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DOI link to article:
https://doi.org/10.1016/j.soildyn.2017.03.008

Date deposited:
12/09/2017

Embargo release date:
05/04/2017

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Deformation Mechanisms for Offshore Monopile Foundations Accounting for Cyclic Mobility Effects

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Abstract

There has been a huge surge in the construction of marine facilities (e.g., wind turbines) in Europe, despite the many unknowns regarding their long-term performance. This paper presents some frameworks for design strategy based on performance measures for cyclic horizontally loaded monopile foundations located in saturated and dry dense sand, by considering pile deformations and pore pressure accumulation effects. A three-dimensional finite element model was developed to investigate the behavior of large-diameter piles. This model accounts for nonlinear dynamic interactions in offshore platforms under harsh combined moment and horizontal environmental loads, with emphasis on the cyclic mobility of the surrounding cohesionless subsoil and associated shear.

The maximum moment applied in the cyclic analyses is varied from 18\% to 47\% of the ultimate resistance. The considered data reflect behavior at the expected load amplitudes and cycle numbers during the service life of operation.
For low numbers of load cycles (<1000 cycles), there were no differences between the power law and logarithmic approaches in terms of describing the accumulated deformations; however, for high numbers of cycles (<10,000 cycles), the logarithmic law was less suited to describe the accumulation response. Magnitude of cyclic loads was found to cause a linear increase in the accumulated rotation. The results from short-term and long-term dynamic response of monopiles indicate that few load cycles with higher load levels are the main concerns in accumulation of pile rotation rather than thousands of load cycles with low amplitudes.

**Keywords:** Wind turbine foundation, Long-term cyclic loading, Dense sand, Cyclic mobility, Transient

**Introduction**

The design and analysis of foundations for offshore turbines are challenging endeavors, due to the harsh environmental conditions that these structures experience. Recently, such structures have been developed extensively in Europe (e.g., see Kuo et al. 2012; Achmus et al. 2009; Barari and Ibsen 2011, 2012, 2014; Barari et al. 2015; Larsen et al. 2013; Ibsen et al. 2014a, 2014b, 2015; Bhattacharya et al. 2013; Doherty and Gavin 2012; Kirkwood and Haigh 2014). The first fully operational offshore wind farm (Hors Rev 1) was installed in the eastern north sea and was supported on a gravity base. The wind farm consists of 80 Vestas V80 type, producing a total amount of 160 MW power. Foundation concepts that are frequently used for offshore wind turbines include monopiles, jackets, and tension-leg floating substructures. Doherty and Gavin (2012) described a state-of-the-art in foundation design for offshore platforms. Under suitable soil conditions, monopiles have shown to be feasible in water depths of up to 35 m. Jacket foundations, consisting of a four-legged steel lattice frame placed on piles, are the preferred choice for water
depths ranging from 35 to 60 m. A deep-water floating turbine was recently installed off the coast of Norway known as Statoil’s Hywind, but the suitability of this design is generally restricted due to commercial issues. Hywind used Spar buoy concept where static stability is achieved through ballast weights situated under a central buoyancy tank (Statoil 2013; Butterfield 2007).

Due to their slender nature, offshore wind turbines are dynamically sensitive when used under adverse environmental conditions. During the lifetime of a wind turbine, the monopile foundation may be subjected to either a small number of lateral load cycles with large amplitudes, in conjunction with a result of severe earthquakes or storms, or to regular of lateral load cycles with intermediate amplitudes, due to wave loading in the fatigue and serviceability limit states (FLS and SLS, respectively) (Wichtmann et al. 2008; Roesen et al. 2013). In the literature, non-continuum analytical approaches (e.g., subgrade reaction methods) and finite-element (FE) techniques have been widely used to determine the response of offshore piles to lateral loading.

The p-y curve is a subgrade reaction technique derived from large-scale testing on two flexible, slender piles, according to the design standards of API (2000) and DNV (2009). Both standards recommend the p-y curves initially formulated by Reese et al. (1974) and O’Neil and Murchison (1983). The p-y methodology is not based on rational mechanics, and material parameters are typically chosen empirically, through observations of pile behavior. Several factors (e.g., diameter and soil-pile stiffness) are not addressed in this methodology, which can lead to severe restrictions.

Applying design standards for stiff offshore piles to wind turbines with a slender slenderness ratio less than 10 is not within the verified range of these standards.

Many authors have provided long-term performance predictions and observations for these relatively novel structures. LeBlanc et al. (2010) was among the first to address the issue of accumulated rotation and stiffness changes for small-scale stiff piles after long-term cyclic
loading between 8000 and 60,000 load cycles. They thoroughly investigated the dependence of accumulated rotation on relative density, which they found to be very sensitive to cyclic load characteristics. Tasan et al. (2010) developed and implemented a fully coupled, two-phase, three-dimensional (3D) FE model for explicitly describing the accumulation of water pressure close to the monopile as a function of the number of cycles. Klinkvort et al. (2012) and Choi et al. (2012) described the change in the bedding resistance in subgrade reaction methods with the number of load cycles.

Aim and scope of the paper

Although several authors have performed small-scale 1-g tests (LeBlanc et al. 2010; Roesen et al. 2013; Rackwitz et al. 2012; Qin and Guo 2007), the applicability of the proposed observations to the design of full-scale monopiles remains questionable. In particular, the stress distribution in a 1-g experiment is not identical to that in the full-scale condition. On the other hand, although values can be scaled in centrifuge experiments conducted at N_g and at the correct stress level corresponding to the full-scale prototype, scaling to prototype is still a difficult task, especially for cyclic tests, and limitations exist (Klinkvort et al. 2012).

