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Permanent accumulated rotation of an offshore monopile wind turbine in sand during a storm

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Abstract

Offshore wind turbines exposed to storm situations are subjected to static and dynamic loads from the same direction over a considerable period of time. Such cyclic loading can potentially result in soil degradation, leading to an undesired permanent rotation of the wind turbine. This paper presents a workflow to predict the permanent accumulated rotation of an offshore monopile wind turbine in sand during an extreme storm event incorporating the use of fully nonlinear irregular waves versus linear waves in current practice. The fully nonlinear irregular waves are realized from a potential flow solver OceanWave3D previously validated at up to near-breaking wave conditions. Given the wave kinematics, the aero-hydro-elastic code HAWC2 is used to calculate horizontal loading and bending moment acting on the embedded pile head. The irregular load series is then decomposed into a set of constant-amplitude load parcels using rainflow counting. Eventually, the permanent accumulated rotation is predicated using the method proposed by LeBlanc et al. (2010b) with Miner's rule-based superposition. In this paper, a case study of the DTU 10MW wind turbine supported by a monopile at 33 m water depth in sand is presented, where the pile is primarily laterally loaded. The simulation results suggest the importance of taking accumulated rotation into design. The permanent accumulated rotation is primarily decided by soil capacity, loading characteristics and pre-loading history. Furthermore, the results show that wave nonlinearity has only limited influence on the permanent accumulated rotation.

Keywords: monopile, cyclic loading, permanent accumulated rotation, fully nonlinear irregular waves, sand

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Nomenclature		H	Horizontal load
β	Wave spectrum shape factor	h	Water depth
$\Delta\omega$	Wave frequency bandwidth	H_S	Significant wave height
$\dot{u}_{rel}$	Relative water particle acceleration	k	Initial modulus of subgrade reaction
η	Wave surface elevation	L	Wave length
γ	Peak enhancement factor	L_{em}	Embedded pile length
γ'	Soil submerged unit weight	M	Bending moment
ω	Wave frequency	M_R	Ultimate soil moment capacity
ω_p	Peak wave frequency	M_{max}	Maximum moment of cyclic loads
ϕ	Wave phase	M_{min}	Minimum moment of cyclic loads
ϕ'	Soil friction angle	N	Number of cyclic loads
ρ	Water density	p	Lateral soil reaction force per unit length
θ	Accumulated rotation	p_u	Ultimate resistance of the sand
θ_s	Static rotation	T_b	Rotation accumulation magnitude fuction
ξ_b	Cyclic load intensity parameter	T_c	Rotation accumulation profile fuction
ξ_c	Cyclic load profile parameter	T_P	Peak wave period
A	Static or cyclic loading condition parameter	u	Water particle velocity
A	Wave amplitude	u_{rel}	Relative water particle velocity
A_i	Internal cross sectional area	V_g	Wave group velocity
A_s	Sand density coefficient	wsp	Mean wind speed
A_t	Total cross sectional area	y	Lateral soil deflection
C_a	Added mass coefficient	z	Soil depth
C_d	Drag coefficient	OWT	Offshore wind turbine
$C_{1,2,3}$	Soil lateral resistance coefficients	SLS	Serviceability limit state
D	Pile diameter		
e	Load eccentricity		

1. Introduction

Offshore wind power is currently gaining an increased popularity around the world as a promising source of renewable energy, especially in a number of European countries such as Denmark, Germany, the United Kingdom and the Netherlands. In order to make offshore wind energy more competitive with traditional energy sources, optimization of foundation designs is of high importance as the foundation can account for up to 35% of the total installed cost (Byrne & Houlsby, 2003). Up to now, monopile is the dominant foundation with 87% of the market share as it is a more cost effective solution with cheaper fabrication and installation cost (WindEurope, 2018). The loading acting on offshore wind turbines (OWTs) is highly cyclic as a result of the stochastic nature of the environmental conditions, which might lead to possible softening/hardening of the surrounding soil and eventually lead to permanent accumulated rotation or lateral displacement at mudline (Zaaijer, 2006; Bhattacharya et al., 2013; Lombardi et al., 2013; Damgaard et al., 2014; Carswell et al., 2016). In terms of the serviceability limit state (SLS) assessment for monopile OWTs, the accumulated rotation prediction is typically a critical aspect as opposed to the prediction of displacement (Arany et al., 2017). DNV (2014) suggests that OWT monopiles are typically designed to not exceed the tolerance of 0.5° for the total rotation at mudline where the installation tolerance at seabed is 0.25° and the limit for permanent accumulated rotation becomes 0.25° . Therefore, the permanent accumulated rotation due to cyclic loading should be a key consideration to achieve a safe design of the OWT, and it has been a subject of intense study.

Extensive attentions have been paid on incorporating cyclic loading effect into numerical simulations to model the lateral response of piles since the development of the so-called $p - y$ curves. Reese et al. (1974) suggested that the soil reaction p should be reduced by a constant factor to account for the cyclic effect, which has been advocated in current design standards such as API (2007) and DNV (2014). The model is independent of the cyclic load characteristics and the number of cycles, hereby it cannot capture the pile behavior cycle by cycle. Little & Briaud (1988) further addressed this issue by introducing the idea of degradation index as a function of the number of cycles. Following this work, Long & Vanneste (1994) refined the degradation index to cover a wide range of soil densities, load conditions, pile types and materials. Alternatively, a versatile dynamic nonlinear Winkler foundation model was developed by Allotey & El Naggar (2008) to account for cyclic soil degradation. Although the current $p - y$ approach has gained broad recognition, the accuracy and validity of these $p - y$ curves, when applied to the soil-pile response of a large diameter monopile, are questionable (Kallehave et al., 2015). As an alternative to the $p - y$ approach, 3D finite element model is widely used for modeling complex 3D soil-structure interaction problems. Cuéllar et al. (2014) and Corciulo et al. (2017) used a comprehensive 3D finite element model with cyclic hydro-mechanical soil coupling to capture the transit response subject to an extreme loading. In principle these models developed with implicit concept can simulate the pile behavior under the loading cycle by cycle, but they are restricted to limited number of cycles considering the expensive computation time. Alternatively, an explicit degradation stiffness model was proposed by Achmus et al. (2009) based on a

combination of a finite element simulation of the pile–soil interaction and an evaluation of drained cyclic triaxial tests. It was further reported by Achmus et al. (2010) that the results
45 using this model compare well with scaled pile test results. Although the 3D finite element model is the most accurate way to simulate soil-structure interaction, it is impractical to use in the design optimization stage regarding the cost of great computation efforts.

