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Seismic considerations in design of offshore wind turbines

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ABSTRACT

Interest in renewable and clean energy over the past decade has motivated immense research on wind energy. The main issues in design of offshore wind turbines in regions of recent development have been aero- and hydrodynamic loads; however, earthquake is a design concern in seismic areas such as East Asia and Western United states. This paper reviews the state of practice in seismic design of offshore wind turbines. It is demonstrated that wind turbines are in particular vulnerable to vertical earthquake excitation due to their rather high natural frequencies in vertical direction; however, inclusion of the radiation damping could contribute considerably reduce the earthquake loads. Moreover, it is demonstrated how soil nonlinearity could lead to settlement and permanent tilting of offshore wind turbines on caisson foundations or tripods. Using these cases, the paper demonstrates that the design of offshore wind turbines for earthquake loading is driven by performance-based considerations.

1. Introduction

For much of the twentieth century there was little interest in using wind energy for generation of electricity. One notable development was the 1250 kW Smith-Putnam wind turbine constructed in the USA in 1941. Shepherd [1] and Divone [2] have presented the history of early wind turbine development. Their reviews include, among others, the 100 kW 30 m diameter Balaclava wind turbine in Russia in 1931, the Andrea Enfield 100 kW 24 m diameter pneumatic generator designed and constructed in the UK in the early 1950s, the 200 kW 24 m diameter Gedser machine in Denmark in 1956, and the 1.1 MW 35 m diameter turbine tested by Electricite de France in 1963.

Despite some technical advances and other initiatives, there was no systematic interest in wind generation until the oil crisis in the early 1970s. The dramatic increase in the oil price motivated considerable government-funded research programs. For example, in the USA, the research led to construction of a series of prototype turbines, starting with the 38 m diameter 100 kW Mod-0 in 1975 and evolving to 97.5 m diameter 2.5 MW Mod-5B in 1987 [3]. Similar research programs were initiated in the UK, Germany and Sweden. Large turbines were constructed with two and three blades until the Danish wind turbine concept emerged of a three-bladed, upwind rotor (i.e. machines that have the rotor facing the wind) and a fixed-speed drivetrain (gearbox and generator). This design proved to be successful and was implemented on turbines as large as 60 m in diameter and at ratings of up to 1.5 MW [3].

Wind quality is better at sea than on land. The smoother surface at sea compared to land results in stronger and less turbulent wind that ensures a greater and more reliable power production. It also allows for the use of larger turbines and lower elevation above ground/water level. Moreover, areas with good wind quality for energy production are limited on land and are often far from big cities. From an environmental viewpoint, placing the turbines offshore makes them less visible and causes less noise for the public.

In 1991 the first offshore wind farm was constructed at Vindeby, Denmark, consisting of eleven, 450 kW wind turbines located up to 3 km offshore. Throughout the 1990s small numbers of offshore wind turbines (OWT) were placed close to shore, until in 2002 the Horns Rev, 160 MW wind farm, about 20 km off the western coast of Denmark, was constructed. This was the first project to use an offshore substation that increased the power collection voltage of 30–150 kV for transmission to shore. OWTs have kept increasing in size over the years. The largest OWT at the time of this publication is 8 MW, is 220 m high, has a rotor blade length of 80 m and weighs 5900 t. A wind farm, consisting of 32 of these turbines, has recently been developed at the Burbo Bank Extension offshore wind farm in Liverpool Bay, off the west coast of England.

From the early 1990s a new driver for use of wind turbines has been the low CO_2 emissions and the potential of wind energy to help mitigate climate change. In 2007, the European Union declared a policy that 20% of all energy should originate from renewable sources by 2020. Because of the difficulty of using renewable energy for transport and

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heating, this implies that in some countries 30–40% of electrical energy should come from renewables. Energy policy continues to develop rapidly with many countries aiming at reducing greenhouse gas emissions of up to 80% by 2050. Currently, the global market for offshore wind energy is dominated by Europe where about 90% of the world's offshore wind turbines are installed [4]. However, the market is expanding further outside of Europe. In 2015, China was ranked fourth in offshore wind capacity when combining operating projects and projects under construction.

This paper reviews some of the key issues in earthquake analysis and design of OWTs. The objective is not present a state-of-the-art review of the various subjects related to design of OWTs. For these subjects, the reader is referred to dedicated papers. For example, a comprehensive review of the literature related to the seismic aspects of wind turbine design has recently been published [5]. Kausel [6] has presented the state-of-the-art in dynamic soil-structure interaction. Dobry and Abdoun [7] and Boulanger and Idriss [8] have presented reference articles on the assessment of liquefaction. Jostad and Andersen [9] have given a detailed account of the various geotechnical issues in the cyclic response of skirted (bucket) foundations that also apply to monopod/caisson foundations in OWTs, and Jardine et al. [10] have reviewed the behavior of piles under cyclic loading. Other researchers have provided detailed account of general dynamic behavior of OWTs [11,12]. They discuss the wind and wave power spectral densities, which essentially describe the frequency content of dynamic excitations from wind turbulence and waves. They also describe the dynamic loading due to the rotation of the blades at the so-called 1P frequency, typically in the range 0.12–0.3 Hz, and the load associated with passing of the blades by the tower with a frequency referred to as 3P frequency, typically in the range 0.35–0.9 Hz. Further, they present a simplified way of translating the wind turbulence and spectra and the wave spectrum into mudline bending moment spectra through linear transfer functions [13].

Mono-pile, Fig. 1(a), is the most common foundation type for OWTs. More recently, monopods (Fig. 1(b)), jacket structures (Fig. 1(c)), and tripods (Fig. 1(d)) have been tried in several windfarms. Use of mono-piles is primarily steered by the weight of the pile and the installation capabilities. Currently, mono-piles can be installed in water depths of up to about 40 m depending on the soil type. Mono-pods and jackets can be used for deeper waters due to the lower weight of the supporting structures. For larger water depths, the economical solution is floating wind turbine anchored to the seabed by various types of anchors (Fig. 1(e)).

For the discussions in the following sections, Fig. 2 presents the terminology used in wind turbine technology. The power produced by wind is proportional to the third power of wind speed and the second power of the rotor radius. Therefore, the means to produce more energy is to increase the rotor diameter and hub height.

