

SHORT REVIEW AND 3-D FEM ANALYSIS OF BASIC TYPES OF FOUNDATION FOR OFFSHORE WIND TURBINES

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ABSTRACT

Some problems of the foundations of offshore wind turbines are considered in this paper. A short review is presented on the two basic types of foundations, i.e. monopiles and gravity foundations, including their basic features and applications as well as general design considerations. Also, some issues regarding analysis are discussed, including geotechnical problems and modelling techniques. A numerical model of offshore turbine and some preliminary computations are presented. Finite element analysis was carried out for wind turbines supported on both gravity and monopile foundations. The wind turbine tower, blades (simplified model), gravity foundation and part of the surrounding soil are included in the model. The turbine was loaded by wind and loads induced by waves, inertia and gravity. Both non-linear static and dynamic analysis of the wind turbine was performed. The displacements and stresses under the tower foundations were calculated and a comparison analysis carried out.

Keywords: offshore wind turbine, monopile, gravity foundation, FEM analysis

Table 1. Properties of turbines for a water depth of 30 m

INTRODUCTION

Wind energy is one of the most widely used types of renewable energy. Recently, there has been a significant development of offshore wind farms. Their construction was an important element of the European government strategy aiming to achieve the target of 20% of energy from renewable sources by 2020. Offshore wind turbines (OWTs) are generally larger than those installed on land, and a nominal capacity of 3 to 5 MW is now the norm. Usually, rotor diameters are larger than 100 m and nacelle locations are more than 80 m above mean sea level. Currently, modern 6–10 MW turbines are being introduced. However, turbines of up to 10 MW and with rotor diameters as large as 150 m are coming soon. The potential of offshore wind energy in Europe is expected to steadily increase up to 25 MW in 2026 [21]. Outline properties of present day and forthcoming turbines are shown in Table 1 [5]. The basic components of a wind turbine system supported on a monopile foundation, including their estimated dimensions, are presented in Fig. 1.

Parameter	Unit	Turbine rated power			
		3.6 MW	5.0 MW	6–7 MW	8.0 MW
Rotor diameter	m	120	126	153	164
Rated wind speed	m/s	13	11.4	13	14
Hub height	m	80	85	100	110
Max thrust at hub	MN	1.20	1.20	2.00	2.30
Max mudline moment M_{max}	MNm	136	137	265	323
Maximum wave height	m	12	12	12	12
Typical monopile diameter	m	6	6.5	7	7.5
Horizontal wave force	MN	4.2	4.81	5.43	6.09
Mudline moment from waves	MNm	120	137	155	175
Unfactored design moment	MNm	256	274	420	498

The main advantages of offshore wind farms are that the wind is more stable, giving more efficient use of turbines; there is lack of technological constraints – a turbine installed at sea may be far higher and thus more effective; wind force at sea is higher at a lower altitude, which enables the use of lower towers. Besides that, wind strength increases with distance from the shore and the sea provides more space for the location of wind turbines. The main disadvantages are greater difficulty in accessing the facility during repair or maintenance, and much more expensive maintenance and service facility power.

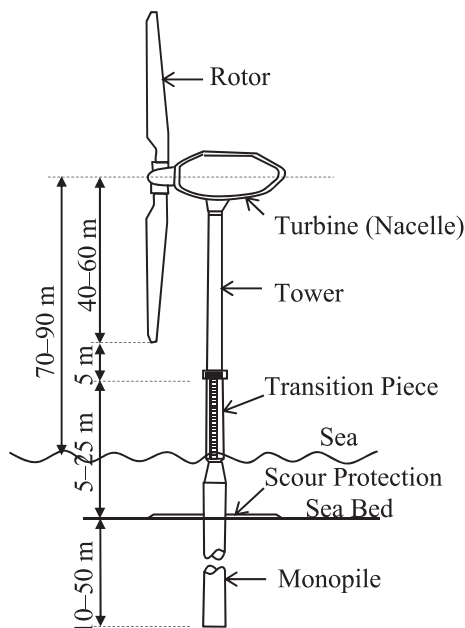


Fig. 1. Offshore wind turbine system components

Foundations of OWTs are considerably more costly than for the equivalent onshore ones. The cost of the support structure varies from 25% to 35% of the overall cost [6] and it can even reach 50% [10]. Such costs mainly depend on the sea depth and distance from shore. Their influence can be introduced by a correction factor, given in Table 2 [9]. It can be assumed that in the next dozen or so years, most OWTs will be located at sites in water depth greater than 30–40 m.

Table 2. Cost factor dependency on distance and depth

Depth (m)	Distance from shore (km)							
	> 0	> 10	> 20	> 30	> 40	> 50	> 100	> 200
10–20	1.00	1.02	1.04	1.07	1.09	1.18	1.41	1.60
20–30	1.07	1.09	1.11	1.14	1.16	1.26	1.50	1.71
30–40	1.24	1.26	1.29	1.32	1.34	1.46	1.74	1.98
40–50	1.40	1.43	1.46	1.49	1.52	1.65	1.97	2.23

The cost of the foundation includes, besides the cost of the structure itself, transport, installation and scour protection. Scour protection consists of a filter layer and an armour layer. In the case of monopiles, depending on the hydrodynamic environment, the horizontal extent of the armour layer can be between 10 and 15 m, having a thickness of between 1 and 1.5 m. Filter layers are usually 0.8 m thick and reach up to 2.5

m further than the armour layer. The total diameter of the scour protection is assumed to be five times the pile diameter. There are many different standards and recommendations for calculating the extent and thickness of scour protection. Interesting design guidelines to determine these parameters in relation to the pile diameter are provided in [24].

