Liquefaction effects on the fundamental frequency of monopile supported offshore wind turbines (OWTs)

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Abstract
Offshore Wind Turbines (OWTs) are dynamically sensitive structures and as a result estimating the natural frequency of the whole system taking into effect the flexibility of the foundation is one of the major design considerations. The natural frequency is necessary to predict the long-term performance as well as the fatigue life. OWTs are currently being constructed in seismically active regions such as the United States, China, India and Taiwan and there is a need for design considerations for OWTs located in seismic zones. Though OWTs are relatively less sensitive to earthquakes than typical structures, liquefaction of soil surrounding monopiles may have serious implications on the dynamic performance of the system. During liquefaction there is a loss of support along the depth of the monopile which noticeably reduces the stiffness of the system. This results in a decrease of the fundamental frequency and may cause resonating effects with high energy wave loads up until liquefaction effects are reduced with time. Therefore, these resonating effects will increase the fatigue damage and/or the overall accumulated rotation of the structure. Therefore, in this paper, the effect of liquefaction on the natural frequency of OWTs was considered through a parametric study which considers various soil conditions, liquefaction depths (D_L), monopile diameters (D) and monopile lengths (L_P). The finite element method used in this parametric study was validated via an analytical solution available in literature and a large-scale experiment which considers a fixed base condition. The study shows that the change in the natural frequency under various liquefaction depths is not significantly influenced by monopile diameter when the length is kept constant. On the other hand, the monopile length has a more detrimental effect and the aspect ratio is that L_P/D is a significant non-dimensional term governing dynamic behaviour of OWT systems in the case of liquefaction. In addition, it is shown that the overall results are more sensitive in looser soils and special design considerations are provided for different ground conditions. Finally, based on analytical solutions provided in literature, a simplified method is presented to estimate the change of the natural frequency taking into account the liquefaction depth which can be easily programmed in a spreadsheet type program followed by a step-by-step solved example which also serves the purpose of verification and validation.

Keywords Liquefaction · Offshore wind turbines · Natural frequency · Monopile

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1 Introduction

The growth of world population resulted in the increase in the global energy demand. Fossil fuels are still the primary sources of energy production and the amount of green-house gas (GHG) emissions is increasing continuously. The amount of harmful gases released and carbon emission into the atmosphere has directed governments to invest more on renewable energy sources (Nikitas et al. 2019). In recent years, offshore wind has become an attractive way of producing electricity due to less influence on communities, no land requirement and high wind speeds in offshore locations.

Offshore wind turbines (OWTs) are being further utilised in Taiwan, US, India, and China which are situated at seismically active regions. Therefore, there is a serious need for seismic design considerations for wind turbines. Natural vibration periods of offshore wind turbine systems are in the range of 3 s as mentioned in the study of Bhattacharya and Goda (2016). The natural periods of these systems are significantly longer than the dominant spectral content of the recorded ground motions. Since there is no overlapping of the periods of the wind turbine systems and the ground, these structures will not resonate. As a result, it can be stated from site observations that offshore wind turbine systems are relatively insensitive to earthquake. However, earthquake-induced effects such as liquefaction may cause several damages to the wind turbine systems.

Estimation of natural frequency of wind turbine systems is an important design requirement as it dictates dynamic load factors and the dynamic amplification factor of the response. The natural frequency of these structures is dependent on structural and foundation parameters. Table 1 shows the details of NREL 5 MW, LW 8 MW, DTU 10 MW and Haliade-X 12 MW turbines. Normalized power spectrum of main forcing frequencies and different design approaches considering a reference 8 MW wind turbine is shown in Fig. 1. Frequencies of 1P (load caused by aerodynamic and mass imbalances of the rotor, whose frequency equals to rotor rotational frequency) and 3P loading (loads due vibrations caused by blade shadowing effect) for 5 MW, 8 MW and 10 MW turbines and their minimum target frequencies are also shown in Fig. 1. The main forces on an offshore wind turbine are wind, wave, 1P, and 3P loadings and their forcing frequencies can be clearly seen in the figure. It is clear from the

Table 1 Summary of NREL, LW, DTU and Haliade-X wind turbine characteristics

| Turbine       | NREL | LW | DTU | Haliade-X | Unit |
|--------------|------|----|-----|-----------|------|
| Rating       | 5    | 8  | 10  | 12        | MW   |
| Rotor diameter | 126  | 164| 178.3| 218.2     | m    |
| Hub height   | 90   | 110| 119 | 135       | m    |
| Rotor speed range | 6.9–12.1 | 6.3–10.5| 6–9.6| 7.81     | rpm  |
| Cut-in, Rated | 3, 11.4 | 4, 12.5| 4, 11.4| 3.5     | m/s  |
| Cut-out wind speed | 25   | 25 | 25  | 28        | m/s  |
| Nacelle mass | 296.78| 375| 551.56| 600     | Tonnes |
| Blade mass   | 17.74| 35 | 41.72| 55        | Tonnes |
| Tower mass   | 347.46| 558| 605  | 2500      | Tonnes |
| Tower height | 87.6 | 106.3| 115.6| 129.1     | m    |
| Tower top diameter | 3.87 | 5  | 5.5  | 5.5       | m    |
| Tower bottom Diameter | 6   | 7.7| 8    | 8         | m    |
figure that engineers have to select a design frequency in order to avoid the resonance of the OWT. Generally, the target natural frequencies of the OWT system are situated between 1 and 3P loading (Soft-Stiff). A black point with red circle in Fig. 1 shows the target natural frequency of the OWT system. In the case of liquefaction, foundations of OWTs lose their supports along the depth of liquefaction which results in a decrease in the natural frequency of the entire system (the “Soft-Stiff” design point is shifting to the left). The decrease in the natural frequency of the system may be critical impacts on the design. Until liquefied soils gain their stiffness back, the natural frequency of the system may match with the frequency of wave loads. This cause the OWTs resonate and the resonating turbine may tilt (SLS criteria).

