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RESEARCH ARTICLE

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Influence of soil–structure modelling techniques on offshore wind turbine monopile structural response

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Abstract

The importance of appropriate offshore wind turbine (OWT) monopile structural modelling technique cannot be overstated in the successful design and installation of a new generation of larger and heavier structures to deliver the increasing capacity demand. The lack of clear design guidance and acceptable structural modelling techniques across the industry results in a range of conservative but expensive design and installation techniques. Most of the OWT monopile modelling efforts lie in the substructure (foundation) and interaction with the supporting soil which is highly nonlinear along the length of the embedment depth of the monopile structure. Typically, monopile offshore wind turbine structural modelling can be completed using, amongst others, one of the following techniques: 3D finite element modelling with mass soil foundation, API *p-y* curve soil springs, JeanJean soil springs, and the newly developed PISA modelling approach. The study presented in this paper considers the application of the 3D finite element modelling with mass soil, API *p-y* soil springs, and the JeanJean soil springs technique. By comparing the structural response, the 3D finite element modelling with mass soil results in an improved natural frequency and harmonic response. Furthermore, a reduced displacement was observed in the 3D finite element model with mass soil which will ultimately result in a corresponding improvement in the structure's useful operational design life. The application of the API *p-y* soil springs, JeanJean soil springs, and other modelling techniques requires extensive calibration to ensure the correct structural response and behaviour are achieved. This becomes a key factor as the boundaries of the size of the structure and turbine capacity are pushed even further for the new concept generation offshore wind turbines, which are required to deliver a higher capacity of 12 to 15 MW with the aim of achieving 20 MW, whilst achieving an efficient cost-effective engineering design and installation process.

KEYWORDS

buckling, harmonic response, monopile, natural frequency, offshore wind, soil–structure interaction

1 | INTRODUCTION

Serviceability limit state design is generally considered the strictest governing design criterion for offshore wind turbines to ensure efficient operational functionality throughout the design life of the structure.[1] Offshore wind turbines are a dynamic-sensitive structure, subjected to complex external dynamic environmental and operational loads such as wind, waves, rotational frequency of the rotor (1P), and blade passing frequency (3P for three bladed turbine).[2] The operating envelope of the wind turbine is defined by the allowable deflection (tilt and rotation) of the rotor-nacelle-assembly (RNA) specified by the manufacturer and the general structural deflection limit according to industry design standards. Modal analysis and harmonic response analyses are performed to establish the natural frequency and response amplitude of the structure which forms an important and fundamental aspect of the serviceability limit state check. The aim is to avoid resonance effects that will ultimately lead to large amplitude stresses and subsequent accelerated structural fatigue damage. One of the primary design aims is to ensure that the natural frequency of the structure does not coincide with the fundamental frequencies of the external loads. The accuracy in estimating the natural frequency of the offshore wind turbine primarily depends on the modelling technique, analytical model verification, and model calibration where data exists. The importance of the natural frequency and structural response cannot be overstated, as this can mean the difference between an accurate and cost-effective design or an over-conservative and expensive design.[3]

The structural modelling technique is crucial in capturing the soil–monopile relationship and interaction. Inaccurate modelling can result in soft soil–structure or on the other hand, stiffer soil–structure than is accurate which directly affects the natural frequency and response of the offshore wind turbine. The problem is exacerbated by the coupled and nonlinear nature of the structure foundation and the variability of the soil properties along the buried monopile length. Typically, the soil–structure modelling can be completed using amongst others, the following approaches:

a. 3D finite element modelling with mass soil;
b. API *p-y* curve nonlinear soil springs;
c. JeanJean nonlinear soil springs;

The famous *p-y* curves in accordance with API-RP 2014 and as described in DNV-ST-0126 are limited to smaller pipe diameters; hence, it is recommended to validate the use of *p-y* curve generated soil springs for application on monopiles with greater than 1.0-m diameters by way of testing, real-time structural monitoring, or the use of finite element analysis or other suitable means.[4] The *p-y* curves for sand were developed by O'Neill and Murchinson (1983), while Dunnavant and O'Neil proposed the *p-y* curves for clay (2000) which were adopted by the API-RP (2000) and still serve as basis for many offshore wind turbine designs. The *p-y* curve model is used to represent the soil nonlinear resistance to the displacement, and the *t-z* curves are used to model the axial loading to structure displacement.[4]

The ongoing PISA project, through European joint-industry academic research, developed a design model for laterally loaded offshore wind turbine monopiles for large diameter, relatively rigid piles, with low length to diameter (L/D) ratios. The project introduces new procedures for site-specific calibration of soil reaction curves that can be applied within a one-dimensional (1D), Winkler-type computational model.[5] Although strides in research are being made in the offshore wind turbine industry such as the PISA project for soil–structure interaction, there is a need for future research and calibrated modelling techniques, including the understanding of soil–structure interactions on future OWT monopile structure concepts.

TABLE 1 Properties of NREL 5-MW reference wind turbine model[3,8]

Description	Value	Units
Rating	5	MW
Rotor diameter	126	m
Hub height	87.6	m
Hub mass	56,780	kg
Nacelle mass	240,000	kg
Tower mass	347,460	kg
Rotor mass	110,000	kg
Cut-in, rated wind	3, 11.4	m/s
Cut-in, rated rotor speed	6.9, 12.1	rpm
Tower base diameter and thickness	6, 0.05	m
Tower top diameter and thickness	3.87, 0.025	m

The monopile support structures, including diameters exceeding 7.5 m, are designed according to the soft-stiff soil approach. Based on several research and industry applications, the *p-y* curve method is considered unsuitable for performance reasons due to its weak nonlinear behaviour under operational loading. The nonlinear implications and the stiffness of the *p-y* methods are questioned by several publications and response comparisons of processed data from in-service monitoring. The *p-y* curve generated soil springs demonstrates an overall underestimation of the soil–structure stiffness.[6] Although the research into the soil–structure interaction continues to be one of the focuses of research, there is yet to be an updated, industry acceptable, and recommended modelling technique. Finite element modelling method is another technique that is adopted for representing and analysing the soil–structure interaction. Previous review and investigation on offshore wind turbine monopiles soil–structure modelling, gaps in knowledge and understanding of structural modelling and dynamic response, and recent improvements can be assessed for further information.[7]

This paper investigates the influence of modelling techniques on offshore wind turbine structural response and the impact on achieving an efficient and cost-effective engineering design. Section 2 of this article presents and discusses the three modelling techniques considered. Fundamental frequencies and generation of the safe design and operational zones of the offshore wind turbine are covered in Section 3. Wind and wave spectra calculations and generations are presented in Sections 4 and 5, respectively. The results and discussions from the investigation are presented in Section 6. Important highlights and conclusions are discussed in Section 7.