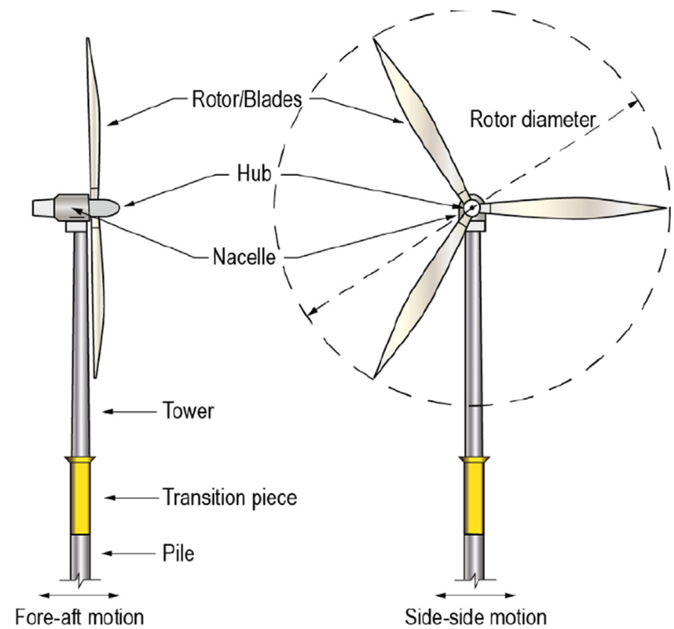

Fig. 2. Terminology used in offshore wind turbine.

2. Earthquake codes and guidelines

There exist less detailed guidelines about earthquake design of wind turbines in comparison to buildings and most other structures. The majority of the available guidelines and codes are based on those for buildings; therefore, they use earthquakes with return period of 475 years. Prowell and Veers [14] have provided a detailed account of relevant codes and guidelines. According to their review, four main guidelines are available for seismic design of wind turbines: Risø Guidelines for Design of Wind Turbines [15], Guideline for the Certification of Wind Turbines by Germanischer Lloyd [16], International Electrotechnical Commission's IEC 61400-1: Wind turbines - Part 1: Design requirements [17], and DNV [18].

Risø [15] proposes a simple structural model for estimation of the natural frequency and computation of the maximum acceleration from a design response spectrum. In this model, the mass is lumped at the top of the tower and includes the nacelle, rotor, and one-fourth of the tower mass (Fig. 2). However, no recommendation is given for the damping ratio. This is indeed an important issue considering that acceleration response spectra are traditionally computed for 5% damping ratio while damping in the side-side vibration (i.e. parallel to the plane of blades, Fig. 2) is generally in the range 0.5%–1% [19]. On the other hand, in the fore-aft direction (i.e. normal to the plane of blades, Fig. 2), the

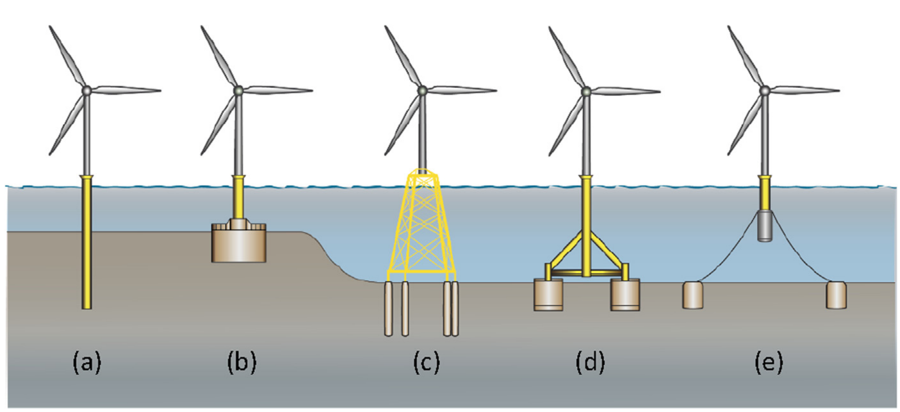

Fig. 1. Common foundation types used in offshore wind turbine design: mono-pile (a), mono-pod (skirted caisson) (b), jacket structure (c), tripod (d) and floating wind turbine with anchors (e).

damping ratio can reach as high as 5% during operation due to aero-elastic effects related to blade rotation as verified by measurements and computations using aero-elastic codes [20].

The GL guidelines [16] suggests use of seismic specifications in either local building codes, Eurocode 8 or the American Petroleum Institute [21] in consultation with the certifying agency. It also prescribes a return period of 475 years and that the resulting earthquake load with a load factor of 1.0 be combined with the design wind load cases during operation. In addition, the guideline considers a load case in which the earthquake load is combined with 80% of reference wind speed for the parked (standstill) wind turbine. Finally, this guideline requires that the tower be designed elastically unless the tower has characteristics that allow ductile response, such as a lattice tower. However, no guidance is provided regarding the level of damping.

The IEC guidelines [17] also prescribe the design earthquake event with 475-year return period, and requires that the resulting loads be combined with the greater of the lifetime-averaged operating loads or the emergency shutdown loads. This guideline recognizes the role of damping by providing a simplified procedure to adjust the design response spectrum from local building codes to damping ratio of 1%. The earthquake load is computed by using only the fundamental mode of the tower response applied at the tower top. The earthquake load is then combined with the loads calculated for an emergency shutdown.

The most recent standard [18] requires the earthquake analyses in one vertical and two horizontal directions, and states that it usually suffices to reduce the analysis in two horizontal directions to an analysis in one horizontal direction due to the symmetry of the dynamic system. However, this is not quite correct because of the difference in the damping and dynamic responses in the side-side and fore-aft directions. This code does not specify any earthquake return period, but it refers the users to the offshore code ISO 19901-2 [22] for the seismic design criteria.

The design philosophy in [22] is reflected in the following two performance expectations:

1. Little or no damage or interruptions to normal operations during frequent earthquakes referred to as Extreme Level Earthquake (ELE).
2. No serious HSE (Health, Safety and Environmental) consequences in rare earthquakes referred to as Abnormal Level Earthquake (ALE) although the facility could be irreparable and result in an economic loss.

