

APPLICATION OF THE KOSECKI'S METHOD IN DESIGNING OF OFFSHORE WIND POWER PLANTS FOUNDATION

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ABSTRACT

The concept of offshore wind power plants has been well developed in many European countries. There is no such thing as design of offshore wind power plants according to national tradition. The main problem is the lack of standards and guidelines. Ones being applied are Scandinavian or American methods which are not fully adapted to the conditions of the Baltic Sea. The article focuses on the monopile design, as it is currently the most often used type of offshore power plants foundation. The results from analysis, according to standard wind turbine monopile design methods, were compared with the results obtained from the Kosecki's method. The geometry analyzed in the article goes beyond the scope of defined geometrical and technological coefficients used to determine the stiffness of spring supports. The adopted stages of the analysis allowed to determine the limit of applicability of the Kosecki's method for monopiles loaded horizontally.

Key words: pile foundation, monopile, offshore wind power plants, wind energy

INTRODUCTION

In 2011, and then in 2015 and 2016, the Act of 21 March 1991 on the Polish Republic's sea areas and maritime administration was amended, which resulted in the increasing interest in the subject of offshore wind energy. The location of wind farms, apart from the previously mentioned act, is also determined by the following conditions (Leite, 2015):

- environmental: sea depth, wind strength;
- spatial: fishing zones, zones of military manoeuvres, shipping routes, raw material deposits;
- economic;
- social;
- legal and administrative: birds protection zones and protected areas;
- political.

The pioneering investment in Poland called the Bałtyk Środkowy III (BS III), is to be launched in 2022,

and it is another part – the BS II in 2026. The main problem, while designing these structures, is the lack of Polish standards and guidelines. Scandinavian or American methods are used, despite not being fully adapted to the climatic conditions on the Baltic Sea. These standards allow to use various design methods. The basic standards for the design of offshore wind turbines are:

- DNV-OS-J101: Design of Offshore Wind Turbine Structures, 2014;
- DNV-DS-J102: Design and Manufacture of Wind Turbine Blades, Offshore and Onshore Wind Turbines, 2010;
- API-RP 2A-WSD: Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Working Stress Design, 2002;
- IEC 61400-1: Wind Turbines – Design Requirements, 2005;
- IEC 61400-3: Design Requirements for Offshore Wind Turbines, 2002.

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Most of common approaches are based on the method of partial safety factors. Alternatively or as a complement to analytical methods, the design can be based on the test results or observations of models in a natural or reduced scale.

FOUNDATION CHARACTERISTICS

The basic structural elements of an offshore wind turbine are: (Fig. 1):

- nacelle – containing electromechanical elements,
- rotor – built with hub and blades;
- tower – tubular construction made of steel or concrete;
- transition piece – connecting the tower with the foundation;
- foundation.

The type of foundation depends on the distance from the shore and on the sea depth. For shallow areas, the best solution is to use gravity based foundations (Fig. 2d) or monopiles (Fig. 2a). As the depth increases, the construction of jackets (Fig. 2c) tripods (Fig. 2b) (often used also in the foundation of drilling units) becomes more beneficial. In deep waters floating structures are the most economical (Fig. 2e).

