Publishing
The Institution of Engineering and Technology
ENGINEERING & TECHNOLOGY REFERENCE

Reference Article
1st published in Month 2017
doi: 10.1049/etr.2016.0019
ISSN 2056-4007
www.ietdl.org

Soil–Structure Interactions for Offshore Wind Turbines

Subhamoy Bhattacharya Chair in Geomechanics, Department of Civil and Environmental Engineering, University of Surrey, Guildford, UK
Georgios Nikitas Postgraduate Researcher, Department of Civil and Environmental Engineering, University of Surrey, Guildford, UK
Laszlo Arany Mechanical Engineer, Atkins, Bristol, UK
Nikolaos Nikitas Associate Professor, School of Civil Engineering, University of Leeds, Leeds, UK

Abstract
Soil–structure interaction (SSI) for offshore wind turbine supporting structures is essentially the interaction of the foundation/foundations with the supporting soil due to the complex set of loading. This study reviews the different aspects of SSI for different types of foundations used or proposed to support offshore wind turbines. Due to cyclic and dynamic nature of the loading that acts on the wind turbine structure, the dominant SSI will depend to a large extent on the global modes of vibration of the overall structure. This study summarises the modes of vibration of offshore wind turbines structures supported on different types of foundations based on observations from scaled model tests and numerical analysis. As these are new structures with limited monitoring data, field records are scarce. Where possible, field records available in the public domain are also used to compare with experimental findings.

Introduction

Foundations for wind turbine generators (WTGs)
Offshore wind turbine (OWT) installation is a unique type of structure due to their geometry (i.e. mass and stiffness distribution along the height) and the loads acting on it. It has been shown that the environmental loads are a mixture of cyclic and dynamic components and depend on the location of the wind farm (wave period, fetch, wind turbulence) together with the size and type of the turbine (see [1, 2]). The main purpose of a foundation is to transfer these loads safely (without excessive deformation) to the surrounding soil. Behaviour of saturated soil under cyclic/dynamic loading is very complex and not well understood and thus, the design of the foundation for these structures is challenging.

Figs. 1 and 2 show WTGs supported on various types of foundations which are either currently used or proposed to be used. Few points may be noted: (i) for water depth typically up to 30–40 m, single foundation (large gravity base or single large diameter pile) may suffice; (ii) for water depths more than 30–40 m to about 60–70 m, multiple foundations (more than one shallow foundations or few piles) may be needed; (iii) for water depths in excess of 80–100 m, bottom-fixed foundations become uneconomic and floating structures become the preferable choice. In each of these cases, the load transfer to the neighbouring ground is essentially a *soil–structure interaction* (SSI).

The difference between the load transfer processes of single foundations and multiple foundations is explained through Fig. 3 by taking the example of single large diameter monopile and multiple piles supporting a jacket. In the case of monopile-supported wind turbine structures or for that matter any single foundation (e.g. Figs. 1*a*–*c*), the load transfer is mainly through overturning moments where the monopile/foundation transfers loads to the surrounding soil and therefore it is lateral foundation soil interaction. On the other hand, for multiple support structure, the load transfer is mainly through push–pull action, i.e. axial load as illustrated in the figure.

It is economical to have many turbines in a wind farm to have the economy of scale by taking advantage of subsea export cables and therefore the modern and future wind farm also requires a large area. If the continental shelf is very steep (i.e. variation of ocean water depth with distance from the shore), grounded (fixed)

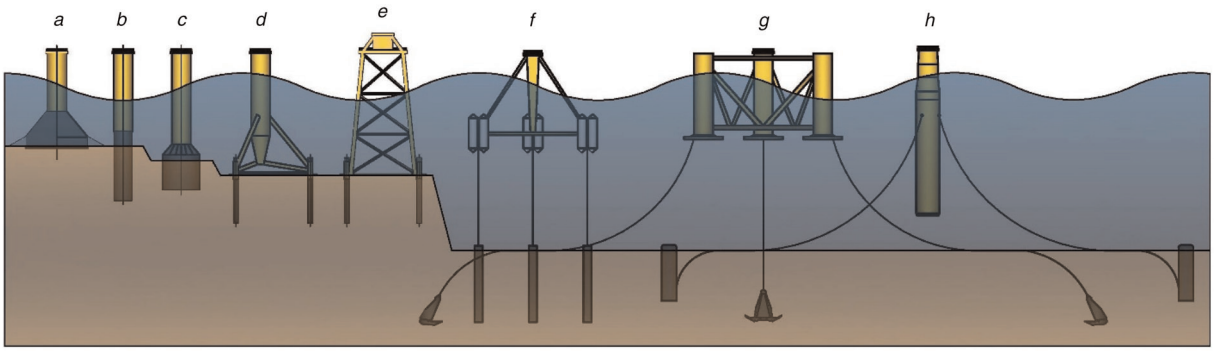

Fig. 1 *Common types of foundations used to support WTGs*

(a) Gravity base foundation, ***(b)*** Monopile foundation connected to the tower with a transition piece, ***(c)*** Suction caisson foundation, ***(d)*** Tripod substructure supported by three pile foundations, ***(e)*** Jacket substructure supported by four pile foundations, ***(f)*** Tension leg platform anchored to three pile foundations, ***(g)*** Semi-submersible floating platform moored to drag anchors, ***(h)*** Ballast-stabilised floating spar platform anchored to three suction caissons

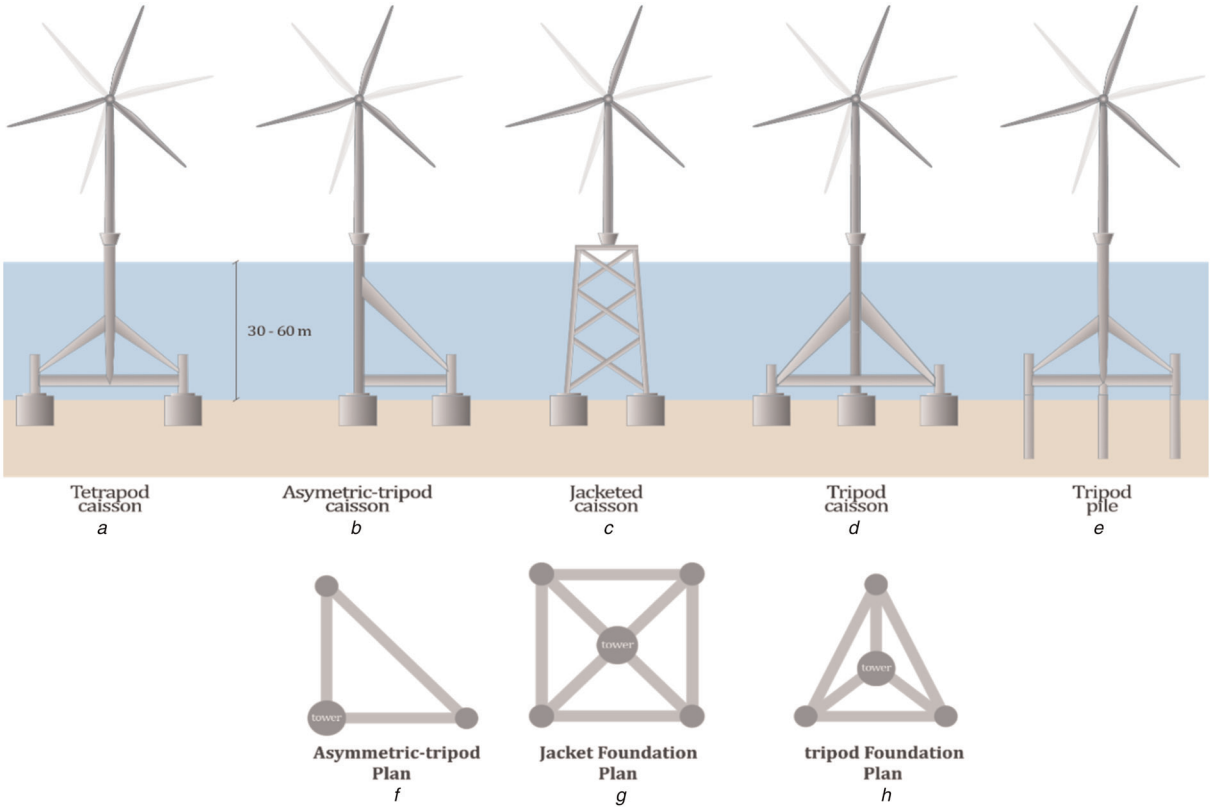

Fig. 2 *Various proposed and existing multi-foundation arrangements to support WTG*

(a) Tetrapod substructure supported by four suction caisson foundations, ***(b)*** Asymmetric tripod substructure supported by three suction caissons foundations, ***(c)*** Jacket substructure supported by four suction caisson foundations, ***(d)*** Symmetric tripod substructure supported by three suction caisson foundations, ***(e)*** Tri-pile substructure and foundation, ***(f)*** Plan view of an asymmetric tripod substructure, ***(g)*** Plan view of a jacket substructure, ***(h)*** Plan view of a symmetric tripod substructure

turbines are not economically viable and a floating system is desirable.

