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Comparative Analysis of Transient Dynamics of Large-Scale Offshore Wind Turbines with Different Foundation Structure under Seismic

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Abstract

In this paper, a structural dynamic response comparison between jacket foundation large-scale offshore wind turbines (OWTs) and monopile ones under wind and seismic loads is demonstrated. The interaction between flexible soil and pile foundation is described by Winkler soil-structure interaction (SSI) model. The National Renewable Energy Laboratory (NREL) 5 MW large-scale OWT is studied via the finite element model. The structural transient dynamic response of these two structures under normal operating conditions at rated wind speed and earthquake is calculated. The results show that under the action of seismic and turbulent wind, the jacket has better wind and seismic resistance, and the displacement of the top of the tower is small, which can effectively protect the blades and the nacelle. Compared with monopile, the range of the Mises equivalent stress amplitude of the jacket wind turbine was reduced and the average value was decreased at the time of the seismic. The study also found that the existing jacket design will have a local strain energy surge.

Keywords: OWTs, dynamic response, SSI, jacket foundation, monopile foundation, seismic

1. Introduction

The need for renewable resources is becoming more evident as the earth's limited reserves of remaining fossil fuels have essentially halted development and utilization. Among renewable energy sources such as wind energy, solar energy, tidal energy and geothermal energy, wind energy is considered to be the most cost-effective alternative energy and one of the cheapest new power source, showing its great potential to meet demand [1]. The 2021 Global Wind Energy Council (GWEC) released an annual wind report that 2020 was the best year in history for the global wind industry showing

year-over-year growth of 53%. Installing more than 93 GW wind power in a challenging year with disruption to both the global supply chain and project construction has demonstrated the incredible resilience of the wind industry. According to the GWEC, more than 6 GW of new offshore wind power capacity was installed worldwide in 2020, with China adding more than 3 GW of new offshore wind power capacity, accounting for half of the world's new installed capacity [2]. These estimations reasonably take into account the new improved technologies in wind turbine systems and the offshore wind energy available in considerable quantity. Those sites primarily located at near coastal or offshore areas and experiencing high wind throughout the year, such as the sea off southeast China and the western coast of the United States. However, these areas are with close proximity to the Pacific seismic belts, making them vulnerable to severe and frequent earthquakes [3]. Or rather, OWTs working under such condition generally have to experience dual external loadings primarily stimulated by turbulent wind and seismic excitation. In addition, so as to shrink the cost of energy, the size of OWTs has increased to 9.5 MW recently [4]. Nevertheless, higher wind turbine capacity is accompanied by higher wind turbine towers, which are destined to withstand higher wind pressures and are more vulnerable to seismic loads.

In the past few decades, a large number of studies have been carried out to analyze the dynamic behavior of OWTs under earthquake excitation, in order to improve the stability of wind turbines [5–9], but there is still the issue of oversimplification in the geometry of the model. It is noted that in most of the aforementioned literatures, the rotor and nacelle were either completely ignored or simplified as a lumped mass. In Ref. [10–13], Asareh and Prowell selected FAST as a design basis, developed a seismic module to examine the coupled effect of wind and seismic loads and employed an improved FAST (also known as NREL Seismic) to investigate the relationship between seismic intensity and structural response. Additionally, in Ref. [14–17], the natural frequency and mode shape of the OWT support structure are calculated using frequency domain, and then the seismic load is calculated according to the seismic response spectrum. However, unsteady aerodynamic loads were often ignored or treated as a rotor thrust, resulting in relatively inaccurate prediction of combined loads acting on blades.

In Ref. [18], the coupled behaviors of seismic and wind loads based on a 2 MW wind turbine was conducted by Witcher. In Ref. [19], Jin et al. also employed the NREL Seismic tool to predict the dynamic responses under multiple hazards associated with earthquake excitation and turbulent wind. In general, the aerodynamic load of large-scale wind turbines increases exponentially with the rotor diameter. Hence, the continuous aerodynamic effects on a larger turbine are determined to be unneglectable in the comparative study of operational and parked states. In Ref. [20], the dynamic response of a 5 MW OWT under seismic loading was further discussed. The finite element model of the SSI was considered for the multi-degree of freedom of fixed foundation. Response spectrum analysis method and transient dynamics analysis method were adopted to verify that the SSI cannot be ignored. In addition, some scholars assumed that there is rigid or linear elastic contact between seabed soil and pile foundation [21–25]. However, due to the severe liquefaction of the seabed soil, the soil porosity and water content change continuously with the pile depth. Furthermore, mechanical dynamic responses on the surface exhibit nonlinear changes in both the horizontal and vertical directions of soil, which can raise the damping of the OWT support structure and lessen the natural frequency. In this case, the relative displacement between soil and structure intensifies the liquefaction of soil during earthquake,

and the soil reaction shows highly nonlinear characteristics related to the structural deformation difference [26–33]. To sum up, an accurate nonlinear SSI model is needed to analyze the dynamic characteristic and response of the OWT support structures under mixed loads.