In the case of a gravity foundation, it should also include seabed preparation.

OWTs are subjected to higher environmental loading than those situated on land. It is necessary to provide the right connection with the subsoil. The foundation of the turbine transfers the forces from the structure to the ground. Thus, the foundation is a critical part in designing the OWT and a thorough analysis is essential. Recently, the finite element method (FEM) has found wide application in soil–structure interaction problems. In this paper, 3-D FEM computations were carried out for wind turbines supported on monopile and gravity foundations. Non-linear static and dynamic analysis of the wind turbine was performed. Both stresses and displacements beneath the foundation were computed. Comparison analysis was performed. A short review of the basic supporting structures of OWTs, i.e. gravity foundations and monopiles, is also presented. The most important characteristics for both cases are discussed as well as some problems of their analysis or design.

TYPES OF FOUNDATION

It is characteristic for an OWT that the wind and wave loadings lead to greater forces on the structure than those that would occur onshore. In the case of a 3.5 MW turbine, the vertical load is of the order of ca. 6 MN, the maximum horizontal load applied on the foundation may be ca. 4 MN and the overturning moment ca. 120 MNm (or equivalent to the horizontal load being applied 30 m above the base), see [6]. The ratio between the horizontal and vertical loads (H/V) for an OWT is between 1.4 and 2.6 [14]. Axial and lateral loads act at a point at the interface level between the monopile and the turbine shaft that is located ca. 20 m above sea level. Thus, it is necessary to ensure that sufficient connection with the ground is provided, otherwise the structure will move irreversibly.

Foundations for wind turbines can be classified into two main types: bottom-fixed (Fig. 2a–d) and floating (Fig. 2e).

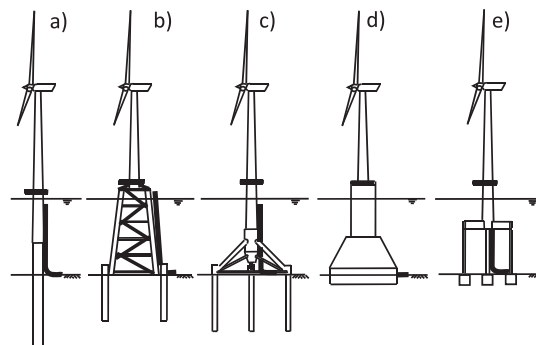


Fig. 2. Foundation types: a) monopile foundation, b) jacket foundation, c) tripod foundation, d) gravity foundation, e) floating structure

Foundations are also classified into three categories according to the depth of the seabed. Gravity-based or monopile foundations are feasible for shallow waters (0–30 m), tripod or jacket foundations for transitional waters (30–50 m) and the floating concept is the best for deep waters (50–200 m). It is worth noting that, presently, steel monopiles can be installed in water depths of 40 to 45 m. Approximately 20% of currently installed OWTs are supported by gravity foundations and 75% by monopile structures [8, 10]. Most small OWTs are located at relatively small depths (up to 20 m) and close to the mainland (up to 20 km). However, it is anticipated that by 2030, the location of wind farms will be standard in 60 m seas and up to 60 km from the mainland [21].

Tables 3 and 4 give very preliminary estimated dimensions for a few different sizes of turbines and the two types of foundation considered in this paper, i.e. gravity and monopile [16].

Table 3. Basic dimensions of gravity foundations for turbines of different power

Gravity	3.0 MW	3.6 MW	4.0 MW	8.0 MW
Shaft diameter	3.5–5.0 m	3.5–5.0 m	4.0–5.0 m	5.0–6.0 m
Width of base	18–23 m	20–25 m	22–28 m	25–35 m
Ballast	Infill sand	Infill sand	Infill sand	Infill sand

Table 4. Basic dimensions of monopile foundations for turbines of different power

Monopile	3.0 MW	3.6 MW	4.0 MW	8.0 MW
Outer diameter at seabed level	4.5–6.0 m	4.5–6.0 m	5.0–7.0 m	6.0–8.0 m
Pile length	50–60 m	50–60 m	50–60 m	50–70 m
Ground penetration (below mud line)	25–32 m	25–32 m	26–33 m	28–35 m
Transition piece				
Length	10–20 m	10–20 m	10–20 m	15–25 m
Outer diameter (based on a conical monopile)	3.5–5.0 m	3.5–5.0 m	4.0–5.5 m	5.0–6.5 m

GRAVITY FOUNDATIONS

A gravity foundation is a shallow foundation resting on the upper sediment layer, where the soil lying directly under the seabed has adequate bearing capacity. It transfers the loading by a large base to the seabed and resists overturning

loads solely by means of its own gravity. Such a foundation must also provide sufficient resistance against sliding and vertical bearing capacity. It is made from concrete, with or without small amount of steel, and its weight is in a range from 1400 to over 3000 tons. Reinforced gravity foundations are built onshore and floated out to sea where they are filled with gravel and sand. They are used on suitable soil sites in shallow waters or where installation of piles in the underlying seabed is difficult. Gravity foundations are uneconomical in deeper waters due to the very high moment loads applied by the wind turbines. The increase in mass of such foundations with water depth follows an approximately quadratic relation. Also, a disadvantage is the necessity of proper preparation of the seabed to ensure the structure's stability. The benefit of installing gravity foundations is that they can be floated out to their location, although heavy lifting vessels are needed for those of great weights. Installation requires little time and favourable weather conditions.

