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# Structural optimization of offshore wind turbine towers under stochastic environmental loads

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**Abstract.** National and international reports on wind energy consistently highlight Brazil's wind power sector as poised for exponential growth in the coming years. In this context, the structural optimization of offshore wind turbine towers presents a promising strategy for reducing costs associated with the commissioning of offshore wind farms, many of which are currently awaiting environmental licensing from IBAMA. Offshore wind turbine towers are subjected to stochastic dynamic loads from wind and wave actions, and accurately modeling these loads is crucial to capturing their inherent variability and avoid underestimating their structural impact. This study presents a structural optimization methodology for offshore wind turbine towers, integrating the Finite Element Method (FEM) and a genetic algorithm, while accounting for the effects of stochastic environmental loads. A numerical model of a 5 MW steel monopile offshore wind turbine (OWT), with a total height of 77.6 meters, was developed, considering soil–structure interaction, the non-deterministic nature of wind and ocean dynamic loads, and aerodynamic forces from the rotor. The optimization results revealed a 21.8% reduction in the total steel volume of the support structure, ensuring adherence to international design standards for offshore wind towers, marking a significant contribution to cost-efficiency in offshore wind turbine development.

**Keywords:** Offshore Wind Turbine Towers, Stochastic Environmental Loads, Finite Element Modelling, structural optimization.

## 1. Introduction

The construction of steel towers for wind turbine support represents a significant part of the total cost in wind farm installations [1]. Specifically, Gentils *et al*. [2] report that these structures account for approximately 45% of the total cost of an offshore wind turbine. Consequently, improving the structural design of offshore wind towers can significantly reduce construction costs, enhance the economic competitiveness of offshore wind energy and promote faster development. Furthermore, offshore wind turbines are continuously exposed to random dynamic loads from wind and wave actions, which presents a major challenge in accurately assessing their impact on the structural behavior of these towers. Therefore, a comprehensive understanding and precise modeling of these stochastic phenomena are essential to ensure structural integrity and performance, with robust structural optimization accounting for uncertainties associated with these forces.

This study presents the structural optimization of an offshore wind turbine tower supporting a 5 MW turbine, using the Finite Element Method (FEM) and genetic algorithms. The goal is to reduce the steel needed for the support structure, thereby significantly lowering commissioning costs for offshore wind farms. The numerical model accounts for the non-deterministic effects of wind and sea loads on the structure, as well as soil-structure interaction. Based on the process results, a statistical analysis is conducted to obtain an optimized structural design with a reduced volume, ensuring a confidence level of 95%.

## 2. Offshore wind tower finite element model development

The development of the finite element model for the offshore wind tower analyzed in this study is based on the characteristics provided by the National Renewable Energy Laboratory (NREL) [3] for a 5 MW turbine. The support structure consists of three sections: a 56-meter monopile foundation with a constant diameter of 6 meters and a 60 mm wall thickness, including a 20-meter section immersed in seawater and the remaining 36 meters embedded in the seabed; a 10-meter transition piece with a constant diameter of 6 meters, tapering from 60 mm at the bottom to 27 mm at the top; and a 76.6-meter wind tower, with tapering diameters from 6 meters to 3.84 meters and wall thicknesses from 27 mm to 19 mm (see Table 1). The structure is modeled using S355 grade steel, with a yield strength ($\sigma_y$) of 350 MPa, Young's modulus (E) of 210 GPa, and Poisson's ratio ($\nu$) of 0.38. To account for additional mass from painting, bolts, coatings, and flanges, a modified material density ($\rho$) of 8500 kg/m³ is used, reflecting an 8.3% increase over the nominal density of S355 steel (7850 kg/m³), providing a simplified yet conservative estimate of the total structural mass [4].

