Finite element frequency analysis of offshore wind turbine structure under soil and structure interaction

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ABSTRACT

In recent years, nations over the world were looking for alternative energy sources to reduce global warming caused by traditional fossil fuel burning. Renewable energy sources like wind power have great opportunities in this direction, because of low pollution. The offshore wind turbine (OWT) structures are frequently subjected to dynamic loads during operation. Hence, knowledge of their dynamic characteristics such as their natural frequencies, is essential to avoid damaging resonances.

In this study, the Abaqus finite element program is used to calculate the natural frequency of a jacket-type offshore wind turbine supporting structure. During the analysis, the nonlinear behavior has also been considered. Four models: the finite element model, the hybrid finite-infinite element model with the soil material simulated by infinite element, the soil spring model adopting the soil equation proposed by the American Petroleum Institute, and the equivalent soil spring model were utilized in this study. The nonlinear behavior of the soil-pile interaction and the natural frequency of the OWT structure were thoroughly analyzed.

KEY WORDS: offshore wind turbines, natural frequency, soil spring

INTRODUCTION

Many offshore wind turbines use the jacket foundation for a cost-effective design. With the small area and high stiffness of the tubular members, the stability and the resistance of the structure to withstand the environmental load is higher than those of other types of foundations. Many other types of foundations such as gravity, monopiles, and tripods, are also used as the support of offshore wind turbine structures (Kaiser and Snyder, 2012). During the design of the structure, it’s essential to correctly estimate the natural frequency to prevent resonances causing damage to the offshore wind turbine (OWT) structure.

In the analysis of the static and dynamic response of offshore wind turbine structures, the soil-pile interaction plays a significant role. Poulos (1971) proposed an elastic solution for the horizontal displacement and rotation of a single pile subjected to lateral loading and moment. Further research focused on the nonlinear behavior of the soil-pile interaction, which may be described as a mathematical function of soil and pile properties. Several researches used the p–y curve method to investigate the nonlinear behavior of different soil types. O’Neill and Murchinson (1983) developed the p–y curves for sand, while Dunnavant and O’Neil proposed a p-y curve for clay. Many results of these researches were adopted by the API standard (2000), which serves as the basis for many designs of OWT structures. Mitwally and Novak (1987) used linear analysis to depict the pile–soil interaction regarding the response of an offshore tower to wave loading. Chaudhry (1994) used finite element method and boundary element method to investigate the pile-soil interaction with single and multiple piles in offshore, and determined the ultimate lateral soil reaction. Another method for soil-pile interaction analysis is the finite/infinite element method. The use of the infinite element method on semi-infinite domains can prevent the reflection of energy from the boundary to the domain of interest. Medina (1992) presented a dynamic soil-structure interaction for homogeneous and layered soil using infinite element method.

In dynamic response and the modal analysis of the OWT structure, Moll and Busmann (2010) investigated the effects of the added hydrodynamic mass on the result of the modal analysis. Lombardi et al. (2013) investigated the dynamic response of offshore wind turbines which stood on monopile foundations, considering the soil-structure interaction, and proposed a practical guideline for choosing the diameter of monopile foundations. Mostafa and El Naggar (2004) discussed the response of offshore platforms subjected to lateral loading due to wave forces, and concluded that the foundation flexibility results in an increase in the velocity, acceleration and the displacement of the offshore tower.

The foundation of an offshore wind turbine should be designed according to the site conditions, like water depth, location, maximum wind speed, wave heights, and currents. In this study, we selected the jacket foundation as prototype, as shown in Fig. 1(a). The jacket foundation is a steel truss template, comprised of a welded frame of tubular members. With the small surface area and the high stiffness of the tubular members, the stability and the resistance is higher than those of other types of foundations. The four piles are driven into the seabed through the legs of the jacket. Jackets can be used in deep water not exceeding 50 meters.

The main purpose of this paper is to investigate the natural frequency of the OWT structure considering pile-soil interaction under geo-static balance. We will be using finite/infinite element method along with the soil spring method based on API standards (2000).
The environmental and geological conditions of the offshore turbine structure influence the soil-pile interaction and the seismic response of the structure. To simplify the simulation of the natural frequency analysis, we assume medium density for sand-type soil. In this study, two different methods with four models are used to investigate the nonlinear behavior of soil-pile interaction, and the natural frequency of the OWT. We want to compare four different kinds of models: the finite element model, the hybrid finite-infinite element model, the soil spring model, and the equivalent soil spring model. The accuracy of the hybrid finite-infinite element model was also verified.

