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Foundation damping for monopile supported offshore wind turbines: A review

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ABSTRACT

Today, an important challenge for offshore wind energy is to design efficient and reliable offshore wind turbines (OWTs). The overall damping of OWTs plays an important role in the design process as it limits the amplitude of the OWT dynamic response at frequencies near resonance. Therefore, an accurate estimation of OWTs damping is necessary for the efficient design of these systems. The foundation damping is one of the main sources overall turbine damping and is the least well understood. This paper presents a critical review of recently published studies on foundation damping for OWTs on monopiles and explains how soil damping contributes to the total damping of OWTs. It also reviews the main methods that have been used for the estimation of foundation damping in numerical and experimental studies. In addition, the importance of damping to the OWTs fatigue life is discussed. Finally, a discussion is provided on the challenges to be overcome and recommendations for the accurate estimation of foundation damping.

1. Introduction

1.1. Background to offshore wind

The first world-wide commitment to sustainable development was proposed by the Brundtland Commission in 1987 [1]. Climate change due to greenhouse gas emissions has led to the development of renewable energy technologies in order to de-carbonise the energy sector. As planning restrictions and local resistance to onshore wind increases, offshore wind energy offers a viable alternative with few of the drawbacks associated with onshore wind. Offshore wind is rapidly becoming one of the most important sources for renewable energy in Europe. By end of 2019, there was 22 GW of offshore wind capacity installed in Europe, with plans to develop a further 70 GW over the next 10 years [2]. This figure is set to increase substantially in the coming decades as nations strive to achieve ambitious targets in terms of CO_2 emission reductions under the Paris Climate Agreement.

OWTs have several advantages compared to their onshore equivalents. Offshore, the wind blows more steadily and at a higher average speed resulting in higher efficiency. Furthermore, OWTs result in less disturbance for local communities which allows for larger turbines to be used [3]. The cost of offshore wind has traditionally been one of the greatest barriers to date in relation to the adoption of this energy source over conventional sources. However, significant cost reductions have been achieved in recent years through increases in turbine size (see Fig. 1) and technological developments covering all aspects of the turbine design, and it is hoped

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that the next generation of advanced large OWTs will further reduce the Levelised Cost of Energy (LCOE). Significant research and development activities are ongoing both in Europe and worldwide in an effort to reduce the capital costs associated with offshore wind and to make technological advances that enable larger capacity wind turbines in deeper waters that can withstand aggressive environmental conditions.

A significant challenge to reduce the cost of offshore wind development is the design of efficient and reliable substructures for turbines. Foundation structures for offshore wind turbines can represent up to 30% of the overall development cost. Currently, monopiles are the most common foundation type supporting OWTs, which represent more than 80% of offshore wind turbine foundations installed to date [2,5]. Monopiles are large diameter steel tubes that are typically driven into the seabed with large impact hammers. Monopiles typically have a diameter, D, between 4 m and 8 m and length between 20 m and 40 m, but with turbines >10 MW being planned for upcoming projects, monopiles up to 10 m in diameter are currently being designed. The wall thickness for these monopiles is typically in the range D/80 to D/120. Recent joint industry research projects (e.g. PISA [6–10]) have led to new geotechnical design approaches for monotonic loading of monopile foundations, which have resulted in considerable cost savings on the overall structure. However, there is the potential to further increase the efficiency of the design by developing new approaches for the dynamics of the foundation, substructure and the turbine.

1.2. *Offshore wind turbine design process*

OWTs are slender and flexible structures, subjected to wind and wave loading, which are dynamically sensitive at low frequencies [11]. Therefore, the provision of sufficient damping is critical for offshore turbines to prevent excessive fatigue damage over the lifetime of the structure. The design of offshore wind turbines is a challenging and complex task, which traditionally involved separate design of the wind turbine, support structure and foundation. Optimizing the design of the OWT using this approach is very challenging due to the high degree of non-linearities and tightly coupled systems and it is now more common to perform an integrated design which allows for optimisation of the turbine, support structure and foundation simultaneously [12]. The interaction of the OWT substructure, foundation and rotor dynamics can be undertaken using commercial software which use either direct time integration methods or modal based methods. Direct time integration methods use an implicit solver for the full system matrices and are more rigorous but more computationally expensive than modal based methods [13]. In both time integration and modal based approaches, the non-linear hysteretic behaviour of soil plays an important role in terms of the dynamic response of the structure due to the foundation damping effects. These effects have neither been accurately quantified nor accounted for in existing design approaches, and yet have the potential to significantly increase the efficiency of offshore wind turbine designs. For a truly optimised design, it is essential that the soil-structure interaction is accurately modelled.

The majority of foundation models used in integrated design software are highly simplified and offer poor predictions of the true soil-structure interaction [14]. Different foundation models used in these codes include (i) assuming a rigid foundation, (ii) apparent fixity length (equivalent beam), (iii) linear macro-element (foundation stiffness matrix) model or (iv) Winkler foundation with discrete distributed springs (see Fig. 2).

Fig. 3 shows a flowchart which represents the main steps of foundation design for OWTs [15]. The process normally starts by guessing initial dimensions for the pile. Then, the loads on the foundation is calculated and will be used to check the global and local stability. If the design is acceptable for ULS, foundations stiffness and maximum mudline deformations will be calculated, otherwise, the pile dimensions need to be revised. Following by this, the natural frequencies and long-term behaviour of the foundation will be evaluated.

Of the different models available in integrated design software, the Winkler foundation model, often referred to as the p-y approach when only lateral soil reaction springs are used, can offer the most accuracy. Traditionally, the most widely used Winkler foundation model for monopiles is the API non-linear p-y approach which is described in various offshore design standards (API [16,17], & DNV

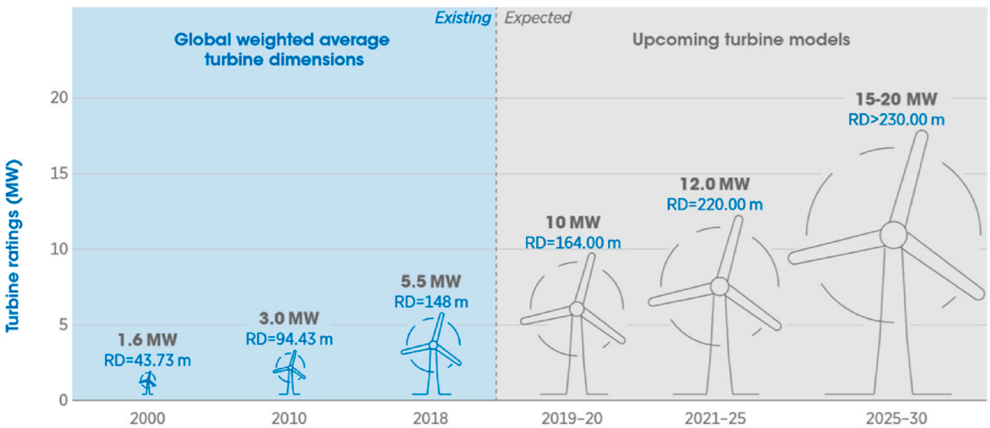

Fig. 1. Soil Growth in capacity and rotor diameter of wind turbines (after [4]).

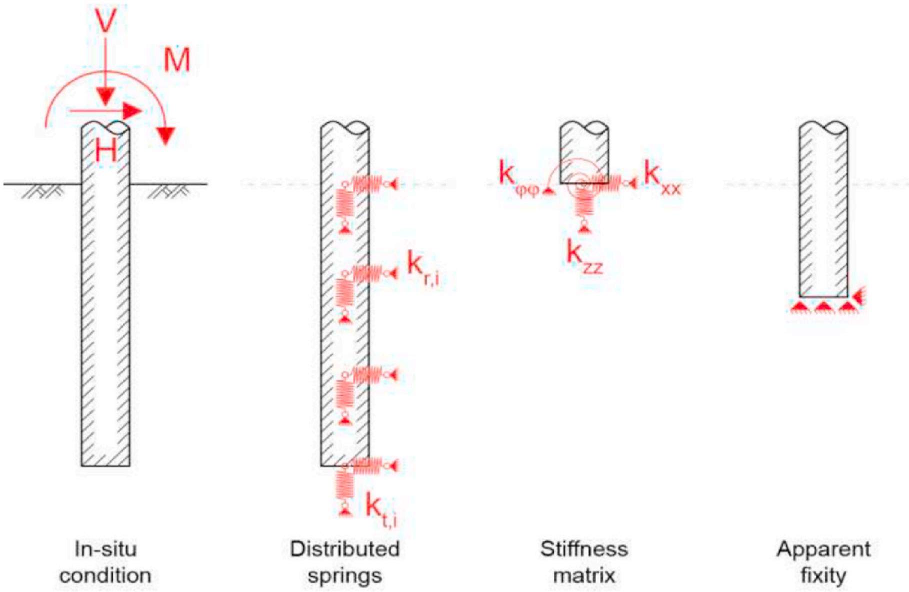

Fig. 2. Monopile foundation models used in integrated analysis (after [12]).

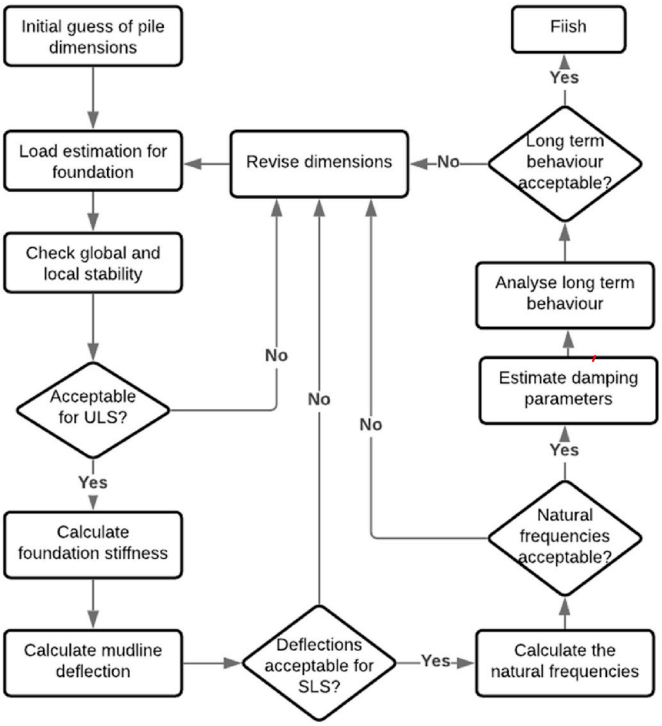

Fig. 3. The design process of monopile foundations (adapted from Ref. [15]).

