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Observations on the influence of soil profile on the seismic kinematic bending moments of offshore wind turbine monopiles

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Abstract

The installed capacity for generation of electric energy from offshore wind power is growing exponentially. At the end of 2015, there were 3230 offshore wind turbines in Europe, 80% of which are founded on monopiles. The expansion in the number of offshore wind turbines may involve the need of installing new wind farms in sites of poorer soil conditions and increasing seismic risk, which puts additional responsibility on the foundation subsystem, which takes a significant part of the total initial investment when setting up new wind farms. These reasons justify the need for analyzing the influence of soil profile on the seismic kinematic bending moment of offshore wind turbine monopiles. To do this, a Beam-On-Dynamic Winkler approach is used to perform a parametric analysis of the system considering large diameter monopiles, realistic material and geometrical properties for soils and pile, and a large set of soil profiles assumed to be composed by several different strata. Each case is assumed to be subject to ground motions described by the elastic response spectrum given by Eurocode 8 (part I) for the ground type corresponding to each profile. Results are presented in terms of envelopes of bending moments along the monopiles, and the large number of results is synthesized in a set of ready-to-use graphs. It will be shown that, for the monopiles studied (with diameters of 3.5 and 6 meters), the peak kinematic bending moments are not necessarily found in the interfaces between strata, as observed in the case of piles with not so large diameters.

Keywords: Monopiles; Kinematic Bending Moments; Seismic Response; Offshore Wind Turbines; Winkler.

1. Introduction

With the number of installed wind turbines growing exponentially all around the world, there appears the need for founding the structures in regions with softer soils, higher seismic risk and, in the case of offshore wind turbines, greater depths. These facts increase the difficulty and responsibility of the design and detailing phases of the foundation, even more if one takes into account the high costs associated with it, specially in offshore parks [1,2]. On the other hand, the growing size and power of the new wind turbines implies that the impact of the failure or collapse of a unit is much more important, which leads to the need of reducing the risk of such failure.

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In Europe there exist regions of high seismic risk, as Greece or Italy, together with other regions of moderate risk. However, the main guides and standards [3–5] focus mainly on the loads generated by wind and, if applicable, waves, currents or ice. It is true that, on the one hand, they also consider earthquake actions, being the study of the seismic loads mandatory only in regions with a reasonable seismic risk and, on the other hand, that most of the existing wind turbines have been installed in zones with low seismic riks. Furthermore, there seems to exist the certitude that wind turbines are structures intrinsically safe under seismic loads [6], due partly to its large fundamental period in comparison to the frequency content of most earthquakes [7], which contributes significantly to the good seismic behavior of these structures and the existence of a very low number of registered failures of seismic origin. In this sense, it is also worth mentioning that, not only most wind turbines are installed in regions of low seismic risk, but also that wind turbines in seismically active zones are relatively recent, which shows that the experience in this field is limited.

For all these reasons, and taking into account that kinematic response may dominate the response of large diameter foundations in soft soils [8] and that soil profile influences very importantly the kinematic response of piles [9,10], this study aims at contributing to the knowledge of the seismic response of wind turbine foundations by analysing the influence of soil profile on the seismic kinematic bending moment of offshore wind turbine monopiles. To do this, a parametric analysis of the system considering large diameter monopiles, realistic material and geometrical properties for soils and pile, and a large set of soil profiles assumed to be composed by several different strata, has been performed. Each case is assumed to be subject to ground motions described by the elastic response spectrum given by Eurocode 8 (part I) [11] for the ground type corresponding to each profile. As a first approach, and considering the size of the set of cases defined for the analysis, a harmonic Beam-On-Dynamic Winkler model is used because of its high computational efficiency. Results will be presented in terms of envelopes of kinematic seismic bending moments for specific cases, and a synthesis of the large set of results.

2. Methodology

The influence of soil profile on the seismic kinematic bending moments of offshore wind turbine monopiles is studied in this paper by analysing the envelopes of bending moments arising in piles embedded in a set of different soil profiles and subjected to ground motions produced by vertically-incident SH waves. The computation of the time history of the internal forces along the piles is performed through the frequency domain method of response analysis [12] after obtaining the corresponding Frequency Response Functions (FRF). To do so, a harmonic Beam-On-Dynamic Winkler model [13,14], in which the pile is modelled as a linear-elastic Euler-Bernoulli beam, and the soil as a series of independent horizontal viscoelastic strata, is used due to its high computational efficiency. The soil horizontal stiffness and damping functions along the pile shaft are defined by the expressions provided by Novak et al. [15], while pile tip reactions are modelled using the horizontal and rocking impedance functions for superficial foundations on homogeneous half-spaces provided by Velestos and Verbič [16]. The resulting equation is solved analytically for each soil strata, and the solution is obtained after coupling all pile sections and applying boundary conditions.

