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Numerical Modelling of large diameter monopiles supporting OWT-Head stiffness assessment in Non-homogeneous Soils

AISSA Mohammed Hemza[1], AMAR BOUZID Djillali[2]

Abstract—Many countries are now planning to build new wind farms with high capacity up to 5MW. Consequently, the size of the foundation increased. This kind of structures are subjected to fatigue damage from environmental loading mainly due to wind and waves as well as from cyclic loading imposed through the rotational frequency (1P) through mass and aerodynamic imbalances and from the blade passing frequency (3P) of the wind turbine which makes their behavior dynamically very sensitive. That is why natural frequency must be determined with accuracy in order to avoid the resonance of the system.

This paper presents a numerical procedure which combines the finite element method based on the 2D discretization in the radial plane and the Fourier series based on expansion of displacements and forces in the circumferential direction. Firstly analytical expressions of stiffnesses of foundations with large diameter embedded in soils with different stiffness profile are established.

Secondly and in order to check the accuracy of the proposed formulas, a mathematical model approach based on non-dimensional parameters is used to calculate the natural frequency taking into account the soil structure interaction (SSI) compared with measured frequency in five wind farms selected from the litterature.

Keywords—Offshore Wind Turbines; Semi-analytical FE analysis; DNV/Risø; Monopiles foundations; Natural frequency.

I. Introduction :

A serious drive to develop Offshore Wind Energy sector is known in the last 20 years. Most Offshore Wind Converters are founded in Monopiles. These kinds of foundations are made of steel or concrete with a diameter ($\boldsymbol{D_p}$) ranging between 4m and 7m, and embedded length ($\boldsymbol{L_p}$) less than 30m. Inhence, they are subjected to lateral loading ($\boldsymbol{P}$) and overturning moment ($\boldsymbol{M}$) due to wind and waves (Figure 1).

The design procedures for Monopiles foundations are based on the experience gained from the oil and gaz field, on which the American Petroleum Institute (API) method[1] has been based.

This semi-empirical approach was then included in the recommended practice of several Offshore Wind Turbines design codes (DNV/Risø[2] for exemple) and already used to simulate the lateral response of monopiles foundations.

The design is based on the p-y method where the pile is simulated as a beam and the soil as series of elastic springs, this was described by Reese *et al.* (1977)[3] and O'Neill & Murchison (1983)[4] who tested full scale piles with small diameter (0.61m). In this topic, a recent report shows the limitations of using conventional models for Offshore Wind Turbine foundations (TRB, 2011)[5]. This report stated that the diameter and slenderness are much more than experienced for oil and gaz platform foundations and consequently this can reduce the life of the Wind Turbines.

Also many researches approve that the famous p-y curves method is not conservative for the design of the Offshore Wind Turbines foundations, for exemple: Lesny & Wiemann (2006)[6], Augustesen *et al.* (2009)[7], Andersen *et al.* (2012)[8], Harte *et al.* (2012)[9] and Swagata and Sumanta (2014)[10].

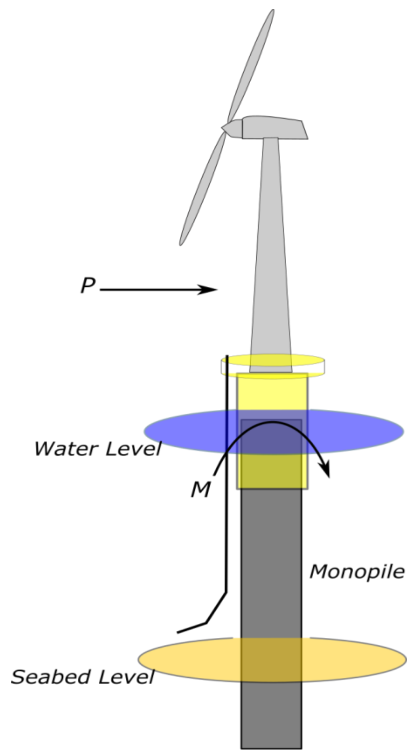

Fig. 1 Loads acting on Offshore Wind Turbine Structure.

The Offshore Wind Turbines experienced during their life service a high dynamic loading cyclic in nature between 10^6 and 10^8 cycles. The main external excitations are:

(1) Environmental loading due to wind and waves. The predominant wave frequency is generally 0.1 Hz.
(2) Rotor loading at a frequency which is referred to as 1P

[1]PhD student
Department of Material Engineering, Faculty of Sciences and Technology, University of Medea
Algeria

[2]Professor
Department of Civil Engineering, Faculty of Technology, University Saad Dahleb of Blida
Algeria

(3) The blade passing frequency is a forced loading generated from the effect of wind deficiency that occurs at each passes of the blade through the shadow of the tower. It refers to as 2P for two blades and 3P for three blades.

