EFFECT OF STIFFNESS DEGRADATION OF CLAY IN THE DYNAMIC RESPONSE OF MONOPILE SUPPORTED OFFSHORE WIND TURBINES

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ABSTRACT: Stiffness degradation studies for monopile supported offshore wind turbines (OWTs) are usually limited to the geotechnical domain, largely ignoring the dynamic loads from the wind and the waves. This paper makes use of a time domain approach, coupling aerodynamic and hydrodynamic loads, to investigate the influence of stiffness degradation in clay on the response of a monopile supported OWT in a water depth of 20 m. *p-y* curves are used to represent the soil-structure-interaction (SSI) in the lateral direction and a suitable degradation method is applied to consider the effects of cyclic loading. It is observed that the influence of stiffness degradation wanes with increasing number of load cycles. OWT's being highly dynamic structures; the debilitating effects of stiffness degradation cannot be entirely discounted.

1 INTRODUCTION

Offshore wind is fast emerging as a source of clean energy, capable of addressing the issue of depleting fossil fuel reserves. Majority of the OWT structures have been installed in shallow waters (depth < 30 m) and are supported on monopiles. They are large diameter steel pipe piles of diameters varying from 4 to 6m. With about 50% of the total cost of an offshore wind farm spent on the foundations, it becomes essential to study the SSI problem in detail.

The design philosophy for offshore structures like oil platforms cannot be directly applied to offshore wind turbines. They are vulnerable to resonance from wind, waves and rotor effects and have to be designed to fall within a narrow safe frequency zone. There exists no specific design procedure for the foundations of monopile supported OWTs. The widely used *p-y* curves suffer from the drawback that they have been experimentally validated only for smaller diameters. While small diameter piles show flexible behavior, monopiles are of large diameters and exhibit rigid body rotation. Latest design standards for OWTs (DNV OSJ101, 2014) specify that *p-y* curves may be used with caution, for large diameter monopiles.

There have been a few studies on cyclic loading and stiffness degradation effects on monopiles. Achmus*et al.* (2009) proposed a degradation model for cyclic loading in sand. From experiments in sand, LeBlanc *et al.* (2010)concluded that cyclic loading increased the pile stiffness. Carswell*et al.* (2015) compared the effect of different *p-y* methods for monopile in clay. Most of these works were limited to the soil domain.

This paper attempts to investigate the influence of stiffness degradation of clayey soil under cyclic wind and wave loads, on the response of a monopile supported OWT, within a time-domain framework. Aerodynamic, hydrodynamic and geotechnical domains were considered and the finite element (FE) method was used. A stiffness degradation scheme by Rajashree and Sundaravadivelu (1996), has been applied on *p-y* curves for clay by Matlock (1970). As the serviceability criteria are governing, for OWTs, the results are presented in terms of displacements and rotations at the head of the monopile.

2 FINITE ELEMENT MODEL

2.1 NREL 5-MW Offshore Wind Turbine

The present study makes use of the NREL 5-MW OWT (Jonkman*et al.*, 2009). It is a 3-bladed horizontal axis wind turbine with an upwind rotor configuration. The hub and rotor diameters are 3 m and 126 m respectively.The tower extends from 10 m above the mean sea level (MSL) to 87.6 m above MSL. The diameter and wall thickness of the tower varies from 6 m and 27 mm at the base, to 3.87 m and 19 mm at the top. The pile has a diameter of 6 m and wall thickness of 60 mm. The water depth is 20 m.

2.2 Soil Conditions

A uniform clay soil profile is used in the study. The effective unit weight (γ') and undrained shear strength (c_u) are 8 kN/m^3 and 100 kPa respectively. The experimental coefficient (J) has a value of 0.25 and the strain at 50% max. stress ε_{50} is 0.005.

2.3 Load Cases

Two load cases are considered – LC1, is an operating case at the rated wind speed of 11.4 m/s and LC2 is an extreme load case, with a wind speed of 39 m/s. They correspond to the design load cases (DLC) 1.2 and 6.4 of IEC (2009) respectively. Sea-states are correlated on wind speeds and are derived using a probabilistic formulation (Johannessen*et al.*, 2002). The met-ocean conditions are given in Table 1. Wind and waves are considered to be co-directional.

Table 1 Load casesused in the study

LC No.	V (m/s)	Hs (m)	Tp (s)
1	11.4	3.1	10.1
2	39	9.5	12.8

Here, V is the hub-height 10-min mean wind speed, $H_s$is the significant wave height and T_p is the peak spectral period.

2.4 Loads on the OWT

The OWT is subjected to dynamic loading, from wind and waves.For fixed OWT's the coupling effect between wind and wave loading can be neglected (Gao*et al.*, 2010).A superposition method is adopted to model their combined influence, usingtwo computer codes. Initially, coupled aerodynamic-hydrodynamic analysis is performed using the aeroelastic program FAST (Jonkman and Buhl Jr., 2005) and six-component time series of the loads acting at the hub are obtained. They are now applied as external forces on the monopile model in USFOS (Søreide*et al.*, 1993), a FE program for offshore structures, and coupled hydrodynamic-geotechnical analyses are carried out.

