

Vibration Characteristics of Offshore Wind Turbine Tower with Caisson Foundation under Wave Excitation

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ABSTRACT

In this study, the vibration characteristics of offshore wind turbine tower with caisson foundation are numerically analyzed under wave excitation. To achieve the objective, the following approaches are conducted. Firstly, the frequency domain decomposition and the stochastic subspace identification methods are selected to extract modal parameters from the structural vibration responses. Secondly, a numerical example of the target structure including a wind turbine tower, a caisson foundation and a foundation bed is simulated using a FEM software, Midas FEA. For wave-induced vibration analysis, a wave field is modelled in the Flow-3D program to extract wave pressures acting on the caisson foundation. Finally, the modal parameters extracted from wave-induced vibration analysis are compared to those from the eigenvalue analysis. Several wave conditions are introduced for the vibration analysis to investigate the change of modal parameters.

1. INTRODUCTION

For offshore wind turbine tower structures, the gravity-type caisson foundation can be a choice dependent on the site condition and the economical construction. During the lifetime, the foundation bed under caisson foundation can be damaged due to the sediment transport and scour phenomenon (Petersen *et al.* 2015). Meanwhile, the wind turbine tower can suffer from serious damages such as cracks or bolt loosening owing to the fact that the slender vertical wind tower exposes to extreme weather conditions. Thus, the dynamic characteristics and the structural health monitoring (SHM) of the offshore wind turbine tower with caisson are clearly essential studies.

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Many researchers have studied the SHM techniques in the civil engineering field by mainly measuring vibration responses of civil structures (Cawley *et al.* 1979), since the late 1970s. This is because vibration responses and structural characteristics alter along with the change of structural mass, stiffness and damping factor (Kim *et al.* 1995, Kim *et al.* 2003, Lee *et al.* 2015). Regarding the inland wind turbine tower, some researches attempt to detect damage in a numerical model via vibration-based damage detection methods and hybrid method (Nguyen *et al.* 2015, Nguyen *et al.* 2016). Regarding the gravity-type caisson breakwater structure, there are several studies about the vibration characteristics with water-level variation and damage detection of foundation (Lee *et al.* 2015, Lee *et al.* 2013). Regarding offshore wind turbine structure, there are several researches on seismic performance and wind-induced response of this structure supported by monopile foundation. Some researchers study about wave-induced and wind-induced dynamic response of the structure supported by jacket foundation. Regarding the offshore wind turbine with caisson foundation, there are research needs to understand the dynamic characteristics of the offshore wind turbine tower with caisson foundation as an integrated system.

In this paper, the following approaches are employed. Firstly, the frequency domain decomposition and the stochastic subspace identification methods are presented to identify modal parameters of the target structure. Secondly, a numerical example of the target structure is simulated using a FEM software, Midas FEA. A nonlinear regular wave field is simulated in the Flow 3D program to measure wave pressures acting on the caisson foundation for wave-induced vibration analysis. Finally, dynamic characteristics of the target structure from the eigenvalue analysis and the several wave-induced vibration analyses are investigated.

2. OPERATIONAL MODAL ANALYSIS METHODS

The frequency domain decomposition (FDD) method is a technique that decomposes the spectral density function matrix and generates a set of single degree of freedom systems from the response (Brinker *et al.* 2000). The PSD matrix $\mathbf{S}_{yy}(\omega)$, which can be constructed by cross-spectrums for all measured accelerations, is decomposed by using singular value decomposition (SVD) algorithm as Eq. (1).

$$\mathbf{S}_{yy}(\omega) = \mathbf{U}(\omega)^T \mathbf{\Sigma}(\omega) \mathbf{V}(\omega) \quad (1)$$

where $\mathbf{\Sigma}(\omega)$ is a diagonal matrix containing the singular values $\sigma_i(\omega)$ ($i = 1, 2, \dots, n$) of its PSD matrices, $\mathbf{U}(\omega)$ and $\mathbf{V}(\omega)$ are unitary matrices. When peak frequencies in the first singular values $\sigma_i(\omega)$ are found, the mode shapes are extracted from anyone of column vectors of $\mathbf{U}(\omega)$ at the corresponding peak frequencies.