ISO [22] has established a procedure for determination of the design return periods based on the seismic hazard condition at the site, ductility characteristics of the structure and accepted risk. The procedure hinges on the concept of Exposure Level (L) that is connected to the probability of failure, P_f. Three exposure levels are considered according to Table 1.

The exposure levels are linked to the Life-safety Category, S, and Consequence Category, C per [23]. There are three life-safety categories, S1 to S3, where S3 is for unmanned structures, and hence applies to OWTs. There are also three consequence categories, C1 to C3, from high to low consequences. Combination of S3 and C1 (high consequence) would result in Exposure Level L-1 with $P_f = 2.5 \times 10^{-4}$. Following the procedures in ISO, one would arrive at return periods typically around 3000 years for ALE events. Using a so-called seismic reserve strength factor 2, principally similar to the ductility factor, one could compute a return period around 500 years for the ELE event. This is consistent with the requirements in [16] and [17] described above. Accepting a lower failure consequence, for example, C2, would result in typical return periods of about 1500 and 300 years for the ALE and ELE events, respectively.

Table 1
Target annual probability of failure [22].

Exposure Level	Probability of failure, P_f
L-1	$4 \times 10^{-4} = 1/2500$
L-2	$1 \times 10^{-3} = 1/1000$
L-3	$2.5 \times 10^{-3} = 1/400$

In summary, one can draw a consensus on the choice of return period for earthquake design of offshore wind turbines being equal to 475 years. This choice of return period has the advantage that allows use of seismic hazard data given in the local building codes. The tower structure is expected to behave elastically during this event. The analysis for the ALE event, under which the structure is allowed to undergo severe damage beyond repair, does not appear to be relevant for OWTs. Nevertheless, it is logical to think that the owners would be interested to know the extent of tower damage during an ALE event.

In regions with medium to high seismicity and soft to medium soil conditions, one would expect a high degree of soil nonlinearity. The additional soil nonlinearity due to the combined effect of wind and inertia forces on the structure would lead to non-symmetrical inelastic response of the foundation that could result in a permanent tilt of the tower, especially in the case of OWTs on mono-pods and tri-pods. An example of this type of response is presented in Section 3.1. It is expected that an OWT would continue operation after an ELE event; therefore, in the cases described above, it might be necessary to perform nonlinear SSI analyses for estimating post-earthquake conditions of the turbine.

2.1. Performance requirements

Deformation tolerances are usually specified in the design basis of the turbine and are often specified in terms of allowable accumulated (permanent) rotation of the foundation. The permanent rotation results from plastic soil deformations caused by the cyclic wave and wind loads. Apart from visual consideration, the deformation tolerances are often related to requirements for the satisfactory operation of the wind turbine and are therefore specified by wind turbine manufacturers. These tolerances include an installation tolerance that is typically 0.25°. From the experience of design of OWTs, the allowable permanent tilt is about 0.75°. Therefore, for an OWT to be acceptable after an earthquake event, it is necessary that the permanent tilt due to the earthquake loading should not exceed 0.5°. In view of this strict performance criterion, one needs robust analysis tools for estimating the permanent rotation of the foundations during the design earthquake.

3. Modelling for earthquake loads

Numerical analyses of seismic soil-structure interaction (SSI) of wind turbines using a variety of linear and nonlinear solution algorithms have clearly shown that inclusion of SSI can reduce the earthquake loads on wind turbines by as much as much as 10% [24–26].

Prowell et al. [27] reported the results of a comprehensive study on the seismic response of a 65-kW wind turbine on the large high performance outdoor shake table at the University of California, San Diego. Prowell et al. [28] also modified the numerical code FAST (*Fatigue, Aerodynamics, Structures, and Turbulence*) [29], which is an open source software for hydro-aero-elasto-dynamic modelling of OWTs under wind and wave loading, to enable base shaking in addition to other loads. The shake table did not have the possibility of including SSI for this large turbine; therefore, Prowell et al. [30] used the finite element software OpenSees (*Open System for Earthquake Engineering Simulation*) [31] and conducted numerical studies of the 5 MW reference wind turbine developed by National Renewable Energy Laboratory (NREL) [29]. This wind turbine was developed by using publicly available information on the structural, operational and other aspects of wind turbines that existed at the time and has been serving as a baseline in research on megawatt-scaled wind turbines. The reference turbine has a weight of about 700 t, height of 88 m, and rotor diameter of 126 m. The tower diameter and wall thickness are 6 m and 27 mm at the base, respectively [29].

Similar attempts have been made to include seismic analyses in other hydro-aero-elasto-dynamic codes. For example, Witcher [32] presented aseismic computational module in the software GH Bladed [33] for analyses of a 2-MW turbine with a 60 m tower. The analyses were performed for four cases during the earthquake: continuous operation; emergency shutdown; parked turbine during, and parked turbine in combination with high winds. The last load case resulted in about 80% increase in moment demand for a time domain simulation from that calculated using a modal solution. More recent studies [34,35] that have included combined effects of earthquake and wind loads in the time domain have highlighted the importance of earthquake loading in the design of wind turbines.

Considering the rather low natural frequencies of OWTs (around 0.3 Hz), these structures are generally not vulnerable to horizontal earthquake shaking in low-to-moderate seismic shaking [36]. On the other hand, the natural frequencies of wind turbines in the vertical direction are fairly high (typically 4–7 Hz) that often coincide with the dominant frequencies of vertical earthquake shaking and make them quite vulnerable to even moderate shaking [37]. A detailed study [38,39] has demonstrated considerable amplification of earthquake motions in the vertical directions in OWTs and has indicated that vertical earthquake shaking could indeed be one of the governing design load cases. Similar observations have been made in other types of structures such as buildings where amplification by as high as 6 have been recorded in instrumented buildings [40].

In view of the types of dynamic behavior described above, it is convenient to treat the subject of seismic response of OWTs separately in the horizontal and vertical directions. In the former case, the challenge is computation of the nonlinear response of the soil and possible tilt of the tower, and in the latter case, the key issue is the potentially large amplification of earthquake motions. It should be noted that the responses in the horizontal and vertical directions are coupled if the earthquake response of the structure causes nonlinear soil behavior. However, in order to be able to highlight the main issues and challenges, it is assumed in the following that the two responses are uncoupled, hence they are treated separately.