Typically, foundations cost 25–34% of an overall project and thus, innovations are underway to reduce the foundation costs [3]. Fig. 4*a* shows a photograph and an artistic impression of a particular type of tripod foundation having a right-angled corner developed by SPT offshore. The advantage of such a configuration is the ease to transport to the location using a barge and easy installation and hence the name self-installed wind turbine (SIWT).

doi: 10.1049/etr.2016.0019

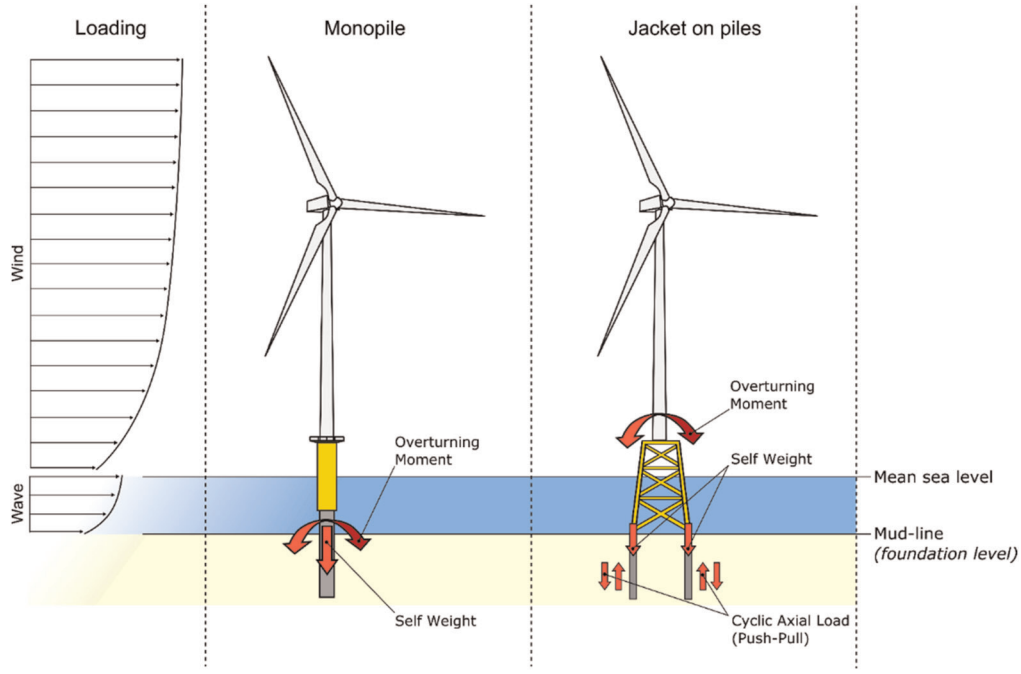

Fig. 3 *Load transfer mechanisms for monopile and jacket supported on piles*

Fig. 4 *Schematic/photographs of some types of foundations*
(a) SIWT: asymmetric type of foundation, ***(b)*** Tripod type of foundation

Fig. 4*b* shows pile-supported tripod system used in Alpha Ventus wind farm. However, monopile (see Figs. 1*b* and 3) due to its simple shape and easy fabrication is one of the preferred types of foundations and will be the main focus in this study.

Ideal foundations for OWTs

The choice of foundation will depend on the following: site condition, fabrication, installation, operation and maintenance, decommissioning and finally economics. Following [4], the definition of an ideal foundation is as follows:

(i) A foundation which is capacity or 'rated power' specific (i.e. 5 or 8 MW rated power) but not turbine manufacturer specific. In other words, a foundation designed to support 5 MW turbines but can support turbines of any type. There are advantages in the sense that turbines can be easily replaced even if a particular manufacturer stops manufacturing them.

(ii) The foundation is easy to fabricate. For example, a large diameter monopile (extra large (XL) piles) can be fabricated by rolling and welding a steel plate, and this process can be automated. On the other hand, a jacket needs extensive welding and often comprehensive manual intervention. From the fabrication point of view, the monopile is preferred.
(iii) Installation of foundation is not weather sensitive, i.e. not dependent on having a calm sea or a particular wind condition. The installation of the first offshore wind farm in the USA took more time due to the unavailability of a suitable weather window.
(iv) Low operation and maintenance (O&M) costs, i.e. needs the least amount of inspection. O&M costs over the lifetime of wind turbines are typically in the same order of magnitude as the capital expenditure cost for the offshore wind farm project. For example, a jacket type foundation needs periodic inspection at the weld joints, often in difficult-to-access locations.

Aim and scope of this study

The aim and scope of this study are as follows:

(a) Review the main loads on the OWTs with their implication on dynamic SSI;
(b) Summarise the SSI issues for the most commonly used monopile type of foundation;
(c) Discuss the SSI aspects on other types of foundations.

Cyclic and dynamic loads on the wind turbine system

As the aim of the foundation is to transfer the loads of the substructure and superstructure safely to the ground, it is necessary to review the loads acting on the wind turbine structure. This section of the paper discusses the loads on the structure. Apart from the self-weight of the whole system, there are four main lateral loads acting on an OWT structure: wind, wave, 1P (rotor frequency) and 2P/3P (blade passing frequency) loads. Fig. 5*a* shows a schematic representation of the time history (wave form) of the main loads.

Each of these loads has unique characteristics in terms of magnitude, frequency and number of cycles applied to the foundation. The loads imposed by the wind and the wave are random in both space (spatial) and time (temporal) and therefore they are better described statistically. Apart from the random nature, these two loads may also act in two different directions (often termed as wind–wave misalignment) to have a steady power output. 1P loading is caused by mass and aerodynamic imbalances of the rotor and the forcing frequency equals the rotational frequency of the rotor. On the other hand, 2P/3P loading is caused by the blade shadowing effect, wind shear (i.e. the change in wind speed with height above the ground) and rotational sampling of turbulence (see e.g. [1, 5]). Its frequency is simply two or three times the 1P frequency. Further details on the loading can be found in [1, 2, 5].

Based on the method developed by Arany *et al.* [2], Table 1 shows typical values of thrust due to the wind load acting at the hub level for five turbines ranging from 3.6 to 8 MW. The thrust load depends on the rotor diameter, wind speed, controlling mechanism and turbulence at the site. The mean and maximum bending moments on a monopile are also listed. Wave loads strongly depend on the pile diameter and the water depth and are therefore difficult to provide a general value. Table 1 contains a relatively severe case of 30 m water depth and a maximum wave height of 12 m.

Typical values of wave loading ranges between 2 and 10 MN acting at about 3/4 of the water depth above the mudline which must be added to the wind thrust. Typical peak wave periods are around 10 s. The pattern of overturning moment on the monopile is schematically visualised in Fig. 5*b*. In the figure, a typical value of the peak period of wind turbulence is taken and can be obtained from wind spectrum data.

Fig. 6 presents a schematic diagram of the main frequencies of these four types of loads so that the dynamic design constraints can be visualised. Current design aims to place the natural frequency of the whole system in between 1P and 3P in the so-called 'soft–stiff' design. In the plot, the natural frequency of two Vestas V90 3 MW wind turbines from two wind farms (Kentish Flats and Thanet) are also plotted. Though the turbines are same, the variation in the natural frequency is due to the different ground and site conditions. Few points may be noted:

(i) In the 'soft–stiff' design, the natural frequency or the resonant frequency is very close to the upper end of 1P (i.e. frequency corresponding to the rated power of the turbine) and lower bound of the 3P (i.e. cut-in speed of the turbine). This will inevitably cause vibration of the whole system as the ratio of forcing

Eng. Technol. Ref., pp. 1–16
doi: 10.1049/etr.2016.0019

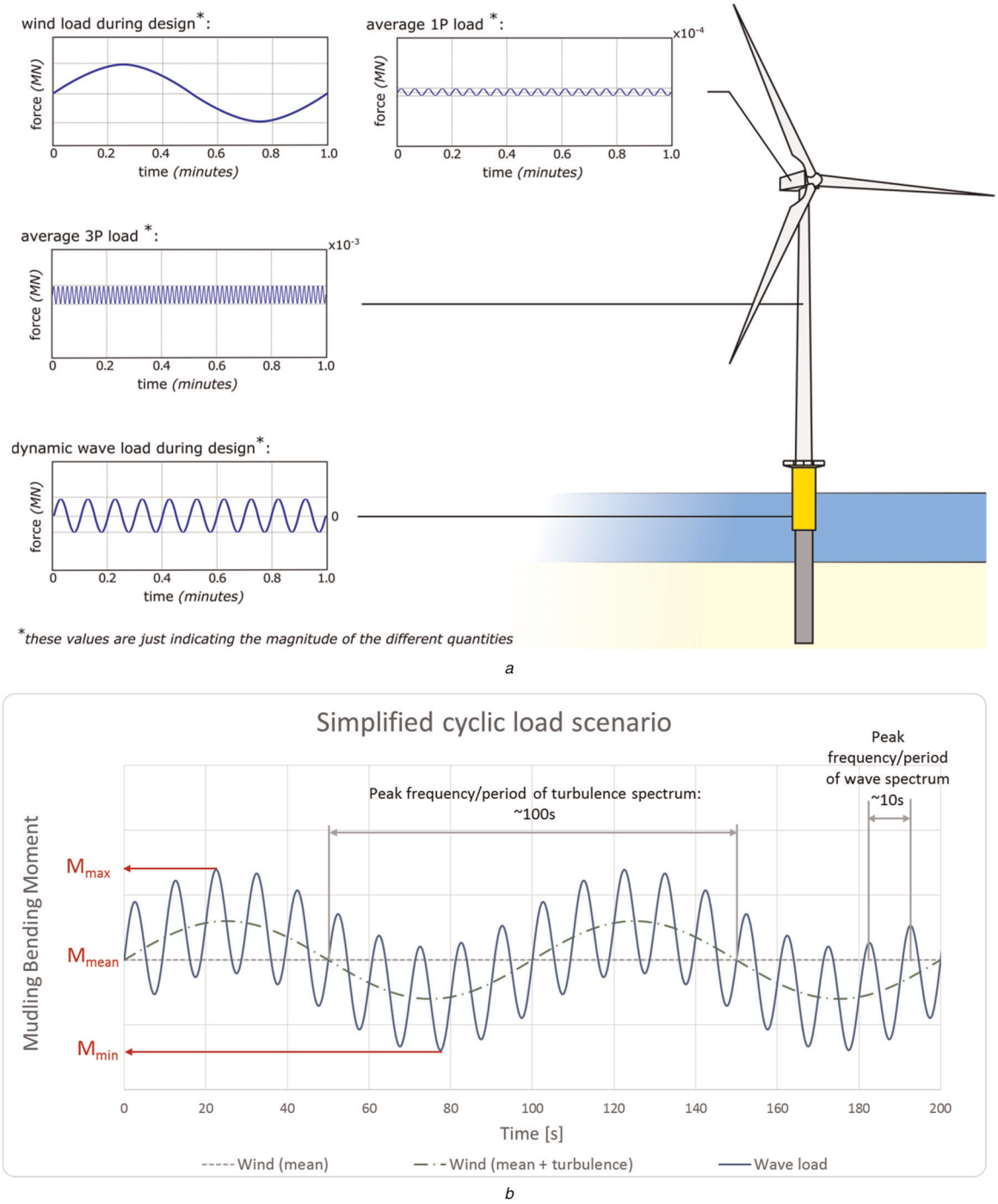

Fig. 5 *Schematic representation of the time history (wave form) of the main loads*
(a) Main loads on OWTs, ***(b)*** Simplified mudline bending moment time history on a monopile under the action of regular waves [2]

to natural frequency is very close to 1. It is worth noting that resonance under operational condition has been reported in the German North Sea projects (see [6]).

