Response of Monopile Offshore Wind Turbine Structure Subjected to Seismic Loads and Degradation

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***Abstract*—The objective of this work is to analyse a monopile offshore wind turbine structure subjected to the coupled loads originating from wind, soil interactions and possible seismic activities. The monopile support structure is designed to be installed at 25 m water depth to support a 5MW wind turbine. The structural assessment is performed based on the finite element method accounting for the nonlinearities associated with the geometry and material. The nonlinear structural response of the monopile structure during the seismic activity is assessed accounting for the time-variant degradation of the load-carrying capacity due to corrosion. The nonlinear response of the degrading structures at different years of the service life is presented.**

Keywords—Monopile; offshore wind turbine; ultimate strength; fatigue; corrosion; FEM; RESET

I. INTRODUCTION

The life-cycle performance assessment of offshore wind turbines is a major challenge, and its significance has been growing even more as the offshore wind industry is looking forward to building more powerful, efficient and economised structures as well as cost-effective operations. Therefore, the development of risk assessment frameworks is necessary not only for the optimal design but also in the optimum operating and maintenance strategy in order to fulfil the primary goal that is to minimise the total expected life-cycle cost [1].

It is imperative to have a good understanding of the structural failure and possible scenarios that may induce the structural failure for developing a comprehensive risk assessment framework. Possible scenarios should include the earthquakes since the seismic activities may jeopardise the serviceability of the offshore wind turbine, or even may even lead the structure to reach the ultimate strength due to the large displacement. To this end, the ultimate load carrying capacity of the structure holds utmost significance for both intact and ageing offshore wind turbine structures. The maximum load carrying capacity of a support structure can only be estimated by performing a structural assessment that accounts for the nonlinear effects arising from the material and geometry.

The present work aims to address this topic by carrying out a nonlinear finite element assessment of an ageing monopile offshore wind turbine support structure under seismic loads. It is intended to investigate the structural response during the earthquake while the wind turbine is in the parked condition. A corrosion wastage model is adopted to define the thickness reduction for the ageing monopile structure, and the nonlinear structural response is analysed under seismic activities at different times in the service life of the monopile support structure in question.

A comprehensive risk assessment framework can be only possible by accurate incorporation of consequence measures with the probability of failure, which depends on engineering demand and damage measures. Also, the risk assessment involves various hazardous scenarios which may result in catastrophic consequence. One of the hazardous scenarios is the earthquakes. There have been a number of studies addressing different aspects of the structural response of offshore wind turbine structures subjected to seismic loadings.

Kaynia [2] reviewed some of the key issues in earthquake analysis and design of OWTs. It is stated that the focus has been given to the aero- and hydrodynamic loads; however, the earthquake was a design concern in seismic areas such as China, USA, India, Southern Europe and East Asia.

Katsanos et al. [3] reported another review where experimental studies demonstrating the significance of the seismic hazard have been discussed. Katsanos et al. [4] also reported a study addressing the adverse effects of the multiple exposures on the vulnerability of the 5MW wind turbine at the top. The dynamic assessment of a 5MW offshore wind turbine is considered to be simultaneously subjected to wind, wave and earth ground motion.

Kjørlaug et al. [5] reported that the excitation originated from an earthquake can produce severe vertical accelerations in upper parts of a wind turbine, which emphasise more research on buckling in the steel tower (when combined with wind), Further, it was also concluded that soil-structure interaction could be of importance regarding the displacement along the wind turbine.

Unobe and Sorensen [6] assessed the vulnerability of the wind turbine foundation to multiple hazards occurring simultaneously. It was assumed that there might be a reduction in the structural integrity of structural systems due to fatigue as a result of cyclic wind loading, which increases their vulnerability to additional non-typical loads such as seismic.

Wang et al. [7] also reported a study investigating the structural response of an offshore wind turbine on a monopile foundation in clay subjected to wind, wave and earthquake loads. The study covered a wide range of aspects to study the nonlinear dynamic behaviour of the structure such as wind velocity, induction factor, wave period, peak ground acceleration and soil parameters; however, the numerical model of the structure was limited with the capabilities of beam elements in the nonlinear analysis for large displacement.

Patil et al. [8] investigated the structural performance of a parked wind turbine tower subjected to the strong ground motion. The potential limit states were defined as global buckling of the tower, the first occurrence of yielding, overturning of the foundation and permanent deformation of the tower. It was found that the wind turbine tower investigated in this study is most vulnerable to the overturning in the event of an earthquake. Yielding of the tower is the second most probable failure mechanism, which is followed by the development of permanent deformation and global buckling of the tower.

The structural response of the fixed offshore wind turbines has been an issue of the study not only for the new-built (intact) structure but also for the ageing support one. In this regards, Jahanitabar and Bargi [9] considered the corrosion deterioration by applying a time-dependent model of corrosion deterioration of the tubular elements in the splash zone of a jacket support structure. The incremental dynamic analysis is performed on the intact and corroded platforms considering soil–pile–structure interaction.