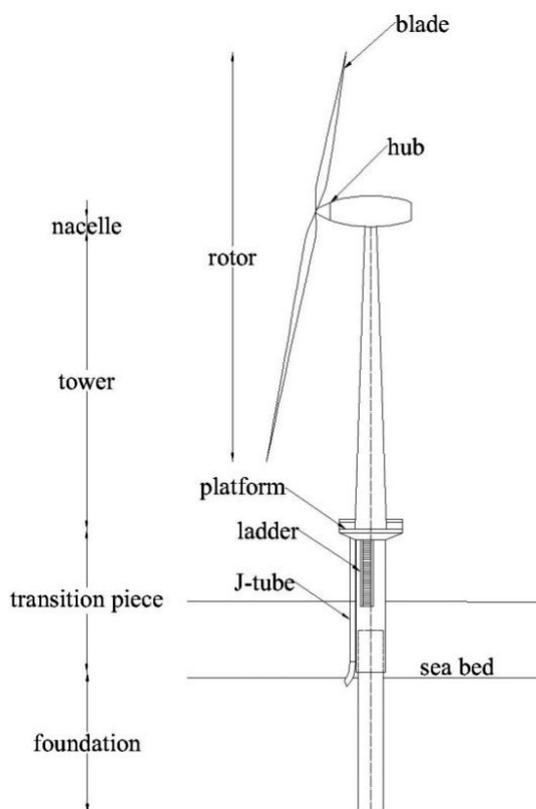


Fig. 1. Elements of an offshore wind turbine

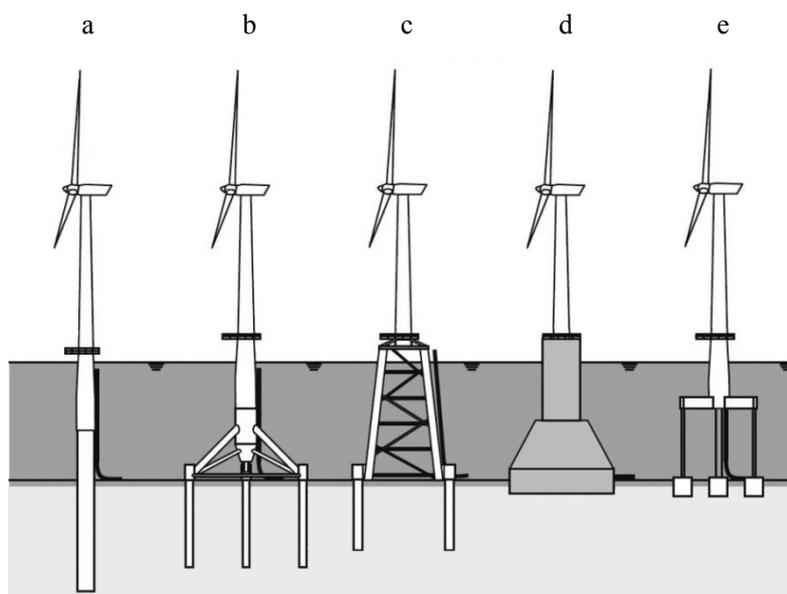


Fig. 2. Wind turbine foundations: a – monopile; b – tripod; c – jacket; d – gravity foundation; e – floating foundation (van der Tempel, 2006)

Currently, most offshore wind farms in Europe are located at depths up to 20 m and mostly used foundation types are monopiles, accounting for about 76% of all existing foundations. They owe their popularity to relatively simple production, installation process and to a wide range of exploitation. This is an efficient solution at depths of up to 35 m and this depth is constantly increasing.

The monopile structure is relatively simple as it is made of a single, steel, cylindrical tube with a diameter of up to 9 m and a wall thickness of up to 150 mm. Pipes are usually made by rolling and longitudinal welding steel plates. Prepared elements are transported to the wind farm area using transport ships or special platforms towed by tugs.

Monopiles can be driven into the seabed with the use of hammers or impact-rotary hammers, depending on the current environmental conditions and the geological structure of the seabed. After installation, the upper elements of the wind turbine, i.e. the tower, nacelle and blades, are assembled. Depending on the structure of the seabed, the tower can be attached to a monopile directly or via a transition piece.

LOADS AFFECTING OFFSHORE WIND FARMS

According to the guidelines of the Norwegian classification society Det Norske Veritas (DNV), the loads acting on the offshore wind farm are divided into five categories (Table 1).

In the analysed example, three most important environmental loads were considered (Fig. 3), i.e. wind, wave and sea currents, which is a good starting point for further, more complex calculations.

Normal and extreme wind conditions are distinguished by the DNV standards. Normal conditions are used to assess fatigue loads under standard load conditions, while extreme conditions apply to extreme situations that cause huge loads on the power plant and the surrounding soil.

Wave data for a specific location are collected over a long period of time. If such data for a given area are not available, parameters from outside areas must be transformed to take differences in water depth and changes in the topography of the seabed into account.

Both waves and currents are generated by the wind, therefore all data should be correlated. The speed of

Table 1. Loads (Det Norske Veritas, 2007)

No	Load category	Type of loads
1	permanent	weight of the wind turbine: rotor, hub, blades, nacelle, tower weight of the monopile and transition piece hydrostatic pressure
2	variable functional	installation operations ship impacts maintenance of the wind turbine load of the staff, equipment and materials
3	environmental	wind wave loads currents and tides snow and ice temperature soil conditions
4	accidental	collision impact from vessel, helicopter or other objects fire load from large breaking waves
5	deformation	settlements temperature loads

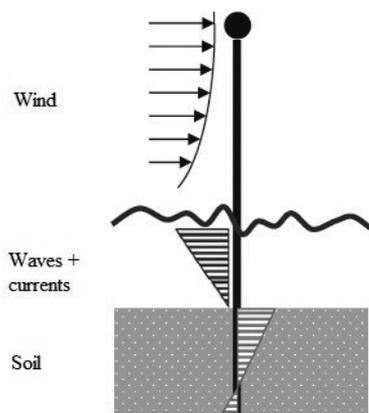


Fig. 3. Environmental loads for the analysed calculation example (van der Tempel, 2006)

sea currents depends on two components: the speed of wind-generated currents and the speed of tidal currents. For the ultimate limit state – ULS, several load

combinations are distinguished by using mean values of load types in different time periods or the frequency of occurrence of extreme values. The load occurring in all combinations is a permanent load, the remaining loads used in the combinations are presented in Table 2.