Gravity foundations for OWTs are usually used at water depths of up to 25 m. A reasonable concept for a gravitational-type foundation intended for waters with a depth of up to 40 m, including numerical analysis, has been proposed by Niklas [22].

MONOPILE FOUNDATIONS

Monopiles consist of a foundation pile and a transition part, on top of which the turbine tower is placed (Fig. 1). The weight of each element usually does not exceed 250 tons. A monopile support structure is exposed to both vertical and horizontal loads. The former are carried by the pile wall friction and tip resistance and the latter are transferred to the soil by mobilizing the lateral resistance of the soil through bending.

The foundation pile is made from steel plate which is rolled and welded together to form a cylindrical section. The length of the monopile in general depends on the overturning capacity under extreme conditions, or the maximum allowable tilt of the OWT due to accumulated rotations from cyclic loading [17]. Its diameter is usually governed by requirements for the frequency of the turbine that is closely related to the stiffness response of the soil. The wall thickness is usually determined by employing either fatigue loads or shell buckling during the installation phase. Monopiles for OWTs are usually 30–40 m long. The diameters of steel piles are in the range 3.5–6 m and wall thicknesses are as much as 150 mm. They can be classified as rigid monopiles for which the length to diameter ratio (L/D) is generally less than 12. Depending on subsoil conditions, piles are driven using hammers or vibrators, using drilling or excavation of seabed material until the final depth, typically 20–40 m, is reached. An embedment length of 30 m (5–6 times the diameter) is usually considered sufficient to meet design criteria, including vertical stability and horizontal deflection requirements.

For larger wind turbines of higher power, the monopile diameter would have to be as large as 7.5 m [3]. Scharff and Siems [28], in a study based on a new-generation 6 MW turbine situated at a water depth of 35 m, found that the diameter of the monopile should be over 8 m. According to Saleem [27],

an increase in the diameter of the monopile by 1 m generally means that the pile can be installed in water 10 m deeper. The diameter limit these days is around 6 m, so a 7 m diameter monopile might be installable in water depths of around 40 m. However, it should not be presumed that this statement, i.e. a linear relation, is true for greater depths.

Although monopile foundations can reduce material costs while maintaining performance, they require heavy equipment for installation that may cause considerable vibration, noise and suspended sediment. So, some environmental issues should be considered for this technology. Advantages include minimal seabed preparation requirements and resistance to seabed movement, scour or ice flow damage. Also, there are relatively small production costs due to the simplicity of the structure, storage and installation although there is the high price of steel and fabrication of the pile itself. At greater depths, the disadvantages are flexibility of the monopile, decreased stiffness or the cost of installation due to the time the grout needs to set. Monopiles are not suitable for locations with many large boulders in the seabed. Also, at sites where drilling is necessary, installation is very slow and expensive. Some difficulties are relevant during production and installation of large-diameter piles because of limitations on available steel plate sizes and pile driving capacity.

FOUNDATION SELECTION

In order to choose the best foundation type, first of all water depth and geotechnical site conditions should be considered. Scour and erosion potential and environmental loading conditions must also be taken into account. In addition, the economic issue should be considered as well.

Comparison analysis considering multiple engineering, economic and environmental attributes for three foundation types (monopile, tripod and jacket) for a 5.5 MW wind turbine and water depth of 40 m has been performed by Lozano-Minguez et al. [18]. They concluded that the tripod is the best option overall although the monopile is the most economical and less harmful to the environment, because the former suffers less from wave-resonance.

Monopiles are commonly used where the water depth is less than 35 m and the near-surface soil conditions are poor. Compared to monopiles, gravity foundations are more onerous considering their transport, installation, scouring protection, dependency on subsoil conditions or removal after the design life. Besides that, they have a minimal and localized environmental impact.

In shallow waters, monopile foundations are the best solution for OWTs up to 3.6 MW, whereas gravity foundations are most suitable for big OWTs [20]. Some considerations aiming to find the optimal foundations of the OWT with a size greater than 10 MW were carried out in [26].

GENERAL ANALYSIS AND DESIGN CONSIDERATION

Foundations for OWTs are generally more complex than for onshore turbines. Proper analysis should include such additional factors as the harsh marine environment as well as the short- and long-term impacts under wave loading. The offshore environment loadings from wind and waves are characterized by a large number of load cycles that affect the seabed and can significantly change soil properties. In turn, they influence the foundation's response to the loads. It is of great importance to investigate the effects of cyclic loading on the structures supporting OWTs. It is essential to perform both short- and long-term analyses, which are neither incompatible nor mutually exclusive. The former in saturated soils leads to a build-up of water pressure and eventually liquefaction phenomena that cause foundation failure. In the latter case, soil densification and hardening are in general observed. It is also important to distinguish fully drained, partially drained and undrained conditions. For example, the accumulating displacements in drained sands can lead to foundation failure. The most remarkable effect of cyclic lateral loading, in comparison to its static behaviour in the case of monopiles, is an increase of deflection and rotation.