[18,19]). Despite its widespread use, however, numerous studies have shown this method to be poor at predicting the response for large diameter monopiles. The recently completed PISA joint industry project has developed a new Winkler foundation model which includes non-linear lateral (p-y) springs and adds distributed rotational springs and offers significantly improved predictions for monopiles (PISA references), however is not yet widely available in integrated design software. In all of the approaches, the soil is typically modelled as elastic (with the same curves used for unloading and reloading), leading to no foundation damping. For direct time integration approaches damping is often added using Rayleigh damping while for modal based methods, it is common to add an overall damping value to each of the main resonance modes [13]. Neither of these approaches can accurately capture the foundation

damping affects.

Over the past few years, many researchers have presented new approaches for the identification of foundation damping for OWTs. In addition, some other techniques have been proposed for modelling of foundation damping in the design process of OWTs. This review is intended to introduce and summarize these approaches and provide recommendations for future development.

2. Damping of offshore wind turbines

In order to correctly estimate the dynamic behaviour of an OWT, an accurate estimation of the whole system damping is required [20]. Damping is generally defined as dissipation of the system energy to environment (normally in terms of heat). It is widely accepted that damping is a critical factor that can limit the amplitude of the dynamic response which would improve the fatigue life of the structure. Estimating damping from different sources is essential for predicting the fatigue life and optimizing the OWT design. OWTs are subjected to a combination of damping originated from various sources. According to the literature, there are five main sources of damping in OWTs [21]:

- Aerodynamic damping
- Hydrodynamic damping
- Structural damping
- Supplemental damping provided by mechanical dissipating devices
- Foundation (or soil) damping

The total damping in the system can be obtained as the sum of the damping from different sources (see Fig. 4). OWTs are lightly damped structures and their total damping varies in a wide range. For the turbines in the parked condition, the damping ratio (ξ) as a percentage of critical damping, may be in a range of 1–3% and for the ones in operational condition, in a range of 7–10% [20]. The following sub-sections briefly review the recent studies have been published about each of these damping sources. However, the main focus of this paper is on foundation damping.

2.1. Aerodynamic damping

The main source of aerodynamic damping is the interaction of wind turbine and forcing air acting on the structure [22]. In operational condition, the aerodynamic damping highly contributes to the overall damping of the OWT. But during the rotor-stop condition, the aerodynamic damping is almost negligible. This is an advantage for the cases that other sources of damping need to be estimated. In this condition, the other damping sources can be identified [23,24]. Researchers have reported aerodynamic damping within a range of 4%–8% in the for-aft (FA) direction and 0.08%–1.43% in side-side (SS) direction [20,25–30]. These values are highly

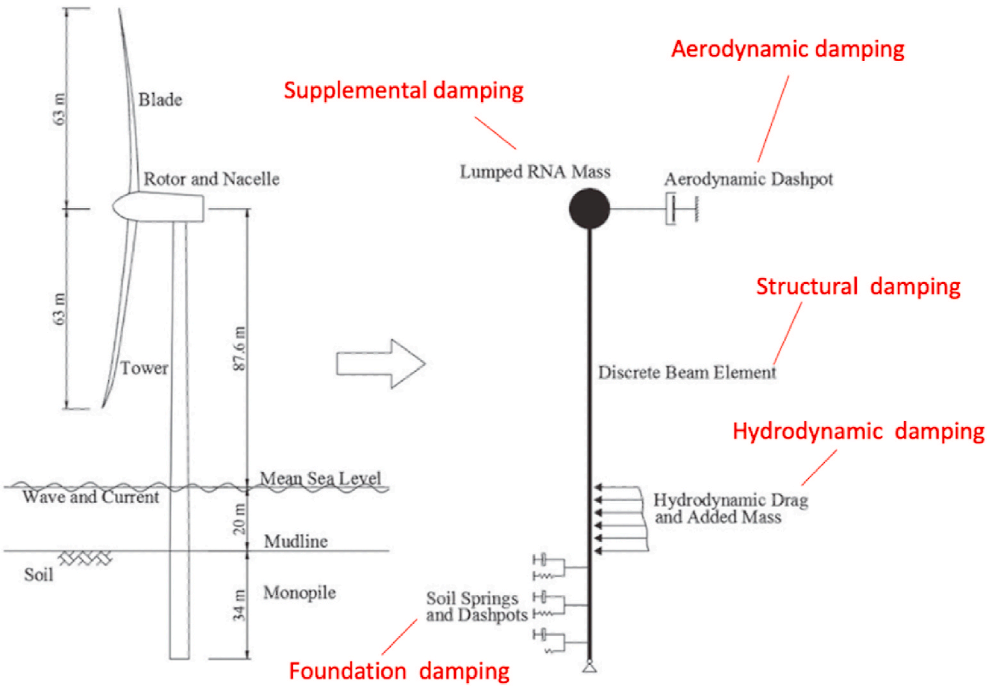

Fig. 4. Different sources of damping (after [20]).

dependent on factors like the wind speed, the rotation speed, the pitch angle of blades and the yaw angle of the rotor [20,24,28]. Several authors [24,25,27] have implemented analytical models for predicting aerodynamic damping of OWT. The general conclusion of analytical studies is that these methods usually underestimate the aerodynamic damping when the wind speed is much higher than the rated wind speed [20].

2.2. Hydrodynamic damping

There are two main sources for hydrodynamic damping; water wave radiation and damping due to hydrodynamic drag [31,32]. The hydrodynamic drag is proportional to the structure velocity and is almost negligible due to low velocity. The wave radiation is a function of relative velocity and has a larger influence. The values have been reported for hydrodynamic damping of OWT vary widely in a range between 0.07% and 0.23% in the literature [33,34]. GL [35] suggested the radiation damping of the order 0.11–0.22% and an upper limit of drag damping of 0.15%. In most cases the conclusion is that the values of hydrodynamic damping are considerably lower compared to the other damping sources.

2.3. Structural damping

Structural damping is the dissipation of energy along the structure when it vibrates. It originates from internal friction of the structure material which transforms energy to heat. The damping values as defined in standards for steel structures are typically used for structural damping of OWTs. For example, Damgaard et al. [36,37] report structural damping for OWT to be 0.19% which is implemented in Eurocode (EN 1991, 2005) [38] while GL [35] recommends steel damping values between 0.2 and 0.3%. Arany et al. [39] suggest values from 0.15% to 1.5% can be expected. Shirzadeh et al. [34] report values in a range of 0.5–1.5% where the lower values are usually considered for pure material damping and higher values are for structures with additional damping sources like joints.

2.4. Supplemental damping

As fatigue is an important parameter in the design of OWTs, it is necessary to reduce the dynamic responses of the structure. One way is the application of structural control techniques usually used in skyscrapers and bridges. Tuned mass dampers are usually used in OWTS to reduce loads and the amplitude of vibrations. Tower oscillation dampers (tuned mass dampers) are systems integrated to OWTs to reduce the amplitude of vibrations [40–44]. These systems usually introduce a high amount of damping to OWTs. For example, Damgaard et al. [36] present a system which introduces 1.36% damping for a particular OWT.

2.5. Foundation damping

During operating conditions, the foundation (or soil) damping is considered to have the second largest contribution to the overall OWT damping, after aerodynamic damping, but when the turbine is idle or when the side-side behaviour is considered, the aerodynamic damping is almost negligible and foundation damping is the most prominent [35]. Foundation damping is sometimes neglected in the design process of OWTs due to lack of a detailed methodology in the current design guidelines [45]. Foundation damping has historically been the least studied and presents the largest discrepancy between measured and theoretical results [35], although there has been a significant drive in recent years to rectify this situation. Foundation damping consists of the energy loss though radiation of elastic waves and soil material damping. The sources of foundation damping for a monopile can be summarized as follows:

- Radiation damping
- Pore-water dissipation (seepage) damping
- Soil material damping

2.5.1. Radiation damping

Radiation damping (also known as geometrical or external damping) is the result of the energy dissipation which occurs due to elastic waves spreading across the soil volume surrounding the monopile [31]. This type of damping is frequency dependant and is important where loadings occurs at high frequencies (>1 Hz). It is generally believed that radiation damping is negligible for OWTs when the loading frequency of wind and wave loadings is typically below 1 Hz [39,46,47] and thus not considered further within this paper. The authors refer to the work of Gazetas and Dobry [48] and Shadlou and Bhattacharya [49] for studies of radiation damping around piles.

2.5.2. Pore water dissipation (seepage) damping

Damping due to the seepage of pore water between soil particles (equalisation of excess pore pressures) is regarded as viscous and is proportional to the velocity and frequency but independent of soil strain level. To understand the importance of seepage damping, one should consider the drainage conditions around a monopile. In fine grained soils with low permeability (e.g. clays), soils will behave in an (almost) undrained manner with zero volumetric strain, where there is insufficient time for excess pore-water pressures to dissipate between load cycles and thus there will be no viscous damping. In coarse-grained soils with medium to high permeability (e.g. sands or silty sands), the soil surrounding a monopile may behave in a drained, partially drained or fully undrained manner, depending on the rate of loading, drainage length and drainage properties of the soil. A number of researchers have attempted to develop hydro-mechanical models to quantify this affect [50,51], however to date there is no widely accepted model which can accurately capture both the stress-strain and drainage response of monopiles. Li et al. [52] used 3D FEM to examine the drainage properties around large diameter (6 m) monopiles in sand and determined that an undrained soil response was generally expected within a single load cycle. It was noted, however, that the average component of cyclic loading (for example due to mean wind speed) would be reasonably expected to result in drained behaviour (due to the lower rate of loading) while the cyclic component (e.g. wave and variable wind loading) would result in undrained loading. Li et al. [52] concluded by suggesting that while drainage conditions are important for large loads, the influence of drainage conditions may be insignificant for typical monopile loads, which are small compared to the ultimate capacity (Fig. 5). Beuckelaers [13] suggests that pore pressure dissipation is not expected to lead to significant damping for monopiles, making only minor contributions in very permeable soils. It can be concluded that the most important contribution to monopile foundation damping comes from the soil material damping.

