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# On the foundation damping evaluation for monopile supported offshore wind turbines

## Évaluation de l'amortissement de fondation pour les éoliennes offshore supportées par monopieu

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ABSTRACT: The majority of offshore structures is installed in non-seismic areas and thus the corresponding design guidelines are mainly focused on static or pseudo-static actions and overlook issues relevant to seismic actions. One such issue is the contribution of the foundation damping to the overall seismic system response. In view of the expanding installation of offshore wind turbines (OWT) in earthquake-prone areas, it has become necessary to take consistently into account the effect of the additional energy dissipation in the foundation soil, so that the structural system is not over-designed at the expense of construction cost. The effect of the soil-foundation-structure interaction and its impact on the dynamic response of the structure (natural period elongation, damping increase) under seismic loading are investigated herein for a typical 5-MW OWT with monopile foundation, following the "beam on nonlinear Winkler foundation" numerical methodology. The foundation damping and the period elongation are estimated from the free vibration response of the structure, following the end of seismic loading. The accuracy and the limitations of this technique are commented in relation to the displacement amplitude, of the tower and the foundation, during and after shaking.

RÉSUMÉ : La majorité des structures offshore est installée dans des zones non-sismiques et, par conséquent, les directives de conception correspondantes sont principalement axées sur les actions statiques ou pseudo-statiques et négligent les problèmes liés aux actions sismiques. L'un de ces problèmes est la contribution de l'amortissement des fondations à la réponse sismique du système globale. Compte tenu de l'installation d'éoliennes offshore (OWT) dans régions sismiques, il est devenu nécessaire de prendre en compte systématiquement l'effet de la dissipation d'énergie supplémentaire dans le sol de fondation, afin que le système structurel ne soit pas surdimensionné, au détriment du coût de construction. L'effet de l'interaction sol-fondation-structure et son impact sur la réponse dynamique de la structure (allongement de la période naturelle, augmentation de l'amortissement) sous charge sismique sont étudiés ici pour un OWT typique de 5 MW avec fondation monopieu, suivant la "poutre sur fondation Winkler non-linéaire" méthodologie numérique. L'amortissement de la fondation et l'allongement de la période sont estimés à partir de la réponse vibratoire libre de la structure après élimination de la charge sismique. La précision et les limites de cette technique sont commentées en fonction de l'amplitude de déplacement, au pylône et à la fondation, pendant et après le secouage.

KEYWORDS: offshore foundations; seismic design; foundation damping.

## 1 INTRODUCTION

Offshore wind turbines (OWT) constitute an alternative and environmentally friendly energy production source which is rapidly expanding the last years. The majority of offshore wind turbines, like all other offshore structures (e.g. oil drilling platforms), in general, are installed in non-seismic areas (e.g. North Sea, Gulf of Mexico) and as a result the associated research has focused on non-seismic actions. The research findings have been incorporated into various Design Codes, such as API (2014) and DNV (2013), which describe analytical methodologies for the static design of foundation piles in cohesive and cohesionless soils as well as methodologies for the estimation of distributed (vertical and horizontal) spring constants along the pile.

The current design practice, as incorporated in the aforementioned Design Codes, is to overlook issues relevant to seismic actions. One such issue is the contribution of the foundation damping to the overall seismic system response. Nevertheless, in view of the expanding installation of OWT in earthquake-prone areas (e.g. Mediterranean Sea, California, Japan, Taiwan), it is necessary to study this effect, as it is directly related to a cost-effective design of the OWTs.

In more detail, high frequency seismic actions would lead to the development of significant energy loss in the foundation soil, due to: (a) radiation damping, which is caused by the propagation of the waves generated by the vibration of the foundation into the surrounding ground and is frequency dependent, and (b) hysteretic damping, which is attributed to the energy loss during each dynamic loading cycle due to the nonlinear – hysteretic behavior of the ground and is independent of frequency but varies with the shear strain level of the ground. Ignoring these sources of foundation damping is not always technically and economically acceptable, given that it may lead to overdesign of the wind turbine superstructure and increase significantly the overall construction cost of the project.

According to the literature, the contribution of the foundation in the overall damping of the OWT for non-seismic environmental loads (i.e. from wind and waves) ranges between 0.17 - 1.50 % (e.g. Tarp-Johansen et al. 2009, Caswell et al. 2015, Chen & Daffour 2018). In addition, incorporation of the foundation damping on the system response led to reduction of the inertial forces in the OWT by 10 - 15 % (e.g. Aasen 2016, Kaynia 2019). However, the effect of foundation damping can become more prominent in the case of seismic loading.