2 | MODELLING TECHNIQUES

The model is based on a validated NREL 5-MW offshore wind turbine embedded 60 m below ground level and in a water depth of 20 m. Three modelling techniques are considered in this study, as described in this section. The basic structural design properties are outlined in Table 1, and the soil properties are presented in Table 2.

The thrust force (*Th*) is applied at the top of the wind turbine tower considering wind acting on the turbine rotor. The thrust force is estimated according to the following expression[9]:

$$Th = \frac{1}{2}\rho_a A_R C_T U^2 \#, \tag{1}$$

where ρ_a is the density of air, A_R is the rotor swept area, C_T is the thrust coefficient, and U is the wind speed. The wind speed can range from cut-in to cut-out, with the appropriate thrust coefficient. The thrust coefficient can be approximated from Thrust Coefficient Approximation Curve or using equations. The thrust coefficient can be estimated for between cut-in and rated wind speed, and after rated wind speed when pitch control is active according to the following equations:

Between cut-in (U_{in}) and rated wind speed (U_R):

$$C_T = \frac{3.5\,[m/s]\,(2U_R + 3.5\,[m/s])}{U_R^2} \approx \frac{7\,[m/s]}{U_R}. \tag{2}$$

TABLE 2 Tower and soil/foundation properties[1,3]

Description	Value	Units
Tower/steel structure material properties		
Density (represents the effective density which accounts for paint, bolts, welds, flanges)*	7850 (8500)*	kg/m^3
Young's modulus	210	GPa
Shear modulus	80.8	GPa
Steel grade	355	
Soil/foundation properties		
Embedment depth	60	m
Density	1800–2000	kg/m^3
Young's modulus	40–70	MPa
Poisson's ratio	0.45	-
Friction angle	20	Deg
Cohesion	150–250	MPa

After rated wind speed, when pitch control is active, and power is assumed to be kept constant

$$C_T = 3.5\,[m/s]\,U_R(2U_R + 3.5\,[m/s]).\frac{1}{U^3} \approx \frac{7\,[m/s]\,U_R^2}{U^3}. \tag{3}$$

Thrust coefficient for different wind scenarios and combinations, including turbulence, extreme, and gust, can be estimated using the appropriate equations from literatures and design codes. This is beyond the scope of this study.

The wave force applied at the mean water level is estimated by assuming linear wave theory such as the Morison's equation.[9] The force on a unit length strip of the substructure is the sum of the drag force F_D and the inertia force F_I:

$$dF_{wave}(z,t) = \frac{1}{2}\rho_w D_S C_D w(z,t)|w(z,t)| + C_m \rho_w A_S \dot{w}(z,t), \tag{4}$$

where C_D is the drag coefficient, C_m is the inertia coefficient, and ρ_w is the density of seawater. The total horizontal force is then calculated by integrating over the water depth and applied at the mean water surface in in the FE model discussed in Section 2.3.

It is worth noting that the tower section properties thickness given in the 5-MW reference wind turbine for offshore system development technical report[8] is based on land-based tower for the NREL 5-MW baseline wind turbine. The technical report specified a baseline thickness of 0.027 m and top thickness of 0.019 m. These section property thicknesses were improved as presented in Table 1 above, according to Senanayake et al.[3]

2.1 | API modelling approach

For soft clay (s_u ≤ 100 kPa) subjected to static lateral loads, the ultimate unit lateral bearing capacity, p_uD, has been found, according to the API standard, to vary between 8 s_uD and 12 s_uD, except in shallow depths where failure occurs in a different mode due to low overburden stress. The lateral bearing capacity can suffer from significant deterioration when subjected to cyclic loads below the static loads.

The soil lateral bearing capacity, p_uD, increases from 3 p_uD to 9 p_uD as z increases from 0 to z_R according to the following equation[10,11]:

$$p_uD = 3\,s_uD + \gamma^I zD + J\,s_u z, \tag{5}$$

$$p_uD = 9\,s_uD for\, z \geq z_R, \tag{6}$$

where p_uD is the ultimate resistance, units of force per unit length; s_u is the undrained shear strength of the soil at the point in question, in stress units; D is the pile outer diameter; γ^I is the submerged soil unit weight; J is the dimensionless empirical constant values ranging from 0.25 to 0.5 having been determined by field testing; z is the depth below the original seafloor; and z_R is the depth below soil surface to the bottom of the reduced resistance zone.

The lateral soil resistance–displacement relationships for piles in clay are generally nonlinear. The p-y curves for short-term static loads and for cases where equilibrium has been reached when subjected to cyclic loads may be generated according to the relationship defined in the design code. The generated soil springs at selected depths, as used in the investigation according to API RP methodology, is presented in Figure 1.

2.2 | JeanJean modelling approach

The JeanJean modelling approach assumes that the shear strength profile increases almost linearly with depth. The method is more suitable for dynamic analysis of structures, subject to cyclic loads that can ultimately lead to fatigue damage.[12] The centrifugal curves from tests according to the JeanJean approach are stiffer than the API RP curves, and the ultimate pressure also exceeds the value of 9 p_uD given by the API RP. The average value of the ultimate pressure is 13.4 p_uD.[13] For shear strength profiles which increase almost linearly with depth, the ultimate unit pressure, P_{max}, can be calculated according to the following expressions:

$$P_{max} = N_p.S_u, \tag{7}$$

$$N_p = 12 - 4.e^{\left(\frac{-z\,Z}{D}\right)}, \tag{8}$$

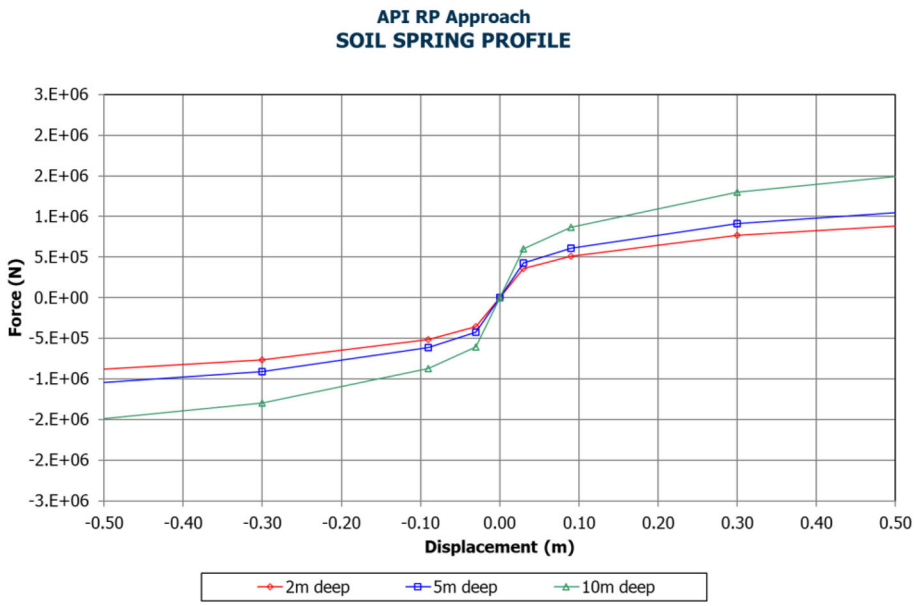

FIGURE 1 Soil spring profile according to API RP

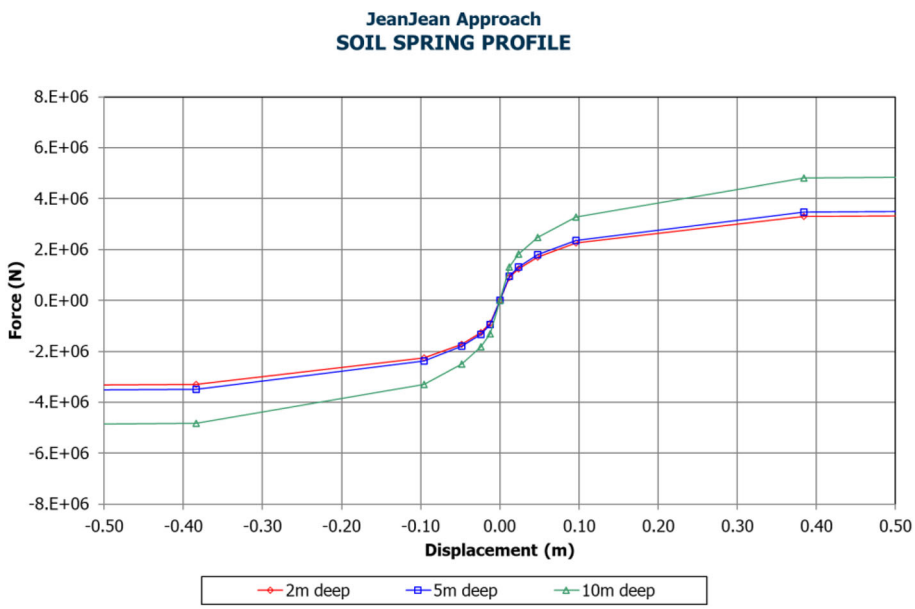

FIGURE 2 Soil spring profile according to JeanJean

$$\xi = 0.25 + 0.55.\lambda, for\ \lambda < 6; and\ \xi = 0.5\ for\ \lambda \geq 6, \tag{9}$$

$$\lambda = \frac{S_{uo}}{S_{u1}.D}, \tag{10}$$

where S_{uO} is the shear strength intercept at seafloor; S_{us} is the rate increase of shear strength with depth; D is the pile diameter; and z is the depth of interest.

The generated soil springs according to the JeanJean approach at selected depths as used in the investigation is presented Figure 2.

2.3 | Finite element modelling approach

The finite element modelling approach is conducted using Ansys Workbench - Static Structural. The model is made up of the tower supporting a lump mass representing the RNA mass. The tower is submerged in a water depth of 20 m and embedded 60 m below mudline. The foundation is modelled using soil mass extending 200 m in diameter (approximately 30 times tower base outer diameter). The foundation is divided into several soil profiles to represent the different soil profiles and properties along the depth. The interaction between the structure and the soil is modelled using friction coefficient of 0.35 calculated based on soil angle of internal friction of 20° as stated in Table 2. The base of the tower is supported on a spring for base bearing support. The interaction and connections of the different sections of the tower are achieved using bonded connections or workbench surface share tool to enable flexibility in meshing the different parts. Mesh sensitivity was completed for the solution.

Free face Quad/Tri mesh type is used to model the tower and foundation structure. Based on findings from the mesh sensitivity, the structure and foundation model mesh sizes are as follows:

- Coarse mesh is used for the soil considering the diameter of 200 m and depth of 60 m. The model successfully solved and converged for the selected mesh size. Refined mesh dependent reactions are not extracted or required from the soil foundation.
- Monopile foundation and tower structure:
  - Between −60 m to −10 m below mudline: the circumferential mesh size is 0.25 m and the longitudinal mesh size is 1.0 m.
  - Refined mesh to capture desired reactions is applied between −10 m below mudline to 10 m above mudline: circumferential and longitudinal mesh size of 0.25 m is applied, respectively. This refinement allowed for stress, bending moment, deflection, and any other desired reactions to be extracted around the mudline region.
  - Between 10 to 20 m above mudline: circumferential mesh size of 0.25 m and a longitudinal mesh size of 0.5 m is applied.
  - Circumferential mesh size of 0.25 m and longitudinal mesh size of 1.0 m is applied at 20 m to the top of the structure.

The model showing sections of the primary structure and soil and meshed profiles is presented in Figure 3. The model was verified by conducting sensitivities, and validated against published literatures and case studies for the NREL 5-MW wind turbine.[1,13]

3 | FUNDAMENTAL FREQUENCIES AND SAFE ZONE

Classical design aims to establish and fix the structure target frequency away from external loads. The target frequency of the structure depends on several variables: installation location, type and capacity of the turbine, wave period and spectrum, wind turbulence, and the operating range of the wind turbine (1P range). The rotational frequency of the rotor (1P) and blade passing frequency (3P for three bladed turbine) both depend on the wind turbine's operating range. Safety margins on the target frequency of the structure are established so that the natural frequency of the turbine should not be within 10% of the 1P and 3P ranges. In addition, high-energy content frequency bands of wind and wave loading are to be avoided to minimise the fatigue damage.[1] This usually leaves a narrow safety zone for the design of the turbine, considering the RNA, tower,

