Review of Numerical Models for Studying the Dynamic Response of Deep Foundations for the Design and Project of Wind Turbines

G. M. Álamo, J. D. R. Bordón, F. García, J. J. Aznárez, L. A. Padrón, F. Chirino and O. Maeso

Abstract Wind turbines support structures must be designed so that the natural frequencies of the entire system are sufficiently separated from the frequency of the different dynamic loads acting on the wind turbine. The design and analysis of the soil-foundation subsystem is subject to significant levels of uncertainty and simplification. Besides, as the number of wind farms increases, so does the need for installing wind turbines in weaker soils, which leads to the use of deeper foundations such as piles and suction caissons. Thus, the need exists for developing computational models able to estimate, with increasing accuracy and efficiency, the dynamic properties of the foundations mentioned above with the aim of being able to reach optimized, safe and long-life designs that help improving the profitability of the technology and reducing the wind energy costs. In this line, this paper presents a review of computational models, with different degrees of accuracy, applicable to the analysis of the dynamic response of deep foundations for onshore and offshore wind turbines.

Keywords Wind turbines · Soil-structure interaction · Boundary element method
Structural dynamics

1 Introduction

The installed capacity of electricity generation from offshore wind power has been growing exponentially in the last few years. So much so, that the production capacity connected to the grid in Europe grew in 2015, nothing less than 108% more than in 2014. Specifically, 3019 MW of power were added through the

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installation of new offshore wind turbines in 15 different farms, connected to the European electric network. For that matter, it is really shocking that, of the 3,230 offshore wind turbines installed in Europe until December 31st 2015, only one is installed in Spain (a 0.03% of the total). Moreover, at the moment, there are 6 projects underway in Germany, the Netherlands and United Kingdom, which will contribute close to 2000 MW more to a growth that is expected to continue in the near future (EWEA 2016).

Almost all offshore wind turbines installed in Europe are located in places where the depth of the sea allows founding them directly to the seabed (compared to the alternative of floating wind turbines, which is still anecdotic for the time being). In fact, the average water depth in wind farms installed or under construction during 2015 was 27.1 m, with an average distance to the coast of 43.3 km. Nonetheless, there are operating farms in depths up to 50 m and located at a distance up to 120 km from the nearest coast. Thus, in Europe, and until December 31st 2015, 80% of wind turbines are installed over monopiles, 9.1% over shallow foundations and the rest over structures with several supports, as groups of three suction caissons (3.6%) or three piles (1.7%) (EWEA 2016).

With the above mentioned growth of the offshore wind farms' number, the need arises to install the wind turbines at greater depths and/or in soils with worse load-bearing characteristics. Given the special characteristics of the loads generated on the foundation by the offshore wind turbines (large bending moments applied to the seabed with significant horizontal loads but very low vertical load), the use of shallow foundations is only feasible in some cases (limited depths and/or rock bed of a very good load-bearing capacity). For the remaining cases, as it is highlighted in the previous paragraph, it is necessary to use deep foundations, mainly piles. Suction caissons, on their part, although still small in number, offer important potential benefits: lower installation costs, lower costs for geotechnical exploration due to their lesser embedding depth, and the possibility of uninstalling the foundation. For these reasons, the analysis and development of this alternative is producing a growing interest. These two foundation types (piles and suction caissons) are the study object of this communication.

In this respect, it is important to take into account that the cost associated with the foundation, represents, in general, a very important proportion of the total required investment for offshore wind farms implementation. Specifically, the design, construction and installation of, for example, the monopile for an offshore wind turbine represents about 15–20% of the total initial cost. In some projects, this cost can be as high as 25–30% (Byrne and Houlsby 2003; Musial and Ram 2010; Bhattacharya 2014). It has also been estimated that foundation costs are estimated to increase approximately linearly with depth (Musial and Ram 2010). It becomes clear then that the reduction of the implementation costs of these foundations by improving the proposed solutions during the project's first phases would represent an important factor when favoring the implementation of new offshore wind farms, and would also improve the profitability of this technology by reducing its initial costs and, at the same time, increasing its useful life.