Table 1. Support structure dimensions.

| | $d_{low}$ (m) | $d_{up}$(m) | $tk_{low}$(mm) | $tk_{up}$(mm) | h(m) |
|---|---|---|---|---|---|
| Tower | 6 | 3.84 | 27 | 19 | 76.6 |
| Transition | 6 | 6 | 60 | 27 | 10 |
| Monopile | 6 | 6 | 60 | 60 | 56 |

To account for the variability in seabed soil properties, three distinct layers are considered: loose sand, medium sand, and dense sand. The loose sand layer extends 5 m below the seabed, the medium sand layer is 9 m thick and lies beneath the loose sand, and the dense sand layer extends to a total depth of 82 m. All layers have an outer diameter of 120 m to minimize artificial stiffness effects at the model boundary. The soil's characteristics are represented using the Drucker-Prager constitutive model [5], where the yield stress $\sigma_{y,s}$ is calculated as a function of the soil's friction angle $\varphi$ and cohesion c as shown in eq. (1). The Young's modulus, cohesion, friction angle, yield stress, and unit weight for each of the three layers are provided in Table 2 [2].

$$\sigma_{y,s} = \frac{6c\cos\varphi}{\sqrt{3}(3-\sin\varphi)} \tag{1}$$

Table 2. Seabed layer properties.

| Soil Layer | E (MPa) | $\varphi$ (deg) | c (kPa) | $\sigma_{y,s}$ (MPa) |
|---|---|---|---|---|
| Loose Sand | 30 | 33 | 50 | 59.2 |
| Medium Sand | 50 | 35 | 50 | 58.5 |
| Dense Sand | 80 | 38.5 | 50 | 57 |

The finite element model of the investigated structure, depicted in Fig. 1, was developed through the ANSYS software. Discretization techniques were employed to create a regular mesh, ensuring accurate numerical results. The tower, transition piece, and monopile were modeled using four-node linear shell elements (SHELL 181), while eight-node linear solid elements (SOLID 185) were used to represent the three soil layers. The inertial effects of the rotor-nacelle assembly (RNA) were modeled as a point mass located at the top of the steel tower, with assigned mass and moments of inertia about the x, y, and z axes: $I_{xx}$ = 2.35 kg/m², $I_{yy}$ = 2.54 kg/m², and $I_{zz}$ = 4.37 kg/m². This concentrated mass is positioned at x = 0 m, y = 1.92 m, and z = 0.41 m in a local coordinate system centered at the geometric center of the tower's circular section, with its local x, y, and z directions aligned with the global X, Y, and Z directions.

Preliminary modal analysis results showed natural frequencies of 0.245 Hz and 1.350 Hz for the first and second fore-aft modes, and 0.247 Hz and 1.449 Hz for the first and second side-to-side modes of the structure. Comparing these values with those from Jung *et al*. [6], differences of 1.2% were observed for the side-to-side

frequencies and 2.5% for the fore-aft frequencies. Additionally, a comparison with Gentils *et al*. [2] showed differences of less than 1%. These results confirm the accuracy of the developed finite element model.

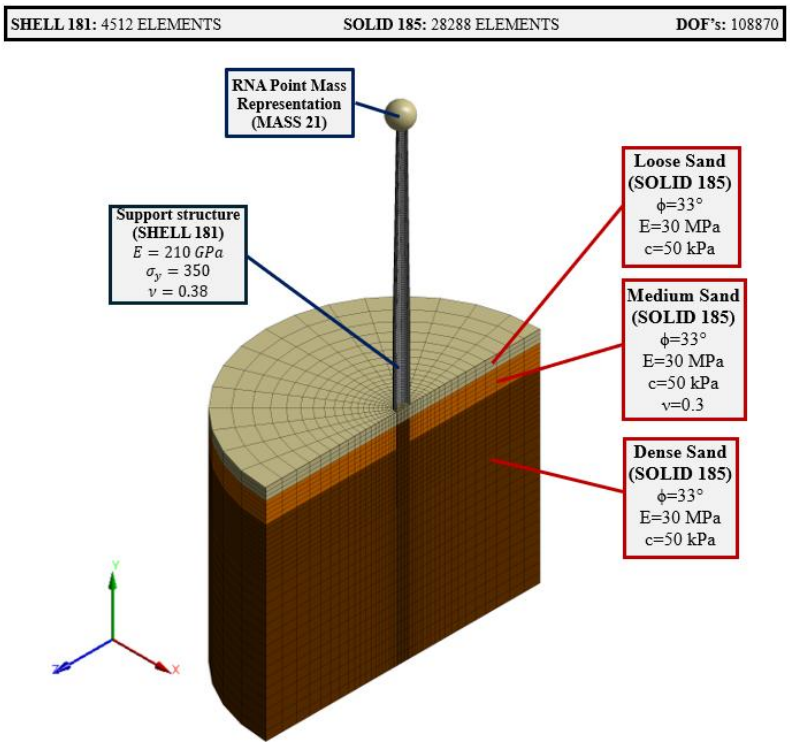



Figure 1. Offshore wind tower finite element model.

