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A review of offshore wind monopiles structural design achievements and challenges

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ABSTRACT

The taller and heavier OWT monopile structures has unique structural engineering design challenges which could lead to catastrophic under-conservative designs or more likely, expensive over-conservative engineering. The need for an efficient engineering design cannot be overemphasised given the current structural configurations and future OWT monopiles novel concepts due to the cost of materials for the increasing towers and foundations diameter and thickness, longer and bigger RNA, which directly impacts the manufacturing, transportation, and installation activities. This increasing trend is projected to continue as the industry utilises shallow to medium water depths for siting new concepts of Offshore Wind Turbine (OWT) monopile structures and development of wind farms. The engineering design through to installation challenges faced by the relatively new OWT industry are exacerbated by the shortcomings of the structural design techniques and practices adopted from the oil and gas industry and in accordance with design codes and standards. This paper presents a concise structural review of the current salient technical aspects, the recent improvements in offshore wind turbine monopile structural design, and the challenges of future OWT monopile concepts considering the increasing monopile structure size and turbine capacity. The review presented in this paper primarily focused on grouted connection between the foundation and tower, damping for monopile structural response analysis, soil scouring, soil-monopile interaction modelling, and corrosion. The aim of this paper is to critically assess, outline, and discuss the current OWT monopile structural design techniques achievements and identify future concepts structural challenges, and to provide structural design direction for OWT monopile research and development activities.

1. Introduction

The guidance on offshore wind turbine support structure is currently being provided in DNVGL-ST-0126: Support structures for wind turbines, (DNVGL. DNVGL-ST-0126) and BS EN 61400–3: Design requirements for offshore wind turbines, (61400 -3 E. 61400-61403, 2009), amongst others. The design achievements and challenges presented in this paper are based primarily on DNVGL guidance, but references are made to other industry recommended codes and standards, as necessary. Due to the less mature technology readiness of the range of offshore wind turbines and the gaps in knowledge and understanding of the structural modelling and dynamic behaviour, the recommended guidance by DNVGL for the design of offshore wind turbine structure is continually updated. These updates are essential in keeping up with the improvements in technology and understanding of the structural modelling, interpretation, and response of the system.

Since the first release of DNVGL guidance on offshore wind turbine structures in 2004 (Veritas, 2004), then DNV, updated revisions were released in 2007 (Veritas, 2007), 2009 (Veritas, 2009), 2010 (Veritas, 2010), 2011 (Veritas, 2011), 2013 (Veritas, 2013), 2014 (Veritas, 2014), 2016 (DNVGL. DNVGL-ST-0126, 2016), and in most recently in 2018. After the merger of Det Norske Veritas (DNV) and Germanischer Lloyd (GL) in 2013, all standards are in the process of harmonization and alignment. The journey of support structures for wind turbines is best presented in Fig. 1.

The offshore wind turbine structures guidance is underpinned with the experiences from the oil and gas industry. However, the design, manufacturing, transportation, installation of offshore wind farms brings new challenges due to the size of the structures and foundation system and the large number of structures per project to install, operate and maintain throughout the intended design life.

The design of offshore wind turbine support structure by DNV provides principles, technical requirements and guidance for design, construction, and in-services inspection of offshore wind turbine structures.

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However, the standard does not cover design of wind turbine components such as nacelle, rotor, generator, gear box, support structures and foundations for transformer stations.

This paper presents and critically reviews the salient updates and design achievements and challenges towards understanding previous and current challenges in the design and analysis of offshore wind turbine structures. This article is structured into the following sections: Section 2 presents and discussed the structural design achievements and challenges, including transition piece connection in Section 2.1; the influence of structural-soil-hydrodynamic-aerodynamic damping is covered in Section 2.2. Section 2.3 discusses soil-structure modelling approach, and the influence of scouring on structural response is presented in Section 2.4. Air-tight corrosion design and impact on fatigue, along with structural considerations for future OWT monopile concepts are discussed in Sections 2.5 through to 2.7. Finally, conclusions and recommendations are presented in Section 3.

2. Discussion on structural design achievements and challenges

Selected salient updates and design achievements and challenges of offshore wind turbine structures and foundation system are discussed in this section following critical holistic review of the design codes and standards, research, and industry contributions.

2.1. *Transition piece: grouted with or without shear keys connection*

The offshore wind turbine tower is connected to the foundation system through the transition piece. The introduction of shear keys in the grouted connection is aimed at improving capacity, but this has the disadvantage of a corresponding poor fatigue strength. The shear keys introduce fatigue hotspots and currently, designers conservatively assume poor fatigue details and design S–N curve data for engineering the joint. Hence, there is the need to refine the design and analysis of grouted joints with or without shear keys. Placing the shear keys within the centre of the connection improves the impact on the fatigue capacity of the joint. Through testing, the presence of shear keys is demonstrated to increase the stiffness of the connection and reduced local sliding distance and gaps by a factor of 2. Although it understood that the shear keys can lead to hotspot and exacerbate fatigue damage, tests shows that plain steel surface grouted joints leads to reduced fatigue and ultimate performance compared to connections with shear keys (Schaumann et al., 2014).

The influence of steel surfaces and shear keys on the fatigue performance of grouted connections was investigated by (Schaumann et al., 2013). The research was conducted for plain grouted joints and for grouted joints with shear keys. The research concluded that local stress concentrations are distributed more evenly with an increase in the number of shear keys and that the local stress plastifications occur at the outer shear keys. It was further concluded that for fatigue design, the shear keys lead to local stress concentrations in the grout layers of the connection. However, the failure modes can be reduced if shear keys beads are arranged at the centre of the joint. The introduction of shear keys is favourable regarding durability of grouted connections.