The natural frequency must be designed to avoid resonance between (1P) and (3P) (Figure 2). So the soil stiffness must be determined accurately from the soil data for a good estimation of the natural frequency of the system.

The main aim of this paper is to propose new design formulas of stiffness coefficients by a semi-analytical approach using the finite element method (Amar Bouzid *et al.* 2004)[11]. Two soil profiles are considered: Gibson's soil and parabolic soil, the nature of the interface between the soil and the pile is taken into account: rough or S=smooth (Amar Bouzid and Vermeer, 2007)[12].

Thereafter, the accuracy of the obtained formulae is checked using a method based on Euler-Bernoulli Beam-Column with elastic end supports as reported by Adhikari and Bhattacharya (2012)[13] and (Laszlo Arany *et al.* 2014)[14].

II. Semi Analytical Method Background:

This so called semi analytical FE approach was proposed first by WILSON (1965)[15] for FE analysis of axisymmetric structures loaded non-axisymmetrically and later used by many authors (Cook *et al.* (1989 for instance)[16]. The main idea of this method is to use Fourier Series to resolve three dimensional problems as a two dimensional harmonic model and superposing each term result. We can found many applications of Semi-Analytical Approach in practical cases (For example Kim et al. 1994 [17]; Zienkiewicz and Taylor[18]). This method finds its applications in the present problem.

The nodal loads applied to the monopile can be expanded in Fourier series as:

$$R = \bar{R}_0 + \sum_{n=0}^{\infty} \bar{R}_n \cos n\,\theta + \bar{\bar{R}}_n \,\mathrm{SIN}\, n\theta$$
$$Z = \bar{Z}_0 + \sum_{n=0}^{\infty} \bar{Z}_n \cos n\,\theta + \bar{\bar{Z}}_n \,\mathrm{SIN}\, n\theta \qquad (1)$$
$$T = \bar{T}_0 + \sum_{n=0}^{\infty} \bar{T}_n \cos n\,\theta + \bar{\bar{T}}_n \,\mathrm{SIN}\, n\theta$$

Where *R*, *Z* and *T* are the radial, axial and circumferential components respectively with respect to the $\theta = 0$ as plane of symmetry.

The displacement can be expressed in the form of Fourier series:

$$u = \sum_{n=0}^{L} \bar{u}_{\mathrm{N}} \,\mathrm{COS}\, n\theta + \sum_{n=1}^{L} \bar{\bar{u}}_n \,\mathrm{SIN}\, n\theta$$
$$v = \sum_{n=0}^{L} \bar{v}_{\mathrm{N}} \,\mathrm{COS}\, n\theta + \sum_{n=1}^{L} \bar{\bar{v}}_n \,\mathrm{SIN}\, n\theta \qquad (2)$$
$$w = \sum_{n=0}^{L} \bar{w}_{\mathrm{N}} \,\mathrm{COS}\, n\theta + \sum_{n=1}^{L} \bar{\bar{w}}_n \,\mathrm{SIN}\, n\theta$$

where $\bar{u}_n$, $\bar{v}_n$, $\bar{w}_n$ are the amplitudes of displacements that are symmetric with respect to the $(\theta = 0)$ plane and $\bar{\bar{u}}_n$, $\bar{\bar{v}}_n$, $\bar{\bar{w}}_n$ are the amplitudes of displacement that are antisymmetric with respect to the $(\theta = 0)$ plane, n is harmonic number, and L is the total number of harmonic terms considered in the series.

For a Monopile subjected to a lateral and/or an overturning moment, only the second term for *i*=1 survives, because this loading has a plane of symmetry. In this situation the components of loading in Eq. (1) reduce to:

$$R = \bar{R} \,\mathrm{COS}\,\theta,\ Z = \bar{Z} \,\mathrm{COS}\,\theta,\ T = \bar{T} \,\mathrm{SIN}\,\theta \qquad (3)$$

Where $\bar{R}$, $\bar{Z}$ and $\bar{T}$ are the amplitudes of nodal loading on the first harmonic. The load system displacements of Eq. (2) will reduce to:

$$u = \bar{u} \,\mathrm{COS}\,\theta,\ v = \bar{v} \,\mathrm{COS}\,\theta,\ w = \bar{w} \,\mathrm{SIN}\,\theta \qquad (4)$$