In alignment with the numerical modeling, a number of experimental tests have been conducted to investigate the cyclic behavior of laterally loaded piles and several empirical
50 formulations have been derived by fitting the recorded measurements. LeBlanc et al. (2010b) performed small-scaled laboratory tests on stiff pile subject to long term cyclic loading, and developed an accumulation model based on a power law. The model is in agreement with the findings from Little & Briaud (1988), Long & Vanneste (1994), Klinkvort & Hededal (2013), Li et al. (2015) and Truong et al. (2018). In addition, logarithmic expressions have
55 also been proposed by Hettler (1981), Lin & Liao (1999) and Achmus et al. (2009). These formulations simulate the long term cyclic soil response as a function of the cyclic load characteristics and the number of cycles based on an explicit concept, in contrast to an implicit concept that the soil response is simulated cycle by cycle. Therefore, they are believed to be efficient in practical applications to monopile designs. Furthermore, it is
60 found out that the accumulated rotation could be better approximated by the power law in contrast to the logarithmic formulation (Abadie, 2015; Barari et al., 2017; Yang et al., 2018). Hereby, the model proposed by LeBlanc et al. (2010b) is used in this study.

Current models are primarily derived from scaled pile tests using a wide range of regular cyclic loadings from extreme one-way loading to extreme two-way loading, which are unre-
65 alistic loading situation experienced by an OWT. To the author’s knowledge, the study on cyclic response of a realistic OWT, exposed to stochastic environmental conditions at least in a single storm (DNV, 2014), is rare and needs further investigation. Therefore, the aim of this paper is to investigate the permanent accumulated rotation experienced by a realistic OWT during an extreme storm. For this purpose, a design framework, which enables the
70 designer to calculate the accumulation of monopile rotation in a realistic and practical way, is established. In this paper, the DTU 10MW wind turbine supported by a monopile in sand sited in 33 m water depth is investigated. A 39-hour storm with 50-year return period is adopted in this study (NORSOK N-003, 2007). In addition, the results from LeBlanc et al. (2010a) indicated that largest load cycles, although very few in number, have a significantly
75 higher impact on the permanent accumulated rotation than small load with thousands of cycles, and it has been reported from a range of available literature that loads predicted from nonlinear wave models are often larger than those predicted from linear wave theory (Agarwal & Manuel, 2011; Marino et al., 2015; Schler et al., 2016; Wang & Larsen, 2017). With these findings in mind, this paper also investigates the effect of using fully nonlinear
80 irregular waves on predicting the permanent accumulated rotation.

This paper starts with a description of the research methodology used in this study, with detailed elaboration on wave modeling, aero-elastic simulations and cyclic accumulation analysis. Afterwards, a case study of the DTU 10MW OWT is presented and the results are summarized and discussed. Finally, the conclusions are given.

2. Research methodology

In principle, the research methodology is based on the explicit concept, which means that the soil deformation is not simulated cycle by cycle, instead it is calculated using a post-processing step without considering the interaction between soil deformation and structural responses. The research methodology is schematically shown in Figure 1 through three steps: wave realizations, aero-elastic simulations and cyclic accumulation analysis. Regarding wave realizations, fully nonlinear irregular waves are realized using a validated potential wave solver OceanWave3D (Engsig-Karup et al., 2009), in addition to linear wave theory widely used in current practice. Consequently, wave kinematics associated with the linear and nonlinear wave realizations are incorporated into aero-elastic calculations using HAWC2 (Larsen & Hansen, 2015). The horizontal loading and bending moment acting on the embedded pile head are obtained and then converted into constant-amplitude cyclic load parcels using the rainflow counting method (Khoa & Jostad, 2017) for the cyclic accumulation analysis. In order to perform cyclic accumulation analysis, static moment-rotation relation of the pile is firstly established using nonlinear $p - y$ curves of sand recommended by API (2007) assuming rigid behavior of the embedded pile. Finally, the model proposed by LeBlanc et al. (2010b) with Miner's rule-based superposition is used for prediction of the permanent accumulated rotation exposed to extreme storms.

Figure 1: Flowchart of the research methodology for prediction of offshore monopile permanent accumulated rotation exposed to an extreme storm event.

2.1. Irregular wave realizations

With respect to a realistic sea, the wave fields vary continuously over space and time in an irregular manner. Hereby, a realistic sea can not be represented by a deterministic regular wave, and it should be treated as a stochastic process.

Linear wave theory is the most common way to represent Gaussian stochastic ocean waves and is widely used in aero-elastic simulations for dynamic load analysis of OWTs. A wave spectrum, typically a Jonswap spectrum (Hasselmann et al., 1973), is the starting point for generating linear irregular waves. The Jonswap spectrum is defined by the following equations:

$$S(\omega) = (1 - 0.287 \ln(\gamma)) \cdot \frac{5}{16} H_S^2 \omega_p^4 \omega^{-5} \exp(-\beta \frac{\omega_p^4}{\omega^4}) \gamma^a \quad (1)$$

$$a = \exp(-\frac{(\omega - \omega_p)^2}{2\omega_p^2 \sigma^2}) \quad (2)$$

$$\sigma = \begin{cases} 0.07 & \omega \leq \omega_p \\ 0.09 & \omega > \omega_p \end{cases} \quad (3)$$

where $\beta = \frac{5}{4}$, $\gamma = 3.3$, H_S is the significant wave height, ω is the wave frequency and ω_p is the peak wave frequency.

A stochastic wave elevation time series $\eta(t)$ from the spectrum can be generated using the following linear superposition:

$$\eta(t) = \sum_i A_i \cos(\omega_i t + \phi_i) \quad (4)$$

$$A_i = \sqrt{2S(\omega_i)\Delta\omega} \quad (5)$$

$$\phi_i = rand(0, 2\pi) \quad (6)$$

where A_i is the i th wave amplitude, ω_i is the i th wave frequency, $\Delta\omega$ is wave frequency bandwidth and ϕ_i is the i th random wave phase.