In the conducted analysis, based on the guidelines presented in DNV Offshore Standard (Det Norske Veritas, 2014) seven load combinations were prepared with the use of the partial safety factors according to Table 3.

OFFSHORE WIND TURBINE MODEL

Soil stiffness according to DNV guidelines

DNV standards recommend the use of $p-y$ curves for the analysis of horizontally loaded piles. The $p-y$ curves give the nonlinear relationship between soil resistance and horizontal displacement of piles under

Table 2. Load combinations for the ultimate limit state

Combination no	Environmental load type and its value
1; 4	wind – extreme sea state – return period: 50 years waves – normal operative loads currents – normal operative loads
2; 5	wind – normal operative loads waves – extreme sea state – return period: 50 years currents – normal operative loads
3; 6	wind – normal operative loads waves – normal operative loads currents – extreme sea state – return period: 50 years
7	wind – extreme sea state waves – extreme sea state currents – extreme sea state

Table 3. Load factors for the ultimate limit state

Combination no	Load factors for:	
	permanent loads	environmental loads
1; 2; 3	1.25	0.7
4; 5; 6	1.0	1.35
7	1.0	1.0

lateral load. The pile is modelled as series of beam-column elements, supported by springs applied in the nodes between the elements. Each spring is placed in the middle of a given soil layer and is characterised by one p - y curve.

In case of cohesive soils, p - y curves are presented in dimensionless form of y/y_c as a function of p/p_u (y – actual deflection; y_c – critical deflection; p – actual lateral resistance; p_u – ultimate resistance).

The value of critical displacement can be determined on the basis of laboratory tests in the triaxial compression apparatus and by using Equation 1 (Det Norske Veritas, 1992).

$$y_c = 2.5 \cdot \varepsilon_{50} \cdot D \quad (1)$$

where:

ε_{50} – strain which occurs at one-half of the maximum stress in laboratory undrained compression tests of undisturbed soil samples [-];

D – pile diameter [m].

The values of ε_{50} , for the purposes of the analysed example, were assumed on the basis of correlations presented in the literature (Chen and Duan, 2003).

For cohesionless soils, p - y curves are also non-linear but they may be determined from the following Equation:

$$P = A_i \cdot p_u \cdot \tanh \left(\frac{k_{init} \cdot z}{A_i \cdot p_u} \cdot y \right) \quad (2)$$

where:

A_i – factor to account for cyclic or static loading condition, evaluated by:

$$A = \left(3 - 0.8 \cdot \frac{z}{D} \right) \geq 0.9 \quad (3)$$

p_u – ultimate resistance at depth H [$\text{kN} \cdot \text{m}^{-1}$];

z – depth below soil surface [m];

y – actual lateral deflection [m];

k_{init} – initial modulus of subgrade reaction [$\text{kN} \cdot \text{m}^{-3}$].

$$k_{init} = (0.008085 \cdot \phi^{2.45} - 26.09) \cdot 10^3 \quad (4)$$

The ultimate resistance depends on the water depth. For shallow waters, the Equation (5) and for deep waters the Equation (6) is used.

$$p_{us} = (C_1 \cdot z + C_2 \cdot D) \cdot \gamma' \cdot z \quad (5)$$

$$p_{ud} = C_3 \cdot D \cdot \gamma' \cdot z \quad (6)$$

where:

γ' – effective unit weight of soil [$\text{kN} \cdot \text{m}^{-3}$];

z – depth [m];

D – pile diameter [m];

C_1, C_2, C_3 – coefficients as a function of ϕ' .

$$C_1 = 0.115 \cdot 10^{0.0405 \cdot \phi'} \quad (7)$$

$$C_2 = 0.571 \cdot 10^{0.022 \cdot \phi'} \quad (8)$$

$$C_3 = 0.646 \cdot 10^{0.0555 \cdot \phi'} \quad (9)$$

where:

ϕ' – effective angle of internal friction [$^\circ$].

