Variable Natural Frequency Damper for Minimizing Response of Offshore Wind Turbine: Principle Verification through Analysis of Controllable Natural Frequencies

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Abstract: Resonance causes extreme stress, acceleration of fatigue, and reduction in lifespan of offshore wind structures. The main factors that cause resonance are environmental loads such as wind and waves, and dynamic loads caused by rotor movement. Estimation of the natural frequency at the design stage is highly uncertain, and natural frequency changes occur due to various factors during long-term operation. Therefore, it is important to ensure structural safety from resonance through a vibration-monitoring system or an additional damper. In this study, the effect of seawater existing inside the substructure on the natural frequency of the structure was dealt with. The natural frequency estimation equation for a fixed offshore wind structure was derived with the “inner fluid simplification assumption”. The finite element modal analysis was performed to verify the principle of Variable Natural Frequency Damper (VNFD), a system that controls the natural frequency of offshore wind structures through a pump, and to find the range of natural frequency control. As a result, interior fluid affects the natural frequency of the wind turbine support structure. Specifically, the variable natural frequency range was very low, at about 0.027% for the monopile model at a depth of 10 m, but increased rapidly to about 3.66% at a depth of 70 m. Furthermore, when estimating the natural frequency of a fixed offshore wind turbine in deep water without consideration of interior fluid, the estimates can be higher than with consideration of it.

Keywords: offshore wind turbine; substructure; damper; natural frequency; fluid structure interaction; soil structure interaction; linear modal analysis

1. Introduction

Installation of wind turbines has increased significantly around the world over the past few years, and special attention is being paid to offshore wind power installations, which are relatively advantageous in terms of efficiency and development location compared to onshore wind power.

Generally, the offshore wind substructure is designed by the Load and Resistance Factor Design (LRFD) method [1] according to the international design standards [2] for the Ultimate Limit State (ULS) and Fatigue Limit State (FLS) of the working environment. ULS corresponds to the resistance of maximum load, such as typhoons and earthquakes, while FLS refers to the accumulation of fatigue in structures due to repeated loads (stresses) such as blade dynamic loads, wind, and waves.

Offshore wind turbines are being installed at sites deeper and farther from the coast, and the turbine systems supported by substructures are becoming increasingly large [3]. Accordingly, the substructure supporting the turbine system also tends to be bigger. Substructures are classified into various types, but monopile and jacket types are the most popular in Europe, and their installed share in 2018 was 81.7% and 6.9%, respectively [4]. In general, the substructure to be used is selected depending on the depth of water at the...
installation site, the condition of the seabed, and the model of the turbine to be installed. The resonance phenomenon of the structure, due to other loads such cyclic loading, should be avoided, and special attention should be paid to the determination of the stress range and the amount of damping of the excited structure in the resonance range during fatigue design [5].

Resonance caused by wind and waves accumulates fatigue in structures and reduces their lifespan. However, resonance still occurs due to various conditions that are difficult to predict during the design stage, such as harmonic-type winds and waves and their misalignment, and break waves [6]. In addition, changes in soil structural stiffness and response due to cyclic lateral loading can lead to structural resonance and risk of fatigue damage [7].

When designing a fixed offshore wind turbine structure, it is common to design a soft-stiff so that the natural frequency of the structure is between the turbine rotation speed (1P) and the blade passing frequency (3P). The monopile type is not suitable for deep water and is known to be economical at depths of 30 m or less. In particular, in the case of dynamic characteristics, as the capacity of the turbine increases, the natural frequency of the structure becomes closer to the 1P frequency caused by the rotor rotation due to the long and slender substructure. To avoid this, the transition piece connecting the turbine system and the monopile must be designed stiffly. In addition, turbine models with a wide rotor rotational speed range (4.5–14.8 rpm), such as Areva M5000 5 MW, does not have a soft-stiff region, so a sophisticated pitch-control system is designed. Resonance is avoided by skipping through this pitch control, but this reduces economic feasibility [8].

Therefore, in order to solve these problems, this paper focuses on the study of VNFD, a system that is designed to actively avoid the resonance phenomenon. VNFD avoids resonance by controlling the Inner Waver Level (IWL) of the substructure with the seawater pump when the frequency of the external force and the natural frequency of the structure match.

The systematic logic of VNFD is shown in Figure 1a and the theoretical configuration is shown in Figure 1b. The VNFD system is designed such that it compares the natural frequency \( f_{\text{natural}} \) with the forcing frequency \( f_{\text{forced}} \) and, whenever the natural frequency is lower than the forcing frequency, seawater is supplied to the inside of the substructure and, whenever the natural frequency is greater than the forcing frequency, the seawater inside the substructure is drained out. The process of calculating natural frequency and forcing frequency can be achieved using the SCADA system, but other methods can also be used in the actual application because it is difficult to post-process the vibration data every time it is required. The inside of the monopile corresponding to the height of the seabed can be assumed to be filled with soil, so it is assumed that the bed equal to the height of the seabed is also present inside the substructure. Therefore, the controllable IWL range is from the seabed to the platform height. Generally, the inside of the transition piece contains some seawater due to the diameter gap of the two tubular steels when grouting. However, sea water may infiltrate by seal leakage related to the bottom outlet of the substructure wall and J-tube and, as a result, tidal changes may occur inside the foundation [9]. Therefore, unless additional measures are taken, an internal coating of steel may be required for the intended operation of the VNFD, and treatment for sealing may be required. Also, since hydrostatic pressure is applied to the tubular steel as much as the difference between the external water level and the inner water level, this must be considered when applying this technology.