(ii) Fig. 7 shows a similar plot as shown in Fig. 6 but for different turbines (2–8 MW). It is clear that as the turbine size/rated power increases, the target frequency is moving towards the left of the spectrum. For example, the target frequency of a 3 MW turbine is in the range of 0.35 Hz. In contrast, the target frequency for a 8 MW turbine is 0.22–0.24 Hz which is within a factor of 2 of a typical predominant wave frequency. This can also be explained through Campbell diagram plotted in Fig. 8 which shows the narrow band of the target frequency for 8 MW turbine.

Table 1 Typical wind and wave loads for various turbine sizes for a water depth of 30 m

Parameter	Unit	Turbine rated power				
		3.6 MW	3.6 MW	5.0 MW	6–7 MW	8 MW
rotor diameter	m	107	120	126	154	164
rated wind speed	m/s	13	13	11.4	13	13
hub height	m	75	80	85	100	110
mean thrust at hub	MN	0.50	0.60	0.60	1.00	1.20
max thrust at hub	MN	1.00	1.20	1.20	2.00	2.30
mean mudline moment M_{mean}	MNm	53	69	70	135	165
max mudline moment M_{max}	MNm	103	136	137	265	323
water depth	m	30	30	30	30	30
maximum wave height	m	12	12	12	12	12
typical monopile diameter	m	5.5	6	6.5	7	7.5
horizontal wave force	MN	3.67	4.2	4.81	5.43	6.09
mudline moment from waves	MNm	104	120	137	155	175
unfactored design moment	MNm	207	256	274	420	498

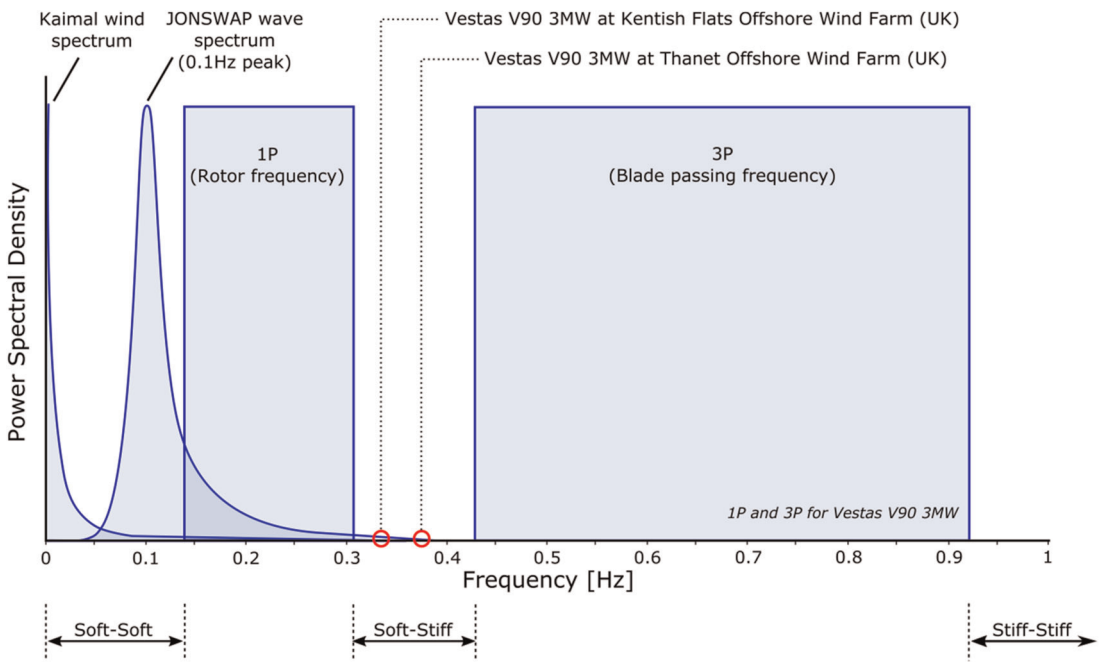

Fig. 6 *Frequency range of the loads along with natural frequency of the turbines for 3 MW turbines*

(iii) For a soft–stiff 3 MW WTG system, 1P and 3P loadings can be considered as dynamic (i.e. ratio of the loading frequency to the system frequency very close to 1). Most of the energy in wind turbulence is in lower frequency variations (typically around 100 s peak period), which can be considered as cyclic. On the other hand, 1P and 3P dynamic loads change quickly in comparison to the natural frequency of the WTG system and therefore the ability of the WTG to respond depends on the characteristics, and dynamic analysis is therefore required.

(iv) As a rule of thumb, if the natural frequency of the WTG structure is more than five times the forcing frequency, the loading can be considered cyclic and inertia of the system may be ignored. For example, for a 3 MW wind turbine having a natural frequency of 0.3 Hz, any load having frequency more than 0.06 Hz is dynamic. Therefore, wave loading of 0.1 Hz is dynamic.

(v) It is easily inferred that for large turbines (8 MW) sited in deeper waters, the wave loads will be highly dynamic (target frequency of the WTG system is 0.22 Hz and the most waves are in the frequency range of 0.05–0.2 Hz) and may control the design.

It has been shown by Bhattacharya [7] and more recently by Arany *et al.* [2] that the design of the foundation is controlled by the foundation stiffness

doi: 10.1049/etr.2016.0019

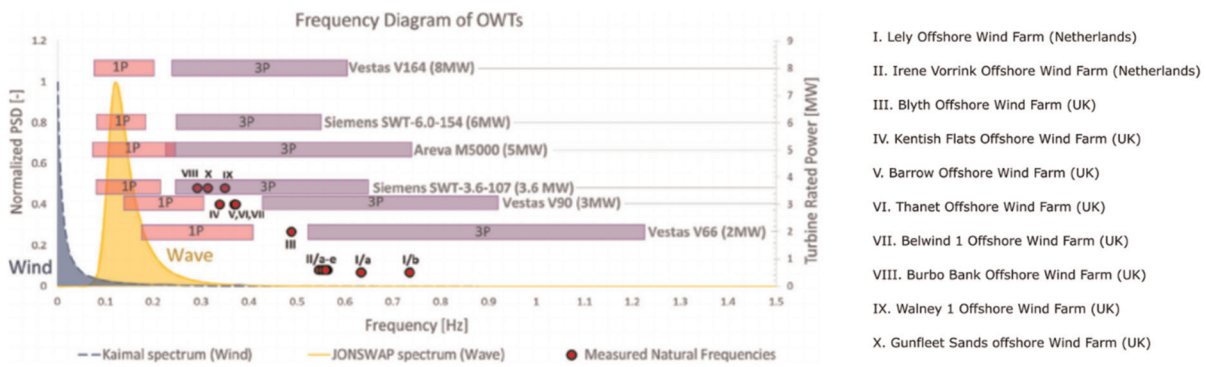

Fig. 7 *Importance of dynamics with deeper offshore and larger turbines*

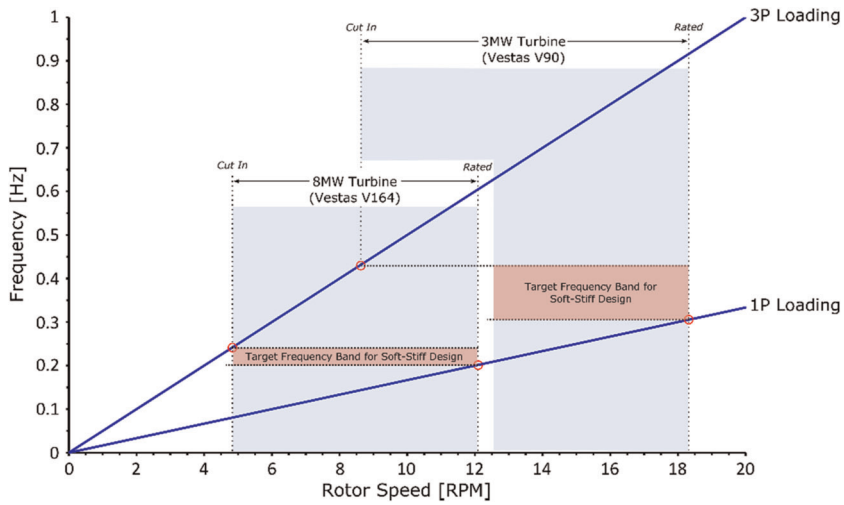

Fig. 8 *Campbell diagram for a 3 and 8 MW turbine*

due to the serviceability limit state (SLS) requirements. It is of interest to take an example to illustrate the salient aspects and complexity of the design and for that purpose monopile is taken. Fig. 9 shows a simplified mechanical model of monopile supported wind turbines and the foundation is represented by a set of springs. From simplified design point of view, the logical steps are:

(a) Obtaining loads on the foundation for different load scenarios, i.e. vertical load (V), lateral load (H) and overturning moment (M) as shown in Fig. 5. Arany *et al.* [2] developed a simplified methodology to estimate the loads at the pile head.
(b) Based on a pile geometry and ground profile (stiffness along the depth of the ground), one can obtain the initial stiffness of the foundation (i.e. K_L, K_R and K_{LR} in Fig. 9) and is explained later in this study. K_L represents lateral stiffness, i.e. force required for unit lateral displacement of the pile head (unit of MN/m), whereas K_R represents moment required for unit rotation of the pile head (unit of GNm/rad). K_{LR} is the cross-coupling spring explained through (1). Detailed explanation of the modelling explained in Fig. 9 is provided in the next section of this paper. Once K_L, K_R and K_{LR} are known, using closed-form solution developed by the authors in [8, 9], the first natural frequency of the whole system can also be predicted. The initial displacements of the pile head (in the linear range) may also be predicted using (1). The terminology can be found in Fig. 9.
(c) Conservative design, i.e. having the foundation stiffness more than necessary may not be a safe solution for soft–stiff type of design as it will impinge on 3P frequency range thereby increasing the response and ultimately higher fatigue damage

$$\begin{Bmatrix} H \\ M \end{Bmatrix} = \begin{bmatrix} K_L & K_{LR} \\ K_{LR} & K_R \end{bmatrix} \begin{Bmatrix} u_L \\ \theta_R \end{Bmatrix} \tag{1}$$

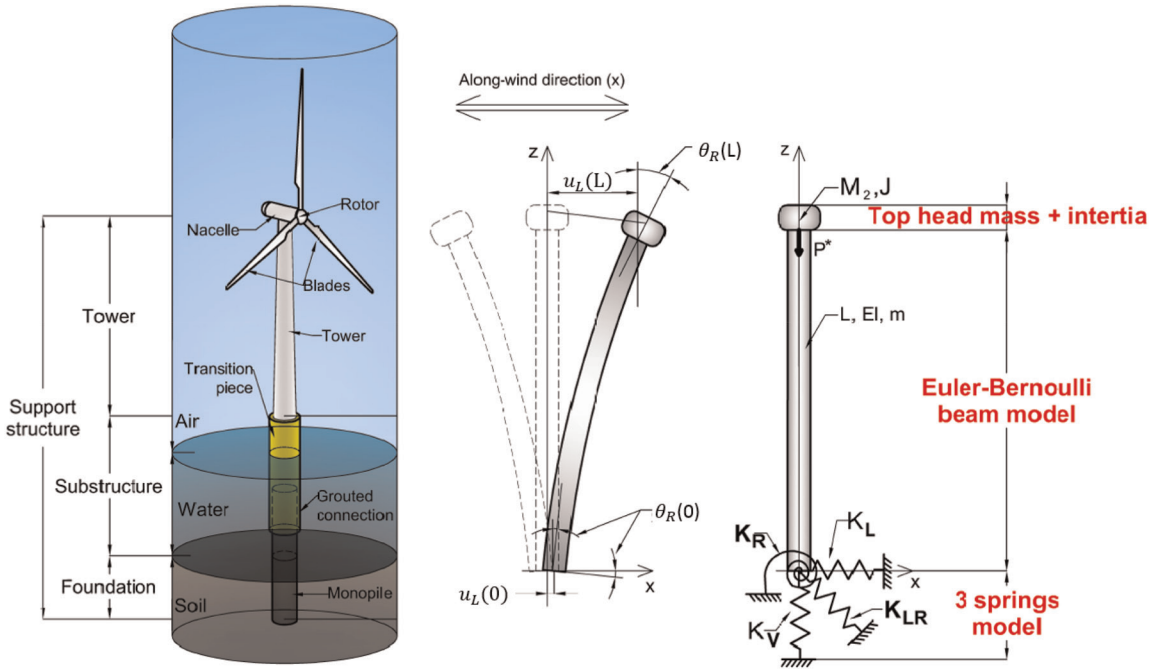

Fig. 9 *Mechanical model of a wind turbine system showing the mass and stiffness distribution*

Modelling strategies for monopile type of foundation

Among the different types of foundations proposed, monopile is very commonly used (about 70% of all operating wind turbines) and is shown schematically in Fig. 9. Effectively, it is the extension of the tower below the ground. The figure also shows a mechanical/mathematical model of the whole system where the foundation is replaced by four springs: vertical spring (K_V having the units of MN/m), lateral (K_L having the units of MN/m), rotational/rocking (K_R having the units GNm/rad) and cross-coupling (K_{LR} having the units of GN). It may be noted that the torsional spring is not included as the effect of torsional loads is minimal due to the yaw bearing at the top of the tower which supports the rotor nacelle assembly (RNA).

This model (which can be conveniently named as *substructure-superstructure model*) allows a two-step design and easy optimisation of the different components. This model can be economic in the conceptual design phase or tender design stage as shown by Arany *et al.* [2]. Expressions of K_L, K_R and K_{LR} for different pile geometry (rigid or flexible) and ground profile can be obtained from the literature [10–13]. Tables 2 and 3 show the equations from [11]. The definitions of the terms are given in the footnote of the tables. Few points may be noted:

(a) For monopiles behaving rigidly, the stiffness terms are function of aspect ratio of the pile (L/D_P) and the soil stiffness (E_{SO})
(b) In contrast, for monopile behaving flexibly, the stiffness terms are function of relative pile–soil stiffness (E_P/E_{SO}) and also on the soil stiffness. For further discussion on these aspects, the readers are referred to [11, 14, 15].

Table 2 Formulas for stiffness of monopiles exhibiting rigid behaviour

Ground profile	K_L	K_{LR}	K_R
homogeneous	$3.2\left(\frac{L}{D_P}\right)^{0.62} f_{(vs)}E_{SO}D_P$	$-1.8\left(\frac{L}{D_P}\right)^{1.56} f_{(vs)}E_{SO}D_P^2$	$1.65\left(\frac{L}{D_P}\right)^{2.5} f_{(vs)}E_{SO}D_P^3$
parabolic	$2.65\left(\frac{L}{D_P}\right)^{1.07} f_{(vs)}E_{SO}D_P$	$-1.8\left(\frac{L}{D_P}\right)^{2} f_{(vs)}E_{SO}D_P^2$	$1.63\left(\frac{L}{D_P}\right)^{3} f_{(vs)}E_{SO}D_P^3$
linear	$2.35\left(\frac{L}{D_P}\right)^{1.53} f_{(vs)}E_{SO}D_P$	$-1.8\left(\frac{L}{D_P}\right)^{2..5} f_{(vs)}E_{SO}D_P^2$	$1.58\left(\frac{L}{D_P}\right)^{3.45} f_{(vs)}E_{SO}D_P^3$

Eng. Technol. Ref., pp. 1–16
doi: 10.1049/etr.2016.0019

Table 3 Formulas for stiffness of monopiles exhibiting flexible behaviour

Ground profile	K_L	K_{LR}	K_R
homogeneous	$1.45\left(\frac{E_P}{E_{SO}}\right)^{0.186} f_{(vs)}E_{SO}D_P$	$-0.3\left(\frac{E_P}{E_{SO}}\right)^{0.5} f_{(vs)}E_{SO}D_P^2$	$0.19\left(\frac{E_P}{E_{SO}}\right)^{0.73} f_{(vs)}E_{SO}D_P^3$
parabolic	$1.015\left(\frac{E_P}{E_{SO}}\right)^{0.27} f_{(vs)}E_{SO}D_P$	$-0.29\left(\frac{E_P}{E_{SO}}\right)^{0.52} f_{(vs)}E_{SO}D_P^2$	$0.18\left(\frac{E_P}{E_{SO}}\right)^{0.76} f_{(vs)}E_{SO}D_P^3$
linear	$0.79\left(\frac{E_P}{E_{SO}}\right)^{0.34} f_{(vs)}E_{SO}D_P$	$-0.27\left(\frac{E_P}{E_{SO}}\right)^{0.567} f_{(vs)}E_{SO}D_P^2$	$0.17\left(\frac{E_P}{E_{SO}}\right)^{0.78} f_{(vs)}E_{SO}D_P^3$

$f_{(vs)} = 1 + 0.6|\upsilon_s - 0.25|$. D_P is the pile diameter; L is the pile length; E_P is the equivalent modulus of the pile, E_{SO} is the Young's modulus of ground at 1 diameter below the ground; vs is the Poisson's ratio.

A more robust model to analyse the foundation is shown in Fig. 10 where the soil can be modelled as continuum. This is very expensive computationally and requires high-quality element test of the soil data to define the constitutive model and an experienced finite-element modeller. This can be used to verify the final design of the foundation and is impractical to use in the design optimisation stage.

Trends in dynamic design of the foundation

A foundation provides flexibility and damping to a wind turbine system and this has been shown experimentally by the authors in [16–22]. As the foundation stiffness increases (i.e. K_L, K_R and K_{LR}), the natural frequency of the whole system (f) will move towards fixed base frequency (f_{FB}), i.e. assuming the bottom of the tower is fixed/encastre. Extensive studies carried out by the authors in [8, 9] showed that amongst the three stiffness terms (K_L, K_R and K_{LR}), rocking stiffness (K_R) dominates the natural frequency calculations for monopile supported OWT. Fig. 11 shows natural frequency of 12 operating wind turbines following the work of Arany *et al.* [9] where the normalised natural frequency (f/f_{FB}) is plotted against the normalised rotational stiffness $\eta_R = K_R L/\mathrm{EI}$ where EI and L are the average stiffness and length of the tower. The study clearly shows that the fundamental natural frequency is about 90–95% of the fixed base frequency. Few points may be noted:

(a) K_R is the foundation stiffness defined in Fig. 9 and it is dependent on the soil stiffness. Following the curves shown in Fig. 11, it may be observed that any change in soil stiffness therefore will alter the natural frequency of the whole system affecting dynamic behaviour as well as fatigue.
(b) This behaviour is non-linear and for soft–stiff design, increase or decrease in natural frequency can impinge in forcing frequencies (see Fig. 6).
(c) The above discussion shows the importance of understanding the change in soil stiffness over time.