Resistancedeflection (p-y) curves

The nonlinear relationship between lateral soil resistance and deflection are analyzed via four empirical formulas based on soil types in A. P. I. (2000). The load-deflection (p - y) curves stand for soft clay, stiff clay, and sand. In this study, we adopt the load-deflection (p - y) curves for sand. The non-linear deflection curves of lateral soil resistance change with the depth of the soil, as follows.

\[ p = A \times p_u \times \tanh \left( \frac{k_h \times H}{A \times p_u} \times y \right) \]  

where \( H \) denotes the depth and \( y \) denotes the lateral deflection. Parameter \( A \) denotes a factor accounted for cyclic or static load conditions. \( A = 0.9 \) for cyclic loading and \( A = 3.0-0.8 \times \frac{D}{H} \) for static loading, where \( D \) is the diameter of pile. Parameter \( k_h \) denotes the initial modulus of subgrade reaction. Parameter \( P_u \) is the ultimate bearing capacity at depth \( H \), where \( P_u \) can be expressed by Eq. (2)-Eq. (4)

\[ P_u = \min \{ p_u, P_{us} \} \]  

\[ P_{us} = (C_1 \times H + C_2 \times D) \times g \times H \]
Parameter $\gamma$ stands for the effective soil weight, $C_1$, $C_2$, $C_3$ are fitting coefficients that can be determined from Eq. (5)- Eq. (7), where $K$ is the horizontal earth pressure coefficient, $\varphi$ is the angle of friction and $\beta$ is a parameter equals to $45 + \varphi/2$.

$$C_1 = \frac{K \tan \beta \sin \beta}{\tan (\beta - \varphi)} + \frac{\tan \beta \tan^2 (45 - \frac{\varphi}{2})}{\tan (\beta - \varphi)} + K \tan \beta (\tan^2 \beta - \tan \beta/2)$$

$$C_2 = \frac{\tan \beta}{\tan (\beta - \varphi)} - \tan^2 \left(45 - \frac{\varphi}{2}\right)$$

$$C_3 = K \tan \beta \tan^4 \beta + \tan^2 \left(45 - \frac{\varphi}{2}\right) \left(\tan^2 \beta - 1\right)$$

Axial load transfer (t-z) curves and pile tip-load—displacement (Q-z) Curve

Many researches intended to determine the relationship – also referred to as t-z curves – between the axial load transfer and pile displacement. The equations proposed in A. P. I. (2000) were based on many studies. Coyle and Reese (1900) proposed empirical t-z curves of clay soil that were based on pile load tests. The t-z curves for clay and sand were provided by Vijayvergiya and Reese (1977). Kraft et al. (1981) also introduced theoretical t-z curves. Reese and M. O’Neill described the load deflection relationship for grouted piles. The recommended curves were introduced in A. P. I. (2000), and the recommended relationships are shown in Table 1. In our formulation, $z$ is the local pile deflection measured in inches, $D$ is the pile diameter, $t$ is the mobilized soil-pile adhesion, and $t_{\text{max}}$ is the maximum soil-pile adhesion, which can be computed by Eq. (8).

$$t = K p_0 \tan \delta$$

Table 1 Load deflection relationships recommended in A. P. I. (2000)

<table>
<thead>
<tr>
<th>Clays</th>
<th>$z/D$</th>
<th>$t/t_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0016</td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td>0.0031</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>0.0057</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>0.0080</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>0.0100</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>0.0200</td>
<td>0.70 to 0.90</td>
<td></td>
</tr>
<tr>
<td>$\infty$</td>
<td>0.70 to 0.90</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sands</th>
<th>$z$ (in.)</th>
<th>$t/t_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.000</td>
<td>0.000</td>
<td></td>
</tr>
<tr>
<td>0.100</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>$\infty$</td>
<td>1.00</td>
<td></td>
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</tbody>
</table>

NUMERICAL SIMULATION

In this study, two different methods with four models are used to investigate the nonlinear behavior of soil-pile interaction, and the natural frequency of the OWT. The superstructure and the foundation of the OWT in these four models are the same, the truss template is modeled by beam elements (B32H), and the four piles are modeled by an eight-node brick element (C3D8), while the difference lies between the models is the method used to simulate the soil-pile interaction. The first model is the finite element model using finite element method to simulate the soil-pile interaction. The second model is the hybrid finite-infinite element model using finite and infinite element method on semi-infinite domain. The third model is soil spring model using soil spring method while the soil was simulated by three types of soil springs, based on empirical API equations. The last model is the equivalent soil spring model using equivalent soil spring obtained from the soil spring model to identical the soil-pile interaction with far few less springs.