In this paper, after introducing the simplified formula derivation for predicting the natural frequency of the monopile type structure with the seawater inside the substructure, three-dimensional Finite Element (FE) modal analysis was performed using ANSYS 2021R1 to find the range of change in the natural frequency of offshore wind structures according to IWL. According to Arany et al. [8], measured and predicted natural frequencies of wind turbine structures have been reported in various studies [10–15], and the causes of these differences are either the theory or physical idealization of the system or parameters, or
a combination of the two. However, the error caused by ignoring the water inside the substructure was not considered. Since the natural frequency of a wind turbine structure is affected by mass, the mass must be considered for accurate prediction, which is particularly important in monopile types with large internal water or deep-water models. Infiltration of seawater and other outlets as described above were not considered. In Section 2, the latest research related to natural frequencies of offshore wind structures is investigated, and theories related to simulation are introduced. Section 3 describes the FE model, and the results and conclusions are discussed.

Figure 1. (a) System logic of VNFD and (b) theoretical configuration.

2. Materials and Methods

2.1. Natural Frequency of Offshore Wind Turbine Structure with Consideration of Inner Water

Modal analysis is used in determining the vibration characteristics of structures, and vibration characteristics such as natural frequency and mode shape of structures are important in the design of offshore wind structures that are regularly subjected to dynamic loads. In terms of the element equations of the FE modal analysis, damping and external forces are ignored and only the inertia and stiffness terms remain. The basic equation for the general eigenvalue problem in undamped modal analysis is Equation (1).

$$ k\phi_i = \omega^2_m \phi_i $$

where, $\phi_i$ is the mode shape vector for mode $i$, and $\omega_m$ is the natural frequency for mode. Various studies have been conducted on the natural frequency theory of offshore wind structures [16,17]. The first natural frequency for a single degree of freedom (SODF)-concentrated mass model of a wind turbine system with a constant tower cross-section can be expressed as Equation (2).

$$ f_1 = \frac{1}{2\pi} \sqrt{\frac{k_T}{m_{RNA} + \alpha m_T}} $$

where, $k_T = \frac{3E_T I_T}{L_T^3}$ means the lateral stiffness of the tower, and $E_T$, $I_T$, and $L_T$ are the tower’s Young’s modulus, moment of inertia, and length, respectively. Furthermore, $m_{RNA}$ is the mass of rotor-nacelle assembly (RNA), $m_T$ is the mass of the tower, and $\alpha$ is the equivalent mass ratio of the tower concentrated to the tower top. According to van der Tempel and Molenaar [18], when the wind turbine is considered as an inverted pendulum...
with flexural stiffness ($E_T I_T$) and tower mass per meter ($m_t$), the first vibrational mode is equal to Equation (3):

$$f_1 \approx \sqrt{\frac{3.04E_T I_T}{(m_{RNA} + 0.227m_t L_T)4\pi^2 L_T^3}}$$  (3)

However, most circular wind turbine towers have a tapered shape in which the diameter and thickness of the cross-section become smaller as the altitude increases, and many simplified methods have been used [18–20]. According to Yung-Yen Ko [20], the closed-form solution of the natural frequency of wind turbine structures with linearly tapered towers in a rigid-base condition is given by Equation (4):

$$f_{RB(tapered)} = \frac{1}{2\pi} \sqrt{\frac{k_{T(tapered)}}{m_{RNA} + \alpha m_{T(tapered)}}}$$  (4)

where, $k_{T(tapered)}$ is the lateral stiffness of a tapered tower [8], and $m_{T(tapered)}$ is the mass of the tapered tower. However, in Equation (4), only the turbine system starting from the tower base was considered. When the substructure is taken into account, the structure becomes more flexible because it has an additional length with a length proportional to the water depth. Utilizing Castigliano’s second theorem, equivalent lateral stiffnesses can be derived assuming that the section rigidity of the monopile continues to the bottom of the tower [8]. The natural frequency of the structure considering the length of the substructure can be obtained through Equation (5) [20]:

$$f_{RB(tapered+SS)} = \frac{1}{2\pi} \sqrt{\frac{k_{T(tapered+SS)}}{m_{RNA} + \alpha m_{T(tapered+SS)}}}$$  (5)

where, SS is an abbreviation of substructure, and $k_{T(tapered+SS)}$ and $m_{T(tapered+SS)}$ are the lateral stiffness and mass of the tapered tower considering the substructure, respectively.