III. Soil stiffness and FE mesh considered in the present analysis:

In our analysis we considered a cylindrical pile with D_p is the diameter of the monopile, and it's length L_p , Young's Modulus E_p . H_0 is the horizontal load acting on the head of the pile and M_0 is the corresponding overturning moment. The soil is described by its Young's modulus E_s and Poisson's ratio ν_s. In the parametric study we assume the stiffness linear increasing with depth which is a typical consideration for sand or Gibson's soil profile (Scott, 1981)[19] and the stiffness increasing with square root of depth for parabolic soil profile. This consideration of soil non-homogeneity has proved to be more realistic in many practical cases where the effective stresses increase with depth. The soil modulus is usually taken to have a power law variation with depth (Booker *et al.* 1985)[20] as expressed by the equation:

$$E_s(z) = E_{sD}(z/D_p)^{\alpha} \qquad (5)$$

Where E_{sD} is the soil modulus at a depth z equal to the monopile diameter D_p and α is an exponent that varies between zero and one. Equation (5) includes a soil with a parabolic variation of stiffness with depth for $\alpha = 1/2$ and a Gibson's soil for $\alpha = 1$

The monopile has a diameter D_p =3m with variable slenderness ratio L_p/D_p ranging between 1 to 15. The relative rigidity of the pile/soil ratio is taken between 10 and 10^6. The flexural rigidity of the pile $(EI)_p$ = 79521564.04kN.m^2 and ν_p=0.25. The poisson's ration of the soil ν_s is taken equal to 0.499. A Fortran program named Pile-Joint has been used. The model consists of 1326 eight-noded quadrilateral elements, whose 200 elements for modeling the monopile and 1126 for the surrounding soil (Figure 3). To ensure a good accuracy and because the diameter of the monopile is large we refine the mesh in the vicinity of the monopile and around the interface area. The distance under the monopile tip is one length L_p.

In the horizontal direction, the distance is taken equal to 40 times the radius of the monopile. The interface is taken into account for this study. We have to cases: Rough Interface (normal rigidity $\boldsymbol{K_n}=\mathbf{10^{12}}$ and the shear rigidity $\boldsymbol{K_s}=\mathbf{10^{12}}$) and Smooth Interface (normal rigidity $\boldsymbol{K_n}=\mathbf{10^{12}}$ and the shear rigidity $\boldsymbol{K_s}=\mathbf{0}$).

A. *Results of the Stiffness Analysis:*

Various criteria for rigid or flexible behavior have been suggested in the literature, for example Poulos and Hull (1989)[21]. The results of the stiffness coefficients (K_H, K_M, K_{MH}) have been plotted as function of L_p/D_p and compared with existing solutions proposed by Higgins *et al.* (2013)[22] in Table 1 and Table 2.

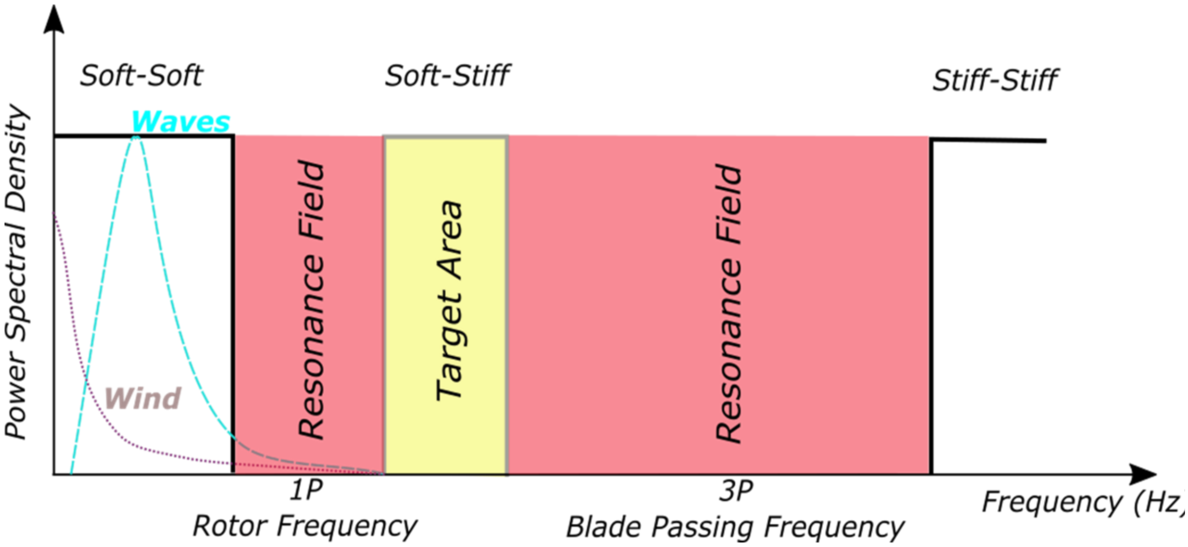

Fig. 2 Dyanmic design approach showing the forcing frequency as function of power spectral density.