In this study, a Finite Element (FE) model was developed to investigate the effects of liquefaction on the natural frequency of Offshore Wind Turbines (OWTs) with monopile foundations. The developed numerical model was calibrated through an analytical methodology proposed by Arany et al. (2016) and an experimental study (Prow-ell et al. 2009) in which fixed base condition was used.

Consequently, a parametric study was conducted to investigate the effects of liquefaction on the natural frequency of Offshore Wind Turbines (OWTs) with monopile foundations under varying liquefaction depths. The analytical methodology suggested by Arany et al. (2016) was used to estimate the natural frequency of OWTs with monopile foundations in non-liquefiable soils.

This analytical methodology was modified to predict the natural frequencies of such structures in liquefiable soils. The modified analytical methodology was calibrated via numerical results under varying liquefaction depths. A worked example explaining the modified methodology is also included in the Appendix.

It is important to note such analytical methods are best suited for the concept design stage where the commercial viability of monopiles in seismic areas is being assessed. This is particularly useful when information is limited about the ground conditions and metocean data. For further optimised monopiles, detailed Finite Element Analysis (FEA) is required.
2 A brief literature review on seismic consideration of offshore wind turbines

Estimation of natural frequency of Offshore Wind Turbine (OWT) + foundation systems is an important design calculation so that dynamic amplifications due to resonance can be avoided. Arany et al. (2016) developed a closed form solution to calculate natural frequency of monopile supported OWT by taking into account soil-structure interaction (SSI) effects and transition piece stiffness. Their analytical model is based on previous research on monopile supported OWTs where mass of rotor and nacelle assembly is modelled lumped mass on the top of tower, the tower is modelled as Euler–Bernoulli beams and it is supported on a set of macro-element springs that represent the foundation (Adhikari and Bhattacharya, 2011, 2012). Stiffness values of these macro-element soil springs for deep foundations exhibiting rigid and flexible behaviour are computed by using various closed form solutions available in the literature (Randolph 1981; Carter and Kulhawy 1992; Pender 1993; Higgins and Basu 2011; Shadlou and Bhattacharya 2016 and Jalbi et al. 2017). The macro-element spring method is highly beneficial in modal analysis and load-deformation analysis at low strains and it has a simple computational capabilities. Apart from the analytical methods, Finite Element (FE) models have been used to calculate natural frequency of whole system.

There are only few studies that focused on soil liquefaction effects on the performance of OWTs resting on monopile foundations (Wang et al. 2019; Esfeh and Kaynia 2020). Wang et al. (2019) investigated seismic performance of an innovative hybrid monopile foundation via a series of centrifuge tests. Various hybrid foundation models with varying foundation thicknesses and diameters were tested for the comparison. The performance of the hybrid foundation system under soil liquefaction in terms of lateral displacements and settlements was also investigated through experiments. Esfeh and Kaynia (2020) investigated the use of advanced numerical modelling in assessment of the effects of liquefaction on Offshore Wind Turbines resting on the monopile and caissons. The performance of caissons and monopiles under combined actions of liquefaction and wind loads was investigated by computing earthquake induced displacements (rotational and horizontal movements of foundations), the excess pore water pressure build-up and the shear stress–strain responses at various points in the soil medium surrounding caisson and monopile foundations. As seen in these works, the performance of OWTs was investigated in terms of settlements, lateral and rotational displacements. The change in the natural frequency of the system due to soil liquefaction was not studied.

Natural frequency of OWT and foundation system reduces due to loss of foundation support during the soil liquefaction. The reduced natural frequency may match with forcing frequencies and this may result in resonance of the whole system. As reviewed in the literature, there is limited work on liquefaction effects on monopile supported OWTs in terms of natural frequency changes with varying liquefaction depth. There is an urgent need to investigate the effects of soil liquefaction on natural frequency change of OWTs resting on monopile foundations and to develop a simple methodology to predict changes in natural frequency due to liquefaction for preliminary design of OWT structures.

3 Model validation

Wind turbines are commonly modelled as a beam-like column with a lumped mass on the top to represent the blades, nacelle and rotor. This modelling approach has been used in this study. The software SAP2000 is used for numerical analysis and developed FE models
are validated by using existing test data and analytical methods. Two different case studies were considered for experimental validation and analytical validation. For the experimental validation, the results of a large-scale experiment available in the literature were considered. In the large-scale experiment, a 65-kW wind turbine model with fixed base conditions was used. For the analytical validation, a reference 8 MW wind turbine supported by a monopile foundation were considered. These two different case studies were used for the validation of the developed Finite Element (FE) models in both experimental conditions and field conditions.

3.1 Experimental validation

It is essential to validate Finite Element (FE) models through experiments or analytical methods to obtain reliable results. The accuracy and validity of FE results can be significantly improved by using experimental data to calibrate numerical models. In this study, the test results of a 65-kW wind turbine that manufactured by Nordtank in Denmark is used to validate the developed FE model. The schematic illustration of the 65-kW wind turbine is demonstrated in Fig. 2. The tower is made of three discrete hollow cylindrical sections: (1) Upper tower section, (2) middle tower section, and (3) lower tower section. The height of these sections is 6.0 m and the height of transition pieces (tapered cross-section) between these sections is 1.9 m. The lower tower section is a cylindrical hollow section with 2.0 m diameter and 6.0 mm wall thickness. The diameter and wall thickness of the middle tower section are 1.6 m and 6.0 mm, respectively while the diameter and wall thickness of the upper tower section are 1.2 m and 6.0 mm, respectively. A summary of other relevant engineering properties of the tested turbine is presented in Table 2.