This article describes a numerical model for predicting the accumulated pile rotation under one-way cyclic and transient lateral loading, as well as applications to investigate stress paths and soil-pile interactions. The full-scale numerical simulations reported in this paper offer promising predictions for the salient features of soil behavior that were previously not accounted for offshore monopiles. This paper complements previous studies in the field by presenting a series of parametric studies of the developed model predictions.

Dynamic considerations of offshore wind turbines
Problem definition and methodology

System analyzed and FE model

A study was performed on a non-slender pile under different loading conditions. A 3D nonlinear static-dynamic analysis model of a soil-pile system was developed in the FE code OpenSees version 1.7.3 (Mazzoni et al., 2010). A total of 384 elements were employed. Soil and pile elements were modeled by using 8-node, fully coupled (solid-fluid) brick and beam elements with 4 and 6 degrees of freedom (DOFs), respectively. Rigid beam-column connections, normal to the pile longitudinal axis, were used to represent the geometric space occupied by the pile. The 3D brick elements of the soil domain were connected to the pile geometric configuration at the outer nodes of the rigid links, this was achieved by equalDOF constraint in OpenSees for translations only.

Three-dimensional modeling relies on the use of a validated fully coupled porous media (soil skeleton)-pore fluid (water) dynamic FE formulation. No special elements were defined for the soil-pile interface as the soil constitutive model accounts for the interface interaction (He 2005).

Figure 2 depicts a typical FE discretisation, together with the detailed loading scenarios applied to the FE models (length = 100 m, depth = 45 m). The generated model can be visualized using GID software (Diaz and Amat 1999). A monopile diameter of 7.5 m with varying embedded length and wall thickness of 9 cm was assumed. In the numerical simulations, the pipe section of the monopile was replaced by a solid section with equivalent bending stiffness. The suitability of this simplification was previously confirmed by Achmus et al. (2009) and Barari et al. (2015). The bottom boundary of the model was taken to be 15 m below the base of the monopile. When a model length of 100 m was used, the calculated behavior of the monopile was not influenced by the boundaries. Tables 1 and 2 provide the characteristics of the model parameters for the physical pile structure and soil parameters for cyclic loading in the model.

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To improve the computational efficiency, only half of the models were discretised. To the bedrock layer, all DOFs were restrained at the bottom boundary of the meshes. All symmetry planes were fixed against displacement normal to the symmetry faces, but were free to move on the surface of the plane. All simulations were undertaken using the OpenSees based on a u-p formulation. The Inelastic soil behavior was described by a multi-surface yield surface with nonlinear kinematic hardening and an associated plastic flow rule.

Three cases of static and dynamic loading, at the head of the pile, were considered. Each case was subdivided into different loading paths and soil characteristics:

- Case A: Cyclic loading on a pile with a diameter of 0.72 m, wall thickness of 0.06 m, and subjected to (i) 12 cycles from 960 up to 480 kN (P32) and (ii) 12 cycles from 960 kN up to 0 kN (P344);
- Case B: Static pushover-type of analysis on a hollow steel monopile of 7.5 m in diameter;
- Case C: Dynamic loading on large-diameter monopile, with (i) 1000 cycles of 0.2 Hz one-way cyclic lateral loads of 7.2, 10 and 18.8 MN for dry sand; (ii) 500 cycles of 0.2 Hz one-way cyclic lateral loads of 7.2 and 10 MN for saturated sand; and (iii) transient effects due to a typical storm combined with cyclic loading.

Soil constitutive modeling

This paper presents a brief overview of the equations used to generate the nonlinear soil model for predicting the cyclic behavior of granular materials and pile responses.

The multisurface-plasticity theory for frictional cohesionless soils described by Yang and Elgamal (2002) was employed to simulate the nonlinear shear behaviour of dense sand next to the foundation. The model uses a purely deviatoric kinematic hardening rule (Prevost 1985) with flow
rules allowing for representing the hysteretic cyclic shear stress-strain response of the soil. The
new flow rules implemented by Elgamal et al. 2003 in OpenSees change the essentials of the
original framework (Prevost 1985) in order to incorporate the cyclic mobility mechanisms.

The model was used to distinguish the nonlinear response as a function of the soil stiffness,
permeability, and dilation potential. It accounted for time-varying strength changes, based on
estimates of the pore-pressure field adjacent to the pile, and simulated the rate-dependency of the
response due to different loading conditions.

If P denotes the direction of plastic flow, the level of dilation or contraction during cyclic
loading is defined by a volumetric component $P^*$ as:

$$
P^* = \begin{cases} 
\frac{1-(\eta/\eta_{pt})^2}{1+(\eta/\eta_{pt})^2} \epsilon_c \text{ (for contraction)} \\
\frac{1-(\eta/\eta_{pt})^2}{1+(\eta/\eta_{pt})^2} \exp(d,\gamma_d) \text{ (for dilation)} 
\end{cases} 
$$

(1)

where $c_1$, $d_1$, and $d_2$ are coefficients modeling the amount of plastic shear work done when
the soil is in contraction ($c_1$) and dilation ($d_1$, $d_2$). Octahedral shear strain and stress ratio are
modeled by $\gamma_d$ and $\eta$ respectively as introduced by Yang et al. (2003). The stress ratio of the
Phase Transformation (PT) surface (Ishihara, 1985) is also defined by $\eta_{pt}$. Please note that if
$(1-(\eta/\eta_{pt})^2)$ is positive, the stress states lies below the PT surface. On the other hand, a negative
value implies that the stress state lies above the PT surface.