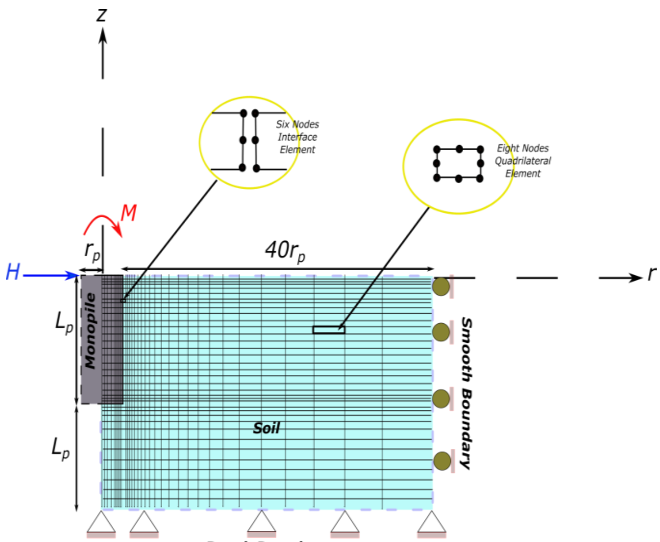

Fig. 3 Geometric model taken in this study.

The Table 3 shows the stiffness equations obtained in the present analysis for the parabolic soil case. For this specific case, expressions for monopile head stiffness were no localized in the literature.

The results of the stiffness coefficients (K_H, K_M, K_{MH}) have been tabulated as function of L_p/D_p and compared with existing solutions proposed by Higgins *et al.* (2013)[22] in Table I and Table II. .

The Table III shows the stiffness equations obtained in the present analysis in the parabolic soil case.

B. *Discussion of stiffness results:*

The obtained results show a good agreement with Higgins *et al.* In Gibson's soil profile for rough interface case. Indeed, the interface state which has a great effect on the results above in both Gibson's soil and Parabolic soil profiles.

IV. Natural frequency assessment:

In the design approach the support structure modeled by static springs which lead to an independency of the stiffness coefficients on the natural frequency of the system, that why the Soil Structure Interaction (*SSI*) is very important in any dynamic analysis of the system. S. Adhikari & S. Bhattacharya (2012)[13] reported the equation of motion of the beam and includes with analytical resolution non-dimensional parameters of the foundation stiffness (In this case we find just K_H and K_M terms).

Laszlo Arany *et al*. (2014)[14] give a coupled stiffness term (K_{MH}) and prove by mean of sensitive analysis that the determination of the natural frequency according to two terms is not sufficient.

For our analysis we choose six Offshore Wind Turbines situated in the North Sea and we compare results of the computed natural frequency determined by two approaches(the approach used in the present study with the equations founded in the DNV/Risø Guideline) with the measured one. Table IV and Table V gives the obtained results from this dynamic analysis:

TABLE I. Stiffness equations obtained in the present analysis in Gibson's soil profile.

Relative Stiffness Term	Rough Interface	Smooth Interface
Relative Horizontal Stiffness	$K_H/E_{sD}D$=1.6471$(L_p/D_p)^{1.6944}$	$K_H/E_{sD}D$=1.2140$(L_p/D_p)^{1.7482}$
Relative Moment Stiffness	$K_M/E_{sD}D^3$=1.1148$(L_p/D_p)^{3.6331}$	$K_M/E_{sD}D^3$=0.8151$(L_p/D_p)^{3.6858}$
Relative Coupling Stiffness	$K_{MH}/E_{sD}D^2$=−1.1889$(L_p/D_p)^{2.6870}$	$K_{MH}/E_{sD}D^2$=−=−0.8967$(L_p/D_p)^{2.7316}$

TABLE II. Stiffness equations proposed by Higgins *et al.* (2013) in Gibson's soil profile.