Inside the monopile substructure, sea water flows in when the foundation is installed as shown in Figure 1b, and its weight is calculated as in Equation (6).

$$m_w = \frac{1}{4}\pi \rho_w H (D_P - 2t_P)^2 \kappa$$  (6)

where $\rho_w$, $H$, $D_P$, and $t_P$ are water density, depth, monopile diameter, and thickness, respectively. Lastly, $\kappa$ is the ratio of water in the interior to the water depth, which is 1.0 when the inner water level is the same as the water depth. For the first natural frequency mode of the cantilever structure, the distance away from the support and the mass value contribute to the vibration mode. Since $m_w$ has a different distance from the mudline than RNA, it is desirable to consider it along with the equivalent mass ratio ($\alpha_w$), and the following “inner fluid simplification assumption” is necessary.

(i) When the dynamic response of the structure is transmitted to the inner water, a sloshing phenomenon occurs due to the inertial effect of the fluid, but this is ignored for the sake of simplification and only considered as a mass.

(ii) Since $m_w$ is not a mass supported by the structure, it is complicated to calculate the mode participation for it. Therefore, the empirical factor ($\psi$) was applied.

Therefore, the equivalent mass ratio ($\alpha_w$) can be assumed to be the mass ratio to the distance of center of gravity with $\psi$ as in Equation (7):

$$\alpha_w = \psi \frac{m_w (C_{Gw} + H)}{m_{RNA} (C_{GRNA} + H)} = \psi \frac{0.5m_w \kappa H}{m_{RNA} (L_T + H)}$$  (7)

where the center of gravity of the RNA ($C_{GRNA}$) is the same as $L_T$ because the sea level is the standard of the height, and the center of gravity of the inner water ($C_{Gw}$) can be calculated as $0.5H(\kappa - 2)$. To consider the distance from the mudline, the depth ($H$) must
be added to $CG_{RNA}$ and $CG_{w}$, respectively. Therefore, the natural frequency considering the additional mass of inner water along with “inner fluid simplification assumption” and can be expressed as Equation (8).

$$f_{RB(tapered+SS+w)} = \frac{1}{2\pi} \sqrt{\frac{k_{T(tapered+SS)}}{m_{RNA} + \alpha m_{T(tapered+SS)} + \alpha w m_{w}}}$$ (8)

There are many studies on the interaction of soil and piles for eigenvalue problems of offshore wind structure [8,16,20–22]. Using the simplified soil-structure interaction (SSI) analysis model proposed in Ref. [23], the flexibility of substructure and foundation can be considered simultaneously with lateral stiffness ($k_{h}$) and rotational stiffness ($k_{r}$), which is given by Equation (9):

$$f_{FF(tapered+SS+w)} = \frac{f_{RB(tapered+SS+w)}}{\sqrt{1 + k_{T}/k_{h} + k_{T}L_{2}^{2}/k_{r}}}$$ (9)

To analyze these resonant frequencies, the log data of the acceleration sensor through the CMS system or SCADA system must be processed using valid data. It is necessary to read the index value of the acceleration sensor and judge the situation while considering not only the operation status of the wind turbine such as start-up, stop, pitching, and yawing, but also wind and wave data; however, this is a lot of work and it is not organized. In particular, it is not easy to establish the effect of the dynamic loads of the rotor and blades with valid data [7]. For this reason, accurate natural frequency estimation is very difficult and unreliable as described above. The natural frequency of an offshore wind turbine always changes depending on the surrounding environment and turbine operation conditions, and it is very difficult to predict at the design stage because different damping factors appear depending on these conditions.

As a result of analyzing the acceleration sensor data for two weeks of an offshore wind power generator installed in the North Sea of Belgium, it was confirmed that the structural frequency changes due to the tidal difference [7]. Through this, it can be seen that the external water level affects the vibration frequency of the structure. Moreover, according to one experiment, the natural frequency for the first mode of the tank containing the fluid decreased as the amount of fluid increased [24]. Therefore, it can be assumed that not only the external fluid but also the IWL affect the natural frequency of the structure, and it is expected that the higher the IWL, the lower the natural frequency of the structure.

When designing an offshore wind structure, it is recommended to avoid the dynamic load frequency due to the rotational speed of the blade and rotor. Accordingly, DNVGL-ST-0126 [5] suggests a 5% range as in Equation (10) so that the excitation frequency and natural frequency of the structure differ by more than 5%.

$$\frac{f_{R}}{f_{0,n}} \leq 0.95 \text{ or } \frac{f_{R}}{f_{0,n}} \geq 1.05$$ (10)

where $f_{R}$ is the excitation frequency, especially the rotational frequency range of the rotor and the transition frequency of the rotor blades in the normal operating range, $f_{0,n}$ is the $n$th natural frequency of the tower and the entire system of the foundation. Additionally, it is recommended to have a margin of $\pm 5\%$ in consideration of the imbalance of the soil and the uncertainty of using the software. It is said that this requirement can be omitted if measures are in place to prevent resonance effects, such as vibration monitoring systems or damping devices [5]. Therefore, international design standards also recommend monitoring based on experience, and an empirical margin far exceeding the avoidance range is recommended.

Since VNFD is different from a general damper in principle, how much influence it will have on these standards can be known through research considering the sloshing effect of inner water and applying it together with other dampers.
2.2. Linear Fluid-Structure Interaction Using ANSYS Code

For the dynamic problem of calculating the natural frequency through the FE model, the modal analysis enables a direct approach. To use this method, eigenvectors must be calculated by solving the symmetric eigenvalue problem, and various formulas can be used for fluid-structure coupled analyses [25].