The tower is modelled using beam-column elements with bending stiffness based on corresponding cross-section at the centre of each element. Each section (lower tower, middle tower, and upper tower) are divided into 20 beam-column elements. Transition pieces are modelled as tapered section by using non-prismatic sections in SAP2000. The nacelle, rotor and blades are modelled as a lumped mass on the top of the tower. The natural frequencies computed by SAP 2000 are presented in Table 3. The Finite Element (FE) of the wind turbine, first and third global mode shapes are demonstrated in Fig. 3. The values in Table 3 indicates that the FE model developed by SAP2000 predicts the behaviour of the turbine quite well, considering experiment results provided by Prowell et al. (2009). Classical mode shape of a cantilever beam-column with lumped mass at the top is clearly observed from the corresponding mode shapes shown in Fig. 3.

3.2 Analytical validation

A reference 8-MW wind turbine whose properties are given in Table 1 is modelled by using SAP2000. The length of the tower, the bottom diameter and the top diameter of the tower are 106, 8 and 5 m, respectively. The wall thickness of the top of the tower is 22 mm while the wall thickness of the bottom of the tower is 36 mm. The total mass of nacelle, rotor and blades is 410 tons. The unit weight of the tower material is taken as 7860 kg/m³. The mass of nacelle, rotor and blades is applied at the top of the tower as a lumped mass. Figure 4 shows FE models of 8-MW wind turbine with fixed base and 8 m diameter monopile with 30 m length. The monopile has a hollow section and its wall thickness is 35 mm. The monopile is assumed to be placed in homogeneous sand with 30° internal friction angle. Soil-pile interaction is modelled using axial and lateral soil
springs suggested by API codes (API 1993). The lateral soil resistance (\( p \))—deflection (\( y \)) relationships for sand are non-linear and the relationships may be approximated at any specific depth \( H \) by Eq. 1. The axial soil resistance (\( t \))—deflection (\( z \)) relationships are calculated by using Eq. 2 and Table 4.

**Fig. 2** Wind turbine configuration for the full-scale shake test conducted at UCSD

**Table 2** Properties of the 65-kW Wind Turbine

| Property                        | Value   |
|---------------------------------|---------|
| Rated power                     | 65 kW   |
| Rated wind speed                | 9.4 m/s |
| Rotor diameter                  | 16.0 m  |
| Rotor hub height                | 22.6 m  |
| Bottom tower segment weight     | 27.8 kN |
| Middle tower segment weight     | 19.8 kN |
| Top tower segment weight        | 12.5 kN |
| Nacelle weight                  | 26.2 kN |
| Hub weight                      | 6.8 kN  |
| Average blade weight (each)     | 3.1 kN  |
where $A$ is factor to account for cyclic or static loading condition, $p_u$ is ultimate soil resistance, $k$ is initial modulus of subgrade reaction, $H$ is depth and $y$ is deflection.

$$P = A \times p_u \times \tanh \left( \frac{k \times H}{A \times p_u} \times y \right)$$  

(1)

$t = \pi \times K \times p_0 \times \tan \delta \times D$ 

(2)

where $t$ is axial soil resistance, $K$ is coefficient of lateral earth pressure, $p_0$ is effective overburden pressure, $\delta$ is friction angle between the soil and pile wall, $D$ is pile diameter, $z$ is relative displacement between pile and soil, and ultimate axial soil resistance.
The fixed base natural frequency of the wind turbine structure can be calculated by using Eq. 3.

\[ f_{fb} = \frac{1}{2\pi} \sqrt{\frac{3E_T I_T}{(m_{RNA} + \frac{33}{140} m_T) L_T^3}} \]  

(3)

where \( f_{fb} \) is natural frequency of the wind turbine with fixed base, \( E_T I_T \) is flexural stiffness of the tower, \( m_{RNA} \) is total mass on the top of the tower (including nacelle, blades, and hub), \( m_T \) is mass of the tower, and \( L_T \) is tower length.

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**Table 4** Axial Load Transfer (t-z) Curves

| Sand | z (m) | \( t/t_{\text{max}} \) |
|------|------|------------------|
| 0.000 | 0.00 |
| 0.0254 | 1.00 |
| \( \infty \) | 1.00 |

---

Fig. 4 Finite Element model of 8-MW wind turbine a with fixed base b with 8 m diameter monopile.
Figure 5 shows the mechanical model of a wind turbine along with macro element springs that represent monopile foundation and lumped mass that represent mass of blade, nacelle and rotor. The foundation stiffness is shown by $K_V$ (vertical stiffness), $K_L$ (lateral stiffness), $K_{LR}$ (cross-coupling) and $K_R$ (rocking stiffness). $K_L$, $K_R$ and $K_{LR}$ are obtained by using appropriate closed-form solutions that are mentioned in literature review. $K_V$ (vertical stiffness) is not taken into account for simplified calculations since the structure is stiff in vertical direction.

The natural frequency of OWT + monopile foundation can be calculated by using Eq. 4 that proposed by Arany et al. (2016).

$$f_0 = C_L \times C_R \times C_S \times f_{fb}$$

(4)

where $f_0$ is the flexible natural frequency of the offshore wind turbine structure, $C_L$ is the foundation flexibility factor, $C_R$ is the foundation flexibility factor, and $C_S$ is the substructure flexibility coefficient to account for the enhanced stiffness of the transition piece.
method to calculate foundation flexibility factors \(C_L\) and \(C_R\) and substructure flexibility coefficient \(C_s\) can be found in the study of Arany et al. (2016).