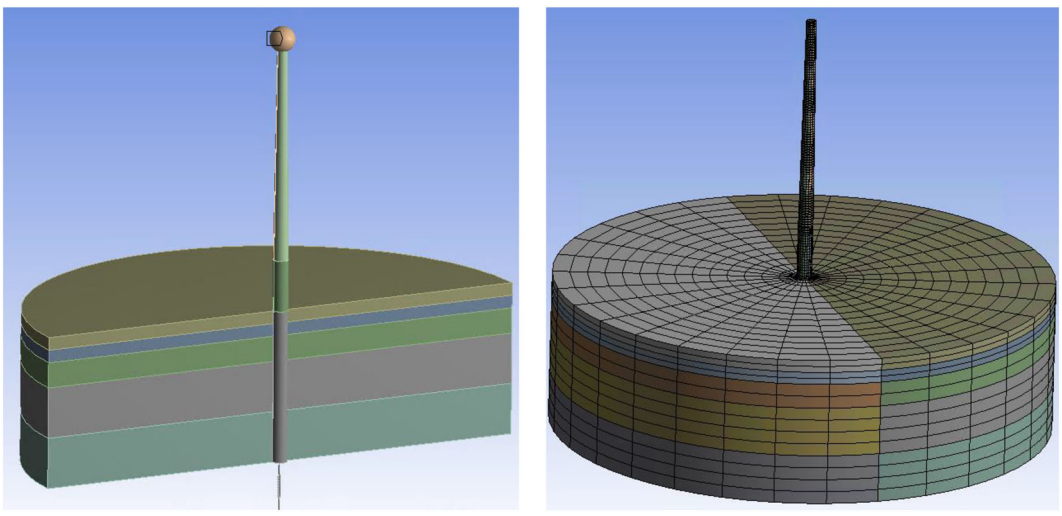

FIGURE 3 FEA model showing soil and meshed profiles

and foundation systems. From an economic standpoint, a softer structure and underestimating the natural frequency of the OWT monopile is desirable; however, the safest design solution is a higher natural frequency target above the 3P range. Stiffer structure and a higher frequency target requires expensive larger and thicker structures and foundation systems which have a corresponding cost impact on transportation and installation. The compromise to the soft or stiffer design is the soft-stiff design where the target natural frequency lies between the 1P and 3P external loads to avoid resonance as presented in Figure 4. The operating range is calculated from the turbine Cut-in and Rated frequencies.

4 | WIND SPECTRUM

Wind spectrum is used to describe short-term stationary wind conditions. This is also known as the power spectral density of the wind speed, usually determined from available measured wind data for site-specific scenarios. Several model spectra exist, and they generally agree in the high frequency range but exhibit significant differences in the low frequency range. Most of the available models may not be suitable for offshore applications as they have been calibrated to onshore or land-based wind data. For example, the Harris spectrum was originally developed for wind over land. Furthermore, the Harris spectrum and many other models are not recommended for use in the low frequency range, that is, for f < 0.01 Hz. Most offshore wind turbines are installed in regions where the structure will experience wind loads, with frequency below the 0.01 Hz. Hence, it is important that the selected model spectra be appropriate for winds with frequencies below 0.01 Hz.[14]

In this study, the empirical *Ochi and Shin spectrum*, applicable for the design of offshore structures, is selected for the generation of the wind spectrum. This Ochi and Shin spectrum is known to have more energy content in the low frequency range (f < 0.01 Hz) than other models. Davenport, Kaimal, and Harris spectra are traditionally developed to represent wind over land or onshore applications. The *Ochi and Shin model spectrum* is developed from measured spectra over a seaway and can be calculated using the following equations:

$$\frac{fS(f)}{u_*^2} = \begin{cases} 583 f_* & \text{for } 0 \le f_* \le 0.003, \\ \dfrac{420 f_*^{0.7}}{\left(1+f_*^{0.35}\right)^{11.5}} & \text{for } 0.003 < f_* \le 0.1, \\ \dfrac{838 f_*}{\left(1+f_*^{0.35}\right)^{11.5}} & \text{for } 0.1 > f_*, \end{cases} \tag{11}$$

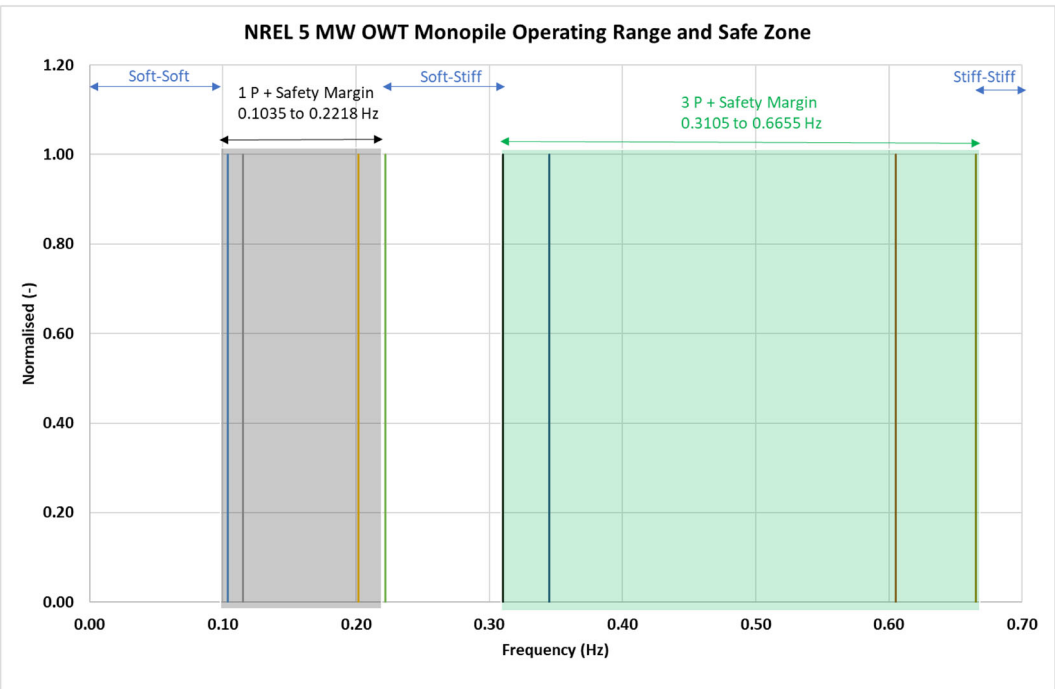

FIGURE 4 NREL 5-MW OWT monopile operating range

where

$$f_* = \frac{f.Z}{U_{10}(Z)}. \tag{12}$$

The generated wind spectrum according to the Ochi and Shin model, along with the external loads (1P and 3P), is presented in Figure 5.

5 | WAVE SPECTRUM

The significant wave height H_s and the peak period T_p are important environmental parameters used to characterise stationary sea-states. Wave spectrum, also known as the power spectral density function of the vertical sea surface displacement, is used to describe short-term stationary irregular sea-states. The JONSWAP spectrum is applied in generating the wave spectrum for this study. The JONSWAP spectrum is recommended for fully developed seas. In addition, it can be applied to describe developing sea-states in a fetch-limited sea according to the following expression[14]:

$$S_J(\omega) = A_\gamma S_{PM}(\omega)\gamma^{exp\left[-0.5\left[\frac{\omega-\omega p}{\sigma.\omega p}\right]^2\right]}, \tag{13}$$

where $S_{PM}(\omega)$ is Pierson–Moskowitz spectrum; γ^I is nondimensional peak shape parameter; σ spectral width parameter; $\sigma = \sigma_a$ for $\omega \leq \omega_p$; $\sigma = \sigma_b$ for $\omega > \omega_p$; $A\gamma = 1–0.287 \ln(\gamma)$ is a normalising factor.

The generated wave spectrum according to the JONSWAP model, along with the external loads (1P and 3P), is presented in Figure 6.