In extend of the above, the effect of the soil-foundation-structure interaction, and especially of the foundation damping, on the seismic response of the superstructure (natural period elongation, damping increase) are investigated herein for a typical NREL 5-MW OWT (Jonkman et al. 2009) with monopile foundation. The foundation is simulated via the commonly used "beam on nonlinear Winkler foundation" numerical methodology, where nonlinear soil springs and dashpots are distributed along the pile. The foundation damping and the period elongation are estimated based on the free vibration response of the structure, following the end of seismic loading. The accuracy and the limitations of this technique are extensively discussed, with emphasis on the effect of the variable displacement amplitude during the co-seismic and the post-seismic stages of the system response.

## 2 METHODOLOGY OUTLINE

### 2.1 *Foundation design*

As mentioned above, the effect of the soil-foundation-structure interaction and its impact on the dynamic response of the structure (natural period elongation, damping increase) under seismic loading are investigated herein for a typical NREL 5-MW OWT (Jonkman et al. 2009) with monopile foundation. The height and the diameter of the OWT tower are equal to 90 m and 6 m respectively, while the water depth was equal to $h_w = 30$ m. The mass of the OWT is equal to 360 tn and the mass of the tower is approximately 340 tn.

For the purposes of the present study, four typical soil profiles that cover the basic soil types commonly encountered in offshore sites were considered: two for the case of non-cohesive seabed (with sand of medium density and dense sand) and two for the case of cohesive seabed (with soft and stiff clay). For the medium density sand, the relative density is assumed equal to $D_r = 50\,\%$, the friction angle to $\varphi = 33\,^o$, while the respective values for the dense sand are $D_r = 75\,\%$ and $\varphi = 40\,^o$. For the case of the soft clay, the upper 15 m were assumed to be overconsolidated with constant undrained shear strength equal to $c_u = 37.5$ kPa, whereas for the stiff clay, the upper 60 m were overconsolidated with $c_u = 150$ kPa. In both cohesive profiles, the foundation soil at larger depths was assumed to be normally consolidated and the $c_u$ values increase linearly with depth (step: $\Delta c_u = 25$ kPa / m).

The first step for analyzing the OWT was the static design of its foundation for vertical as well as horizontal loading for all the examined soil profiles. The vertical loads come from the weight of the OWT, its tower and its foundation, while the lateral loads are caused by wave and wind action. Based on a literature review for typical environmental loads on OWT of this size (e.g. Bhattacharya et al. 2017), the wind loading was equal to 1.2 MN, applied at the OWT rotor level, and the wave loading was equal to 4.8 MN, applied at a depth of $3h_w/4$ from the seabed.

The design for axial loading was performed according to the API (2014) and DNV (2013) recommendations, while the design against lateral loading was performed following the Broms (1964a&b) methodologies. The foundation pile properties (i.e. outer diameter *D*, wall thickness *t* and length *L*) that fulfill the static axial and lateral loading requirements are summarized in Table 1, separately for each soil profile.

Table 1. Foundation pile properties for each soil profile

| Soil Profile | *D (m)* | *t (mm)* | *L (m)* |
|---|---|---|---|
| Medium Sand | 6 | 70 | 28 |
| Dense Sand | 6 | 70 | 23 |
| Soft Clay | 6 | 70 | 36 |
| Stiff Clay | 6 | 70 | 22 |

### 2.2 *Numerical modelling*

The pile foundation-wind turbine interaction was examined via nonlinear numerical analyses using the Finite Element method and the commercial computer program Ansys v 17.2. The pile and the OWT tower were discretized into 2-noded isoparametric, 3-dimensional elastic pipe elements, of 2 m length each, featuring the mechanical characteristics of a hollow cylinder with $D = 6$ m outer diameter and $t = 70$ mm wall thickness. Eight integration points are defined along the perimeter of each cross-section and the strains and stresses are computed at these eight points, taking into account the contribution of axial forces and bending moments. The steel pile and the tower were considered elastic with Young's Modulus equal to $E = 210$ GPa, Poisson's ratio $v = 0.30$, density 7.85 Mg/m$^3$ and Rayleigh damping of 3 %. A concentrated mass of 360 Mg was added in the tower's top to account for the weight of the OWT.