The foundation flexibility factors \(C_L\) and \(C_R\) are expressed in terms of the non-dimensional foundation stiffness values:

\[
\eta_L = \frac{K_L L^3}{EI}, \quad \eta_{LR} = \frac{K_{LR} L^2}{EI}, \quad \eta_R = \frac{K_R L}{EI}
\]

(5)

where \(\eta_L\) is non-dimensional lateral stiffness, \(\eta_{LR}\) is non-dimensional cross coupling stiffness, \(\eta_R\) is non-dimensional rotational foundation stiffness, \(K_L\) is lateral stiffness of the foundation, \(K_R\) is rotational stiffness of the foundation, \(K_{LR}\) is cross-coupling stiffness of the foundation (see Arany et al. 2016).

Using the calculated non-dimensional stiffness values, the foundation flexibility factors \(C_L\) and \(C_R\) are calculated as:

\[
c_R = 1 - \frac{1}{1 + 0.6 \left( \frac{\eta_R}{\eta_L} - \frac{\eta_{LR}}{\eta_L} \right)}, \quad c_L = 1 - \frac{1}{1 + 0.5 \left( \frac{\eta_L}{\eta_R} - \frac{\eta_{LR}}{\eta_R} \right)}
\]

(6)

The distance between the bottom of the tower and mudline is \(L_s\), bending stiffness of the monopile is \(E_I P\). The substructure flexibility coefficient \(C_s\) is expressed in terms of two dimensionless parameters, length ratio \(\psi\) and bending stiffness ratio \(\chi\):

\[
\psi = L_s / L_T, \quad \chi = (E_T I_T) / (E_I P)
\]

(7)

\[
c_s = \sqrt{\frac{1}{1 + (1 + \psi)^3 \chi - \chi}}
\]

(8)

Table 5 shows the values of natural frequencies of reference 8-MW wind turbine comparing with the analytical solution. The values calculated by SAP2000 exhibits quite a good match with the analytical solution. The numerical model predicted the natural frequency for fixed base with a 3% error while the error between the analytical and numerical model with monopile foundation is 6.73%. These results indicate that the numerical model is able to capture modal behaviour of the OWT system.

### 4 Methods to model liquefied springs

Two different approach were used to model liquefied soil in this study: Case I and Case II as seen in Fig. 6. Figure 6 shows a Beam on Non-linear Winkler Foundation (BNWF) with axial \(t-z\) and lateral \(p-y\) soil springs for Case I and Case II. In Case I, liquefied soil is modelled...
by removing soil springs in the liquefaction zone. In Case II, liquefied soil is modelled by using a practical method proposed by Lombardi et al. (2017) and Dash et al. (2017) for construction of p-y curves (lateral soil springs) for liquefiable soils. The p-y curve after full liquefaction is represented by four parameters such initial lateral resistance \( p_1 \), initial lateral displacement \( y_1 \), ultimate lateral resistance \( p_u \) and ultimate lateral displacement \( y_u \) as seen in Fig. 6. These parameters are calculated by using Eqs. 9–12.

\[
p_1 = N_s \times 1.25 \times \gamma_{to} \times G_1 \times D
\]

\[
y_1 = \frac{1.25 \times \gamma_{to} \times D}{M_s}
\]

\[
p_u = N_s \times \tau_{max} \times D
\]

---

**Fig. 6** Beams on Non-linear Winkler Foundation (BNWF), a no-liquefaction case, b p-y curves for liquefied soils in Case I, and c p-y curves for liquefied soils in Case II
where \( N_s \) and \( M_s \) are scaling factors, \( \gamma_{to} \) is take-off strain, \( G_1 \) is initial shear modulus, \( D \) is pile diameter, \( \tau_{\text{max}} \) is ultimate shear strength and \( G_2 \) is critical state shear modulus. Readers are referred to following studies for detailed information on the construction of p-y curves for liquefiable soils (Lombardi et al. 2017; Dash et al. 2017). \( N_s \) is taken as 9.2 (smooth interface) and \( M_s \) is taken as 1.87 for fully liquefied soil.

Figure 7 shows the variation of normalized natural frequency \((f/f_0)\) of OWT + monopile system with normalized liquefaction depth \((D_L/L_p)\) for Case I and Case II. The natural frequency of the system under no liquefaction condition is denoted by \( f_0 \) and \( f \) is for the natural frequency of the system under various liquefaction depths \((D_L)\). The monopile length is denoted by \( L_p \). In the FE models, the reference 8 MW wind turbine whose properties are given in Table 1 and 6 m diameter monopile with 30 m length are used. The comparison of normalized natural frequencies for Case I and Case II shows that the results are of close proximity to each other.

For simplicity, it is concluded that removing soil springs in the liquefaction zone instead of using liquefied soil springs in this zone will have minimal influence on the modal analysis results. Therefore, in the parametric study, liquified soil was modelled by removing soil springs in the liquefaction zone.

Material properties such as the internal friction angles considered in the parametric study were used for soils outside the liquefaction zone. No friction angles were considered for liquefiable soils in the parametric study since the soil springs in the liquefaction zone were removed in the analysis.

However, in the detailed design stage, an indepth analysis of the liquefied soil material properties such as the liquefiable friction angle needs to be considered for more optimised monopile design.

\[
y_u = \left( 1.25 \times \gamma_{to} + \frac{\tau_{\text{max}} - (G_1 \times 1.25 \times \gamma_{to})}{G_2} \right) \times \frac{D}{M_s} (12)
\]
5 Parametric study

A parametric study is performed to investigate the effects of various parameters such as monopile diameter (D), monopile length (L_p), internal friction angle of the soil (\(\phi\)) and liquefaction depth (D_L) on the natural frequency (f) of offshore wind turbine (OWT) + monopile system. The reference 8 MW wind turbine whose properties are given in Table 1 is used in the parametric study. Figure 8 shows a schematic illustration of the OWT + monopile system along with liquefaction depth (D_L), monopile length (L_p) and monopile diameter (D). Table 6 summarises the parameters used in the parametric study. As seen in Table 6, three various monopile diameters such as 6, 8 and 10 m are used to study the effect of monopile diameter on the natural frequency of OWT + monopile system with varying liquefaction depths. Five different monopile lengths such as 25, 30, 35, 40 and 45 m are used to investigate the effect of monopile length on the natural frequency of OWT + monopile system under varying liquefaction depths. Three various internal friction angles such as 30°, 35° and 40° are used to study the effect of internal friction angle on the natural frequency of OWT + monopile system with varying liquefaction depths.
as 25°, 30° and 35° are also used to study the effect of internal friction angle on the natural frequency of OWT + monopile system under varying liquefaction depths.

