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Preliminary evaluation of monopile foundation dimensions for an offshore wind turbine by analyzing hydrodynamic load in the frequency domain

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A B S T R A C T

Although design of offshore wind turbines has many similarities to that of onshore turbines, a lot of considerations should be made for the additional substructure imposed on hydrodynamic loads. The additional substructure prolongs the total tower length, increasing the tower bending moment and lowering the natural bending frequencies of the tower. Accordingly, system dynamic analyses associated with hydrodynamic load should be performed in the frequency domain in order to avoid bending modes of tower from the operation frequency ranges. In this paper, a method to generate hydrodynamic load for a finite element analysis is introduced, considering the characteristics of sea conditions for a candidate site of demonstration offshore wind farm in the west sea of Korea. In addition, a wind energy conversion system with a monopile foundation is fully modeled using the finite element method to simulate the various conditions based on IEC standard. Based on the FEM analyses of tower bending modes, optimal dimensions of the monopile for the candidate site are proposed.

1. Introduction

Offshore wind farms provide higher energy density, have lower spatial limitations, and are less likely to generate complaints from the public than onshore wind farms. Despite such advantages, because the installation of offshore wind turbines is difficult and construction costs are far higher than those of onshore wind turbines [1], a stringent feasibility study must be conducted in advance of construction.

Various researches regarding foundation design have been carried out. R. S. Nehal et al. [2] designed a monopile foundation for 3.6 and 6.0 MW wind turbines to investigate the possibility of placing wind turbines in the North Sea, some kilometers from the Dutch west coast. B. W. Byrne et al. [3] evaluated foundation modeling methods from a civil engineering point of view to support and accelerate the large-scale commercial development of offshore wind farms in the UK. H. J. T. Kooijman et al. [4] described the pre-design and analyzed the stationary aerodynamic performance and natural frequency of a 6 MW wind turbine with PHATAS in the framework of the DOWEC project. While intensive research, as aforementioned, has been carried out on the foundations of wind turbines, systematic studies on the foundation of offshore wind turbine that consider the characteristics of sea conditions for candidate offshore wind farm site have not been performed to date.

In this paper, a method to generate the hydrodynamic load for a finite element analysis considering the characteristics of a candidate site is introduced. A full model of the wind turbine and foundation using the finite element method is then designed and an analysis technique involving a multi body dynamic system is introduced to simulate the various conditions based on IEC code [5]. A wind turbine for low wind speed is introduced in order to enhance the economic feasibility of the offshore wind farm. Optimal dimensions of the monopile for the candidate site are finally proposed based on an analysis of the hydrodynamic load and operation range in the frequency domain.

2. Hydrodynamic load

2.1. Sea conditions

To select the candidate site for a wind farm, not only wind resources but also diverse factors including the distance from substations, water depth, and the use of the sea must be taken into consideration [6]. Through an analysis of these factors, the Korea Electric Power Corporation Research Institute (KEPCO Research Institute) has determined that the area near the island Wi-do in the West Sea (Yellow Sea) is the optimal site for demonstration offshore wind farm.

The IEC stipulates the evaluation of hydrodynamic loads in the normal sea state and extreme sea state in a recurrence period of 50 years to estimate the integrity of wind turbines and foundations [5].

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Nomenclature

a	wave acceleration
A_i	i-th magnitude at spectral density
c_d	drag coefficient
c_m	inertia coefficient
D	diameter of foundation
f	frequency
f_i	i-th frequency at spectral density
f_p	peak frequency
F	wave force per unit length of foundation
F_{total}	total wave load
h	depth of water
H	height of foundation
H_s	significant wave height
N	shape function
S_{PM}	Pierson–Moskowitz spectrum
S_{PM_a}	spectrum density of wave acceleration
S_{PM_d}	spectrum density of wave elevation
S_{PM_u}	spectrum density of wave velocity
u	wave velocity
U	velocity of the flow resolved normal to the member
$\dot{U}$	acceleration of the flow resolved normal to the member
z	mean water depth of site
ρ	density of water
ϕ_i	random phase in the $[-\pi, \pi]$ range at spectral density

For reliability assessments of the wind turbine and foundation, significant wave height and the peak frequency of waves with a recurrence period of 50 years and waves for the normal sea state should be measured in order to describe the wave conditions of the candidate site. In this paper, hindcasted wave data estimated for the past 24 years from January 1979 to December 2002 by the KORDI (Korea Ocean Research and Development Institute) [7] has been analyzed, as there is no long-term measurement data for the candidate site. Fig. 1 denotes the calculation lattices of the hindcasted wave data for the candidate site.