3.1. Earthquake response in horizontal direction

The earthquake response of OWTs in the horizontal direction is governed by issues that have been known to the geotechnical earthquake engineering community for several decades. They include classical subjects such as liquefaction and its impact on the foundation, especially on mono-pods and tripods where there is a larger possibility of tilting, analysis of piles by rigorous solutions or p-y concepts, and use of direct finite element (FE) methods vs. springs/macro-elements through the conventional three-step SSI method. Each of these subjects have been addressed in detail in many specialized publications; therefore, in passing, only a brief account of the more important topics in relation to OWT is given in the following.

Mono-piles are the most common types of foundations in OWTs. In many cases, they are still analyzed by the traditional p-y spring approach [18,19] although many studies have pointed out to the inaccuracies of these springs especially for large piles and because of the soft behavior of these springs at small-to-medium range of deformation which is most important in design of OWTs. A number of studies have been carried out for improving the p-y curves with respect to both the diameter effect and stiffness at small deformations. A recent publication has provided an account of these studies and has summarized the proposed improvements mostly with focus on the developments made in the offshore oil industry [41]. Another study, initiated by the offshore wind industry has focused on piles with very large diameters and has used results of pile load tests and extensive FE modelling to propose new sets of lateral springs combined with rotational springs along the pile length [42]. These models can be extended to earthquake analyses by defining appropriate hysteretic cyclic behavior for the springs (e.g. [43]) and incorporation of additional elements to account for far-field response under dynamic loading [44]. Considerable research is needed to adapt these springs to soils prone to liquefaction or considerable pore-pressure generation.

Liquefaction is indeed a major challenge in areas of high seismicity. This is becoming more evident as development of wind farms is moving to seismic areas such as East Asia, for example Taiwan that has an ambitious program of building wind farms. Although liquefaction impacts all types of foundations, the effect is more dramatic on shallow foundations such as caissons, pods and anchors (Fig. 1). In the case of caissons and pods, the consequence of liquefaction (or large pore pressure generation) is primarily the permanent tilt, and in the case of anchors for floating OWTs (Fig. 1(e)), the extreme situation is that the anchors lose their holding capacity and are pulled out during earthquake. While centrifuge and small-scale shake-table tests are needed to shed light on the failure mechanisms and calibration of numerical tools, the constitutive models recently developed and implemented in FE codes (e.g. [45–47]) have provided convincing evidence of reliable performance, and that they could be used in OWT design during liquefaction until more specialized models are developed. The following section presents the use of a numerical tool in a closely related problem in which the earthquake load is applied simultaneously with the static wind load on a wind turbine. Kourkoulis et al. [48] performed a series of nonlinear dynamic analyses of two OWTs on caissons in two different clay soil profiles. They considered the effect of environmental loads, namely wave and wind loading, with and without earthquake excitation. Fig. 3 shows the key elements of the model.

The turbine was modelled as a single-degree-of-freedom system consisting of a beam, representing the tower, and a concentrated mass at the nacelle level. The turbine was founded on a caisson foundation with two alternative realistic designs: one with D = 20 m and L/D = 0.5, and the other with D = 25 m and L/D = 0.2 (Fig. 3). The other parameters for the 3.5 MW turbine were as follows: Height, H_0 = 90 m, Rotor diameter = 20 m, and sum of the masses of the nacelle and rotor = 220 t. The tower's diameter and wall thickness were selected such that the natural frequency of the structure was about 0.28 Hz

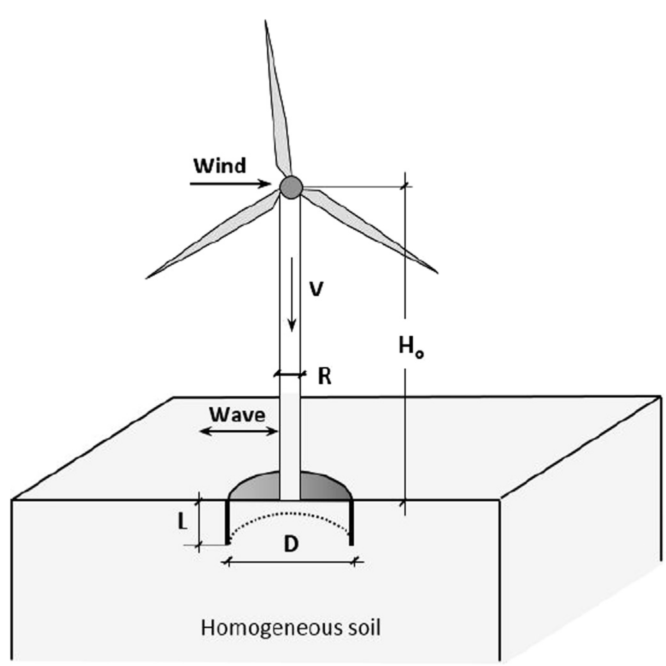

Fig. 3. Key elements and parameters of turbine on caisson foundation (modified after [48]).

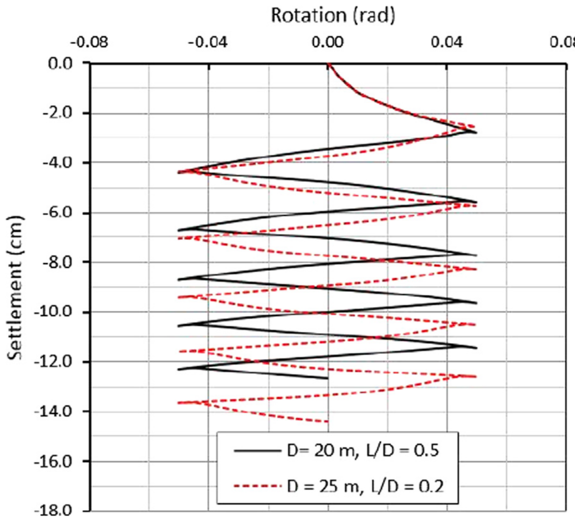

Fig. 4. Settlement as function of rotation of caisson for cyclic loading on tower.

which is in the typical range of natural frequencies of OWTs. The nonlinear soil behavior was modelled using a simple kinematic hardening model with Von Mises failure criterion and associated flow rule available in Abaqus [49] and validated for simulation of soil–structure interaction responses under cyclic and seismic loading for undrained condition [50]. To examine the performance of the model under cyclic loading, the foundation was subjected to a cyclic rotation with amplitude 0.03 rad. The undrained shear strength of the soil was taken s_u = 60 kPa. Fig. 4 displays the accumulation of vertical settlements for the two foundation cases.