An appropriate technical design for the foundations of OWTs is crucial for the safe, efficient and economic development of offshore wind farms. The main objective of the support structure design is to determine the dimensions of its components, taking into account operability, load resistance and economics. The design procedure for OWT foundations is typically based on design standards. The detailed specification of foundation design procedures is referred to in [7].

The most commonly used method for OWT foundation analysis is the limit states design method. There are four types of limit state: the ultimate limit state (ULS), fatigue limit state (FLS), accidental limit state (ALS) and serviceability limit state (SLS). ULS verifies the strength and stability of the foundation while SLS checks its maximum displacement. FLS verifies that the structure is able to withstand accumulated damage throughout its design life, which is critical for large modern OWTs. For offshore foundation design, only ULS uses non-degraded soil parameters. Other limit states use cyclically degraded soil properties for design calculations.

For design purposes of gravity-based foundations, the following factors should be accounted for: 1) total stability failure, 2) rupture in soil-carrying capacity, 3) sliding ruptures, 4) combined ruptures in soil and structure, 5) ruptures due to foundation movements, 6) unacceptable movements and oscillations, 7) eigenfrequency analysis, 8) liquefaction risk analysis (when set upon sandy soils), 9) zones of local strong soil or rock and 10) design of the gravel bed (differential settlement analysis, requirement for grading curve, levelling of gravel base, base minimum thickness) [29, 30].

Piled foundations should be examined in the following contexts: 1) elastic ULS, where only one pile per foundation is allowed to reach the yield point as a maximum, 2) plastic ULS (accounting for cyclic load strength degradation), where piles

are allowed to yield if still absorbing design loads, 3) fatigue in terms of both actual fatigue load on structure and the damaging effects of pile driving, 4) pile driving analysis, 5) eigenfrequency analysis (i.e. characteristic representation of the dynamic response of the pile) and 6) a soil damping estimate [4, 30].

When designing monopile foundations, it is rare that the ultimate lateral bearing capacity is a decisive factor. More important is the monopile's stiffness due to possible fatigue failure. Thus, the pile–soil interaction problem must be analysed. Design recommendations for monopiles [7] adopt the soil pressure–pile deflection (p-y) method. However, there are other methods of analysing laterally loaded piles like the limit state method, subgrade reaction method, elasticity method and FEM. The last one is an important research tool for soil–structure interactions and optimizing the support design.

A review of modelling of soil–OWT structure interaction, including modelling of soil and foundations, is given in [23]. Presently, FEM is commonly used in studies on modelling of OWT foundations [2, 11, 12, 13, 15, 19]. Many computer programs using FEM have been developed for offshore settings (e.g. ABAQUS, PLAXIS and ROSAP).

NUMERICAL ANALYSIS

DESCRIPTION OF THE WIND TURBINE TOWER

Finite element calculations were carried out for a steel wind turbine tower with a height of about 80 m and rotor blade of about 80 m. The tower of the wind turbine was tubular with diameter of 4 m at the bottom and 2.3 m at the top, with a varying cross-section thickness ($t = 14\text{--}36\text{ mm}$) along the column height. Two types of turbine foundation were considered.

In the first variant, a gravity foundation was analysed (Fig. 3a). The wind turbine tower was set on a concrete column, 8 m high and diameter $D = 5\text{ m}$, that was supported by a gravity base with a circular fundamental slab of diameter 18 m and

thickness 2 m. Two soil blocks in the shape of a cylinder were included in the numerical model. The bottom block height was 32 m and the upper block height was 6 m. The block diameter was 32 m. In the upper part of the soil, a cut was made for the concrete foundation block.

In the second variant, a turbine resting on a monopile was considered (Fig. 3b). The monopile was 32 m long while it penetrated 24 m into the seabed. The diameter of the steel pile was 4 m and wall thickness was 70 mm. Two cylinder soil blocks were modelled. The bottom block height was 10 m and the upper block height was 24 m. The block diameter was 32 m. The soil block inside the monopile was also modelled. The Mohr–Coulomb model was used for the soil. The following soil parameters were assumed: density 2.1 t/m^3 , Young modulus 20 MPa, Poisson's ratio 0.25, friction angle 15° , cohesion 27 kPa and dilation angle 0.1° .