There is lack of details on guidance for state-of-the-art fatigue assessment of offshore wind turbine structures grouted joint connections that accurately captures the occurrence of non-linear effects. The non-linear effects of ovalisation and S-shaped buckle mode as the grout punches into the slender steel shell must be correctly captured as stress riser. Where analytical method leads to a non-favourable design solution due to simplifications and approximations, it is then recommended to complete numerical analysis by means of 3D finite element modelling and analysis (–0126: Sup). However, several parameters required for assessment of capacity of grouted connections using finite element analysis are encumbered with uncertainty such as element types, element mesh in the region of the highest stresses, friction coefficient, characteristics of the grout materials, material modelling, contact formulation, and convergences criterion. Therefore, grouted connection design and analysis by finite element require calibration.

The research project "Grouted Joints for Offshore Wind Turbine Structures" (GROW) and a follow-up research project "GROWup" investigated improving the strength of hybrid connections by applying shear keys and to address reported sliding damage which occurred at several offshore wind farms in 2009 having plain cylindrical grouted joints. An important task within the GROW project was to develop detailed finite element models (FEM), calibrated against the large-scale tests. The primary objective of the calibrated detailed FEM is to replace expensive experimental verifications and for carrying out parametric studies and analysis to improve on the designs of grouted connections. It was concluded that the calibrated detailed finite element model gave a good understanding of how the loads in the grout are transferred mainly between the shear keys and a good tool for detailed design verifications. The research made recommendations for further work aimed at providing guidance on geometric boundary conditions or simplified analytical verification concepts where tests data may not be available for calibration. Details of the experiments, calibration process, and the refined finite element model for the design and analysis of grouted connections are presented in (Klose et al., 2012). This is in line with recommendation that is appropriate to perform grouted connection design and analysis using finite element model; however, such finite element analysis must be calibrated and bench-marked with reliable experimental test data or well-known cases where such data exist (–0126: Sup).

The design of grouted joint primarily accounts for loading due to bending, shear, axial, and torque which usually results in complex combination and response. Bending moment is the predominant of all the loading modes, hence, there is the assumption that the grouted connection capacity is improved due to increased frictional resistance generated by the induced bending moment. This assumption resulted in the simplified modelling approach of separating the axial load and torque from the bending and shear. Earlier version of DNV–OS–J101

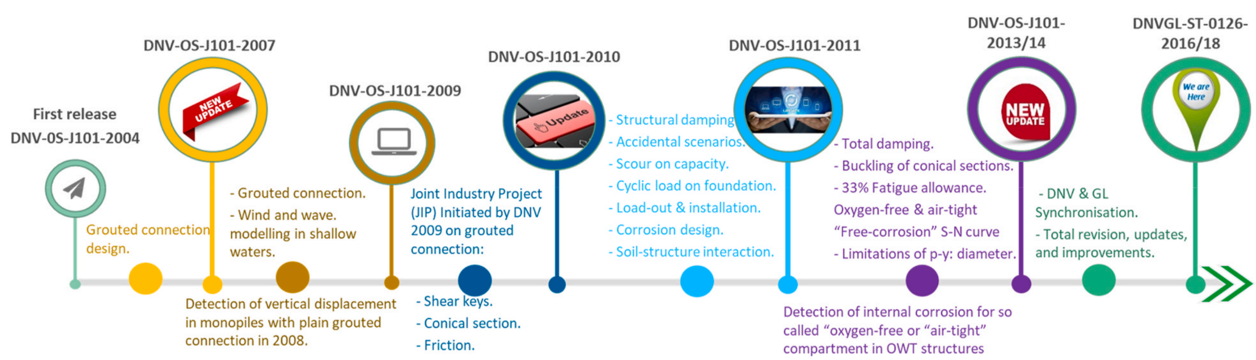

Fig. 1. The journey of support structures for wind turbines to DNVGL

recommended to demonstrate that the axial loads and bending moment do not interact, then the design conditions of two separate loadings can be justified:

1. Axial load and torque without bending and shear.
2. Bending and shear without axial load and torque.

Distribution of the contact pressure between the grout and steel is presented in Fig. 2. There is an increased contact pressure at the near-face top and far-face bottom in direction of the bending moment. The bending moment leads to vertical rotation of the pile and the sleeve, giving rise to two opposing areas of contact pressure at the top and bottom of the connection, couple. Load transfer between the transition piece and the monopole is made possible by the resulting force couple (Alwan and Boswell, 2014) and (Schaumann et al., 2013).

2.1.1. For tubular and conical grouted connection without shear keys

The maximum nominal contact pressure, $P_{norm,M}$ at the top and at the bottom of the grouted connection, caused by an applied bending moment M, may be calculated from the following expression:

$$\boldsymbol{P}_{norm,M} = \frac{3\pi\boldsymbol{M}}{\boldsymbol{R}_p\,\boldsymbol{L}_g^2(\boldsymbol{\pi}+3\boldsymbol{\mu}) + 3\boldsymbol{\pi\mu R}_p^2\,\boldsymbol{L}_g} \tag{1}$$

where:

μ is the friction coefficient.

$L_g = L - 2.t_g$ is the effective length of the grouted section.

L is the full length of the grout thickness.

t_g is the grout thickness.

R_P is the outer radius of the innermost tube for tubular connections and the average of the outer radius of the innermost for conical connections over the effective area.