Many other studies incorporated the structural assessment into the performance-based assessment to define fragility curve accounting for seismic activities. Risia De Risi et al. [10] developed the finite element model of a monopile offshore wind turbine subjected to natural seismic records with no scaling. Zheng et al. [11] reported an experimental study on the joint earthquake and wave action on the monopile wind turbine foundation. Zareian and Krawinkler [12] applied simple performance-based assessment for design procedure where the focus was given to the mean values of the ground motion intensity, building response, and losses. Abhinav and Saha [13] analysed the structural response of jacket supported OWTs considering a range of soil density from loose to dense and the wind speeds by performing a simplified nonlinear finite element technique known as the Idealized Structural Unit Method (ISUM). However, the parked operation mode when the turbine is shut down to prevent overloading, structural damage or seismic activities were neglected.

The seismic response of the foundations is a very complex process involving interactions between the structural components, interactions between structure soil, the response of the soil to the seismic activities, and the non-linear response of the structure to the seismic load coupled with other environmental loads. The researches cited above and many more aimed to have a correct model to assess the structural behaviour through many interdisciplinary methods have been used together. Nonetheless, these methods neglect several of the factors that may strongly affect response. The present study aims to contribute by performing nonlinear structural response of a monopile OWT subjected to simultaneous wind and seismic loads accounting for the soil-pile interactions. A sophisticated shell model of the monopile is developed where the initial imperfections are introduced. The performed nonlinear finite element analysis accounts for the nonlinearities associated with the material and geometry, and it can capture the progressive collapse of the monopile structure due to the buckling. A time-variant corrosion model is adopted to model the ageing support structure, and lastly, the time histories of the nonlinear structural responses of the ageing monopile are presented.

II. STRUCTURAL DESCRIPTION

The monopile support structure is the most commonly used foundation type for offshore wind turbines. Monopile offshore wind turbines consist of a wind turbine (blades, hub, rotor, and nacelle), tower structure, transition piece, and foundation. The monopile foundation consists of a single large diameter pile with a thick wall thickness that is driven into the seabed by an impact or vibratory hammers. The monopile foundation is connected to the transition piece, which is bolted to the tower structure holding the wind turbine. The foundation transfers the vertical and horizontal loads acting on the offshore wind turbine to the subsoil and constrains the excessive motion of the offshore wind turbine using the soil pressure acting on the foundation. Furthermore, Figure 1 shows a typical monopile offshore wind turbine.

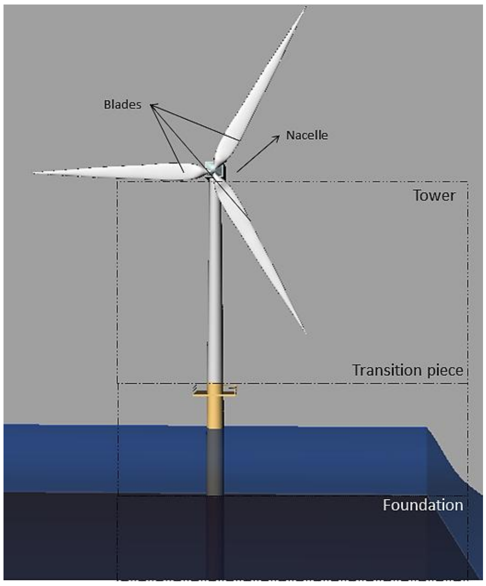


Fig. 1. Monopile offshore wind turbine

Depending on the soil characteristics and the allowable rotation of the pile head, the typical penetration depth for a pile may vary from 15 to 40 m. The present study adopts the monopile OWT optimised by Yeter et al. [14]. Table 1 presents the location of each tubular segment of the monopile OWT from the mean sea level (MSL) together with the thicknesses (t) and the diameters at the top and bottom of the segments (D_{top}, D_{bottom}).

Table 1. Dimensions of the monopile OWT components

	D_{top} (m)	D_{bottom} (m)	t (mm)	From MSL (m)
Tower_1	4.000	4.000	22	+83.76 – +82.76
Tower_2	4.000	4.118	22	+82.76 – +77.76
Tower_3	4.118	4.329	26	+77.76 – +68.76
Tower_4	4.329	4.565	30	+68.76 – +58.76
Tower_5	4.565	4.800	32	+58.76 – +48.76
Tower_6	4.800	5.082	34	+48.76 – +36.76
Tower_7	5.082	5.318	36	+36.76 – +26.76
Tower_8	5.318	5.600	38	+26.76 – +14.76
T. Piece_1	5.318	5.600	50	+14.76 – +8.50
T. Piece_2	5.600	5.600	50	+8.50 – +5.00
T. Piece_3	5.600	5.600	50	+5.00 – -5.40
T. Piece_4	5.600	6.200	50	-5.40 – -11.40
T. Piece_5	6.000	6.000	50	-11.4 – -13.00
T. Piece_6	6.000	6.000	50	-13.00 – -25.00
Pile_1	6.000	6.000	70	-25.00 – -35.00
Pile_2	6.000	6.000	70	-35.00 – -40.00
Pile_3	6.000	6.000	70	-40.00 – -45.00
Pile_4	6.000	6.000	70	-45.00 – -50.00

A 5 MW wind turbine is used on the offshore wind turbine structure. This wind turbine is commonly used in the already installed JOWT structures in various wind farms. The characteristics of the used wind turbine are given in Table 2.