SSI and long-term performance of wind turbines

Research carried out by the authors in [21–23] showed that SSI is important to predict the long-term performance of this relatively new type of structure. SSI can be cyclic as well as dynamic and will affect the following three main long-term design issues:

(a) Whether or not the foundation will tilt progressively under the combined action of millions of cycles of loads arising from the wind, wave and 1P (rotor frequency) and 2P/3P (blade passing frequency). Fig. 5*b* shows a simplified estimation of the midline bending moment acting on a monopile type foundation and it is clear that the cyclic load is asymmetric which depends on the site condition, i.e. relative wind and wave component. It must be mentioned that if the foundation tilts more than the allowable, it may be considered failed based on SLS criteria and may also lose the warranty from the turbine manufacturer.
(b) It is well known from the literature that repeated cyclic or dynamic loads on a soil causes change in the properties which in turn can alter the stiffness of foundation (see [16, 19]). A wind turbine structure derives its stiffness from the support stiffness (i.e. the foundation) and any change in natural frequency may lead to the shift from the design/target value and as a result the system may get closer to the forcing frequencies. This issue is particularly problematic for soft–stiff design (i.e. the natural or resonant frequency of the whole system is placed between upper bound of 1P and the lower bound of 3P) as any increase or decrease in natural frequency will impinge on the forcing frequencies and may lead to unplanned resonance. This may lead to loss of years of service, which is to be avoided.

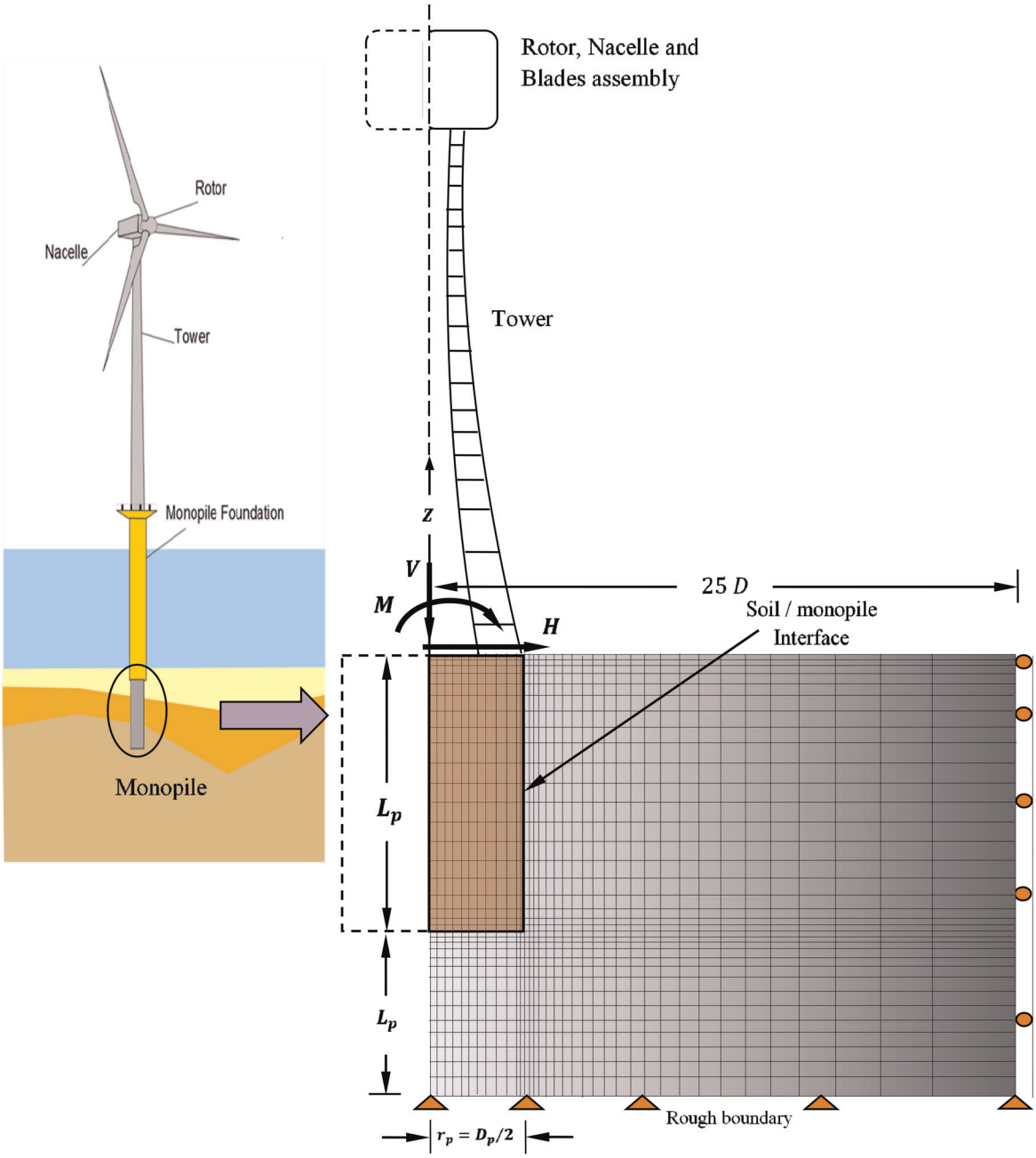

Fig. 10 *Modelling the whole problem considering SSI*

(c) Predicting the long-term behaviour of the turbine taking into consideration wind and wave misalignment aspects. Wind and wave loads may act in different directions. While the blowing wind creates the ocean waves and ideally they should act collinearly. However, due to operational requirements (i.e. to obtain steady power), the rotor often needs to feather away from the predominant direction (yaw action) which creates wind–wave misalignment.

It is therefore essential to understand the mechanisms that may cause the change in dynamic characteristics of the structure and if it can be predicted through analysis. An effective and economic way to study the behaviour (i.e. understanding the physics behind the real problem) is by conducting carefully and thoughtfully designed scaled model tests in laboratory conditions simulating (as far as realistically possible) the application of millions of cyclic lateral loading by preserving the similitude relations. Considerable amount of research has been carried out to understand various aspects of cyclic and dynamic SSI (see [23–25]). The studies showed that to assess the SSI, it is necessary not only to understand

doi: 10.1049/etr.2016.0019

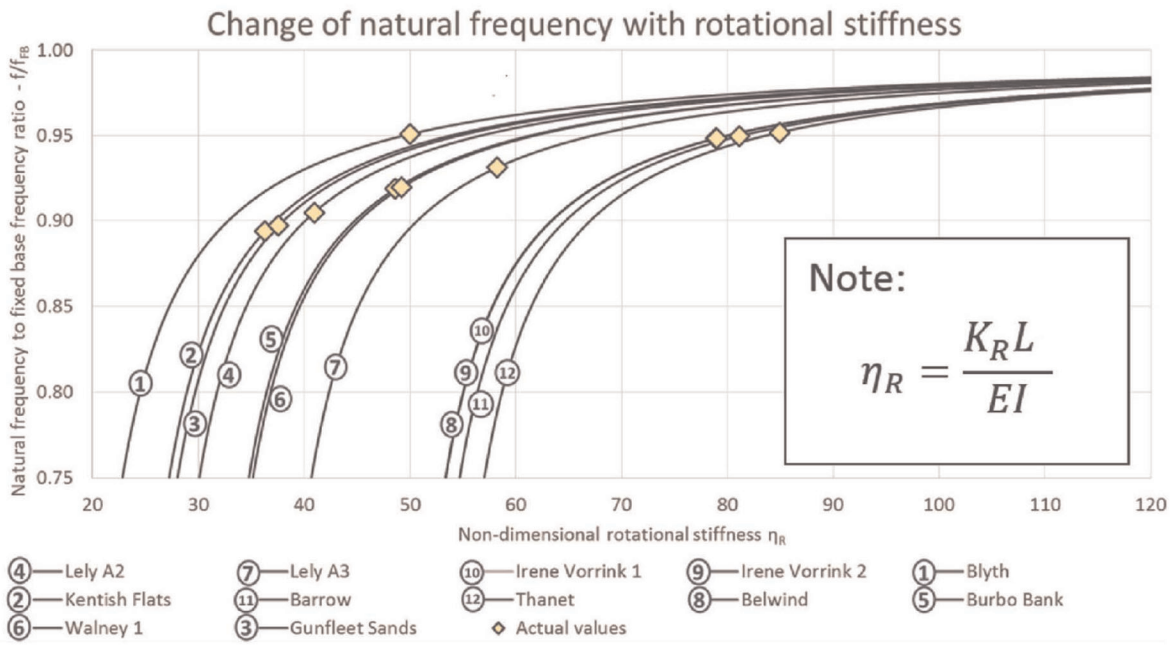

Fig. 11 *Ratio of fundamental natural frequency to the fixed base frequency of installed wind turbines*

the loading on OWTs but also the modes of vibration of the overall system. The aspect of load transfer to the foundation is discussed in the earlier section. It can be easily envisaged that the modes of vibration will dictate the interaction of the foundations with the supporting soil. Furthermore, if the foundation–soil interaction is understood, the long-term behaviour of the foundation can be predicted through a combination of high-quality cyclic element testing of soil and numerical procedure to incorporate the different interactions. The next section of this paper summarises the modes of vibration of a wind turbine system.