In terms of the nonlinear irregular wave realizations, the fully nonlinear potential flow solver OceanWave3D, developed by Engsig-Karup et al. (2009), is applied, which solves the 3D Laplace equation for the velocity potential, the free surface elevation up to the point of breaking with nonlinear boundary conditions at the free surface and the impermeability condition at the seabed. The interested reader is referred to Engsig-Karup et al. (2009) for further information.

2.2. Load simulation and transformation

2.2.1. Aero-hydro-elastic simulations

DTU Wind Energy developed aero-elastic code HAWC2 (Larsen & Hansen, 2015; Larsen et al., 2014) is used to perform all the aero-hydro-elastic simulations based on multibody

formulation with floating frame. For OWT exposed to a storm, the wind turbine is at a standstill condition by setting rotational speed as zero and pitch angle to a feathered position. In addition, control is not active when the turbine is at a standstill condition. Therefore, the aerodynamic loads are negligible, and the hydrodynamic loads and wave-excited structural vibrations dominate the loads at the monopile mudline. Regarding hydrodynamic loads, the Morison equation is used with three force terms: the Froud-Krylov force, the water added mass and the drag force. The hydrodynamic loading per unit length of flooded members is written as:

$$f = \frac{1}{2}C_d\rho D u_{rel}|u_{rel}| + \rho(A_t - A_i)\dot{u} + \rho(C_a A_t + A_i)\ddot{u}_{rel} \quad (7)$$

where C_d and C_a are drag and added mass coefficients respectively. For monopiles, the wave force is dominated by inertial force where the theoretical value of added mass coefficient C_a is 1. In addition, the value of C_d is also chosen as 1 considering a rough structure due to corrosion and marine growth. u is the water particle velocity, and u_{rel} , $\dot{u}_{rel}$ represent the relative velocity and acceleration respectively. ρ is the water density and D is the diameter of slender piles. A_t and A_i are the total and internal reference cross sectional areas. The internal area, A_i , accounts for water mass inside the cylinder to include its inertial force.

2.2.2. Transform irregular loading to cyclic loading

The typical load series used for monopile geo-technical design consist of the simultaneous lateral force H and bending moment M . The combined load pairs (H, M) can be represented by a single loading M or H using the deterministic relation $M = H \cdot e$, where e is called as load eccentricity representing the average correlation coefficient between the bending moment M and the lateral force H . Thus, the moment time series can be used as the design load series, which consists of a considerable number of load cycles with highly irregular load amplitudes. It requires a great deal of computation effort for direct modeling in time domain. For the sake of convenience, the irregular load time series are usually rearranged into the so-called load parcels, which have been widely addressed in practical design (Andersen, 2015; Carswell et al., 2016). For each load parcel, it is a N cycles constant-amplitude load series bounded by the maximum and minimum moment, M_{max} and M_{min} respectively in this study. The transformation is performed with either the rainflow counting method (Matsuishi & Endo, 1968) or the method developed at Norwegian Geotechnical Institute (NGI) (Andersen, 2015). In the case of the NGI method, pore pressure accumulation is considered for determining the equivalent number of cycles, which is likely more accurate. However, the computational expense and complexity of the model are limiting factors for its practical use (Carswell et al., 2016). Therefore, the rainflow counting method is used in this study to perform the transformation. The procedure is illustrated in Figure 2.

For each load parcel, the cyclic loading is characterized in terms of two non-dimensional parameters, ξ_b and ξ_c , determined by the ultimate soil capacity M_R (LeBlanc et al., 2010a,b).

$$\xi_b = \frac{M_{max}}{M_R} \quad (8)$$

Figure 2: Illustration of transforming an irregular load series into load parcels of constant-amplitude cyclic loading using the rainflow counting method (modified from Khoa & Jostad (2017)).

$$\xi_c = \frac{M_{min}}{M_{max}} \quad (9)$$

where ξ_b is a measure of the load magnitude normalized with the static moment capacity within the range of 0 to 1. ξ_c quantifies the characteristics of the cyclic loading bounded with -1 to 1, which takes the value 1 for a static loading, 0 for one-way loading and -1 for two-way loading.

2.3. Permanent accumulated rotation analysis

2.3.1. API $p - y$ curves for sand

API (2007) standard recommends the use of Reese's sand $p - y$ curves (Reese et al., 1974) for soil lateral resistance in case where no more specific information is available. In general, $p - y$ curves are nonlinear and a function of depth, soil type, pile dimensions and properties. The curves for sand are described with depth z as:

$$p = Ap_u \tanh\left(\frac{kz}{Ap_u}y\right) \quad (10)$$

where p_u is the ultimate resistance of the sand at depth z , the factor A accounts for cyclic or static loading condition, which is chosen as 0.9 at all depths as the loading is cyclic. k is the initial modulus of subgrade reaction and kz is called subgrade modulus. y is the soil lateral deflection. The schematic $p - y$ curve comprises an initial portion of a straight line with a constant slope kz , followed by a hyperbolic curve until it reaches the soil resistance p_u , after which the $p - y$ curve becomes flat and extends to a certain extent (Jia, 2018), which is depicted in Figure 3 at different depths for sand.

The initial modulus of subgrade reaction k , which is also called coefficient of subgrade reaction, is approximated as (Terzaghi, 1955):

$$k = \frac{A_s \gamma'}{1.35} \quad (11)$$

Figure 3: Schematic representation of nonlinear $p - y$ curves at different depths for the geological condition of sand.

where γ' is the submerged unit weight, and A_s is a coefficient which depends only on sand density, which is chosen as 900 for medium dense sand leading to a value of 6000 kN/m³ for k .

The ultimate lateral soil resistance for sand has been found to vary with depth. Based on the recommendations from API (2007), the ultimate lateral bearing capacity at shallow soil depth can be calculated as:

$$p_{us} = (C_1 z + C_2 D) \gamma' z \quad (12)$$

at deeper depth, the ultimate lateral bearing capacity can be calculated as:

$$p_{ud} = C_3 D \gamma' z \quad (13)$$

where C_1, C_2, C_3 are the lateral resistance coefficients as a function of the effective internal friction angle of sand. γ' is the effective density of soil and D is the pile diameter.