Two different systems were analyzed: *(i)* a rigid system, where only the OWT is modelled (Figure 1a), and *(ii)* the complete soil-structure interaction system (SSI system) which includes both the OWT and the monopile (Figure 1b). In the rigid system, the tower is fixed on the ground surface and the seismic shaking is applied as a displacement time-history at the base of the tower.

In the latter case, the monopile was simulated using the "Beam on nonlinear Winkler foundation (BNWF)" approach and, in particular, a set of parallel nonlinear spring and dashpot was attached to each pile node. The spring stiffness was obtained from the static p-y curves (*p*: lateral resistance and *y*: lateral deflection) proposed by API (2014) for monotonic loading, while the Masing (1926) rule was adopted in order to simulate the hysteretic response during unloading - reloading. The dashpot coefficients representing radiation soil damping were calculated following the methodology proposed by Gazetas & Dobry (1984).

The seismic loading is applied at the free end of the springs and dashpots. The corresponding free-field displacement time-histories at each depth were estimated by conducting equivalent linear 1-D wave propagation analyses (Schnabel et al. 1972) in order to deconvolute the seismic motion that was considered for the ground surface. Detailed information regarding the numerical modelling can be found in Papadopoulou & Vallianatou (2020).

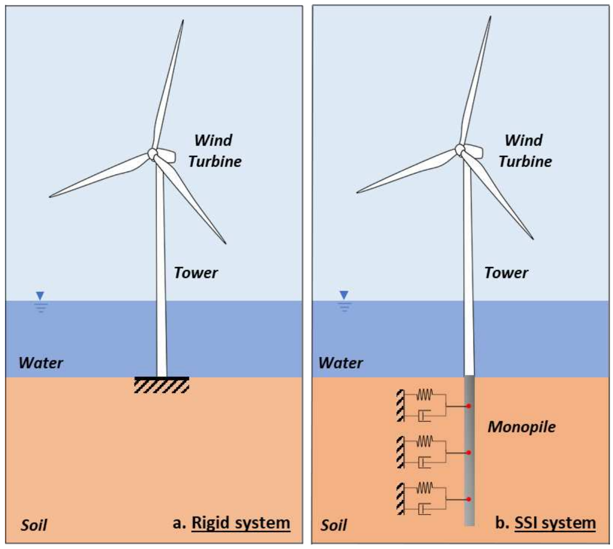


Figure 1. Illustration of the (a) rigid and (b) soil-structure interaction (SSI) wind turbine system.

All numerical analyses were conducted in two stages. In the first stage, the wind turbine was subjected to 15 cycles of forced harmonic vibration with excitation period $T_{exc} = 0.5$ sec. In the first 5 loading cycles, the peak ground acceleration values are gradually increasing and then stabilize to the value $PGA = 0.50$ g (Figure 2d) for 10 more loading cycles. In the second stage, the system (either rigid or SSI) was allowed to vibrate freely in order to estimate the natural frequency as well as the total damping of the vibrating system.

A parametric investigation was conducted for: *(i)* the same ground profile (medium density sand), but more stiff wind turbines, and *(ii)* the same wind turbine but for different ground profiles: dense sand, soft and stiff clay. All parametric analyses were performed for both the rigid and the SSI system. In addition, the analyses of the SSI system were repeated with the options of *(i)* "only hysteretic damping", *(ii)* "only radiation damping" and *(iii)* "no foundation damping", i.e. for nonlinear elastic Winkler springs and no dashpots. Thus, a total number of 35 nonlinear Finite Elements analyses was finally conducted in order to examine the effect of the above aspects of the coupled OWT-pile-soil response.

## 3 NUMERICAL ANALYSIS RESULTS

### 3.1 *Baseline analysis results*

The displacement and acceleration time-histories, at the top of the OWT tower and at the ground surface, for the baseline numerical analysis of the SSI system founded in medium sand are depicted in Figure 2. Note that in this analysis both the hysteretic spring and the radiation dashpot options were incorporated ("total foundation damping"). It is observed that during the forced vibration (seismic) loading stage, both the displacement and the acceleration amplitude on the top of the tower are significantly de-amplified. In particular, the $PGA = 0.50$ g on the ground surface has reduced to an acceleration amplitude of $a_{max} = 0.10$ g on the tower top.