2.5.3. Soil material damping

Soil material damping (also known as internal damping) is the dissipation of energy within the soil mass due to friction, sliding between particles and structural rearrangement. Zhang et al. [53] suggests that the mechanisms which contribute to soil material damping are friction between the soil particles, strain rate effects and non-linear soil behaviour. One of the complexities of soil material damping is the relationship with pore water pressures through the changes in effective stress within the soil. While the pore water seepage can lead to viscous damping, changes in excess pore water pressure result in changes in the soils effective stress which affects the material damping. O'Reilly and Brown [54] suggests that "not all time dependent stress-strain responses of real soils are of viscous origin" and that some time-dependent characteristics of soils which are associated with the flow of pore water can be explained through an inviscid constitutive model in which soil behaviour is controlled by effective stresses. Despite these suggested rate affects, soil material damping for monopiles is thought to be generally insensitive to frequency [55] or rate of loading [3], but highly dependent on the soil strain, ε. In geotechnical engineering, an equivalent damping ratio, D, (discussed later) is typically used to characterize energy dissipation in soils:

$$D = \frac{1}{4\pi}\frac{\Delta W}{W} \tag{1}$$

where ΔW is the energy dissipated in one cycle of loading, and W is the maximum strain energy stored during the cycle (see Fig. 6). In theory if a loading stress-strain curve is purely elastic, the area under the unload curve, and hence the material damping ratio, will be zero. However, Zhang et al. [53] suggest that even at very low strain levels (which should behave linear-elastically) there is always some energy dissipation measured in laboratory soil tests. They also suggest that the damping ratio at very low strain levels is a constant value, referred to as the small-strain damping ratio, D_{min}. At higher strain levels, the non-linear hysteretic behaviour leads to an increase in the material damping ratio.

Soil material damping can be measured on a soil specimen in the lab using a different equipment including resonant column, torsional shear, cyclic triaxial or cyclic direct simple shear. There has been extensive research on soil material damping from laboratory testing on soil samples, mainly focused on the seismic response of soils, and it has been shown that the soil damping ratio, D, can be linked to the shear modulus degradation with shear strain ([53,57–64] add 2005 and others), see Figs. 7 and 8. In the absence of site specific lab tests to determine the soil shear modulus degradation or damping curves, these databases of soil lab tests can be used to reasonably assess the range of soil material damping ratio for a given soil element as a function of shear strain for a given material type. While the majority of lab tests relate the soil damping ratio and shear strain, it should be noted that volumetric strain may also result in damping. Soils will typically change volume (dilate or contract) during shearing, and prolonged cyclic loading may result in

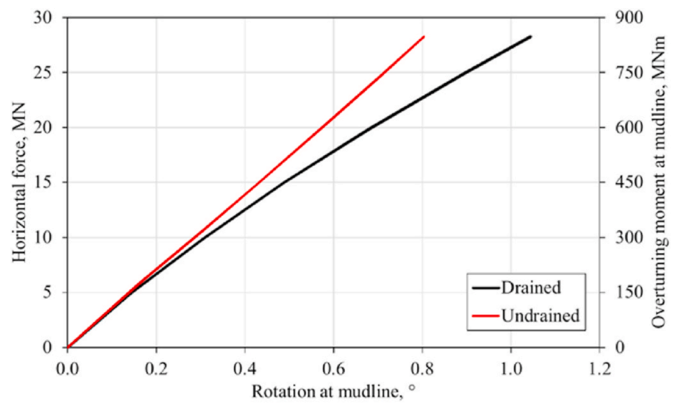

Fig. 5. Comparison of load-displacement responses for drained and undrained conditions (after [52]).

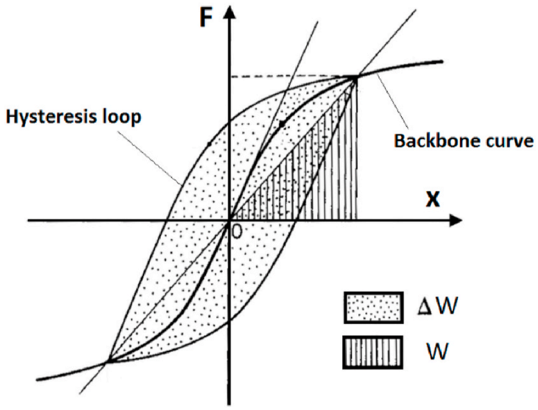

Fig. 6. Soil Hysteresis loop (adopted from Ref. [56]).

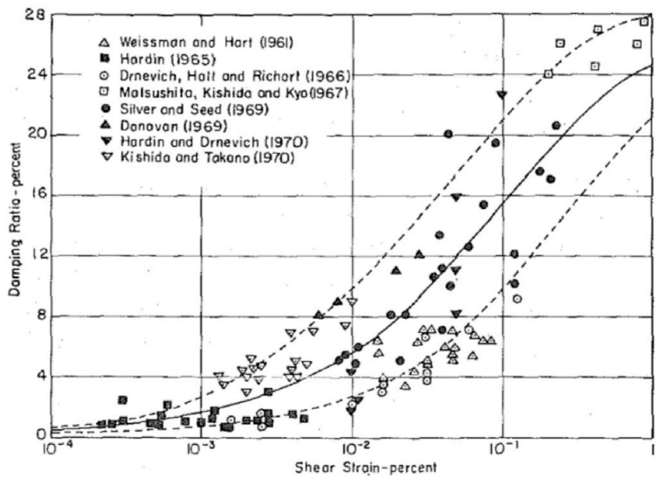

Fig. 7. Database of Soil Damping Factor for sands (after [57]).

irrecoverable plastic volume changes (e.g. densification) in the soil, which may also contribute to the soil material damping, although this affect is expected to be small relative to the damping due to plastic shear strains.

When discussing soil material damping for a system such as a monopile, it can be useful to further subdivide the damping into intrinsic damping, which describes the energy loss at a specific point within the material, and extrinsic damping which characterises the global energy loss in a finite volume [65]. For a monopile supported OWT, the energy lost through soil-material damping is the integrated effect of the material damping from all soil elements influencing the monopile response:

$$D_{global} = \frac{1}{V}\int D(\varepsilon)dV \tag{2}$$

where D_{global} is the global (extrinsic) material damping ratio, $D(\varepsilon)$ is the material intrinsic damping as a function of strain and V is the volume of soil strained by the monopiles response. If the soil damping is linear (i.e. not a function of strain), then $D_{global} = D$, [65]. However, as the soil response and damping is highly non-linear, solving equation (2) is not straight forward, and therefore it is a challenge to accurately determine the global soil material damping ratio for monopiles directly from intrinsic soil damping values or damping values measured from soil element lab tests. The following sections review the various approaches which have applied to determine monopile foundation damping. Fig. 9 summarizes all the sources of damping for OWTs which have been discussed in this section.

3. Numerical modelling approaches for foundation damping in integrated OWT design

In most basic analyses soils can be modelled as a simple elastic (non-dissipative) material. Energy dissipation or damping during cyclic loading can be added by the introduction of extra plasticity or viscosity or both [54].

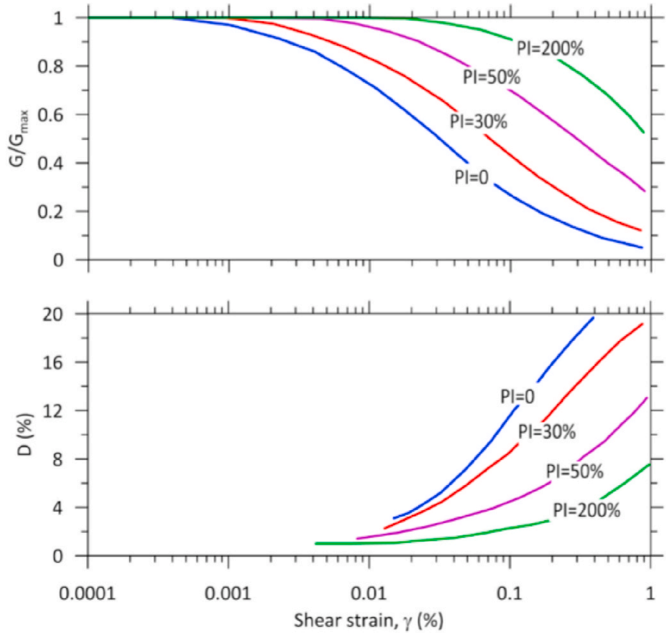

Fig. 8. Shear modulus and damping ratio curves for clays (after [58]).

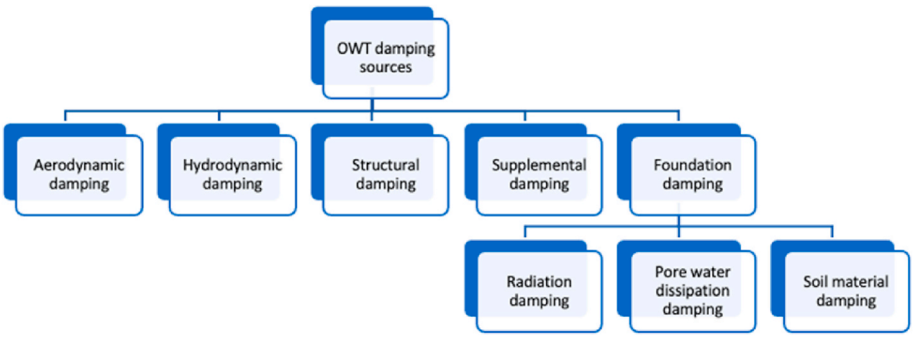

Fig. 9. The main sources of damping in OWTs.