5.1 Results

The results of the parametric study are presented by using non-dimensional groups in order to create a framework of understanding. The natural frequency of the monopile + Offshore Wind Turbine (OWT) after liquefaction (f) is normalized by the natural frequency of the monopile + OWT without liquefaction (f₀). Depth of liquefaction (D_L) is normalized by the length of monopile (L_P). The values of D_L used in the parametric study range between 1 and 13 m for the numerical models.

Figure 9 shows variation of normalized natural frequency of the OWT system (f/f₀) with respect to normalized liquefaction depth (D_L/L_P) for various monopile diameters. Normalized natural frequency of the system shows nonlinear trend with varying normalized depth of liquefaction. The values of f/f₀ for various values of monopile diameter are very close to each other, showing that the change of natural frequency of the system is slightly influenced by monopile diameter under varying liquefaction depths.

Figure 10 shows the variation of the values of f/f₀ with respect to the ratio of D_L/L_P for various normalized monopile lengths (L_P/D). The ratio of f/f₀ experiences nonlinear decrease with increasing ratio of D_L/L_P for all values of L_P/D. The amount of change in the ratio of f/f₀ decreases with increasing ratio of L_P/D. Therefore, it can be concluded that L_P/D is a significant non-dimensional term governing the natural frequency of the system in the case of soil liquefaction.

As mentioned in the work of Arany et al. (2015), the structures are relatively insensitive to the lateral stiffness (K_L) and the most important factor influencing natural frequency of the system is the rotational stiffness (K_R). According to their study, it can be concluded that the change in rotational stiffness (K_R) of the foundation causes the greatest change in

![Figure 9](image-url)  
Fig. 9 Normalized natural frequency (f/f₀) versus normalized liquefaction depth (D_L/L_P) for various monopile diameters (D)
natural frequency of the soil-structure system. The equations of lateral stiffness of the foundation ($K_L$), cross-coupling stiffness of the foundation ($K_{LR}$) and rotational stiffness of the foundation ($K_R$) are given in Eqs. 13–15.

\begin{align}
K_L &= 2.65 \times \left(\frac{L}{D_p}\right)^{1.07} \times f_{(vs)} \times E_{SO} \times D_p \\
K_{LR} &= -1.8 \times \left(\frac{L}{D_p}\right)^2 \times f_{(vs)} \times E_{SO} \times D_p^2 \\
K_R &= 1.63 \times \left(\frac{L}{D_p}\right)^3 \times f_{(vs)} \times E_{SO} \times D_p^3
\end{align}

where $L$ is monopile length, $D_p$ is monopile diameter, $f_{(vs)}$ is a function dependent on Poisson’s ratio of the soil, $E_{SO}$ is Young’s Modulus of the soil at a depth of a pile diameter ($D_p$).

As seen from the equations above, monopile diameter ($D_p$) has a relatively smaller effect on the equation of $K_{LR}$, $K_R$ and even in the equation of $K_L$. Therefore, it can be concluded that the natural frequency of the system is less sensitive to the variation of monopile diameter. On the other hand, foundation stiffness values ($K_L$, $K_{LR}$ and $K_R$) are function of monopile length ($L_p$) as seen in the equations. The variation of the monopile length influences foundation stiffness values which in turn influences the natural frequency values of the soil-structure system.

Figure 11 shows the variation of the ratio of $f/f_0$ with respect to the ratio of $D_t/L_p$ for different internal friction angle ($\phi$) of the soil. There is a nonlinear relation between the ratio of $f/f_0$ and the ratio of $D_t/L_p$ for $\phi = 30^\circ$ and $\phi = 35^\circ$ whereas an almost linear relation between the ratio of $f/f_0$ and the ratio of $D_t/L_p$ is observed for $\phi = 25^\circ$. The change in the ratio of $f/f_0$ increases with decreasing internal friction angle. For instance, the value of $f/f_0$ becomes 0.58 for $D_t/L_p = 0.30$ for $\phi = 25^\circ$ whereas the value of $f/f_0$ is equal to 0.85 for $\phi = 35^\circ$ for the same value of $D_t/L_p$. 

![Fig. 10 Normalized natural frequency ($f/f_0$) versus normalized liquefaction depth ($D_t/L_p$) for various normalized monopile lengths ($L_p/D$)](image-url)
As seen in Eqs. 13–15, values of foundation stiffnesses ($K_L$, $K_{LR}$, $K_R$) are dependent on the Young’s modulus of the soil. As known, Young’s modulus of a soil is proportional to its internal friction angle. Therefore, an increase in internal friction angle of the soil increases foundation stiffnesses ($K_L$, $K_{LR}$, $K_R$). Hence, an increase in foundation stiffnesses increases the natural frequency of OWTs with monopile foundations.