Classification of OWTs based on modes of vibration

The modes of vibration depend on the combination of the foundation system (i.e. single foundation such as mono caisson or monopile or a group of piles or a seabed frame supported on multiple shallow foundations) and the superstructure stiffness. The fundamental modes of vibration can be mainly two types:

(a) *Sway-bending modes*: This consists of flexible modes of the tower together with the top RNA mass which is sway-bending mode of the tower. Effectively in these cases, the foundation is very stiff axially when compared with the tower and the tower vibrates and the foundation provides stiffness and damping.

(b) *Rocking modes*: This occurs when the foundation is axially deformable (less stiff) and is typical of WTG supported on multiple shallow foundations. Rocking modes can be also coupled with flexible modes of the tower.

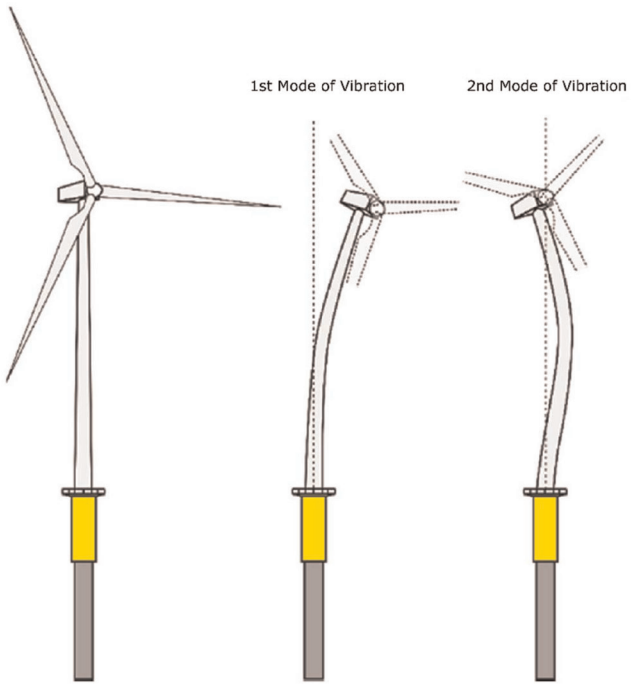

Fig. 12 *Modes of vibration for monopile supported wind turbines*

The next section describes the modes of vibration through some examples. These aspects were investigated by Bhattacharya *et al.* [21] through experimental testing where the modes of vibration were obtained from snap back test.

Sway-bending modes of vibration: Essentially this form is observed when the foundation is very rigid compared with the superstructure. Wind turbines supported on monopiles and Jackets supported on piles will exhibit such kind of modes. Fig. 12 shows a schematic diagram of modes of vibration for monopile supported wind turbines and Fig. 13 shows schematic diagram of a jacket supported wind turbine system. It is important to note that the first two modes are quite widely spaced – typical ratio is about four to six times.

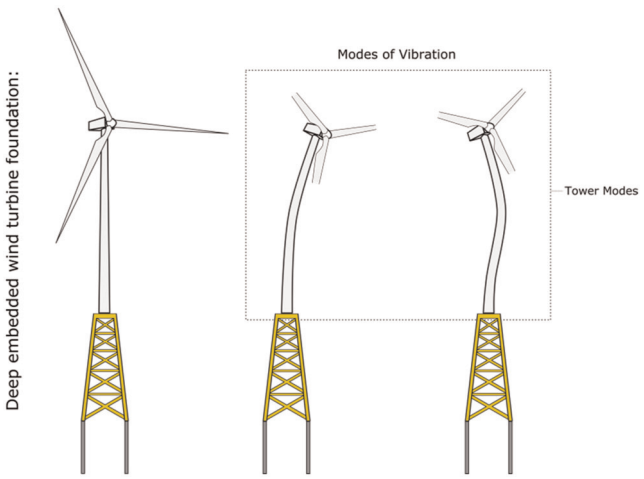

Fig. 13 *Schematic diagram of modes of vibration for jacket structures supported on piles*

These analyses can be easily carried out using standard software. Numerical simulation of a typical 3 MW monopile supported wind turbine system is carried out (but not presented) and it was observed that the natural frequency in first mode and second mode are 0.37 Hz and the third mode is 2.85 Hz. Similar observations were also noted for different types of jackets on piles (see e.g. Figs. 13 and 14).

The natural frequency of a monopile supported wind turbine system can be estimated following [8, 9]. This simplified methodology builds on the simple cantilever beam formula to estimate the natural frequency of the tower, and then applies modifying coefficients to take into account the flexibility of the foundation and the substructure.

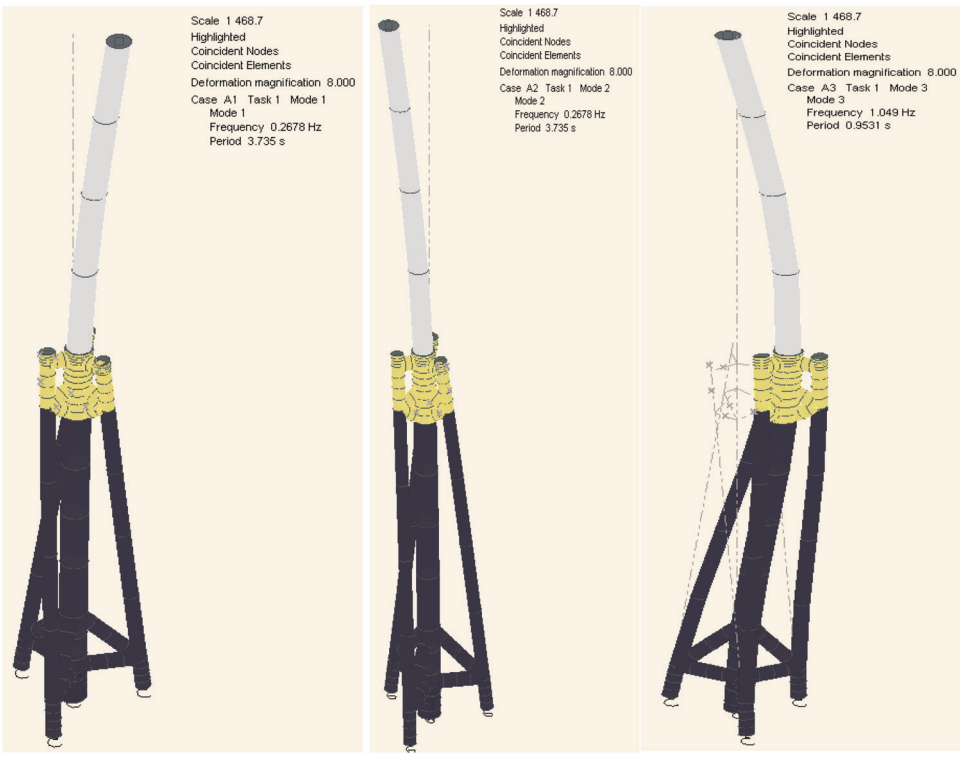

Fig. 14 *Twisted jacket – modes of vibration*

doi: 10.1049/etr.2016.0019

This is expressed as

$$f_0 = C_L C_R C_S f_{FB} \tag{2}$$

where C_L and C_R are the lateral and rotational foundation flexibility coefficients, C_S is the substructure flexibility coefficient and f_{FB} is the fixed base (cantilever) natural frequency of the tower.

Rocking modes of vibration: Rocking modes of foundation is typical of wind turbines supported on multiple shallow foundations (see e.g. Fig. 15, where wind turbine structures are supported on multiple bucket type foundations). This has been observed through scaled model tests and reported in [20, 21]. The foundation may rock about different planes and is dictated by the orientation of the principle axes, i.e. highest difference of second moment of area. Fig. 16 shows a simplified diagram showing the modes of vibration where the tower modes can also interact with the rocking modes, i.e. the tower may or may not follow the rocking mode of the foundation. Rocking modes of a foundation can be complex as they interact with the flexible modes of the tower. Few cases are discussed below:

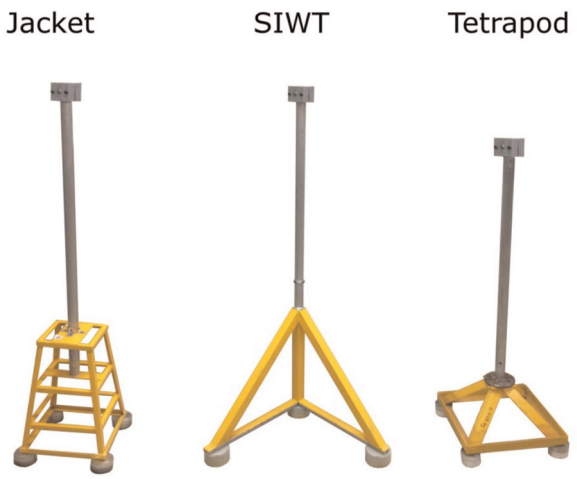

Fig. 15 *Different configuration of foundation*

(a) Jacket structure supported on four suction bucket (symmetric), ***(b)*** Seabed frame supported on three suction buckets (asymmetric, see Fig. 4*a*), ***(c)*** Tetrapod frame supported on four suction buckets

(a) *Wind turbine supported on symmetric tetrapod foundations*: Examples are given in Figs. 17 and 18 and a simplified model for analysis is also shown. Research shown by the authors in [20, 21] shows that even for same foundations under each support, there will be two closely spaced vibration frequencies. This is due to different vertical stiffness of the foundation associated with variability of the ground. However, after many thousands of cycles of loading and vibration, these closely spaced vibration frequencies will converge to a single peak.