It is important to take into account that, contrary to what happens in many other types of projects, the final design and dimensioning of offshore wind turbines foundations are not totally conditioned by the requirements associated with Ultimate Limit States, but for those of the Fatigue Limit State (accumulation of damage due to repetitive loads) and Serviceability Limit States, because compliance with these last two states is very restrictive (Arany et al. 2015) and often implies a loose compliance of the Ultimate Limit States requirements. Among these Ultimate Limit States, is the structural failure of the substructure itself or the surrounding soil (load-bearing capacity loss), or the overturning or sliding stability loss. The Serviceability Limit States include excessive deflection or rotation at the foundation head or at the nacelle, permanent rotations, excessive settling, differential settling or excessive vibrations (DNV 2014).

A vital importance is granted to the detailed analysis of the Fatigue Limit State, because it is directly related both with the planned useful life for the wind turbine as a whole, and with the possibility of extending the operational life (thus, increasing the facility profitability). The foundation dynamic characteristics, in terms of rigidity and capacity to dissipate energy (damping), play a fundamental role in the reduction of the accumulated damage, because both of them are key to reducing structure vibration amplitude. In the rigidity case, its proper estimation in the project's design and planning phases allows preventing the structure from vibrating in resonance with any of the multiple dynamic loads to which it is subjected (waves, currents, winds, blades passing in front of the shaft, rotor and blades imbalances…). In the damping case, higher values of this parameter in the foundation allow maximizing the mechanical energy evacuation from the structure through the ground.

Still, it is the compliance of requirements associated with Serviceability Limit States that generally determines the foundation's final design and dimensioning. For example, in the case of a monopile, the criterion for maximum rotation at pile head can be associated with an oversizing higher than 15% in diameter, with regard to the majority of the remaining criteria (Arany et al. 2015). Another criterion related with this can lead to more important oversizing, for instance, the achievement of a given rigidity, guarantying that the system's natural frequencies remain sufficiently far from the loads' characteristic frequencies. For this reason, and as it has been numerically observed (Damgaard et al. 2014a) deformation and tension levels achieved in the ground (and of course, in the foundation itself) tend to be lower than the yield point, so the foundation is operating fundamentally in the elastic and linear range. This justifies that many of the proposed models that are later used are of the linear-elastic type.

These low deformation levels will also favor a certain stability over time of the foundation performance in terms of dynamic rigidity and damping. This way, the global system natural frequencies do not deviate from the established ones. As it was mentioned, this aspect is especially relevant with regard to the need that the natural frequencies are maintained sufficiently away from the frequencies defining excitations. Still, with the accumulation of load cycles, a certain degradation of the dynamic properties of the turbine could occur. Several researchers are occupied in finding out whether this aspect could influence negatively and significantly the structure response in the long term (Foglia et al. 2015; Bhattachary et al. 2012, 2013; Cox et al. 2014). For this end, the main tool is the elaboration and test of scale empirical models, such as the proposed by Bhattacharya et al. (2012) and Bhattacharya et al. (2013) over a model 1:100 and 1-g of a wind turbine over monopiles or a group of three or four suction caissons; where degradation, compaction and stiffening phenomena of the surrounding soil are observed, leading to significant changes in the foundation properties. Subsequently, Cox et al. (2014), when performing a centrifugal study of individual suction caissons behavior, found similar tendencies in this foundation types, but more limited than in the monopiles case. In any case, these degradation phenomena, if they are produced, occur slowly and progressively in the long term, so the study on the system's response at a given moment is made through models that do not necessarily consider these phenomena.