Comparison with results of centrifuge modeling

The well-documented centrifuge test of Rosquoët et al. (2004) on a pile founded on dense sand
exposed to a cyclic horizontal load was modeled numerically (case A). One-way cyclic force-time
history (Fig. 3) varying from 960 to 480 kN (p32 test) was modeled numerically in 3D to verify and calibrate the developed framework for the dynamic analysis of the model. Lateral loads were applied at 1.6 m above the soil surface, 1 m below the head of the pile model with an embedded length of 12 m in dense sand.

It can be inferred from the literature that, overall, the response observed in a soil element can not be directly transferred to a soil-foundation system owing to the difference between drainage conditions in test and in situ conditions (i.e., generation of 3-D hydraulic gradients), existence of local high excess pore pressures around the pile, and difference in boundary conditions.

It is pointed that constitutive soil model validated against element tests needs additional calibration to capture the behaviour of soil-pile response. To validate the numerical results, a procedure similar to one developed by Achmus and Abdel-Rahman (2012) for pile foundations in sand was applied and the main modeling parameters were listed in Table 2.

Figure 4 compares the force-displacement curve of the pile head under the P32 load cycles to the experimental data. Comparison of the numerical predictions for the Soil Structure Interaction (SSI) with the data measured and computed by Rosquoët et al. (2004) indicated that, as expected, the developed model realistically reflected predicted the accumulation of pile displacements in dense sand under cyclic loading.

Rayleigh damping was implemented into the model in which a frequency range of 0.1 to 5 Hz was set as effective range and 5% Rayleigh damping was assigned to the soil producing the mass and stiffness coefficients as 0.061 and 0.003, respectively.

Encouraged by the ability of the proposed model to capture such a complex nonlinear behavior, the model was used to examine the influence of material nonlinearities on the dynamic response of a monopile.
Results and discussion

Cyclic loading: harmonic excitation

It seems logical to evaluate the behavior of a monopile subjected to cyclic lateral loading through the accumulated rotation rather than the lateral deformation of the pile. Therefore, the main part of this article involves the evolution of rotational deformations. Initially, a series of FE static pushover-type of analyses was performed at different values of moment arm, $h$ to derive the bearing strength envelope of the soil-monopile system (Fig.5). Also shown on the figure are ranges of $\xi_h$ for $h=30$ m. This parameter is described in subsequent sections modeling the cyclic loading ranges. Figure 5 interpreted the FE results in terms of non-dimensional groups as suggested by LeBlanc et al., (2010):

$$\frac{M}{D\gamma(L)^2} = \frac{H}{D\gamma(L)^2}$$  \hspace{1cm} (2)

In which $H$ and $M$ denote horizontal load and moment at the sea-bed resulting lateral displacement and rotation.

Figure 6 outlines the evolution of the soil yield computed from the FE analysis and its progressive extension to greater depths in conjunction with load levels. Soil plastification developed in the vicinity of the monopile head at small strain levels, although only the upper part (i.e., active length) participated in failure. By contrast, the failure mechanism in a bucket foundation extends towards the base of the caisson (Barari and Ibsen, 2012).

It was however difficult to identify a failure point from static analyses. A limiting value for the horizontal capacity was defined as \(~40\) MN which was chosen by taking into account the limiting pile head displacement, $0.1D$ criterion proposed by Cuéllar (2011).
Comparisons of the calculated bending moments, shear forces and soil reactions at different stages of virgin loading are presented. The shear force and soil reaction may be calculated indirectly from the bending moment by double differentiation (Fig. 7) at different stages of virgin loading. The model captured the increase in the magnitude of the maximum soil reaction, and the depth at which the maximum occurred.

Cutoff frequency

The developed model, considering homogeneous soil conditions, was subjected to a sinusoidal dynamic load of 10 MN, applied at the top of the monopile. Two vibration periods which are deliberately higher and lower than the first natural period of the soil profile, \( T = 5 \text{s}, \frac{T_{\text{mol}}}{s} = 0.45 \text{s} \) and \( 0.13s, \frac{T_{\text{mol}}}{s} = 0.45 \text{s} \), and two model sizes (50 and 100 m) were investigated. Under a short excitation period (Fig. 8(b)), the lateral boundaries had a clear influence on the response. A large amount of undamped wave energy was present in the model domain. In contrast, the response under the long-period excitation (Fig. 8(a)) was hardly distinguishable between the two model sizes. However, the radiated seismic energy was negligible, providing additional evidence for the existence of a cutoff frequency. Considering the above model verification, to reduce the computational cost in the nonlinear dynamic analysis, a distance of 50 m was adopted in all of the subsequent cases.

In spite of the existence of the lateral loading for offshore monopiles, four important design loads for offshore monopiles have been described (Det Norske Veritas 2007; LeBlanc et al., 2010; Zhu et al. 2013):

- Ultimate Limit State (ULS), experienced once during the wind turbine lifetime;
- Worst expected transient load (WETL = ULS/1.35), experienced once during the lifetime;
SLS (~47% of the ULS), experienced frequently (~100 times) during the lifetime; and

FLS (25~30% of the ULS), experienced very frequently (~10^7 times) during the lifetime.