To achieve the same end, Yang et al. [34] proposed a numerical analysis framework coupled with FAST to obtain more precise responses. However, this method is only applicable to OWTs with monopile support structure, and the results are relatively macroscopic. Yet, it is challenging to examine the local response characteristics of OWTs with different support structures under joint loads.

To tackle these challenges, it is necessary to propose a proper and practical control mechanism of large-scale OWTs under persistent earthquake-and-wind-induced excitations, so as to decrease the volume, prolong the working life, and improve the reliability of the support structures. In the paper, instead of 2 MW OWT, the NREL 5 MW OWT is referred to as the research object, utilizing Winkler soil-structure interaction (SSI) model to describe the interaction between flexible soil and pile foundation. Finite element analysis software ANSYS is used to simulate the dynamic response of OWT foundation under different soils and different seismic intensities, and the influence of seismic load on tower-top displacement and tower-base bending moment is studied. Moreover, the local response characteristics of OWTs with different support structures under seismic load are strictly analyzed. Several principles of nonlinear dynamic response of OWTs structures experiencing sorts of earthquakes are identified. These conclusions will be beneficial in the phase of guaranteeing the optimum performance of OWTs foundations in any operating conditions and in any seismic area.

The remainder of the paper is presented as follows. In Section I, numerical modeling and physical properties are explained. Section II describes wind and seismic loads. In Section III, the finite element method is outlined. Reliability is validated in Section IV. The comparison between the numerical results of two foundations are presented in Section V. Finally, Section VI summarizes the work done and concludes the paper.

2. Numerical simulations

This paper takes NREL 5 MW OWT as the research object. The support structures of offshore wind power are mainly monopile and jacket, which are usually installed in shallow sea areas. The jacket is mainly composed of grid truss, low steel consumption, convenient transportation and assembly, and good anti-wind and wave performance, suitable for deep water [35]. Since this paper mainly studies the difference between monopile and jacket under the combined action of earthquake and turbulent wind, and wave load has little influence on the jacket, the influence of wave on them is ignored.

2.1 Structural model

Design parameters were detailed in a report published by the National Wind Technology Center (NWTC), proposed by Jonkman et al. [36]. Besides, the main parameters of NREL 5 MW OWT are shown in **Table 1**.

The wall thickness of the tower decreases linearly with height; the outer diameter of the tower base is 6 m and the wall thickness is 27 mm; the diameter of the tower top connecting the engine room connecting flange is 3.87 m and the wall thickness is 19 mm. A kind of A709 circular section high-strength steel with a density of

Parameters	Numerical value
Rated power/MW	5
Tower height/m	87.6
Diameter of impeller/m	126
Rated wind speed/m.s ⁻¹	11.4
Tower quality/kg	347, 460
Cabin quality/kg	240, 000
Wheel and blade quality/kg	111, 000
1st natural frequency fore-aft of the tower/Hz	0.32
1st natural frequency lateral of the tower/Hz	0.31
2nd natural frequency fore-aft of the tower/Hz	2.90
2nd natural frequency lateral of the tower/Hz	2.93

Table 1.
Main parameters of wind turbine.

7850 kg/m³, Young’s modulus of 210 GPa, Poisson’s ratio of 0.3 and material yield limit of 380 MPa is chosen as the material of tower. So as to take into account the influence of structural paint, flanges, bolts and welding point masses, the calculated density has been modified to 8500 kg/m³ [37]. The structure model and parameters are shown in **Figure 1**.

Although more accurate structural dynamics results can be obtained by using FEM, the FEM method requires high computational resources, so almost all studies ignore or simplify the wind wheel and engine room as the bulk mass. Using a similar simplification, Lavassas et al. [38] studied the structural characteristics of 1.0 MW wind turbine tower under earthquake action, in which the seismic load referred to the relevant Greek seismic codes and guidelines. Hacıfendioğlu [39] used a similar simplified model to study the seismic dynamic characteristics of a 3.0 MW offshore wind turbine under shutdown condition, considering SSI effect. Ma et al. [20] calculated the structural response and load distribution characteristics of the concrete tower of 5.0 MW wind turbine under six different earthquake actions by ABAQUS using a simplified model without considering wind turbines at all.