Thus, to analyze the dynamic response of deep foundations for wind turbines, several different models. The numeric modeling of the dynamic response of deep foundations for wind turbines is important because the interaction between the support structure and the soil modifies the dynamic response of the structure as a whole. For this reason, the structure cannot be studied as perfectly fixed to the soil. These soil-foundation-structure interaction phenomena are much more important in offshore wind turbines than in the onshore ones, not only because of the seabed nature, the wider variety of dynamic loads that the structure withstands and the higher magnitude of these loads, but also due to the larger height of the support structure (which has an aerial plus a submerged section that can frequently be longer than 30 m). This higher height increases the influence of the inertia forces over the foundation and produces a higher flexibility of the structural group.

For these reasons, this communication describes in the first place, the diversity of existing models for the estimation of the foundation dynamic response, so the phenomena soil-foundation-wind turbine can be evaluated (Sect. 3). Subsequently, in Sect. 4, a group of advanced numeric models is presented, each one with their particularities and application cases, proposed and developed by the authors of this communication, for the dynamic response analysis of deep foundations for wind turbines. Their utilization is shown in Sect. 5 with a selection of results.

2 Objectives

The objective of this communication is to present a state of the art review regarding the dynamic modelling of foundations for wind turbines and subsequently to describe different advanced computational models, related mainly to the Boundary Element Method (specially indicated for the study of not bounded media such as the soil). These models allow analyzing the dynamic response of deep foundations for wind turbines and, thus, studying soil-foundation-wind turbine interaction phenomena. This is done with the intention of contributing to obtain optimal support structures designs, secure and with a larger useful life, and as a consequence, to improve this technology's profitability and to reduce costs.

3 Review of Commonly Used Numerical Models for the Study of the Dynamic Response of Deep Foundations for Wind Turbines

In scientific and technical literature, a significant number of models are commonly used to study the dynamic response of deep foundations for wind turbines. In the first place, the two approximations with a higher level of simplifications are: (a) the rigid base model (in which soil-structure interaction is directly disregarded), and (b) the model of apparent (or effective) fixation length, which consists in considering that the wind turbine tower is longer than it actually is, adjusting this added length such that the lost flexibility due to the presence of a flexible foundation in an also flexible soil will be partially incorporated (Zaaijer 2006). However, due to their excessive simplicity, these models are no longer used when the interest lies in considering the effects of the soil-structure interaction. Thus, the most used models for the study and analysis of the structural response of deep foundations for wind turbines including the soil-structure interaction effect, can be classified in the following four groups.

3.1 Stiffness Matrix Models

The models based on the stiffness matrix at mudline are those in which the interaction with the soil-structure system is modeled through actions equivalent to one or several punctual linear springs and dashpots (generally, viscous) in the tower base at the foundation terrain height. In some cases, such elements consider only stiffness and damping to rotation. In the most complete models not only rotation and horizontal interactions are included, but also their cross influences, which are significant in deep foundations. For that matter, Zaaijer (2006) is the reference work regarding the application of these concepts to the case of offshore wind turbines, comparing this model with the more simple ones above described, and with the model exposed in the next point. He arrives to the conclusion that the complete stiffness matrix model defined at the foundation bed depth is the best approximation for the case of monopiles.

On the other hand, more simple models consider static coefficients, while the more complete models include their dynamic characteristics, even with concentrated parameters models (Lumped Parameter Models). These models allow the use of frequency-dependent stiffness and damping coefficients in analyses performed in the time domain (for example, see Damgaard et al. 2014a).

In any case, the key aspect in the model utilization is, of course, obtaining and/or defining the necessary stiffness and damping functions. They can be taken from expressions found in the literature, generally for the static case (Poulos and Davis 1980; Randolph 1981; Gazetas 1984 or Eurocode 8 – Part 5, 2003) or they can be derived from the direct application of the methodologies described in the following sections.

3.2 Winkler-Type Models for Monopiles

Models of distributive rigidity of the Winkler type, are those in which the pile is generally modeled as a monodimensional, linear and elastic beam subjected to bending, while its interaction with the soil is represented through a series of springs and dashpots distributed along the depth.