Cyclic creep, defined as the accumulation of plastic strain in the vicinity of the pile, is accompanied by hardening or softening of the soil. The cyclic stress ratio of the soil elements, which is used to determine the cyclic creep, corresponds to the cyclic load ratio of the whole soil-pile interaction system. It is defined by the ratio of the cyclic load amplitude to the static bearing capacity of the pile. Two different parameters were defined to characterize the cyclic lateral load by Buckingham’s hypothesis (Roesen et al. 2013):

\[
\begin{align*}
\xi &= \frac{M_{\text{max}}}{M_x} \\
\xi_c &= \frac{M_{\text{max}}}{M_{\text{max}}} 
\end{align*}
\]

in which \(M_{\text{max}}\) and \(M_{\text{min}}\) are the maximum and minimum moment in the load cycle (plotted in Fig. 9), and \(M_x\) is the static capacity. The loading type is denoted by the dimensionless parameter \(\xi_c\), which typically ranges from 0 (one-way cyclic loading) to -1 (full two-way cyclic loading).

Several long-term cyclic loading analyses were performed, in both saturated and dry sands. Analyses were conducted with one-way cyclic loading and a target \(\xi_c = 0\). The loading magnitudes were chosen to reflect realistic loading conditions for the FLS and SLS. Different load regimes of types A, B, and C are listed in Table 3.

Analysis of cyclic rotation

The moment-rotation curves governed by static and cyclic loads are presented in Fig. 10. Dense sand can be expected to show a stiffer response than the original soil in the region as previously

Commented [MR12]: Inconsistent use of symbols from here (see Figs 9-11)? I think 'xi' is the correct symbol and not 'zeta'?

Commented [MR13]: Does not make sense? What is meant by the soil in the region?
reported by Roesen et al. (2012). The rotation in the first loading cycle was almost equal to the rotation obtained in the static pushover analysis (Fig. 10). The cyclic triaxial tests carried out by Niemunis et al. (2005) indicated that the stress exponent (herein called the load exponent) of the stress-strain curve follows a quadratic relationship. Figs. 11 and 12 present the pile rotation at the soil surface as a function of the number of cycles (N).

According to Roesen et al. (2013), most functions for the displacement of structures under cyclic loading are either exponential (e.g., Eq. (4a)) or logarithmic as follows: (e.g., Eq. (4b)):

\[
\frac{\Delta \theta}{\theta_1} = aN^b \quad (a) \\
\frac{\Delta \theta}{\theta_1} = a + \ln(N)^b \quad (b)
\]

where \( \theta_1 \) is the rotation obtained in the first loading cycle. Two dimensionless constants \( a \) and \( b \) can be determined empirically from either physical modeling or numerical analyses.

Both expressions were fitted to the maximum values of the rotation computed in the FE simulations. A varied normalization of the rotation is found in the literature by LeBlanc et al. 2010. The proposed normalization of \( \frac{\Delta \theta}{\theta_s} \) (where \( \theta_s \) is the rotation occurring in a static analysis when load is equivalent to \( \tau \times M_R \)) is only valid when evaluating the maximum rotation. Figure 11 shows the computed response of the evolution of the pile rotation, \( \theta_s \), at the soil surface as a function of the number of cycles under three types of continuous cyclic loading for dry sand. Rotation accumulated throughout the entire analysis, and similar trends were obtained in two additional analyses for saturated sand (Fig. 12). In these evaluations, the maximum values of the rotation were used (dashed lines in Figs. 11 and 12).

Commented [MR14]: Remove the symbols (a) and (b) as they can be confused with the parameters a and b?
Further inspection of Fig. 10 and Table 3 reveals that $\theta_1 = \theta_1$; therefore, the accumulated rotations obtained from different cyclic analyses in Fig. 13 were normalized to the rotation obtained in the first loading cycle. Predictions obtained with Eqs. (4a) and (4b) shown in Figs. 13 and 14 suggested that the model well-predicted the accumulated rotation of the stiff pile with a power function, as previously shown by Peralta and Achmus (2010). For dry sand, $b$ ranged from 0.47 to 0.52; these values are larger than the value of $b = 0.31$ reported by LeBlanc et al. (2010) for small-scale stiff piles.

Using the data from these simulations, expressions fitting the first 1000 cycles were determined. Similar plots were obtained for saturated sand when the maximum values were fitted. As shown in Table 4, the Pearson correlation coefficients between the computed and fitted expressions were between 0.951 and 0.999, representative of the accumulated rotation of an offshore monopile.

Equation (4a) can be described more conveniently herein:

$$\frac{\Delta \theta}{\theta_1} = T_s \left( \frac{\xi}{\zeta} \right) N^b$$

(5)

where the coefficient $a$ in Eq. (4a) is interpreted by dimensionless functions $T_s$ and $T_c$ in terms of the load characteristics. As such, the function $T_s$ can be assumed here equal to unity, which arises from $\xi \zeta = 0$.

Hence, on rearranging Eq. (5), the normalised form of accumulated rotation is given by:

$$\frac{\Delta \theta}{\theta_1} = T_s \left( \frac{\xi}{\zeta} \right) N^b$$

(6)

The results arisen from the assumption of a nearly constant $b$ observed in analyses with respect to the sand saturation (for example $b = 0.49$ for dry sand), indicates that $T_s$ depends linearly on the load magnitude, $\xi$, varying in a range between 0.18 and 0.47 (Fig. 15). A similar tendency

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was observed from small scale tests by LeBlanc et al. (2010) and Roesen et al. (2013), although the reported values for $T_s$ cover a lower range of variation. However, further studies are required to verify the form of models developed for both dry and saturated sands, and to extend them for predicting long-term behaviour of offshore piles.