The eigenperiod $T_{str}$ as well as the total damping ratio $\zeta$ of the system can be estimated from the displacement time-history on the tower top during the free vibration loading stage (i.e. for time $t > 7.5$sec), as shown in Figure 3. In particular, the damping ratio can be estimated from Equation (1) using the displacement amplitude of two consecutive peaks ($u_i$ and $u_{i+1}$), as follows:

$$\zeta = \frac{1}{\sqrt{1 + \left[\frac{2\pi}{ln(u_i/u_{i+1})}\right]^2}} \quad (1)$$

Following this methodology, the values of the acceleration amplitude during the forced vibration stage $a_{max}$, the eigenperiod $T_{str}$ and the total damping ratio $\zeta$ of the system were calculated in all numerical analyses in this study.

Table 2 summarizes the $a_{max}$, $T_{str}$ and $\zeta$ values for the rigid system as well as for the four different scenarios that are considered for the SSI system (i.e. "total foundation damping", "only radiation damping", "only hysteretic damping" and "no foundation damping"). It is observed that incorporation of the complete SSI system in the analysis had beneficial effect of the OWT, as the $a_{max}$ value was reduced from $a_{max} = 0.22$ g for the rigid system to $a_{max} = 0.10$ g for the SSI system. In addition, the eigenperiod and the damping ratio were increased by 37 % (from $T_{rigid} = 2.16$ sec to $T_{SSI} = 2.95$ sec) and 97 % (from $\zeta_{rigid} = 3.0$ % to $\zeta_{SSI} = 5.9$ %) respectively.

Regarding the contribution of each damping source (hysteretic or radiation), it is observed that the values of $a_{max}$ and $T_{str}$ are practically constant in all SSI systems, even in the case that there are no sources of foundation damping in the system. Hence, the beneficial effect in these values is attributed to the addition of the pile as a structural element in the system and not to the response of the surrounding soil. In more detail, with the addition of the pile, the system became more flexible (from $T_{str} / T_{exc} = 2.16 / 0.50 = 4.3$ to $T_{str} / T_{exc} = 2.96 / 0.50 = 5.9$) and led to the reduction of the inertial forces in the OWT. In this period range, which is way far from resonance, damping has practically no effect on the system response.

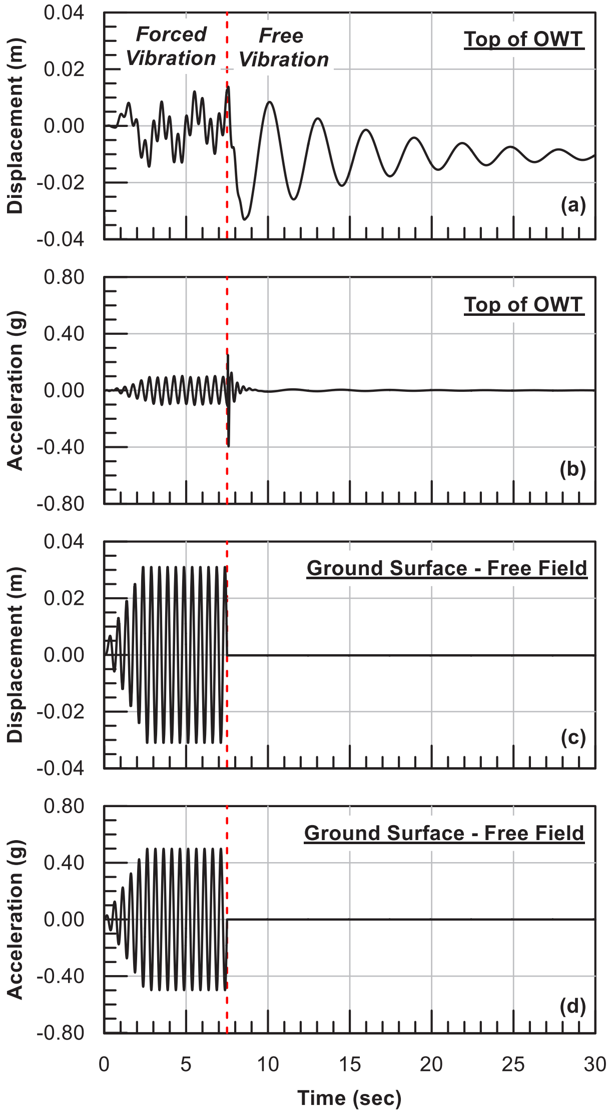


Figure 2. (a) Displacement and (b) acceleration time-histories at the top of the OWT tower as well as (c) displacement and (d) acceleration time-histories at the ground surface for the baseline case of the SSI system in medium sand.

