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Preface

Madrid, 20 de Junio de 2018,

La dinámica estructural es un campo de interés común y de importancia creciente en diversas especialidades de la ingeniería y de la ciencia. Mientras que en algunos campos como las máquinas o los vehículos de transporte ha sido siempre un elemento básico, en otros como la ingeniería civil y la arquitectura, más preocupados tradicionalmente con la estática, se ha convertido en un aspecto muy relevante.

Esta primera conferencia a nivel nacional pretende ser un foro en el que tengan cabida los trabajos de investigación, desarrollo y aplicaciones, permitiendo la discusión, difusión, contacto con otros grupos y establecimiento de colaboraciones. Se organiza con proyección internacional y europea, contando con el apoyo de la European Association for Structural Dynamics (EASD) organizadora de los congresos EURODYN, así como con el apoyo de la Sociedad Española de Métodos Numéricos (SEMNI).

La participación incluye tanto trabajos basados en métodos teóricos y computacionales como experimentales. Por otra parte abarca todos los campos de la dinámica estructural, como son la ingeniería mecánica, el transporte, ingeniería civil y arquitectura, ingeniería sísmica e ingeniería de materiales. Aunque ubicados en especialidades de ingeniería distintas todos estos campos comparten conceptos y métodos comunes de dinámica.

Esta primera conferencia pretende iniciar una serie que se desarrolle de forma periódica. Asimismo se propone constituir una Asociación Española de Dinámica Estructural que articule las actividades de colaboración y difusión, y que sirva de interlocutora con otros órganos nacionales e internacionales como la EASD.

Desde el comité organizador queremos dar la bienvenida a todos los participantes y ponernos a disposición para el desarrollo de la conferencia.

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SECTION 6: Seismic engineering, soil-structure dynamic interaction

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KINEMATIC BENDING MOMENTS IN OWT MONOPILES AS A FUNCTION OF THE GROUND TYPE

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Abstract. Offshore wind energy has already proven to be a competitive technology for contributing to the generation of renewable electrical energy. With costs falling very rapidly as the technology matures, the capacity installed, for instance, in Europe, grew a significant 25% in 2017, with 13 new offshore wind farms and additional 3.1 GW. This expansion will lead to the installation of offshore wind turbines (OWT) in locations even more challenging from the geotechnical point of view, with greater depths, less capable soils and/or increasing seismic risk. In order to contribute to the field of earthquake resistant design of foundations for OWTs, this papers tackles the computation of kinematic bending moments in OWT monopiles, which is the most common type of foundation in this kind of structures. A parametric study involving different foundations and geotechnical profiles was carried out, involving large diameter monopiles, realistic material and geometrical properties for soils and pile, and a large set of layered soil profiles. For each case, compatible earthquake excitation, described through the elastic response spectra provided by Eurocode 8 - Part 1 for each ground type, is used. Kinematic bending moments were estimated using a Beam-On-Dynamic-Winkler approach. For these large–diameter monopiles, the peak bending moments are not necessarily found in the interfaces between strata, as observed with not so large diameters. Results are presented as a function of the ground type (according to Eurocode 8) and an empirical regression, based on the parameter $c_{s,30}$, for the estimation of a normalized maximum kinematic bending moments is proposed. As expected, the largest kinematic bending moments arise for C, D and E ground types.

Key words: Offshore wind turbines, piles, kinematic interaction, earthquake response

1 INTRODUCTION

This paper summarizes the results of a parametric study on the kinematic bending moments arising in OWT monopiles embedded in different soil profiles. Results are synthesized so that they can be used to estimate these maximum moments as a function of the average value of the S waves velocity in the upper 30 m of the soil profile ($c_{s,30}$).

2 PROBLEM DEFINITION

Four different steel pipe monopile configurations, defined by two different pile external diameters D and two different slenderness ratios L/D are considered (see Table 1). 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 adopted. The minimum pile wall thickness

Table 1: Pile configurations.

Configuration	L (m)	D (m)	t (mm)	L/D	$\delta = D_{int}/D$
1	10.5	3.5	41.37	3	0.97636
2	24.5	3.5	41.37	7	0.97636
3	18.0	6.0	66.37	3	0.97788
4	42.0	6.0	66.37	7	0.97788

recommended by the API 2A-WSD [1] is adopted. Two different boundary conditions at the pile head (free head and zero–rotation head are considered).