Table 2. REpower 5M Wind turbine specifications

Power	5 MW
Rotor diameter	126 m
Wind class	IEC IIb (DIBt 3)
Weight (hub, rotor, nacelle, blades)	525 tons
Minimum rotor speed	6.9 rpm
Maximum rotor speed	12,1 rpm
Start-up wind speed	3,5 m/s
Nominal wind speed	13 m/s
Maximum wind speed	30 m/s
Minimum hub height	90 m

III. Finite Element Modelling

A. *Modelling of monopile support structure*

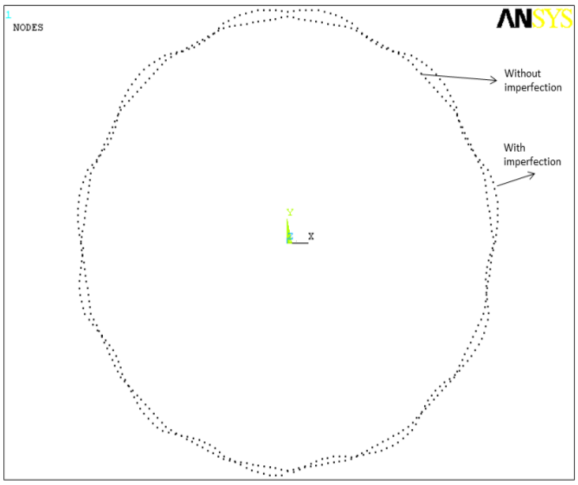
The element type SHELL181 is utilised to model the tubular monopile structure. The element has four nodes with six degrees of freedom at each node: translations in the X, Y, and Z-axes, and rotations about the X, Y, and Z-axes. The element type includes the stress stiffness terms by default and supports the nonlinear material models, which makes the element type well-suited for linear, large rotation, and large strain nonlinear applications. Furthermore, the material model used in the nonlinear FE analysis is a bilinear elastic-perfectly plastic stress-strain relationship where the yield stress is of 355 MPa.

The finite element model using a coarse mesh density may overestimate the ultimate strength due to the over-stiffening. On the other hand, choosing a refined mesh density not only increases the computational time significantly but might also cause a convergence problem. Thus, a mesh sensitivity analysis is carried out based on the relation between the gradient of the ultimate bending moment and element size to identify the optimal finite element size. The result of the sensitivity analysis and the appropriate finite element size is found to be 0.002 m.

The monopile offshore wind turbine is analysed based on the FEM employing the commercial software ANSYS [15]. The FEM model of the monopile OWT involves the tower, transition piece and pile. A very thick plate that is as heavy as the blades, hub and nacelle are modelled at the top of the tower structure in order to account for the overall weight of the wind turbine.

B. *Imperfections*

A structure may suffer from buckling as a result of the imbalanced loading or displacement, which is very difficult to observe in a perfect structure. Both buckling and ultimate strength can be much less in experiments than the predictions made in theory for the perfect structures. Introducing an imperfection to the geometry of a structure is common and the recommended practice, and it results in out of balance load or displacement so that more realistic results can be achieved. Therefore, the present study introduces the geometrical imperfection into the FEM model in order to create a more accurate finite element model by modifying the vertical and horizontal position of the nodes, as can be seen in Figure 2.



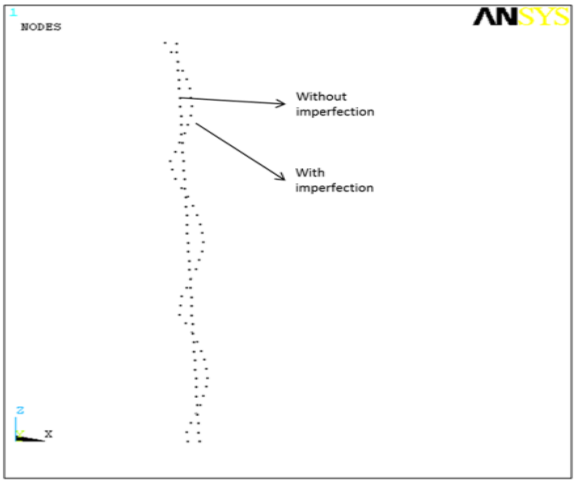


Fig. 2. Imperfection XY (up) and XZ plane (down)

The imperfections are applied based on a function generating superimposed periodic buckling wave shapes throughout the structure. The modified nodal location is defined based on a function superimposing sine and cosine functions onto a 2-D surface. The number of half wave around the circumference of the monopile and the amplitude of the half wave are defined under the consideration of the plate thickness and the length of the support structure.