(b) *Asymmetric tripod foundation*: Example is provided in Fig. 19 inspired by the concept shown in Fig. 4*a*. Study reported in [21] showed that there will two modes of vibration with closely spaced frequencies but with millions of cycles of loading, these two closely spaced peaks will not converge. This is because the foundation has two different stiffness in two orthogonal planes.

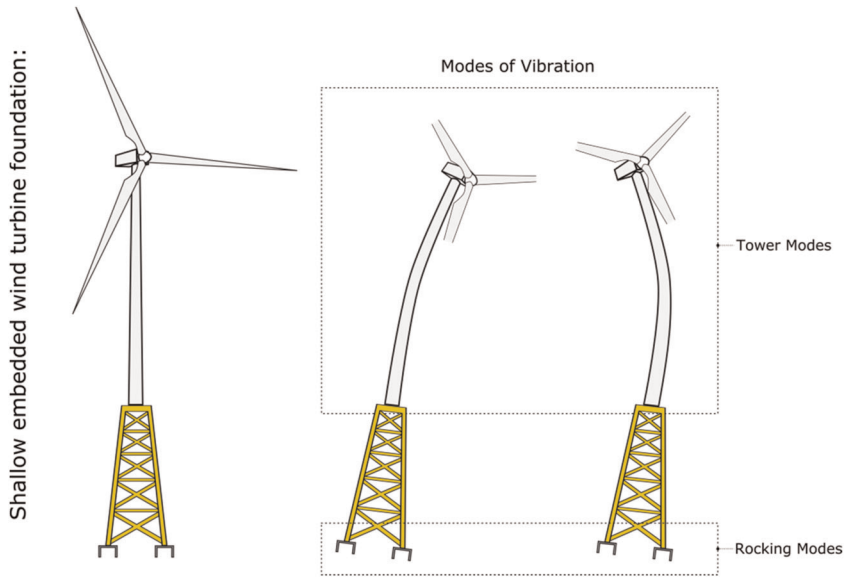

Fig. 16 *Rocking modes of vibration*

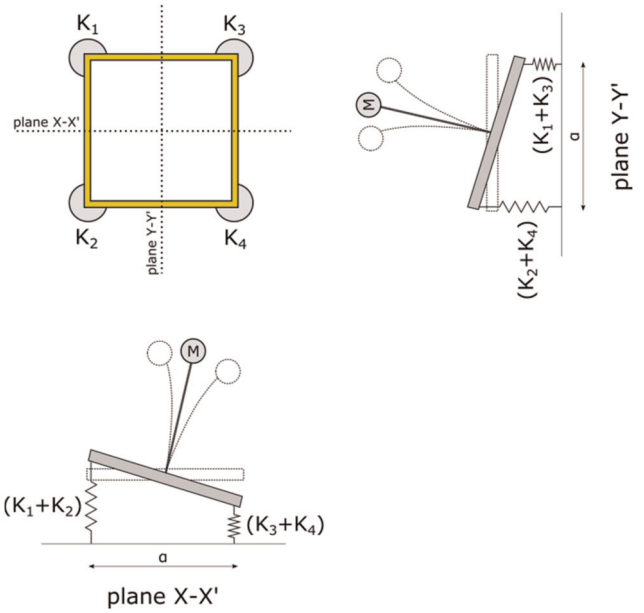

Fig. 17 *Rocking modes for a symmetric tetrapod about X–X′ and Y–Y′ plane*

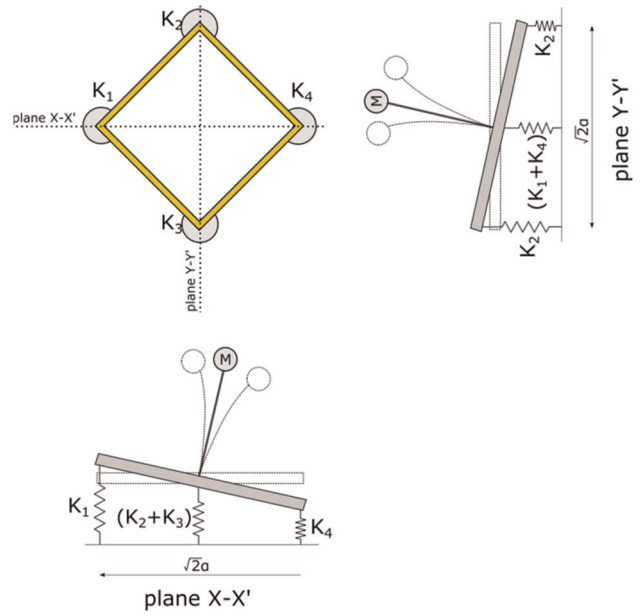

Fig. 18 *Rocking modes about diagonal plane*

(c) *Symmetric tripod foundation:* In a bid to understand the modes of vibration for a symmetric tripod, tests were carried out on a triangular foundation shown in Figs. 20 and 21. Free vibration tests were carried out and a typical result is shown in Fig. 22. The mode is like a '*beating phenomenon*' well known in physics which is possible for two very closed spaced vibration frequencies with low damping.

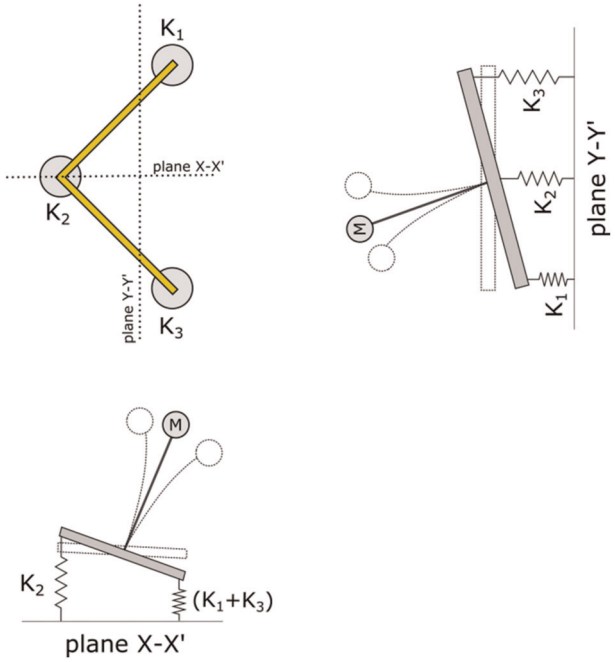

Fig. 19 *Modes of vibration for symmetric tripod*

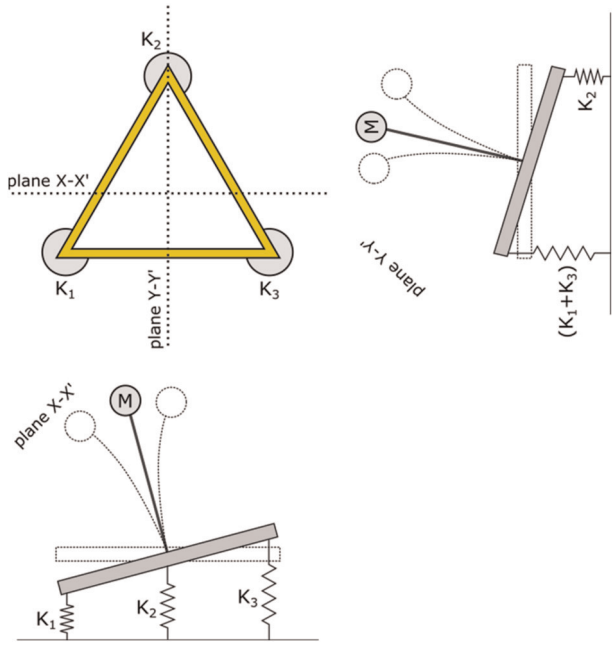

Fig. 20 *Symmetric foundation*

Taking into consideration Fig. 6 where the design of first natural frequency of the whole system is to be targeted between 1P and 3P, it is important not to have two closely spaced modes of vibration. In practical terms, it is therefore recommended to avoid an asymmetric system. The above study also shows that

doi: 10.1049/etr.2016.0019

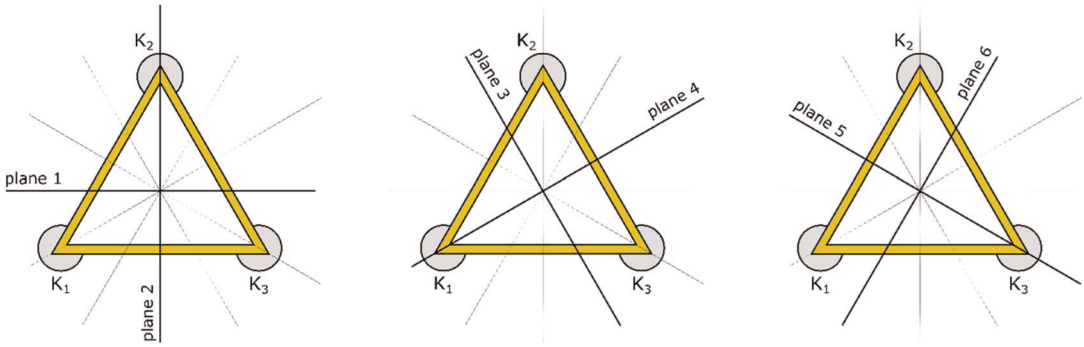

Fig. 21 *Planes of vibration*

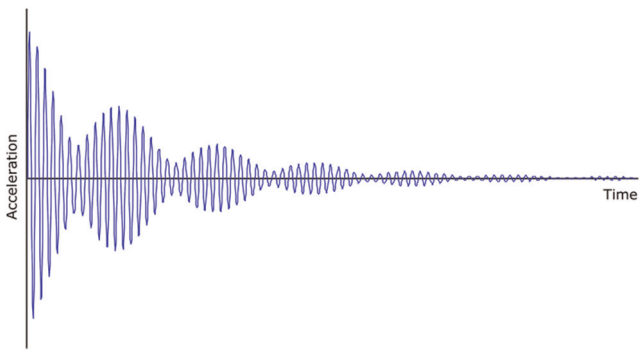

Fig. 22 *Free vibration acceleration response*

a symmetric tetrapod is better than symmetric tripod due to higher damping. It may be noted that beating phenomenon is typical of low damping and two closely spaced modes. Gravity based foundation will also exhibit rocking modes of vibration and it may also interact with tower flexible modes. Fig. 23 shows a schematic diagram of observed modes of vibration from a small-scale model test.

