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Wind, wave and earthquake responses of offshore wind turbine on monopile foundation in clay

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ABSTRACT

This paper investigates the dynamic responses of offshore wind turbine (OWT) supported on monopile foundation in clay subjected to wind, wave and earthquake actions. Based on the open-source software platform OpenSees, a three-dimensional finite model of the system is developed. The tower and monopile is modeled using beam element, the pile-soil interface behavior using nonlinear Winkler foundation approach, and the pile-water interface using hydrodynamic added mass. The wind, wave and earthquake actions are applied as loadings on the system. The effects of several parameters, such as wind velocity, induction factor, wave period, peak ground acceleration, and soil parameters on the dynamic responses of the system are studied. The results indicate that it is necessary to consider the combination of wind, wave and earthquake actions in the design of offshore wind turbine.

1. Introduction

Monopile foundation is the most widely used in offshore wind turbines (OWTs) at shallow water depth. The choice of monopile foundation results from its simplicity of installation, economical, and the proven success of driven piles in supporting offshore oil and gas infrastructures. Monopile for OWTs is a long steel member, commonly 22–40 m long and 3–6 m diameter [1]. Wind and wave loads are usually the most important environment loads for the design of offshore wind turbine structures. However, OWTs are also under threat of earthquake in areas of active seismicity, such as the Eastern coast of China [2]. Therefore, it is necessary to investigate the responses of the OWTs subjected to combined wind, wave and earthquake actions [3,4].

OWT supported by monopile foundation is a soil-pile-tower system. Several studies on pile-soil interaction without the effects of superstructure have been performed by numerical model [5–7] and physical experiment [8]. However, the structure and foundation interaction must be treated in order to estimate the behavior of the OWT system, since soil-pile-tower interaction significantly changes the system responses [9,10]. In order to check the stability of a flexible OWT system, the structure and foundation interaction must be treated jointly because the serviceability criteria for monopile at seabed level and tower top are different. Present design approaches are mainly relying on quasi-static load on the offshore wind turbines [5,11,12]. Nevertheless, soil-structure interaction affects significantly the dynamic response of offshore structures [13,14]. Consequently, a dynamic analysis of offshore wind turbine considering soil-structure interaction is indeed necessary for a rational structure design [15–19].

Currently, two methods are always adopted to model the soil-structure interaction on the dynamic responses of OWT. The first method is to model the OWT foundations using three-dimensional solid model [20–24]. Kourkoulis et al. [20] investigated the response of wind turbines founded on suction caissons and subjected to wave and earthquake loading using non-linear three-dimensional finite-element analyses. Kjørlaug and Kaynia [21] studied the vertical earthquake response of wind turbines including the soil-structure interaction effects. Corciulo et al. [22] carried out the dynamic analysis of OWTs subjected to wave and wind loading using 3D finite element model, which is proposed as a valuable support to current design practice. Utilizing the substructure methods, where the soil and the structure are analyzed separately and then coupled at the interface enforcing compatibility and equilibrium conditions, Taddei et al. [23] investigated the effects of the influential factors of the soil-structure interaction on the dynamic response of onshore wind turbines supported by a shallow foundation, and Galvín et al. [24] studied the seismic response of onshore wind turbines account for a monopile foundation and different soil conditions. In addition, physical experiments [25–27] were used to investigate the dynamic properties and dynamic responses of the OWTs considering soil-structure interaction.

Another method is to model the OWT foundations replaced by linear

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springs [28–32], using simple beam on Winkler foundation model based linear p-y curves [33,34] or nonlinear p-y curves [35–40], and replaced by other simplified foundation modeling approaches [41–44]. Adhikari and Bhattacharya [29,30] developed an analytical model based on an Euler-Bernoulli beam-column with elastic end supports. The elastic end supports were considered to model the foundation by two springs (translational and rotational). The proposed model was experimentally validated by Bhattacharya and Adhikari [31]. Feyzollahzadeh et al. [32] proposed an analytical transfer matrix method to determine wind load response of OWT, where the foundations are simplified as coupled springs, distributed springs and apparent fixity length models. Darvishi-Alamouti et al. [34] developed a simplified method to obtain the fundamental frequency of OWTs supported by monopile foundations, where the soil is assumed to be cohesionless and the foundation is simplified as distributed springs. Bisoi and Haldar [36] performed a comprehensive study on the dynamic behavior of OWT supported on monopile foundation in clay under combined wind and wave loading, and Bisoi and Haldar [37] investigated the feasibility of soft-soft and soft-stiff design approaches considering monopile supported OWT founded in clay. In their model, the pile resistance to the pile movement was modeled using the static p-y, t-z and q-z curves to account for soil nonlinearity as suggested in API [45]. Harte et al. [41] studied the wind load response of wind turbines using an Euler-Lagrangian approach. Damgaard et al. [42] presented semi-analytical frequency-domain solutions to evaluate the dynamic impedance functions of the soil-pile system at a number of discrete frequencies. Zania [43] presented a rigorous analytical solution of the modified soil-structure-interaction eigenfrequency and damping. Ghaemmaghami [44] investigated the seismic behavior of wind turbines sitting on a finite flexible soil layer in frequency domain, where the underlying soil is represented by complex dynamic stiffness functions based on cone models.