6 | FINDINGS AND DISCUSSIONS

The findings from the study and the discussion of the results are presented in this section by considering the 3D soil mass model, API-RP, and the JeanJean springs supported models. The responses and comparison of the models are also presented and discussed. The monopile is subject to the following external and machine loads according to[15]

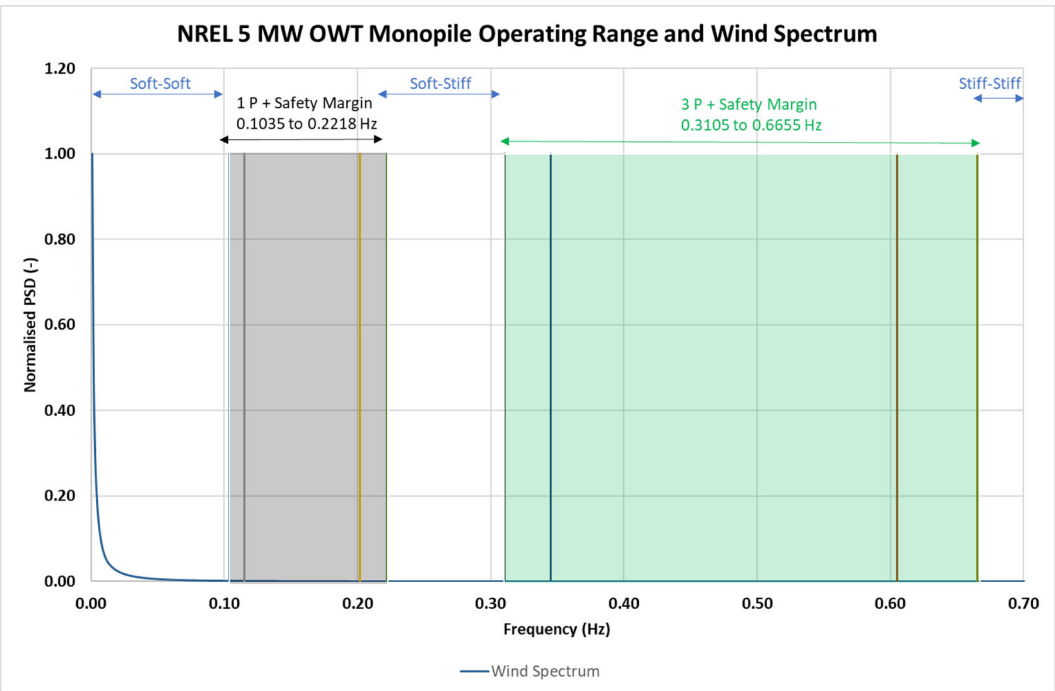

FIGURE 5 NREL 5-MW OWT monopile operating range and wind spectrum

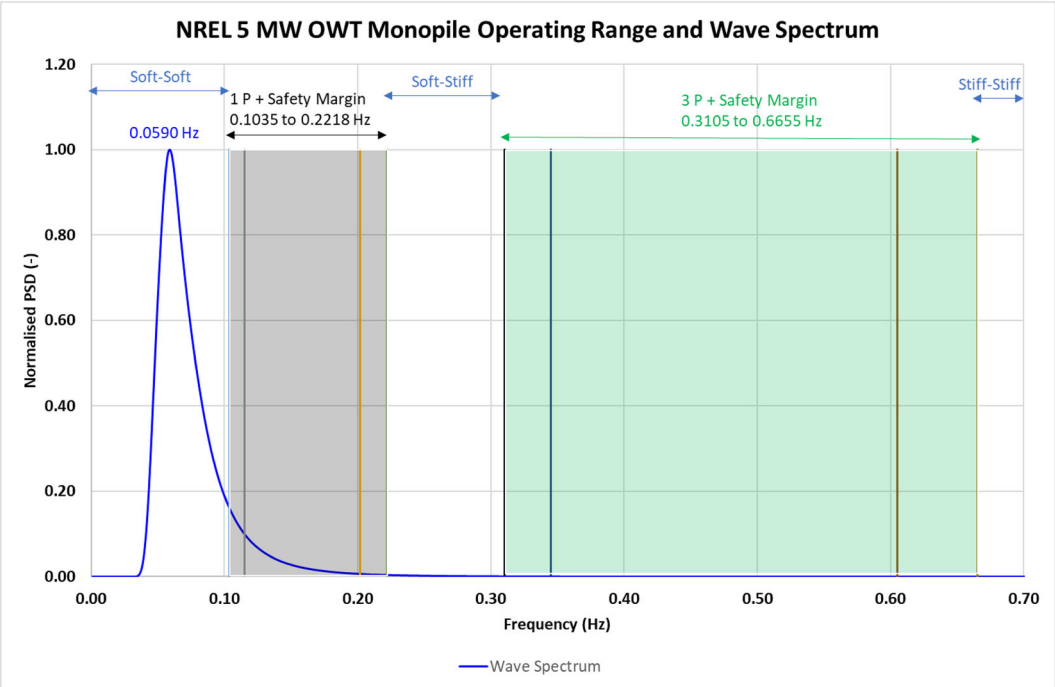

FIGURE 6 NREL 5-MW OWT monopile operating range and wave spectrum

- Self-weight of the RNA is modelled using lump mass at the top of the tower;
- Maximum thrust force due to wind applied at the hub height;
- Horizontal wave force applied at the mean sea level;
- Wind pressure on the tower;
- Hydrostatic pressure.

The models were iteratively tested, and sensitivities performed for different total damping ratios, which include steel damping, tower oscillation damping, aerodynamic, hydrostatic, and soil damping. A total damping ratio of 10% is selected and applied, considering previous studies and guidance provided on damping estimation of offshore wind turbine structures.[7,16]

6.1 | Influence of springs-supported modelling techniques

Three springs-supported modelling techniques are conducted, and the results are compared and benched marked against the 3D soil mass model.

Traditionally, the springs-supported curves are generated and modelled considering a single longitudinal springs application on the monopile foundation, supported at 180° circumferential cardinal point along the pile length; relative to the wind pressure, wave, and thrust force loads on the monopile at 0° heading. This modelling technique is primarily adopted due to the modelling computational efforts and associated costs in handling eight to 16 times more springs around the circumference cardinal points and convergence issues. Therefore, the influence of the springs-supported modelling techniques is investigated, considering the circumferential positions of the springs at 180° (denoted as near-face scenario, considering the predominate direction of displacement) from the thrust-wave-wind heading. In addition, springs at 90° and 270° (denoted side-face springs) are investigated, and at 0° denoted as far-face longitudinal springs. The different springs-supported modelling positions are presented in Figure 7. The models are presented and described in detail below:

- Base case near-face longitudinal primary springs (+x): the primary near-face springs are responsible for providing the main structural foundation support against the machine loads and environmental loads. The primary near-face springs are located opposite to the loading direction, providing support to the monopile generated through compressive soil strength as shown in Figure 7. The maximum top deflection of the monopile for the API-RP and JeanJean models are 4.03 m (1.74°) and 3.75 m (1.62°), respectively. Comparing the spring models against the 3D mass soil model, with a maximum deflection of 2.46 m (1.06°), leads to an increase of 64% and 52% in deflection in the API-RP and JeanJean models, respectively.