Equation (1) assumes that the dependency on a horizontal shear force on the grouted connection is insignificant. This is assumption is valid for grouted connection for monopoles.Wherever the pressure from the shear force is significant, then the effects of this shear force on the pressure, $\mathrm{P}_{\mathrm{norm,Q}}$ may be calculated from following expression:

$$\boldsymbol{P}_{norm,Q} = \frac{\boldsymbol{Q}}{2\boldsymbol{R}_p\ \times\ \boldsymbol{L}_g} \tag{2}$$

The maximum nominal contact pressure due to bending moment M and shear force Q becomes:

$$\boldsymbol{P}_{norm} = \boldsymbol{P}_{norm,\,M} + \boldsymbol{P}_{norm,\,Q} \tag{3}$$

The design tensile stress in the grout can be calculated using the following expression:

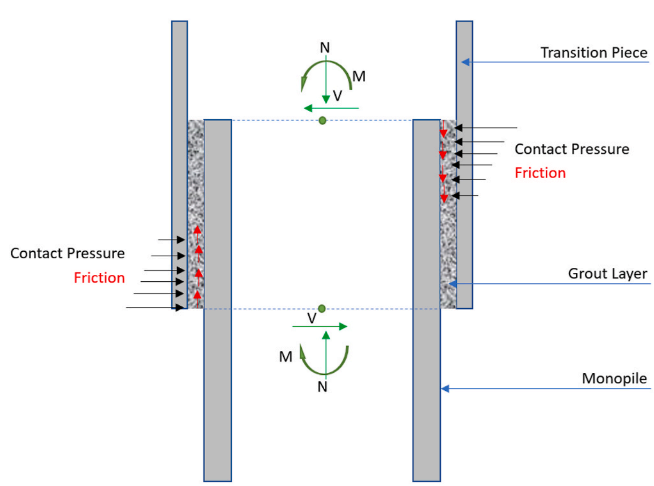

Fig. 2. Grouted joint contact pressure distribution.

$$\boldsymbol{\sigma}_d = 0.25\ \times \boldsymbol{P}_{local,d}\left(\sqrt{1+4\boldsymbol{\mu}_{local}^2}-1\right) \tag{4}$$

where:

μ_{local} is the local friction coefficient representative at the top and bottom of the grouted connection.

$P_{local,d}$ is the design value of the local contact pressure, P_{local}.

2.1.2. For tubular grouted connection with shear keys

The maximum nominal radial contact pressure, $P_{norm,d}$ at the top and at the bottom of the grouted connection, caused by an applied bending moment M_d may be calculated from the following expression:

$$P_{norm,d} = \frac{3\pi M_d E L_g}{\begin{bmatrix} EL_g \times \left\{R_p L_g^2(\pi+3\mu) + 3\pi\mu R_g^2 L_g\right\} \\ \\ +18\pi^2 k_{eff} R_p^3 \left\{\frac{R_p^2}{t_p} + \frac{R_{TP}^2}{t_{TP}}\right\} \end{bmatrix}} \tag{5}$$

where:

k_{eff} is the effective spring stiffness for the shear keys

μ characteristics friction coefficient, equal to 0.7.

R_P outer radius of the pile.

R_{TP} outer radius of transition piece

t_P wall thickness of transition piece.

L_g = L - 2.t_g = effective length of grouted section.

L full length of grouted section from the grout packers to the top of the pile.

T_g nominal grout thickness.

Following observation and detection of vertical settlements in monopiles with plain grouted connections in 2008, a Joint Industry Project (JIP) was initiated by DNV in 2009. It was realised that the industry practice used for the design of large diameter connections did not correctly represent the in-service behaviour and response of the physical structure. The JIP tests revealed salient design parameters and considerations that influences the long-term behaviour of large diameter grouted connections (Alwan and Boswell, 2014) and (Klose et al., 2012):

1. Surface irregularities and fabrication tolerances: using the correct design data can lead to increase in capacity generated by friction between irregular surface/interface of the grout and steel.
2. Slenderness ratio and connection flexibility affects the stiffness and ovality and buckling behaviour of the grouted connection. The flexibility and ovality is increased when the structures are subjected to horizontal wind and wave action inducing bending moment.
3. Accumulated sliding length due to cyclic loading leads to reduced frictional and joint capacity.
4. Friction coefficient at the steel-grout interface: application of the appropriate friction coefficient. Higher friction can lead to under-conservative/incorrect resistance capacity, while lower friction leads to over-conservatism and expensive design.
5. Abrasive wear at the steel-grout-interface due to combination of moment from sliding, bending, reduced friction and slenderness ratio, leading to ultimate reduction in surface roughness and loss of friction and capacity.

Conical grouted joint or straight grouted joint with shear keys is favourable and identified for improved performance and capacity to resist bending, shear, axial and torsional loads. The design of grouted connection requires improvements and appropriate design guideline, outlining the acceptable design approximation for modelling the grout geometry, friction, ovality, and the contact behaviour upon bending and axial loads and response under dynamic loads.

2.2. Appropriate structural-soil-hydrodynamic-aerodynamic damping

Total structural damping due to simultaneous occurrence of different loads and structural behaviour does not always follow a linear combination of the separately determined individual loads and damping coefficient. The total damping is influenced by the character of the individual loads and the combined effects and total structural damping may be established from structural analytical investigations and sensitivity checks. The total damping depends on the wind loading and its direction relative to other loads, such that for example, the wave load effect becomes dependent on the characteristics of the wind loading. The aerodynamic damping depends on whether there is wind or not, and if the turbine is in power production or at stand-still, including if the wind is aligned or misaligned with other loads such as wave loads on the structure. This is required as an input for calculating the total damping of the OWT structure.

Currently, assumptions are made regarding the stiffness and damping of both soil and structural members. Although, these assumptions may be tested through sensitivity analysis and parametric studies, this allows for subjective applications which may lead to over-conservative but expensive design or under-conservative unsafe design. Appropriate individual damping ratio and total damping are crucial for dynamic analysis and in avoiding resonance, the estimation and control of the natural frequency for the overall structure away from the excitation frequencies (–0126: Sup).