3.1. *Viscosity and complex stiffness based approaches for integrated foundation damping*

There are numerous mathematical damping formulations which are commonly applied in the analysis of OWTs. The Kelvin-Voigt (KV) model is one of the simplest approaches to adding damping to a single degree of freedom (SDOF) system and it involves adding a purely viscous damper in parallel to an elastic spring (see Fig. 10a), so that the strains in the spring and the damper are equal. For a viscoelastic SDOF system, the general equation of motion is given by:

$$m\ddot{x}(t) + c\dot{x}(t) + kx(t) = F(t) \quad (3)$$

where $x(t)$ is the displacement and $F(t)$ is the applied force as a function of time (t), m is the mass, c is the viscous damping coefficient

Fig. 10. (a) Kelvin-Voigt viscous damping model with inertia and (b) Complex Stiffness (CS) model (after [65]).

and k is the elastic stiffness. The damping ratio for a SDOF KV system, ξ, can be expressed as:

$$\xi = \frac{c}{c_r} = \frac{c}{2\sqrt{k.\,m}} \tag{4}$$

where, c_r is the coefficient of critical damping. Due to damping, the displacement (or strain) response within a system lags the input force (or stress) by a phase angle, φ, which for a SDOF KV system with inertia can be given as:

$$\varphi = \tan^{-1}\left(\frac{c\,\omega}{k - m\,\omega^2}\right) \tag{5}$$

where ω is the circular frequency of the applied force. For a constant value of the damping coefficient, c, the response of a KV model is frequency dependent. In geotechnical engineering, an equivalent damping ratio, D, is used to characterize energy dissipation in soils, as the term damping ratio ξ only applies to KV SDOF systems, and they can be related by Ref. [41]:

$$D = \xi\Omega = \xi\left(\frac{\omega}{\omega_n}\right) \tag{6}$$

where Ω is the frequency ratio and ω_n is the circular natural frequency. The equivalent damping ratio is equal to the KV SDOF damping ratio (D $= \xi$) when at resonance [65]. The amplitude of the dynamic response can also be represented by a magnification or quality factor, Q, defined as the ratio of the amplitude of displacement to the equivalent static displacement, and can be related to the damping ratio as:

$$Q = \frac{1}{\sqrt{(1-\Omega^2)^2 + (2\xi\Omega)^2}} \tag{7}$$

At resonance, $\Omega = 1$ and the equation simplifies to $Q = \frac{1}{2D}$. Q is a measure for skewness of a frequency peak where the higher and narrower they are, the lesser these frequencies are damped (see Fig. 11). Hysteretic material damping can alternatively be modelled using a complex stiffness (CS), $k^* = k + Bi$ where B is the imaginary parts of the complex stiffness and, $i = \sqrt{-1}$ [65]. As seen from Fig. 10, the KV and CS models are essentially equivalent with the basic difference being the constants c and β applied in each model. The parameter B in the CS model is equivalent to $c\omega$ in the KV model [65]. Due to its frequency independence, the CS model is often used in soil dynamics to model hysteretic behaviour and is typically applied to the soil shear modulus, G, such that [41]:

$$G^* = G(1 + \eta\,i) \tag{8}$$

where η is the loss factor. For small values of the damping, the loss factor η can be related to the equivalent damping ratio, D through the expression [41]:

$$\eta = \frac{c\omega}{G} = 2D \tag{9}$$

Rayleigh damping is a slightly more complex form of viscous damping commonly used in structural analysis, where for a SDOF system the damping coefficient c is expressed as a linear combination of mass and stiffness ($c = \alpha\,m + \beta\,k$). Rayleigh damping is often used to represent foundation damping in direct time integration approaches in integrated monopile design software. While useful, one of the disadvantages of this approach is that the damping ratio varies with the response frequency. The values of α and β are chosen so that the damping is equal to the expected value at two chosen frequencies, which results in a fixed damping value at other frequencies which may not accurately represent the foundation damping [13]. Another disadvantage is that the foundation damping is

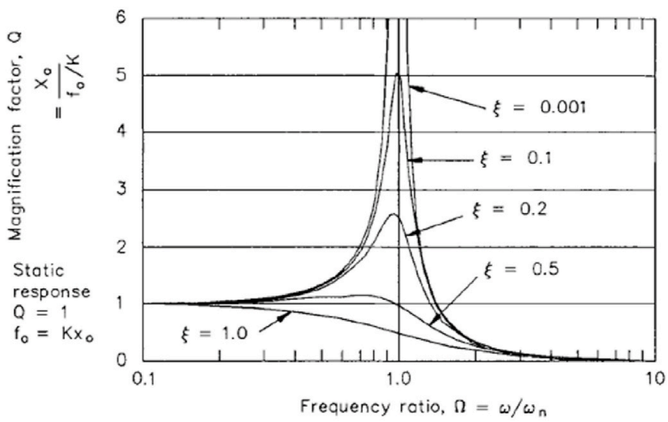

Fig. 11. Dynamic magnification factor, Q, as a function of frequency ratio with different levels of damping ratio (from Ref. [66]).

strain-independent and thus cannot accurately capture the entire foundation damping response at different load/strain levels.

3.2. Plasticity based approaches for integrated foundation damping

The shortcoming of the viscosity type damping models can be overcome by introducing plasticity. A number of researchers [13, 67–71] have focused on integrating plasticity models into commercial monopile design software using either elasto-plastic macro--element or distributed Winkler spring models. Page [72] argued that macro-element models have some advantages compared to distributed spring models such as reduced complexity, lower computational effort and easier calibration, especially in layered soils. While this is true, the Winkler foundation model offers a better numerical representation of the monopile and is better able to capture different monopile geometries and soil conditions without the need to re-calibrate the model each time. It can also be argued that with ever improving computational power, the Winkler foundation model is generally the most suitable for the range of design requirements of monopile foundations. The Winkler approach is regarded as being a reasonable compromise between model accuracy and computational cost when compared to macro-element models and full 3D finite element continuum models [3].

The starting point for the introduction of plasticity into macro-element or Winkler foundation models is the often the use of a kinematic hardening elasto-plastic model. In a hardening elasto-plastic model, the displacement (or strain) is decomposed into an elastic and a plastic part, with the elastic strain being proportional to the applied stress through an elastic stiffness matrix, and the plastic strain depending on a yield criterion, a flow rule and a hardening law [69]. In a kinematic hardening elasto-plastic model, the yield surface translates upon plastic straining, but does not change in size (as opposed to an isotropic hardening model). A kinematic hardening model is often preferred over other plasticity models as its application can directly result in the extended Masing's rule [73] upon unloading and reloading [3], which has been found to be reasonable for modelling the cyclic hysteretic response of a soil-structure interactions in certain situations. While there are different approaches to applying Kinematic hardening, a multi-surface plasticity model is often favoured due to its more flexible calibration, where the plastic displacements depend on how a family of nested surfaces move in the load space, with kinematic hardening defined by piece-wise linear curves [69]. Fig. 12a shows a typical Masing's rule hysteresis curve and Fig. 12b shows the stress-strain response under load cycles using a multi-surface kinematic hardening model (note the curve is broken into piecewise linear sections). It can be seen that the unloading and reloading curves are geometrically similar to the initial-loading (backbone) curve except the direction is reversed and it is increased in scale two-fold.

Page [72] developed a series of papers presenting different macro elements foundation models for OWTs using multi-surface plasticity with kinematic hardening. Three separate models were developed starting with the simplest model 1 [68] which coupled the horizontal force, H, and moment, M, at the seabed with the displacement and the rotation and used a simplistic band shaped yield surface in H–V space (V = vertical force) and was capable of modelling hysteretic behaviour under uni-directional loading. Model 1 was verified using 3D Finite Element analysis of the soil but the shape of the yield surface meant that the model was only suitable for low-load levels and was not suitable for layered soils. Page [72] advanced on this by developing a second model which had an elliptical shaped yield surface in H–V space which resulted in improved performance in non-homogenous soils and was validated using the results of the PISA field tests in clay. Model 2 was again only suitable for uni-directional loading. A third model [70] was developed which was capable of modelling hysteretic behaviour under multi-directional lateral loading which was again validated using the PISA field tests in clay. It should be noted that all of the models were developed and verified/validated for clay soils. Some of the drawbacks to the approaches adopted by Page are the inability of the models to include ratcheting (accumulation of strain under cyclic loading), gapping (when gaps form around the monopile as it displaces) or rate effects on the soil.

Houlsby et al. [74] described the development of a kinematic hardening macro element model, based on the principles of hyperplasticity, which can account for ratcheting behaviour. The model, referred to as the Hyperplastic Accelerated Ratcheting Model

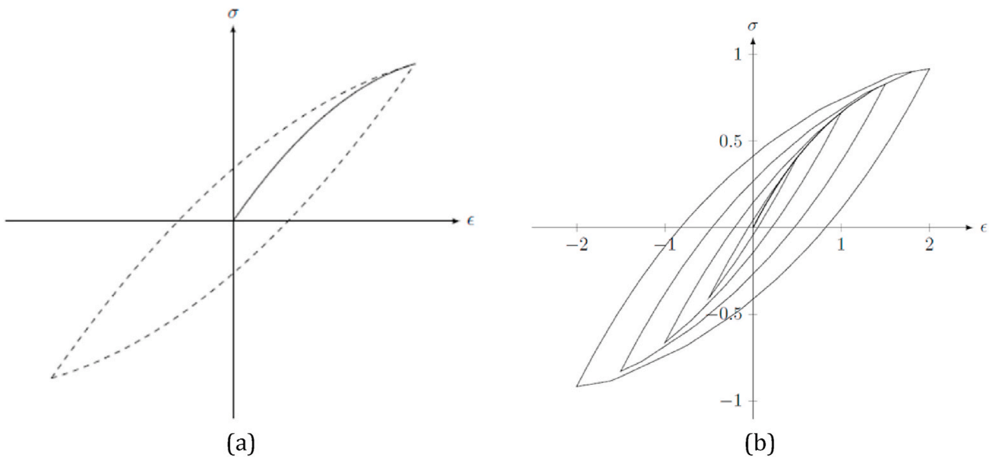

Fig. 12. (a) Initial backbone curve and steady state hysteretic response using the Masing rules and (b) illustration of the multi-surface kinematic hardening model upon cyclic loading at varying strain amplitudes (from Beuckelaers 2017 [3]).