Finite element model

In the finite element model, the soil material, the truss template and the pile are all modeled by finite element. In the field of geotechnical engineering, the semi-infinite domain problems are usually considered. In the case of static analysis, it is customary to set the boundary condition on the finite element when the depth of the soil under the pile is 1 – 1.5 times the length of the pile, and the diameter of the soil body is up to 20 – 30 times the radius of the pile. Therefore, in our finite element model, we took the depth of the soil as twice the length of the pile, while the diameter of the soil body as 200m, that is, 100 times the diameter of the pile. The result obtained from the finite element model was used to compared and prove the accuracy and the reliability of the hybrid finite element-infinite element model.

Hybrid finite-infinite element model

However, when dynamic load is applied to the finite element model, the wave reflects into the soil body from the truncated boundary. That modeling phenomenon is unrealistic when the size of the soil body simulated by finite element is not enough to dissipate the energy, thus we adopted the infinite element method to solve the semi-infinite domain problems.

According to the theory of the infinite element method, the distance between the pole node and the boundary of the infinite element should be twice the distance between the pole node and the finite element boundary. Considering the continuity of the type of the
element, the CIN3D8 was used for infinite element, and the C3D8 was used for the finite element, the model and its schematic diagram are shown in Fig. 4 and Fig. 5.

Since the infinite elements do not provide any stiffness to the system, the infinite elements was assumed linear elastic while the finite elements are nonlinear in the dynamic analysis. The dynamic equation in one dimension can be expressed as Eq. (11).

\[-\rho \ddot{u} + E \frac{\partial^2 u}{\partial x^2} = 0\]  

Here $\rho$ is material density, $E$ is the Young's modulus. The solution of the above equation is $u = f(x \pm ct)$, where $c$ is the wave velocity which can be shown as $c = \sqrt{E/\rho}$ . The boundary of the finite element and the infinite element was located at $x=L$. The stress on the boundary is expressed by Eq. (12), with a damping boundary condition in effect and $d$ being the impedance. The incident wave and the reflected wave can be expressed as $u = f_1(x-ct), u = f_2(x+ct)$, respectively.

\[\sigma = -d \frac{\partial u}{\partial x} - d\left(\frac{c_1^2}{c_1^2} - c_2^2 (x) + c_2^2 (x)\right)\]  

In general, the tensile action was not considered in non-cohesive soil behavior. Therefore, the soil springs work only if they are subjected to pressure. Considering the three dimensional analysis, two pairs of lateral springs and vertical springs (four $p - y$ springs and four $t - z$ springs) were used to represent the horizontal and vertical soil resistance in each layer. In addition, a $q - z$ spring was used to model the pile tip resistance. The diagram is shown in Fig. 6. The non-linear soil springs were constructed in Abacus by inputting the load-displacement into the axial connector. Our follow-up simulation was done by 1444 soil springs: with 4 $q - z$ springs, 720 $p - y$ springs, and 720 $t - z$ springs. The parameters used in the soil spring model are shown in Table 3.

**Soil Spring Model**

In this study, the soil was discretized into 45 layers, soil springs were set per unit length along the pile, and the length of the pile was 45m. In that case, the soil springs were set at -0.5 m, -1.5 m, -2.5 m and so on till -44.5 m. Three types of soil springs were set: $p - y$ spring, $t - z$ spring and $q - z$ spring. The $p - y$ spring described the lateral soil–pile resistance and the lateral relative displacement. The $t - z$ spring described the end-bearing resistance of the soil.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value</th>
</tr>
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<tbody>
<tr>
<td>$C_1$</td>
<td>2.17</td>
</tr>
<tr>
<td>$C_2$</td>
<td>2.82</td>
</tr>
<tr>
<td>$C_3$</td>
<td>35</td>
</tr>
<tr>
<td>Friction angle of soil $\phi$</td>
<td>31.5</td>
</tr>
<tr>
<td>Initial modulus of subgrade reaction $K$</td>
<td>17527.65</td>
</tr>
<tr>
<td>Frictional coefficient $\mu$</td>
<td>0.466</td>
</tr>
<tr>
<td>$Z/D$</td>
<td>0.01</td>
</tr>
<tr>
<td>Friction angle between the soil and pile wall $\delta$</td>
<td>25</td>
</tr>
<tr>
<td>At rest lateral earth pressure coefficient $k_0$</td>
<td>1</td>
</tr>
<tr>
<td>Dimensionless bearing capacity factor $N_q$</td>
<td>40</td>
</tr>
</tbody>
</table>
Equivalent soil spring model