NREL's TurbSim (Jonkman, 2009) is used to generate 3-D stochastic wind fields. The frequency content of the wind velocity is described using the Kaimal spectrum. Aerodynamic forces on the blades are computed by means of the blade element momentum theory. Here, the total load on the blade is obtained by summing up the component loads on the individual blade elements long its span. Ocean waves are irregular and their surface profile is generated from the JONSWAP spectrum, using an equal area method. The wave loads on the monopile are computed using the Morison equation (Morison *et al.*, 1950), as the sum of inertia and drag forces.

$$f = \rho C_M \frac{\pi D^2}{4}\dot{u} + \frac{1}{2}\rho C_D |u|u \quad (1)$$

Here, *f*is the horizontal force per unit length, *D* represents the diameter of the pile and *u* stands for the relative water particle velocity in the horizontal direction. C_M and $C_D$are the empirical coefficients for inertia and drag, respectively and ρ is the density of water. The upper dot represents the time derivative.

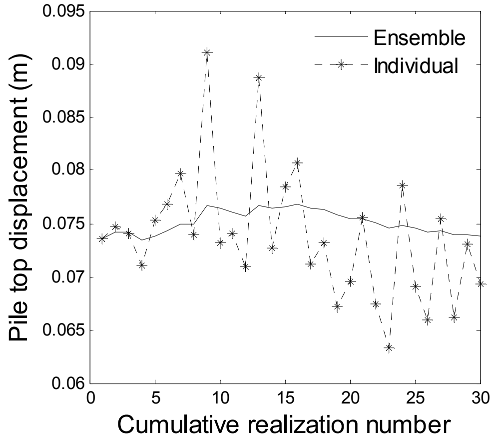

Figure 1.Eliminating uncertainty effects

Random phases are used tosimulate the time series of wind and waves result in statistical uncertainty. Varying the random phases would result in the generation of different time series for the same met-ocean state. Such an uncertainty can be eliminated by increasing the number of simulations for the same load case. This is illustrated in Figure 1. The pile top displacementat mudline under LC2 is studied for 30 individual realizations of the load process. It is observed that the use of individual estimates results in significant variations of the response. However, ensemble averaging is a reliable method for computing the response, with the values converging after about 25 realizations. In the present study, 30 different simulations are performed for each load case and the ensemble mean values are reported.

3 MODELLING

3.1 Structural Model

USFOS makes use of the Idealized Structural Unit Method (Yukio and Rashed, 1984), where the structure is discretized into actual physical units. The Hilber, Hughes and Taylor (HHT)-α method (Hilber*et al.*, 1977) is used for numerical time integration.Each simulation is run for 600 s.

3.2 Soil Model

The soil spring properties in the lateral direction are modelled using the static *p-y* curves by Matlock (1970). Here, *p* is the soil resistance and *y* is the pile deflection at a particular depth (*x*) below the mudline. The static p-y curves are developed as follows:

$$p = 0.5 p_{ult}\left(\frac{y}{y_c}\right)^{\frac{1}{3}} \quad (2)$$

where $y_c = 2.5\epsilon_{50}b$. $p$remains constant beyond a value of $y = 8y_{50}$. 'b' is the diameter of the pile. $p_{ult}$is the ultimate soil resistance per unit length of the pile, taken as the smaller of the following:

$$p_{ult} = \left(3 + \frac{\gamma'}{c_u}x + \frac{J}{b}x\right)c_u b;\ p_{ult} = 9c_u b \qquad (3)$$

Stiffness degradation is implemented using the model by Rajashree and Sundaravadivelu (1996). The ultimate soil resistance is degraded based on the number of cycles *N*, as follows:

$$p_{ultN} = (1 - \lambda_N)p_{ult} \qquad (4)$$

The degradation factor λ_N is given by:

$$\lambda_N = \frac{y_1}{0.2b} logN \leq 1 \qquad (5)$$

y_1 is the static displacement of the soil spring and is assumed as 1% of the pile diameter, in the present study (Carswell*et al.*, 2015). Spring elements are inserted along the depth of the pile, at a spacing of 3 m. The soil stiffness is represented by the initial slope of the *p-y* curve. Figure 2 shows the staticand the degraded *p-y*curves for a depth of 1.5 m.