In practical design, at a given depth the equation giving the smallest value of p_u should be used as the ultimate lateral resistance:

$$p_u = \min(p_{us}, p_{ud}) \quad (14)$$

2.3.2. Cyclic accumulation model

A number of experimental investigations have been performed and several models have been proposed (Niemunis et al., 2005; LeBlanc et al., 2010b; Klinkvort & Heddal, 2013; Wichtmann & Triantafyllidis, 2017). The cyclic accumulation process and its relation with a moment rotation curve are illustrated in Figure 4. The static moment-rotation curve is used to determine the rotation in the first loading cycle, similar to a static pushover analysis. The most commonly used cyclic accumulation models in the literature for embedded pile rotation subject to cyclic loading are primarily in exponential or logarithmic type, some of which are summarized in Table 1.

Figure 4: Illustration of cyclic accumulation for the rotation θ_N after cyclic loading with number of cycles N (reproduced from LeBlanc et al. (2010b)). The value for θ_0 can be replaced by θ_s corresponding to the static rotation.

Table 1: Cyclic accumulation models proposed by different researchers for long-term accumulated rotation estimation of lateral loaded pile (θ is the accumulated rotation after N cycles, θ_s is the rotation that would occur in a static test, the other parameters are not elaborated here).

Reference	Evolution Model
Hettler (1981)	$\theta = \theta_s(1 + t \ln N)$
Little & Briaud (1988)	$\theta = \theta_s N^m$
Achmus et al. (2009)	$\theta = \theta_s N^{b_1} X^{b_2}$
LeBlanc et al. (2010b)	$\theta = \theta_s(1 + T_b T_c N^{0.31})$
Klinkvort & Hededal (2013)	$\theta = \theta_s N^{T_b T_c}$

In this paper, the model proposed by LeBlanc et al. (2010b) is applied for predicating the cyclically accumulated rotation in sand, which is built up on the model from Little & Briaud (1988) following a power function written as:

$$\theta = \theta_s + \Delta\theta(N) \quad (15)$$

$$\Delta\theta(N) = T_b T_c N^{0.31} \theta_s \quad (16)$$

where θ is the maximum rotation after a load parcel with N cycles and θ_s is the static rotation responding to the load magnitude $M_{\max}$. T_b and T_c are dimensionless coefficients primarily depending on the load characteristics ξ_b and ξ_c . T_b is also affected by the relative density of sand, and the value of 38% is assumed in this study, the same data as used in LeBlanc et al. (2010b). The functions of T_b , T_c are determined by a series of centrifuge tests (LeBlanc et al., 2010b) and subsequently linearly approximated by Arany et al. (2017).

$$T_b(\xi_b) = \begin{cases} 0 & 0.0 \leq \xi_b \leq 0.051 \\ 0.4238\xi_b - 0.0217 & 0.051 < \xi_b \leq 1.0 \end{cases} \quad (17)$$

$$T_c(\xi_c) = \begin{cases} 13.71\xi_c + 13.71 & -1 \leq \xi_c < -0.65 \\ -5.54\xi_c + 1.2 & -0.65 \leq \xi_c < 0 \\ -1.2\xi_c + 1.2 & 0 \leq \xi_c \leq 1 \end{cases} \quad (18)$$

The function T_b denotes whether the yield strength is breached, which shows a linear dependence of the load magnitude ξ_b . Moreover, T_b has a value of 0 under cyclic loading with a small magnitude $\xi_b \leq 0.051$, which implies that no accumulation of rotation would occur as the pile-soil interaction is elastic and reversible in this condition. The function T_c represents the influence of load amplitude, which follows a third-step established linear relation ensuring that $\Delta\theta = 0$ for $\xi_c = 1$ corresponding to a static loading. It is also found that the maximum value of T_c is reached at $\xi_c = -0.65$. This observation disagrees with the conclusion from Klinkvort & Hededal (2013) that one-way loading, corresponding to $\xi_c = 0$, is the most critical load scenario. Figure 5 shows the power function relation between the cyclically accumulated rotation θ with number of cycles N for a given load characteristics. One can also observe a significant decrease of accumulation rate with increasing number of cycles. Such a decrease is possibly a result of sand densification where the near-field soil is stiffened under cyclic loading (Niemunis et al., 2005; LeBlanc et al., 2010b; Bhattacharya et al., 2013; Klinkvort & Hededal, 2013; Wichtmann & Triantafyllidis, 2017).

Figure 5: Illustration of cyclically accumulated rotation and its corresponding accumulation rate as a function of the number of cycles.

2.3.3. Miner's rule-based rotation superposition

The permanent accumulated rotation subject to variable cyclic loadings can be superposed based on a relation similar to Miner's rule (LeBlanc et al., 2010a). The assumption using Miner's rule is that the damage is independent of loading consequence, which has been validated against experimental data (LeBlanc et al., 2010a; Li et al., 2015; Wichtmann & Triantafyllidis, 2017). For example, the accumulated rotation θ_a for N_a cycles of load parcel a is calculated as:

$$\Delta\theta_a = (\theta_s T_b T_c)_a N_a^{0.31} \quad (19)$$

Alternatively, the accumulated rotation $\Delta\theta_a$ can be represented by N_b^0 cycles for load parcel b and the equivalent number of cycles N_b^0 can be calculated as:

$$N_b^0 = \left(\frac{\Delta\theta_a}{(\theta_s T_b T_c)_b} \right)^{1/0.31} \quad (20)$$

Therefore, the resulting accumulated rotation for load parcel b with N_b cycles following load parcel a with N_a cycles can be evaluated as:

$$\Delta\theta_b = (\theta_s T_b T_c)_b (N_b^0 + N_b)^{0.31} \quad (21)$$

And the permanent accumulated rotation under cyclic load parcel (a, b) according to Eq. (15) is written as:

$$\theta_b = \Delta\theta_b + \max(\theta_{s,a}, \theta_{s,b}) \quad (22)$$

With this approach, more load parcels with variable cyclic loading characteristics can be applied while maintaining the rotation accumulated in previous loading history. The procedure for prediction of accumulated pile rotation subjected to variable cyclic loadings is further illustrated in Figure 6 with three load parcels. After N_1 cycles for the first load parcel, a rotation θ_1 is generated, which is equivalent with N_2^0 cycles for the second load parcel. The cyclically accumulated rotation for second load parcel with N_2 cycles then starts from the point (N_2^0, θ_1) and reach the point $(N_2^0 + N_2, \theta_2)$. In a similar manner the procedure is applied to the third load parcel with N_3 cycles leading to the final rotation at θ_3 . The procedure can be applied to more load parcels with random sorting order as the accumulated rotation is independent of the loading consequence.