The model used for the application of imperfections can be improved by implementing a parameter in the sinusoidal functions that account for the phase shift. By doing so, randomness can be involved in the imperfection. However, randomness is not considered in the present study.

C. *Modelling of load application*

For the application regarding the load and boundary conditions, two master nodes are defined at both ends of the FEM model of the tubular monopile structure. These master nodes are connected to the nodes at the bottom and top of the structure through rigid beams, which allow the structure to follow the behaviour of the master nodes. Figure 3 demonstrates the load and boundary conditions applied in the nonlinear FE analysis.

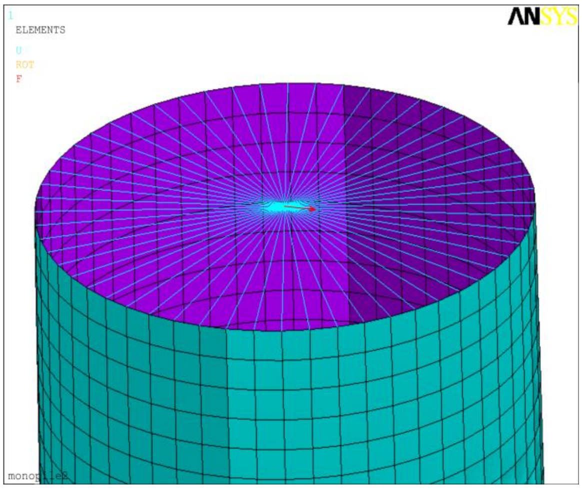


Fig. 3. Load application at the tower top

The wind-induced loadings at tower top are applied to the tower structure through a concentrated load. The concentrated load is applied to the master node at the centre of the circular cross-section. The nodes located at the circumferences of the tower top are connected to the master node by very rigid beams, and these nodes have the same translation and rotation at all dimensions. By using the given modelling technique possible stress concentrations and local failures are avoided.

D. *Modelling of boundary conditions*

In the present study, the developed FE model to carry out the time variant nonlinear analysis takes into consideration the soil-structure interactions (SSI). For the characteristics, the Winkler spring model is adopted, which uses the subgrade reaction forces associated with the soil to define the stiffness of the implemented spring. The subgrade reaction represents the overall soil characteristic accounting for the soil characteristic such as the soil density, soil porosity, shear modulus, and Young's modulus of the soil.

The nodes are connected to the horizontal Winkler spring elements, forming a circular layer. The stiffness coefficients of the springs are calculated for each layer depending on the pile penetration depth, the soil profile and the unit length of the pile. The unit length of the pile depends on the pile length. The number of layers with a series of the Winkler springs is assumed 25.

Similar to the load application, the boundary conditions are applied by employing a master node. These master nodes are connected to the nodes of the tubular shell structure through a rigid beam, which allows the structure to follow the behaviour of the master nodes. Following this, the master nodes at each layer are connected to the Winkler springs to constrain the structure. Figure 4 demonstrates the boundary conditions applied in the nonlinear FE analysis.

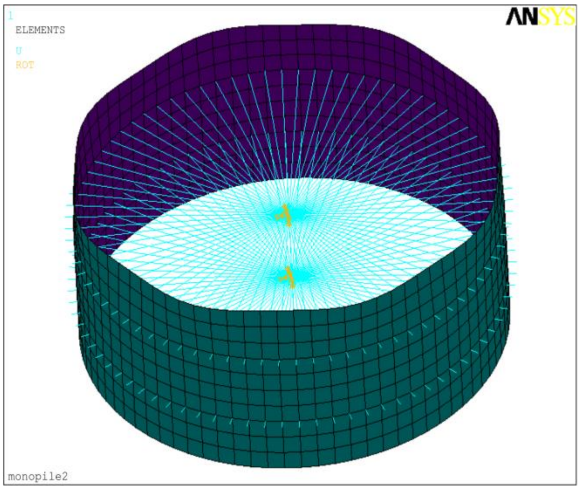


Fig. 4. Master node application for the boundary conditions

Solid particles and water are the principal components of the soil. Depending on the geological history of the soil, the size of these particles varies from soft clay to hard sand and boulder rock, which defines the soil characteristics and therefore the local constraint and strength. A wide range of soil characteristics exists in the coastal waters where OWTs have been installed. The geotechnical investigations on the soil strength and profile are carried out before the construction of an offshore wind turbine structure. The geotechnical investigations usually involve topographic, subsurface, seismic surveys, which define the feasibility to install an offshore wind farm. Also, the cone penetration testing is carried out in order to predict the behaviour of the soil.