Soil stiffness according to the Kosecki's method

Parameters of spring supports, modelling the horizontal cooperation between piles and the surrounding soil, in case of standard pile structures can be determined according to the Kosecki's proposal based on the module of horizontal soil reaction determined by the Equation (10) (Kosecki, 2006). Spring supports, in this method, are placed every 0.50 m.

$$K_x = n_0 \cdot n_1 \cdot n_2 \cdot S_n \cdot \kappa \cdot \phi \cdot E \quad (10)$$

where:

n_0 – correction factor for the influence of pile diameter ($n_0 = 1.0$ for $0.3 \leq D \leq 1.5$, $n_0 = D/1.5$ for $1.5 \leq D \leq 3.0$, $n_0 = D$ for $D \geq 3.0$ m);

n_1, n_2 – coefficients dependent on the diameter and distance between piles ($n_1, n_2 \leq 1.0$);

S_n – coefficient dependent on the pile technology, assumed $S_n = 0.9$;

κ – coefficient dependent on the shape of the pile cross section;

ϕ – coefficient dependent on the durability and repeatability of loads;

E – Young's modulus [kPa].

Coefficients n_1 and n_2 are determined from following Equations:

$$n_1 = 0.2 \cdot \frac{R_1}{D} + 0.4 \leq 1.0 \quad (11)$$

$$n_2 = \beta + \frac{(1-\beta) \cdot (R_2 - D)}{1.8 \cdot D} \leq 1.0 \quad (12)$$

where:

R_1 – axial distance of piles in the surface perpendicular to the direction of spring supports;

R_2 – axial distance of piles in the surface parallel to the direction of spring supports;

D – pile diameter [m];

D_0 – equivalent pile diameter, equal to $D_0 = D + 1$ [m];

β – coefficient dependent on the number of rows of piles, assumed $\beta = 1$.

RESULTS

For the purpose of the analysis, a 2MW turbine was adopted. Dimensions of each component were assumed relying on the guidelines presented in *Technical Project Description for Offshore Wind Farms* (Energinet.dk, 2015).

The geotechnical aspect in this project is dimensioning the substructure (foundation). In the case of the described monopile, the task was to determine geometry of the pile, i.e. diameter, length and wall thickness, so that the structure could transfer loads safely.

Geotechnical parameters were determined based on cone penetration test data (CPTU) carried out in the Gulf of Gdańsk, 15 km from shore, at site where water depth was about 12 m.

Loads were determined in accordance with procedures described in

chapter “Loads effecting offshore wind farms”. Values of measured climatic parameters were obtained from the Institute of Meteorology and Water Management in Gdańsk and based on the publication by Marcinkowski and Szymkiewicz (2016).

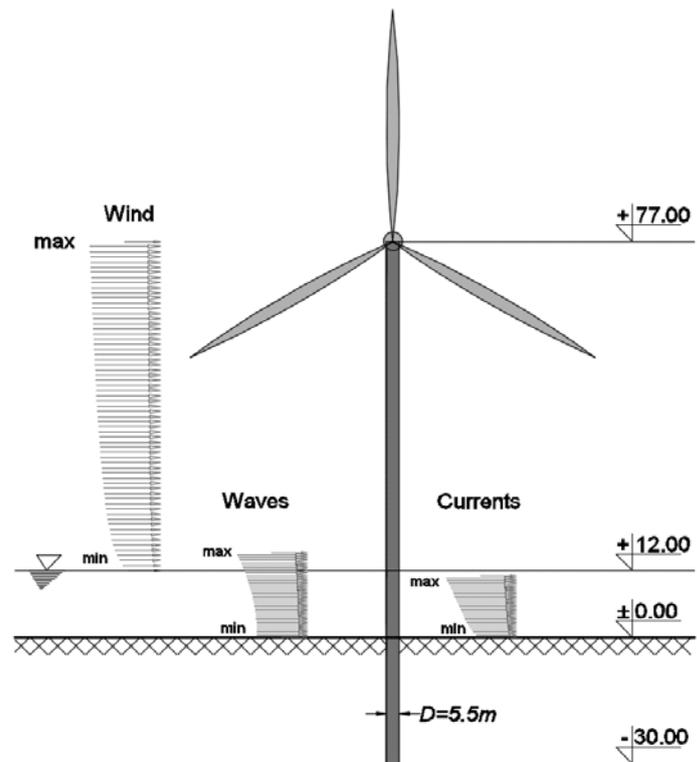
Results obtained with the model defined according to the DNV guidelines are shown in Figure 5, while those obtained with the model in which the support springs were defined in accordance with Kosecki's method are shown in Figure 6.