Lee *et al.* 2018 summarized the stochastic subspace identification (SSI) method into following five fundamental steps based on several existing studies. In the first step, the cross-correlation matrices are calculated from the measured time signals. In step 2, the Hankel matrix $[\mathbf{H}]$ is constructed from the obtained correlation matrices. In step 3, the invertible weighting matrices \mathbf{W}_1 and \mathbf{W}_2 are pre- and post-multiplied to the Hankel matrix. Then, this matrix is decomposed into the observability \mathbf{O}_{n1} and system matrix \mathbf{A} .

$$\mathbf{A}\Psi = \Psi\mathbf{M} \quad (2)$$

$$(\mathbf{M} = \text{diag}(\mu_1, \mu_2, \dots, \mu_N) \in \mathbf{R}^{N \times N}, \Psi = [\psi_1, \psi_2, \dots, \psi_N] \in \mathbf{R}^{N \times N})$$

In step 4, the eigenvalues μ and eigenvectors ψ of the system are computed by decomposing the system matrix A as Eq. (2). In step 5, several criteria are applied to classify a stable mode, unstable mode and noise mode. Then a proper system order is decided via the stabilization chart.

3. NUMERICAL MODELLING OF OFFSHORE WIND TURBINE TOWER WITH CAISSON FOUNDATION

As shown in Fig. 1(a), the design of wind turbine tower is based on an onshore wind turbine tower, which is located in Hankyung II Wind Park, Jeju Island (Nguyen *et al.* 2015). The nacelle and rotor weigh 68 and 39.8 ton respectively. The tower structure has thickness varying from 40 mm to 16 mm and a total height of 77.3 m. The top diameter is 2.316 m and the bottom diameter is 4.150 m. The geometry of the caisson foundation is roughly scaled down with a factor of 0.75 from an example design of Daewoo E&C offshore wind turbine supporting structure (Daewoo E&C 2017). The foundation consists of a tube part (length of 17.3 m, diameter of 4.15 m) and a conical part (length of 17.25 m, bottom diameter of 11.5 m and slab thickness of 1.4 m). The caisson thickness is 0.5 m. The design of foundation bed is based on the Thornton Bank Offshore Wind Farm (Peire *et al.* 2008). The bed is composed of four layers, namely scour protection, backfill, gravel and filter layer. The diameter of the foundation bed is selected as 92 m. The thickness of the scour protection layer and the backfill layer are 1.3 m and 6.3 m respectively; the gravel and filter layers are altogether 2 m thick. The foundation bed is supported by the natural ground of 13 m thick. The material properties are listed in Table 1.

As shown in Fig. 1(b), the tower is modeled by shell elements. On the top, the rotor and nacelle are simplified as lump masses. The caisson foundation is modeled using shell elements for the cover and solid elements for the sand fill. All layers of the foundation bed (except scour protection layer) and the natural ground are simulated using solid elements. The scour protection layer is considered as added masses. Regarding boundary condition, the elastic compressive and shear coefficient of sand $C_z = 7.5 \text{ kg/cm/cm}^2$ and $C_{x,y} = 3.75 \text{ kg/cm/cm}^2$ are selected (Barkan 1962). The added mass of seawater is calculated based on Eq. (3) (Westergaard 1933). The M_w is the hydrodynamic mass, ρ_w is the seawater density as 1027 kg/m^3 , H_w and h are the depth from the water level to the foundation and the water level to the action point of hydrodynamic pressure, respectively.

$$M_w = \int_{h_1}^{h_2} \frac{7}{8} \rho_w \sqrt{H_w h} \, dh \quad (3)$$

The numerical model of wave field surrounding the target structure is established using Flow 3D program. As schematized in Fig. 2(a), the wave field has the following dimensions: 1500 m in length, 70 m in width, and 18 m in height. The still water level is 13.69 m above the foundation bed. The incident waves propagate in the X direction

(along-wave direction) and are normal to the target structure, which locates at 70 m far from the wave generation source. The dimensions of the wave field are determined so that the influence of reflecting waves could be minimized. Regarding boundary condition, the min-X plane is the plane of wave generation while the max-X plane is the outflow of the domain. The min-Z plane is set to be wall as the representation of seabed. The remaining planes are defined as symmetry condition. The wave data observed near Jeju Island (Korea) is used as the input for the wave field (KHOA 2017) as in Table 2. The wave pressure in front of caisson (depth -2.2 m) is shown in Fig. 2(b).