It is a common practice for offshore foundations to use a linear distribution function to model the soil profile through the pile penetration depth based on the assumption that the soil stiffness increases directly proportional to the pile penetration depth, which means that the soil stiffness increases as moving towards to more profound in the soil. As a result of this assumption, the $p - y$ curves for different depth can be obtained. The tangent of each curve represents the elastic modulus of the soil at a given depth. The required load per a unit pile length p increases as the pile penetrates deeper into the subsoil.

The present study employs a different approach regarding the change in the soil profile with the depth since the soil of a possible offshore wind farm site does not necessarily need to be linear it can be of a parabolic shape. In this study, the standard logistic sigmoid function is employed to address this issue. The sigmoid function can also provide the linear soil profile; however, the sigmoid function can also explore various soil profiles by manipulating the shape factor, which is essential for the performed uncertainty analysis.

The standard logistic sigmoid function is a mathematical relationship forming a number of different shapes, which is very useful to fit data. The function is described in the literature as a bounded differentiable real function that is defined for all real input values and has a positive derivative at each point [16]. The sigmoid function is expressed as:

$$g(x) = \frac{1}{1+e^{-(Wx-B)}} \tag{1}$$

where x is the distance from the seabed surface. $g(x)$ is the function of the variation of the soil characteristics along the depth, and it varies from 0 to 1. W is the shape factor and B is the scale factor. The shape factor W is associated with the steepness of the curve denoting how the soil stiffness changes with the distance from the seabed surface. The scale factor B is associated with the soil stiffness at the seabed surface.

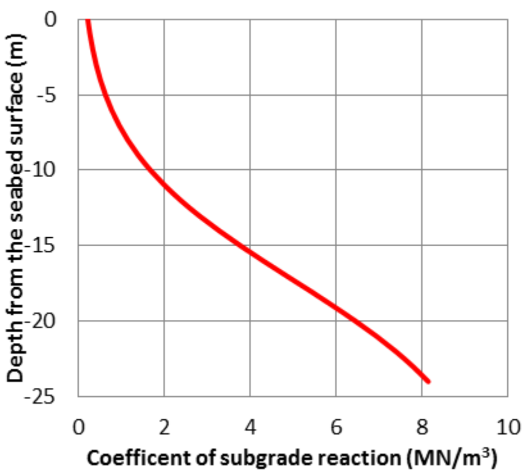


Fig. 5. Soil profiles as a function of the depth

The monopile foundation must be driven into the subsoil as much as it is needed in order to guaranty support to the offshore wind turbine as required by the design criteria. This means that in order to achieve solid support the pile has to be driven until it reaches a soil layer of very dense sand or rock. This affects the pile length, thereby the structural stiffness.

Therefore, by defining the penetrated pile length depending on the shape factor W in the sigmoid function, the effect of the soil-pile interaction on the structural natural frequency can be analysed more realistically.

If the behaviour of each spring is assumed linear and the soil reaction corresponding to the lateral displacement of the pile can be formulated as [17]:

$$p = E_{py} y \tag{2}$$

where p is the soil resistance regarding force per unit length, y is the lateral pile deflection, and E_{py} is termed as a reaction modulus and represents the slope of the $p - y$ curve. As far as a linear soil profile is concerned, the reaction modulus E_{py} may be expressed as:

$$E_{py} = K_{hs} z \tag{3}$$

whereas for the nonlinear soil profiles defined based on the sigmoid function can be ex-pressed as:

$$E_{py}(z) = K_{hs} \frac{1}{1+e^{-(Wz-B)}} \tag{4}$$

where z, W and B are the distance below the soil surface, the shape factor of the sigmoid function and the scale factor of the sigmoid function, respectively. K_{hs} is the coefficient of subgrade reaction.

IV. Nonlinear Finite Element Analysis

The OWT support structures are not only subjected to tensile, but also compressive loading, which may result in local or global instability of the structure. To design the support structure ensuring the lightest acceptable design that can the demanded ultimate load, the ultimate strength assessment is of necessity.

The ultimate strength of the monopile OWT structure is numerically analysed based on the finite element method (FEM) by performing a nonlinear finite element (FE) analysis. The nonlinearities that are associated with the material and structural geometry are considered. Using the nonlinear FE analysis, it is possible to observe the phases that the structure goes through under progressive load such as proportional limit, buckling, ultimate strength and post-collapse.

The nonlinear FE analysis is carried out using the commercial software ANSYS. The large deformation option is activated to solve the geometric and material nonlinearities and to pass through the extreme points. The full Newton–Raphson equilibrium iteration scheme is utilised in the nonlinear FE analysis. However, the Newton-Raphson approach attempts to solve by a linear increment with only positive slopes, which is a source of a convergence problem when the slope of the load-deflection curve becomes zero or negative. The arch-length method is adopted to solve the convergence problem by introducing an arc instead of line to converge the Newton-Raphson equilibrium, and following the post-collapse behaviour can be obtained.