## 3. Desing load analysis

Offshore wind turbine towers are usually subjected to combined wind and wave loading. The IEC 61400-3 [7] standard defines various design load cases (DLCs) that account for turbine operation and different environmental conditions. Among the DLCs presented by the design code, DLC 6.1 is recognized as the most critical due to its comprehensive representation of the most severe operational and environmental conditions [2][4]. Thus, DLC 6.1 will be used, considering the wind and sea characteristics detailed in Table 3.

Table 3. Sea and wind conditions.

| Wind condition | | Marine condition | |
|---|---|---|---|
| Reference velocity | $V_{ref}$=35 m/s | 50-Year significant wave height | $H_{50}$=4 m |
| Annual average velocity | $V_{avg}$=10 m/s | 50-Year peak spectral period | $T_{s50}$=7 s |
| 50-Year extreme 3 sec gust wind velocity | $V_{g50}$=50 m/s | Average significant wave height | $H_{avg}$=2 m |
| | | Average wave period | $T_{avg}$=5 s |
| | | 50-Year extreme current velocity | $V_{c,50}$=0.8 m/s |

The numerical model developed in this study incorporates the combined effects of wind and wave loading on an offshore wind turbine supported by a monopile foundation. Specifically, the model accounts for aerodynamic loads on the turbine rotor and tower, and hydrodynamic loads on the monopile foundation. A schematic of the applied loads is presented in Figure 3. Aerodynamic loads on the wind turbine were computed using the FAST [8] code, employing the dynamic blade element momentum (BEM) theory considering airfoil data as outlined in JONKNAM *et al*.[2]. Wind loading on the tower and transition part was modeled using a stochastic approach,

assuming a weak stationary, second-order ergodic process, as presented in Castilho *et al*. [1]. Current and wave loading were computed using the FAST [8] considering the potential flow theory and Pierson-Moskowitz irregular wave spectrum, respectively.

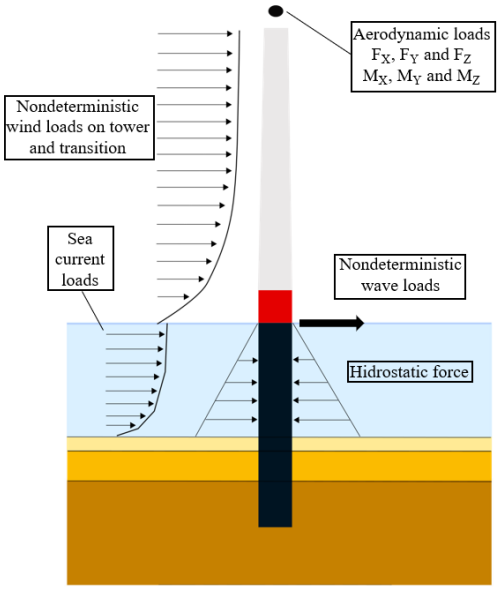



Figure 2. Representation of wind and sea loads on investigated offshore wind tower.

## 4. Structural optimization of offshore wind tower

This study investigates an optimization problem aimed at minimizing the total volume of an offshore wind turbine support structure through genetic algorithms, considering constraints on natural frequency, allowable stress, displacement, and buckling. The design variables considered in this study (see Table 4) are defined as follows: $d_1$ – diameter of the monopile foundation and of the transition piece; $d_2$– top diameter of the steel tower; $t_1$ – constant wall thickness of the monopile and bottom thickness of the transition piece; $t_2$ – top thickness of the transition piece and bottom thickness of the tower; $t_3$ – top thickness of the tower. To construct the design space, the variables are uniformly distributed within the lower and upper limits, with diameters varying by 0.1 m and thickness by 1 mm.