A first subgroup of this case comprises elastic and linear models in which the key aspect is to obtain the expression that defines the interaction between pile and soil. These models are widely used in different fields of structure dynamics (Novak et al. 1978; Kavvadas and Gazetas 1993; Mylonakis 2001; Dezi et al. 2009).

Another subgroup is that in which the properties of the distributed springs and dashpots depend, not only on the pile and the soil in each case, but also on the depth, the deformations magnitude and the previous states, so it is non-linear in nature. The most used behavior laws in this case are the ones called *p-y*, *t-z*, *Q-z*. They relate horizontal and vertical loads and deflections, acting along shaft and pile tip, respectively. The *p-y* curve, for instance, represents the soil's horizontal resistance (*p*) per unit length of the pile when it undergoes a lateral displacement (*y*) against the soil. These *p-y* type models are probably the more extended and commonly used, among other reasons because they appear in Sect. 10.1.5 (soil-structure interaction) of the design standard for offshore wind turbine structures (DNV 2014) as an example of a possible model to be used in the dynamic analysis of the wind turbine. However, more and more specialists insist that this model is not appropriate for the specific problem here analyzed, mainly because the *p-y* curves were obtained for diameters and load conditions very different to those found in the wind turbines case, and also because they do not adequately consider the inertial effects involved, among other reasons. In fact, appendix F (Sect. 2.4) of the above mentioned standard (DNV 2014) includes a series of notes mentioning the precautions with which this type of models should be used, due to several possible problems associated with their use in different situations.

Specific examples of the use of this methodology in the field of dynamic response of wind turbines are the works of Jonkman et al. (2008) and Abhinav and Saha (2015). Another example is the work of Damgaard et al. (2014b) in which the influence of pore pressure in the saturated seabed is also researched, justifying the need for model development which considers the terrain's porous nature and the different possible water saturation states, as is the case of one of the models described in Sect. 4.

3.3 Advanced BEM-FEM Coupled Models

The Boundary Elements (BEM) and Finite Elements (FEM) coupled models allow modelling more rigorously the geometries and characteristics of the foundation and soil, even though these analyses are basically performed in an linear-elastic regime, as it was stated in this document's introduction. They also allow considering the real foundation geometry and all their elements simultaneously. This is also true in the case of foundations formed by pile groups or suction caissons, also directly including the superstructure if necessary. An application of this methodology to the study of the dynamic response of foundation caissons is the work of Liingaard et al. (2007), while in the field of piled foundations, examples of works in this line are those of Kaynia and Kausel (1991), Maeso et al. (2005) or Padrón et al. (2007).

3.4 Advanced Non-linear Finite Element Models

When it is necessary to include nonlinear behaviors in three dimensional foundation models, it is generally required to use models based on the Finite Element Method. In these cases, the nonlinear phenomena are usually centered in the soil behavior and the soil-foundation contact (slippage, separation….), although in some cases the structural element can also develop nonlinear behaviors, as in the case of buckling (Madsen et al. 2015). Except in this last case, the justification for the use of these complex and costly models is not still clear, as it was argued in the introduction. In these cases, the first step (and the main obstacle) is the selection of the material behavior law (generally the soil) and all the parameters that these models include and which are not always easily estimated.

An interesting example of this model use is found in the work of Jung et al. (2015), who use a three dimensional and nonlinear model of Finite Elements to model the dynamic response of an offshore wind turbine monopile. They incorporate the failure criteria of sand or clay soils, according to the Mohr-Coulomb and Tresca models respectively and validate it with previous experimental results. The authors compare the results of this model with the results of a Winkler type model, based on the *p*-*y* curves, and with those of a model which does not consider foundation flexibility. They show that this last model is too simple and produces erroneous results and, for example, underestimates moments at the structure base. At the same time, they show that the nonlinear FEM model produces larger rotations at the pile tip, but that the Winkler and MEF models offer very similar results regarding natural frequencies and maximum stresses. This supports the idea that this type of advanced models is only justified for very specific calculations or cases. Some of them can be seen in the works of Anastasopoulos and Theofilou (2016) and Carswell et al. (2015) for piles, or Jin et al. (2014) for groups of suction caissons.