The wind turbine tower was loaded by inertia and gravity loads, wind and wave loads (Fig. 3c). In the present analysis, the wind load was taken according to the Polish code [25] for a characteristic wind speed of 30 m/s and air density of 1.23 kg/m^3 , resulting in a characteristic wind speed pressure of 550 Pa. Wave load predictions should account for the size, shape and type of the proposed structure. In this paper, the wave forces on a slender wind turbine tower cylinder submerged in water were predicted by Morison's equation. In this model, wave load F per unit length is given in [26]:

$$dF = dF_M + dF_D = C_M \rho \pi \frac{D^2}{4} \ddot{x} dz + C_D \rho \frac{D}{2} |\dot{x}| \dot{x} dz, \quad (1)$$

where the inertia coefficient C_M was set at 2, drag coefficient C_D was equal to 1, and ρ , the density of water, is 1000 kg/m^3 . Water depths and wave data for an example case were taken according to [26]. The wave height was assumed to be 3.5 m and its length 45 m. The wave force is a function of time and reaches a maximal value described by following formula:

$$F_{\max} = C_M \rho \pi \frac{D^2}{4} U \frac{\sinh(kd)}{k}, \quad k = \frac{2\pi}{\lambda}, \quad (2)$$

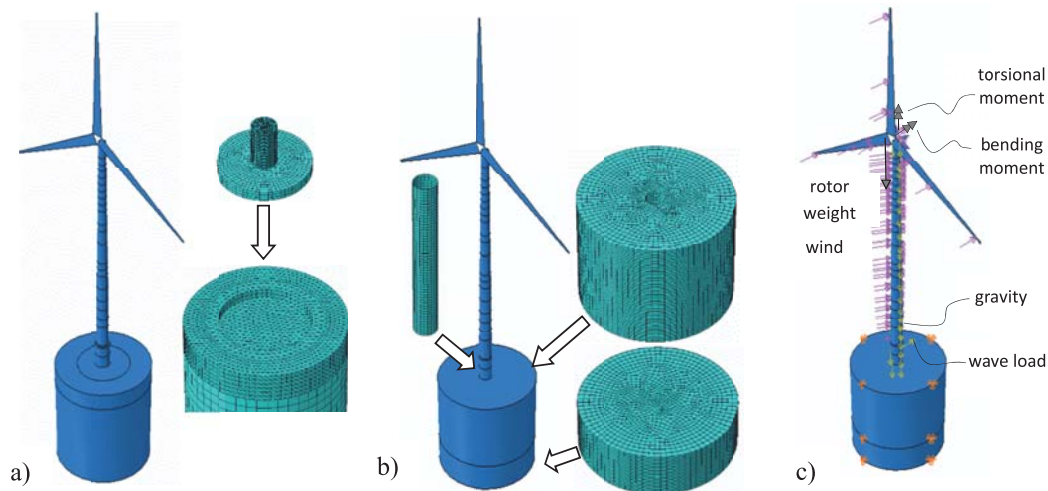


Fig. 3. Wind turbine tower resting on: a) gravity foundation, b) monopile foundation; c) loads applied on the OWT

where the constant U represents the maximal velocity at the bottom and λ is wavelength. The concrete column supporting the wind turbine tower was subjected to waves with a ratio between inertia and drag force amplitude of 9.8, causing a maximum horizontal wave force of 2793 kN whose vertical arm measured from the sea floor was 4.37 m. The forces caused by the blades and rotor were taken according to [31]: concentrated vertical force 1200 kN, torsional moment 215 kNm and bending moment 2733 kNm at the top of wind turbine tower.

Numerical static analysis was performed by means of a commercial program [1]. Both non-linear static (the arc-length method by Riks) and implicit dynamic (quasi-static) analysis was used.

In the numerical model of a tower set on a gravity foundation, 21756 four-node doubly curved thin or thick S4R shell elements with reduced integration were used to simulate the wind turbine tower, and 21492 eight-node hexahedral finite C3D8R elements with reduced integration were used for the fundament slab, concrete column and soil. The upper part of the soil was modelled with 37195 ten-node quadratic tetrahedral C3D10 elements. For the tower set on the monopile, 22428 S4R shell elements were used to simulate the wind turbine tower, and 59276 eight-node hexahedral C3D8R elements for the soil. The soil parameters were the same in the two models and the ground layers. The soil was divided into different zones in order to save the number of finite elements. The wind turbine rotor blades were modelled by an approximate method using shell S4R elements as a flat plate. The blades were included in the model to calculate the wind load only. The gravity of the blades was taken into account in the form of the concentrated force at the top of the tower. The rotor blades were eccentrically connected to the wind turbine tower by a rigid connection. The contact between the fundament or monopile and the soil was modelled with possible separation between the slab and the soil. The contact conditions between the fundament slab or monopile and the soil were controlled in the normal and tangential directions. The friction coefficient was 0.2. The current load and ice loading were omitted in the analysis performed.

RESULTS OF NUMERICAL SIMULATIONS

Wind turbine tower on a gravity foundation

In the case of the tower on a gravity foundation, the load was applied in two steps: in the first, the tower dead weight was applied; in the second, the remaining load as for example wind or wave loads. The results of the numerical analysis of the wind turbine tower are presented in Figs. 4–6. The static solution lost convergence in the second step at load level 0.817 that corresponded to a tower top displacement of 1.09 m. In this case, dynamic analysis was used. In the quasi-static implicit dynamic analysis, the dead weight was applied in 1 s and the wind, sea and turbine loads in 5 s. The displacement of the tower top increased to 1.39 m (Fig. 4b); the vertical displacement of the foundation slab was 5.4 cm for static and 7 cm for dynamic analysis (Fig. 5). The zone of separation of the foundation slab from the ground increased significantly. It expanded to the

reinforced concrete column (Fig. 5). The displacement of the top of the tower for different soil conditions was 0.96 m [26], while the vertical displacement of the fundament slab was 0.027 m. Geodetic monitoring of a similar wind turbine tower situated on the land on a square fundament slab (16 m × 16 m × 2 m) [31] showed maximum displacement of about 0.40 m for a measured wind speed of 10 m/s and predicted displacement of about 0.9 m for a wind speed of 24 m/s.