Table 4. Lower and upper bounds for the design variables.

| Variable | $d_1$ (m) | $d_2$ (m) | $t_1$ (mm) | $t_2$ (mm) | $t_3$ (mm) |
|---|---|---|---|---|---|
| Lower bound | 5.5 | 5.5 | 50 | 20 | 15 |
| Upper bound | 6.5 | 6.5 | 70 | 40 | 35 |

Considering the first natural frequency $f_{01}$, it should be higher than the turning rotor induced frequency $f_{1p}$ and blade passing frequency $f_{3p}$. According to DNV-GL [9], the first natural frequency should avoid rotor induced frequencies by 5%. Considering the cut-in and rated rotor speed of the NREL [2] 5 MW wind turbines are equal to 6.9 rpm and 12.1 rpm; the natural frequency constraint is given by:

$$0.212 \leq f_{01} \leq 0.328 \text{ Hz} \tag{2}$$

The allowable stress must be established with respect to the ultimate limit state, considering the maximum von Mises stress $\sigma_{M,\,max}$ experienced by the structure, as shown in Eq. (3), where $\gamma_m$=1.1 and $\gamma_f$=1 are the partial safety factors for material and consequence of failure according to IEC 61400-1 [10] .

$$\sigma_{M,max} \leq \frac{\sigma_y}{\gamma_m \gamma_f} = 323 \text{ MPa} \tag{3}$$

Displacement constraints are applied to the tower top $d_{max}$ and monopile foundation rotation at the seabed level $\theta_{max}$. Following DNV-OS [9] criteria, these displacement limits are given by eqs. (4) and (5), where $H_T$ is the distance from the seabed to the tower top.

$$d_{max} \leq 1.25 \frac{H_T}{100} = 1.345 \text{ m} \tag{4}$$

$$\theta_{max} \leq 0.25° \tag{5}$$

According to DNVGL [9], structural failure due to buckling is prevented if the first multiplier $L_m$ of the structure is below a reference value $L_{m,allow}$, which is taken based on the structure's nature. Considering $L_{m,allow} = 1.4$, the buckling constraint is given by:

$$L_m \leq L_{m,allow} \tag{6}$$

## 5. Results and discussion

To achieve an initial understanding of the structural dynamic behavior of the offshore wind turbine tower, a transient analysis was performed using the original design described in Section 2, subjected to a random combined wind and wave dynamic loads as described in Section 3. Figure 3 illustrates the structural dynamic response to a random series of wind and wave loads. The displacement at the tower top, rotation at the base, and maximum stress all remain below the limits established in the previous section. The maximum horizontal displacement was 20.4% below the limit specified by DNV-OS [9], while both rotation and stress values were approximately 10% of the limits defined by DNV-OS [9] and IEC 61400-1 [10], respectively.