Table 4. Construction specifications

	turbine manufacturer	Siemens
Turbine	turbine model	SWT-2.3-93 Offshore
	capacity	2.3 MW
Operational	cut-in wind speed	4.0 m·s ⁻¹
	rated wind speed	13 m·s ⁻¹
	cut-out wind speed	25 m·s ⁻¹
	rotor position	upwind
Rotor, hub, blades	rotor diameter	93 m
	rotor area	6,800 m ²
	rotor speed (minimum)	6 rpm
	rotor speed (maximum)	16 rpm
	hub height (AMSL)	65 m
	blade length	45 m
	number of blades	3
	total weight (rotor, hub, blades)	60 t
Nacelle	weight	82 t
Tower	structure type	tubular, steel
	height	65 m
	weight	610.08 t
	diameter	5.5 m
Substructure	wall thickness	70 mm
	weight	394.2 t
	length driven into sea bed	30 m
	length above sea bed	12 m

Table 5. Soil layers for the analysed static penetration test data

Layer No	Soil type	Depth below the seabed [m]	
		top of the layer	bottom of the layer
1	clay	0.00	1.00
2	sand/sandy silt/clayey silt	1.00	7.10
3	clay/silty clay/sandy silt	7.10	9.60
4	sand/silty sand	9.60	11.60
5	sand	11.60	13.30
6	sand/silty sand	13.30	14.80
7	silty-clayey sand	14.80	16.10
8	silty clay/clay	16.10	21.50
9	silty sand/sandy silt	21.50	30.00

**Fig. 4.** Environmental loads**Table 6.** Values of environmental loads

Load type	Normal operative [$\text{kN}\cdot\text{m}^{-1}$]		Extreme sea state [$\text{kN}\cdot\text{m}^{-1}$]	
	max	min	max	min
Wind	0.22	0.04	1.81	1.60
Waves	0.17	0.02	8.36	0.85
Currents	9.07	2.52	46.62	16.62