The nonlinear structural behaviour is path-dependent. Therefore, it is necessary to choose the incremental load step as small as possible that the model follows the load carrying path as closely as possible; however, extensive computation effort must be avoided. To this end, the minimum time stepping is set to be 20 meaning that the solver may decrease the load step if necessary depending on the convergence of the solution.

In the present study, the structural response of the transition piece is discussed at four points. The first point is associated with the proportional limit where the structure follows a linear force-displacement relationship. The second point is where the deformation becomes obvious. The point 3 is associated with the ultimate strength of the transition piece, and the last point accounts for the failure. These points are illustrated in Figure 6 on a force-displacement relationship of a transition piece design.

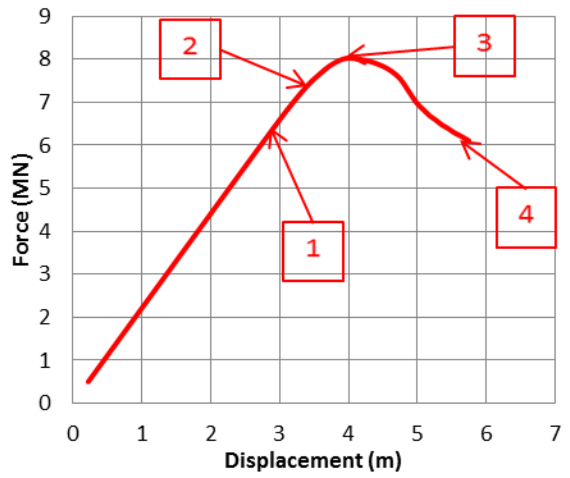


Fig. 6. Force-displacement diagram of the monopile OWT

Figure 7 shows the deformation shape after reaching the ultimate strength. The local buckling failure occurs at the bottom of the monopile structure, and as the structure continues to be subjected to the bending moment, the deformation becomes more evident. The ovalisation of the circular cross-section enforces the further stiffness reduction and the structural loses the capacity to carry the more loads. Due to the resulting plastic hinge the monopile OWT bends more reaching significant displacement values (see Figure 8).

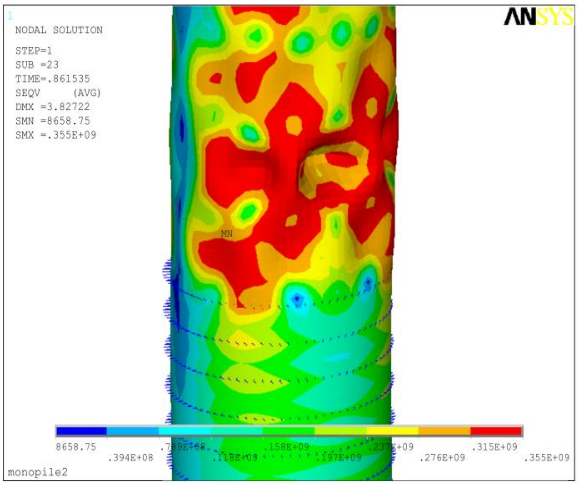


Fig. 7. Deformation shape of local buckling

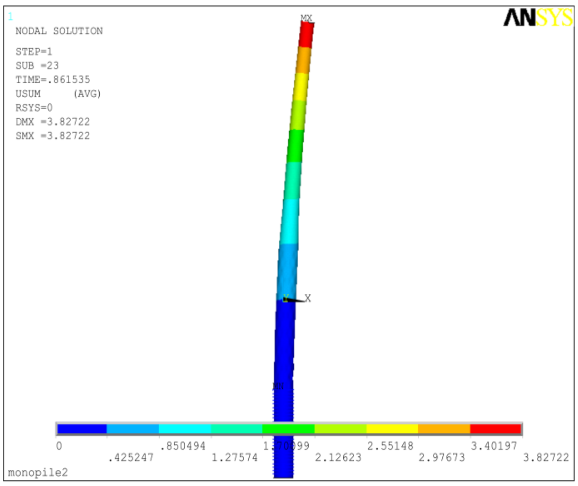


Fig. 8. Displacement distribution on the deformed monopile

V. NONLINEAR STRUCTURAL RESPONSE EARTHQUAKE

A. *Seismic load application*

Earthquakes are known to be a consequence of active tectonic movements, and they occur mostly on plate boundaries [18] The plate boundaries can be divided into three zones as divergent zones, convergent, transform zones. Most of the seismic energy release, inter-plane earthquake, is explained by the tectonic theory. Shallow and intermediate earthquakes happen at convergent zones in bands can be quite broad and vast; whereas, divergent plate boundaries form narrow bands of moderate shallow earthquakes [19].