Studies show that due to the complexity of modelling soil structure and interpreting the behaviour upon loading, verification of the soil stiffness and determination of soil damping ratio for offshore wind turbine structures is by full scale testing (Damgaard and Andersen, 2012). Soil damping is the highest contributor to the total damping after tower oscillation dampers. Damgaard et al. (Damgaard and Andersen, 2012) conducted "rotor stop" test to determine the soil damping. Cyclic motion was observed to take place during the "rotor stop" test which results to material damping and geometric damping. The material damping is also known as internal damping which is the dissipation of energy within the soil mass due to friction, sliding between particles and rearrangement. Geometric damping is also known as radiation (external) damping of waves into the subsoil and can be ignored for frequencies below 1 Hz. From the "rotor stop" test, the irreversible deformations in the soil were established as a measure of the energy dissipation in the first cycle after the "rotor stop" takes place. The tower oscillation damper was determined from full scale "rotor stop" test as 1.36%, the steel material damping and aerodynamic damping according to (E–. EN, 1991-1-4, 2005) were estimated as 0.19% and 0.062%, respectively. The hydrodynamic damping was assumed to be 0.12%, hence, the soil damping was calculated to be 0.58%, deduced from the system total damping of 2.31% following the tests.

Study by Malekjafarian et al. (2021) presented several field tests and experimental research where signals were measured using accelerometer, and strain gauges measured the structure motions and vibrations for determining the OWTs foundation damping. The damping ratio were determined using the well-known logarithmic decrement method for identifying the damping ratio from free decay response. The calculated natural bending frequency and soil damping ratio depends on the measured and calculated soil strength. Once the appropriate soil stiffness and damping ratio are determined, these can be used in the soil-foundation structure interaction model for local and global design and analysis.

Aerodynamic damping significantly effects in the fatigue life of offshore wind turbine structures. According to Rezaei et al., 2018), normal or unforeseen shutdowns of the wind turbines is likely to induce fatigue damage of up to 60%, this is primarily driven by the significant reduction in aerodynamic damping influences on the structural responses than the corresponding reduction in operational dynamic loads. Proper calculation of damping ratio and appropriate application can lead to significant improvements in the structural fatigue life. The fatigue life is reported to increase almost linearly with applied damping (Rezaei et al., 2018).

The total structural damping ratio is also influenced the presence of marine growth on the foundation and turbine structure. The impact of marine growth is greater for the hydrodynamic damping ratio and overall structural response. The thickness and imposed weight of the marine growth as damper are necessary for estimating the influence on the natural response of the structure. The uncertainties in estimating the damping contribution from tower oscillation damper, structural damping, soil, aerodynamic, and hydrodynamic effects is highlighted by several researchers. Hence, the relevance of further research including investigation into stand-still, faulty, and shut-down conditions. The damping coefficient is an important dynamic parameter for modelling and conducting representative dynamic analysis of offshore wind turbine structures (Damgaard and Andersen, 2012).

2.3. Soil-structure stiffness and modelling approach

The soil-structure modelling technique, analysis and interpretation is crucial in the overall offshore wind turbine structure design. The natural frequency as well as the fatigue loading, and response are significantly affected by the soil-structure interaction understanding and modelling technique.

The famous p-y curves in accordance with API-RP 2014 and as described in DNV-ST-0126 is limited to smaller pile diameters, hence it is recommended to validate the use of p-y curves generated soil springs for more than 1.0 m diameter piles by means of finite element analysis or other suitable means (Hu et al., 2016). The p-y curves for sand were developed by O'Neill and Murchinson (1983), while Dunnavant and O'Neil proposed the p-y curves for clay (2000) which were adopted by the API-RP (2000) and still serve as basis for many offshore wind turbine designs. The p-y curve model is used to represent the soil resistance to the displacement by the non-linear the transfer curve, and the t-z curves are used to model the axial loading to structure displacement (Hu et al., 2016). The soil resistance to structure deflection is constructed using stress-strain data interpreted from soil samples, Fig. 3.

The monopile support structures, including diameters exceeding 7.5 m are designed according to the soft-stiff approach. Based on several research and industry applications, the p-y curve method is considered unsuitable for performance reasons due to the weak non-linear behaviour under operational loading. The linear implications and the stiffness of the p-y methods are questioned by several publications and considering the frequency of processed data from in-service monitoring. The p-y curve generated soil springs demonstrates an overall underestimation of the soil-structure stiffness (Dubois et al., 2016). Although, the research into the soil-structure interaction continues to be one of the

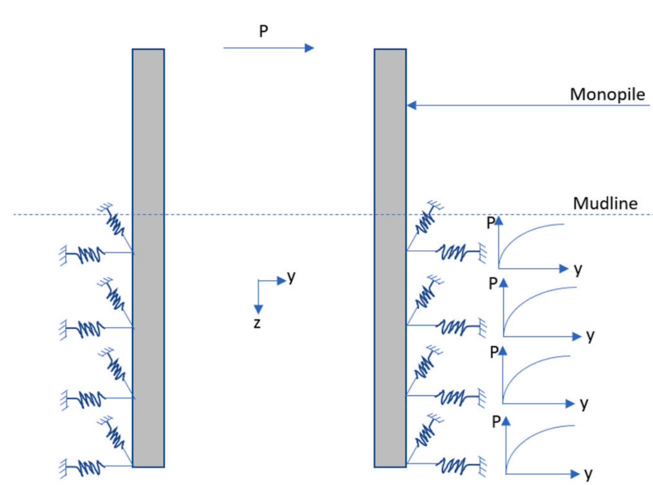

Fig. 3. Schematic of P–Y curve method.

focus of research, there is yet to be an updated and acceptable recommended modelling technique. Finite element modelling method is another technique that is adopted for representing and analysing the soil-structure interaction.