The model was next excited at the base of the soil domain in horizontal direction by the modified Takatori, Kobe earthquake scaled to PGA = 0.35*g* (Fig. 5) and manipulated to represent the soil type D according to Eurocode 8. The soil profile was assumed to have a constant undrained shear strength, s_u = 120 kPa and shear modulus 600 times s_u. For simplicity, the environmental loads were taken as constant loads acting throughout the earthquake excitation. The simultaneous wind and wave loads were taken equal to 1000 kN and 2000 kN, respectively.

Fig. 6 displays the time histories of the caisson rotations. In some response cycles one can identify the natural frequency of the turbine around 0.28 Hz. The effect of one-way wind load on accumulating permanent tilt of the tower is quite evident in the figure. For earthquakes with longer return period, the permanent tilt could increase by two to three folds which might bring the tilt to unacceptable levels at the end of the earthquake. Similar responses can be observed in sandy soil that are prone to pore-pressure generation during earthquake. The strict performance considerations point to the importance of using robust computational tools for reliable prediction of permanent tilt of OWTs due to earthquake loading.

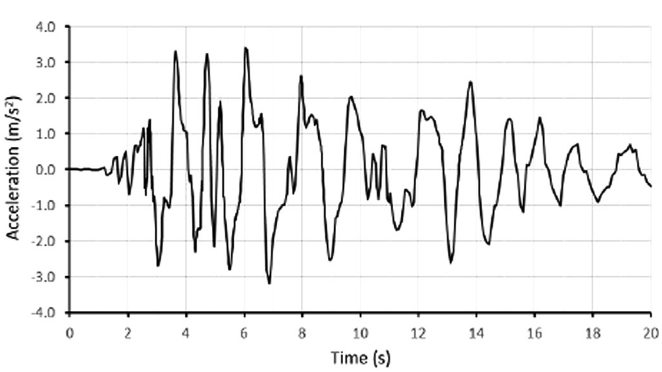

Fig. 5. Acceleration time history of Takatori, Kobe, earthquake scaled to PGA = 0.35*g*.

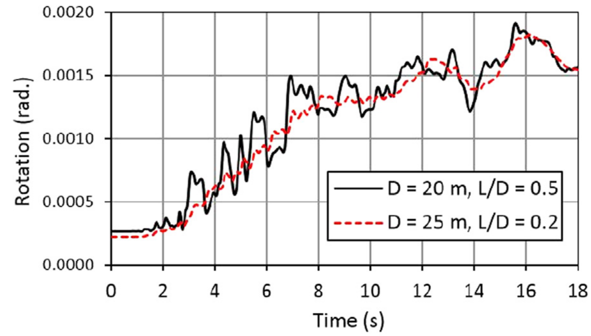

Fig. 6. Time histories of foundation rotation and accumulation of tower tilt for two design alternatives (modified after [48]).

3.1.1. Simple models for nonlinear dynamic analyses

Advanced FE models, such as the one presented in the preceding section, are gaining more popularity; however, the use of nonlinear spring models or macro-elements for representing foundation response at the base of the structure/tower (Fig. 7) is indeed more attractive in OWTs. This is because the existing computational tools for OWTs are based on hydro-aero-elasto-dynamic codes that cannot easily be adapted to conventional FE tools. One of the most popular hydro-aero-elasto codes is FAST [29] which is an open-source code maintained and distributed by National Renewable Energy Laboratory (NREL). The only built-in foundation model in the latest version of this code is a beam at the tower base.

It appears that Krathe and Kaynia [51] are the first to implement a nonlinear hysteretic foundation spring in a hydro-aero-elastic code. They used a multi-surface kinematic hardening model based on Iwan's model [52] and implemented it in FAST. Then they carried out several simulations using NREL 5 MW reference turbine [29] with harmonic wind and wave loads to demonstrate the performance of the turbine and especially the foundation for nonlinear soil behavior.

Fig. 8 displays the results of some of these simulations for three idealized wind environments with a period of 30 s and velocity amplitudes ranging from 5 m/s to 10 m/s. The large hysteresis loops of

Fig. 7. Schematics of foundation springs or macro-elements in OWTs.

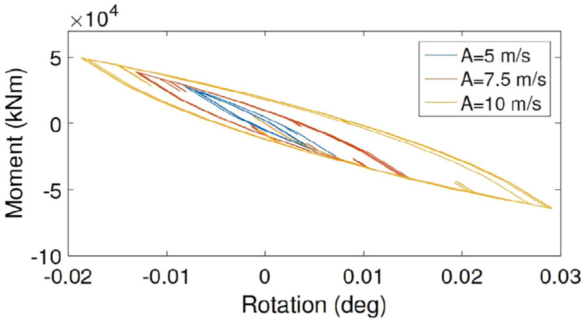

Fig. 8. Nonlinear foundation response for NREL 5 MW reference turbine for harmonic wind load with amplitudes from 5 m/s to 10 m/s.

rotation vs. moment are due to the harmonic wind loads whereas the smaller inner loops are due to the vibration of the tower with a natural period of about 3 s. As it is evident in the figure, the hysteretic loops of the foundation response get broader with increasing wind speed, indicating larger soil nonlinearity and damping. This model can readily be extended for earthquake analyses by treating the earthquake excitation as inertia loads on the masses of the structure.

Spring models as described above have been in use in geotechnical earthquake engineering for some time. These models have been utilized both as macro-elements representing, for example, the foundation response of large gravity-based offshore platforms [53] or as distributed foundation springs used in pile-soil interaction analyses in terms of nonlinear p-y curves [44].