Figure 6: Illustration of accumulated pile rotation subjected to variable cyclic loadings following a order of 1 - 2 - 3.

The phenomenon of superposition subject to variable cyclic loadings demonstrates that the cyclic accumulation rate is highly dependent on the initial rotation accumulated from the previous loading history.

3. Application to the DTU 10MW offshore turbine

3.1. Offshore wind turbine structure

The DTU 10MW reference wind turbine (Bak et al., 2013), which is a conventional horizontal axis, three bladed and upwind type turbine on a tubular tower, supported by a large monopile sited in 33 m water depth is studied, shown in Figure 7. It is considered to be representative for the latest multi-MW wind turbine on the market. The monopile is designed as 7.5 m wide, 85 mm thick and the embedded lengths below seabed are assumed to be 30, 35 and 40 m. Key design properties are summarized in Table 2. It is remarkable that this model is assumed to be fixed at seabed with a rigid foundation and the wind turbine locked during the storm. The load transfer is primarily through lateral soil structure interaction where the horizontal loading and bending moment acting on the surrounding soil are substantial compared to the vertical loading also shown in Figure 7.

Figure 7: The DTU 10MW wind turbine supported by a monopile sited in 33 m water depth. The pile embedded lengths below seabed are 30, 35 and 40 m and the load transfer mechanism is also demonstrated (modified from Bhattacharya et al. (2012)).

Table 2: Key design properties of the DTU 10MW reference wind turbine supported by a large monopile sited in $h = 33$ m (Nat.Freq means natural frequency and damping is given in the format of logarithmic decrement).

Parameters	Values
Rated power	10MW
Rated wind speed	11.4 m/s
Cut-in, cut-out speed	4 m/s, 26 m/s
Controller	Variable-speed pitch control
Rotor speed	6 rpm - 9.6 rpm
Rotor diameter	178.3 m
Hub height	119 m above MSL
Water depth	33 m
Embedded pile length	30, 35, 40 m
Monopile diameter	7.5 m
Monopile thickness	0.085 m
1 st Nat.Freq, Damping	0.22 Hz, 5.8%
2 nd Nat.Freq, Damping	0.23 Hz, 5.9%

3.2. Bathymetry and storm profile

215 A representative seabed profile is used in this paper with a slope around 1:100, similar to Schler et al. (2016), depicted in Figure 8. The total distance of the wave domain is 11500 m between water depth $h = 100$ m and $h = 30$ m. The monopile-supported OWT is sited in $h = 33$ m water depth. The waves in the storm event are generated in 100 m water depth and move up over the sloping seabed until 30 m water depth.

Figure 8: Seabed profile with a 1:100 slope, the black dot shows the location of the OWT and the black dashed lines show the wave generation zone and relaxation zone respectively in the numerical wave tank within OceanWave3D.

A typical storm sea state with 50-year return period in the Northern North Sea located at approximately E3.0, N58.5 with 100 m water depth was provided by DHI. As a detailed investigation on storm profiles is not the focus of this study, a generic storm profile taken from NORSOK N-003 (2007) is considered reasonable. This involves three phases: a build-up phase, a peak phase and a decay phase, depicted in Figure 9. The peak phase lasts 3 hours, while the build-up and decay duration are each 18 hours (Tarp-Johansen, 2005). The temporal development of wind speed is assumed to be similar to the trend of wave condition during the storm with a turbulence intensity assumed to be 0.11. The storm profile is discretized into 1 hour sea state. Therefore, 19 sea states are obtained in total, listed in Table 3. The relation between peak wave period T_P and significant wave height H_S was established by DHI as described in the following equation:

$$T_P = 4.38\sqrt{H_S} \quad (23)$$

220 The bathymetry shown in Figure 8 was modeled within OceanWave3D in order to capture the nonlinear effects such as re-distribution of the energy between free and bound components in the nonlinear wave realization as well as wave transformation along the sloping seabed (Schler et al., 2016).

225 Linear irregular waves from a Jonswap spectrum are generated in 100 m water depth within the 1000 m wave generation zone and assumed as uni-directional to simplify the

Figure 9: Temporal development of a generic storm profile with 1-hour discretization (NORSOK N-003, 2007).

problem as two dimensional. At the end of the fluid domain, a wave relaxation zone is defined where waves are damped out numerically. The length of wave relaxation zone is identical with wave generation zone, depicted in Figure 8. The OWT location is also shown as a black dot in 33 m water depth.

Based on the convergence study performed by Schløer et al. (2016), at least 10 points underneath wave surface and at least 8 points per wave length are necessary to ensure all waves of interest are resolved properly using OceanWave3D. The energy spectrum shown in the sea states is approximately bounded between 0.05-0.30 Hz. The shortest wave component with the frequency of 0.30 Hz has the wave length of $L = 17.3$ m in 33 m water depth based on linear dispersion relation and therefore the grid spacing is set as 1.5 m in the x -direction. In addition, 10 points are resolved underneath the wave surface elevation.

The transition time for these waves to travel from the wave generation zone to the investigated location is decided by travel distance and wave group velocity. The slowest wave with the frequency of 0.30 Hz has a group velocity as $V_g = 2.6$ m/s, thus it takes approximately 4000 s to reach the location in 33 m water depth. In order to ensure that all the interested wave components in the energy spectrum are captured at the investigated location, the transition time is set as 4400 s, shown in Figure 10.