Other soil modelling techniques used for offshore structures foundation are the Matlock (1970) and Jeanjean (2009) p-y models. The API-RP p-y curves are originally generated from the Matlock model, although research shows that the stiffness of API-RP p-y curves to be significantly lower than that of Matlock p-y curves for very small displacements. The Jeanjean p-y model is suitable for assessing the fatigue life of offshore well conductors and applied in designing offshore wind turbines for serviceability limit state (Senanayake et al., 2017). The Jeanjean curves are stiffer than the API-RP p-y curves at all lateral displacements and stiffer than the Matlock curves at all but very small displacements, Fig. 4. Several researchers recommend improvements to the Matlock p-y curves; however, these modifications are known to only work well for the cases studied and not hold on wider applications. Therefore, the modifications to the Matlock p-y formulation is yet to be implemented, awaiting a comprehensive review to develop an alternative design method for monopiles that is robust, and provides efficient and effective designs for different soil conditions (Haiderali and Madabhushi, 2013). Refined design models and predictions using FEA techniques and measured data to establish the most appropriate soil-structure models are acceptable practice.

Recent work completed through a major European joint-industry academic research project, known as the PISA project, designed to develop soil modelling approach for laterally loaded offshore wind turbine monopoles. The PISA project focused on large diameter, relatively rigid piles, with low length to diameter (L/D) ratios. The PISA project introduces new procedures for site specific calibration of soil reaction curves that can be applied within a one-dimensional (1D), Winkler-type computational model. The 1D model incorporates the standard *p-y* lateral soil reaction, denoted as *p-v* in the PISA design model, but extended to allow for a distribution of moment along the pile length, as well as a horizontal and a moment soil reaction at the pile base. The 1D model is verified against data from 3D FE analysis of layered soil profiles, calibrated using inputs from field tests. The PISA project identified that, for piles under lateral loading with a low L/D ratio (buried pile length/diameter), the failure mode is more complex than assumed with the traditional *p-y* method (Byrne et al., 2019).

2.4. *Scour and cyclic loading on capacity of foundation and influence on structural response*

The effects of scour and cyclic loading on soil properties shall be considered in foundation design for offshore wind turbine structures. Effects of wave- and wind-induced forces on soil properties for a single storm shall be investigated, for normal operating conditions followed by a storm or an emergency shutdown. Geotechnical design of foundation is completed for both strength and the deformations of the foundation structure and of the soil in ULS and SLS. Cyclic loading may reduce the ultimate bearing capacity of the soil in the ultimate limit state (ULS), hence, the effects of cyclic loading on the ground strength and stiffness shall be addressed for ULS and SLS design conditions for the different loading situations.

Considering steady current, the scour process is mainly caused by the presence of horseshoe vortex combined with the effect of contraction streamlines at the edges of the pile. Measured data across different offshore wind farms indicates significant variation in scour hole shape which tends to be elongated with steep upstream slope and gentle downstream slope. For cases of waves, the horseshoe vortex and lee-wake vortex form the processes that govern scour, dictated primarily by the Keulegan-Carpenter number, KC as follows:

$$KC = \frac{u_m.T_P}{D} \tag{6}$$

where T_P is the peak wave period, D is the cylinder diameter and $U_m = 1.41u_{ms}$. u_{ms} is the standard deviation of the velocity at the seabed.

Long-term cyclic lateral loading induced by waves and wind can lead to change in soil stiffness during the lifetime of the offshore wind turbine structure and foundation system (Nicolai and Ibsen, 2015). Therefore, it is important to understand and refine the soil-structure modelling technique and analysis in generating the response of the offshore wind turbine subjected to cyclic lateral loading. The change in soil-structure stiffness and response due to cyclic lateral loads can lead to the risks of resonance and fatigue damage of the structure.

The stiffness of a structure is a function of the deflection and the natural frequency, fundamental in the design of OWT monopile structure. Deflection and natural frequency using reference 5 MW NREL OWT monopile, modelled in a 20 m water depth is presented in Table 1 for API p-y supported springs. The model and analysis are completed using Ansys, as presented in Fig. 5. The impact and sensitivity of soil scour on

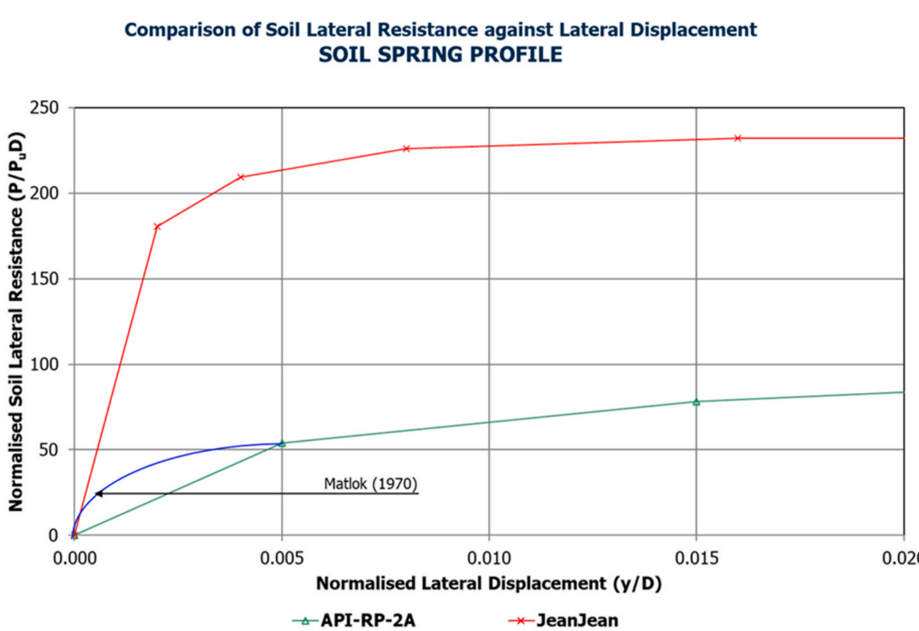

Fig. 4. –Comparison of matlock (1970), AP-RP 2GEO (2011), and JeanJean for normally consolidated clay.