- Far-face longitudinal springs in additional near-face primary springs (−x): far-face springs acts by generating additional resistance as the monopile tends to overturn or bend about the mudline. The far-face springs is illustrated in Figure 7. Introducing the far-face springs leads to an improvement of up to 2% on the base-case near-face, for the API-RP springs. However, the JeanJean springs showed an insignificant improvement on the base-case model.
- Side-face springs (skin friction resistance) in addition to both near- and far-face springs (±z): the side-face springs are introduced to generate skin friction resistance from the large monopile side-face contact interaction with the soil as the monopile responds to the imposed operational and environmental loads. The skin friction generated from the monopile side face contact and interaction with the soil leads to an improvement of 21% and 27% in deflection on the base-case using the API-RP and JeanJean models, respectively. The side-face springs are often ignored for smaller monopile applications, for example, in the oil and gas sector. However, this is shown to be important as it leads to a significant improvement in the monopile structural response for springs-supported structure-foundation models.

Comparison of the 3D mass soil and API-RP springs-supported models performed as part of the PISA project supports the findings of this study. Ratio of horizontal load at the top of the OWT structure to the pile head displacement (force/displacement) was reported to be 19.38 for the 3D mass soil model and 8.75 for the API-RP model.[5,17] This represents a 121% increase in 3D mass soil model stiffness on the API-RP spring-supported model stiffness.

Summary of the deflection and stress utilisation results considering the influence of spring-supported modelling techniques and 3D mass soil is presented in Table 3. Stress utilisation is calculated based on selected steel grade 345 MPa. The stress infographs at the mudline for the 3D mass soil model and spring-supported models is presented in Figure 8.

6.2 | Buckling of 3D mass soil and springs-supported models

The first buckling mode is investigated in this study by considering the 3D mass soil model supporting the monopile, the API-RP and JeanJean springs-supported monopile models. The buckling is initiated by the compressive axial loads from the RNA, the monopile self-weight, and the bending loads. The bending loads are from the horizontal thrust force at the RNA, wind action on the tower above water level, and wave loads on the tower.

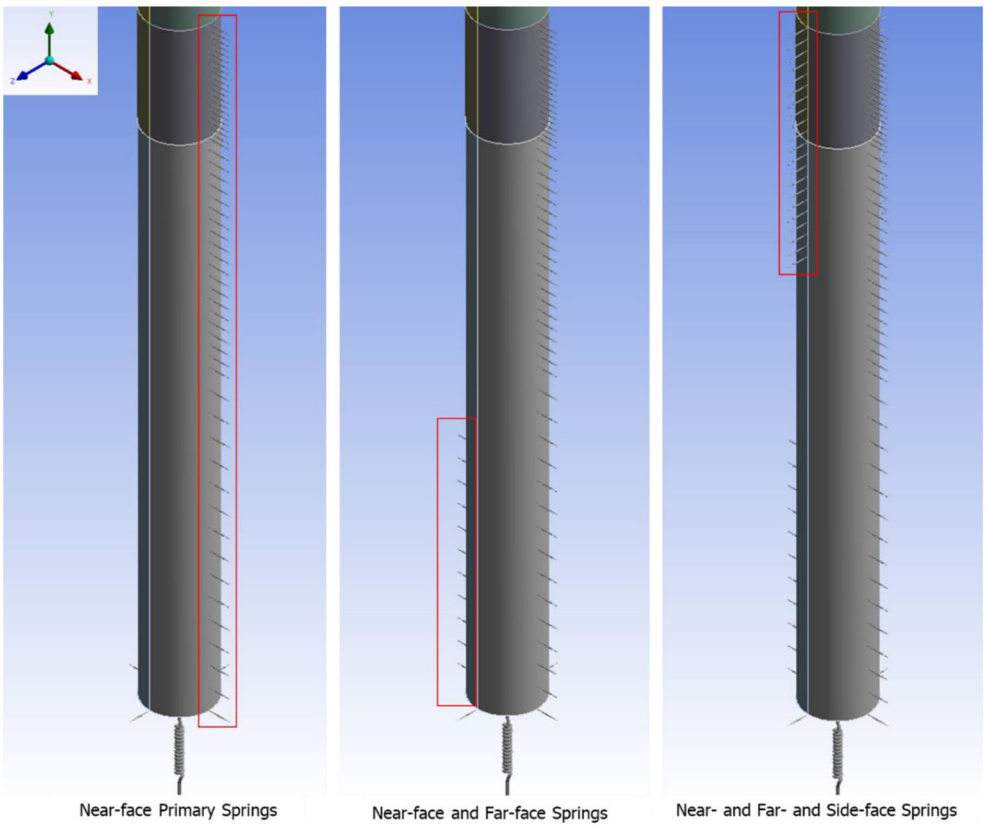

FIGURE 7 Foundation springs supported modelling techniques

TABLE 3 Summary of springs-supported modelling techniques

		API-RP springs			JeanJean springs		
Description	**3D mass soil**	**BC**	**FF**	**SF**	**BC**	**FF**	**SF**
Deflection (m)	2.46	4.03	3.96	3.12	3.75	7.74	2.74
Total Rotation (°)	1.06	1.74	1.71	1.35	1.62	1.62	1.19
von Mises Stress (MPa)	269	670	662	417	779	778	460
Stress Utilisation (%)	78	194	192	121	226	225	133

Note: BC is base-case near-face, FF is far-face, and SF is side-face.