In the equivalent soil spring model construction, the soil-pile interaction was simplified to four horizontal and one vertical soil spring working at the four bottom joints of the truss template. The number of soil springs was reduced to 20 which increased the speed of the simulation. The load-displacement relationships of the equivalent-horizontal and equivalent-vertical soil springs were obtained by applying horizontal and vertical force at the top of the pile, respectively. The equivalent soil spring model is shown in Fig. 7.

Geometry of support structure and piles

Our prototype of the OWT structure followed Passon and Branner (2014). A detailed tower model was used as a simplified representation of the superstructure. Note that the Rotor–nacelle assembly (RNA) is equivalent to a mass on the top of the tower as shown in Fig. 1(a). In principle, if the pile spacing is less than eight times its diameter, group effects have to be taken into account. The diameter of the pile is 1.829 m, and the distance between the piles equals 20 m, which is over 10 times the diameter, therefore, the group effect (i.e., pile–soil–pile interaction) is ignored. Our model of the OWT structure is shown in Fig. 1(b). The details of the size and the parameters can be found in the research conducted by Passon and Branner (2014).

CONSTITUTIVE MODELS

In this research, the material used to simulate the superstructure of the OWT was steel. The turbine structure was treated as a linear elastic model. The Young’s modulus of steel equals to 2 ×10^8 kPa, and the soil was plastic-elastic. The Mohr-Coulomb plasticity model was used in connection with the classical Mohr-Coloumb yield criterion, which assumes that the failure of soil material is controlled by the maximum shear stress. The failure model can be defined as Eq. (15) and Eq. (16).

Mohr Coulomb plasticity

In the analysis of the soil-pile interaction, the pile was elastic, while the soil was plastic-elastic. The Mohr-Coulomb plasticity model was used in connection with the classical Mohr-Coloumb yield criterion, which assumes that the failure of soil material is controlled by the maximum shear stress. The failure model can be defined as Eq. (15) and Eq. (16).

Soil material damping

In order to simulate the dynamic response of the soil material, a viscous damping – called Rayleigh Damping – had to be added to the system. The damping matrix [C] of Rayleigh Damping is proportional to a linear combination of the mass matrix [M] and the stiffness matrix [K], Eq. (13).

\[
[C] = \alpha [M] + \beta [K] 
\]

where \(\alpha\) is the mass-proportional damping coefficient, and \(\beta\) is the stiffness-proportional damping coefficient. Also, the relationship between the modal equations and the orthogonality conditions allows the natural frequency and the damping ratio \(\xi_i\) to be connected as in Eq. (14).

\[
\xi_i = \frac{\alpha}{2\omega_i} + \frac{\beta\omega_i}{2} 
\]

where \(\omega\) is the natural frequency of the system, the subscript \(i\) means the \(i\)-th modal of the system. The damping coefficients \(\alpha\) and \(\beta\) can be obtained by solving Eq. (13), except for the problem with the first two low modal mass proportions. Therefore, \(\alpha\) and \(\beta\) are obtained by the least squares method, using the first 100 natural frequencies of the soil obtained from modal analysis. The effective modal mass of the first 100 modal is greater than 90% of the total mass of the soil.

Table. 4 Material properties

<table>
<thead>
<tr>
<th>Young’s Modulus E</th>
<th>Unit Weight γ</th>
<th>Poisson’s Ratio ν</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>2 ×10^8 kPa</td>
<td>8 tf/m^3</td>
</tr>
<tr>
<td>Soil</td>
<td>30100 kPa</td>
<td>1.75 tf/m^3</td>
</tr>
</tbody>
</table>

Mohr Coulomb Plastic Coefficients

<table>
<thead>
<tr>
<th>Friction Angle Φ</th>
<th>Dilatation Angle ψ</th>
<th>Cohesion c</th>
<th>Abs Plastic Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>31.5</td>
<td>27.6</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Here \(\phi\) is the slope of the Mohr-Coulomb yield surface in the stress plane of \(p-R_{ncf}\) also referred to as the frictional angle of the material, \(\Theta\) is the deviatoric polar angle, \(p\) is the equivalent pressure stress, and \(q\) is the Mises equivalent stress. However, since the yield surface of the Mohr–Coulomb model has corners, it is sometimes inconvenient to use the original model to determine the direction of the plastic flow. Instead, we used a smooth elliptic approach proposed by Menetrey and Willam in Eq. (17) and Eq. (18).