6 Estimation of the change natural frequency due to liquefaction

The change in natural frequency of OWT + monopile system can be predicted by using the equations proposed by Arany et al. (2016). The steps for estimating the ratio of $f/f_0$ is summarised below:

1. The natural frequency of the OWT for fixed base ($f_{fb}$) is calculated by using Eq. 3.
2. The natural frequency of OWT + monopile foundation ($f_0$) is calculated by using Eq. 4.
3. The natural frequency of the OWT for fixed base in the case of liquefaction ($f_{fbL}$) is calculated by considering depth of liquefaction ($D_L$). The mass of unsupported monopile length is added to tower mass.
4. The natural frequency of OWT + monopile foundation after soil liquefaction ($f_L$) is calculated by using Eq. 4. Foundation flexibility factors ($C_L$, $C_R$, $C_s$) are calculated considering the depth of liquefaction. The unsupported length of monopile is considered as transition piece to calculate the value of $C_s$. The value of $C_L$ and $C_R$ are also calculated considering depth of liquefaction ($D_L$). The details of the estimation of the change of natural frequency of OWT + monopile system is given with a worked example in Appendix.

Several modifications were made to the analytical model which was developed by Arany et al. (2016) to estimate natural frequency of OWTs with monopile foundations in non-liquefiable soils in order to predict natural frequencies of such structures in liquefiable soils.

In the modified analytical methodology, following modifications have been made:
(1) Stiffnesses of the foundation \((K_L, K_{LR}, K_R)\) are calculated by considering new embedded length of the monopile \((L')\) after liquefaction where \(L' = L_p - D_L\) (\(L_p\) is monopile length and \(D_L\) is liquefaction depth) (see Step 6 in the appendix).

(2) Non-dimensional foundation stiffness parameters \((\eta_L, \eta_{LR}, \eta_R)\) are calculated by considering new values of \(K_L, K_{LR}, K_R\) (see Step 7 in the appendix).

(3) Foundation flexibility coefficients \((C_L, C_S, C_R)\) are calculated considering new values of \(\eta_L, \eta_{LR}and \eta_R\). The substructure flexibility coefficient of \(C_S\) is expressed in terms of two dimensionless parameters: (a) bending stiffness ratio \((\chi)\) and (b) length ratio \((\psi)\). Length ratio \((\psi)\) where \(\psi = L'S/L_T\) is re-calculated depending on the liquefaction depth \((D_L)\). The distance between the mudline and the bottom of the tower is denoted by \(L_S\) while tower length is denoted by \(L_T\). New distance between the mudline and the bottom of the tower after the liquefaction is denoted by \(L'S\) where \(L'S = L_S + D_L\). (see Step 8 in the appendix)

(4) The mass of the monopile \((m_M)\) along the liquefaction depth are calculated and added to the tower mass in the equation of fixed base natural frequency \((f_{bbL})\) (see Step 9 in the appendix). Finally, natural frequency of OWTs with monopiles under liquefaction \((f_L)\) is obtained by using newly calculated foundation flexibility factors \((C_L, C_S, C_R)\) and fixed base natural frequency \((f_{bbL})\) under liquefaction.

Figures 12 and 13 show variation of the ratio of \(f/f_0\) with respect to the ratio of \(D_L/L_P\) for Finite Element and analytical models. As seen in Figs. 12, 13, the comparisons between FE model and analytical model show that the analytical method can reasonably predict the change in the ratio of \(f/f_0\) under varying \(D_L/L_P\).

A comparison between numerical and analytical changes in the natural frequency of OWT for \(\phi = 30^\circ\) for various normalized liquefaction depths \((D_L/L_P)\) were shown in Figs. 12, 13. In order to assess the applicability of the analytical model for various range of soil parameters, the variation of \(f/f_0\) with \(D_L/L_P\) for \(\phi = 25^\circ\) and \(\phi = 35^\circ\) was predicted through the analytical model. Figure 14 shows a comparison between calculated (numerical model) and predicted (analytical model) changes in the natural frequency of the system \((f/f_0)\) with respect to normalized liquefaction depth \((D_L/L_P)\) for \(\phi = 25^\circ\) and \(\phi = 35^\circ\).

**Fig. 12** Variation of the ratio of \(f/f_0\) with respect to the ratio of \(D_L/L_P\) for Finite Element and analytical models for \(D = 8\) m
As seen in the figure, the discrepancy between analytical model predictions and numerical calculations increases with an increase in normalized liquefaction depth ($D_L/L_P$). The maximum difference between analytical predictions and numerical calculations are 8.4% and 3.5% for $\phi = 25^\circ$ and $\phi = 35^\circ$, respectively. Considering maximum divergence between calculated and predicted values of $f/f_0$, it can be inferred that analytical model can reasonably estimate the change in the ratio of $f/f_0$ with varying values of $D_L/L_P$.

Figure 15 shows the change of natural frequency of the OWT + monopile system due to 8 m of soil liquefaction for the case of $D = 10$ m, $L_P = 24$ m and $\phi = 30^\circ$. The black point with red circle shows the natural frequency of the whole system before the liquefaction. The green point with red circle shows the natural frequency of the whole system after 8 m of liquefaction of the soil surrounding the monopile. Under 10 m of soil liquefaction, the natural frequency decreases by 39% (see red curve in Fig. 13) and it approaches to the...
value of frequency of wave loads as seen in Fig. 15. It is clear that the natural frequency of
the system after liquefaction may resonate with the waves which might have serious fatigue
implications on the structure until liquefaction effects are neutralized.