These monopiles are assumed to be embedded in 28 different soil profiles ranging from homogeneous half-space (profiles 1 to 4) to layered soils characterized by 6 layers over an stiff half-space (profile 12). These profiles are defined in Table 2, where h is the depth of the layer, ρ_s is the soil density and G_s is the elastic shear modulus, with each layered modeled as a viscoelastic region with 5% histeretic damping and a 0.3 Poisson's ratio. The table also presents the corresponding ground type according to the EC-8 classification.

The system is assumed to be subjected to vertically–incident S seismic waves producing ground surface earthquake motions compatible with the corresponding Eurocode 8 type 1 elastic response spectrum for 5% damping, with $a_g = 0.25\,\text{g}$. Three different artificial earthquakes are used for each soil configuracion.

3 METHODOLOGY

The time histories of the bending moments along the piles are computing through the frequency domain method of response using the Frequency Response Functions obtained from an efficient semi-analytical harmonic Beam-On-Dynamic Winkler model [2, 4] of the problem in which the pile is modelled as a linear–elastic Euler–Bernoulli beam, and the expressions provided by Novak et al. [3] are used to model the soil response as a series of independent horizontal viscoelastic springs and dashpots. Maximum bending moments at a given depth computed for any time during the seismic excitation are used to build bending moment envelopes from which the maximum bending moments at any depth shown below are obtained.

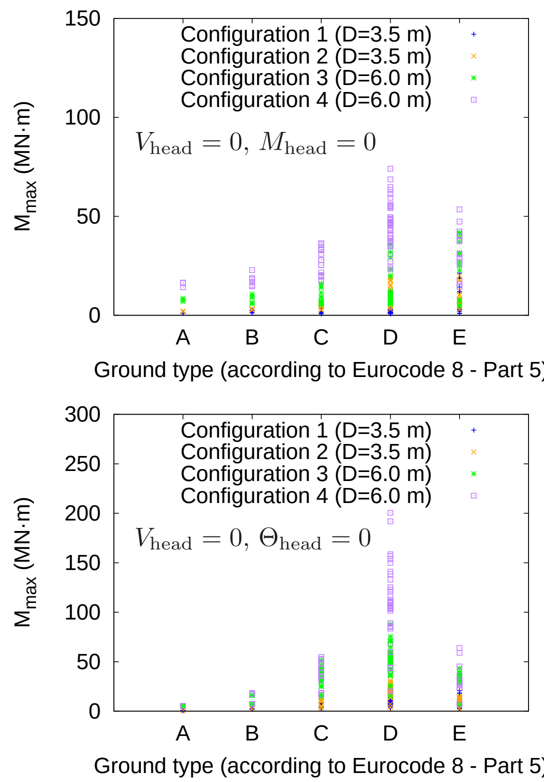

Figure 1: Maximum kinematic bending moments as a function of ground type and pile configuration. Free head (top) and Zero–rotation head (bottom) boundary conditions

4 RESULTS

Figure 1 summarizes the results as maximum bending moments, at any depth, for each ground type, pile configuration and boundary condition at the pile head. Each point represents one specific case (given pile configuration and soil profile). The results are presented in figure 2 as a function of the parameter $c_{s,30}$ together with a representation of the regression analysis performed on the data. For each boundary condition and pile configuration, an expression of the type $M_{\max}(c_{s,30}) =$

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 (5-42)	70 (100)	1650 (1750)	8.08 (17.50)
P5 B	113.75	D	0-5 (5-42)	70 (130)	1650 (2000)	8.08 (33.80)
P5 C	131.77	D	0-5 (5-42)	70 (160)	1650 (2000)	8.08 (51.20)
P5 D	175.00	D	0-5 (5-42)	70 (250)	1650 (2000)	8.08 (125.0)
P5 E	224.00	C	0-5 (5-42)	70 (400)	1650 (2000)	8.08 (320.0)
P5 F	292.14	E	0-5 (5-42)	70 (800)	1650 (2500)	8.08 (1600.0)
P6 A	87.50	D	0-10 (10-42)	70 (100)	1650 (1750)	8.08 (17.50)
P6 B	101.11	D	0-10 (10-42)	70 (130)	1650 (2000)	8.08 (33.80)
P6 C	112.00	D	0-10 (10-42)	70 (160)	1650 (2000)	8.08 (51.20)
P6 D	134.62	D	0-10 (10-42)	70 (250)	1650 (2000)	8.08 (125.0)
P6 E	155.55	D	0-10 (10-42)	70 (400)	1650 (2000)	8.08 (320.0)
P6 F	178.72	E	0-10 (10-42)	70 (800)	1650 (2500)	8.08 (1600.0)
P7 A	154.07	D	0-5 (5-42)	130 (160)	2000 (2000)	33.80 (51.20)
P7 B	216.66	C	0-5 (5-42)	130 (250)	2000 (2000)	33.80 (125.0)
P7 C	297.14	C	0-5 (5-42)	130 (400)	2000 (2000)	33.80 (320.0)
P7 D	430.34	B	0-5 (5-42)	130 (800)	2000 (2500)	33.80 (1600.0)
P8 A	148.57	D	0-10 (10-42)	130 (160)	2000 (2000)	33.80 (51.20)
P8 B	191.18	C	0-10 (10-42)	130 (250)	2000 (2000)	33.80 (125.0)
P8 C	236.36	C	0-10 (10-42)	130 (400)	2000 (2000)	33.80 (320.0)
P8 D	294.34	E	0-10 (10-42)	130 (800)	2000 (2500)	33.80 (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