4 BEM-Based Advanced Numerical Models for the Analysis of the Dynamic Response of Deep Foundations for Wind Turbines

The previous section showed the range of methodologies available for modelling the dynamic response of deep foundations for wind turbines. As seen above, it is still necessary to make progress in the development of methods and advanced and rigorous numerical models. These methods and models have to be capable of taking into account the real geometry and properties of all involved elements, and also of estimating the actual system response characteristics. Moreover, these models have to be computationally efficient to make their use feasible in the different project phases. This computational efficiency also has to support the study of the vast amount of different load cases that need to be analyzed during the design and verification of the wind turbine structure (Zaaijer 2006; DNV 2014).

For this end, as it has been previously argued, linear and elastic models seem to be the best option to study all eminently linear aspects of the analysis (study of frequencies and eigenmodes, study of the system response in nominal regime of operation, etc.). For these cases, this section presents several models developed by the authors of this document, which are optimal for their application in different situations.

4.1 Multidomain Boundary Elements Model for the Analysis of Problems Involving Poroelastic Regions

Historically, this was the first model developed by the group to address the dynamic impedances calculation of piles and pile groups in saturated soils (Maeso et al. 2005). Additionally to being able to model the soil as an elastic medium, it allows the analysis of a particular case of water saturated or quasi-saturated soils by considering them as pore-elastic media, according to Biot's theory (Biot 1956, 1962). In these soils, that can be characteristic of the seabed, the interaction phenomena between soil and structure are of a special importance.

In the case of water saturated or quasi-saturated soils, the problem is addressed three dimensionally, considering the piles as a viscoelastic continuous medium and the surrounding soil as a pore-elastic medium, taking into account its two-phase nature. The BEM equations are applied to each one of the regions, performing the coupling through the equilibrium and compatibility conditions along the pile-soil interface.

The model presents some not negligible advantages: it allows to reproduce any foundation geometry, including different piles or inclined piles with their real cross section; it is possible to model the contact condition between pile and soil, being it permeable or impermeable; it is possible to include subsoil geometries with layers with different characteristics (elastic or poroelastic), or it is even possible to take into account the presence of a water layer over the foundation soil, as it occurs in the offshore wind turbines installation. As the principal disadvantage, it must be mentioned that a great computational effort is required, but it is compensated with a higher versatility, generality and precision.

4.2 Finite Elements—Boundary Elements Coupled Model for the Analysis of the Dynamic Response of Piles and Pile Groups

In the search for a way to reduce the great number of degrees of freedom necessary to use models based on BEM (see Fig. 1), the authors have developed a BEM-FEM coupled model for the dynamic study of pile foundations in the frequency domain (Padrón et al. 2007). The soil is modeled through BEM, taking advantage of this formulation possibilities to represent semi-infinite spaces, as one (or several) homogeneous, isotropic, viscoelastic and linear domains; while the piles are represented using FEM as Bernoulli beams, disregarding their torsional stiffness. It is assumed that soil continuity is not altered by the piles presence, but they are treated as monodimensional load lines acting in its interior. This way, the discretization of the soil-pile interface is not necessary, and the only variables left associated with the piles are the displacements and rotations of the pile section and the distributed tractions of soil-pile interaction. Despite these simplifications, the BEM-FEM model reaches results which are equivalent to those from a complete formulation based on BEM, such as the one presented in the previous section. Thus, the BEM-FEM model is a tool that can be used to obtain dynamic responses for piled foundations in homogeneous terrains or in soils with irregular stratigraphy. Nonetheless, if the number of layers is very high, the BEM-FEM coupled model, as well as a BEM multidomain model, implies a high computational cost, due to the necessity to discretize each one of the boundaries that constitute the interfaces, with the subsequent increase on the degrees of freedom and size of the model matrices.