A major challenge in this type of models is realistic representation of damping which is an important parameter in dynamic analysis of wind turbines. Although correct representation of damping is a challenge in almost all constitutive soil models in FE codes, it sounds intuitively easier to account for it in foundation springs. Most available models represent the nonlinear cyclic response, often referred to as hysteretic response, with the help of Masing's rule [54] which is a kinematic hardening model easily represented by Iwan's model [52]. Fig. 9 shows an example of nonlinear hysteretic response at different levels of loading in a rotational foundation spring of a mono-pile following this rule. The amount of hysteretic damping, which is directly related to the area circumscribed in a closed response loop, could exceed 60% at large deformations. For the largest loop in Fig. 9, the damping ratio is about 33%.

Different solutions have been proposed to limit the foundation hysteretic damping. One of these solutions, which has been implemented and verified against actual measurements of Troll Platform [55,56], consists of the following steps: i) computation of the nonlinear load-response of the foundation, for example moment-rotation relationship, the so-called backbone curve, using FE methods, ii) computation of the foundation damping, ζ_{eq}, as function of the foundation rotation using the conventional formula for hysteretic damping (Eq. (1)) for the whole FE domain by using the relevant strain-dependent damping curves, e.g. [57], and iii) modifying the backbone curve at larger rotations to correctly reproduce the damping computed in step ii (see [55] for details and example of application). In Eq. (1), E_S and E_D are respectively the elastic energy and energy loss for each element in the finite element model. Energy loss is computed from elastic energy using the same formula (Eq. (1)) for each element and damping-vs-strain curves.

$$\zeta_{eq} = \frac{1}{4\pi}\frac{\sum E_D}{\sum E_S} \tag{1}$$

Another solution for reducing hysteretic damping is deviate from the Masing's principle by defining different unloading rules that would result in slimmer hysteresis loops. A simple way to achieve this is through modification of Iwan's mechanical model [52]. This model reproduces a backbone curve by a series of parallel elastic-perfectly plastic springs with a hysteretic response similar to Masing's rule shown in Fig. 10(a). By replacing some of the elastic-plastic springs with nonlinear elastic springs with similar stiffness and yield moment, one can reduce the damping to a desired level. This is shown schematically in Fig. 10(b) where one of the elastic-plastic springs has been replaced by the corresponding nonlinear elastic spring. The resulting hysteretic curve has a damping ration of only 19%. A drawback of this solution is that, due to the inclusion of the nonlinear elastic springs, the stiffness at unloading points can be less than the stiffness at loading points. A more elaborate method of reducing damping has recently been presented in [43].

3.2. Response of OWT in vertical direction

As stated earlier, the low damping of wind turbines combined with the fairly large natural frequencies of wind turbines in the vertical direction make OWT vulnerable to vertical earthquake motions [39]. The vulnerability could be related to large accelerations in the rotor and nacelle, overstress in the tower structure and local buckling. Kjørlaug and Kaynia [39] constructed an FE model of the NREL 5 MW reference turbine in SAP2000 by including a soil volume with horizontal dimensions 30 m by 80 m and depth 40 m. The blades, nacelle and tower were modelled by beam, solid and shell elements, respectively (Fig. 11). For the case of the tower on mono-pile, the length of the pile was taken 30 m and the pile was modelled by shell elements. The structural model was verified by comparing its natural frequencies on fixed base compared to those reported by NREL [39]. The natural frequencies in the fore-aft and side-side directions are 0.32 and 0.31 Hz, respectively. The natural frequencies in the vertical direction have not been reported in NREL; therefore, no direct verification has been possible. However, the good match in the horizontal direction implies that the structural model is satisfactory in terms of dimensions, elasticity and masses. Therefore, the vertical dynamic characteristics of the model, including the natural frequency discussed in the next section, are believed to be reliable.

For the soil medium, a generic clay soil profile with Poisson's Ratio 0.45 was used with the small-strain shear modulus increasing linearly with depth from almost zero on the surface to about 300 MPa at 40 m depth corresponding to Vs = 400 m/s. The soil was assumed linear elastic under vertical shaking. The base of the model was excited in the vertical direction by the vertical component of 1985 Nahanni, Canada, earthquake time history. The acceleration time history was frequency-domain scaled to match the response spectrum in Eurocode 8 for bedrock outcrop with PGA = 0.12g (Fig. 12). The vertical base motion is propagated by pressure waves that have a wave velocity around 1500 m/s in typical saturated clay soils. However, to avoid numerical problems in shallow depths, which will have Poisson's Ratio very close

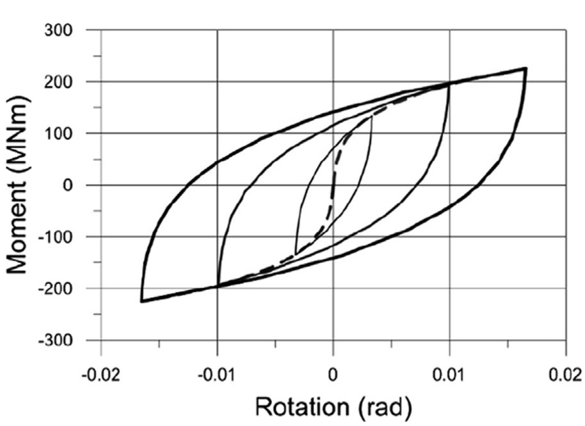

Fig. 9. Nonlinear backbone curve of foundation response (dashed curve) and hysteresis nonlinear loops following Masing's rule [54].