The three-dimensional Computational Fluid Dynamics (CFD) numerical method is advantageous because the surface pressure distribution can be obtained through CFD software, and this pressure is applied to the structural system. According to [26], how the interactions between fluids and solids are described offers the greatest opportunity of reducing the computational effort, and by using one-way simulations where only fluid pressure data are applied to structures, the computational effort of structural simulation can be greatly reduced.

The equations for coupled fluid-structure problems using pressure-based equations for fluids are described in the frequency domain. The structure problem is written as Equation (11), and the fluid problem is written as Equation (12):

\[-\omega^2 \int_{\Omega_S} \rho_S u_i \delta u_i - \int_{\Gamma} p n_i \delta u_i = 0\]  

(11)

where \(\Omega_S\) is the structure domain with boundary: \(\partial\Omega_S = \partial\Omega_{S_r} \cup \partial\Omega_{SO} \cup \Gamma\) where \(\partial\Omega_{S_r}\) is the boundary part with imposed forces, \(\partial\Omega_{SO}\) is the boundary part with imposed displacement, and \(\Gamma\) is the fluid-structure interface. Furthermore, \(u\) is the structure displacement, \(\rho_S\) stands for structure density, \(n\) is the inward normal on \(\partial\Omega_S\) and \(\sigma(u)\) is the stress tensor.

\[\int_{\Omega_F} \frac{\partial p}{\partial x_i} \frac{\partial \delta p}{\partial x_i} - \omega^2 \int_{\Omega_F} \frac{p \delta p}{c^2} - \rho_F \omega^2 \int_{\Gamma} u_i n_i \delta p = 0\]  

(12)

where \(\Omega_F\) is the fluid domain with \(\partial\Omega_F = \partial\Omega_{FO} \cup \partial\Omega_{SO} \cup \Gamma\), where \(\partial\Omega_{FO}\) is the boundary part with imposed normal gradient pressure (rigid wall or symmetry plane), and \(\partial\Omega_{FO}\) the boundary part with imposed pressure (pressure release surface or anti-symmetry plane). The vector \(n\) is the outward normal on \(\Gamma\), and \(n_F\) is the outward normal on \(\partial\Omega_F\). The fluid density and velocity waves are denoted \(\rho_F\) and \(c\) respectively.

For spatial discretization using finite elements, the mass and stiffness matrices for fluid and structural problems are defined by Equations (13)–(16). The fluid structure interaction matrix used to model the coupling term is defined as Equations (17) and (18):

\[\int_{\Omega_S} \rho_S u_i \delta u_i \to \delta U^T M_S U\]  

(13)

\[\int_{\Omega_S} \sigma_{ij}(u) \epsilon_{ij}(\delta u) \to \delta U^T K_S U\]  

(14)

\[\int_{\Omega_F} \frac{\partial p}{\partial x_i} \frac{\partial \delta p}{\partial x_i} \to \delta U^T K_F U\]  

(15)

\[\int_{\Omega_F} \frac{p \delta p}{c^2} \to \delta U^T M_F U\]  

(16)

\[\int_{\Gamma} p n_i \delta u_i d\Gamma \to \delta U^T R P\]  

(17)

\[\int_{\Gamma} u_i n_i \delta p d\Gamma \to \delta U^T R^T U\]  

(18)

Finally, the coupled problem is defined in Equation (19):

\[
\begin{bmatrix}
K_S & -R \\
0 & K_F
\end{bmatrix}
\begin{bmatrix}
U(\omega) \\
P(\omega)
\end{bmatrix}
= \omega^2
\begin{bmatrix}
M_S & 0 \\
R^T M_F & M_F
\end{bmatrix}
\begin{bmatrix}
U(\omega) \\
P(\omega)
\end{bmatrix}
\]  

(19)
where \( U(\omega) \) and \( P(\omega) \) are the displacement and pressure in frequency domain, respectively, and \( R \) is fluid-structure interaction matrix. The mass and stiffness of structure and fluid are denoted \( M_S \), \( K_S \), \( M_F \), and \( K_F \), respectively. The displacement-pressure formulation leads to an unsymmetric eigenvalue problem that can be solved with the unsymmetric Lanczos algorithm \([27,28]\).

In this paper, the ANSYS modal acoustic analysis model was used for modal analysis considering fluid–structure interaction (FSI). The interaction between the structure and the fluid is formed by pressure rather than velocity \([29]\). This involves modeling the fluid medium as well as the surrounding structure to determine frequencies and standing wave patterns within the structure. Instead of using the two-way FSI analysis in which the deformation of the structure is reflected in the fluid solver at the same time, by using the one-way FSI, the VNFD can reduce damping caused by the air-conditioning of the fluid inside the substructure such as the principle of other additional dampers. Ignoring it means accepting the interaction with the fluid outside the substructure in the form of one-way pressure, and through this assumption, the efficiency of the analysis can be greatly improved.