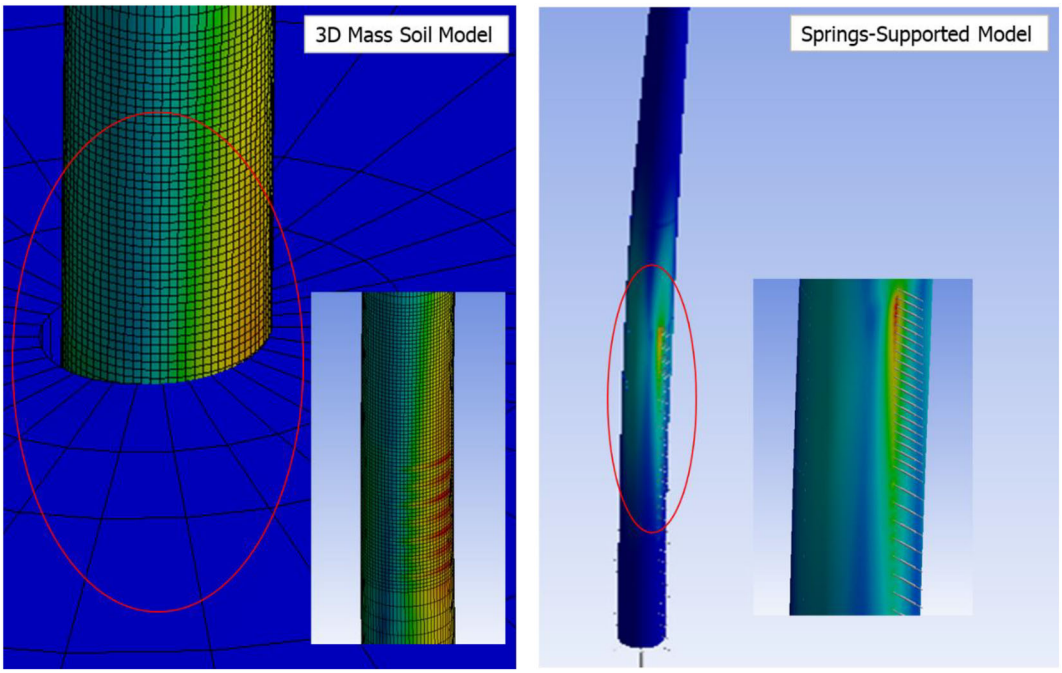

FIGURE 8 Three-dimensional mass soil and springs-supported models stresses

TABLE 4 Summary of spring-supported modelling techniques

		API-RP springs			JeanJean springs		
Description	**3D mass soil**	**BC**	**FF**	**SF**	**BC**	**FF**	**SF**
Buckling utilisation (%)	74	148	146	176	140	140	178

Note: BC is base-case near-face, FF is far-face, and SF is side-face.

The monopile utilisation ratio due to buckling loads is 74% for the mass soil supported model. For the same loading conditions, the API-RP model buckling utilisation is 148% for the base-case near-face springs supported model described in 6.1. The JeanJean springs supported model buckling utilisation is 140%.

Summary of the buckling utilisation ratio for the different models is presented in Table 4.

The buckling response of the springs supported models is reduced by at least a factor of 2 when compared with the mass soil model. The poor buckling performance of the springs supported model is primarily due to the local punching of the springs on the monopile shell. The reduced buckling capacity is exacerbated when side-face springs are introduced to the model, aimed at generating additional resistance and stiffness from monopile–soil contact, but instead this leads to a reduced average buckling capacity of 23% on the base-case springs supported models. Studies on wind turbine tower buckling behaviour based on energy method supports the findings on how bending moments affects the buckling evolution

paths, leading to section distortions (oval shaped) and curvature, resulting in change in the strain energy dissipation. The shell geometry along with local imperfections show a strong influence on the buckling and noticeable reduction in the monopile capacity during combined loading scenarios.[18]

An infograph of the 3D mass soil model and the springs supported models buckling response is presented in Figure 9.

6.3 | Harmonic response

The monopile structure is assessed for different modelling techniques in the frequency domain through the application of a forced frequency response, also known as the harmonic response analysis (a branch of linear dynamic analysis), to establish the structure excitation's natural frequency. As discussed in Section 3, this is fundamental for the design of the OWT monopile and avoidance of resonance which can lead to rapid damage and reduced operational life when the natural frequency or first bending mode shape of the structure is excited by external loads. The harmonic response analysis provides insight into the significance of the different modelling techniques and their responses to the external environmental and machine loads.

The first natural frequency mode for the mass soil model is 0.2460 Hz, located in the soft-stiff zone and away from the external loads, including the 1P and 3P loads. The API-RP and JeanJean springs supported models' first natural frequencies are 0.2021 and 0.2132 Hz, respectively. They were found to be within the excitation region of the rotor rotational frequency (1P). The results from the harmonic response analyses showing the models first natural frequency and their interaction with external loads in frequency domain are presented in Figure 10.

The presentation in Figure 10 highlights the significance of modelling techniques on the design of OWT monopile structures. For the same loading conditions, the mass soil model, although computationally expensive, indicates an improved model response and design in comparison with the springs supported models. Even after incorporating the three improvements outlined in Section 6.1 for the springs supported models, the API-RP and JeanJean models were still not sufficient to clear the "1P + Safety Margin" frequency excitation region, which will ultimately lead to reduced useful operational life of the monopile structure.

The springs supported models can still be applied in the design of OWT monopiles, but this would require significant deviation from current springs force–stiffness calculation and instead rely heavily on calibration using both the mass soil model and measured monitoring data to verify,