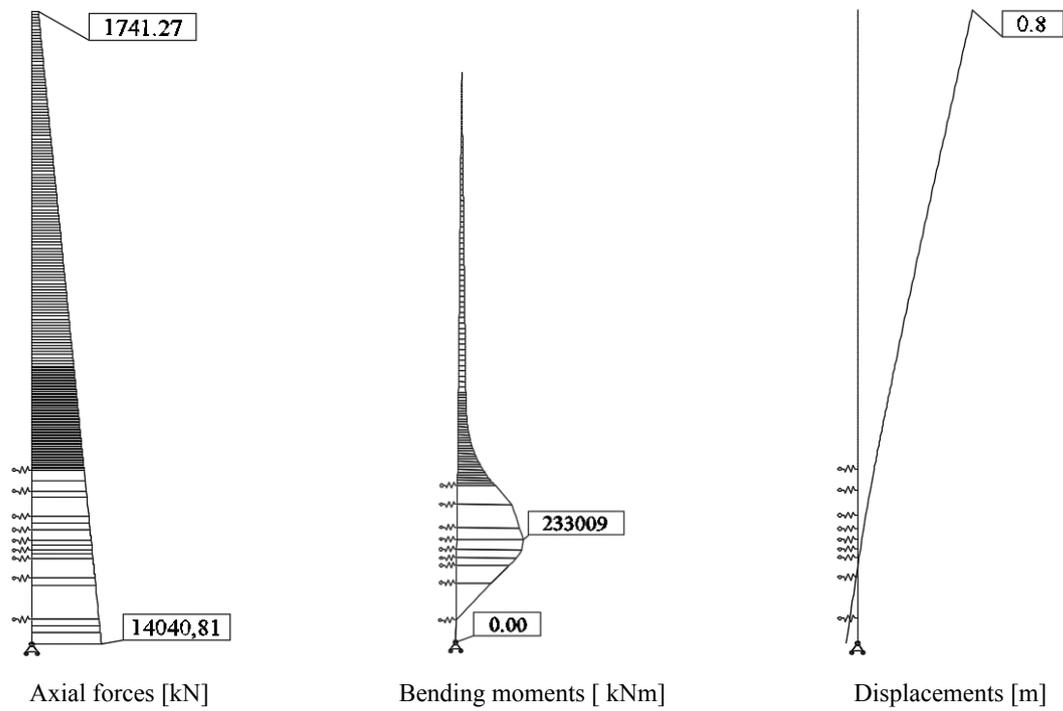


Fig. 5. Analysis results for the model according to DNV

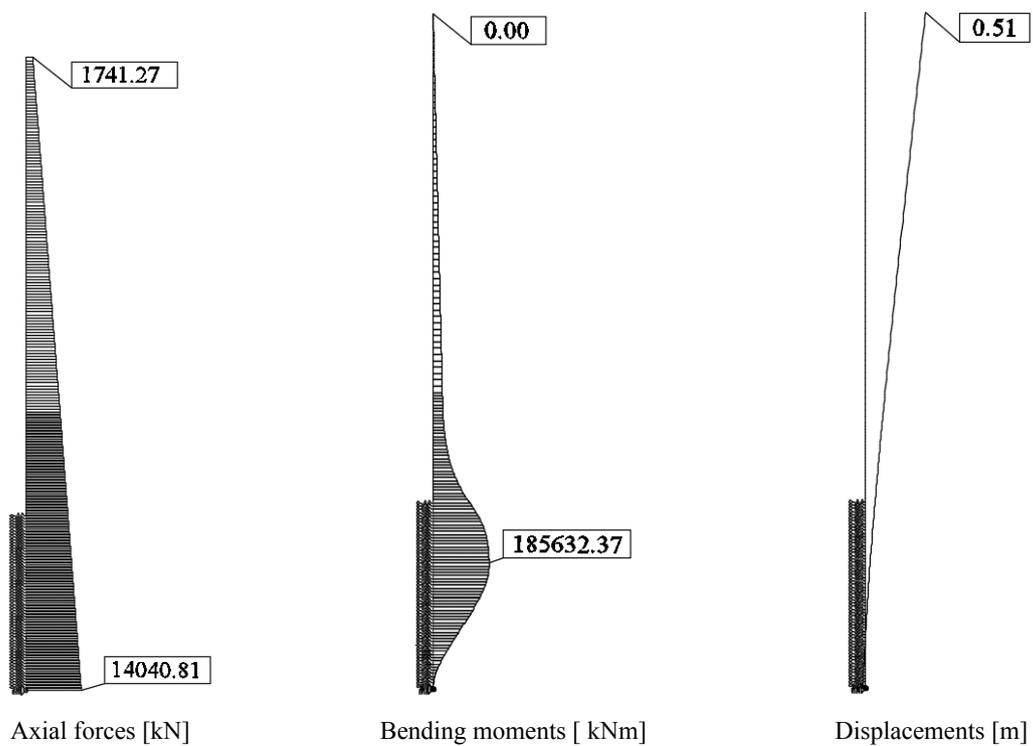


Fig. 6. Analysis results for the model according to Kosecki's method

CONCLUSIONS AND SUMMARY

Based on the analysis, it appears that in case of offshore monopile wind turbines, main criterion is the serviceability limit state (SLS), especially horizontal displacements of the structure. According to the DNV, permitted displacement is 1% of the total construction height. In the considered case, the allowed displacement is 1.07 m. The SLS was met in both analyzes. In Kosecki's method, displacements were significantly lower.

The main differences between applied approaches are related to the methodology for defining stiffness of spring supports. They include not only the number of springs in each soil layers, but more importantly the values of soil stiffness (Kx). In cohesive soils, the discrepancies of Kx are slight (Fig. 7). However, the attention should be paid to the sensitivity of the Norwegian method to changes in the pile diameter. This aspect is not evident in Kosecki's method, it is only visible at a certain depth.

For cohesionless soils, the procedure for determining stiffness of spring supports requires the determination of p - y curves. The best way is to carry out a pile load test, which is rather expensive, especially at design stage. Based on analytical method, the Kx values according to DNV are several times higher than those determined with the use of Kosecki's method (Fig. 8).

Kosecki's method is popular and has often been used for calculating piles with standard parameters. In case of very large diameters, obtained results are divergent in relation to the p - y curves method. Mistakes and deviations from proper displacement values increase with pile diameter. The relation to the values obtained according to DNV is shown in Figure 9 and Table 7.

Based on the additional simulation, it was found that the possible scope of Kosecki's method refers to piles with a maximum diameter of 1.5 m. Despite significant discrepancies in the assumptions regarding determination of the Kx the theoretical displacement of the top of the structure was the same in both methods for the pile diameter of 1.5 m. It is also important that the trend in the difference in displacements depends on the diameter and is linear.