(HARM), can employ an accelerated ratcheting term for a high number of cycles, negating the need to model each cycle individually. Aside from macro-element models, a series of extensions for the Winkler foundation model have been proposed based on the same hyperplastic theoretical framework, primarily through the work of Beuckelaers [3]. Beuckelaers [13] proposed a kinematic hardening extension to the industry standard API p-y model which follows the extended Masing's rules and could capture the hysteretic response of a pile under uni-directional loading. Beuckelaers et al. [71] extended this model to multi-directional lateral loading and integrated this model into the commercial wind turbine software HAWC2. Beuckelaers [3] used soil reaction curves calibrated from the PISA field tests, and developed multi-surface kinematic hardening models which were capable of modelling the effects of rate, ratcheting and gapping for a monopiles.

It should be noted however that at the time of writing these advanced kinematic hardening models are not widely applied in industry. While all these models represent important theoretical advancements in modelling approaches for offshore wind turbine foundation, significant challenges remain particular regarding the calibration of ratcheting and gapping affects. The authors believe that the use of these foundation models will become more prevalent in the future as experience in calibrating and validating these models improves. Other approaches to estimate foundation damping for monopiles can be made through the use of 3D Finite Element continuum models, although currently these cannot be directly used with integrated design software, or through experimental measurements from OWTs. The following section reviews some of the studies undertaken in this area.

4. Damping studies for offshore wind turbines founded on monopiles

The identification and the modelling of the foundation damping has been identified as an important challenge in recent years and several researchers have employed experimental and numerical studies to quantify foundation damping [75]. Table 1 summarizes the most important studies undertaken so far. Some of them identify foundation damping separately and some others as part of total damping. This section reviews the most recent studies for quantification of foundation damping.

4.1. Experimental studies

The logarithmic decrement method is a well-known method for identifying the damping ratio from free decay responses. For example, Fig. 13 shows a sample a displacement free vibration response measured at the RNA level of an OWT. The logarithmic decrement damping is defined by:

Table 1
Summary of the main studies for foundation damping in OWTs.

Reference	Overall damping	Foundation damping	Method for determining damping			OWT conditions		Experimental	
			Modal damping	Free decay	hysteretic	Rotor-stopped	Operating	Full scale experimental	Lab scale experimental
Carswell et al. (2015) [46]	✓	✓	×	✓	✓	✓	×	×	×
Lombardi et al. (2013) [11]	✓	×	×	✓	×	×	✓	×	✓
Shirzadeh at al. (2013) [34]	✓	×	✓	×	×	✓	✓	✓	×
Chen et al. (2018) [20]	✓	✓	×	×	×	✓	✓	×	×
Koukoura et al. (2015) [76]	✓	✓	✓	✓	×	✓	✓	✓	×
Fontana et al. (2015) [77]	✓	✓	×	×	×	✓	×	×	×
Aasen et al. (2017) [47]	✓	✓	×	✓	✓	×	✓	×	×
Bayat et al. (2016) [50]	×	✓	×	×	×	×	×	×	×
Versteijlen at al. (2017) [21]	×	✓	✓	×	×	×	×	✓	×
Damgaard et al. (2013) [55]	✓	✓	✓	✓	✓	✓	✓	✓	×
Rezaei et al. (2018) [78]	✓	✓	×	×	×	✓	✓	×	×
Weijtjens et al. (2014) [79]	✓	×	×	×	×	✓	✓	✓	×
Versteijlen et al. (2011) [80]	✓	×	✓	✓	×	✓	×	✓	×
Damgaard et al. (2015) [81]	✓	✓	×	×	×	✓	×	×	×
Beuckelaers (2017) [3]	✓	✓	×	✓	✓	✓	×	✓	×

$$\delta = \frac{1}{n}\ln\left(\left|\frac{A_0}{A_n}\right|\right) \tag{10}$$

where A_0 and A_n are two extremes in the response placed with the time interval. The following equation can be used for converting logarithmic decrement, δ to equivalent damping ratio, D:

$$D = \frac{1}{\sqrt{1+\left(\frac{2\pi}{\delta}\right)^2}} \tag{11}$$

One of the earliest studies on the use of logarithmic decrement method for the estimation of OWTs damping is the work done by Versteijlen [80] in 2011. They measured vibration signals from twelve 'rotor stop'- test on an OWT at Dong Energy owned - Burbo Banks wind farm where an accelerometer and strain gauges were installed along the tower to measure free decay responses. Fig. 14 shows the transition from feathering position to production and then again feathering position to create free decay in for-aft direction.

The authors use the Quality, Q, factor technique in frequency domain and logarithmic decrement technique in time domain for identifying damping from the free decay. The average magnitude of the total identified damping in the measurements was estimated to be 19% logarithmic decrement (3% damping ratio) for the first natural bending frequency of 0.296 Hz and 9.5% logarithmic decrement (1.5% damping ratio) for the second frequency. The design value for this turbine is 2.5% logarithmic decrement (0.4% damping ratio) of damping for the first natural frequency which is much smaller than the identified value in the measurements. Damgaard et al. [55] investigated more than 1500 "rotor-stop" tests in the period 2006–2011 at four offshore wind farms. Two accelerometers were installed at top of the tower in the fore-aft and the side-side directions. The authors use least-squares fitting of a linear function to the natural logarithm of the rate of decay of the transient response to find the damping. Each time history was divided into bins with 10 maxima and 10 minima and then the logarithmic decrement was estimated. The authors assumed that the tower behaves almost as a single-degree-of-freedom (SDOF) system with linear viscous damping. Mean values of the first modal damping in terms of the logarithmic decrement were found to be in the range of 15–16% (2.4–2.5% damping ratio). The contribution of the soil damping to the measured system damping for a specific turbine in terms of the logarithmic decrement was estimated to be 6% (1% damping ratio). In addition, a mean value of the modal damping in terms of the logarithmic decrement for the side-side vibration was found to be in the range of 16–18% (2.5–2.9% damping ratio). Bajric et al. [82] employed three modal identification techniques, Eigensystem Realization Algorithm (ERA) [83], covariance driven Stochastic Subspace Identification (COV–SSI) [84] and the Enhanced Frequency Domain Decomposition (EFDD) [85] to estimate the OWT damping. The authors used a numerical model of an OWT in non-operating condition, where the total damping was assumed to be governed by structural and foundation damping. Damping was also estimated from accelerations measured on a real wind turbine. The average damping was estimated to be about 0.7% damping ratio for the fore-aft mode and 1.2% damping ratio of critical damping for the side-side mode. Lombardi et al. [11] performed a series of laboratory tests using a scaled model wind turbine supported on a monopile in kaolin clay in operating condition. Between 32,000 and 172,000 cycles of horizontal loading were applied to the turbine and the changes in natural frequency and damping of the model were monitored. The damping was evaluated in time domain using the logarithmic decrement method and showed that damping increased with the number of cycles and higher damping variations were recorded for larger strain amplitudes in the soil.

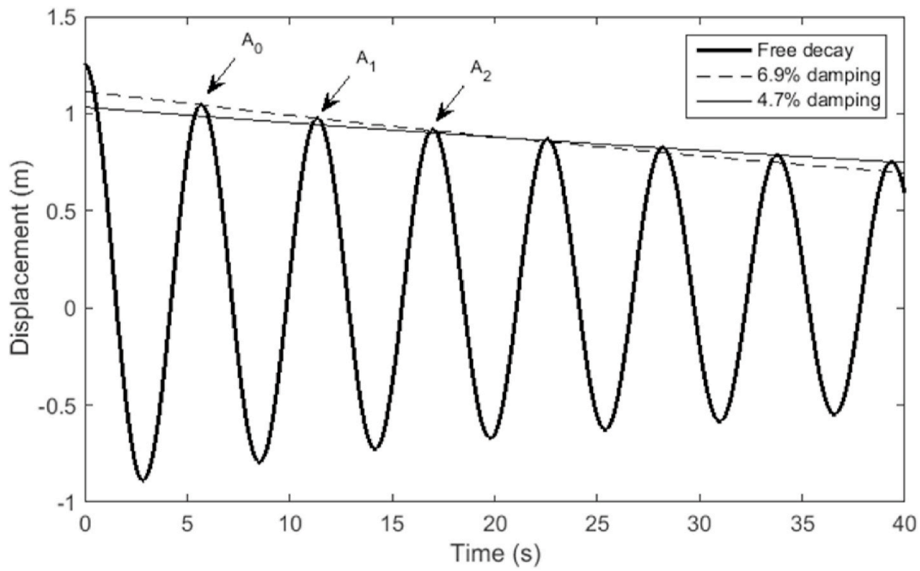

Fig. 13. A sample of free decay response.

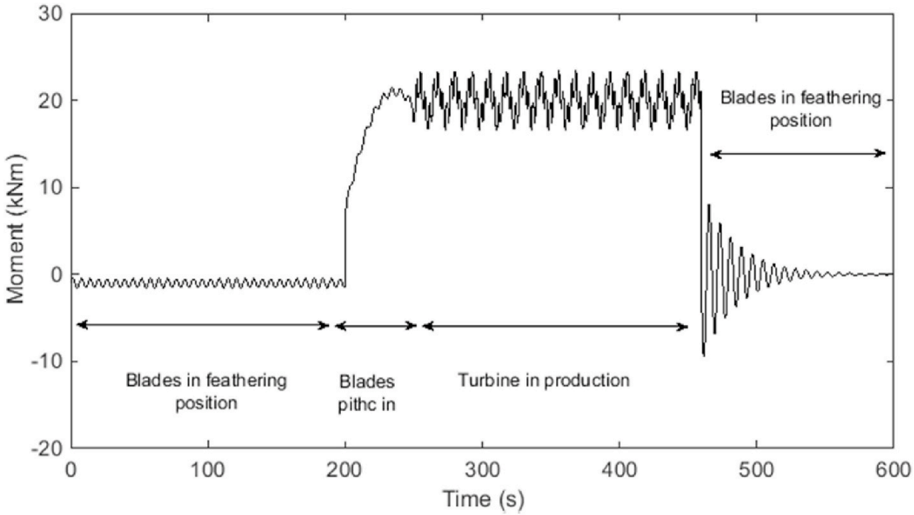

Fig. 14. The transition from feathering position to production and then again feathering position (adopted from Ref. [80]).