Elastic strain energy accumulates and suddenly releases a rupture as the groundmasses move together. As the distorted plates crack for the energy equilibrium, an earthquake ground motion is created. When the earthquake occurs due to the sudden fault slip, seismic waves travel from the focal point to the observed site. The slip of the plates can take place horizontally, vertically or both. As a result, two types of seismic waves are generated, which are known as body and surface waves.

Body waves (primary waves) are longitudinal (primer) and transversal (secondary) waves. The primary waves travel very fast; however, they have a little damage potential, whereas the secondary waves lead to horizontal and vertical motions, and both can cause substantial damage. At the earth, body waves manifest as surface waves such as the Love waves and Rayleigh waves. While the body waves equally represented in the seabed at all depths, the surface waves most likely occur in shallow earthquakes. The ground motions are the combination of the mentioned waves occurred, as a result, the energy release, and they can be measured regarding displacement, velocity and acceleration [19].

The present study adopts the displacements in three dimensions as the ground motions of the earthquake studied for the “Gulf of Thailand”. The given ground motions are associated with the scaled seismic data which was measured during the earthquake occurred on September 28, 2004, on the San Andreas Fault near Parkfield, California, with a magnitude 6.0. The scale factor is 0.297 and pulse period is 1.078.[20].

The ground motions in X, Y, and Z-axes are given in Figure 9-11. It can be seen that the prevailing displacement of the seabed surface takes place in Y-axes. A conservative approach is adopted to be able to assess the worst possible scenario; therefore, the applied seismic and wind-induced loads are considered to be unidirectional.

Although it is a crude assumption that the ground motion would be the same for two earthquakes of similar magnitude at different locations, the primary objective of the study is to perform an exemplary assessment how to analyse the nonlinear structural response of an offshore wind turbine structure subjected to simultaneous loads such as seismic and wind-induced loads.

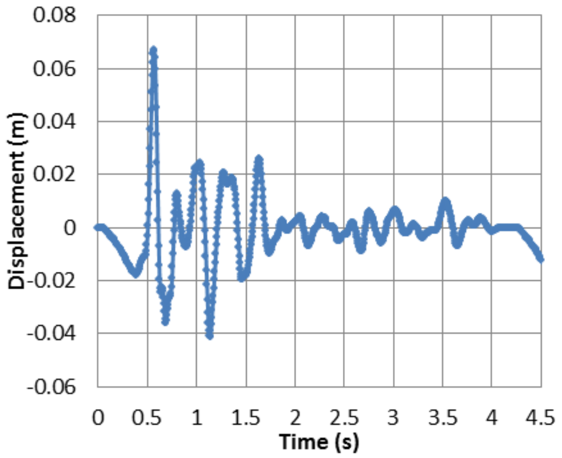


Fig. 9. Ground motion at the seabed surface, displacement in X-axis

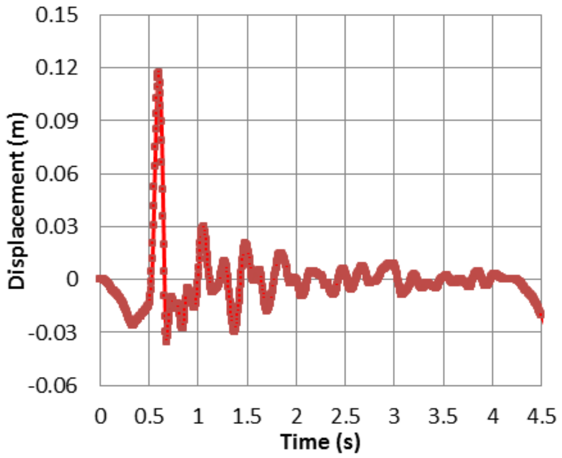


Fig. 10. Ground motion at the seabed surface, displacement in Y-axis

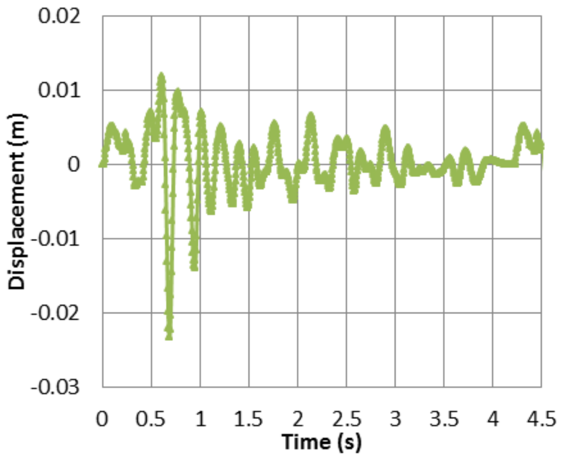


Fig. 11. Ground motion at the seabed surface, displacement in Z-axis

These waves travel with a varying velocity depending on the characteristics of the trough material they travel. Therefore, it is reasonable to assume the motion of the seabed at different levels of the depth in proportion to the soil profile. The displacement of the soil at different depth is then estimated based on the soil profile defined by the sigmoid function, which is also used to define the stiffness of the Winkler springs (see Figure 5).