Table 1
Effect of scour on OWT monopile stiffness.

Elevation Below Mudline (m)	Natural Frequency (Hz)		Deflection (Deg)	
	API p-y Springs	JeanJean p-y Springs	API p-y Springs	JeanJean p-y Springs
0.0	0.2028 Hz	0.2132 Hz	1.35°	1.19°
2.5	0.1979 Hz	0.2080 Hz	1.41°	1.25°
5.0	0.1902 Hz	0.1999 Hz	1.52°	1.34°
7.5	0.1796 Hz	0.1895 Hz	1.65°	1.46°

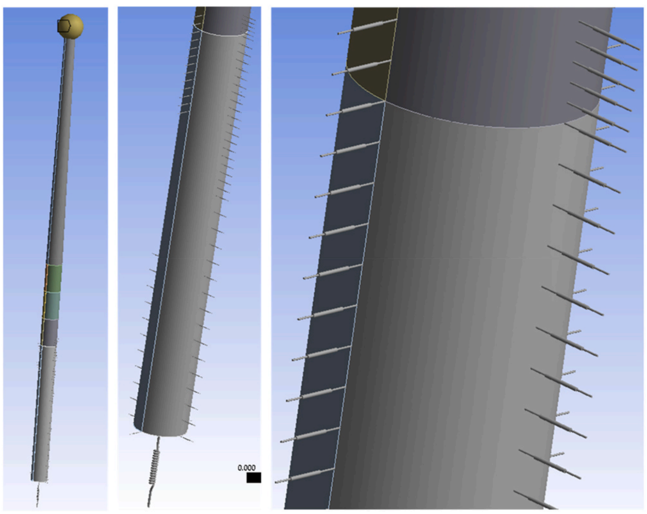
Fig. 5. Typical Finite Element Model: API p-y and JeanJean p-y Generated Soil Springs.

the structure stiffness is conducted for different scour depths below mudline: no scour, 2.5 m scour, 5.0 m scour, and 7.5 m scour. The result shows an increase of 5.0%, 12.6%, and 22.4% in deflection for 2.5 m scour, 5 m scour and 7.5 m scour, respectively, compared with a model with no scour. A corresponding reduction in natural frequency of −2.4%, −6.2%, and −11.4%, respectively, is observed compared with no scour. The analysis is repeated but for new finite element model supported using stiffer p-y springs generated according to JeanJean technique, this shows an average reduction (improvement) in deflection of −11.6% and a corresponding average improvement in stiffness of 5.2% compared with the p-y springs generated according to API method. It is worth noting that the scouring angle is not accounted for in this sensitivity analysis. Detailed analysis to quantify the impact of soil scour depends on the quality of the design data from field measurement that characterises the scour such as the scouring angle, depth of scour, predominate scouring direction, and radius and/or diameter of the scour. Analytical models can also be calibrated and validated using measured monitoring data to enhance accuracy of the predictions.

To avoid resonance, the first frequency of the OWT system must be away from the frequencies of external excitations of wind, wave, and current, including the rotational frequency of rotor (1 P) and the blade passing frequency (3 P for three bladed turbine) (Huang et al., 2016). There is a conflicting outcome as studies show an increase in stiffness even for dense sand during cyclic loading, but current design guidelines consider cyclic loads by reducing the soil-foundation stiffness (Nicolai and Ibsen, 2015). The secant stiffness of soil springs is observed to decrease with an increase in load and the effects are further exacerbated for dynamic compared with static or quasi-static analysis considering wind, wave, and current (Huang et al., 2016). Stiffening of the soil attributed to cyclic loading leads to reduction in the soil structure deformation and increasing the bending moment in the pile around the mudline, dismissing the understanding and description of stiffness degradation due to cyclic horizontal loading of pile foundations (Zachert et al., 2016).

2.5. *Air-tight corrosion design and control by exclusion of oxygen*

The inside of the offshore wind turbine tower and foundation was in recent times considered to be airtight, assuming no corrosion due to lack of oxygen to complete the chemistry. However, this assumption is shown to be invalid as both seawater and oxygen have access to the inside of the monopole under certain conditions, including sites where significant tidal variations exists. This can lead to active corrosion and impact the integrity and capacity of the offshore wind turbine structure. Industry recommended codes and standards are revised to reflect these findings.

Corrosion control by exclusion of oxygen is primarily an option for structural compartments which are only externally exposed to seawater, e.g., internal of legs and bracings of jacket structures that are completed free flooded at installation. Any compartments potentially exposed to air will need to be kept permanently sealed by welding or by maintenance of overpressure by nitrogen to prevent any air ingress. Some compartments such as the interiors of monopiles are periodically accessed for inspection and repair and can therefore not be considered completely sealed. Levels and zones in sea water environment schematic representation is shown in Fig. 6. Effects of large tidal variations on internal water level should be considered. In addition, even in the virtual absence of oxygen in the seawater, corrosion by anaerobic bacteria can occur. It is recognised that an air-tight compartment in monopile structures is not feasible, hence, it is recommended that these issues are taken into consideration when evaluating options for corrosion control for internal compartments.

Presently, new offshore wind turbine structures are conservatively designed for internal corrosion by application of protective coatings, corrosion allowance designs and implementation of corrosion protection and monitoring as additional corrosion control and mitigation strategy. However, these are expensive; therefore, research to establish an optimised cost-effective solution is required.