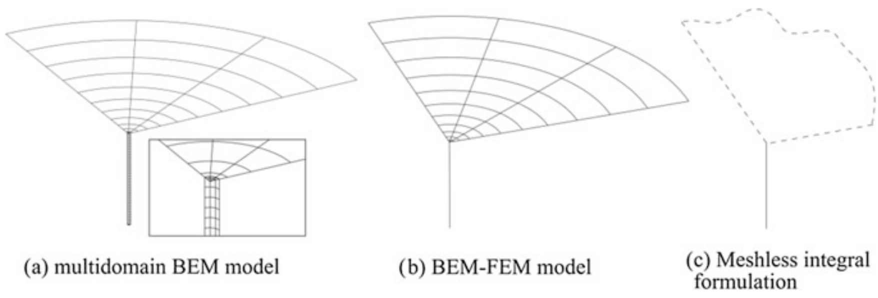

Fig. 1 Comparison between BEM multidomain, BEM-FEM and integral models

4.3 Meshless Integral Model for the Analysis of the Dynamic Response of Piles and Pile Groups

To avoid this inconvenience and be able to assess problems with more realistic soil profiles, in the last few years the authors have developed an integral model (Álamo et al. 2016) where the fundamental solution of the infinite elastic domain used in the previous BEM-FEM model is substituted by a fundamental solution for the stratified half-space (Pak and Guzina 2002). By using this fundamental solution, which already verifies the free surface condition, as well as the compatibility and equilibrium between the layers, the discretization of the terrain boundaries is avoided. Thus, the problem's degrees of freedom are reduced only to the ones pertaining to the piles (displacements, rotations and soil-pile interaction tractions). For this reason, this model is ideal to study terrains with several horizontal layers, or those which properties vary continuously with depth, through approximation by a sufficient number of layers.

As has been mentioned, both models can be used to obtain impedance functions of foundations constituted by one or more piles, and also to directly analyze the wind turbine-foundation system as a whole. To do this, it is only necessary to attach a finite element model of the shaft to the upper part of the foundation.

4.4 Finite Elements—Boundary Elements Coupled Model for the Analysis of the Dynamic Response of Suction Caissons

A suction caisson is a buried laminar structure, cylindrical in shape, which is geometrically defined by its diameter D, length L and thickness t. The metallic sheet has a thickness t which is much smaller than any of the other characteristic dimensions of the caisson, that is, $t \ll L, D$.

For the study of this type of foundations, BEM-BEM multidomain models can be used, such as the one mentioned earlier. However, their use has important disadvantages of numerical, computational and methodological nature. On the one hand, the presence of a domain (the skirt) of a small thickness creates difficulties in the numerical calculation of the necessary integrals to construct the BEM equation system. Furthermore, the linear equation system's matrix is very bad conditioned due to the closeness of the points in which the integral equations are situated, at both sides of the sheet. These two factors combined lead to numerical errors very difficult to resolve. On the other hand, due to the laminar nature of the structure, the Kirchoff (thin sheet) or Reissner-Mindlin (thick sheet) hypotheses are perfectly suitable and lead to a reduction of the degrees of freedom with respect to a continuous model, which reduces computational costs associated to the equation system resolution. Finally, the mesh construction, as well as the calculation of the caisson variables of interest, e.g., stresses, requires an important pre- and post-processing time investment.

For the reasons stated in the previous paragraph, the authors have developed a BEM-FEM model (Bordón et al. 2016) for buried laminar structures, where discretization is completely direct and natural. The laminar structure in this case, the suction caisson, is discretized through sheet type finite elements, and its interaction with the surrounding soil is treated with crack type boundary elements. In this case, unlike in the Liingaard et al. model (Liingaard 2007), the use of artificial interphases is not necessary. This model is being used for impedance calculation of elastic and poroelastic soils, but the direct study is also possible, by incorporating the superstructure to the model.

5 Results

In this section, a selection of results obtained by the authors of this communication are presented, which show in a very succinct manner some of the possibilities of the codes developed by the research group and mentioned in the previous section.