2.2 SSI modeling

In this paper, the current Wenkel SSI model of the American Petroleum Institute (API) is adopted to consider the nonlinearity of the lateral and longitudinal soil stiffness [40]. Meanwhile, the SSI model is represented by a nonlinear spring, as shown in **Figure 2**. The calculated monopile and jacket *p-y* curves are illustrated in **Figure 3**.

The burial depth of two structures is 25 m and the burial part is divided into 5 sections, starting from 1.25 m below the seabed, with an interval of 5 m at each calculation step. The nonlinear stiffness of the soil *K_n* is based on piecewise calculation, the outside diameter of the jacket pile and the monopile is 2 m and 6 m respectively.

The ultimate bearing capacity of sand varies in depths. The ultimate bearing capacity of shallow and deep sand below the seabed is denoted as Eq. (1) and Eq. (2).

Figure 1. Structure model and parameters. (a) Wind turbine support structure model diagram, (b) jacket model parameters and (c) jacket finite element model and grid.

$$p_s = (C_1H + C_2D)\gamma H \tag{1}$$

$$p_d = C_3D\gamma H \tag{2}$$

where p_s and p_d are the ultimate bearing capacity of shallow and deep sand respectively; γ is the effective gravity of sand; H is the depth of sand below the seabed; Coefficient C_1 , C_2 and C_3 is determined by the internal friction angle φ , displayed in **Table 2**; D is the outside diameter of the pile foundation.

Figure 2.
Schematic diagram of soil structure interaction.

Figure 3.
p-y curve. (a) Single pile structure and (b) jacket structure.

When the pile foundation is laterally displaced, the sand reacts to the pile foundation due to the deformation of the pile. The nonlinear relation between the force and the lateral displacement of the sand is denoted as follows:

$$\begin{cases} p = Ap_s \tanh \left[\frac{kH}{Ap_s} y \right] \\ p = Ap_d \tanh \left[\frac{kH}{Ap_d} y \right] \end{cases} \quad (3)$$

Soil parameters/unit	Numerical value
Internal friction angle $\varphi/^\circ$	36
C_1 coefficient/—	3.2
C_2 coefficient/—	3.6
C_3 coefficient/—	60
Effective soil severity $\gamma/(\text{kN} \cdot \text{m}^{-3})$	20
Ground reaction coefficient $k/(\text{kN} \cdot \text{m}^{-3})$	24,440

Table 2.
 Soil parameters.

where A is the empirical adjustment coefficient with regard to different loads; A is 0.9 with cyclic load and $A = (3.0 - 0.8H/D)$. When with static load, k is the ground reaction coefficient; p is the sand reaction force and y is the lateral displacement.

3. Loading calculation

3.1 Turbulent wind load

According to the IEC Kaimal turbulence wind spectrum model defined by the classic IEC61400–1 [41], the three component calculations of the wind ($K = u \cdot v \cdot w$) are determined by:

$$S(f) = \frac{4\sigma^2L/\bar{u}_{\text{hub}}}{(1 + 6fL/\bar{u}_{\text{hub}})^{5/3}} \tag{4}$$

where f is the frequency; $\bar{u}_{\text{hub}}$ is the mean wind speed at the hub height; σ is the standard deviation of the wind speed and L is the integral scale parameter of each velocity component. Wind field calculations are arranged at 11 grid points in the y and z directions.

The turbulent component $V(t)$ is calculated by applying an Inverse Fast Fourier Transfer (IFFT) to the Kaimal turbulent spectrum described by Power Law wind profile and Logarithmic wind profile as follows:

$$\begin{cases} V(z) = V(z_{\text{hub}})(z/z_{\text{hub}})^{0.3} \\ V(y) = V(y_{\text{hub}})\left(\ln \frac{y}{z_0} - \psi\right) / \left(\ln \frac{z_{\text{hub}}}{z_0} - \psi\right) \end{cases} \tag{5}$$

where $V(z)$ and $V(y)$ represent the wind velocity in the lateral and vertical directions; $V(z_{\text{hub}})$ is the mean velocity at the hub height z_{hub} ; The value of $V(z_{\text{hub}})$ is selected as 12.0 m/s equal to the rated wind speed; z_0 is land surface roughness with a value of 0.021; ψ is vertical stability of dimensionless function.