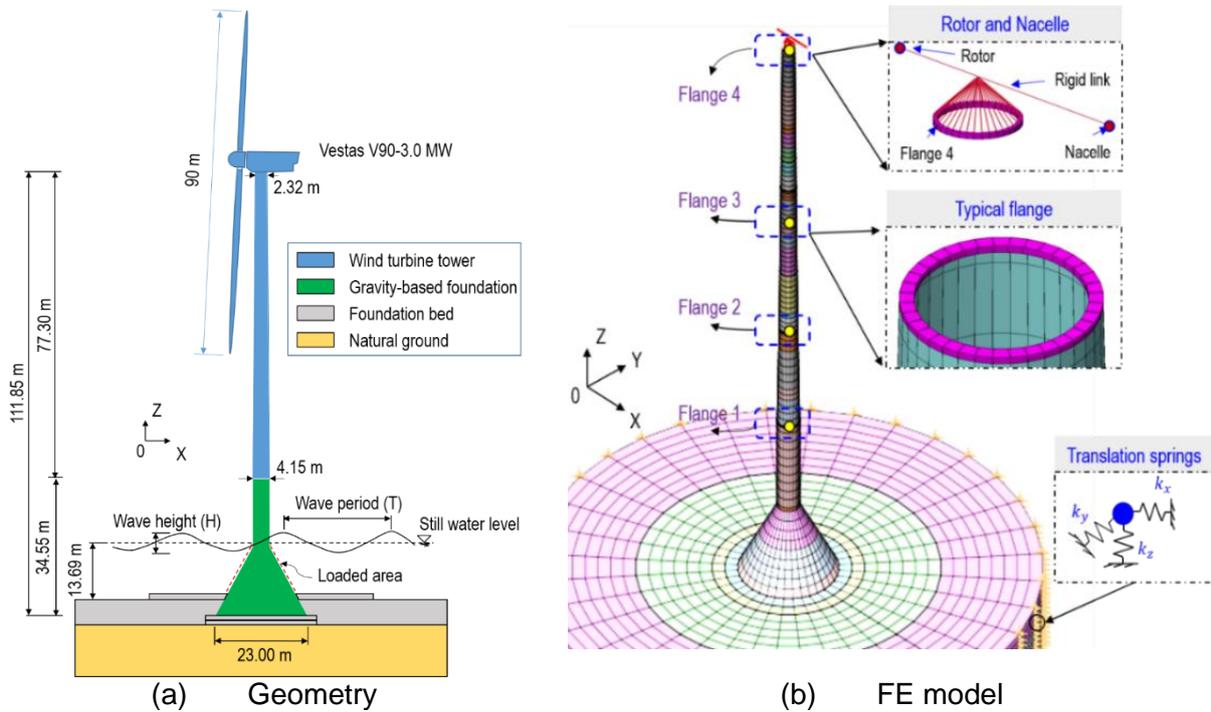


Fig. 1 Wind turbine tower with caisson foundation

Table 1 Material properties of target structure

Component	Material	Modulus of elasticity, E (MPa)	Poisson's ratio, ν	Mass density, ρ (kg/m ³)
Wind turbine tower	Steel	2.10E+05	0.3	7698
Caisson foundation	Concrete	3.35E+04	0.2	2500
	Sand fill	66.5	0.325	1620
Foundation bed	Scour Protection Layer	-	-	1800
	Backfill Layer	66.5	0.325	1620
	Gravel Layer	140	0.3	1500
	Filter Layer	140	0.3	2100
Natural ground	Sand Layer	66.5	0.325	1620

Table 2 Wave properties at 32°N-127°E near Jeju Island, Korea

Cases	Observation time	Type	Wave Height (m)	Wave Period (s)
Wave 1	1 st Jan 2017	Significant	0.5	6
Wave 2	3 rd quarter of 2017	Significant	1.34	6.1
Wave 3	5 th Jan 2017	Max	2.8	5.5
Wave 4	9 th Jan 2017	Max	4.6	6.5

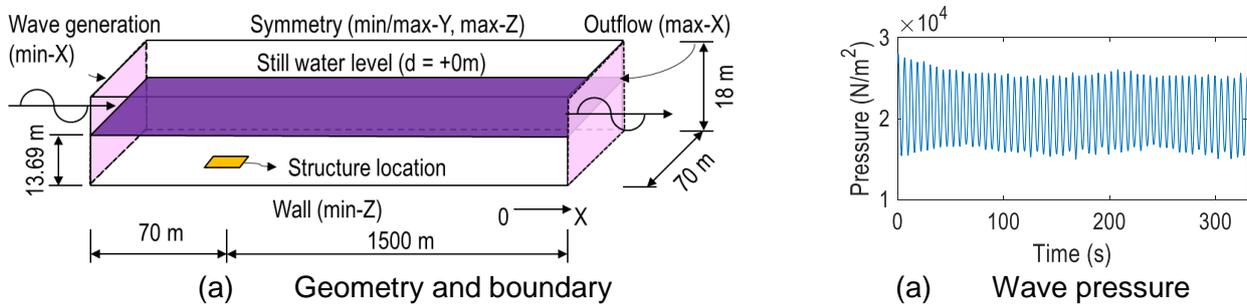


Fig. 2 Simulation of wave field

The vibration response of the target structure is recorded from 11 sensors, which are equally distributed on the wind turbine tower. The measuring period and sampling rate are 330 s and 50 Hz, respectively. The acceleration signals at sensor 7 under wave 2 are illustrated as examples. The max value of acceleration signal along-wave direction at sensor 7 is 0.076 m/s² within 5 s from the time when wave approaches the caisson foundation as shown in Fig. 3 (a). Meanwhile, the maximum value of acceleration signal across-wave direction at sensor 7 is only 0.021 m/s² within first 10 s as shown in Fig. 3 (b).