The time-domain nonlinear structural response of the monopile OWT structure during the earthquake is given in Figure 12. As a result of the coupled wind and seismic load, the maximum displacement of 0.518 m occurs at the tower top of the offshore wind turbine; however, the resulting displacement is not sufficient to cause local buckling at the bottom monopile near seabed surface.

The maximum von Mises stress of 180 MPa occurred at the bottom of the transition piece, which is low enough not to suffer from the local buckling. This is mainly because the wind turbine is idle in the parked condition after detecting the seismic activity.

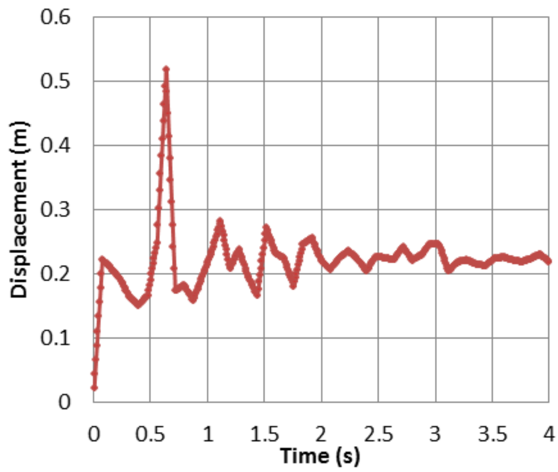


Fig. 12. Structural response of the monopile OWT during the earthquake

The structural response presented in Figure 12 is associated with an intact monopile OWT at the beginning of its service life. Here in this study, it is also aimed to assess the response to the seismic and wind loads for a degrading monopile OWT at different years of its service life, which is discussed in the following section.

VI. SEISMIC RESPONSE OF DEGRADING MONOPILE

A. *Corrosion model*

Three fundamental approaches are typically applied for corrosion deterioration modelling. The conventional approach is considered that corrosion grows linearly, which may lead to a very big overestimation of the corrosion deterioration or underestimation of the corrosion effects in early life. Garbatov and Guedes Soares [21] developed the corrosion model that is based on a non-linear time-dependent function of general corrosion wastage. The model is based on the solution of a differential equation for the corrosion wastage, which leads to the mean value of the corrosion depth as:

$$d_{corr}(t) = \begin{cases} 0 & t \le \tau_c \\ d_\infty \left[1 - \exp\left\{ -\dfrac{t-\tau_c}{\tau_l} \right\} \right], & t > \tau_c \end{cases} \tag{5}$$

The parameters of the corrosion depth as a function of time are determined under the assumption that it is approximated by the exponential function. The long-term corrosion wastage for monopile structure near the splash zone as d_∞= 4 mm. The time without corrosion τ_c is assumed to be 5 years and the transition period τ_l is assumed to be 12. years.

Figure 13 represents the maximum displacement that might occur at the tower top of the monopile OWT as a result of simultaneous wind-induced and seismic loads, at a different time of the service life. As the corrosion depth increases with time the structural stiffness reduces, as a result, the monopile is subjected to higher displacements under external loads.

Fig. 13. Structural response of the monopile subjected to seismic loads and corrosion degradation

The results presented in Figure 13 also indicate that as the monopile structure remains below the proportional limit, the trend that the maximum displacement follows over the years shows similarity with the increase of the corrosion depth in function of time.

Furthermore, the maximum displacement is still far from reaching the ultimate strength even after 25 years of service life being subjected to a corrosive environment.

VII. Conclusions

The present work investigated the seismic response of the monopile structure taking into account the interactions between the structural components, the interactions between structure soil, the response of the soil to the seismic activities, and the non-linear response of the structure to the seismic load coupled with other environmental loads. A sophisticated shell model of the monopile was developed where the initial imperfections were introduced. The performed nonlinear finite element analysis accounted for the nonlinearities associated with the material and geometry, and it could capture the progressive collapse of the monopile structure due to the buckling. A time-variant corrosion model was adopted to model the ageing support structure, and lastly, the time histories of the nonlinear structural responses of the ageing monopile were presented.

The maximum displacement at the tower top is estimated as 0.518 m for the intact monopile OWT structure subjected to the couple wind-induced load, soil-pile interactions and the ground motions. However, the calculated displacement was not sufficient to cause local buckling. Moreover, the maximum von Mises stress was calculated 180 MPa for the OWT structure including the pile component, which is lower than the buckling limit. It was predicted that the monopile structure would not experience any local buckling as a result of the seismic loads.

The results indicated that the maximum displacement occurring at the tower top increases with the level of the corrosion degradation. Because the stress level occurred throughout the monopile OWT structure remains below the proportional limit, the trend that the maximum displacement follows over the years shows similarity with the increase of the corrosion depth in function of time.

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