As said before, the system is assumed to be subjected to vertically-incident SH seismic waves producing ground surface earthquake motions compatible with the corresponding Eurocode 8 [11] type 1 elastic response spectrum for 5% damping. The magnitude of the design ground motion in terms of reference peak ground acceleration is $a_g = 0.25$ g. Three different artificial earthquake are used for each configuration.

3. Results

3.1. Problem definition

The problem under consideration is depicted in figure 1. Being the aim of this study the analysis of the influence of the soil profile only on the kinematic bending moments along monopiles, the model used does not include the inertial response produced by the vibration of the wind turbine structure itself. Therefore, only pile and soil are considered, and two limit boundary conditions will be assumed at the pile head: *a)* free head, with zero shear force and bending moment at the top ($V_{head} = 0$ and $M_{head} = 0$) and *b)* zero-rotation head, with zero shear force and rotation at the top

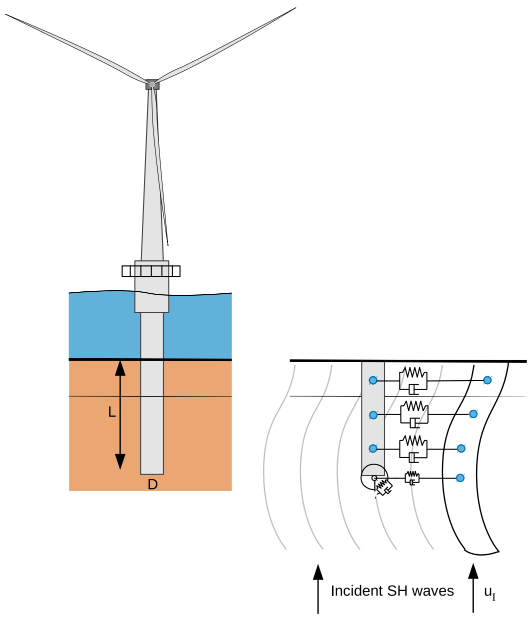

Fig. 1. Problem depiction (left) and illustration of the BDWM foundation model used in the analysis (right).

Table 1. Pile configurations.

Configuration	L (m)	D (m)	t (mm)	L/D	$\delta = D/D_{int}$
1	10.5	3.5	41.37	3	0.97363
2	24.5	3.5	41.37	7	0.97363
3	18.0	6.0	66.37	3	0.97788
4	42.0	6.0	66.37	7	0.97788

($V_{head} = 0$ and $\theta_{head} = 0$). Four different steel pipe pile configurations, as described in table 1 are considered, being L, D, D_{int} and t the embedded length, external diameter, internal diameter and steel thickness. Steel Young's modulus, density and Poisson's ratio of $E = 210\,\text{GPa}$, $\rho = 7850\,\text{kg/m}^3$ and $\nu = 0.25$, respectively, have been assumed. On the other hand, table 2 defines the set of 28 different soil profiles that have been used for the study. The table presents a reference name for each profile followed by the average value of the propagation velocity of S waves in the upper 30 m of the soil profile, the corresponding ground type according to Eurocode 8 [11] classification, and the definition of the depth range h of each one of the individual strata composing each profile together with the corresponding shear wave velocity c_s, density ρ_s and shear modulus G_s. Poisson's ratio and hysteretic damping ratio in all soil strata are assumed to be 0.3 and 5%, respectively. This set tries to include many different kinds of profiles, from homogeneous half-spaces of type A, B, C and D (profiles P1 to P4) to profiles composed by 7 different layers along the length of the embedded pile.

3.2. Seismic kinematic bending moments

Figure 2 presents envelopes of maximum seismic kinematic bending moment for four selected soil profiles with different characteristics (P3, P6E, P8D and P11 according to table 2). For each profile, for different subplots are presented, one for each one of the pile configurations defined in table 1. For each configuration, a set of three different envelopes (corresponding to three different artificial compatible earthquake records) is presented for each boundary condition at the pile head, so that the influence of both aspects can be evaluated. When interfaces between soil layers are present in the soil profile, a grey horizontal line is also plotted at the corresponding depth.

Very significant peak kinematic bending moments are usually found in piles of small to moderate diameters at depths corresponding to the interfaces between soil layers [9,10]. However, it is found that this is not necessarily the

Table 2. Definition of soil profiles.