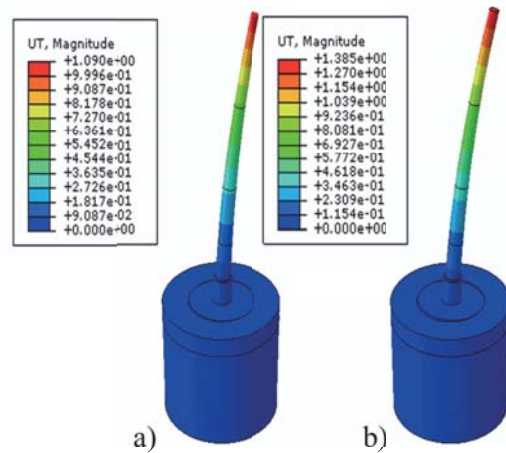


Fig. 4. Displacement of wind turbine tower on gravity foundation: a) non-linear static analysis for load factor 0.817, b) implicit dynamic analysis for load factor 1.0 (scale by factor 10 in m)

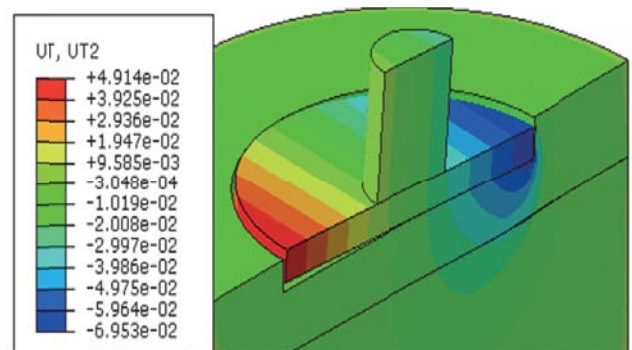


Fig. 5. Vertical displacement of wind turbine tower gravity foundation: implicit dynamic analysis for load factor 1.0 in m

Forces acting on the foundation, including loads exerted by wind and waves, caused maximal compression stress in the soil equal to 170–200 kPa (Fig. 6a). This effect caused a non-linear relation between the load and the displacement of the tower, resulting in less rotational stiffness of the foundation. The maximal von Mises stress in the steel tower equal to 125 MPa was calculated at the bottom of the steel tower (Fig. 6b).

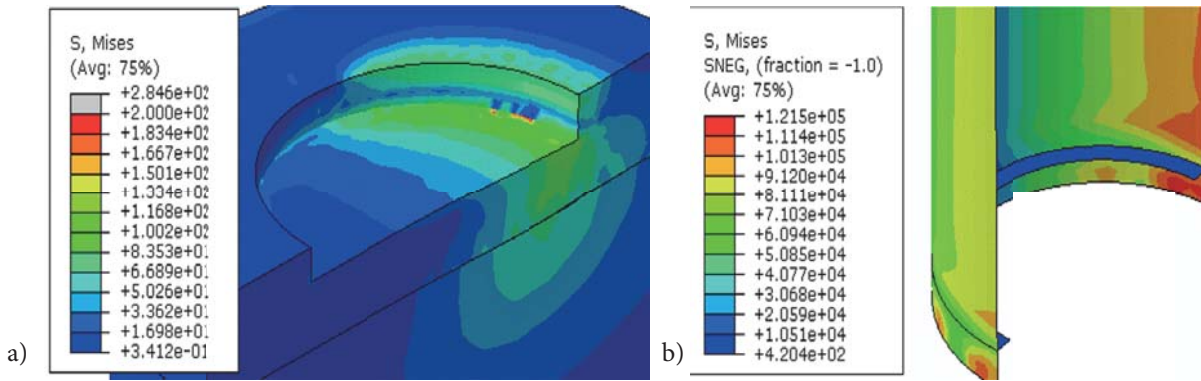


Fig. 6. Von Mises stress for wind turbine tower resting on gravity foundation in: a) soil, b) steel tower in kPa

Wind turbine tower resting on a monopile

In the case of the tower resting on a monopile, the load was applied in one step. The computed results of the wind turbine tower numerical analysis are presented in Figs. 7 and 8. The static solution lost convergence at a load level of 0.387 that corresponded to tower top displacement of 0.60 m. In this case, dynamic analysis was used. In the quasi-static implicit dynamic analysis, all of the load was applied in 5 s. The displacement of the tower top increased to 2.49 m (Fig. 7a), while the displacement of the foundation slab was 0.21 m. A zone of separation of the monopile from the ground was observed. The displacements of the top of the tower were 1.8 times higher than for a gravity foundation.