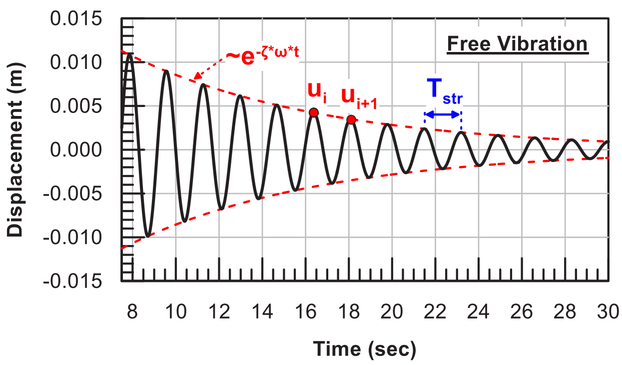


Figure 3. Example for the estimation of the eigenperiod and the equivalent viscous damping ratio from the free vibration of the tower – monopile system.

Regarding the effect of foundation damping, it is observed that the radiation damping is practically the only source that contributes to the system, as the corresponding damping value ($\zeta = 5.89\,\%$) is practically the same with the total foundation damping for the system with both hysteretic springs and radiation dashpots ($\zeta = 5.90\,\%$). On the other hand, the hysteretic damping has practically no contribution on the system as the damping ratio when only hysteretic springs are considered is almost equal ($\zeta = 3.17\,\%$) to the case without any foundation damping source in the system ($\zeta = 3.14\,\%$). As it will be shown in the following paragraphs, this observation can be attributed to the small displacement amplitude during the free vibration analysis stage and highlights one of the main limitations of the "free vibration" technique for the estimation of the total damping of the OWT-foundation system.

It is also noted that the small difference between the damping ratio of the SSI system without any foundation damping and the rigid system (i.e. $\zeta = 3.14\,\%$ versus $\zeta = 3.00\,\%$) can be attributed to the Rayleigh damping formulation which is not constant but depends on the frequency. Finally, the contribution of the foundation damping to the overall damping of the system is $\zeta_f = 5.9 - 3.14 = 2.76\,\%$. It is observed that this damping ratio value is significantly higher than the corresponding damping ratio values for the foundation that were found in literature (i.e. $\zeta = 0.17 - 1.50\,\%$) for environmental loading conditions (i.e. wind and waves).

Table 2. Result summary for the baseline case

| System | $a_{max}$ (g) | $T_{str}$ (sec) | $\zeta$ (%) |
|---|---|---|---|
| Rigid | 0.22 | 2.16 | 3.00 |
| SSI - total foundation damping | 0.10 | 2.95 | 5.90 |
| SSI - only radiation damping | 0.10 | 2.94 | 5.89 |
| SSI - only hysteretic damping | 0.10 | 2.96 | 3.17 |
| SSI - no foundation damping | 0.09 | 2.96 | 3.14 |

### 3.2 *Effect of tower stiffness*

To study the effect of the superstructure properties on the soil-structure-interaction of the OWT, the baseline case (i.e. medium density sand) with tower stiffness $K_o$ was repeated for stiffness values $K/K_o = 3$, 9 and 27, or equivalently for tower heights equal to 2/3, 1/2 and 1/3 of the initial tower height of 90 m. Table 3 summarizes the $a_{max}$, $T_{str}$ and $\zeta$ estimates for the "total damping systems" (i.e. for both radiation and hysteretic damping) and $K/K_o = 1 - 27$. In addition, Figure 4 presents the variation of *(i)* the acceleration amplitude ratio $a_{max,SSI} / a_{max,rigid}$, *(ii)* the eigenperiod elongation ratio $T_{SSI} / T_{rigid}$ and *(iii)* the damping ratio $\zeta$ with the $T_{str} / T_{exc}$ ratio for the examined $K/K_o$ ratio values and all the damping scenarios for the SSI system.