In the first place, Fig. 2 shows the influence of the soil poroelastic properties over the dynamic response of a piled foundation obtained with the multidomain boundary elements code (Maeso et al. 2005) described in Sect. 4.1. Specifically, it shows the horizontal impedance functions (dynamic stiffness and damping of the foundation) of a square group of 4 piles of diameter d, length L and separation between centers s, with geometrical ratios $s/d = 5$ y $L/d = 15$. With regard to the mechanical properties of soil and pile, the presented results have been calculated for a poroelastic soil with a porosity $\Phi = 0.35$, solid skeleton damping factor $\beta = 0.05$, a reinforced concrete pile, a relation between the pile Young modulus and the soil drained Young modulus $E_p/E = 343$, a relation between pile and soil densities $\rho_p/\rho = 1.94$, a Poisson coefficient in the pile of 0.2 and dissipation constants $b^* = 59.3$, 0.593 or 0.00593. Additionally, the null or infinite values corresponding to the ideal cases of perfectly drained soils or undrained soils are considered (see

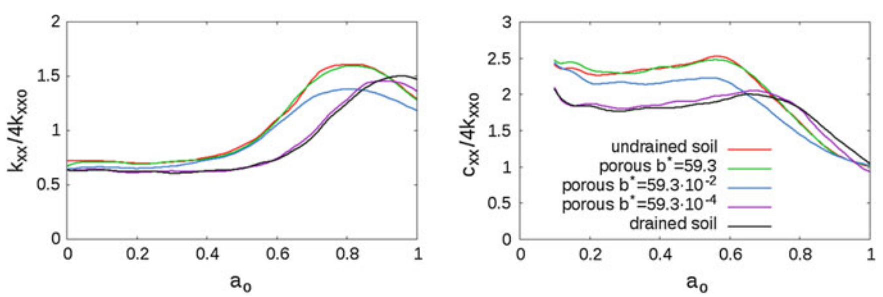

Fig. 2 Horizontal impedances of 2 × 2 pile groups in poroelastic soils with different permeabilities

Maeso et al. (2005) for additional details over the poroelastic soil properties and their definitions). Results are presented with regard to the dimensionless excitation frequency $a_o = \omega d/c_s$, being ω the excitation frequency in radians per second and c_s the shear waves (S) propagation velocity in the soil. There are significant differences, no less than 15%, between stiffnesses of the same foundation in limit properties soils (drained or not).

In second place, and as a performance illustration and comparison of the BEM-BEM, BEM-FEM and integral without mesh models described in Sects. 4.1, 4.2 and 4.3 respectively, Fig. 3 presents results corresponding to the response at pile head of a single pile embedded in a homogeneous half-space, subjected to vertical S (left) and P (right) waves. Specifically, the horizontal and vertical displacements are presented of the head of a pile with a slenderness ratio $L/d = 20$, normalized by the corresponding displacement produced by incident seismic waves at the free surface without foundation. The internal damping ratio of the soil is $\beta = 0.05$, its Poisson coefficient is 0.4, pile-soil Young modulus ratio is $E_p/E = 1000$, density ratio is $\rho_s/\rho = 0.7$. The results are presented according to the dimensionless excitation frequency a_o. As can be observed, the results obtained through the three methodologies are practically the same, but the computational cost and the effort needed to prepare input data decreases significantly when going from BEM-BEM (or multidomain BEM), to BEM-FEM or to the meshless model.

Finally, in Fig. 4, verification results of the model described in Sect. 4.4 are presented for the study of suction caissons, comparing results previously presented by Liingaard et al. (2007) for the same problem. Specifically, horizontal and rocking impedances of suction caissons with slenderness ratios $L/D = 1/4$, 1 and 2 are shown. The viscoelastic soil has a shear modulus of 1 MPa, Poisson coefficient 1/3, density 1000 kg/m^3 and a hysteretic damping factor of 0.025. The suction caisson is made of steel with Young modulus 210 GPa, Poisson coefficient 0.25 and hysteretic hysteretic factor 0.01 (considered massless). The caisson radius is $D/2 = 5$ m, and the skirt thickness is $t = 5$ cm. The results are made dimensionless with respect to the static values and are very close to the functions used as a reference in this case.