Forces acting on the foundation, including loads exerted by wind and waves, caused maximal compression stress in the soil equal to 150 kPa (Fig. 8a). The maximal von Mises stress in the steel tower equal to 235 MPa was calculated at the bottom of the tower (Fig. 8b). Displacements of the top of a similar tower grounded on a monopile with different soil conditions for short-term cyclic loading in SLS were 0.764 m and for ULS were 1.608 m [19].

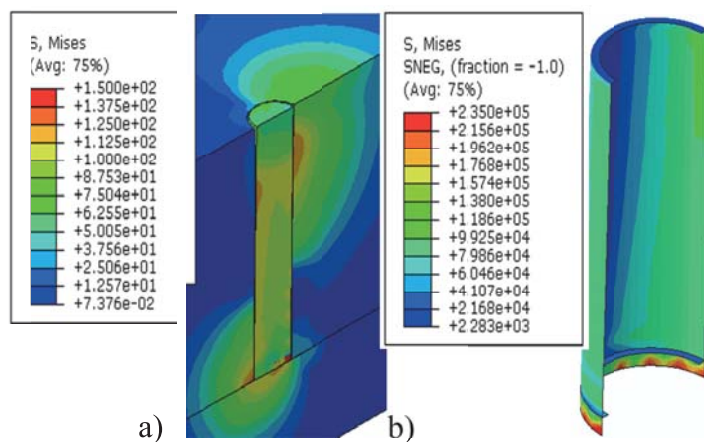


Fig. 8. Von Mises stress for wind turbine tower resting on monopile in: a) soil, b) steel tower in kPa

CONCLUSIONS

Construction of OWTs seems to be one of the best ways of producing renewable energy. However, besides some technical problems or more expensive maintenance and service facilities, their main disadvantages are the costs of building. Thus, an optimization procedure for both the structure and foundation should be performed. Considering that the dimensions of turbines are still increasing, their proper analysis is particularly important. Foundations of OWTs are one of the most important and expensive parts of turbines; therefore, they require special treatment. The short review presented in this paper should provide some knowledge about the two basic types and direct further research.

Numerical analysis of wind turbine towers requires advanced and costly computational programs. The performed analysis leads to the conclusion that an approximate approach for wind towers may be carried out by FEM with Abaqus software [1]. The soil–structure interaction is a non-linear problem, thus obtaining a solution by static analysis could be difficult or even impossible due to the lack of convergence. In such a case, dynamic analysis

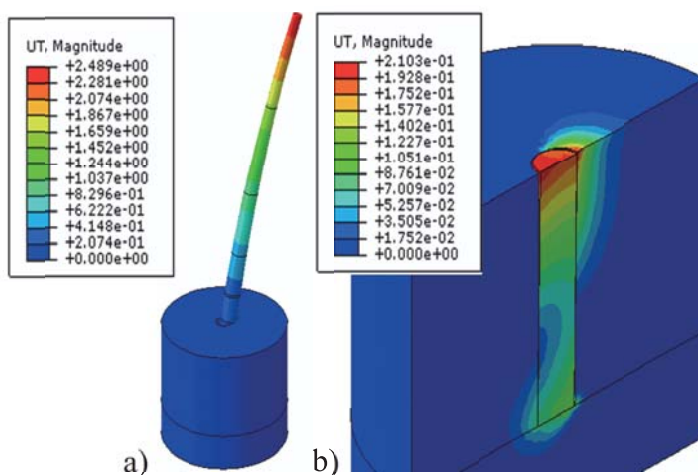


Fig. 7. Displacement of wind turbine tower resting on monopile for load factor 1.0: a) whole tower, b) anchorage in the ground in m

should be applied. In order to choose the best foundation type, first of all water depth and geotechnical site conditions should be considered. The detachment of a gravity foundation results in a reduction of foundation stiffness. In the case of a wind tower resting on a monopile, deformation of the soil at the top and bottom of the monopile also causes a decrease in foundation stiffness.