Profile	$c_{s,30}$ (m/s)	Ground Type	h (m)	c_s (m/s)	ρ_s (kg/m^3)	G_s (MPa)
P1	160	D	0-42	160	2000	51.20
P2	250	C	0-42	250	2000	125.0
P3	400	B	0-42	400	2000	320.0
P4	800	A	0-42	800	2500	1600.0
P5 A	93.33	D	0-5	70	1650	8.08
			5-42	100	1750	17.50
P5 B	113.75	D	0-5	70	1650	8.08
			5-42	130	2000	33.80
P5 C	131.77	D	0-5	70	1650	8.08
			5-42	160	2000	51.20
P5 D	175.00	D	0-5	70	1650	8.08
			5-42	250	2000	125.0
P5 E	224.00	C	0-5	70	1650	8.08
			5-42	400	2000	320.0
P5 F	292.14	E	0-5	70	1650	8.08
			5-42	800	2500	1600.0
P6 A	87.50	D	0-10	70	1650	8.08
			10-42	100	1750	17.50
P6 B	101.11	D	0-10	70	1650	8.08
			10-42	130	2000	33.80
P6 C	112.00	D	0-10	70	1650	8.08
			10-42	160	2000	51.20
P6 D	134.62	D	0-10	70	1650	8.08
			10-42	250	2000	125.0
P6 E	155.55	D	0-10	70	1650	8.08
			10-42	400	2000	320.0
P6 F	178.72	E	0-10	70	1650	8.08
			10-42	800	2500	1600.0
P7 A	154.07	D	0-5	130	2000	33.80
			5-42	160	2000	51.20
P7 B	216.66	C	0-5	130	2000	33.80
			5-42	250	2000	125.00
P7 C	297.14	C	0-5	130	2000	33.80
			5-42	400	2000	320.0
P7 D	430.34	B	0-5	130	2000	33.80
			5-42	800	2500	1600.0
P8 A	148.57	D	0-10	130	2000	33.80
			10-42	160	2000	51.20
P8 B	191.18	C	0-10	130	2000	33.80
			10-42	250	2000	125.00
P8 C	236.36	C	0-10	130	2000	33.80
			10-42	400	2000	320.0
P8 D	294.34	E	0-10	130	2000	33.80
			10-42	800	2500	1600.0
P9	140.54	D	0-5	130	2000	33.80
			5-10	100	1750	17.50
			10-42	160	2000	51.20
P10	200.00	C	0-5	160	2000	51.20
			5-10	130	2000	33.80
			10-42	250	2000	125.0
P11	201.82	E	0-5	70	1650	8.08
			5-10	130	2000	33.80
			10-15	250	2000	125.0
			15-42	800	2500	1600.0
P12	179.22	E	0-5	70	1650	8.08
			5-10	100	1750	17.50
			10-15	130	2000	33.80
			15-20	160	2000	51.20
			20-25	250	2000	125.0
			25-30	400	2000	320.0
			30-42	800	2500	1600.0