Regarding nonlinear wave realizations, waves move from $h = 100$ m to $h = 33$ m following the shoaling process by which waves entering shallower water change in wave height. Specifically, the significant wave height H_S in the investigated water depth $h = 33$ m is reduced, listed in Table 3. It is consistent with the findings of Schløer et al. (2016). The significant wave heights H_S in $h = 33$ m were then used to generate linear irregular waves in order to ensure that the obtained wave kinematics from linear and nonlinear waves have the same H_S and T_P values.

Table 3: Metocean data of 19 sea states in a generic storm (wsp : mean wind speed; $H_{S,100}$: significant wave height in 100 m water depth; $H_{S,33}$: significant wave height in 33 m water depth; T_P : peak wave period).

Sea State	wsp [m/s]	$H_{S,100}$ [m]	$H_{S,33}$ [m]	T_P [s]
1	25.7	6.6	5.9	11.3
2	27.1	7.0	6.2	11.6
3	28.5	7.3	6.5	11.9
4	29.9	7.7	7.0	12.2
5	31.3	8.1	7.4	12.5
6	32.6	8.5	7.8	12.7
7	34.0	8.8	8.1	13.0
8	35.4	9.2	8.4	13.3
9	36.8	9.6	8.8	13.5
10	38.2	9.9	9.1	13.8
11	39.6	10.3	9.4	14.1
12	41.0	10.7	9.8	14.3
13	42.4	11.0	10.1	14.6
14	43.8	11.4	10.4	14.8
15	45.1	11.8	10.8	15.0
16	46.5	12.2	11.1	15.3
17	47.9	12.5	11.4	15.5
18	49.3	12.9	11.7	15.7
19	50.0	13.2	12.0	15.9

Figure 10: The nonlinear wave surface elevation for the severest sea state $H_S = 12.0$ m, $T_P = 15.9$ s. The transition period is 4400 s and in total 8000 s is required for the simulation.

250 3.3. Geological condition and static soil response

It is challenging to obtain the geological data considering the requirement of site investigation with expensive cost. A first reasonable estimation for the uppermost layers is loose to medium dense sand in the North Sea (Arany et al., 2017). The loose to medium sand in the upper layers used in this paper has a submerged unit weight of $\gamma' = 9 \text{ kN/m}^3$ and friction
 255 angle of $\phi' = 35^\circ$. The geological data for sand are summarized in Table 4. In the absence of site-specific information, the nonlinear $p - y$ curves recommended in API (2007) and DNV (2014) are used to establish soil resistance in the lateral direction. The soil resistance is assumed to increase linearly with depth.

Table 4: Geological parameters and $p - y$ curve coefficients of sand recommended by API (2007).

Parameter	Symbol	Value	Unit
Submerged unit weight	γ'	9	kN/m^3
Angle of internal friction	ϕ'	35	deg
Relative density	R_d	38	%
Lateral resistance coefficient	C_1	2.8	-
Lateral resistance coefficient	C_2	3.4	-
Lateral resistance coefficient	C_3	52.8	-
Sand density coefficient	A_s	900	-
Initial modulus of subgrade reaction	k	6000	kN/m^3

Static pile moment rotation relation is the starting point for its accumulation analysis subject to cyclic loading, which can be established by either a static load test in experiments (LeBlanc et al., 2010b) or using finite element analysis (Achmus et al., 2009). In this study, HAWC2 was applied by setting up the pile supported with distributed $p - y$ curves of sand, shown in Figure 11(a). For the DTU 10MW wind turbine supported by a large diameter pile with three embedded length $L_{em} = 30, 35, 40 \text{ m}$, pile diameter $D = 7.5 \text{ m}$ and pile wall thickness $t_p = 0.085 \text{ m}$, the embedded pile behaves in a rigid manner without significant flexibility. The rigid pile behavior is described as a “toe kick” under simultaneous lateral load H and bending moment M (LeBlanc et al., 2010b). Because H and M are not perfectly correlated, there is no precise way of linking rotation θ to a simultaneous pair of (H, M) . Therefore, the combined pair of loads (H, M) is represented by a single loading M or H with the relation $M = H \cdot e$, where e is called load eccentricity. A constant load eccentricity, the average correlation coefficient between mudline moment M and horizontal force H , was used as a simplification. The eccentricity was determined to be 25 m, shown in Figure 11(b). The pile moment rotation relations were then established by static load tests with increasing load level H acting on the pile with a value of 25 m for eccentricity, depicted in Figure 12. The initial stiffness is also demonstrated, which can be determined via the following equation

(Darvishi-Alamouti et al., 2017):

$$\theta = \frac{24L_{em} + 36e}{kL_{em}^4 e} M \quad (24)$$

where L_{em} is pile embedded length, e is load eccentricity, and k is the initial modulus of subgrade reaction. Detailed derivation of the initial stiffness is given in the Appendix. The initial stiffness is normally used to determine the natural frequency of OWTs.

Figure 11: (a) Lateral loaded rigid pile with distributed $p-y$ curves modeled in HAWC2. (b) Determination of eccentricity using the average relation between mudline moment M and horizontal force H .

It is remarkable that a clear point of sand failure could not be established from the tests to determine the ultimate soil capacity. Here, the definition from LeBlanc et al. (2010b) is used, that the ultimate soil capacity is the bending moment causing 4° rotation. It shows that a slightly larger embedded length gives a significantly higher ultimate soil capacity.

Figure 12: Static moment rotation relations established using HAWC2 ($L_{em} = 30/35/40$ m, $D = 7.5$ m, $e = 25$ m).

4. Results and discussion

4.1. Linear and nonlinear wave realizations

Wave skewness is used to quantify the departure from Gaussianity to compare the difference between linear and nonlinear wave realizations. The wave with higher peaked crest and rounded trough will be displayed by a positive skewness. Positive skewness obtained from nonlinear waves are stated in Table 5, denoting that nonlinear surface elevation has crest values larger than trough values. The skewness is around zero for all linear wave realizations, indicating a Gaussian distribution for linear wave surface elevation. Besides, the skewness of nonlinear wave realizations increases with more severe sea states. An exemplary time series of linear and nonlinear wave realizations is depicted in Figure 13, where higher peaked crest and rounded trough are noticeable in the nonlinear wave elevation.

Table 5: Wave skewness of the 19 sea states in the storm.