These findings suggest that lateral cyclic loading on monopiles embedded in cohesionless soils will not typically result in complete shakedown, but instead will result in the progressive accumulation of deformations. Hettler (1981) called this phenomenon shakedown evolution. In this attenuation mechanism, the rate of displacement will constantly decrease, but will never reach zero, because the effect is modeled in a logarithmic fashion with load cycles. For certain loading cycles, the logarithmic function of the number of cycles (proposed by Hettler 1981; Rosquoët et al. 2004; and Lin and Liao 1999) is eventually followed by an over-logarithmic accumulation stage for higher numbers of load cycles. The validity of this issue was investigated for the FLS, and deformation accumulation was approximated by power laws in contrast to the logarithmic formulation.

Influence of load order

Next, variable cyclic load patterns and the validity of Miner’s accumulated damage concept for monopile-dense soil systems were investigated. The accumulated rotations for three load patterns were calculated, considering the effect of load sequence based on Miner’s damage concept. Figure 16 shows rotation envelopes from three loading regimes operated with 1000 cycles of type A, 100 cycles of type B, and 10 cycles of type C, respectively and inversely. The resulting accumulated pile rotations differed by $\Delta \theta = 0.33^\circ$. Denser sands had a higher sensitivity to the load sequence, as was reported in the cyclic triaxial tests of Shen et al. (1978). However, the dry sand analyses showed some minor scatter in the order of the load applications.
Stewart (1986) advocated the strain superposition theory for load cycles with different amplitudes. This theory states that strains accumulate through the strain accumulation curves developed for different load amplitudes and while maintaining the strains implemented in previous events. The accumulated rotation for load type A, $\Delta \theta_a$, may alternatively be obtained through the application of $(N_a)_\text{equivalent}$ load cycles of type B, through Eq. (7) (LeBlanc et al. 2010):

$$\Delta \theta_a = (\theta_T T_a) \times (N_a)_\text{equivalent}$$  \hspace{1cm} (7)

where $T_a = T_a(\zeta_a, R_a)$ and $T_r = T_r(\zeta_r)$ are dimensionless variables, and $R_r$ represents the soil relative density. If $N_a$ cycles of loading type A are applied to the monopile, followed by $N_b$ cycles of loading type B, the resulting rotation may be obtained by LeBlanc et al. (2010):

$$\theta_s = (\theta_T T_a) \times (N_b)_\text{equivalent} \times (N_a)^\nu + \max\{\theta_{\Delta a}, \theta_{\Delta b}\}$$  \hspace{1cm} (8)

Figure 17 shows the predicted monopile rotation in response to the load sequence 200A---200B. In general, satisfactory agreement was found between the theoretical solution and computed curves.

Effect of load reversal

Next, consider load pattern type A, defined by the corresponding $\zeta_a$. Another cyclic load, identified by $-\zeta$ with equivalent amplitude but in the opposite direction, was considered to analyze
the effect of load reversal on the accumulation of rotation in the soil-pile system. Figure 18 depicts
the pile rotation under the loading pattern of \((N \times A) \rightarrow (N \times -A) \rightarrow (x \times A)\), where \(x\) is the number
of cycles needed to diminish the effect of the loading reversal. The accumulated rotation is
eventually equal to the rotation prior to load reversal. The results showed that 2.31N cycles were
required to counteract the effects of load reversal on a monopile installed in dense sand, which is
higher than the 1.963N cycles found by Leblanc et al. (2010) for small-scale stiff piles in medium
dense sand. Therefore, the hypothesis that 1N cycles are required to neutralize this phenomenon
(i.e., subtraction of the number of reversed load cycles) underestimates the accumulated rotation.

**Fatigue analysis**

In the absence of a sufficient number of well-calibrated investigations, carefully conducted
numerical modeling offers a means for understanding the performance of non-slender piles
founded on dense sands. The model proposed by Lin and Liao (1999) suggests that the
accumulated rotation is proportional to \(\ln(N)\). This model was investigated in the previous section
for dry sand with \(N < 10,000\). A relatively good fit was observed, with a better fit when the
accumulated rotation was modeled as an exponential rather than a logarithmic expression.
In this section, this issue was further studied for the FLS. The results from fatigue analysis
showed a much better fit with the exponential expression \((R = 0.999)\) than with the logarithmic
expression \((R = 0.851)\). Constants \(a\) and \(b\) for the exponential expression were 0.0324 and 0.58,
respectively, and for the logarithmic expression were -2.78, and 0.851, respectively. Table 5
reports the load characteristics for a realistically designed wind turbine, including the FLS, SLS,
and the worst expected transient load equal to ULS/1.35. The results include \(~10^4\) cycles, whereas
FLS is governed by \(10^7\) load cycles. In the absence of further information, and due to the closeness
of the exponential fit up to $10^4$ cycles, care should be advised when extrapolating we may be able expression to the FLS.

LeBlanc et al. (2010) proposed the following expression based on small-scale tests:

$$\Delta \theta = \left( \left( \Delta \theta_{\text{avg}} \right)^\alpha + N \times (\theta_{T1}T_{e})^\alpha \right)^\alpha$$

$$\theta = \Delta \theta + \max (\theta_{1}, \ldots, \theta_{n})$$

(10)

The evolution of pile rotation due to cyclic loading in the above expressions was obtained by employing different assumptions, such as Miner’s rule, the strain superposition theory, and the extended rainflow-counting method (Rychlik 1987). For this research, in line with the non-dimensional frameworks given above, the load-time histories are decomposed into a set of load regimes relevant to wind turbines (Table 5).