2.3. Three-Dimensional Soil-Structure Interaction

The Soil–Structure Interaction (SSI) is a very important factor in vibration studies because it is the largest damping factor in offshore wind turbine structures \([30,31]\), as well as designing capacity \([9,32]\). Jalbi et al. \([33]\) broadly classified methods to consider these SSI into three groups as simplified SSI, standard SSI (see Figure 2a), and advanced method (finite element shown in Figure 2b). In this paper, the FE model method is used because it is difficult to converge the analysis using p-y, t-z, and q-z spring \([34]\), which is a nonlinear spring theory, and it is convenient to implement the condition in which the fluid elements inside the substructure is supported by soil elements. Additionally, the layered soil in this paper consist of sand. In general, unlike sand, clay exhibits physical properties such as a high damping factor due to its adhesion. If clay is used instead of sand, and other dynamic characteristics and other conditions are the same, the natural frequency is generally reduced.

Figure 2. Soil-pile interaction of (a) distributed nonlinear spring model and (b) FE model.
3. Finite Element Model

3.1. FE Model for Wind Turbine and Soil Layers

This section deals with the descriptions of modeling and verification of the FE model. Under the VNFD principle, the substructure type was selected as monopile because of the assumption that the bulky type containing the fluid inside the substructure is advantageous for application and the reason that the accumulated installed capacity is the largest among the offshore wind power substructure types [4]. The software used in the analysis was ANSYS 2021R1. A reference 5.0 MW turbine model of National Renewable Energy Laboratory (NREL) was used for reliability, and the specifications for mass and dimensions of RNA and tower are shown in Table 1 [35]. The weight of the RNA was applied as a lumped mass. The thickness of the tower is linearly decreased 0.027 m to 0.019 m. The dimensions of the geometry are shown in Figure 3. Since the parameter for the depth, \( H \), was applied in this paper, it is denoted by a symbol in Figure 3a. The density of the tower material was set at 8500 kg/m\(^3\), taking into account bolts, paint, welds, and flanges, with a typical density of 7850 kg/m\(^3\) for steel [35]. The grout material is Ducorit D4 [36], and the details of the materials are shown in Table 2. Material strength is not described in this paper because the properties for linearity are considered and stress is not of interest. The soil was assumed to be a sand layer without clay, and the properties for each layer are shown in Table 3. The fluid was assumed to be sea water and had a density of 1025 kg/m\(^3\) [37].

Table 1. Specification of turbine model of NREL 5 MW reference wind turbine [35].

| Contents                               | Value       |
|----------------------------------------|-------------|
| Length of tower                        | 87.6 m      |
| Total mass of tower                    | 347,460 kg  |
| Outer diameter of tower top            | 3.87 m      |
| Thickness of tower top                 | 0.019 m     |
| Outer diameter of tower bottom         | 6.0 m       |
| Thickness of tower bottom              | 0.027 m     |
| Total weight of RNA                    | 350,000 kg  |
| CM height from tower top of RNA        | 1.817 m     |

Table 2. Material properties of steel, grout [36].

| Material                    | Young's Modulus (GPa) | Poisson's Ratio | Density (kg/m\(^3\)) |
|-----------------------------|-----------------------|----------------|----------------------|
| Steel                       | 210                   | 0.38           | 8500                 |
| Grout                       | 70                    | 0.19           | 2740                 |

Table 3. Material properties of soil layers [38].

| Material | Depth (m) | \( \varphi \) (kN/m\(^3\)) | \( c \) (kPa) | \( \varphi \) (degree) | \( \sigma \) (-) | \( E_s \) (MPa) | Dilation Angle (degree) |
|----------|-----------|-----------------------------|---------------|------------------------|-----------------|-----------------|------------------------|
| Sand 1   | 0.0-5.0   | 10.0                        | 0.05          | 33.0                   | 0.3             | 30.0            | 3.0                    |
| Sand 2   | 5.0-14.0  | 10.0                        | 0.05          | 35.0                   | 0.3             | 35.0            | 5.0                    |
| Sand 3   | 14.0-36.0 | 10.0                        | 0.05          | 38.5                   | 0.3             | 47.0            | 8.5                    |

For the contact between the grout and the steel interface, a bonded contact that does not allow sliding or gaps was used. Bonded contact conditions in three-dimensional FE analysis permanently hold the target and contact surfaces [39–41]. The frictional coefficient of contact between the soil and the structure is obtained through the angle of friction (\( \varphi \)) in Table 3 [38]. However, through the linear assumption of the modal analysis, this non-
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| Material | Young’s Modulus (GPa) | Poisson’s Ratio (-) | Density (kg/m³) |
|----------|------------------------|---------------------|-----------------|
| Steel    | 210                    | 0.38                | 8500            |
| Grout    | 70                     | 0.19                | 2740            |

Table 3. Material properties of soil layers [38].

| Material | Depth (m) | γ (kN/m³) | c (kPa) | φ (Degree) | ν (-) | E_s (MPa) | Dilation Angle (Degree) |
|----------|-----------|-----------|---------|------------|-------|-----------|-------------------------|
| Sand 1   | 0.0–5.0   | 10.0      | 0.05    | 33.0       | 0.3   | 30.0      | 3.0                     |
| Sand 2   | 5.0–14.0  | 10.0      | 0.05    | 35.0       | 0.3   | 35.0      | 5.0                     |
| Sand 3   | 14.0–36.0 | 10.0      | 0.05    | 38.5       | 0.3   | 47.0      | 8.5                     |

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For the boundary condition, the outer and bottom surfaces of the soil were set to be completely fixed except for vertical DOF of outer surfaces (see Figure 2b), and the gravitational acceleration was 9.81 m/s² in the -z direction.