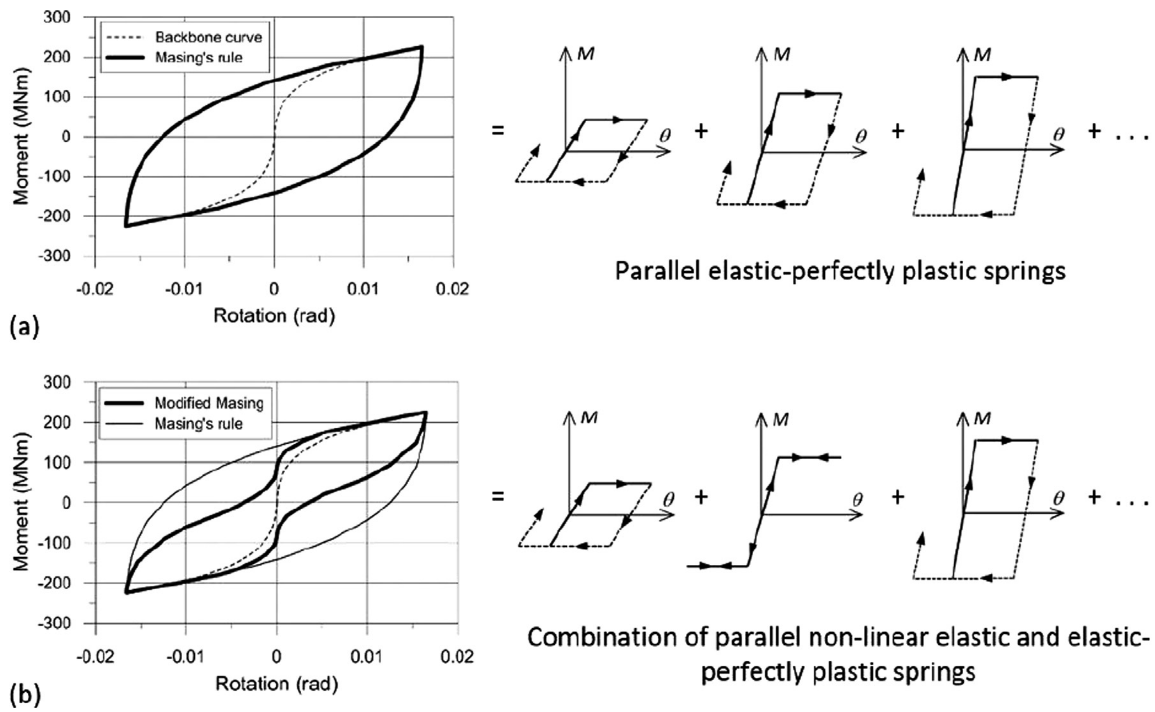

Fig. 10. Iwan model with elastic-plastic springs consistent with Masing's rule (a) and Modified Iwan model with combined elastic-plastic and nonlinear elastic springs (b).

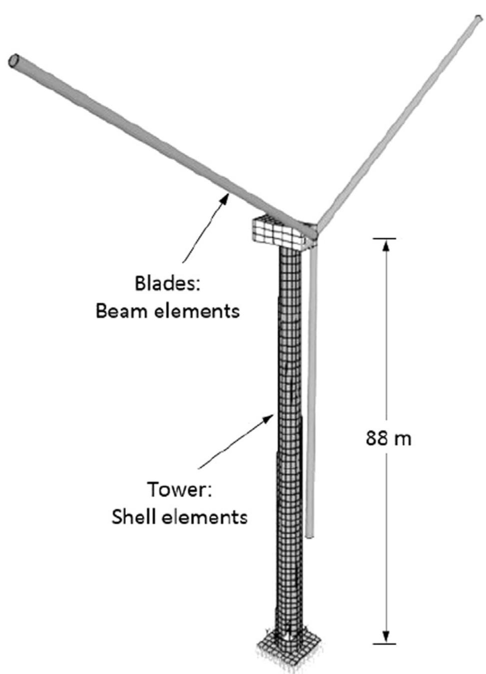

Fig. 11. Finite element model of NREL 5 MW turbine on fixed base used for verification of dynamic structural model (modified after [38]).

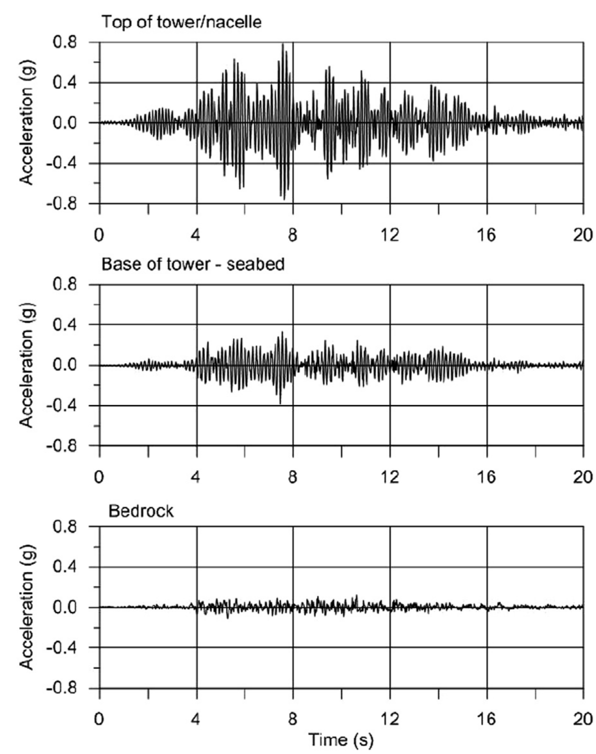

Fig. 12. Vertical earthquake response of soil-turbine model upward: earthquake time history at base (bedrock), response at base of tower (seabed), acceleration response at top of tower (modified after [39]).

to 0.5, a constant ratio 0.45 was selected. This would result in pressure wave velocities considerably less than 1500 m/s at shallow depths; however, it will not affect the trend of the results nor the main conclusions of this study, especially as regards the amplification of the motions along the tower.

A major challenge in dynamic modelling of soil media is handling of lateral boundaries Most FE models use viscous dashpots that only partially absorb the propagating waves. While radiation damping is generally small for vibration in the horizontal direction, it could indeed be large for the vertical response of an OWT due to its high natural frequency. In order to highlight the role of radiation damping on the vertical earthquake response, two different models were used.

In the first model, the sides of the soil body were fixed in the horizontal direction (i.e. only free to move vertically). The FE model would thus be able to perfectly represent the earthquake wave propagation in the soil under vertical excitation (i.e. free-field condition) but unable to absorb the waves due to the soil-structure interaction, known as inertial interaction. Therefore, this model is not able to capture the radiation damping. A damping ratio of 2% was used in both the structure and the soil for all frequencies. This was possible by carrying out the dynamic analysis using the mode superposition method. The second model was based on the conventional three-step method in which the OWT model was mounted on soil spring and dashpot, representing the stiffness and damping of the mono-pile. The spring and dashpot were computed by rigorous frequency-domain solutions that include radiation damping. This model was then excited at the base of the spring and dashpot by the vertical motion computed at the top of the pile that was shown to be practically identical to the vertical motion on the ground surface in the free field.