The maximum accumulated rotation is determined as $\theta_p = 1.917^\circ$. Fatigue and serviceability limit states contribute to $\theta_p$ by 46% and 20% respectively, rather than 31.82% caused by the worst expected load. From this point, the high-level cycling, even though in very few cycles gave a scope for accelerated higher evolution of accumulation of ed rotation.

Figure 19 shows the lateral deflection lines from two of two model monopiles. The monopile with an embedded length of 30 m behaved like a flexible pile, whereas the short monopile with a length of 20 m showed a rigid response. A significant head-and-toe displacement with increasing load cycles was observed, with a slight downward movement of the rotation point of the pile. Greatest displacement was observed during the first cycle, and the accumulated cyclic displacement increased with the number of cycles. This figure sheds light on some complicated features of the plastic shakedown response of the pile.

Subgrade reaction
The developed model is capable of simulating the partially drained loading condition for Cyclic loading: transient effect

A large-diameter monopile is intended to maintain the serviceability of offshore platforms over several years. However, a monopile with an unfavorable drainage system can lead to the accumulation of PWP, followed by pile displacements. The combined loading of a foundation is a fundamental problem in offshore wind farms, which experience harsh environmental conditions (Barari and Ibsen 2012). There can be significant changes in the induced loads when waves pass a structure; therefore, transient effects need to be considered.

The PWP pattern changes dramatically with the loading rate, although the loading test frameworks in literature are limited to very poorly drained conditions for small-scale soil-pile models. The change in PWP, \( \Delta \rho \), can be defined as follows (Cuéllar 2011):

\[
\Delta \rho = f \left( \frac{1}{k}, \frac{1}{L_d}, \gamma_w \right)
\]

where \( \tau_L \) is the loading period, \( k \) is the soil permeability, \( L_d \) is the drainage length, and \( \gamma_w \) is the unit weight of fluid. Although \( f \) is not explicit, it is evident that an increase in monopile size, when \( \tau_L \) and \( k \) are constant, will lead to greater PWP development. When the period is large, there is very little tendency for PWP development. The developed finite element model was utilized to examine the effects of load repetition, loading rate, and loading history under extreme loading events (Fig. 22(a)). The loading rate had a significant effect on the lateral response of the foundation. Typical responses to such a load history are shown in Figs. 22(b) and 22(c).

To identify the ground displacement pattern at the soil-pile interface, two cases were defined: the pore fluid behavior in soil adjacent to the monopile shaft (case I), and the free-field behavior at a point approximately midway between the pile and the model boundary (case II).
interface exhibits significant displacements because of the complex effects of deviatoric soil
deformation (due to SSI-induced cyclic loading near the foundation edges) and volumetric effects
resulting from dynamic loads. However, there is still insufficient knowledge regarding the
mechanisms of foundation-induced dilation in engineering practice, which may lead designers to
erroneous decisions.

Under the given level of excitation, a soil compaction mechanism dominated in the free-
field, resulting in steadily increasing PWP over ~ 146 seconds (Fig. 23(b)). Shear stress
components were imposed due to transient excitation, causing the cycle-by-cycle accumulation of
lateral deformations. The computed excess PWP histories (Fig. 23) displayed several instantaneous
sharp drops in PWP.

During undrained loading, a zone formed around the pile, in which the PWPs were
considerably different from those in the free-field (Figs. 23 and 24). This finding can be attributed
to the foundation-induced dilation effect during cyclic loading. Observations from numerical
analyses showed that the dilation phenomenon caused by the soil-pile motion on the lateral loading
of the monopile head was confined within a zone of approximately up to two diameters in the
vicinity of the pile. With the development of 3D transient hydraulic gradients, it is anticipated that
partial drainage, PWP, and void redistribution may occur simultaneously. Partial drainage may
occur with excess PWP development, in response to transient hydraulic gradients, and as rapidly
as the 3D PWP redistribution occurs (Fig. 24). As a result of the horizontal flow towards the free-
field, the excess PWP will be dissipated downward.

Shear stress components are imposed by transient effects due to accumulated lateral
deformations (Fig. 25). The excess PWP generation rate and soil softening are highly dependent
on the confining pressure and foundation-induced shear stresses. Figure 26 shows the psy
responses at three different depths when the cyclic displacement amplitude successively increased.

Storms caused transient and permanent deformations of the foundation and surrounding soil settlements (Fig. 27). The few centimeters of soil settlement indicated that storms, at least of a certain duration, might cause a transient softening of the foundation, until the excess PWP dissipates. Importantly, these a few centimeters pile head deformations were induced in only 146 seconds. For storms with a return period during the lifetime of a wind turbine, the accumulated deformations may exceed tens of centimeters, which can interrupt the turbine serviceability.

Finally, the SSI-induced settlements were quite similar in shape within the soil layers. The observed re-stiffening behavior towards the end of the motion was mostly due to the vertical downward water flow from the surrounding soil adjacent to the pile shaft toward the free-field (Fig. 24). Settlements in the free-field may be mainly attributed to settlements within the upper layers of saturated sand. Interestingly, the downward flow away from the upper layers made late displacements within the lower layers.

Conclusions

This paper presents results from the numerical modeling of the lateral long-term cyclic loading of large-diameter monopiles due to wind and tidal waves. A well-calibrated critical state multi-surface plasticity model was employed for modeling the accumulated deformations of offshore piles in dense sand associated with cyclic mobility. This model can be useful for qualitative analysis during the design of foundations for offshore wind turbines, as well as for analyzing how the soil-pile interaction affects the overall response of the system.