\[ G = \sqrt{\left(\varepsilon \cdot e \cdot \tan \varphi\right)^2 + \left(R_{mod}\right)^2} - p \tan \varphi \]  

(17)

\[ R_{m0} = \frac{4(1-e^{-2}) \cos \Theta + (2e-1)^2}{2(1-e^{-2}) \cos \Theta + (2e-1)^2} \tan \left(\frac{\pi}{3} \varphi\right) \]  

(18)

Here, \( c_b \) is the initial cohesion yield stress; \( \varepsilon \) is the meridional eccentricity, and \( e \) is the deviatoric eccentricity which equals to 3-sin\( \varepsilon \)/3+sin\( \varepsilon \). The details of the description of the parameters are shown in Abaqus Analysis User’s Manuals (2014). The advantage of using the approach proposed by Menetrey and Willam rather than the original Mohr-Coloumb model is that it helps the possibility of divergence during calculations.

**Boundary condition and contact algorithm**

In this study, the superstructure and the piles of the OWT structure were simulated by different types of elements, the truss template was modeled by beam elements, and the four piles were modeled by an eight-node brick element. In order to connect these two parts, a constraint was used to couple the reaction at the four joints with the displacements along the x, y, z directions.

There are differences between the settings of the boundary condition for the finite element model and for the hybrid finite-infinite element model. In order to better simulate the site condition, we limited the displacements at the bottom of the soil body with boundary conditions \( u=v=w=0 \), and the artificial boundary conditions \( u=v=0 \), respectively. Since the infinite element is able to simulate the far field condition of the soil body, it can be referred to as a boundary condition. Therefore, there is no need to set extra boundary conditions on the infinite element boundary. The only boundary condition in this model is on the bottom of the soil body of finite element, i.e., \( u=v=w=0 \).

In general, both normal and tangential contacts occur on the contact surface. We used the so-called hard contact in Abaqus with constraint enforcement. When the gap between two surfaces disappears, they come into contact, therefore, pressure and friction can be transmitted between them. When the friction exceeds \( \tau_{c,n0} \), which equals \( \mu_p \) relative displacement between the two surfaces. The value of the frictional coefficient can be obtained through \( \mu = \tan(\frac{\pi}{3} \varphi') \), and \( \varphi' \) is the friction angle of soil which is the same value determined in the soil spring model.

**SIMULATION RESULTS**

**Comparison of finite element model, Infinite element model and existing p-y curve**

We used static analysis to study the behavior of a single pile subjected to monotonic lateral loading at the top of the pile. The actual input values were 1 MN and 4 MN. The soil-pile interaction in the finite element model, hybrid finite-infinite element model, and the soil spring model were evaluated in Fig. 8. The finite element model and the hybrid finite-infinite element model showed similar displacement-depth relationships on the top of the pile. We can interpret this as justification to use the hybrid finite-infinite element model as well.

However, the depth-displacement relationships in Fig. 8 showed discrepancies between the soil spring model and the hybrid finite-infinite element model. Namely, the displacements calculated by the soil spring model based on the API guideline were smaller than those of the hybrid finite-infinite element model. The cause is that the initial slope of the \( p \)-\( y \) curve based on the API guideline was overestimated (Pitilakis 2007 and Hearn et al. 2010). The \( p \)-\( y \) curve of the hybrid finite-infinite element model incorporated less stiffness than the API curve, which followed a perfectly plastic behavior. The comparison between the different \( p \)-\( y \) curves is shown in Fig. 9 (a) and (b). Similar phenomena were observed by Hearn et al. (2010), McGann et al. (2011) and Tak Kim et al. (2004), in which they compared three existing \( p \)-\( y \) curves with their testing model, including the API curve, Reese's \( p \)-\( y \) curve (1974) and Wesselink's \( p \)-\( y \) curve (1988). The results are similar to those observed in Fig. 9.

Therefore, it is reasonable that the displacement of the single pile subjected to a monotonic lateral loading in the hybrid finite-infinite element model is larger than that of the soil spring model based on the API guidelines.