3.2. Verification of FE Models

To secure the reliability of the meshing method of the FE model, a convergence test of the natural frequency analysis results was performed while increasing the number of elements using the soil lateral direction, height direction, and bias as parameters, and the results are shown in Table 4. The difference term in Table 4 is the error rate for the natural frequency of each test step and the natural frequency of the previous step. The reason for applying the parameter based on the soil is that, the discretization of the lateral, axial, and circumferential directions of the soil coincides with the nodes at the interface of the elements of the structure or fluid. This technique is also used in many other studies using FE analysis, and the reliability of the FE model can be secured [36,43,44]. The soil model used in this process used a different model from the scenarios of this paper, but other conditions were the same, so it was judged that the reliability test result of the meshing method could be accepted.

Table 4. Results of mesh convergence test.

| Test No. | Number of Elements (-) | Natural Frequency (Hz) | Difference (%) |
|----------|------------------------|------------------------|----------------|
| 1        | 5148                   | 0.26025                |                |
| 2        | 6894                   | 0.26030                | 0.0192         |
| 3        | 8580                   | 0.26046                | 0.0615         |
| 4        | 10,386                 | 0.26035                | 0.0346         |
| 5        | 13,788                 | 0.26064                | 0.0345         |
| 6        | 18,673                 | 0.26073                | 0.0345         |
| 7        | 24,275                 | 0.26072                | −0.0038        |
| 8        | 35,700                 | 0.26077                | 0.0192         |
Eight mesh convergence tests were performed, and small differences occurred from test 6 as shown in Figure 4. As a result, the soil elements were divided into 32 pieces in the circumferential direction and 1500 mm in the axial direction, and 8 layers were gradually densely formed in the area adjacent to the structure in the transverse direction. The diameter of the soil model was modeled as 20 times the monopile diameter (see Figure 5).

![Figure 4. Mesh convergence graph.](image)

In order to secure the reliability of the tower model, as shown in Table 5, the simulation results of the FE model of this paper were compared with the results of other studies of the NREL turbine model. Since the main concern of VNFD is the soft–soft domain or soft–stiff domain, it is the first natural frequency, so the results are analyzed only for the first mode. As a result, the natural frequency of the rigid base model was about 0–4% lower than that of other studies, considering only the shape of the tower. The result of fixed soil in which the fixed condition was set in the mudline was about 0–0.5% higher. Considering the error of the rigid base model, it is assumed that the error in the geometry of the substructure and the error in the tower are offset. Through the results of the flexible soil model, it is expected that an error of about 2–5% occurs from the physical properties of the soil or the interaction between the soil and the foundation. Consequently, the model for the first mode analysis and the model’s meshing method are acceptable.

![Figure 5. FE model for (a) structures with soil layers and (b) tubular steels.](image)
the fixed condition was set in the mudline was about 0–0.5% higher. Considering the error of the rigid base model, it is assumed that the error in the geometry of the sub-structure and the error in the tower are offset. Through the results of the flexible soil model, it is expected that an error of about 2–5% occurs from the physical properties of the soil or the interaction between the soil and the foundation. Consequently, the model for the first mode analysis and the model’s meshing method are acceptable.

Table 5. Comparison of the simulation results of the FE model with the results of other studies of the NREL turbine model for the rigid base (RB), fixed soil (FS) and flexible soil (FF) condition.

| Support Conditions | Method | 1st Tower Fore-Aft (Hz) | 1st Tower Side-to-Side (Hz) | Relative Difference (%) |
|--------------------|--------|------------------------|-----------------------------|-------------------------|
|                    |        |                        |                             |                         |
| Rigid base         | $f_{RB, \text{ FE}}$ (present) | 0.3245                  | 0.3250                      | -                       |
|                    | FAST [35] | 0.3240                  | 0.3120                      | 0.14                    |
|                    | ADAMS [35] | 0.3195                  | 0.3164                      | 1.54                    |
|                    | $f_{RB}$ [20] | 0.3123                  | 0.3123                      | 3.76                    |
| Fixed soil         | $f_{FS, \text{ FE}}$ (present) | 0.2763                  | 0.2766                      | -                       |
|                    | $f_{FS}$ [36] | 0.2760                  | 0.2780                      | 0.09                    |
|                    | $f_{FS}$ [45] | 0.2770                  | 0.2780                      | -0.27                   |
| Flexible soil      | $f_{FF, \text{ FE}}$ (present) | 0.2548                  | 0.2550                      | -                       |
|                    | $f_{FF}$ [38] | 0.2420                  | 0.2410                      | 5.02                    |
|                    | $f_{FF}$ [36] | 0.2450                  | 0.2480                      | 3.85                    |

4. Results and Discussion

4.1. Impact of Inner Water Level and Water Depth Parameters

The theoretical manageable IWL range through VNFD is from the seabed of the installation site to the height of the maintenance platform, and is determined according to the depth of the site. A monopile type is considered to be economical when the water depth is not more than 30 m [3], but the water depth was selected as a parameter to investigate the effect of the installation depth on the variable natural frequency region. Therefore, the effect of the IWL on the natural frequency of the entire structure model, except for rotor nacelle assembly, (RNA) was analyzed for a depth of between 10 to 70 m, and the controllable natural frequency range using VNFD was analyzed. When the manageable IWL was fully drained out, it was named M0, and when it was fully supplied, it was named M100, and IWL parameters were set from 0 to 100% in 20% increments. Through this, natural frequency analysis according to the change of IWL was conducted. The name of the analysis scenarios to which parameters are applied are shown in Table 6.