Table 3. Result summary for different tower stiffnesses

| $K/K_o$ | $a_{max}$ (g) | $T_{str}$ (sec) | $T_{str}/T_{exc}$ | $\zeta$ (%) |
|---|---|---|---|---|
| 1 | 0.10 | 2.95 | 5.9 | 5.90 |
| 3 | 0.12 | 1.91 | 3.8 | 11.03 |
| 9 | 0.30 | 1.41 | 2.8 | 22.37 |
| 27 | 0.92 | 1.26 | 2.5 | 35.55 |

It is observed that the tower stiffness has no systematic effect on the reduction of the $a_{max}$ values when the soil-structure-interaction effects are considered, as the variation of the $a_{max,SSI} / a_{max,rigid}$ with $T_{str} / T_{exc}$ is limited (Figure 4a). On the other hand, the contribution of the foundation on the elongation of the natural period, as well as on the increase of the total damping ratio, becomes more prominent as the system moves towards resonance (i.e. as the $T_{str} / T_{exc}$ values are reduced towards unity). It is also observed for the stiffer tower (i.e. $K/K_o = 27$) the eigenperiod for the SSI system becomes almost triple, while the corresponding damping ratio has increased to 35.6 %, i.e. to a much higher value than those found in the literature.

Finally, the contribution of each foundation damping component is similar to that described previously in Section 3.1. Namely, that foundation damping is attributed only to the radiation damping and the effect of hysteretic damping component is negligible (Figure 4c). However, it should be noted that the contribution of either component on the reduction of the $a_{max}$ values during the seismic shaking is comparable (Figure 4a).

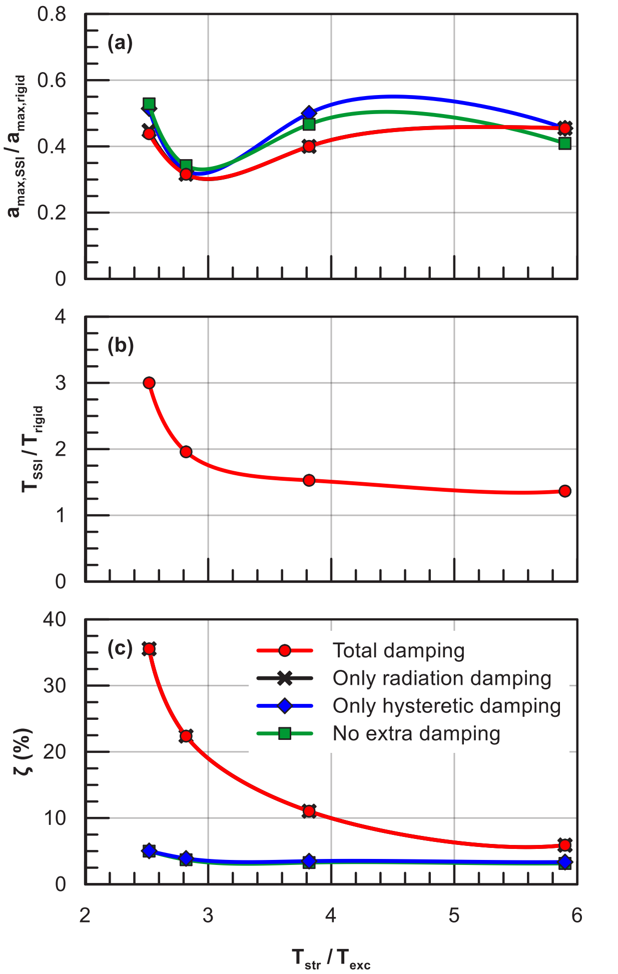


Figure 4. Variation of (a) acceleration amplitude ratio $a_{max,SSI} / a_{max,rigid}$, (b) eigenperiod elongation ratio $T_{SSI} / T_{rigid}$ and (c) damping ratio $\zeta$ with $T_{str} / T_{exc}$ ratio for medium sand soil profile and $K/K_o = 1 - 27$.

### 3.3 *Effect of soil type*

The effect of soil type on the soil-structure-interaction of the OWT was examined for the case of $K/K_o = 3$. Table 4 summarizes the values of the $a_{max}$, $T_{str}$ and $\zeta$ obtained for the "total damping system" in the four different soil profiles. It is observed that the soil type has a generally small impact on the results. In more detail, the incorporation of the SSI effect in sand profiles leads to an acceleration amplitude reduction of $a_{max,SSI} / a_{max,rigid} = 0.40 - 0.43$ for medium and dense sand respectively, while for the clayey seabed the respective reduction is $a_{max,SSI} / a_{max,rigid} = 0.53 - 0.63$. The elongation of the eigenperiod is practically constant in each soil type, and equal to $T_{SSI} / T_{rigid} = 1.46 - 1.53$ for the sandy seabed and $T_{SSI} / T_{rigid} = 1.58 - 1.64$ for the clayey one. Finally, according to Table 4, the foundation damping reduces as the sand densifies (i.e. from $\zeta = 11.03\,\%$ to $\zeta = 7.52\,\%$), while the shear resistance of the clay has a minor effect on the damping values ($\zeta = 13.77$ - $14.51\,\%$).