**Natural frequency of OWT**

Natural frequency analysis is indispensable for the design of the OWT structure to help prevent resonance. At the time of an earthquake, resonance causes great damage to the OWT. We obtained values of the natural frequency from the hybrid finite-infinite element model, the soil spring model, and the equivalent soil spring model, the comparison is shown in Table. 8. Besides, in the hybrid finite-infinite element model, the frequency response functions with damping ratios of 0.01, 0.03 and 0.05 were obtained and compared.

In the hybrid finite-infinite element model, the natural frequency
obtained from the modal analysis is not truly the natural frequency of the OWT structure, because the soil and the OWT structure are considered together. The frequency response function is used to solve this problem by applying the sinusoidal loading at the top of the structure with different frequencies. Sinusoidal loading with amplitudes of 1 kN and 10 kN were applied at the top of the structure between time=0 sec and time =30 sec, with different frequencies. The results are shown in Fig. 10 (a) and (b). The displacements of the OWT structure were obtained from the maximum displacement between time=20 sec and 30 sec, when the OWT structure entered the steady state. In the resonant case – since there is no structural damping included in the model –, the displacement would become infinite. Therefore, the maximum displacement was also obtained from the maximum simulated data between time=20 sec and 30 sec. The natural frequency of the OWT obtained from the hybrid finite-infinite element model is 0.3 Hz, the period is 3.3 sec, and there is no difference between the natural frequency estimations obtained with different damping ratios, the natural frequency obtained from different models are shown in Table. 5. The results of the modal analysis of the soil spring model show that the frequency of the first mode is 0.336 Hz, which is close to the natural frequency obtained from the hybrid finite-infinite element model. In the case of shallow earthquake, the frequency is ranged from 5 Hz to 20 Hz, has a higher probability to cause damage to structure comparing with the deep earthquake with the frequency range from 0.2 Hz to 3Hz. Therefore, the result shows that the natural frequency of the OWT structure is acceptable.

<table>
<thead>
<tr>
<th>Table. 5 Natural frequency obtained from different models</th>
</tr>
</thead>
<tbody>
<tr>
<td>soil spring model</td>
</tr>
<tr>
<td>Natural frequency</td>
</tr>
<tr>
<td>Calculating time</td>
</tr>
</tbody>
</table>

Effect of equivalent springs

In order to increase the efficiency of the calculations and the simulation of the follow-up analysis, in the equivalent soil spring model, 20 equivalent nonlinear soil springs were used to replace the complicated soil-pile interaction modeled by 1444 soil springs in the original soil spring model. In this model, four horizontal and one vertical soil springs were set up at each of the four joints located at the bottom of the OWT structure. The stiffness of the equivalent-horizontal and the equivalent-vertical soil springs were obtained from the load displacement relationship by applying horizontal and vertical load at the bottom four joints of the original soil spring model. The stiffness of the equivalent-vertical and equivalent-horizontal soil springs are shown in Fig. 11. The natural frequency obtained from the equivalent soil spring model is 0.334 Hz, which shows good agreement with the results from the hybrid finite-infinite element model and the original soil spring model.

Comparing the operational speeds, we found the calculating time of the equivalent soil spring model as short as 7 secs for the natural frequency analysis, about 5 minutes for the original soil spring model, and 5.3 hours for the hybrid finite-infinite element method – for only one point in the frequency response function. This advantage of the equivalent soil spring model is beneficial to the follow up research when considering different environmental load combinations and earthquakes.

CONCLUSION

The soil-pile interaction plays a significant role in structural response. With the simulation of the soil-pile interaction, the natural frequency obtained is accurate and reliable. In the hybrid finite-infinite element model, with the use of infinite element to the semi-infinite
domain problems and the consideration of the initial geostatic balance, a comprehensive procedure for the static and dynamic analysis of the OWT structure was established. Also, good agreements were obtained for the natural frequencies using the hybrid finite-infinite element model, the soil spring model based on the API guideline, and the equivalent soil spring model. The natural frequency of the OWT structure was estimated as 0.3 Hz, 0.336 Hz, and 0.334 Hz, respectively. The difference between the natural frequency estimations was attributed to the higher initial stiffness of the soil spring obtained from the empirical API equation.

In addition, the equivalent soil spring model offers the following advantages compared to the hybrid finite-infinite element model: a more comprehensive analysis which is important in the problems considering earthquake. Future work, following this study, should take into account the more comprehensive site condition such as the soil properties and environmental loads. Also, the dynamic soil-pile interaction would need to be considered if the seismic responses of the OWT structure are analyzed.

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