Designing of offshore wind farms according to national tradition does not exist. The main problem is the lack of standards and guidelines. Scandinavian or American methods are used despite not being fully adapted to the climatic conditions on the Baltic Sea. Based on the conducted analysis, it is stated that after defining appropriate coefficients the Kosecki's method can be used, as an alternative approach, for designing pile foundations for offshore wind farms. Those coefficients, should take into account particularly large pile diameters, specific monopile technology and installation process. This method should also be

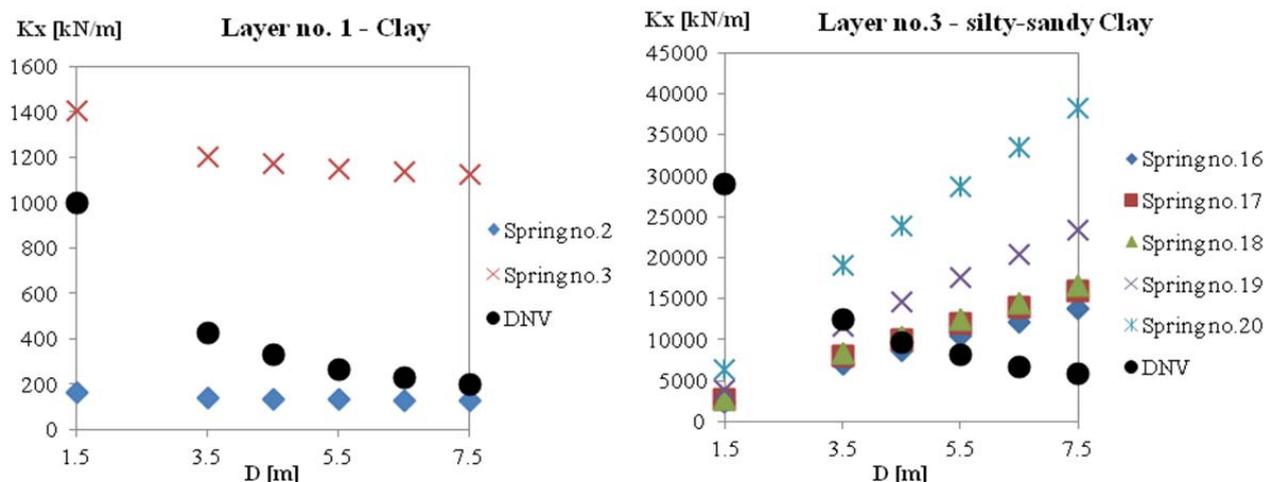


Fig. 7. Soil stiffness in cohesive layers according to DNV and Kosecki's method

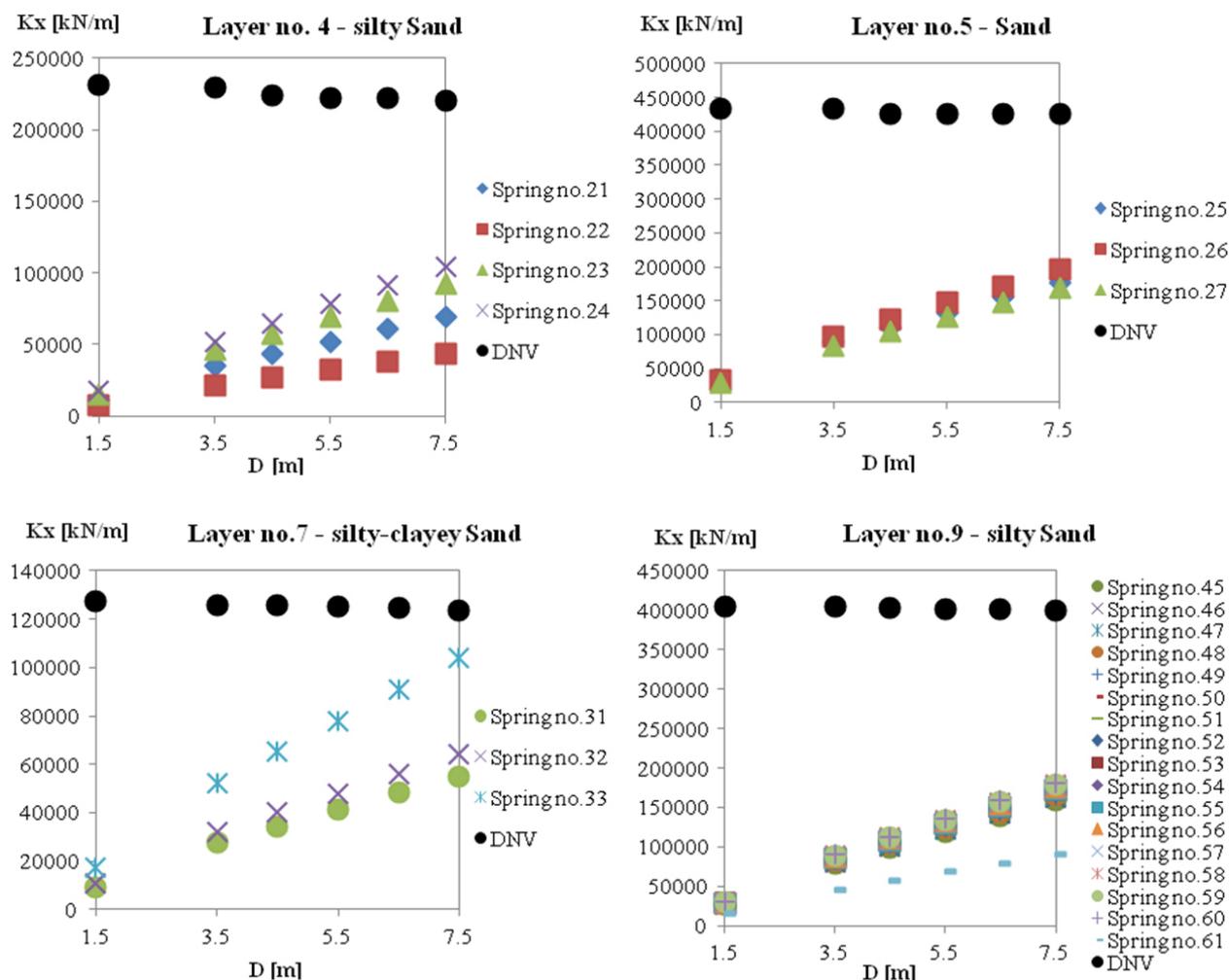


Fig. 8. Soil stiffness in cohesionless layers according to DNV and Kosecki's method

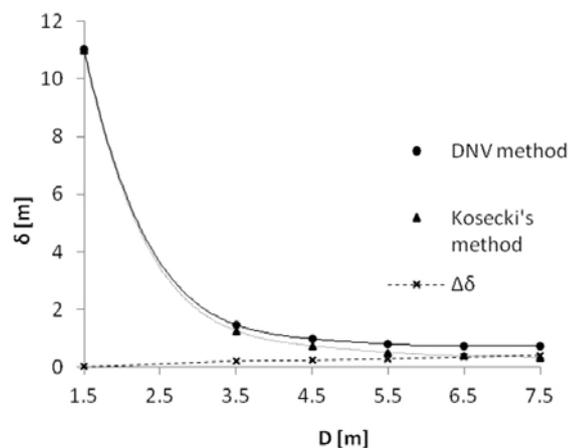


Fig. 9. Relation between horizontal displacement and pile diameter

Table 7. Horizontal displacements

Diameter [m]	δ (DNV) [m]	δ (Kosecki) [m]	$\Delta\delta$ [m]
1.5	11.02	11.01	0.01
3.5	1.47	1.26	0.21
4.5	0.98	0.74	0.24
5.5	0.8	0.51	0.29
6.5	0.74	0.39	0.35
7.5	0.73	0.32	0.40

calibrated to local soil conditions. The advantage of Kosecki's method is a clear, analytical determination of the characteristics of spring supports.