Loading conditions corresponded to high lateral loads and bending moments, which could be induced by environmental events, especially storms. When evaluating the cyclic loading, the accumulated rotation was normalized to the rotation obtained in the first loading cycle. For one-
way cyclic loading, an attenuation mechanism was observed for different cyclic load ratios, wherein the plastic increments still occurred after a certain number of load cycles and at a decreasing rate to zero, as, for instance, in a logarithmic evolution with the number of cycles. On the other hand, no load level led to an elastic response; therefore, for the dense sand, an attenuating evolution without pure shakedown can generally be expected.

The new expressions, which are based on full-scale behavior, are more specific than previously reported relationships because they incorporate additional terms that reflect the soil characteristics during cyclic behavior. Additionally, it was shown that excess PWP may accumulate around the pile, depending on the drainage conditions, system geometry, soil permeability, and load frequency.

References


Commented [MR28]: Could talk about the transient/storm analysis here — seemed very interesting to me??


**Figures Captions**

Fig. 1. Frequencies distribution for a fully operational Vestas 4.5 MW wind turbine (LeBlanc 2009)
Fig. 2. Typical FE mesh of the developed numerical model: schematic illustration in dense sand subjected to the one-way cyclic and transient loading: (a) typical load time histories showing sinusoidal loading and an extreme event (b) plan view (c) side view (d) three dimensional view of Finite Element mesh

Fig.3. Load time histories of the tests P32 and P344 (Giannakos et al. 2012)

Fig. 4. Experimental and computed force-displacement curves at pile head for tests (a) P32 and (b) P344

Fig. 5. Failure envelope obtained from bearing capacities obtained at different values of h for large-diameter monopile (L=30 m)

Fig. 6. Contours of plastic strain magnitude (plotted as deformed mesh) at selected loading levels (as a fraction of the total applied load) for a shear force monotonically applied at the head of the monopile (a 57000 kN shear force monotonically applied at the head of monopile: h = 30 m). a) H= 0.175 Hult, b) H= 0.33 Hult, c) H= 0.7 Hult

Fig. 7. Computed (a) bending moment b) shear force c) soil reaction distribution for virgin loading of monopile at 10000 kN, 5780 kN, 2544 kN

Fig.8. Model verification: proof of the existence of a cutoff period for radiation damping. Harmonic excitation at the top of the monopile. The response for two vibration periods (a) \(T/T_{soil}\); (b) \(T/T_{soil}\) and two model sizes (FE model length=50 and 100 m) is analyzed.

Fig.9. Cyclic parameters (Roesen et al. 2013; LeBlanc et al. 2009)

Fig. 10. Moment-rotation relationships of the static reference analysis and the four cyclic loads (D=7.5 m, L = 30 m, h=30 m)

Fig.11. Rotation of the monopile at soil surface as a function of the number of cycles in the FE simulations with \(\xi_b = 0.18, 0.25\) and 0.47. Maximum values of rotation are indicated by the dashed lines.

Fig.12. Rotation of the monopile at soil surface as a function of the number of cycles in the FE simulations with \(\xi_b = 0.18\) and 0.25. Maximum values of rotation are indicated by the dashed lines.

Fig. 13. Normalised accumulated rotation as a function of the number of cycles for the three amplitudes of cyclic loadings for dry sand

Fig. 14. Normalised accumulated rotation as a function of the number of cycles for the three amplitudes of cyclic loadings for saturated sand
Fig. 15. Fitted empirical constant $T_b$ as a function of the loading magnitude $\zeta_b$ in the three cyclic analyses for dry sand.

Fig. 16. Envelope of monopile rotation computed for an increasing load sequence. a) 1000A........100 B........10C (Dry) b) 10C........100 B........1000A (dry)

Fig. 17. Prediction of accumulated rotation using superposition concept.

Fig. 18. Effect of the reversed loading on the behavior of monopile rotational deformations:

$$(\zeta_b = 0.18) \rightarrow (\zeta_b = -0.18) \rightarrow (\zeta_b = 0.18)$$

Fig. 19. a) Pile deflection lines under cyclic loading b) Relative increase of the lateral pile displacement at ground level.

Fig. 20. Evolution of p-y curves derived from numerical analysis at the depth of 7.5 m ($\zeta_b = 0.25$)

Fig. 21. Time histories of excess pore pressure generation for ($\zeta_b = 0.25$) at different depths in free field.

Fig. 22. (a) Loading time history, (b) corresponding displacement response, and (c) the load displacement behavior showing increasing hysteresis for large cycles.

Fig. 23. Representative excess pore water pressure-time histories at various depth. a) soil-pile interface b) interface.

Fig. 24. Three dimensional hydraulic gradients at different instants.

Fig. 25. (a) Computed shear stress histories (b) Computed effective stress path.