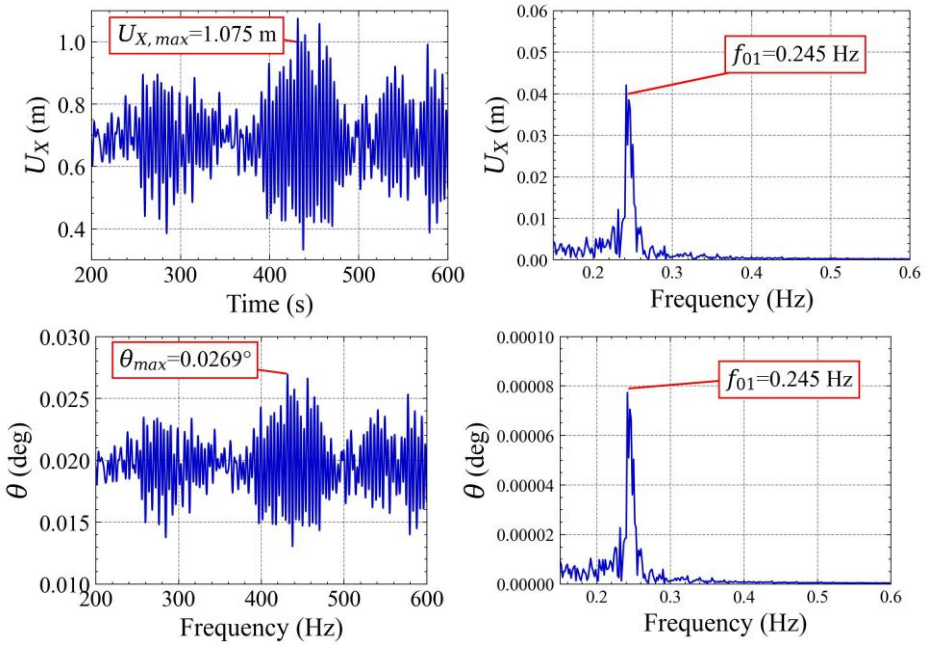



Figure 3. Top displacement and bottom rotation in both time and frequency domain.

Considering the response in the frequency domain, it is evident that the 0.245 Hz frequency has the greatest influence on the dynamic response of the offshore wind tower. Additionally, a buckling analysis yields a load amplification factor of 3.92, indicating that the initial structural design meets the design criteria considered.

The optimization process is carried out considering six random wind and sea combined loading series [7]. Figure 4 presents a detailed overview of all the optimization process steps considered in this study. For each of

these series, the optimization process based on genetic algorithms is performed with 20 generations, 100 individuals, and a mutation probability of 10%, aiming to obtain the design variables that result in the minimum volume for the offshore wind tower. Upon completion of all six random loading series, a statistical analysis of the structural responses, as well as the optimized volume of the wind tower, is conducted, considering a 95% confidence level. This framework aims to achieve an optimized and robust structural design that accounts for uncertainties in modeling wind and sea loads.

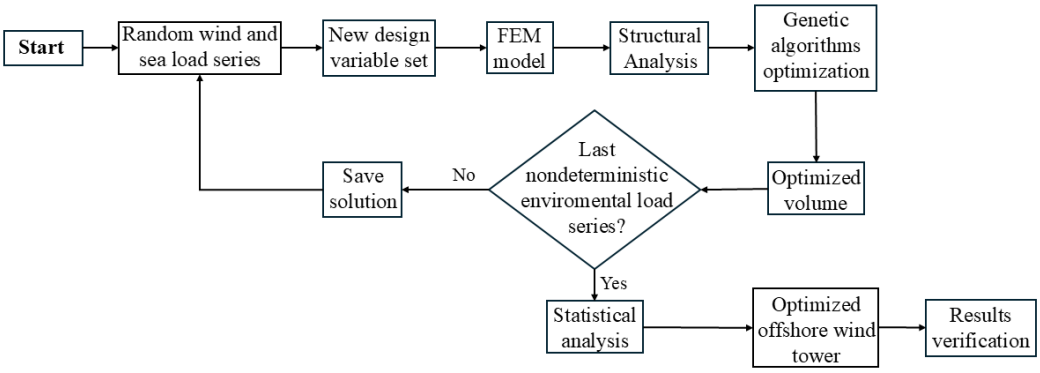



Figure 4. Offshore wind tower structural optimization framework.

Table 5 presents the results for the structural optimization considering the mean μ, standard deviation σ and maximum values with a confidence level of 95% for the design variables $d_1$, $d_2$, $t_1$, $t_2$ and $t_3$, and the structural response for the top maximum displacement $U_x$, bottom rotation $\theta_{max}$, fundamental frequency $f_{01}$, buckling load multiplier $L_m$ and maximum von Mises stress $\sigma_M$. Analyzing these results, it can be concluded that the constraints of the fundamental frequency, horizontal displacement at the top of the structure, and buckling resistance had the greatest influence on the optimization process, while the constraints of rotation at the base of the tower and the maximum von Mises stress had a lesser influence on this process.