TurbSim [42] developed by NREL is adopted to simulate the full-field turbulent wind. The generated wind field is presented in **Figure 4**. The time-varying wind speed has a peak value of over 20 m/s at the hub and an average magnitude value of 11.4 m/s as expected. The variation of wind speed is irregular in time domain and non-uniform

Figure 4.
Turbulent wind at hub height of the OWTs.

Figure 5.
Time history accelerations of ground motions.

in spatial distribution, which indicates the turbulent characteristics of the generated wind field.

3.2 Seismic load

The earthquake record was selected from the Pacific Earthquake Engineering Research Center (PEER) NGA database [43]. The earthquake event occurred in Morgan Hill, USA, with a magnitude of 6.19 and a peak acceleration of 2.44 m/s^{-2} . The time history of seismic acceleration is shown in **Figure 5**.

4. Finite element method

Transient dynamics analysis is used to analyze the dynamic response of a load structure subjected to any time variation. The Newmark time integral method is adopted to solve the whole structural system matrix for all kinds of large nonlinear elastic–plastic deformation structures [44, 45].

The basic equation of structural transient dynamics analysis is:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{F(t)\} \quad (6)$$

Figure 6.
Resolving flow chart of transient dynamics direct method.

where $[M]$ is mass matrix; $[C]$ is damping matrix; $[K]$ is stiffness matrix; $\{\ddot{u}\}$ is acceleration vector; $\{\dot{u}\}$ is velocity vector; $\{u\}$ is displacement vector; $\{F(t)\}$ is variable load vector.

The accuracy of the calculation result is improved by using the complete Newmark time integration method. The specific steps are as shown in **Figure 6**.

5. Reliability compliance

Like any other numerical models, finite element models should be validated before the formal application. Only in this way can we ensure the accuracy of material models, element formulations and mathematical calculations. In this study, the calculation results are compared with those of NREL wind turbine aerodyn-servodyn-hydrodyn-elasdyn open source software FAST. The dynamic response of tower-top displacement is shown in **Figure 7**.

Figure 7.
Time-domain response of tower top displacement under seismic.

The result from finite element calculation is basically consistent with the result of FAST in the turbulent wind load before 20 s. Both of the curves manifest severe fluctuations, and the trend is highly consistent, with only minor differences in amplitude. The average tower-top displacement is 1.295 m via FAST, but it is 1.234 m using finite element merely with an error value of 4.7%. Therefore, the finite element model is validated.

6. Results and discussion

6.1 Modal analysis

In the modal analysis module, Block Lanczos solver performance output is used to extract mode shapes and natural frequencies. The Block Lanczos method uses the sparse matrix solver, overriding any specified solver via the Eqsolv command, and can be used for large symmetric eigenvalue problems. The 1st and 2nd modal natural vibration frequencies of the two structures involved in SSI are shown in **Table 3**. The 1st and 2nd fore-aft tower vibration mode shapes are illustrated in **Figure 8**.

The vibration characteristics of the OWTs are greatly changed due to the different structures. The 1st natural frequency of the jacket structure is higher, and the 2nd natural frequency is lower, compared with those of the monopile structure. Also, the largest displacement location of the 1st mode at the nacelle is different from that of the 2nd mode.

6.2 Structural dynamic response analysis

To investigate the effect of wind-seismic interaction on the dynamic responses of the OWTs, a nonlinear time-history analysis is more appropriate compared to the

Modal	Single pile structure	Jacket structure	Amplification
Fore-aft 1st	0.157	0.180	+14.6%
Lateral 1st	0.157	0.181	+15.3%
Fore-aft 2nd	0.979	0.913	−6.7%
Lateral 2nd	0.989	0.922	−6.8%

Table 3.
Wind turbine natural frequency.

Figure 8.
Fore-aft mode shape of tower.

other analysis schemes. In this study, the dynamic responses of two structures are analyzed under two conditions, with and without seismic load.

Numerical simulations in four diverse cases are carried out:

1. Jacket OWTs under wind and seismic load;
2. Monopile OWTs under wind and seismic load;
3. Jacket OWTs under wind load merely;
4. Monopile OWTs under wind load merely.

Under such circumstance, the simulation duration is 100 s and the time step is 0.002 s. The turbulent wind speed is 11.4 m/s and the earthquake is assumed to occur at 20th s with a magnitude of 6.19. Tower-top displacements in x directions (fore-aft) and y (side-to-side) are presented in **Figure 9**.