REFERENCES

1. Dassault Systèmes. (2008): *ABAQUS Theory Manual, Version 6.8*, Hibbit, Karlsson & Sorensen Inc.
2. Achmus M., Abdel-Rahman K. (2005): *Finite element modelling of horizontally loaded monopile foundations for offshore wind energy converters in Germany*. In: Proceedings of the First International Symposium on Frontiers in Offshore Geotechnics (ISFOG), 391–396, doi: 10.1201/NOE0415390637.ch38
3. Achmus M., Kuo Y., Abdel-Rahman K. (2009): *Behaviour of monopile foundations under cyclic lateral load*. Computers and Geotechnics, Vol. 36(5), 725–735.
4. Arany L., Bhattacharya S., Macdonald J., Hogan S. J. (2017): *Design of monopiles for offshore wind turbines in 10 steps*. Soil Dynamics and Earthquake Engineering, Vol. 92, 126–152.
5. Bhattacharya, S., Nikitas, G., Arany, L., Nikitas, N. (2017): *Soil-structure interactions for offshore wind turbines*. Engineering and Technology Reference, Vol. 1(1), 1–16. <http://digital-library.theiet.org>
6. Byrne B. W., Houlsby G. T. (2003): *Foundations for offshore wind turbines*. Philosophical Transactions of the Royal Society of London Series A: Mathematical, Physical and Engineering Sciences, Vol. 361, 2909–2930.
7. DNV (2010): *DNV-OS-J101 Design of Offshore Wind Turbine Structures*.
8. EWEA (2013): *Deep Water—the Next Step for Offshore Wind Energy*. July 2013.
9. EWEA (2009): *Pure Power: Wind Energy Targets for 2020 and 2030*. Technical Report of the European Wind Energy Association. URL <http://www.ewea.org/>.
10. Gavin K. G., Igoe D. I., Doherty P. (2011): *Piles for offshore wind turbines: A state of the art review*. Proceedings of ICE Journal, Geotechnical Engineering, Vol. 164(4), 245–256.
11. Haiderali A., Cilingir U., Madabhushi G. (2013): *Lateral and axial capacity of monopiles for offshore wind turbines*. Indian Geotechnical Journal, Vol. 43(3), 181–194, doi: 10.1007/s40098-013-0056-4
12. Hansen N. M. (2012): *Interaction Between Seabed Soil and Offshore Wind Turbine Foundations*, PhD thesis, Department of Mechanical Engineering, Technical University of Denmark.
13. Hearn E. E., Edgers L. (2010): *Finite element analysis of an offshore wind turbine monopile*. In: GeoFlorida 2010: Advances in Analysis, Modeling & Design (GPS 199), 1857–1865.
14. Houlsby G., Ibsen L., Byrne B. (2005). *Suction caissons for wind turbines*. In: G. A. Cassidy (Ed.), Frontiers in Offshore Geotechnics: ISFOG 2005, pp. 75–93, Taylor and Francis Group, London.
15. Jung S., Kim S.-R., Patil A., Hung L. C. (2015): *Effect of monopile foundation modeling on the structural response of a 5-MW offshore wind turbine tower*. Ocean Engineering, Vol. 109, 479–488.
16. Kriegers Flak (2013): *Technical Project Description for the Large-Scale Offshore Wind Farm (600 MW) at Kriegers Flak*. Technical project description.
17. LeBlanc C., Houlsby G. T., Byrne B. W. (2009): *Response of stiff piles in sand to long-term cyclic lateral loading*. Geotechnique, Vol. 60, 79–90.
18. Lozano-Minguez E., Kolios A. J., Brennan F. P. (2011): *Multi-criteria assessment of offshore wind turbine support structures*. Renewable Energy, Vol. 36(11), 2831–2837.
19. Maa H., Yanga J., Chen L. (2017): *Numerical analysis of the long-term performance of offshore wind turbines supported by monopiles*, Ocean Engineering, Vol. 136, 94–105.
20. Madariaga A., Martinez de Alegria I., Martin J. L., Eguia P., Ceballos S. (2012): *Current facts about offshore wind farms*. Renewable and Sustainable Energy Reviews, Vol. 16(5), 3105–3116.
21. Myszkowska A. (2014): *Fundamentowanie morskich farm wiatrowych. Doświadczenia z województwa zachodniopomorskiego*. Inżynieria Morska i Geotechnika, Vol. 3, 238–244.
22. Niklas K. (2017): *Strength analysis of a large-size supporting structure for an offshore wind turbine*. Polish Maritime Research, Special Issue S1 (93), Vol. 24, 156–165.
23. Oh K. Y., Namb W., Ryuc M. S., Kimc J. Y., Epureanu B. I. (2018): *A review of foundations of offshore wind energy convertors: Current status and future perspectives*. Renewable and Sustainable Energy Reviews, Vol. 88, 16–36.
24. Petersen T. U. (2014): *Scour Around Offshore Wind Turbine Foundations*, PhD thesis, Department of Mechanical Engineering, Technical University of Denmark.

25. PN-77/B-02011. (1990): *Obciążenia w Obliczeniach Statycznych. Obciążenie Wiatrem*, PKN, Warszawa.
26. Przewłócki J., Iwicki P. (2016): *Some problems of supporting offshore wind turbines*. In: L. Małyszko, R. Tarczewski (eds.), XXII Lightweight Structures in Civil Engineering, pp. 71–76, Wydawnictwo UWM, Olsztyn.
27. Saleem Z. (2011): *Alternatives and modifications of monopile foundation or its installation technique for noise mitigation*. Report by Delft University of Technology for Stichting De Noordzee (the North Sea Foundation).
28. Scharff, R., Siems, M. (2013): *Monopile foundations for offshore wind turbines – solutions for greater water depths*. Steel Construction, Vol. 6(1), 47–53.
29. Tempel J., Diepeveen N. F. B., Salzmänn D. J., Vries W. E. (2010): *Design of support structures for offshore wind turbines*. WIT Transactions on State of the Art in Science and Engineering, Vol. 44, 559–591. www.witpress.com
30. Westgate, Z. J., DeJong J. T. (2005): *Geotechnical Considerations for Offshore Wind Turbines*, MTC-OWC.
31. Widerski T. (2013): *Pomiary i Prognozowanie Przemieszczeń Elektrowni Wiatrowych Metodami Geodezyjnymi*, PhD thesis, Department of Geodesy, Faculty of Civil and Environmental Engineering, Gdańsk University of Technology.

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