Table 4. Result summary for different soil profiles

| Soil Profile | $a_{max}$ (g) | $T_{str}$ (sec) | $\zeta$ (%) |
|---|---|---|---|
| Medium sand | 0.12 | 1.91 | 11.03 |
| Dense sand | 0.13 | 1.83 | 7.52 |
| Soft clay | 0.16 | 1.97 | 13.77 |
| Stiff clay | 0.19 | 2.05 | 14.51 |

## 4 EVALUATION OF THE FREE VIBRATION TECHNIQUE

As mentioned in the previous paragraphs, the only source of foundation damping in the performed numerical analyses is essentially the radiation of seismic waves, while the hysteretic soil behavior does not seem to contribute to the overall foundation damping. On the other hand, it has been shown that the contribution of each component in the reduction of the acceleration amplitude on the OWT during the seismic shaking is comparable. To further investigate this inconsistency, the variation of the maximum horizontal displacements with depth during the forced vibration stage $y_{max}^{forced}$ (i.e. for $t < 7.5$ sec) and during the free vibration stage $y_{max}^{free}$ (i.e. for $t > 7.5$ sec) are compared in Figure 5 for the case of a medium sand with $K / K_o = 3$. The variation of the yield displacement of the nonlinear p-y curves $y_o^{API}$ is also presented in the same figure.

Thus, it becomes evident that the displacement amplitude during the free vibration stage is significantly smaller than the corresponding values during the forced vibration stage. Consequently, the estimation of hysteretic damping is significantly underpredicted as the response during the free vibration phase is practically linear, given that the yield displacements are more than an order of magnitude larger than the applied ones. This observation explains why the contribution of the hysteretic damping in the previously presented analyses is negligible, but most importantly highlights the limitations of using the free vibration technique for the damping estimation.

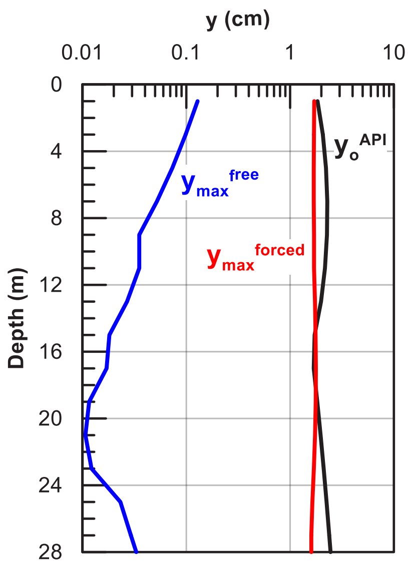


Figure 5. Variation with depth of the maximum horizontal displacements during the forced and the free vibration as well as the yield displacement of the lateral p-y curves $y_o^{API}$.

Coming next to the radiation damping evaluation, Figure 6 shows the variation with vibration frequency of the dashpot coefficient $C$, normalized against the theoretical value $C_R$ for high frequencies adopted in the numerical analyses (Gazetas & Dobry 1984). It is thus observed that the free vibration frequency for the same analysis falls within the area of the cut-off frequencies, for which a serious reduction of the theoretical value of the dashpot coefficients $C_R$ is required. On the other hand, no reduction is required for the forced vibration period (i.e. with the seismic excitation frequency). This observation deserves to be further explored as it may explain, partially at least, why the estimated damping values are consistently higher than those found in the literature.

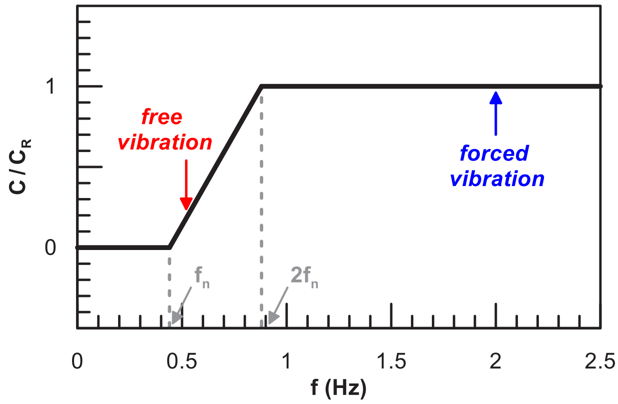


Figure 6. Correction factor of the dashpot coefficient for excitation frequencies in the cut-off frequency range (Gazetas & Dobry 1984).