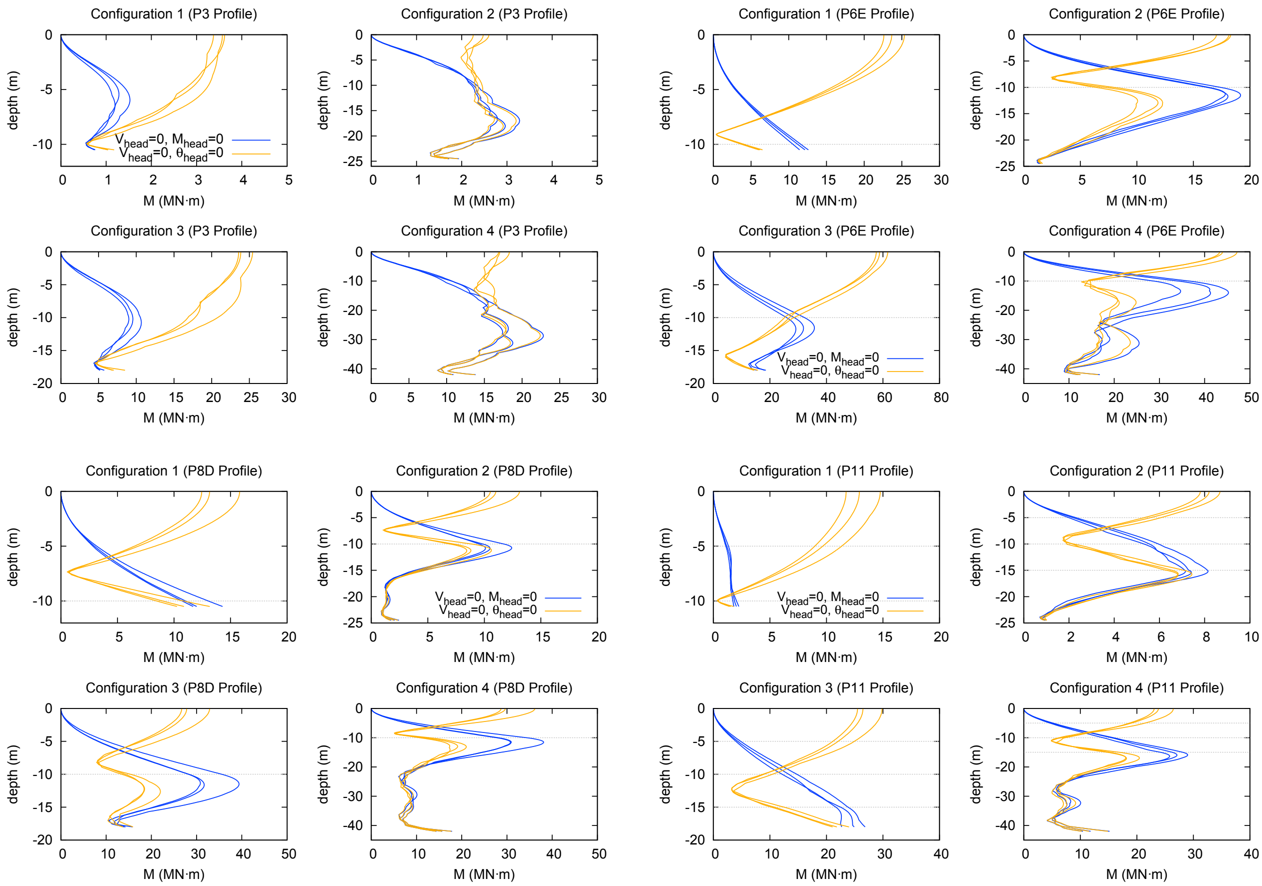

Fig. 2. Envelopes of seismic kinematic bending moments for different soil profiles

case in piles with large diameters. In fact, when kinematic bending moments at an interface are studied for increasing diameters (results not shown due to space constraints), the very significant peak values that appear for small diameters tend very clearly to lose importance when the large diameters used for wind turbine monopiles are reached. In the present cases, these peaks remain only when stiffness contrasts between consecutive thick layers are large enough (as in profiles P6E and P8D), appearing slightly below the interface and being the largest value under the assumption of free head. The assumption of zero rotation at the pile head yields, on the contrary, maximum bending moments not at the interface but at the pile head. For profiles like P11, with smaller stiffness contrast and thinner layers, peak bending moments are not observed at all at the interfaces. On the other hand, and as expected, the boundary condition at the pile head has a large influence on the envelope of maximum bending moments, mostly along the short piles (configurations 1 and 3) and near the pile head in all cases.

3.3. Maximum kinematic bending moments as a function of $c_{s,30}$

Figure 3 synthesizes the results from the parametric study defined above presenting peak seismic kinematic bending moments as a function of the average value of the propagation velocity of S waves in the upper 30 m of each soil profile. Each point represents the maximum bending moment along the pile for each one of the cases. Results for zero-rotation head and free head boundary conditions are presented at the left and right plots, respectively. In the first case, maximum seismic kinematic bending moments are found to be much larger in soft soils, while for the second case, the maximum values are obtained for $c_{s,30} \simeq 150$ m/s and always below the first interface (if it exists). In all cases, and as expected, the values grow with both pile diameter and length.

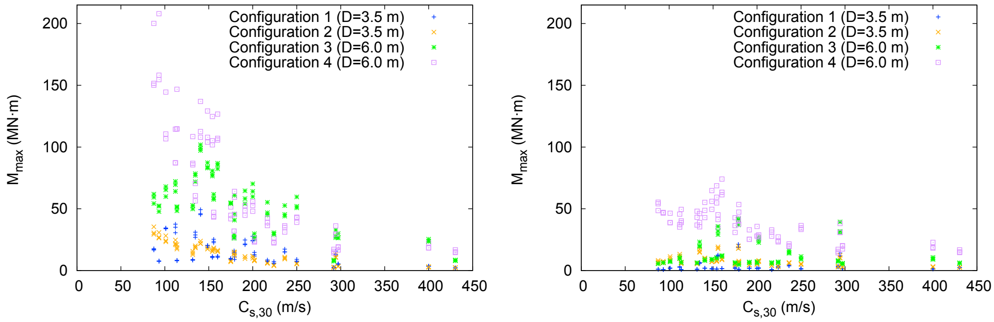

Fig. 3. Representation of the maximum seismic kinematic bending moments obtained for all cases studied at any depth along the pile for zero–rotation head (left) and free head (right) assumptions.

4. Conclusions

The influence of soil profile on the maximum kinematic bending moments of offshore wind turbine monopiles has been explored by performing a parametric analysis on a large set of cases representative of that structural typology. It has been found that, for these large diameters, peak values do not necessarily arise at layer interfaces, and that boundary conditions at the pile head show a strong influence on the response of the whole foundation. As kinematic response may dominate the system behaviour in these cases of piles with large diameters, further research is needed, and these values should now be added to those of normal operation and inertial response in case of earthquake. To this end, the response of the whole system, including both kinematic and inertial responses and, eventually, operational loads, should be investigated, as well as the magnitude of the influence of non-linear effects.

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