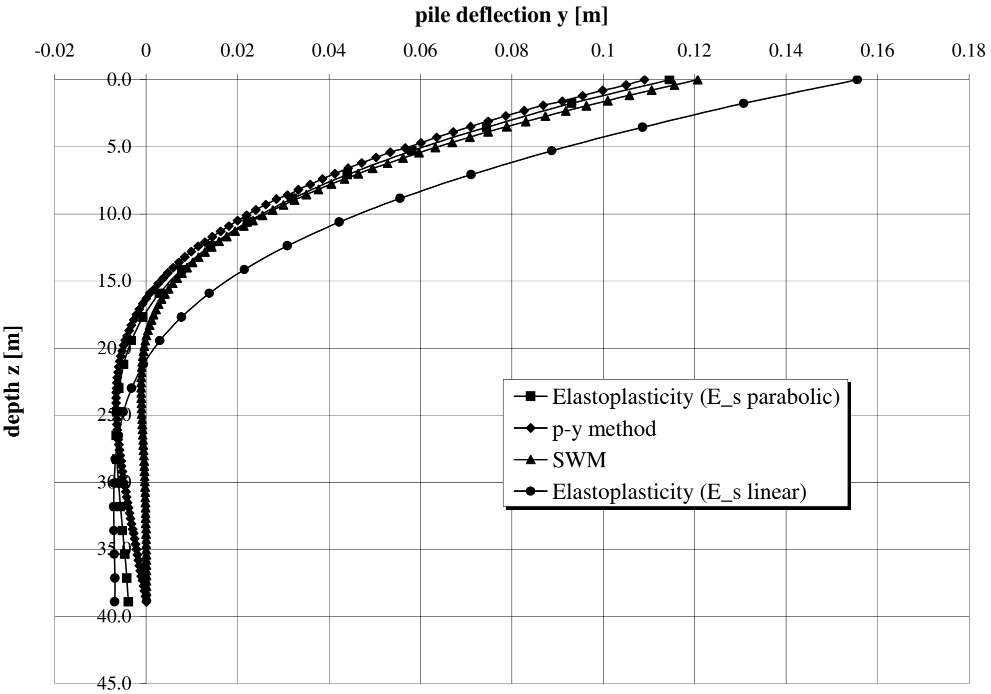


Figure 5. Deflection lines of the 6 m pile according to the p-y-method and the SWM compared to FE-results with back calculated oedometric modulus.

CONCLUSIONS

The monopile is one of the favoured foundation concepts for offshore wind energy converters. Monopiles are usually designed using the well-known p-y-method. An alternative design method is the strain wedge method developed by Norris (1986). Utilizing FE- analysis it has been demonstrated that both methods overestimate the pile-soil-stiffness of large diameter monopiles at great depths which may result in an insufficient pile length design. These observations may be attributed to the linear distribution of the soil stiffness implied in these methods. This assumption leads to unrealistic pile-soil stiffness relations of large diameter piles and therefore cannot properly reflect the pile response. Hence, these method should not be directly applied to large diameter monopiles.

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