Table 6. Parameter setting (water depth for site & inner water level inside of substructure).

| IWL (%) | 10 | 20 | 30 | 40 | 50 | 60 | 70 |
|---------|----|----|----|----|----|----|----|
| 0       | D10M0 | D20M0 | D30M0 | D40M0 | D50M0 | D60M0 | D70M0 |
| 20      | D10M20 | D20M20 | D30M20 | D40M20 | D50M20 | D60M20 | D70M20 |
| 40      | D10M40 | D20M40 | D30M40 | D40M40 | D50M40 | D60M40 | D70M40 |
| 60      | D10M60 | D20M60 | D30M60 | D40M60 | D50M60 | D60M60 | D70M60 |
| 80      | D10M80 | D20M80 | D30M80 | D40M80 | D50M80 | D60M80 | D70M80 |
| 100     | D10M100 | D20M100 | D30M100 | D40M100 | D50M100 | D60M100 | D70M100 |

The natural frequency results of first mode according to water depth and IWL are shown in Tables 7 and 8. Based on the rotor, the results for the fore-aft (FA) direction are shown in Table 7, and the results for the side-to-side (SS) direction are shown in Table 8. There was a difference between the FA mode and the SS mode as the center of gravity of RNA was spaced apart from the axis of the support structure, and it can be seen that the difference was not large. Therefore, we will discuss only the FA mode results from now on.
In the previous work, the natural frequency of the first mode of 5.0 MW turbine model is generally in the range of 0.115 to 0.250 Hz [46]. The first fore–aft natural frequencies of this study were analyzed in the range from 0.1520 (D70M100) to 0.2802 (D10M0) Hz from Table 7.

Table 7. Results of natural frequency analysis for first fore-aft mode.

| IWL (%) | 10   | 20   | 30   | 40   | 50   | 60   | 70   |
|---------|------|------|------|------|------|------|------|
| 0       | 0.27984 | 0.25591 | 0.23333 | 0.21222 | 0.19257 | 0.17441 | 0.15775 |
| 20      | 0.27983 | 0.25590 | 0.23331 | 0.21220 | 0.19254 | 0.17438 | 0.15771 |
| 40      | 0.27982 | 0.25587 | 0.23326 | 0.21212 | 0.19242 | 0.17421 | 0.15751 |
| 60      | 0.27981 | 0.25582 | 0.23314 | 0.21189 | 0.19205 | 0.17369 | 0.15683 |
| 80      | 0.27979 | 0.25573 | 0.23289 | 0.21139 | 0.19121 | 0.17245 | 0.15518 |
| 100     | 0.27976 | 0.25558 | 0.23246 | 0.21046 | 0.18959 | 0.17003 | 0.15196 |

Table 8. Results of natural frequency analysis for first side-to-side mode.

| IWL (%) | 10   | 20   | 30   | 40   | 50   | 60   | 70   |
|---------|------|------|------|------|------|------|------|
| 0       | 0.28018 | 0.25617 | 0.23352 | 0.21237 | 0.19267 | 0.17448 | 0.15780 |
| 20      | 0.28017 | 0.25615 | 0.23350 | 0.21234 | 0.19264 | 0.17445 | 0.15777 |
| 40      | 0.28016 | 0.25613 | 0.23345 | 0.21226 | 0.19252 | 0.17429 | 0.15756 |
| 60      | 0.28015 | 0.25608 | 0.23333 | 0.21203 | 0.19215 | 0.17376 | 0.15688 |
| 80      | 0.28013 | 0.25598 | 0.23308 | 0.21153 | 0.19131 | 0.17253 | 0.15523 |
| 100     | 0.28010 | 0.25584 | 0.23265 | 0.21059 | 0.18969 | 0.17010 | 0.15201 |

Figure 6 shows the natural frequency range by IWL change as a percentage. Overall, the deeper the water depth, the lower the natural frequency of the support structure linearly, and the lower the IWL, the higher the natural frequency nonlinearly, and this will be discussed in Section 4.2. The controllable range of natural frequency of the 10 m was about 0.027%, which was very small, but at a depth of 70 m, the relative frequency was 3.666%, which was relatively large. Since the splash zone was ignored and the external water level was considered constant, only the influence on the IWL was used as a variable.
Table 9 shows the comparison of the natural frequencies estimated without considering the internal fluid for the flexible soil model \( f_{FF(tapered+SS)} \) and the estimated natural frequencies considering the internal fluid \( f_{FF(tapered+SS+w)} \), and the simulation results of this paper. The low-depth model showed no significant difference in the error rate of the two natural frequency estimation equations, but the high-depth model had a large error.