7 Conclusions

Offshore Wind Turbines (OWTs) are dynamically sensitive structures since their natural
frequency is very close to the forcing frequencies such as wave, wind, 1P and 3P loading
frequencies. Estimation of natural frequency of OWTs is very significant to avoid fatigue
damage of OWTs due to resonance. In the case of liquefaction, the natural frequency of
the whole system decreases as foundations of OWTs lose their supports along the lique-
faction depth. The decrease in the natural frequency of the system may result in matching
forcing frequencies specially wave load frequency with natural frequency of the system.
In order to investigate the effects of soil liquefaction on dynamic behaviour of OWTs in
terms of natural frequency of the system, a parametric study is performed considering vari-
ous monopile diameters, monopile lengths, internal friction angle of the soil and liquefac-
tion depths. A simple methodology to predict liquefaction effects on natural frequency of
OWTs with monopile foundations is proposed. The main findings of the study are summa-
rised as below:

1. The change of normalized natural frequency ($f/f_0$) under various liquefaction depths
   ($D_L$) is not significantly influenced by monopile diameter (D).
2. The ratio of $f/f_0$ under various liquefaction depths is significantly influenced by normal-
   ized monopile length ($L_p/D$) showing that $L_p/D$ is a significant non-dimensional term
governing dynamic behaviour of OWT systems in the case of liquefaction. The ratio of
   $L_p/D$ needs to be increased in order to design relatively insensitive OWTs to depth of
   liquefaction ($D_L$) in terms of natural frequency.
3. A dramatic decrease in the ratio of $f/f_0$ occurs for weaker soil conditions under increas-
   ing depth of liquefaction. The soil conditions of OWTs located in seismically active

Fig. 15 Qualitative power spectrum of main forcing frequencies and different design approaches considering a reference 8 MW wind turbine along with the natural frequency change due to liquefaction
regions needs to be examined carefully. In the case of weak soil conditions, the ratio of \( L_p/D \) can be increased to avoid drastic change in natural frequency of OWT systems.

4. The proposed methodology to estimate change in the natural frequency of OWT systems provides reasonable results and it can be used for preliminary design of OWTs in seismically active regions.

Appendix

**Worked example on prediction of natural frequency of the OWTs after liquefaction**

**Wind turbine data**

The basic wind turbine data that is essential for these analyses are summarised in Table 7. These data is taken from the study of Arany et al. (2017).

**Metocean data**

The required Metocean data for the analysis are listed in Table 8. These are turbulence characteristics, wind speed, wave characteristics and water depth at the site.

**Geological and geotechnical data**

For this worked example, it is assumed that the monopile is placed in loose to medium sand. A monopile with 6 m diameter (D) and 30 m length (L_p) is used for the example. A parabolic stiffness variation is adopted for sand soil profile. The stiffness variation with depth for the adopted soil profile is shown in Fig. 16. The green dashed line in the figure shows the mudline level before liquefaction while red dashed line shows the mudline level.

| Table 7  | Turbine data |
|----------|--------------|
| Parameter | Symbol | Value | Unit |
| Hub height | \( z_{hub} \) | 87 | m |
| Tower height | \( L_T \) | 68 | m |
| Rotor diameter | \( D \) | 120 | m |
| Tower bottom diameter | \( D_b \) | 5 | m |
| Tower top diameter | \( D_t \) | 3 | m |
| Tower wall thickness | \( t_T \) | 0.027 | m |
| Density of tower material | \( \rho_T \) | 7860 | kg/m³ |
| Tower mass | \( m_T \) | 250 | Tons |
| Mass of rotor-nacelle assembly | \( m_{RNA} \) | 243 | Tons |
| Operational rotational speed range of the turbine | \( \Omega \) | 5–13 | rpm |
| Rated wind speed | \( U_R \) | 12 | m/s |
after liquefaction. The depth of liquefaction (DL) is assumed to be equal to 10 m as seen in the figure. The initial soil Young’s modulus (\(E_{S0}\)) at 1D (one pile diameter) depth is 75 MPa for the given soil profile (Fig. 16).

Three parameters are necessary to obtain idealized stiffness profile of the soil in the simplified procedure. These are Young’s modulus of the soil at 1D\(_p\) below mudline denoted by \(E_{S0}\), monopile diameter (\(D_p\)) and the depth (\(z\)). The stiffness profile can be mathematically expressed as follow:

\[
E_{S}(z) = E_{S0} \left( \frac{|z|}{D_p} \right)^n
\]  

(16)
where homogeneous, square root inhomogeneous and linear inhomogeneous are given by \( n = 0, n = 0.5, \) and \( n = 1. \) The stiffness profile in this worked example is assumed as a square root inhomogeneous profile such that \( n \) is taken as 0.5.

**Step 1** Calculation of fixed based natural frequency of the OWT.

The fixed base natural frequency of the OWT is calculated by using Eq. 16.

\[
f_{fb} = \frac{1}{2\pi} \sqrt[3]{\frac{3EI_T}{m_{RNA} + \frac{33}{140}m_T}L_T^3} = 0.422 \text{ Hz}
\]  (17)

where

**Step 2** Calculation of \( K_L, K_{LR}, K_R. \)

Formulas for stiffness of monopiles exhibiting rigid behaviour:

\[
K_L = 2.65 \times \left(\frac{L}{D_p}\right)^{1.07} \times f_{(vs)} \times E_{SO} \times D_p = 2.65 \times \left(\frac{30}{6}\right)^{1.07} \times 1.03 \times 75 \times 6
\]  (18)

\[
K_L = 6873.75 \text{ MN/m}
\]

\[
K_{LR} = -1.8 \times \left(\frac{L}{D_p}\right)^2 \times f_{(vs)} \times E_{SO} \times D_p^2 = -1.8 \times \left(\frac{30}{6}\right)^2 \times 1.03 \times 75 \times 6^2
\]  (19)

\[
K_{LR} = -125145 \text{ MN}
\]

\[
K_R = 1.63 \times \left(\frac{L}{D_p}\right)^3 \times f_{(vs)} \times E_{SO} \times D_p^3 = 1.63 \times \left(\frac{30}{6}\right)^3 \times 1.03 \times 75 \times 6^3
\]  (20)

\[
K_R = 3399773 \text{ MNm/ rad}
\]

\[
f_{(vs)} = 1 + 0.6\left|\theta_p - 0.25\right| = 1 + 0.6 \times (0.3 - 0.25) = 1.03
\]  (21)

where \( L \) is the pile length, \( D_p \) is the pile diameter, \( E_{SO} \) is the soil stiffness at pile diameter depth \( (D_p). \)