Carswell et al. [46] presented a method for finding an equivalent lumped parameter model to represent OWT foundation damping due to hysteretic energy loss. The lumped model represents the foundation damping as a viscous rotational dashpot at mudline level. A free vibration and stochastic time history analysis was developed using a two-dimensional finite element model. The stochastic time history analysis included extreme storm conditions of sea state and wind conditions. This analysis is important because it represents the actual loading of OWTs. The authors have assumed that the wind turbine is subjected to six different 1-h stochastic load histories corresponding to extreme wind and wave loading. The OWT foundation damping ratio was estimated as 0.17%–0.28% of critical damping to total damping using logarithmic decrement method. It was also shown that when OWT foundation damping was included in the model, the maximum mudline moment was reduced by 7–9% compared with the case where no foundation damping was considered. The shear force at mudline level also reduced by approximately 2%. Koukoura et al. [76] employed vibration measurements from a 3.6 MW pitch controlled variable speed offshore wind turbine on a monopile foundation for estimating the support structure damping both in parked condition and in normal operation. The damping was estimated using the EFDD method under ambient excitation. The damping in parked condition was identified through the parameters of an exponential curve fitted to the relative maxima of a decaying response and also half-power bandwidth method. A mean value of the total damping ratio of 1.93% was identified in the parked condition. In the operating condition, the damping ratio was estimated to be around 8.1% for the FA mode and

Table 2
Different types of the sensors used for damping estimation.

Reference	Sensor specifications	Measurement details
Damgaard et al. [41]	Acceleration transducers were used.	The fore-aft acceleration and the side-side acceleration were measured at a sampling frequency of 10 Hz.
Bajric et al. [55]	A triaxial accelerometer was mounted on the tower 7 m below the centre of gravity of the nacelle	The accelerations were recorded at a sampling frequency of 10 Hz.
Lombardi et al. [7]	Piezoelectric accelerometers were used at two locations (pile head and at the top of the tower).	The acceleration responses were recorded in two orthogonal directions along the tower.
Koukoura et al. [48]	44 strain gauges, 6 accelerometers and 2 inclinometers were used. The gauges are located at 11 different heights throughout the length of the sub structure with 4 gauges per height.	The sampling rate of the data acquisition system is 35 Hz.
Devriendt et al. [59,60] Shirzadeh et al. [27]	The accelerometers were installed at four levels. Two mounted at the lower three levels and four at the top level. The chosen configuration is primary aimed at identification of tower bending modes. The two extra sensors on the top level are placed to capture the tower torsion mode.	High sensitivity accelerometers which are able to measure very low frequent signals were used.
Versteijlen [15]	7 rings of strain gauges are attached to the inner pile wall, with 6 rings below, and 1 ring just above mudline. Each ring contains 4 strain gauges. 2 Althen AAA320 accelerometers were attached to the inner pile wall at the top of the pile for measuring the pile accelerations in North-South and East-West direction.	The pile was excited using a shaker consists of 2 large cogwheels that are hydraulically powered. The shaker can deliver a maximum hydraulic power of 50 kW, rotate at a maximum frequency of 8.6 Hz and was designed to not supersede an excitation force of 160 kN.
Beuckelaers [3]	Multiple displacement gauges and inclinometers at different elevation above ground level, multiple levels of fibre-optic strain gauges, extensometers and inclinometers below ground level. Advanced PID control system used for controlling load and displacement levels.	Multiple tests with pile diameters ranging from 0.76 m to 2 m. 1- and 2-way cyclic loading covering multiple loading amplitudes and pile displacement levels.

6.7% for the SS mode.

Devriendt et al. [86,87] implemented long-term monitoring of an OWT in Belgian North Sea. They use a state-of-the-art modal parameter estimator, the polyreference least squares complex frequency-domain estimator-commercially known as PolyMAX [88]. The total damping in the parked condition was estimated under various wind speed conditions. The damping ratio was shown to vary in a range of 1.1–2.1% when the wind speed was in a low range of 0–5 m/s, while it varies in a range of 1.5–2.6% for higher wind speeds of 10–15 m/s. As higher wind speed may create larger pile deflections (and hence larger soil strains), higher damping values were estimated. Shirzadeh et al. [34] estimated the frequency and damping ratio of the first for-aft mode of a wind turbine using real measurements and numerical simulations. They employ a time-domain output-only modal analysis method called PolyMAX for the modal identification. The measurements are performed at the Belwind wind farm, which is located in the North Sea on the Bligh Bank. The measurements were taken from overspeed stop condition and ambient excitation. The first FA frequency is found to be 0.3529 Hz and 0.3565 Hz from the overspeed stop test and ambient excitation respectively. The damping ratio for this mode was identified as 1.05% for both conditions. Versteijlen [21] employed a large-size hydraulic shaker to excite a full-scale, in-situ monopile foundation in stiff, sandy soil at a near-shore wind farm. The measurement setup included 7 rings of strain gauges each contained 4 strain gauges, that were attached to the inner pile wall, where 6 rings embedded below the mudline level, and 1 above. 2 accelerometers were also installed at the top of the pile for measuring the pile accelerations. The soil damping ratio contribution was estimated to be around 20% for the pile-only situation which equated to a 0.14% damping ratio for the full wind turbine including the tower and RNA. Table 2 summarizes different types of sensors used in the recent experimental case studies. It also provides some details about each measurement set up. An important point in choosing proper sensors is that the range of interested frequencies is normally in a very low range. However, the type of the sensors needs to be chosen based on the test environmental condition and required results.

Perhaps some of the most useful measurements of foundation damping on monopiles stems from the PISA project and PISA damping project field testing which involved 1-way and 2-way cyclic load testing on medium (0.76 m diameter) and large scale (2 m diameter) monopiles and was reported by Byrne et al. [89] and Beuckelaers [3]. The testing took place at two well characterised test sites: a stiff clay test site in Cowden, UK, and a dense marine sand site in Dunkirk, France, and could be regarded as some of the most rigorous and complex geotechnical testing ever undertaken. The damping ratios were calculated from the cyclic hysteresis loops and covered a large range of load (and pile displacement) levels and loading rates. The testing showed measured damping ratios in the range of 5–15% in stiff clay at Cowden and in the range of 3–13% in dense sand at Dunkirk, albeit at strain levels higher than typical operational strains around offshore monopiles. It is the authors opinion that the PISA testing offers the most realistic measurements of large strain foundation damping of all the aforementioned studies. The range of measured foundation damping values were generally significantly higher than would typically be used in design, highlighting the potential conservatism in current practice.

4.2. Numerical studies

Dilas [24] employed a geotechnical 3D Finite Element software (Plaxis 3D) with the Hardening Small Strain (HSS) constitutive soil model for modelling of soil structure interaction for an OWT and the soil model was verified against experimental field data. The operational loading in Ultimate Limit State (ULS) and Serviceability Limit State (SLS) conditions was applied before starting a free vibration analysis. The damping values were then estimated using logarithmic decrement method from the displacement free decay responses. The total and soil damping ratios under SLS loading are estimated as 0.62–0.70% and 0.43–0.51% respectively. The total and soil damping ratios under ULS and SLS were calculated as 1.53–1.64% and 1.34–1.45% respectively. It was concluded that soil damping value increased as the wave and wind loads increase. Malekjafarian et al. [90] used a similar approach to Dilas [31] where they employed Plaxis 3D software with the HSS soil model to model an OWT. They measure free decay response at RNA level. The damping were estimated using successive peaks in the response using the first 27 peaks as the starting point in logarithmic decrement. They have shown that as the time passes and the amplitude of the response decreases, the overall damping of the system decreases from about 7% to 2.5%. Aasen et al. [40] created a 3DFE model of soil-pile system using Plaxis 3D. Simalar to Malekjafarian et al. [90], the pile was modelled in half-space due to the symmetry of loading. The model was used to calibrate a non-linear 1D rotational model using the moment-rotation and horizontal load-displacement curves at the mudline level. The 1D model was ultimately used for investigating the effects of soil damping.

Kementzetzidis et al. [91] built a hydro-mechanical 3D finite element model of an OWT using OpenSees simulation platform. A unique feature of this model is including the low-frequency dynamics of the water-saturated soil. They have employed this model to study the influence of load amplitude and sand permeability to the foundation damping. They used simulated rotor-stop tests to find the free decay lateral acceleration responses. The damping values were obtained using the logarithmic decrement method. Ishihara et al. [92] created an OWT numerical model where the foundation is modelled using a Winkler model. The also used free decay analysis for the estimation of damping ratios. They used two different test cases including emergency stop and ambient excitation. The model was used to evaluate the effects of soil properties on the modal damping ratios. It is shown that the modal damping of the first and second modes strongly depend on the soil stiffness. Alamo et al. [93] presented a simplified sub-structuring model to study the change in the damping of OWTs sue to soil structure interaction effects. The authors used a single-degree-of-freedom system to represent the fixed-based super-structure and impedance functions to model the foundation stiffness. The authors used this model to perform a parametric study which evaluates the influence of soil profile, pile diameter and pile length to the foundation damping. Chen and Duffour [20] employed a decoupled finite element model of monopile-supported OWTs which was created using the modelling package FAST for the foundation damping estimation. They used a sonogram FFT technique on the free decay response of the OWT to identify the damping of the first mode. They used the model to evaluate the impact of the wind speed to the damping ratio.