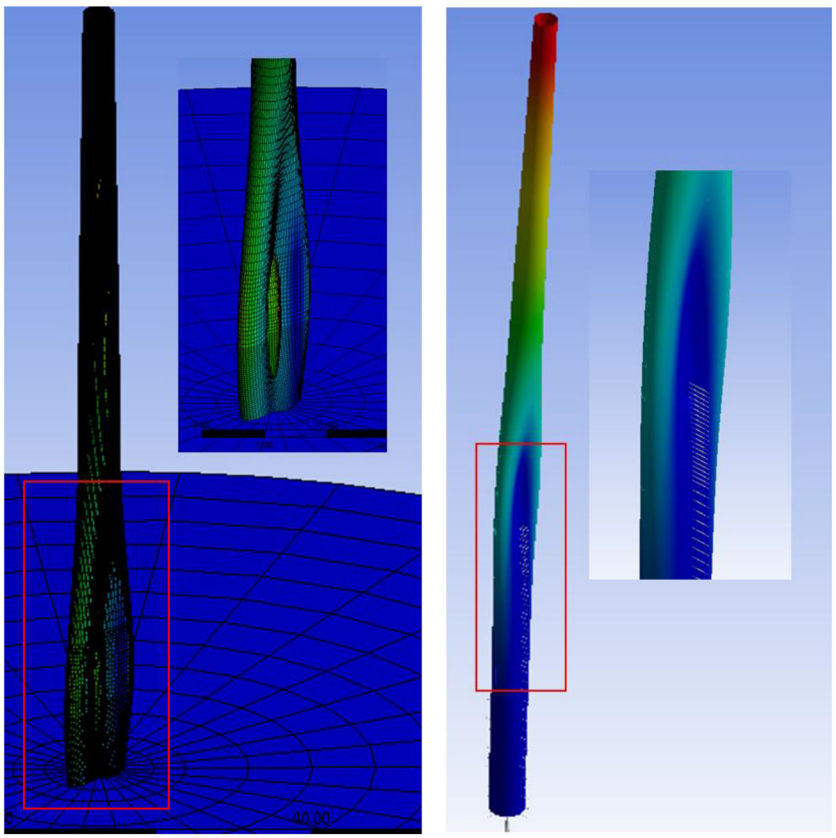

FIGURE 9 Three-dimensional mass soil and springs-supported models buckling

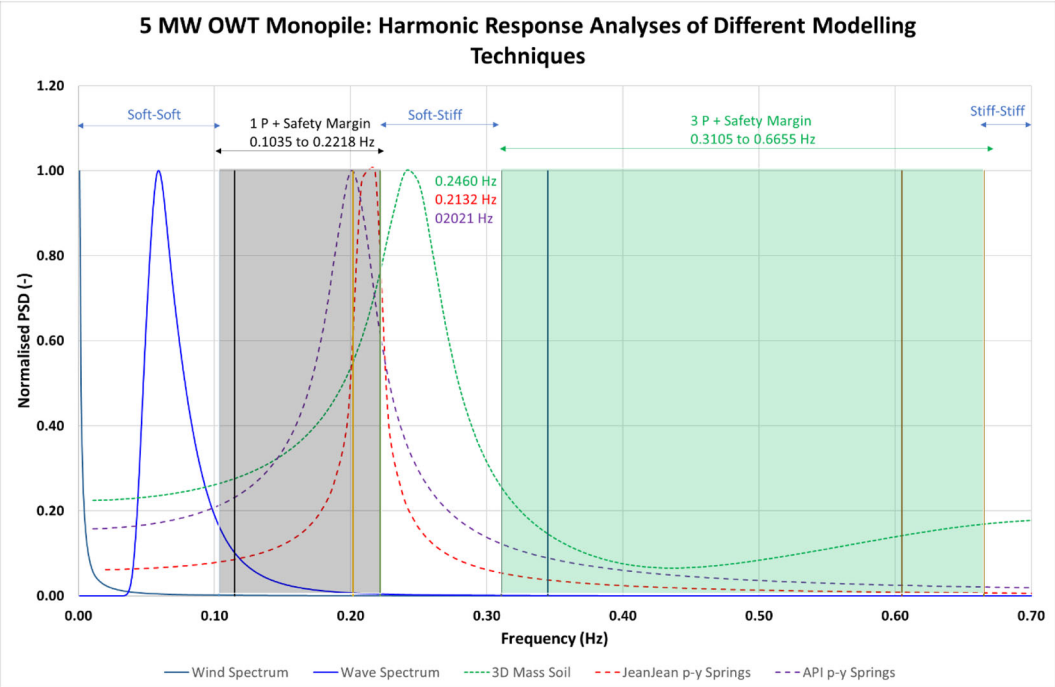

FIGURE 10 Harmonic response analyses

investigate, and correct uncertainties in the design.[19] Engineering justification and benefit for undertaking the design and analysis using this method can be made on the grounds of reducing computational costs and improving efficiency considering the sheer number of modelling iterations, improvements, loading conditions, and combinations required for the complete structural design of OWT monopiles.[17]

7 | CONCLUSIONS

This research investigates the relationship between offshore wind turbine monopiles and the influence on the structural response considering external environmental loads and the turbine machine loads. The modelling approaches investigated in this study include the 3D mass soil supported monopile model, API-RP springs supported model, and JeanJean springs supported monopile model. The models are a 5-MW OWT monopile, subjected to the same external environmental and machine loading conditions (maximum thrust force, wave force, wind pressure on tower, and hydrostatic pressure).

The following conclusions can be drawn from the investigation:

A. Three modelling improvements on the API-RP and JeanJean springs-supported monopile models were investigated. The study shows that the modelling refinements resulted in a corresponding average improvement of 24% on the base-case springs-supported models. Despite these improvements, the total deflections for the refined API-RP and JeanJean springs-supported models were observed to be 27% and 12% more than the 3D mass soil model, respectively.
B. The investigation further reveals the influence and significance of modelling techniques on the buckling capacity and response of the monopile structure when subjected to external loads. The 3D mass soil model monopile structure utilisation due to buckling is 74%. This compares with 148% and 140% for the API-RP and JeanJean springs-supported models, respectively, for the same design and loading conditions. The poor buckling capacity and response of the springs-supported models are exacerbated by the local punching of the springs on the monopile shell.
C. Harmonic response analyses are conducted with the aim of providing insights into the influence and significance of the modelling techniques on the monopile structural response when in frequency domain, considering the applied loads and operational conditions. The determination of the structural frequency response with respect to external loads is crucial in the design of the offshore wind turbine monopile, including the avoidance of resonance excitations that can rapidly reduce the design operational life. From the investigation, the following findings were made:

i. The 3D mass soil supported model's first natural frequency mode is 0.2460 Hz, located in the soft-stiff zone and away from the external loads capable of causing resonance excitation, including the 1P (0.1035 to 0.2218 Hz) and 3P (0.3105 to 0.6655 Hz) loads for the 5-MW OWT.
ii. The API-RP springs-supported model's first natural frequency mode is 0.2021 Hz which falls comfortably within the "1P + safety margin" excitation frequency zone.
iii. Similarly, the JeanJean springs-supported model resulted in a natural frequency of 0.2132 Hz which is within the "1P + safety margin" excitation frequency zone.

D. Although, the springs-supported models may currently be suitable in some industries and applications, such as smaller pipes as used in the Oil and Gas sector, this investigative study on the influence of modelling techniques shows that the springs-supported models would benefit from extensive refinements and calibration for offshore wind turbine monopile applications with bigger and heavier structure sections. The PISA project for soil–structure modelling and interaction is an example of research progress aimed at addressing some of the modelling issues in the offshore wind turbine industry, including the influence on the structure fatigue life which is outside the scope of this study.[5,17]

E. Understanding the upper bound capacity limits of OWT monopile is an important and interesting question presently facing the industry. Although this is outside the scope of this study, extensive research and industry studies are required on the low technology readiness levels of future bigger and heavier OWT monopile structure concepts. Some areas of interests include but are not limited to the following[7]:
i. Understanding the influence of modelling techniques and refinements on future monopile concepts.
ii. Applying the appropriate modelling techniques in defining the design envelope and limits of future concepts up to and including 20-MW OWT monopiles, and maybe higher.
iii. Understanding and outlining the limiting structural criteria, installation depth, and installation considerations such as acceptable and excessive pile inclination that may arise from driving larger diameter piles.

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AUTHOR CONTRIBUTIONS

The authors are responsible for ensuring that the descriptions are accurate and agreed by all authors.

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CONFLICT OF INTERESTS

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

PEER REVIEW

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DATA AVAILABILITY STATEMENT

Data sharing not applicable—no new data generated, or the article describes entirely theoretical research. Data sharing not applicable to this article as no datasets were generated or analysed during the current study. References have been provided in the manuscript regarding inputs used in the modelling and analysis and are derived from public domain.

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