**Step 3** Calculation of the non-dimensional foundation stiffness parameters \( (\eta_L, \eta_{LR}, \eta_R) \)

\[
\eta_L = \frac{K_LL^3}{EI_\eta} = \frac{6873.75 \times 68^3}{7.1186 \times 10^5} = 3036.16
\]  (22)

\[
\eta_{LR} = \frac{K_{LR}L^2}{EI_\eta} = \frac{-125145 \times 68^2}{7.1186 \times 10^5} = -812.898
\]  (23)

\[
\eta_R = \frac{K_RL}{EI_\eta} = \frac{3399773 \times 68}{7.1186 \times 10^5} = 324.761
\]  (24)

**Step 4** Calculation of \( C_L, C_S, C_R \)

\[ \text{Springer} \]
\( c_R = 1 - \frac{1}{1 + 0.6 \left( \frac{\eta_R}{\eta_L} \right)^2} = 1 - \frac{1}{1 + 0.6 \left( \frac{324.761 - \left( \frac{-812.898}{3036.16} \right)^2}{3036.16} \right)} \) (25)

\( c_R = 0.985 \)

\( c_L = 1 - \frac{1}{1 + 0.5 \left( \frac{\eta_L}{\eta_R} \right)^2} = 1 - \frac{1}{1 + 0.5 \left( \frac{3036.16 - \left( \frac{-812.898}{324.761} \right)^2}{3036.16} \right)} \) (26)

\( c_L = 0.998 \)

\( c_s = \sqrt{\frac{1}{1 + (1 + \psi)^3 \chi - \chi}} = \sqrt{\frac{1}{1 + (1 + 0.294)^3 \times 0.3808 - 0.3808}} = 0.832 \) (27)

\( \psi = \left( \frac{L_s}{L_T} \right) = \frac{20}{68} = 0.294 \)

\( \chi = \left( \frac{(E_T I_T)}{(E_p I_p)} \right) = 0.3808 \) (29)

**Step 5** Calculation of natural frequency of the OWT + monopile system

\( f_0 = C_L \times C_R \times C_S \times f_{jb} = 0.998 \times 0.985 \times 0.832 \times 0.422 = 0.345 \text{ Hz} \) (30)

**Step 6** Re-calculation of \( K_L, K_{LR}, K_R \) for selected liquefaction depth

\( K_L = 2.65 \times \left( \frac{L'}{D_p} \right)^{1.07} \times f_{(\psi)} \times E_{SO} \times D_p = 2.65 \times \left( \frac{20}{6} \right)^{1.07} \times 1.03 \times 75 \times 6 \) (31)

\( K_L = 4454.26 \text{ MN/m} \)

\( K_{LR} = -1.8 \times \left( \frac{L'}{D_p} \right)^2 \times f_{(\psi)} \times E_{SO} \times D_p^2 = -1.8 \times \left( \frac{20}{6} \right)^2 \times 1.03 \times 75 \times 6^2 \) (32)

\( K_{LR} = -55620 \text{ MN} \)

\( K_R = 1.63 \times \left( \frac{L'}{D_p} \right)^3 \times f_{(\psi)} \times E_{SO} \times D_p^3 = 1.63 \times \left( \frac{20}{6} \right)^3 \times 1.03 \times 75 \times 6^3 \) (33)

\( K_R = 1007340 \text{ MNm/rad} \)

\( L' = L_p - D_L = 30 - 10 = 20 \) (34)
where $D_L$ is the liquefaction depth.

**Step 7** Re-calculation of non-dimensional foundation stiffness parameters ($\eta_L$, $\eta_{LR}$, $\eta_R$)

$$\eta_{LR} = \frac{K_{LR}L^2}{EI} = \frac{-55620 \times 68^2}{7.1186 \times 10^5} = -361.29$$

(35)

$$\eta_R = \frac{K_R L}{EI} = \frac{1007340 \times 68}{7.1186 \times 10^5} = 96.23$$

(36)

**Step 8** Re-calculation of $C_L$, $C_S$, $C_R$ for selected liquefaction depth

$$c_R = 1 - \frac{1}{1 + 0.6\left(\eta_R - \frac{\eta_{LR}}{\eta_L}\right)} = 0.947$$

(37)

$$c_L = 1 - \frac{1}{1 + 0.5\left(\eta_L - \frac{\eta_{LR}}{\eta_R}\right)} = 0.997$$

(38)

$$c_s = \sqrt{\frac{1}{1 + (1 + \psi)^3 \chi - \chi}} = 0.754$$

(38)

$$\psi = \frac{L_s'}{L_T} = \frac{30}{68} = 0.441$$

(40)

$$L'_s = L_s + D_L = 20 + 10 = 30\text{m}$$

(41)

$$\chi = \frac{(E_T I_T)}{(E_p I_p)} = 0.3808$$

(42)

**Step 9** Re-calculation of fixed based natural frequency of the OWT considering liquefaction depth

$$f_{fb_L} = \frac{1}{2\pi} \sqrt{\frac{3E_T I_T}{m_{RNA} + \frac{33}{140}(m_T + m_M)L_T^3}} = 0.413 \text{Hz}$$

(43)

$$m_M = A_{p} \times D_L \times \rho_{pile} = 0.656 \times 10 \times 7860 = 51561.6 \text{kg}$$

(44)

where $m_M$ is the mass of monopile in the liquefaction depth and can be calculated as:

**Step 10** Re-calculation of natural frequency of the OWT + monopile system considering liquefaction depth

$$f_L = C_L \times C_R \times C_S \times f_{fb1} = 0.997 \times 0.947 \times 0.754 \times 0.413 = 0.294 \text{Hz}$$

(45)
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