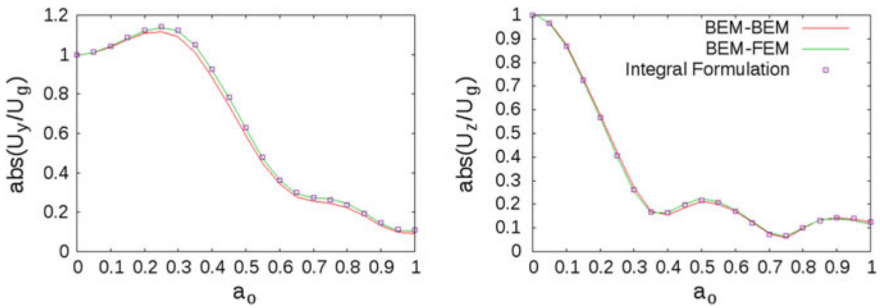

Fig. 3 Kinematic response at the head of a simple pile to S waves (left) and P waves (right) obtained through different models

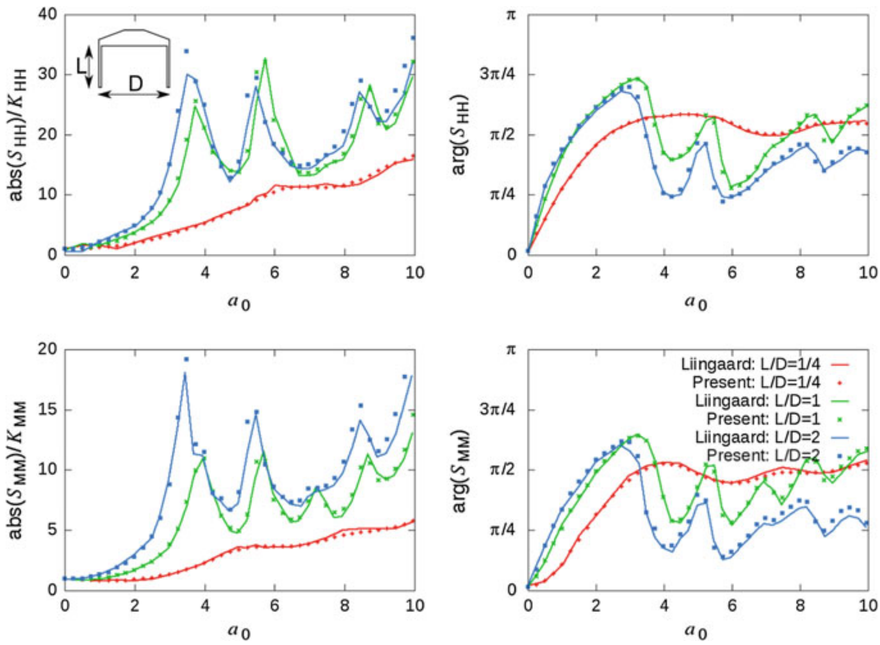

Fig. 4 Horizontal (up) and rocking (down) impedances of a suction caisson in viscoelastic soil for different slenderness ratios

6 Conclusions

This communication emphasizes the need to develop advanced computational models for estimating the dynamic response of offshore wind turbine foundations. A state of the art revision of the most commonly used models in the design and dimensioning of these elements is presented, highlighting their application fields, virtues and shortcomings. Four different numerical models developed by the authors have been succinctly described, highlighting different abilities such as the capacity of modeling the actual foundation geometry, considering media of a poroelastic nature, or studying the dynamic response of pile or suction caisson foundations with coupled models of finite elements and boundary elements, or finite elements and integral formulations of the half-space, which allows to drastically reduce the number of degrees of freedom (when compared to pure BEM multidomain or finite element models) without reducing the validity of the results.

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