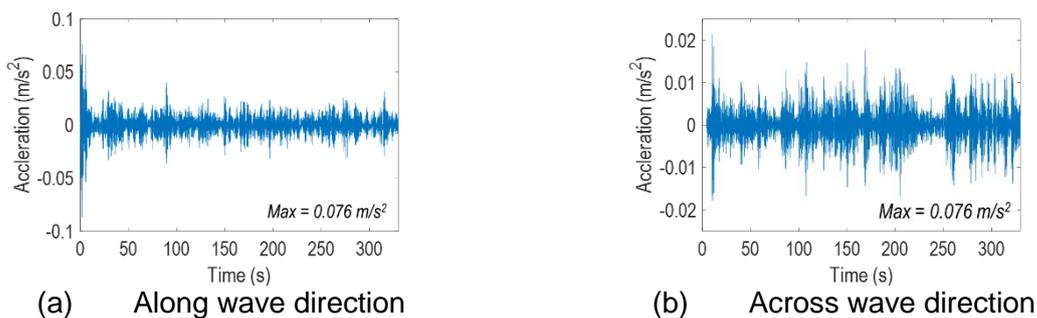
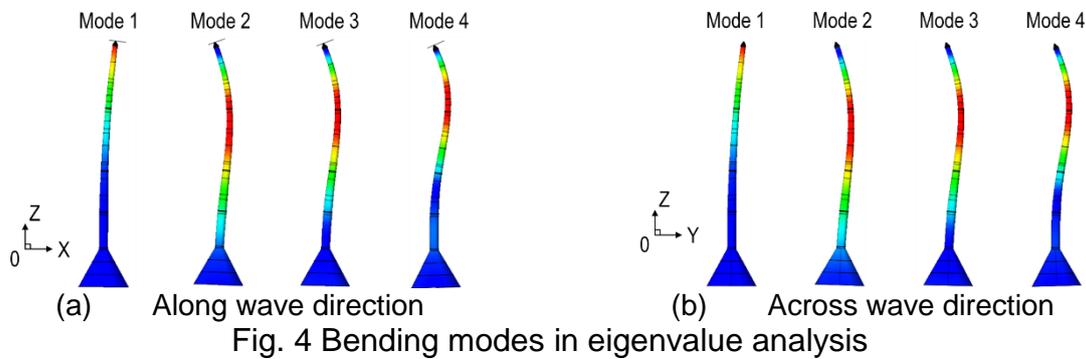


Fig. 3 Acceleration signals at sensor 7 under wave 2

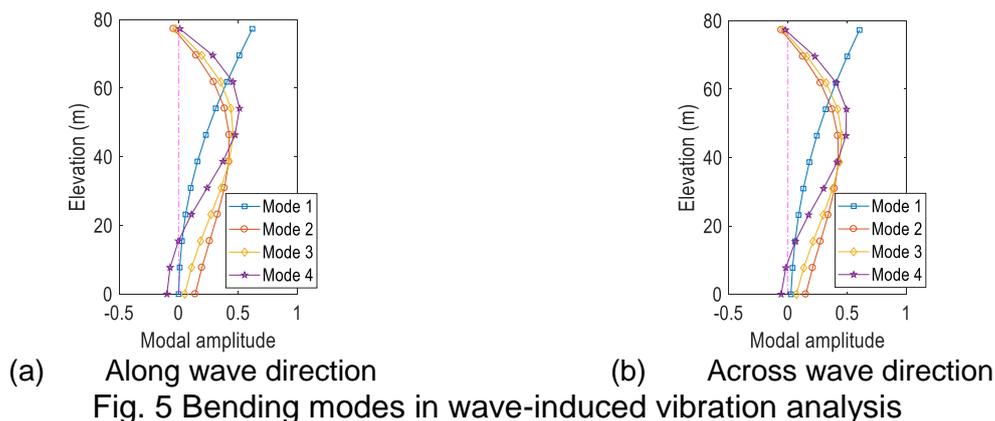
4. VIBRATION CHARACTERISTICS OF OFFSHORE WIND TURBINE TOWER WITH CAISSON FOUNDATION