2.6. *Corrosion allowance and fatigue design*

Fatigue calculation is affected by the corrosion allowance applied to the structural component. The corrosion allowance corresponds and is determined by the corrosion rate and conforms to the assumed corrosion conditions which dictates the S–N curve used for the fatigue calculation. It is recommended that if substantial metal loss is expected, free

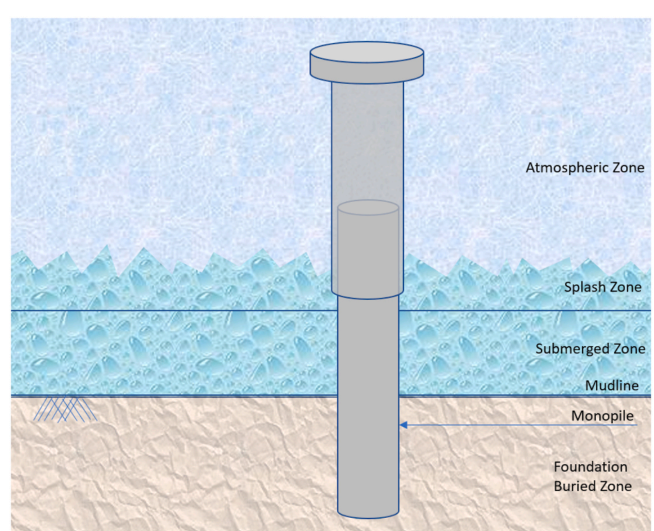

Fig. 6. Schematic representation of levels and zones.

corrosion conditions must in general be assumed, and the "free-corrosion" S–N curve is then required. This aspect of design requires further research to understand the extent and envelope definition of "substantial metal loss" and if the "free-corrosion" S–N curve is appropriate or other S–N curves suitable for the condition along with engineering quantifiable justification. Further guidance is given that the "free-corrosion" S–N curves can be applied for the internal surfaces of monopiles below the waterline.

Cathodic protection is one of the establish methods to mitigate corrosion using sacrificial anodes. However, challenges exist and no established methods on how to control the current output from the anodes due to the partial close compartments (non-airtight). This was recorded in several cases showing acidification and health issues which compromised the structural integrity and safety (Jensen, 2015). Acidification can result in inadequate protection of the steel surfaces due to a higher current requirement, leading to shorter life of anodes.

Localised accelerated low water corrosion of up to 0.5 mm/year is observed where the J-tube seal failed, leading to substantial ingress of seaweater and tidal variations occurring directly inside the foundation. The water level changed daily and at extreme spring tide. The recorded corrosion of 0.5 mm/year exceeded corrosion rate design allowance by DNV of 0.10 mm/year for submerged internal surfaces, 0.15 mm/year for splash zone in temperate climates and 0.20 mm/year in tropical/subtropical climates (Black et al., 2015). For surfaces of primary structural parts exposed in the splash zone and for internal surfaces of the submerged zone, which are without CP, the corrosion allowance (CA) for the surface with or without coating is according to the expression:

$$CA = V_{corr}(T_D - T_C) \tag{7}$$

where V_{corr} is the expected maximum corrosion rate, T_C is the design useful life of the coating and T_D is the design life of the structure.

Corrosion protection system (CPS) on offshore wind turbine structures around the splash zone is a challenging case to design and control due the continuous and intermittent exposure to seawater and oxygen in response to tidal and wave variations. Corrosion protection systems are ineffective around the splash zone, considered as severe corrosive environment compared with atmospheric and submerged zones, hence, further research is required for design and control measures (Aeran et al., 2016). Furthermore, there is no established detailed method to model and analyse patch-type and pitting corrosion which is an area of active research. The localised forms of corrosion such as pitting, and crevice can lead to local stress concentrations and a corresponding reduction in fatigue life and utilization.

Studies recommend corrosion design using time-dependent corrosion rate model instead of corrosion design by assumed generic allowance based on corrosion wastage thickness. The time-dependent corrosion rate model assumes deterioration of the CPS and reduced effectiveness.

2.7. OWT monopile concepts future outlook and other structural considerations

Some important and interesting questions presently being faced by the industry includes understanding the upper bound capacity limit of OWT monopile, the limiting structural criteria, how big and heavy can the structures in line with the capacity increase, how deep and the limit of the installation water depth for OWT monopile structures, manufacturing, and installation considerations. The impact on the dynamic response and structure modes arising from refined hydrodynamic and aerodynamic loads on the larger and bigger structures is also an area of on-going research. The findings and understanding from these topics are required to enhance and improve the structural design techniques and methodology for these bigger and heavier OWT monopiles future concepts. These topics are still unresolved and in fact, is a current research area of interests being conducted for 5 MW–20 MW OWT monopiles in the Ocean and Marine Engineering in the University of Strathclyde.

The transportation and installation analysis and operations of future bigger and heavier OWT require update to industry design codes and standards in-line with new technologies. Although current industry codes and standards provides guidelines for best practices, it does not fully cover novel transport and installation activities and assessments required for future concepts of OWT structures installations. Structural evaluation of new concepts OWT transportation and installation can be addressed using finite element tools with the help of codes and standards but require relevant structural experience and technical know-how on how to deal with these future OWT structures. Another challenging area of interests considering the future concepts is the assessment and control of construction peak noise, noise exposure level, excessive pile inclination, and plastic deformation of the thin-shell pile head associated with driving bigger and heavier OWT monopiles to the designed embedment depth.