Table 3: Regression coefficients for each case

	Free head					Zero-rotation head				
Conf.	a	b	d	E^2	S	a	b	d	E^2	S
1	$-9.5 \cdot 10^{-5}$	0.049	-1.9	1462.0	4.3	$5.3 \cdot 10^{-5}$	-0.093	31.1	6947.2	9.4
2	$-1.1 \cdot 10^{-5}$	-0.017	11.5	1215.3	3.9	$3.6 \cdot 10^{-4}$	-0.26	47.1	790.6	3.2
3	$-2.6 \cdot 10^{-5}$	0.13	-1.4	6875.7	9.4	$-3.5 \cdot 10^{-5}$	-0.17	86.3	21896.2	16.8
4	$1.8 \cdot 10^{-4}$	-0.20	68.5	7457.5	9.8	$2.4 \cdot 10^{-3}$	-1.58	269.3	42683.2	23.4

$a\,c_{s,30}^2 + b\,c_{s,30} + d$ has been fitted to the results. The fitted parameters for each case are given in Table 3, together with the root mean square of the residuals $S = \sqrt{E^2/N}$, with N the number of degrees of freedom of the regression and E^2 the weighted sum of the squares residual.

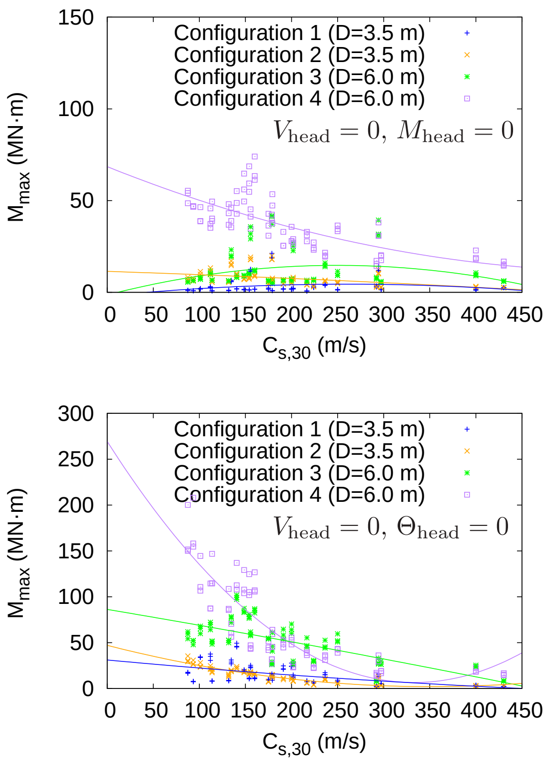

Figure 2: Maximum kinematic bending moments as a function of $c_{s,30}$ and pile configuration. Free head (top) and Zero–rotation head (bottom) boundary conditions

5 CONCLUSIONS

Seismic kinematic bending moments of 4 different offshore wind turbine monopiles in 28 different layered soil profiles have been computed for three different compatible earthquake signals and two different boundary conditions at the pile head, making a set of 672 cases modelled through a Beam–On–Dynamic Winkler formulation.

The largest maximum kinematic bending moments are obtained for D ground types (Deposits of loose–to–medium cohesionles soil or of predominantly soft–to–firm cohesive soil with $c_{s,30} < 180\,\text{m/s}$. The value of these maximum kinematic bending moments are fitted to quadratic polynomials that can be used during the initial stages of the analysis and design of these foundations in earthquake–prone areas.

6 ACKNOWLEDGMENTS

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