3.2.1. Analysis ignoring radiation damping

Fig. 12 displays the results of the first analysis, namely, the integrated analysis of the soil-wind turbine model with fixed lateral boundaries and no radiation damping. The plots in this figure show the following results from bottom to top: vertical earthquake time history at the base of the model (bedrock), earthquake response at the base of the tower (ground surface), and vertical acceleration at the top of the tower/nacelle. It should be noted that, although there is a significant amplification in the acceleration, most of the amplification occurs in the soil. Indeed the amplification from the base of the tower to its top is about two. It is interesting to note that a mild vertical acceleration of 0.12*g* on bedrock is amplified to about 0.8*g* which is extremely high for the performance of the nacelle and rotor and their connections to the tower. The earthquake-induced stresses, however, are not larger than 10% of the yield stress; therefore, they are not expected to represent a major challenge in the structural design of the tower or the pile for the level of shaking considered in this study. However, they need to be addressed carefully, especially as the level of shaking increases.

3.2.2. Analysis including radiation damping

The preceding analysis did not take advantage of the radiation damping. Considering the natural frequency in the vertical direction, about 7.5 Hz, being larger than the cut-off frequency of the soil layer (about 3.5 Hz), one would expect a large radiation damping in the vertical vibration of the turbine.

Fig. 13 plots the variations of the real and imaginary parts of the pile's vertical impedance with frequency computed by PILES [58]. The figure also displays the equivalent damping ratio of the soil-pile system as a function of frequency. For a frequency around the SSI natural frequency the damping ratio is about 30%.

In this analysis, the impedance of the pile at 7.5 Hz was converted to equivalent spring and dashpot constants that were set under the FE model of the tower (Fig. 11). Then the model was excited by the effective input motion computed at the pile head, known as kinematic interaction motion, which as stated above, was almost identical to the free-field vertical response. This analysis also showed an amplification of the earthquake motions in the tower, however, it was less than the one computed from the first analysis. The acceleration at the top of the tower was about 0.6*g* (compared to 0.8*g* from the first analysis).

Fig. 14 displays the results of analyses at the base and top of tower in the same format and scale as in Fig. 12. The noteworthy reduction of the accelerations highlights the importance of considering the radiation damping in vertical earthquake analyses of OWTs.

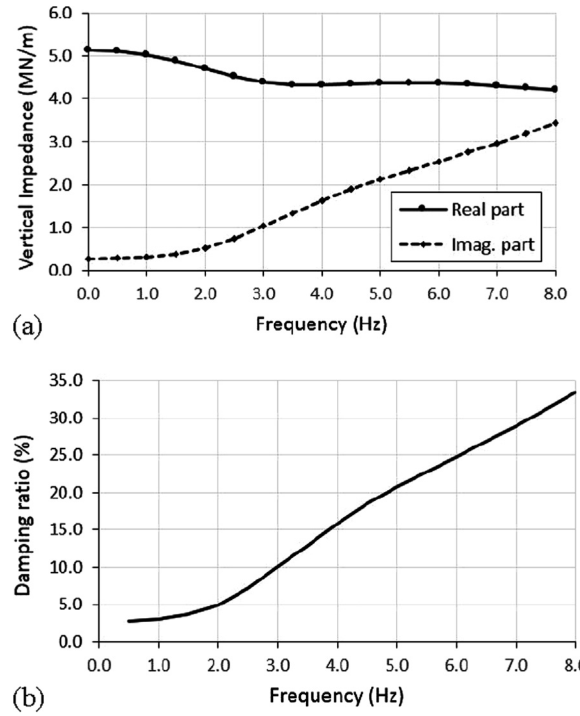

Fig. 13. Variation with frequency of vertical impedance of pile (a) and corresponding damping ratio (b) computed by PILES.

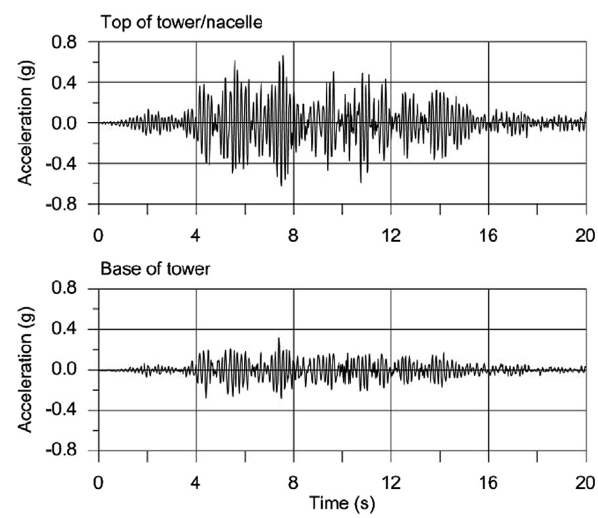

Fig. 14. Vertical earthquake response of soil-turbine model with radiation damping; plots from bottom to top: earthquake time history at base (same as Fig. 12), response at base of tower, acceleration at 1/3 height of tower, acceleration response at top of tower (modified after [39]).

4. Summary and conclusions

This paper has presented a brief history of the development and growth of wind turbines and OWTs. It has also presented the state of practice in seismic design of offshore wind turbines and existing design cods. In addition, the paper has addressed, among others, the use of foundation macro-elements for earthquake response of wind turbines under lateral earthquake excitation and has highlighted the vulnerability of wind turbines to vertical earthquake excitations due to their high vertical natural frequencies. It has also been shown that for such loading conditions, use of radiation damping is a key to a more economical design. Moreover, the paper has demonstrated how soil nonlinearity and pore-pressure generation could lead to settlement and permanent tilting of offshore wind turbines on mono-bucket foundations. Using these cases, the paper has clearly highlighted the importance of performance-based analyses in seismic design of OWTs.

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