Sea State	1	2	3	4	5	6	7	8	9	10
Linear	0.01	-0.02	-0.07	0.00	0.00	0.02	-0.01	-0.01	0.01	-0.01
Nonlinear	0.19	0.14	0.19	0.22	0.23	0.17	0.23	0.24	0.23	0.23
Sea State	11	12	13	14	15	16	17	18	19	
Linear	0.01	-0.01	0.01	-0.01	0.00	-0.10	0.14	-0.02	0.09	
Nonlinear	0.28	0.28	0.30	0.34	0.38	0.39	0.42	0.42	0.42	

Figure 13: Comparison of the linear and nonlinear wave realizations of the most severe sea state $H_S = 12.0$ m and $T_P = 15.9$ s.

Alternatively, the amplitude spectrum of surface elevation was calculated to further understand the influence of wave nonlinearity. The amplitude spectral densities of three

representative sea states are depicted in Figure 14. They are intended to represent the relatively small, medium and large sea states studied in this paper. The spectrum of nonlinear wave shows a secondary peak at around twice of the peak wave frequency and another small peak close to zero frequency. Such a redistribution of energy arises due to sum and difference interactions of frequencies from nonlinear wave transformation. The phenomenon is more pronounced with severe sea states.

Figure 14: Amplitude spectral density of wave surface elevation for the linear and nonlinear waves of three representative sea states.

4.2. Aero-elastic simulation results

In order to investigate the wave forces independent from the dynamic properties of the OWT structures, a fixed monopile was firstly considered to evaluate the loading differences between using linear and nonlinear waves together with the Morison equation. For the purpose of linking the wave elevation and its corresponding loading, exceedance probabilities of the peak values of wave elevation η , horizontal force H and mudline moment M are calculated using Eq. 25:

$$P(x_i) = 1 - \frac{i - 1}{N} \quad (25)$$

where x_i is the i th peak value of η, H, M identified using zero down-crossing method and sorted in increasing order. N is the number of peak values.

Figure 15 presents the exceedance probabilities of the peak values of η, H, M when linear and nonlinear waves are used for the three representative sea states. The probability plot shows that using the nonlinear wave realizations gives higher peak values of η, H, M at rare probability level compared to those predicted for the linear waves, and their trends are similar. The deviation between the linear and nonlinear waves becomes more notable in more severe sea states.

Afterwards, the simultaneous aerodynamic loads and hydrodynamic loads were taken into account in the aero-elastic simulations. Figure 16 shows the amplitude spectral density of fore-aft mudline moment using the linear and nonlinear waves. The spectrum has a significant peak at around 0.22 Hz, which is the natural frequency of the first bending mode

Figure 15: The exceedance probability of the peak values for η, H, M using the linear and nonlinear waves of three representative sea states ($1^{st} - H_S : 5.9m, T_P : 11.3s$; $10^{th} - H_S : 9.1m, T_P : 13.8s$; $19^{st} - H_S : 12.0m, T_P : 15.9s$; LW-linear wave, NW-nonlinear wave).

of tower in the fore-aft direction. The peak is pronounced due to poor suppression as a result of low aerodynamic damping value with parked rotor. Another significant peak is seen at around the peak wave frequency showing a quasi-static behavior subjected to hydrodynamic force. The small peak at around twice the peak wave frequency also shows in the plots.

4.3. Permanent accumulated rotation

As stated earlier, the highly irregular loading series obtained from aero-hydro-elastic simulations need to be transformed into a set of load parcels using the rainflow counting method. Figure 17 shows the histogram of load magnitude for all the load parcels transformed from the mudline moment series during the storm. Almost identical numbers of cycles are found

Figure 16: Amplitude spectral density of the mudline moment using the linear and nonlinear waves of three representative sea states.

for the load parcels with small loading magnitude as a result of the dominating structural vibration in its first tower bending natural frequency. More cycles, nevertheless, are obtained for the load parcels with large loading magnitude when the nonlinear waves are used. This phenomenon is consistent with the exceedance probability of hydrodynamic force acting on the structure, shown in Figure 15. The cyclically accumulated rotations are calculated for each load parcel and depicted in Figure 17 for the embedded pile of $L_{em} = 35$ m. The results agree with the conclusion from LeBlanc et al. (2010a) that the largest load cycles, although very few in number, generate higher accumulated rotation than the thousands of small load cycles. This behavior is also valid for the monopile with L_{em} as 30 and 40 m. It indicates that the number of loading cycles has a relatively small effect on the accumulated rotation compared to the effect from load characteristics.

The accumulation of permanent rotation during the 39-hour storm was calculated based on Miner's rule, depicted in Figure 18. In general, less rotation is accumulated for a longer pile which has more ultimate moment capacity. Moreover, using linear waves slightly underestimates the permanent accumulated rotation. It also shows that the build-up of permanent accumulated rotation becomes slower due to pre-accumulated rotation. This is because the surrounding soils are typically densified as a result of the cyclic pre-loading. These results demonstrate that the accumulated rotation primarily depends on the soil capacity, loading characteristics and pre-loading history.

Table 6 lists the permanent accumulated rotation for the three embedded lengths studied. Static pushover analysis was also performed so that the rotation was calculated directly from the static moment rotation relation subject to the largest moment in the load series. The values calculated using cyclic analysis are significantly larger than these from static analysis. Regarding nonlinear waves, 67% – 195% more rotation is predicted using cyclic analysis compared to static analysis. It is also seen that 6% – 10% more rotation is predicted when nonlinear waves are used for cyclic analysis.

Figure 17: The histogram of load magnitude M_{max} for all the load parcels transformed from the mudline moment series during the storm and its corresponding cyclically accumulated rotation for monopile with $L_{em} = 35$ m.

Table 6: Permanent accumulated rotation calculated using static pushover and cyclic analysis.