Damgaard et al. [55] used complex stiffness hysteresis loops to approximate the soil material damping to an equivalent viscous damping. A static deformation analysis was performed with a load level based on the measured wind speed during the “rotor-stop” test. The energy dissipation in first cycle after the “rotor-stop” was approximated using the irreversible deformations in the soil. Carswell et al. [46] converted hysteretic energy loss into a viscous, rotational mudline dashpot to represent OWT foundation damping. The soil was assumed to have a hysteretic behaviour where the hysteric foundation-energy loss was computed with the INFIDEL computer model. This hysteric foundation energy loss is converted to a viscous damping constant in a lumped parameter model at the mudline level of the structural model which is created in a finite element modelling package, ADINA. Carswell et al. [94] derived mudline stiffness and damping matrices for the National Renewable Energy Laboratory (NREL) 5 MW reference turbine. The contribution of soil damping to OWT dynamic behaviour was quantified by recompiling the NREL code FAST with mudline stiffness and damping matrices. The dynamic responses of the monopile including and excluding soil damping were compared for this purpose. Aasen et al. [47] computed hysteretic damping using a non-linear elastic two-dimensional finite element model and implemented it in the foundation model using a viscous rotational damper at mudline. In this model, damping is a function of load amplitude due to hysteresis, which gives more damping at higher load levels.

Markou and Kaynia [95] presented a parametric study where different generalized nonlinear mechanical formulations were used with different damping characteristics for the soil modelling. A recently developed nonlinear mechanical models (previously used for the simulation of high-damping rubber isolators) was employed to describe the nonlinear hysteretic soil behaviour. Two new models were introduced and compared with well-known models within the industry. The authors concluded that the most commonly models used within the industry practice are inappropriate to describe the soil-pile interaction phenomena and more appropriate models should be used to account for soil-pile interaction phenomena.

Page et al. [96] examined the impact of foundation modelling by comparing simulations and full scale field data from a OWT in the North Sea. Three different foundation models were compared: (i) using elastic API p-y curves, (ii) elastic p-y curves calibrated from 3DFE and (iii) the 2D kinematic hardening elasto-plastic macro-element model calibrated from 3DFE discussed in section 3. The analysis showed that the API elastic p-y approach resulted in a significant under-estimation of the first eigenfrequency compared with the measured results and resulted in an over-estimation of the 1 Hz Damage Equivalent Load (DEL) by 166% on average, demonstrating the inadequacy of the API elastic p-y approach. Using 3DFE calibrated elastic p-y curves (Winkler approach) resulted in significantly improved predictions of the natural frequency but still resulted in an over-estimation of the DEL by 26% on average. The elasto-plastic macro element model offered the best prediction of the DEL but was still over-predicted by 15%. The primary difference between DEL from the 3DFE calibrated macro element and elastic p-y curves is attributed to the damping resulting from elasto-plastic behaviour in the macro-element model, highlighting the benefits of using non-linear elasto-plastic kinematic hardening models. Table 3 provides a summary of the software used in studies reviewed in this section.

Beuckelaers et al. [71] developed a Winkler p-y model utilising kinematic hardening elasto-plastic p-y curves using the API p-y curves as the backbone curve. The model was integrated into the HAWC2 commercial software for OWT rotor dynamics to simulate the behaviour of offshore wind turbines in the time domain. The behaviour of a wind turbine was simulated for a simplified rotor stop at different wind speeds using the fully integrated model and the damping was estimated using the logarithmic decrement approach. Comparisons between the elastic API p-y approach and elasto-plastic API p-y approach showed a 30–90% increase in overall damping, depending on the wind velocity.

4.3. Studies of the effect of foundation damping on fatigue analysis

The fatigue analysis for OWT is normally carried out by calculating the stress at critical locations (For example, the tower base [77] or the mudline [98]). Time domain methods are normally used for predicting the fatigue life of the structure. In these methods,

Table 3
The summary of the software used for estimating the hysteretic damping.

Reference	Software used
Damgaard et al. [41]	Matlab is used for implementing the Winkler approach.
Carswell et al. [39]	The primary model of the OWT structure and foundation used for free vibration and stochastic time history analyses was created in the finite element modelling package ADINA. The software package FAST was used for developing an aero-hydro-elastic model which generates the stochastic load time histories. The INFIDEL software was used to compute foundation stiffness and damping which define the LPM at the mudline of the ADINA model.
Carswell et al. [62]	The software package FAST was employed for performing an aeroelastic time history analysis. The INFIDEL software was used to determine a linearized mudline stiffness and damping.
Aasen et al. [40]	The software is called 3DFloat was used for modal analysis and time domain simulations. 3DFloat [97] is an aeroservo-hydro-elastic finite-element-method code, developed by the Institute for Energy Technology (IFE) and the Norwegian University of Life Sciences (NMBU). Plaxis 3D was used for finite element analysis of the soil–pile system to obtain moment–rotation and horizontal load displacement curves at the mudline.
Markou and Kaynia [63]	The software used was not clearly mentioned.
Beuckelaers [4]	HAWC2 was used to simulate the dynamics of OWTs. The dynamic calculations of the wind turbine using HAWC2 are controlled by a script written in Matlab.
Page et al. [70]	The macro-element model has been implemented in the OWT simulation software 3D Float and SIMA viadll (DynamicLinkLibrary) interface. The models built in Plaxis 3D were used for verification.

nonlinear and stochastic load effects due to the environmental loads and soil-structure interaction can be taken into account [78]. The impact of damping to the fatigue life of OWTs has been mostly considered in parked/non-operational conditions in the literature.

Fatigue life calculation for OWTs is normally done using the design. DNV [99] recommends using S–N curves given in Eq. (12) to assess the fatigue capacity the structures [78].

$$\log(N) = \log(\overline{a}) - m\,\log\left((\Delta\sigma(SCF))\left(\frac{t}{t_{ref}}\right)^{k}\right) \tag{12}$$

where N is the number of cycles to failure, $\log(\overline{a})$ refers to the intercept of $\log(N)$ axis, $\Delta\sigma$ is the stress range, m is the negative inverse slope of the curve and SCF is the stress concentration factor. t and t_{ref} are the thickness at which the crack is likely to grow and a reference thickness, respectively. The stress results then will be used to find the damage caused by every stress bin and then added together to obtain the total damage in the monopile for a given stress time-history. The damage in the pile can be calculated using Eq. (13) [78]:

$$D_j = \sum_{i=1}^{N_C} \frac{n_i}{N_i} \tag{13}$$

where n_i is the number cycles counted within a given stress bin, N_i is the number of cycles to fatigue failure for the nominal stress cycle amplitude i, and N_C is the total number of bins counted overthe 1 h time history. The total fatigue damage is obtained by summing each damage contribution, DC_j, according to Eq. (14), with 20 years the resultant fatigue life of the monopile [78]:

$$D = \sum_{i=1}^{N_j} DC_j \tag{14}$$

Aasen et al. [47] studied the impact of soil-foundation models on the fatigue damage of an OWT with a monopile foundation. Both the stiffness and damping properties were shown to have a noticeable effect on the fatigue damage, particularly for rotor-stop condition. The authors concluded that the accumulated fatigue damage at mudline level can be reduced by up to 22% depending on the foundation model used. Fontana et al. [77] employs a numerical model of an OWT to study the influence of overall structural damping and foundation damping to the fatigue life of the structure. The NREL open-source wind turbine simulation software FAST was used to demonstrate that increased damping will decrease both the resultant mudline moment and the accumulation of fatigue damage for all the cases. Increasing damping from 1% to 5% may cause up to 47% reduction in the fatigue damage accumulation in the operating cases and up to 69% in the parked. Like other studies, greater damage reduction could be expected in the parked condition due to the lack of aerodynamic damping. They also show that fatigue damage is more sensitive to damping ratio than the resultant moment because the relationship between stress and fatigue life is nonlinear.

Rezaei et al. [78] also investigated the effects of damping on the fatigue life of an offshore wind turbine. They stated that shutdowns of the wind turbine would increase fatigue damage up to 60% due to the significant reduction in aerodynamic damping. This means that long-time maintenance or shut-down periods could result in a reduction in the fatigue life. Their study also showed that the predicted fatigue life of offshore wind turbines linearly increases with the level of damping e.g. from 16 years at 4% overall damping to 53 years at 11% damping (Fig. 15). Damgaard et al. [81] evaluated how a change of the soil properties could affect the fatigue loads in OWTs in parked conditions. They showed that changes of the soil stiffness and damping may significantly change the fatigue damage equivalent moment at mudline. Their conclusion is that a 50% reduction in soil damping may imply 21% increase in the fatigue

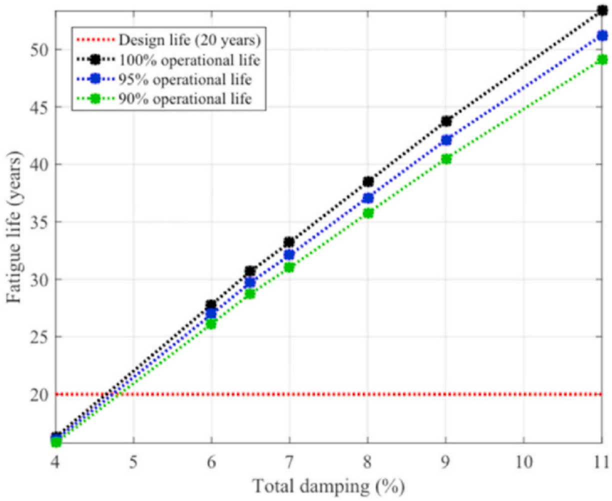

Fig. 15. Fatigue life comparison of operational life and different levels of damping (After [78]).

damage equivalent moment at mudline level. Schafhirt et al. [100] investigated the impact of changes in soil parameters on the fatigue lifetime for an OWT founded in loose sand. Nonlinear elastic API p-y curves were used to model the soil-pile interaction and the fatigue lifetime was shown to vary between −9% and +4% when considering changes in soil conditions including soil softening or stiffening, respectively.

4.4. Summary of damping studies

Table 4 summarizes the damping ratios estimated from different studies in recent years. It can be seen that the foundation damping varies in a range from 0.17% to 1.5% in different studies. These studies consider different soil profiles of clay, sand or combinations of both. They also employ different monopile sizes with different slenderness (l/D) ratios. In addition, various environmental conditions were used in these studies. In particular, most of them avoid high speed winds which means the applied loads are in a low range. This keeps the soil behaviour in the linear part of p-y curves which contains low level of energy dissipation. It can be concluded here that the foundation damping is function of many parameters. Therefore, these parameters need to be defined as inputs of any methodology for identifying of OWTs foundation damping.