When comparing the structure behaviors in different directions, the tower-top displacement in x direction is obviously larger than that in y direction, this is because the x direction suffers from wind load. The displacement curve varies significantly in earthquake, and it is obvious larger in y direction than that in x direction, due to the damping of wind load.

When comparing the different structure behaviors, the dynamic response of the jacket during initial period of the earthquake was smaller than that of monopile, with a 32.3% decrease in peak.

After the earthquake, energy is quickly dissipated in the turbulent wind, the tower-top displacement curve of jacket drops sharply in x direction, with a 39.2% decrease in peak compared with monopile. Most of the energy can be dissipated through the vibration of the jacket tower, which shows that the jacket can be used in aseismic design.

Figure 9. Tower top displacement response in time domain. (a) Earthquake and turbulent wind combined action conditions and (b) turbulent wind action.

However, the stress of the supporting structure of OWTs will increase sharply when the earthquake occurs, and it is necessary to conduct further research on the response characteristics of shear stress. The average equivalent stress curve of the structures in time-domain is shown in **Figure 10**.

It is shown that the average equivalent stress manifests low-frequency fluctuations as wind speed vary, with a range of 0.92 ~ 64.27 MPa for monopile and 0.66 ~ 53.31 MPa for jacket. The average value of equivalent stress of jacket decreased

Figure 10.
Average equivalent stress curve in time domain. (a) Earthquake and turbulent wind combined action conditions and (b) turbulent wind action.

by 21.6% compared with monopile. It is indicated that the jacket structure has a higher and more reliable ultimate bearing capacity of support structures subjected to accumulated fatigue damage.

When seismic load is added, the shear curve shows high frequency fluctuations, accompanied by severe vibration of the tower. The equivalent stress amplitude varies from 1.91 to 82.05 MPa for monopile, and from 5.13 to 59.03 MPa for jacket. The average value of equivalent stress of jacket decreased by 48.7% compared with monopile. Therefore, the jacket structure can reduce the collapse possibility in that increasing rapid stress presumably exceed the tower yield limit when the earthquake hits the coast.

The principle of virtual work states that a virtual (very small) change of the internal strain energy must be offset by an identical change in external work due to the applied loads, and then the virtual strain energy value related to the nodal displacements can be got. The maximum strain energy position of the structure is shown in **Figures 11** and **12** is the cloud picture of strain energy peak moment.

The strain energy fluctuation has been proven to be associated with the change of wind speed, with an average of 228.7 J for monopile and 365.5 J for jacket. Both the two structures receive the same energy from the blades of OWT. However, due to the large displacement response of the monopile, part of the energy is dissipated. The stain energy of stable jacket is 59% higher than that of monopile.

When the earthquake occurred, the local strain energy surged in both structures. The maximum strain energy appeared at 20 s ~ 30 s, the average value of jacket is 1510.8 J, which is 313% higher than that before the earthquake, and the monopile is 708.9 J, which is 94.0% higher than that before the earthquake. Compared with monopile, strain energy of the jacket is larger and more concentrated, which is prone to suffer local deformation.

Figure 11. Maximum strain energy position curve in time domain. (a) Earthquake and turbulent wind combined action conditions and (b) turbulent wind action.

Figure 12. Strain energy of OWT foundation structure.

7. Conclusion

In this study, the NREL 5 MW OWT was chosen as the study object and an earthquake with a magnitude of 6.19 is selected as input ground motions. Transient dynamics of large-scale OWTs under combined loads with different foundation structures is analyzed. The structural dynamic response of jacket foundation and that of monopile foundation was carefully compared. In view of the calculations and discussions described, the four key conclusions are as follows:

- a. Compared with monopile-type OWT, jacket-type OWT has higher 1st natural frequency and lower 2nd natural frequency. The 2nd modal response of jacket-type OWT is smaller.
- b. The tower-top displacement of the jacket-type OWT is smaller under the action of turbulent wind, which can guarantee the stability of a multitude of important components at the top of the tower. Compared with the monopile, the equivalent stress of jacket has a low average value and a small fluctuation range, which improves the structural safety.
- c. When the earthquake occurs, the monopile response is more violent and lasts for a longer time, while the jacket response is moderate, and can be fully restored to the stationary state more quickly. Thus, the jacket performs better in anti-seismic behaviors.
- d. The earthquake can cause the jacket-type OWT at the connection point between the jacket structure and the tower, while the monopile-type OWT deformation occurs in the deep buried location. Therefore, the maintenance cost of jacket is lower, which is beneficial to reduce the cost of electricity.

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