Discussion and conclusions

SSI can be classified based on the following:

(a) *Based on load transfer mechanism*: Monopiles will load the soil very differently than jackets. For a monopile, the main interaction is lateral pile–soil interaction due to the overturning moment and the lateral load. On the other hand, for a jacket, the main interaction is the axial load transfer. Therefore, the SSI depends on the choice of foundation and essentially how the soil surrounding the pile is loaded.
(b) *Modes of vibration*: The modes of vibrations are dependent on the types of foundations, i.e. whether the foundation is a single shallow or a summation of few shallow foundations or a deep foundation. Essentially, if the foundation is very stiff, we expect sway bending modes, i.e. flexible modes of the

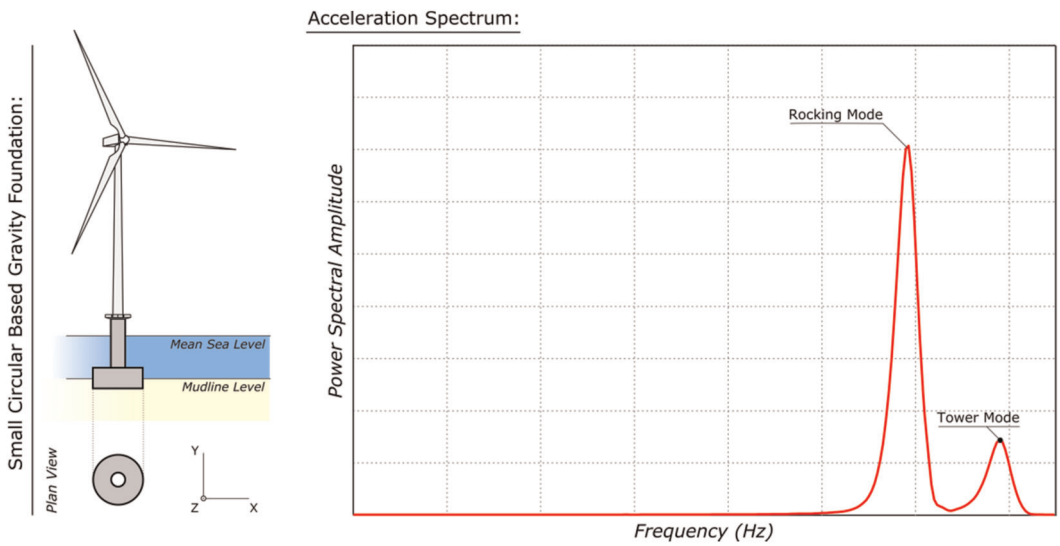

Fig. 23 *Modes of vibration for a small circular gravity based foundation*

tower. On the other hand, WTG supported on shallow foundation will exhibit rocking modes as the fundamental modes. This will be low frequency and it is expected that there will be two closely spaced modes coinciding with the principle axes. Two closely spaced modes can create additional design issues: such as beating phenomenon which can have an impact in fatigue limit state.

REFERENCES

[1] Arany, L., Bhattacharya, S., Macdonald, J., *et al*.: 'Simplified critical mudline bending moment spectra of offshore wind turbine support structures', *Wind Energy*, 2015, **18**, pp. 2171–2197

[2] Arany, L., Bhattacharya, S., Macdonald, J., *et al*.: 'Design of monopiles for offshore wind turbines in 10 steps', *Soil Dyn. Earthq. Eng.*, 2017, **92**, pp. 126–152

[3] Bhattacharya, S., Wang, L., Liu, J., *et al*.: 'Civil engineering challenges associated with design of offshore wind turbines with special reference to China', in 'Chapter 13 of wind energy engineering: a handbook for onshore and offshore wind turbines' (Academic Press (Elsevier), 2017), pp. 243–273

[4] Bhattacharya, S.: 'Chapter 12: civil engineering aspects of a wind farm and wind turbine structures', in Letcher, T.M. (Ed.): 'Wind energy engineering: a handbook for onshore and offshore wind turbines hardcover' (Elsevier, 2017), ISBN: 9780128094518

[5] Burton, T., Sharpe, D., Jenkins, N., *et al*.: 'Wind energy handbook' (John Wiley & Sons, Ltd, Chichester, UK, 2nd edn., 2011)

[6] Hu, WH, Thöns, S., Said, S., *et al*.: 'Resonance phenomenon in a wind turbine system under operational conditions'. Proc. of the 9th Int. Conf. on Structural Dynamics, EURODYN 2014, Porto, Portugal, 30 June–2 July 2014

[7] Bhattacharya, S.: 'Challenges in the design of offshore wind turbine foundations' (Engineering and Technology Reference, IET, 2014)

[8] Arany, L., Bhattacharya, S., Adhikari, S., *et al*.: 'An analytical model to predict the natural frequency of offshore wind turbines on three-spring flexible foundations using two different beam models', *Soil Dyn. Earthq. Eng.*, 2015, **74**, pp. 40–45, doi:10.1016/j.soildyn.2015.03.007

[9] Arany, L., Bhattacharya, S., Macdonald, J.H.G., *et al*.: ' Closed form solution of eigen frequency of monopile supported offshore wind turbines in deeper waters incorporating stiffness of substructure and SSI', *Soil Dyn. Earthq. Eng.*, 2016, **83**, pp. 18–32, doi:10.1016/j.soildyn.2015.12.011

[10] European Committee for Standardization (CEN): 'EN 1998-5:2004 – Eurocode 8: design of structures for earthquake resistance'. Part 5: Foundations, retaining structures and geotechnical aspects, 2004

[11] Shadlou, M., Bhattacharya, S.: 'Dynamic stiffness of monopiles supporting offshore wind turbine generators', *Soil Dyn. Earthq. Eng.*, 2016, **88**, pp. 15–32

[12] Abed, Y., Bouzid, D.A., Bhattacharya, S., *et al*.: 'Static impedance functions for monopiles supporting offshore wind turbines in nonhomogeneous soils – emphasis on soil/monopile interface characteristics', *Earthq. Struct.*, 2016, **10**, (5), pp. 1143–1179

[13] Jalbi, S., Shadlou, M., Bhattacharya, S.: 'Chapter 16: practical method to estimate foundation stiffness for design of offshore wind turbines', in 'Wind energy engineering: a handbook for onshore and offshore wind turbines hardcover' (Elsevier, 2017), ISBN:9780128094518

[14] Shadlou, M., Bhattacharya, S.: 'Dynamic stiffness of pile in a layered elastic continuum', *Geotechnique*, 2014, **64**, (4), pp. 303–319

[15] Aissa, M., Bouzid, D.A., Bhattacharya, S.: 'Monopile head stiffness for servicibility limit state calculations in assessing the natural frequency of offshore wind turbines', *Int. J. Geotech. Eng.*, 2017, pp. 1–17, doi:10.1080/19386362.2016.1270794

[16] Adhikari, S., Bhattacharya, S.: 'Vibrations of wind-turbines considering soil–structure interaction', *Wind Struct., Int. J.*, 2011, **14**, pp. 85–112

[17] Bhattacharya, S., Lombardi, D., Muir Wood, D.M.: 'Similitude relationships for physical modelling of monopile-supported offshore wind turbines', *Int. J. Phys. Model. Geotech.*, 2011, **11**, (2), pp. 58–68

[18] Bhattacharya, S., Adhikari, S.: 'Experimental validation of soil–structure interaction of offshore wind turbines', *Soil Dyn. Earthq. Eng.*, 2011, **31**, (5-6), pp. 805–816

[19] Adhikari, S., Bhattacharya, S.: 'Dynamic analysis of wind turbine towers on flexible foundations', *Shock Vib.*, 2012, **19**, pp. 37–56

[20] Bhattacharya, S., Cox, J., Lombardi, D., *et al*.: 'Dynamics of offshore wind turbines supported on two foundations', *Geotech. Eng., Proc. ICE*, 2013, **166**, (2), pp. 159–169

[21] Bhattacharya, S., Nikitas, N., Garnsey, J., *et al*.: 'Observed dynamic soil–structure interaction in scale testing of offshore wind turbine foundations', *Soil Dyn. Earthq. Eng.*, 2013, **54**, pp. 47–60

[22] Lombardi, D., Bhattacharya, S., Muir Wood, D.: 'Dynamic soil–structure interaction of monopile supported wind turbines in cohesive soil', *Soil Dyn. Earthq. Eng.*, 2013, **49**, pp. 165–180

[23] Nikitas, G., Arany, L., Aingaran, S., *et al*.: 'Predicting long term performance of offshore wind turbines using cyclic simple shear apparatus', *Soil Dyn. Earthq. Eng.*, 2017, **92**, pp. 678–683

[24] Yu, L., Wang, L., Guo, Z., *et al*.: 'Long-term dynamic behavior of monopile supported offshore wind turbines in sand', *Theor. Appl. Mech. Lett.*, 2015, **5**, (2), pp. 80–84

[25] Guo, Z., Yu, L., Wang, L., *et al*.: 'Model tests on the long-term dynamic performance of offshore wind turbines founded on monopiles in sand', *ASME J. Offshore Mech. Arctic Eng.*, 2015, **137**, (4), pp. 041902–041902-11

doi: 10.1049/etr.2016.0019