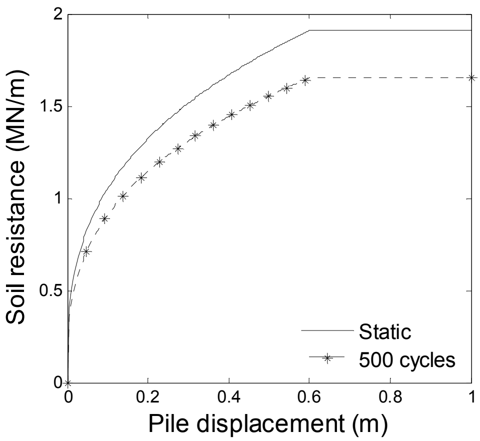

Figure 2.*p-y* curves for static and cyclic cases

4 RESULTS

Figures 3 and 4show the variation in the pile head displacement and rotation, with respect to the increasing number of load cycles. '0' number of cycles refers to the initial case, where stiffness degradation is disregarded. At high wind speeds, the power production is suspended and the blades are parked. However, due to increased wave height and loads, the response of the monopile at LC2 is significantly higher than that at LC1. The influence of stiffness degradation reduces the resistance of the *p-y* curves and results in increased responses.

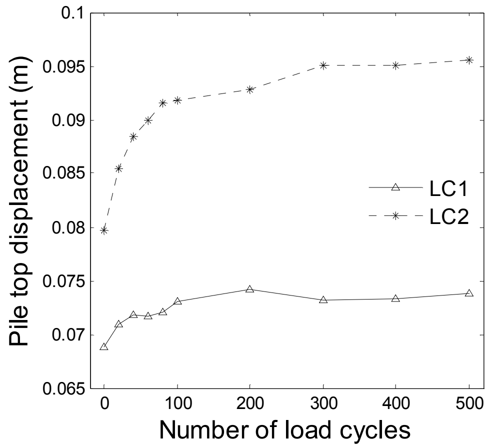

Figure 3.Variation of pile head displacement

The effect of stiffness degradation is predominant only during the initial stages (for load cycles < 100 in number). With further increase in the number of load cycles, the percentage reduction in the strength of the soil curves is reduced and beyond 300 cycles, the response levels off, as seen in Figures 3and 4. The pile top rotation is safe with respect to the serviceability criteria of 0.5° (Achmus*et al.*, 2009).

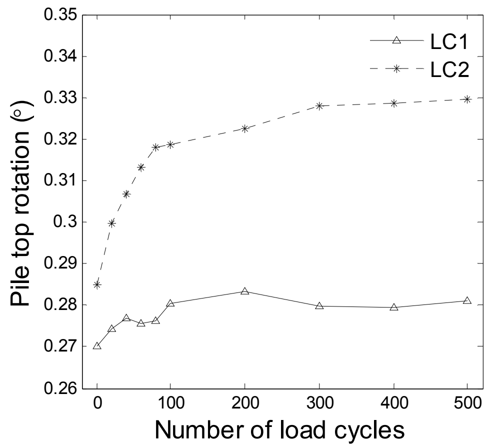

Figure 4.Variation of pile head rotation

Figure 5 shows the profile of maximum displacement, along the depth of the pile, for both the static case and after 500 load cycles. SSI for large diameter monopiles in clay shows flexible behaviour with a 'toe-kick' phenomenon, whereas the toe of the pile rotates in the negative direction. The point of inflection is observed at a depth of 22 m below the mudline. The pile in the degraded soil exhibits displacements with a percentage increase of 7% and 13% at the top and toe respectively, over the static case.

OWT foundations are usually designed for 50-year load cases and are rarely subjected to even 100 load cycles. However, stiffness degradation studies are aimed at understanding the performance of these highly dynamic structures under cyclic loads, in a bid to ensure that the safe natural frequency criteria or the serviceability criteria are not violated.

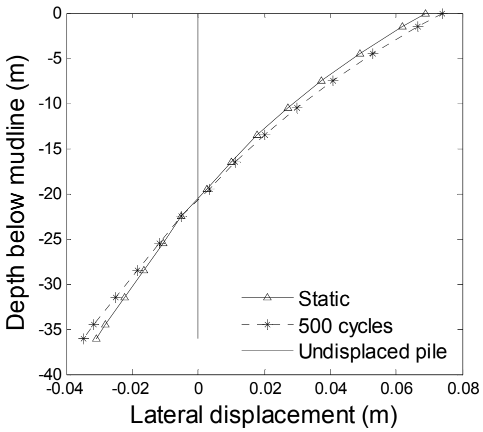

Figure 5. Pile displacement profile

5 CONCLUSIONS

The influence of stiffness degradation of clay on the lateral response of a monopile OWT in a water depth of 20 m has been investigated by means of the finite element method. Aerodynamic, hydrodynamic and geotechnical domains are considered together. A method proposed by Rajashree and Sundaravadivelu (1996) have been used to account for the reduction in soil strength due to cyclic loading.

The influence of stiffness degradation in increasing the lateral response of the pile top is visible during the initial stages (< 100 cycles), but tails off with further increase in the number of cycles. Large diameter monopiles in clay exhibit flexible behavior. While the limit state criteria with respect to pile top rotation is satisfied even at heavy sea-states, such studies may still serve as a safety check for monopiles designed using the *p-y* method.

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