Relative Stiffness Term	Rough Interface
Relative Horizontal Stiffness	$K_H/E_{sD}D$=0.9161$(L_p/D_p)^{2.0415}$
Relative Moment Stiffness	$K_M/E_{sD}D^3$=0.6625$(L_p/D_p)^{3.9417}$
Relative Coupling Stiffness	$K_{MH}/E_{sD}D^2$=−0.6244$(L_p/D_p)^{3.0618}$

TABLE III. Stiffness equations obtained in the present analysis in Parabolic soil profile.

Relative Stiffness Term	Rough Interface	Smooth Interface
Relative Horizontal Stiffness	$K_H/E_{sD}D$=2.8300$(L_p/D_p)^{0.9958}$	$K_H/E_{sD}D$=2.0555$(L_p/D_p)^{1.0675}$
Relative Moment Stiffness	$K_M/E_{sD}D^3$=3.9374$(L_p/D_p)^{2.5707}$	$K_M/E_{sD}D^3$=2.5554$(L_p/D_p)^{2.6863}$
Relative Coupling Stiffness	$K_{MH}/E_{sD}D^2$=−=−2.9421$(L_p/D_p)^{1.7824}$	$K_{MH}/E_{sD}D^2$=−=−2.0997$(L_p/D_p)^{1.8649}$

TABLE IV. Computed and Measured Natural Frequency of various Wind Turbines in Gibson's soil profile.

Wind Turbine Name	Computed Frequency with Rough Interface	Computed Frequency with Smooth Interface	Computed Frequency with DNV/Risø (Hz)	Measured Frequency (Hz)
Walney 1	0.3454	0.3446	0.3342	0.3500
Lely A2	0.7613	0.7608	0.7358	0.6300
North Hoyle	0.4481	0.4481	0.4258	0.3500
Irene Vorrink	0.5509	0.5505	0.5297	0.5400-0.5600
Sheringham Shoal	0.4996	0.4989	0.4648	0.8500-0.9600
Kentish Flats	0.6282	0.6273	0.5460	0.8500-0.9600

TABLE V. Computed and Measured Natural Frequency of various Wind Turbines in Parabolic soil profile.

Wind Turbine Name	Computed Frequency with Rough Interface	Computed Frequency with Smooth Interface	Computed Frequency with DNV/Risø (Hz)	Measured Frequency (Hz)
Walney 1	0.3454	0.3445	0.3315	0.3500
Lely A2	0.7609	0.7603	0.7302	0.6300
North Hoyle	0.4480	0.4480	0.4212	0.3500
Irene Vorrink	0.5506	0.5501	0.5251	0.5400-0.5600
Sheringham Shoal	0.4990	0.4981	0.4574	0.8500-0.9600
Kentish Flats	0.6272	0.6259	0.5302	0.8500-0.9600

A. *Discussion of the dynamic results:*

A preliminary analysis of the results above shows that the difference between the natural frequency in the two profile is very smaller. Furthermore, the interface state has no effect on the natural frequency of the system. In comparison with the measured natural frequency the error is admissible in Walney 1 and Irene Vorrink Wind farms, but for example in Kentish flats the error is more than 60% which is not allowable.

In all cases, the DNV/Risø approach underestimates the natural frequency (Walney 1, Irene Vorrink, Sheringham Shoal and Kentish flats cases) or overestimates the natural frequency (Lely A2 and North Hoyle cases). Returning to our results we remark that the computed natural frequency with the present approach is closer to those measured in four cases: Walney 1, Irene Vorrink, Sheringham Shaol and Kentish flats Wind turbines.

Conclusion:

This paper focused on the monopile head stiffness fo Offshore Wind Turbinres using a numerical analysis. Analy tical expressions were proposed both Gibson's soil and parabolic soil profiles with different interface states and validated with existing solutions

the dynamic behavior of piles. Most monopiles are sensitive to dynamic/cyclic loading are those supporting Offshore Wind Turbines in a great depth which are caracterised by large diameter.

To check the applicability of the present study equations we use it in the calculation of the natural frequency of Wind Turbines structures and compared with the DNV/Risø Guideline's equations and measured data. The results of the natural frequency of Wind Turbines structures and compared with the *p-y* equations and measured data.

In a hand, the results of our study show a well accuracy with the measured natural frequency contrary to the DNV/Risø results (Walney 1 and Irren Vorrink), this is due to the rigid behavior of these piles, which leads to the useless of the equations founded in the famous DNV/Risø Guideline sited previously.

In the other hand, the error of the present method is much higher in the others case study (The error in Kentish flats is more than 60% of the measured natural frequency). This is explained by the flexible behavior of the foundations, it means that the length of these piles exceeds the critical length. So this make the applicability of our formulas outside of them range.

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