Fig. 26. a) Bending moment for soil profile. b) p-y response at $z=7.5$ m c) p-y response at $z=15$ m d) p-y response at $z=22.5$ m.
### Table 1. Pile characteristics

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Symbol</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total length</td>
<td>$L_s$</td>
<td>20, 30, 40, 60 m</td>
</tr>
<tr>
<td>Embedded depth</td>
<td>$L$</td>
<td>20, 30 m</td>
</tr>
<tr>
<td>Outer diameter</td>
<td>$D$</td>
<td>7.5 m</td>
</tr>
<tr>
<td>Pile wall thickness</td>
<td>$t_p$</td>
<td>0.09 m</td>
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<tr>
<td>Equivalent diameter</td>
<td>$D_e$</td>
<td>4.1 m</td>
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<tr>
<td>Young’s modulus</td>
<td>$E$</td>
<td>$2.1 \times 10^8$ kPa</td>
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<tr>
<td>Moment of inertia Young’s modulus</td>
<td>$I$</td>
<td>14.84 m$^4$</td>
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<tr>
<td>Bending stiffness</td>
<td>$EI$</td>
<td>$3.12 \times 10^9$ kNm$^2$</td>
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### Table 2. Material parameters used for dense sand

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Dense sand</th>
</tr>
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<tbody>
<tr>
<td>Low strain shear modulus, $G_s$ (kPa)</td>
<td>60000</td>
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<tr>
<td>Friction angle, $\phi$ (Degree)</td>
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<td>Liquefaction yield strain, $\gamma_s$</td>
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<td>Contraction parameter, $c_1$</td>
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<tr>
<td>Dilation parameter, $d_1$</td>
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</tr>
<tr>
<td>Dilation parameter, $d_2$</td>
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</tr>
<tr>
<td>PT angle, $\phi_{PT}$ (Degree)</td>
<td>30</td>
</tr>
<tr>
<td>Mass density, $\rho$ (kg/m$^3$)</td>
<td>1700</td>
</tr>
<tr>
<td>Permeability coefficient, $k$ (m/s)</td>
<td>$6.6 \times 10^{-5}$</td>
</tr>
</tbody>
</table>
Table 3. Loading characteristics

<table>
<thead>
<tr>
<th>Load regime</th>
<th>$\xi_0$</th>
<th>$\xi_c$</th>
<th>$\theta_1$: rad</th>
<th>$\theta_t$: rad</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.18</td>
<td>0</td>
<td>1.89 $\times 10^{-3}$</td>
<td>1.45 $\times 10^{-3}$</td>
</tr>
<tr>
<td>B</td>
<td>0.25</td>
<td>0</td>
<td>4.29 $\times 10^{-3}$</td>
<td>3.24 $\times 10^{-3}$</td>
</tr>
<tr>
<td>C</td>
<td>0.47</td>
<td>0</td>
<td>5.1 $\times 10^{-3}$</td>
<td>4.65 $\times 10^{-3}$</td>
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</table>

Table 4. Pearson’s correlation coefficient, $R$, between the fitted and computed results for the accumulated rotation

<table>
<thead>
<tr>
<th>$\xi_0$ (Sand type)</th>
<th>No. of Cycles</th>
<th>Power fit (Eq. 4a)</th>
<th>Logarithmic fit (Eq. 4b)</th>
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<tr>
<td></td>
<td>$a$</td>
<td>$b$</td>
<td>$R$</td>
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<tr>
<td>0.18 (Dry)</td>
<td>1000</td>
<td>0.05</td>
<td>0.474</td>
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<tr>
<td>0.25 (Dry)</td>
<td>1000</td>
<td>0.045</td>
<td>0.528</td>
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<tr>
<td>0.47 (Dry)</td>
<td>1000</td>
<td>0.064</td>
<td>0.492</td>
</tr>
<tr>
<td>0.18 (Saturated)</td>
<td>500</td>
<td>0.157</td>
<td>0.450</td>
</tr>
<tr>
<td>0.25 (Saturated)</td>
<td>500</td>
<td>0.185</td>
<td>0.452</td>
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Table 5. Prediction of the cumulative rotation based on the numerical analysis of a full-scale offshore monopile

<table>
<thead>
<tr>
<th>Load type</th>
<th>$N$</th>
<th>$\theta_1$ [degree]</th>
<th>$T_s$</th>
<th>$T_c$</th>
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<tbody>
<tr>
<td>FLS</td>
<td>$10^7$</td>
<td>0.1155</td>
<td>0.0324</td>
<td>1</td>
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<tr>
<td>SLS</td>
<td>100</td>
<td>0.237</td>
<td>0.0648</td>
<td>1</td>
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<tr>
<td>WETL</td>
<td>-</td>
<td>0.61</td>
<td>-</td>
<td>-</td>
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</table>

Accumulated rotation obtained by Eq. (10)

<table>
<thead>
<tr>
<th>Load type</th>
<th>$I$</th>
<th>$\Delta \theta$</th>
<th>$\max(\theta_{i,1},...,\theta_{i,j})$</th>
<th>$\theta_i$</th>
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</thead>
<tbody>
<tr>
<td>FLS</td>
<td>0</td>
<td>0.783</td>
<td>0.1155</td>
<td>0.89</td>
</tr>
<tr>
<td>SLS</td>
<td>1</td>
<td>0.79</td>
<td>0.237</td>
<td>1.027</td>
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<td>WETL</td>
<td>2</td>
<td>----</td>
<td>0.61</td>
<td>$\theta_p = 1.917$</td>
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Accumulated rotation for continuous cyclic loading obtained by Eq. (4)

<table>
<thead>
<tr>
<th>Load type</th>
<th>$I$</th>
<th>$\Delta \theta = \left(\theta T_s T_c\right) \times N^{a}$</th>
<th>$\theta = \theta_i + \Delta \theta$</th>
<th>$\theta_i$</th>
</tr>
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<tbody>
<tr>
<td>FLS</td>
<td>0</td>
<td>0.783</td>
<td>0.899</td>
<td>46%</td>
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<tr>
<td>SLS</td>
<td>1</td>
<td>0.148</td>
<td>0.385</td>
<td>20%</td>
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<tr>
<td>WETL</td>
<td>2</td>
<td>----</td>
<td>0.61</td>
<td>31.82%</td>
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