The average fixed-bottom offshore wind turbine size for European deployment in 2018 was 6.8 MW (Gaertner et al., 2020). GE Renewable Energy have recently introduced the Haliade-X offshore wind turbine range which features a 14 MW, 13 MW or 12 MW capacity, 220 m rotor diameter, 107 m long blade, and 260 m high. In a similar move, Vestas also introduced 15 MW offshore wind turbine in 2021. New reference 15 MW fixed-bottom offshore wind turbine was presented by the National Renewable Energy Laboratory (NREL) in joint effort with the Technical University of Denmark (DTU) (Gaertner et al., 2020). These reinforces the pressing questions of understanding the design envelops for future bigger and heavier OWT monopile structure and how deep they can be safely and successfully installed, operated, and maintained.

Investigation into 5 MW OWT monopile diameter, thickness, and tower height performed by (Huang et al., 2016) gives an insight into the stiffness of the system. The investigation shows reduction in the tower stiffness as the height increases and/or reduction in wall thickness. The research concluded that the impact of height change is of greater impact on the stiffness of the tower in comparison with the corresponding change in wall thickness as presented in Fig. 7. Influence of tower diameter was also investigated which showed a decrease and increase in bending moment of 19% and 6.3% for 5 m and 7 m pile diameter compared with original 6 m diameter, respectively. Although the investigation did not include the extensive definition of the design envelop for the 5 MW OWT monopile and the governing factors for increasing water depth and structure size, it however highlighted the impact on the structure stiffness and natural frequency which is fundamental to the design of OWT monopiles.

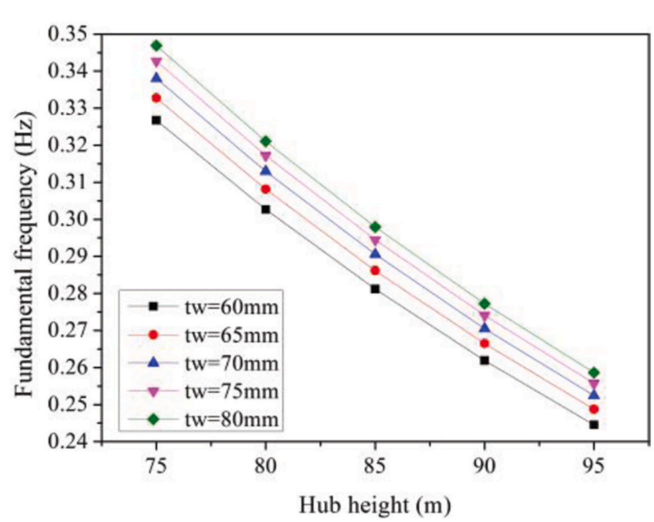

Fig. 7. Fundamental frequencies for different tower heights and wall thicknesses (Huang et al., 2016).

3. Conclusions and recommendation

The holistic review of offshore wind turbine structural design techniques and practices in accordance with industry design codes and standards is presented in this paper. The review is primarily focused on fixed-bottom OWT monopile structural design and analysis. Several academic works and existing industry techniques and technologies are reviewed, highlighting the salient design achievements, challenges, and opportunities for future research and development activities of bigger and heavier OWT monopiles concepts structural design. The following conclusions can be drawn:

1. Grouted connection structural response and capacity is improved by the introduction of shear keys; however, this limits the fatigue strength through stress hotpots and should be addressed through detailed refined local analysis. There is lack of detailed guidance for state-of-the art fatigue assessment of OWT and the use of finite element analysis are encumbered with significant uncertainties, requiring calibration with experimental test data or well-known cases where such data exist.
2. Total damping is influenced by the character of the individual loads and the combined effects may be established from structural analytical investigations and sensitivity checks. Calculation of the total damping currently suffers from assumptions and subjective applications on estimating the individual damping ratio. The damping coefficient is an important dynamic parameter for modelling and conducting representative dynamic analysis. Hence, further research is required to address this issue and to improve the calculation of the offshore wind turbine design fatigue life. Previous research has attempted to calculate the tower oscillation and soil damping through "rotor stop" tests and assumption, estimation of the steel material and aerodynamic damping, and assuming the hydrodynamic damping. There are strong interests on improving the total (and individual) damping coefficient, including the influence on the structural dynamic behaviour and response of future bigger and heavier OWT monopile concepts.
3. Modelling and analysis of cyclic loading and foundation scour remains a challenging issue. Measured data across different offshore wind farms indicates significant variation in the scour hole shape (horseshoe). The horseshoe vortex and lee-wake vortex form the processes that govern scour, dictated primarily by the Keulegan-Carpenter number. Results from sensitivity demonstrates the influence of scour on the global stiffness and modes of the OWT monopile structures, including the impact of soil-structure foundation modelling techniques. However, more work is required to fully capture the influence of scour on bigger diameters, thicknesses, and turbine capacity loads.
4. The natural frequency as well as the fatigue loading, and response are significantly affected by the soil-structure interaction understanding and modelling. The famous p-y curves method is limited to smaller pile diameters. Although strides in research are being made in the offshore wind turbine industry such as the PISA project for soil-structure interaction, there is the need for future research and calibrated modelling technique, including the understanding on future OWT monopile structure concepts.
5. It is recommended that if substantial metal loss is expected, "free-corrosion" conditions and S–N curve must be assumed and applied. This aspect of design requires further research and justification to understand the extent and definition of "substantial metal loss" and the appropriateness of "free-corrosion" S–N curve. In addition, there is no established method and the need for future research into modelling and analysis of patch-type and pitting corrosion.
6. Extensive research and industry studies are required on the low technology readiness levels of future bigger and heavier OWT monopile structure concepts. Some areas of interests include but not limited to defining the design envelope and limits of future concepts up to and including 20 MW OWT monopiles, and maybe higher, the limiting structural criteria, installation depth, and installation considerations such as acceptable noise exposure level and excessive pile inclination that may arise from driving bigger diameter pile. Although, financial models and economic analysis are not reviewed in this paper, the costs impact of structural design techniques and methodology may be worth investigating for the future OWT monopile concepts.

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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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