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ZASTOSOWANIE METODY KOSECKIEGO W PROJEKTOWANIU POSADOWIENIA MORSKICH ELEKTROWNI WIATROWYCH

STRESZCZENIE

Koncepcja morskich elektrowni wiatrowych została dobrze rozwinięta w wielu krajach europejskich. Obecnie nie projektuje się morskich elektrowni wiatrowych zgodnie z krajowymi tradycjami. Głównym problemem jest brak standardów i wytycznych. Stosowane są metody skandynawskie lub amerykańskie, które nie są w pełni dostosowane do warunków panujących na Morzu Bałtyckim. Artykuł skupia się na projektowaniu monopali, ponieważ są one obecnie najczęściej stosowanym rodzajem fundamentów morskich elektrowni. Wyniki analizy, przeprowadzonej zgodnie ze standardowymi metodami służącymi projektowaniu monopali pod turbiny wiatrowe, porównano z wynikami uzyskanymi metodą Koseckiego. Analizowana w artykule geometria wykracza poza zakres zdefiniowanych współczynników uwzględniających kształt i technologię pała, służących do wyznaczania sztywności podpór sprężystych. Przyjęte etapy analizy pozwoliły określić granicę stosowalności metody Koseckiego dla monopali obciążonych siłami poziomymi.

Słowa kluczowe: fundamenty palowe, monopale, morskie elektrownie wiatrowe, energia wiatrowa