$L_{em} = 30m$	Static [deg]	Cyclic [deg]	Ratio
Linear	0.409	1.125	2.75
Nonlinear	0.422	1.243	2.95
Ratio	1.03	1.10	
$L_{em} = 35m$	Static [deg]	Cyclic [deg]	Ratio
Linear	0.195	0.399	2.05
Nonlinear	0.200	0.432	2.16
Ratio	1.03	1.08	
$L_{em} = 40m$	Static [deg]	Cyclic [deg]	Ratio
Linear	0.114	0.181	1.59
Nonlinear	0.116	0.194	1.67
Ratio	1.02	1.06	

4.4. Discussion on practical implications for design

Although for practical design full scale validation is required, the research findings from the case study can reflect some practical implications for future design approaches. The current $p - y$ approach with a constant reduction factor to account for the cyclic effect is clearly shown not represent the rotation accumulation process. Alternatively, the explicit methodology presented in this paper based on the model proposed by LeBlanc et al. (2010b) and Miner's rule-based superposition shows the potential suitability for the early stages of the design process. The permanent accumulated rotation of an OWT is found to be sensitive to the embedded pile length, hereby the pile geometry should be carefully selected by the

Figure 18: Accumulation of the permanent rotation during the 39-hour storm using linear waves and non-linear waves for $L_{em} = 30, 35, 40$ m.

designer to fulfill the SLS requirements. Additionally, the results show that investigation of the storm peak condition should be sufficient for preliminary design because all largest load cycles present in this period. Furthermore, wave nonlinearity has only limited influence on OWT permanent accumulated rotation, hereby it should be sufficient to use linear waves in the permanent rotation analysis. The design procedure should be iterated in order to find a safe, but less conservative design.

5. Conclusions

According to DNV (2014), prediction of permanent accumulated rotation of the pile head, at least for a single storm, is typically critical in terms of fulfilling the requirement for SLS. In this paper, we present a workflow to predict the permanent accumulated rotation in a storm by including the effect from fully nonlinear irregular waves versus commonly used linear wave theory. The DTU 10MW wind turbine sited in 33 m water depth with three embedded lengths below seabed as 30, 35 and 40 m in sand was investigated.

Regarding the analysis for wave realizations, the nonlinear waves realized from Ocean-Wave3D tend to be more non-Gaussian causing wave crests to be sharper and wave troughs flatter. The corresponding extreme wave forces due to nonlinear waves are predicted to be larger than those calculated from linear waves based on the Morison equation.

The aero-elastic simulations of wind turbine response were performed using HAWC2 for 19 1-hour sea states, using both linear and fully nonlinear wave models. The dynamic response is dominated by the first bending mode of tower in the fore-aft direction due to

low aerodynamic damping value with parked rotor. Moreover, the monopile responds to the wave force in a relatively quasi-static manner. Following this, the rainflow counting method was used to rearrange the highly irregular load series into a set of so-called load parcel, a N cycles constant-amplitude load series bounded by the maximum and minimum moment. The number of cycles for the load parcels with small loading magnitude is found out to be almost identical. However, a few more cycles are obtained for the load parcels with extremely large loading magnitude when the nonlinear waves are used.

The method proposed by LeBlanc et al. (2010b) with Miner's rule was used to predict the cyclically accumulated rotation during the storm. The extremely large load cycles, although very few in number, generate higher accumulated rotation than the thousands of small load cycles. Furthermore, the analysis shows that the cyclically accumulated rotation is significantly higher than the value predicted from static analysis. The results show that the accumulated rotation primarily depends on the soil capacity, the pre-loading history and loading characteristics. Ultimately, this study suggests that nonlinear irregular waves have only limited influence on the permanent accumulated rotation.

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Appendix

The subgrade modulus in sand is expressed mathematically as kz , where k is the initial modulus of subgrade reaction and z is soil depth. The unit soil reaction forces at each soil layer above pivot point and below it, denoted as dF_1 and dF_2 respectively, can therefore be calculated as:

$$\begin{cases} dF_1(z) = kz \cdot (\lambda L_{em} - z)\theta = kz(\lambda L_{em} - z)\theta & 0 \leq z \leq \lambda L_{em} \\ dF_2(z) = kz \cdot (z - \lambda L_{em})\theta = kz(z - \lambda L_{em})\theta & \lambda L_{em} < z \leq L_{em} \end{cases} \quad (26)$$

where λ is used to determine the ratio of length above pivot point to the total embedded length. The total soil reaction forces can be derived as:

$$F_1 = \int_0^{\lambda L_{em}} dF_1(z)dz = \int_0^{\lambda L_{em}} kz(\lambda L_{em} - z)\theta dz = \frac{k\theta L_{em}^3 \lambda^3}{6} \quad (27)$$

$$F_2 = \int_{\lambda L_{em}}^{L_{em}} dF_2(z)dz = \int_{\lambda L_{em}}^{L_{em}} kz(z - \lambda L_{em})\theta dz = \frac{k\theta L_{em}^3 (2 - 3\lambda + \lambda^3)}{6} \quad (28)$$

The resultant moments of soil reaction around the ground point at pile head are calculated by:

$$M_1 = \int_0^{\lambda L_{em}} dF_1(z)zdz = \int_0^{\lambda L_{em}} kz^2(\lambda L_{em} - z)\theta dz = \frac{k\theta L_{em}^4 \lambda^4}{12} \quad (29)$$

$$M_2 = \int_{\lambda L_{em}}^{L_{em}} dF_2(z)zdz = \int_{\lambda L_{em}}^{L_{em}} kz^2(z - \lambda L_{em})\theta dz = \frac{k\theta L_{em}^4(3 - 4\lambda + \lambda^4)}{12} \quad (30)$$

θ and λ are obtained using the force and moment equilibrium equations:

$$\sum F = 0 \Rightarrow H + F_2 - F_1 = H + \frac{k\theta L_{em}^3(2 - 3\lambda)}{6} = 0 \quad (31)$$

$$\sum M = 0 \Rightarrow M + M_1 - M_2 = M + \frac{k\theta L_{em}^4(4\lambda - 3)}{12} = 0 \quad (32)$$

replacing H with $\frac{M}{e}$ and substituting Eq. (31) into Eq. (32) yield:

$$\theta = \frac{24L_{em} + 36e}{kL_{em}^4 e} M \quad (33)$$

$$\lambda = \frac{3L_{em} + 4e}{4L_{em} + 6e} \quad (34)$$

It is noted that the moment-rotation relation is linear for a rigid pile. The derived Eq. (33) and Eq. (34) are identical with what derived by Darvishi-Alamouti et al. (2017).

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