In the eigenvalue analysis, regarding along-wave bending modes, the first four natural frequencies are 0.2830 Hz, 1.5619 Hz, 1.8495 Hz and 2.4006 Hz respectively. In the first mode, the most rigorous vibration area is in the top of wind turbine tower as shown in Fig. 4(a). Meanwhile the area in two-third elevation of wind turbine tower has the largest modal amplitude among other three modes. It is noted that the first two modes of wind turbine tower have the distinct mode shapes while the remaining ones

are similar to the second mode. This can be due to the effect of foundation vibration. Regarding across-wave bending modes, the first four natural frequencies are 0.2840 Hz, 1.5897 Hz, 1.8820 Hz and 2.4576 Hz respectively. It is also noteworthy that the natural frequency from across-wave direction in each mode is always higher than that from along-wave direction. The first four mode shapes of target structure are similar to those from previous direction as shown in Fig. 4.



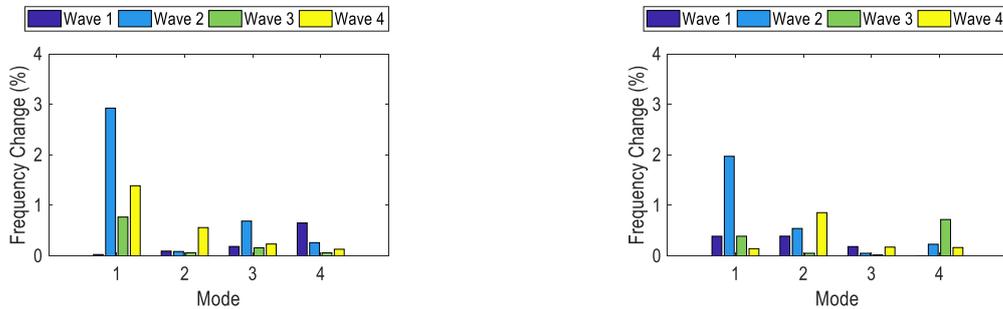
In the wave-induced vibration analysis, the vibration characteristics of target structure are identified via the FDD and SSI method. Regarding along-wave bending modes, the first four natural frequency along wave direction are 0.2933 Hz, 1.5606 Hz, 1.8368 Hz and 2.4069 Hz. These extracted natural frequencies match those from the eigenvalue analysis under 4%. It is obtained that the gap between natural frequencies from the eigenvalue and vibration analysis differs largest in the first mode.



Regarding across-wave bending modes, the first four natural frequencies across wave are 0.2784 Hz, 1.5983 Hz, 1.8829 Hz and 2.4632 Hz. These extracted natural frequencies match those from the eigenvalue analysis under 3%. It is obtained that the gap between natural frequencies from the eigenvalue and vibration analysis differs largest in the first mode. As shown in Fig. 5, the corresponding mode shapes in both direction are similar to those from the eigenvalue analysis.

All waves in Table 2 are employed for the vibration analysis to examine the influence of wave condition on modal parameters. The relative variation between natural frequencies from the eigenvalue and wave-induced vibration analysis are calculated.

As shown in Fig. 6, the first natural frequency varies largely compared to others because the frequencies of incident waves are similar to the first natural frequency. Therefore, if the frequency wave is near the structural natural frequency, it has influence on the modal parameters of that mode. This leads to a situation that structural damages can be hidden from the variation of coming wave in field practices.



(a) Bending modes along wave (b) Bending modes cross wave
 Fig. 6 Comparison of natural frequencies: eigenvalue analysis and wave-induced vibration analyses

5. CONCLUSION

In this paper, the vibration characteristics of offshore wind turbine tower with caisson foundation are numerically analyzed under wave excitation. To achieve the objective, the following approaches were conducted. Firstly, the frequency domain decomposition and stochastic subspace identification methods were presented to identify modal parameters of the target structure. Secondly, the numerical example of the target structure was simulated in the structural analysis program, Midas FEA. The wave field was simulated in the Flow 3D program to measure wave pressures acting on the caisson foundation. Finally, dynamic characteristics of the target structure from the eigenvalue analysis and the several wave-induced vibration analyses were investigated. The following observations are concluded. Firstly, the vibration of foundation bed has influence on the mode shape of target structure. Secondly, if the frequency of incident waves is similar to the fundamental frequency of the offshore wind turbine tower with caisson foundation, the damage-induced frequency variation can be overshadowed by the wave-induced frequency variation.

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