Extreme wave conditions for a recurrence period of 50 years per wave direction at locations 055120 and 056120 between the islands Wi-do and Anma-do, where a test bed is to be constructed, were analyzed and are shown in Table 1. Through the height and period of the waves thus calculated retrogressively, the candidate site was observed to have the strongest wave height and period at NW for a recurrence period of 50 years. This is identical to the results of investigations of wind resources on the west coast, where NW was the main wind direction [8]. When the correlation between the wind and the waves is taken into consideration, it is possible to infer that the derived results are reliable. Because both lattice points were being considered as candidate sites for an offshore wind farm, in the present study, it was decided that the wave height and the wave period of lattice point 055120 NW, which had the greatest wave height and the longest wave period from among the two lattice points, would be selected as the extreme waves for a recurrence period of 50 years and the hydrodynamics would be reproduced. In addition, it could be observed that the average and maximum wave heights were 0.6 m and 1.8 m, respectively, and the average and maximum peak periods were 3.6 s and 6.7 s, respectively, in the normal sea state.

2.2. Non-stationary random hydrodynamic load

Morison's equation, given as Eq. (1), is commonly used to depict hydrodynamic loads [5]:

$$F = \frac{1}{2}c_d\rho D|U|U + c_m\rho\frac{\pi D^2}{4}\dot{U}, \tag{1}$$

where F represents the wave force per unit length of the foundation, ρ is the density of water, D is the diameter of the foundation, U denotes the velocity of the flow resolved normal to the member, $\dot{U}$ denotes the acceleration of the flow resolved normal to the member. c_d and c_m are the drag coefficient and the inertia coefficient. When Eq. (1) is examined, to calculate the hydrodynamic load, the velocity and acceleration of the waves that act on the structures must be calculated. To simulate the waves that act on the structures, the Pierson–Moskowitz spectrum delineated in Eq. (2) was used [5]:

$$S_{\text{PM}}(f) = 0.3125\cdot H_s^2\cdot f_p^4\cdot f^{-5}\cdot\exp\left(-1.25\left(\frac{f_p}{f}\right)^4\right), \tag{2}$$

where H_s denotes the significant wave height (m), f_P is the peak frequency$\left(=\frac{1}{T_p}\right)$ (Hz), and f is the frequency (Hz). The spectrum of velocity and acceleration can be converted by substituting Eq. (2) into Eqs. (3) and (4) [9]:

$$S_{\text{PM}_u}(f) = 2\pi f\cdot S_{\text{PM}_d}(f), \tag{3}$$

$$S_{\text{PM}_a}(f) = 2\pi f\cdot S_{\text{PM}_u}(f), \tag{4}$$

where S_{PM_d} represents the spectral density of wave elevation, S_{PM_u} the spectral density of wave velocity, and S_{PM_a} the spectral density of wave acceleration. Calculated spectrums are shown in Fig. 2.

The velocity and acceleration spectrums generated using Eqs. (3) and (4) can be transformed into non-stationary time series data by using Eqs. (5) and (6) [10]:

$$u(t) = \sum_{i=1}^{N} A_{u_i}\sin(2\pi f_i t + \phi_i), \tag{5}$$

$$a(t) = \sum_{i=1}^{N} A_{a_i}\sin(2\pi f_i t + \phi_i), \tag{6}$$

where A_i and f_i represents the i-th magnitude and frequency at the spectral density. ϕ_i is generated randomly in the $[-\pi, \pi]$ range. A_i can be calculated using Eq. (7).

$$A_i = \sqrt{S_{\text{PM}}(f_i)\cdot\Delta f}. \tag{7}$$

By substituting Eqs. (5) and (6) into Eq. (1), $F(t)$ can be calculated. Total wave load can be estimated by using Eq. (8).

$$F_{\text{total}}(t) = \int_0^z N(h)\cdot F(t)\mathrm{d}h, \tag{8}$$

where h denotes depth of water, z is the mean water depth of site and $N(z)$ denotes the shape function as given in Eq. (9):

$$N(z) = \frac{3}{2}\left(\frac{z}{H}\right)^2 - \frac{1}{2}\left(\frac{z}{H}\right)^3, \tag{9}$$

where H is the height of the foundation. The non-stationary random hydrodynamic load regenerated using the above procedure is described in Fig. 3 (Fig. 4).