Table 9. Comparison of natural frequency estimates for flexible soil (Equation (9)) without consideration of water (A), with consideration of water (B), and simulation result (C) \( (\kappa = 1.0) \).

| Item | Water Depth | 10 m | 20 m | 30 m | 40 m | 50 m | 60 m | 70 m |
|------|-------------|------|------|------|------|------|------|------|
| A    | \( f_{FF(tapered+SS)} \) | 0.2885 | 0.2662 | 0.2447 | 0.2241 | 0.2048 | 0.1866 | 0.1699 |
| B    | \( f_{FF(tapered+SS+w)} \) | 0.2884 | 0.2658 | 0.2436 | 0.2221 | 0.2014 | 0.1820 | 0.1639 |
| C    | Results    | 0.2801 | 0.2558 | 0.2326 | 0.2106 | 0.1897 | 0.1701 | 0.1520 |

| Difference between A and C (%) | 2.90 | 3.90 | 4.93 | 6.05 | 7.36 | 8.86 | 10.51 |
| Difference between B and C (%) | 2.89 | 3.77 | 4.51 | 5.17 | 5.84 | 6.54 | 7.28 |

4.2. Comparison with Derived Equation for Considering Inner Water

In Section 4.2, a discussion of the impact of IWL is dealt with. By substituting Equations (6) and (7) into Equation (8), it can be expressed as Equation (20):

\[
\frac{k_{T(tapered+SS)}}{m_{RNA} + \Delta m_{T(tapered+SS)} + \psi \frac{\pi^2 p_w H^3 (D_p - 2t_p)^3 \kappa^3}{32 m_{RNA} (L_T + H)}}.
\]  

(20)

where the ratio of water in the inner to the water depth \( \kappa \) means IWL with a range from 0 to 1. The empirical coefficient \( (\psi) \) created by the “inner fluid simplification assumption” can be assumed to be 0.0152 through the simulation results of this paper.

We can expect that the natural frequency will change non-linearly with a linear change of \( \kappa \). Since all the variables constituting the term \( \Delta m_w m_w \) of Equation (8) are constants when the water depth, density of water, and the cross section of the substructure are constant, \( \kappa \) dominates. Figure 7 shows the natural frequency according to the change of IWL. The prediction that it would change non-linearly according to the change in \( \kappa \) was also found in the analysis results. Raising \( \kappa \) from 0.8 to 1.0 is about 3.7 times more effective than raising it from 0 to 0.2 in 10 m depth, while about 95.2 times more effective in 70m depth. Even if equal volumes of water are compared, it is clearly effective in deep water depth models.
5. Conclusions

This paper is a study on the impact of natural frequency of interior fluid of fixed offshore wind turbine substructure and the development of a Variable Natural Frequency Damper (VNFD) for resonance avoidance of offshore wind turbine structures, and the conclusion of this study is as follows.

First, the inner water of fixed offshore wind turbine substructure affects the natural frequency of the wind turbine structure: the deeper the model, the larger the controllable natural frequency range. Specifically, the difference between fully supplied and drained
was very small at 0.027% in the 10 m water depth model, but was relatively large at 3.67% in the 70 m model.

Second, when estimating the natural frequency of fixed offshore wind turbine in deep water without consideration of interior fluid, a higher estimate is obtained than with consideration of it. The natural frequency estimation formula considering the internal fluid together with “Inner Fluid Simplification Assumption” had a smaller error rate when compared with the simulation than the estimation formula without considering the internal fluid. In particular, the difference in error rates increased rapidly as the water depth increased.

Third, even when controlling the same amount of water, controlling water at a high altitude has a greater effect on natural frequency. In particular, when 20% of the total was supplied from the lower part and upper part of the substructure, the difference in the effect on natural frequency was 3.7 times in the 10 m model and 95.2 times in the 70 m model.

As the water depth increases, the diameters of the monopile and the transition piece increase, and accordingly, the amount of fluid inside the substructure increases significantly. In this paper, the diameter of the substructure according to the water depth was kept constant. Therefore, in actual application, the natural frequency control effect through VNFD is expected to be greater. Also, the deeper the water depth, the higher the significant wave height, and this was considered in calculating the height of the maintenance platform when designing the offshore wind structure. In this study, the splash zone was not reflected in the FE model because water depth parameters were applied and wave height data were not used. Therefore, if the wave height is considered, the effect of VNFD is expected to be greater. In the deep water, for the application of this technique, the effect of hydrostatic pressure must be taken into account. The VNFD should be utilized so that the water level difference is not large and the controllable natural frequency is high.

A pressure-based formulation is used when considering fluid-structure interactions. Therefore, the tuning effect or sloshing mode of the inner water of the substructure was not analyzed with the “inner fluid simplification assumption”. However, as for other tuned dampers used in practice, it is expected that the inner water of the substructure will also have a damping effect due to the tuning effect. These simplifying assumptions should be simulated to reflect actual physical phenomena in future studies. In addition, the effect on other types of substructures will be different from those of monopile types, so this also requires further study.

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