The aim of this research is to investigate the dynamic responses of OWT supported by monopile foundation in clay subjected to combined wind, wave and earthquake loadings. In this paper, monopile supported an offshore wind turbine is modeled as a beam on nonlinear Winker foundation model. A finite element model of the offshore wind turbine is developed in the open-source software platform OpenSees [46], and dynamic response of the system subjected to wind, wave and earthquake actions is analyzed in time domain.

2. Modeling of offshore wind turbine

An offshore wind turbine model with three blades is shown in Fig. 1. The detailed description of the tower, pile, pile-soil interface and pile-water interface are provided herein.

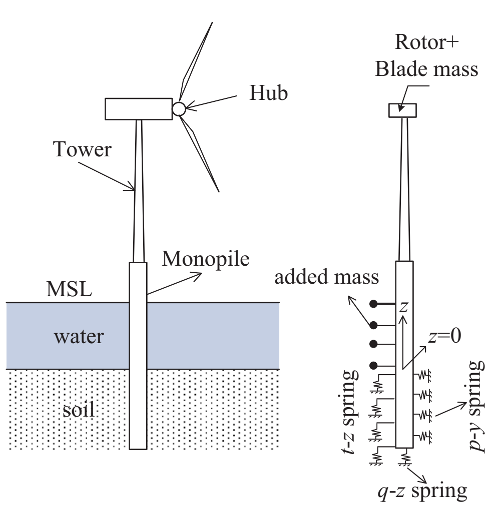

Fig. 1. A monopile supported offshore wind turbine system in clay.

2.1. Tower and pile

The tapered tower is modeled as a number of segments connected according to continuity condition, and each segment of the continuous beam is assumed to obey the characteristics of uniform cross section [47]. Each segment is modeled as one beam element. The equivalent bending stiffness and equivalent linear density of the ith segment of the continuous beam are expressed as

$$EI_i = \frac{1}{l_i}\int_{z_{i+1}}^{z_i} EI(z)\mathrm{d}z \tag{1}$$

$$\rho A_i = \frac{1}{l_i}\int_{z_{i+1}}^{z_i} \rho A(z)\mathrm{d}z \tag{2}$$

where E is the young's modulus, ρ is the density, $I(z)$ is the inertia moment along the height, $A(z)$ is the area along the height, and l_i is the length of the ith segment.

The tower and pile are modeled using 3D fiber-section displacement-based beam column elements with nonlinear fiber material. Each tower and pile node has six degrees of freedom.

2.2. Pile-soil interface

Nonlinear Winkler foundation (BNWF) approach after Boulanger et al. [48] is used to model the pile-soil interface behavior. The soil springs are zero-length elements assigned different uniaxial materials in the lateral and vertical directions. The spring nodes are created with three translational degrees of freedom. One spring node, the fixed node, is initially fixed in all three degrees of freedom. The other nodes, the slave nodes, are initially fixed in only two degrees of freedom, and are later given equal degrees of freedom with the pile nodes.

Laterally oriented p-y spring elements are used to represent the lateral resistance of the pile-soil interface, whereas t-z and q-z spring elements are used to represent the frictional resistance along the length of the pile and the tip resistance at the base of the pile. PySimple1, TzSimple1, and QzSimple1 uniaxial materials from OpenSees are used to define the constitutive behavior of the above-mentioned springs. The details on their backbone equations, validation and applications can be found in Boulanger et al. [48] and Boulanger [49]. The parameters to define the backbone curves for each spring is derived based on the properties of each layer of the deposit. Three parameters including p_{ult}, y_{50}, and C_d are required for PySimple1, two parameters including t_{ult} and z_{50} for TzSimple1, and two parameters including q_{ult} and z_{50} for QzSimple1.

Note that p_{ult}, t_{ult} and q_{ult} represent the ultimate capacity of p-y, t-z and q-z materials, respectively, whereas y_{50} and z_{50} represent the displacement at one-half the load capacity in the respective directions. In this study, p_{ult} and y_{50} for clay, as proposed by Matlock [50] are defined as

$$p_{ult} = \begin{cases} 3s_u + \gamma_0 z + Js_u z/D & z < z_R \\ 9s_u & z \geq z_R \end{cases} \tag{3}$$

$$y_{50} = 2.5\varepsilon_{50} D \tag{4}$$

where γ_0 is the submerged unit weight in kN/m^3, s_u is the undrained shear strength in kPa, z is depth in m, D is the diameter of monopile in m, J is an empirical dimensionless constant, ε_{50} is the strain corresponding to one-half the maximum stress on laboratory undrained compression tests of undisturbed soil, and $z_R = 6s_u D/(\gamma_0 D + Js_u)$ is the depth below soil surface to bottom of reduced ultimate soil strength in m. Matlock [50] suggested that ε_{50} has a range of 0.005–0.02 and he found that J = 0.25 and 0.5 fitted the field test results well at two different sites. The value of J = 0.25 may be applicable for stiffer clays. This study uses J = 0.25 for all cases. Similarly, t_{ult}, q_{ult} and z_{50} are