5. Remaining challenges and conclusion

The foundation damping of OWTs is a relatively new field of study in renewable energy. Despite of numerous studies which have been completed in recent years, there are still several challenges which need to be addressed in order to accurately capture an OWTs damping response. The main remaining challenges are discussed in this section:

5.1. Soil structure interaction modelling

There have been several attempts for dynamic analysis of soil structure interaction. Most of these studies are limited by several assumptions and are mostly carried out for particular cases. The main challenges of soil structure interaction modelling can be summarized as:

1. *Soil modelling:* Complex constitutive soil models and rigorous calibration procedures are required to accurately capture the soil-structure interaction for an offshore monopile. The influence of several aspects of soil behaviour including cyclic loading, rate effects, ratcheting and gapping effects require further study [3,45,101]. In addition to the intrinsic material behaviour of a particular soil type, more research needs to be carried out on the effects of soil layering on OWT foundation damping. The overall soil damping is the sum of energy dissipated at each soil layer. Therefore, future works should define the contribution of each soil layer to the overall foundation damping. This contribution could be function of soil properties and the pile horizontal displacement at each layer.
2. *Drainage conditions in soil model:* Most monopile damping studies do not explicitly consider the effect of pore water in the soil model. As offshore monopiles OWTs may behave in a drained, undrained or in a partially drained manner, there is a need for the application of advanced soil models that can capture the appropriate dynamic pore pressure regime. This will provide further insight to the foundation damping problem where the development of pore water pressures could play a significant role.
3. *Load Modelling:* In addition, the influence of the main load parameters e.g. load amplitude (ULS, SLS etc.), load frequency and combined static cyclic loading on foundation damping and stiffness need to be investigated [101]– see Fig. 16. Larger loads normally create larger deflections below mudline level, which may increase the foundation damping.
4. *Dominant mode of vibration:* So far, most of the studies assume that the fore-after first vibration mode is the dominant mode in all cases (see Fig. 17). It is also a common practice to use only unidirectional wind and wave loads. But in practice, the loads may be applied to OWTs from different directions which may make the other modes dominant in the dynamic response. Therefore, there is need for a methodology to define the dominant frequency in different design load scenarios. In addition, it is important to investigate how the direction of wave and wind loads may influence the foundation damping and ultimately fatigue life of OWTs [78].
5. *Seabed changes during time:* As suggested by Beuckelaers [3], an important geotechnical challenge for foundation damping is the influence of seabed changes throughout the lifetime of OWTs on damping of the system. This could include scour and gapping that can cause loss of contact between foundation and soil [103].
6. *Modelling challenges:* Numerical modelling of soil structure interaction using software like Plaxis 3D is normally computationally expensive, particularly when dynamic analysis is required (see Fig. 18). This would limit the efficiency of parametric studies with many simulations needed. Potentially the most useful developments for estimating the design values for damping would be the widespread adoption of kinematic hardening elasto-plastic models such as the macro-element approach proposed by Page et al. [69] or Houlsby et al. [74], or the Winkler (p-y type) approach proposed by Beuckelaers [3]. Care should be taken as to how these models are verified (e.g. 3DFE) and validated (e.g. field testing) and the use of appropriate backbone monotonic loading curves for these methods is crucial to accurately capture the foundation damping.

5.2. Experimental challenges

Many researchers have carried out experimental investigations on foundation damping of OWTs. The main conclusions that there is

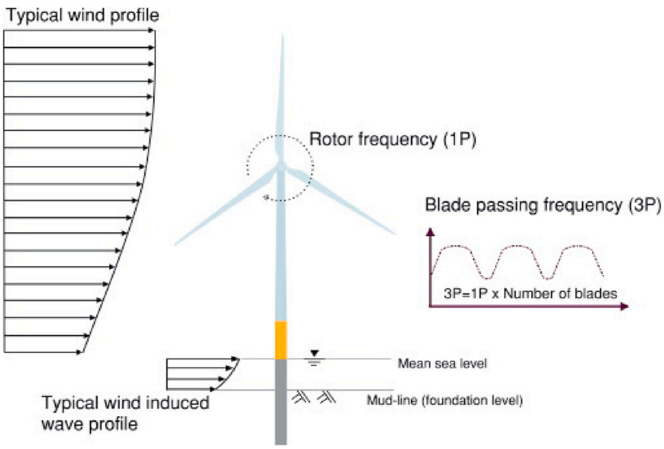

Fig. 16. External loads acting on monopile-supported OWTs (After [11]).

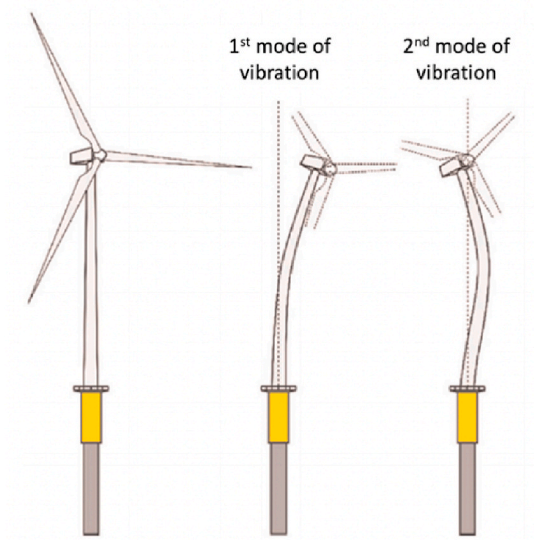

Fig. 17. The first two modes of vibrations for monopile-supported OWTs (After [102]).

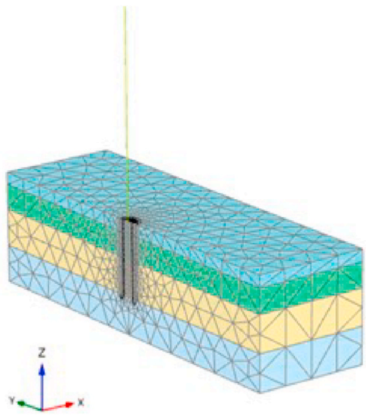

Fig. 18. 3D FE model of OWT in Plaxis 3D (After [90]).

Table 4
Damping values estimated in different studies adapted from Refs. [31,46].

Study	Method	Soil	MP l/D	Total damping ratio [%]	Soil damping ratio [%]
Tarp-Johansen et al. (2009) [26]	Experimental	Sand or Clay/North Sea	4.26	0.75–0.99	0.56–0.80
Versteijlen et al. (2011) [80]	Experimental	Sand/Clay	4.68	3.00	1.5
Damgaard et al. (2012) [36]	Experimental	Sand/Clay	5.38	0.77	0.58
Damgaard et al. (2013) [55]	Experimental	Sand/Clay	–	0.8–1.3	0.8–1.3
Beuckelaers (2017) [3]	Experimental	Clay	5.25	–	5–15
Beuckelaers (2017) [3]	Experimental	Sand	5.25	–	3–13
Shirzadeh et al. (2013) [34]	Numerical	Sand/Clay	4.12	0.85	0.25
Carswell et al. (2015) [46]	Numerical	Clay	5.67	1.17–1.28	0.17–0.28
Dilas (2018) [31]	Numerical	Sand	5	0.62–1.53	0.43–1.45

still lack of experimental works for the accurate identification of dynamic behaviour of OWTs. The main remaining challenges for experimental works are listed and discussed here:

1. *Investigation of large diameter monopiles:* An important experimental challenge is the need for more experimental data from large diameter monopiles embedded in a range of different soil profiles. The recently completed joint industry PISA project (and associated damping project) [6] has gone some way to address the dearth of high quality field test data from monopiles. However, further large scale testing is required to further the knowledge of foundation damping in layered or partially drained soils.
2. *Different test conditions:* A more comprehensive experimental database is needed from full-scale OWTs during different working conditions such as: operational (production), parked, storm and shut-down. The future test set ups should include more information from the experimental measurements (e.g accelerometers placed on the hub level) [31].
3. *Presence of harmonics:* Presence of harmonics in experimental results is a recent challenge in the OWTs experiments. It is important to well investigate the harmonics and eliminate them before the identification process.
4. *The contribution of other damping types:* There is a need for a detailed study on the influence of different hydrodynamic and wind parameters including the different wave angle, wave period, tidal level (wave height) and wind speeds to the foundation damping [20,34]. The contribution of aerodynamic damping at different wind speeds for both operational and parked wind turbines should be experimentally investigated. In addition, a comparison between numerical results and measurements will be helpful for determining of the aeroelastic behaviour of OWTs at different operational and ambient conditions [34]. This is especially important for cross-wind vibrations, where the aerodynamic damping is negligible [78]. Cases with wind-wave misalignment will be examined to emphasize the significance of damping in the fatigue of the structure, when aerodynamic damping is not present [76].
5. *Laboratory-scale experiments:* Full-scale experiments are usually expensive and time consuming. Small-scale experimental test in a laboratory can be used to provide insight into particular mechanism or behaviours and the conditions are easier to control in a laboratory environment. The development of small-scale lab models are also useful for the calibration and verification of numerical models [31].
6. *Model updating:* A common practice in numerical modelling is model updating in which the numerical model is updated by measured data of an offshore wind turbine [55]. This is particularly important for the cyclic pile–soil interaction behaviour in order to ensure a correct behaviour of dynamically loaded offshore turbines [55].

5.3. Developing of integrated structural and geotechnical models

One of the main remaining challenges for accurately determining foundation damping of OWTs is the lack of a widely accepted or industry standard integrated structural geotechnical methodology for the accurate determination of foundation damping. Such a methodology is necessary for more efficient and accurate designs for OWTs and have started to be addressed by a number of researchers [3,72,104]. To be easily applicable for different phases of engineering design, both simple and complex approaches for estimating soil damping are required. For simple models, used in early stage design, the foundation soil damping effects need to be defined for various turbine geometries, seabed conditions, and loading effects. In addition, the development of a database of foundation damping for a wide range of soil conditions and monopile geometries be very useful to offshore developers and designers.

Declaration of competing interest

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

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