Table 5. Structural optimization of the offshore wind tower.

| Load series | $U_X$ (m) | $\theta_{max}$ (deg) | $f_{01}$ (Hz) | $L_m$ | $d_1$ (m) | $d_2$ (m) | $t_1$ (mm) | $t_2$ (mm) | $t_3$ (mm) | V (m³) |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 1.18 | 0.0271 | 0.2353 | 1.77 | 6.5 | 3.3 | 44 | 26 | 11 | 75.9 |
| 2 | 1.16 | 0.0271 | 0.238 | 1.9941 | 6.5 | 3.3 | 0.044 | 0.026 | 0.011 | 78.1 |
| 3 | 1.21 | 0.0311 | 0.234 | 2.0851 | 6.5 | 3.1 | 0.042 | 0.029 | 0.012 | 76.6 |
| 4 | 1.21 | 0.0306 | 0.234 | 1.70338 | 6.4 | 3.2 | 0.044 | 0.028 | 0.012 | 75.9 |
| 5 | 1.25 | 0.031 | 0.230 | 2.06331 | 6.5 | 3.1 | 0.041 | 0.027 | 0.011 | 75.1 |
| 6 | 1.29 | 0.0307 | 0.228 | 2.01363 | 6.4 | 3.2 | 0.043 | 0.026 | 0.012 | 75.2 |
| μ | 1.22 | 0.03 | 0.233 | 1.938728 | 6.43 | 3.16 | 0.0426 | 0.0272 | 0.0115 | 76.2 |
| σ | 0.034 | 0.03008 | 0.00269 | 0.133742 | 0.067 | 0.067 | 0.001 | 0.000889 | 0.0005 | 0.827 |
| CL95% | 1.273 | 0.00102 | 0.2377 | 2.159403 | 6.54 | 3.28 | 0.044 | 0.028 | 0.0123 | 77.5 |

Upon analyzing the optimized design variables in Table 5, it was observed that variable $t_1$, associated with the monopile foundation and lower transition segment, was reduced by 26.7% compared to its initial value of 60 mm. Conversely, diameter $d_1$ increased by 8%. Additionally, a reduction of 14.6% in the upper diameter and 35.2% in the wall thickness of the steel tower was observed. indicating that the most significant impact of the optimization process occurred in the monopile foundation. These changes demonstrate that the structural optimization carried out has resulted in significant modifications throughout the entire support structure. As a result of these changes, the total steel required for constructing an equivalent structure for a 5 MW wind turbine was reduced by 21.8%.

# 6. Conclusions

This study performed structural optimization by integrating the Finite Element Method (FEM) with genetic algorithms. The numerical model accounted for the stochastic effects of wind and wave loads on the steel tower, as well as the soil-structure interaction. Given the random nature of the environmental loads, a statistical analysis was conducted to determine the structural response of the offshore wind turbine (OWT) and obtain the optimized design variables.

A preliminary modal analysis showed that the finite element model for the offshore wind tower exhibited satisfactory accuracy compared to the results from Gentils *et al*. [2] and Jonkman [3]. A subsequent structural dynamic analysis revealed that the tower's response is primarily governed by the first fore-aft natural frequency.

Considering the optimization process, the most significant changes in the design variables were observed in the thickness and diameter parameters related to the monopile foundation. These modifications resulted in a 21.8% reduction for steel required for constructing an offshore wind turbine equivalent to the initial design. This outcome stands for a substantial decrease in commissioning costs for new offshore wind farms.

For each of the six randomized environmental load series considered, the methodology successfully identified design variables that results in a structure with a dynamic response closely aligned with design constraints. Among the design constraints, the fundamental frequency, maximum horizontal displacement, and buckling load multipliers had the greatest influence on the optimization process, with the optimized design operating near the established limits for these parameters. Conversely, the rotation at the base of the tower and the maximum von Mises stress had a lesser influence.

Given the dynamic response, particularly the horizontal displacement at the top of the tower, a potential risk for structural fatigue failure is evident. As wind and wave loads induce repeated stresses on the structure, future investigations should incorporate a comprehensive fatigue life analysis. This would ensure the development of a more robust design that addresses uncertainties related to dynamic environmental forces and enhances the long-term performance and reliability of offshore wind turbine towers.

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