## 5 CONCLUSIONS

The effect of the soil-foundation-structure interaction and its impact on the dynamic response of the structure (natural period elongation, damping increase) under seismic loading were investigated herein for a typical 5-MW OWT with monopile foundation following the “beam on nonlinear Winkler foundation” numerical methodology. The main conclusions that were derived from this study are:

(a) Including the foundation response in the computations leads to reduction of the mass (spectral) acceleration. This is mostly due to the contribution of the pile, as an additional structural element, which is more important than the contribution of the surrounding soil, and essentially controls the response of the wind turbine. The otherwise secondary contributions of the energy dissipation due to the hysteretic behavior of the springs (hysteretic damping) and due to the viscous dampers (radiation damping) are comparable.

(b) In all parametric analyses, the addition of the monopile foundation leads to increase of the vibration eigenperiod of the system. The eigenperiod values for a full system of soil springs and dampers are practically the same with the values obtained for a system with soil dampers only, for a system of soil springs only or for a system without any springs or dampers.

(c) The addition of the monopile foundation pile led to an increase of the energy dissipation, described by the equivalent viscous damping ratio. The increase in the energy damping comes almost exclusively from radiation damping, as the hysteretic damping is practically zero.

(d) The displacement amplitude during the free vibration stage is significantly lower than the amplitude during the forced vibration stage and remains well below the yield displacement of the soil springs. This explains the practically zero contribution of the hysteretic damping to the free vibration of the system and also reveals a significant limitation of total system damping estimates using the “free vibration” method.

(e) Numerically computed damping values are significantly higher than those reported in the literature. This discrepancy may be due to the fact that the majority of the values at the literature has been derived for wind and wave loading of the wind turbine, in contrast to the seismic loading, which is examined in this study. Another possible reason is that the theoretical value of the dashpot coefficients used in the numerical analyses should be somewhat reduced because the free vibration frequencies of the system fall within the area of the cut-off frequencies for radiation damping.

It is finally noted that the research is ongoing aiming to estimate the effect of the soil-foundation-structure interaction during seismic shaking utilizing other methods for the estimation of the foundation damping (e.g. by the response amplitude at resonance). Additional aspects of the numerical simulation that will be examined include the shape of the nonlinear hysteretic p-y curves (e.g. monopile specific instead of API recommended), and the spring-dashpot configuration (e.g. "in series" rather than "in parallel").

## 6 REFERENCES


Aasen S. 2016. *Soil-structure interaction modelling for an offshore wind turbine with monopile foundation*. Master Thesis, Norwegian University of Life Sciences.

Ansys 2016. *Ansys Multiphysics Release 17.2*. Ansys Inc and Ansys Europe, Ltd.

API 2014. *Geotechnical and Foundation Design Considerations, ANSI/API RP 2GEO*. American Petroleum Institute, Washington, D.C.

Bhattacharya S., Nikitas G., Arany L., and Nikitas N. 2017. Soil-Structure Interactions (SSI) for Offshore Wind Turbines. *Engineering & Technology Reference*, 1(1) 1–23.

Broms B.B. 1964a. The Lateral Resistance of Piles in Cohesive Soils. *Journal of the Soil Mechanics and Foundations Divisions*, ASCE, 90(3), 27-63.

Broms B.B. 1964b. Lateral Resistance of Piles in Cohesionless Soils. *Journal of the Soil Mechanics and Foundations Divisions*, ASCE, 90(3), 126-156.

Carswell W., Johansson J., Løvholt F., Arwade S.R., Madshus C., DeGroot D.J. and Myers A.T. 2015. Foundation damping and the dynamics of offshore wind turbine monopiles. Renewable Energy, 80, 724-736.

Chen C. and Duffour P. 2018. Modelling damping sources in monopile - supported offshore wind turbines. *Wind Energy*, 21, 1121-1140.

DNV. 2013. *Design of Offshore Wind Turbine Structures; Offshore Standard*. Det Norske Veritas, 214.

Gazetas G. and Dobry R. 1984. Horizontal Response of Piles in Layered Soils. *Journal of Geotechnical Engineering*, 110(1), 20-40.

Jonkman J., Butterfield S. Musial W. and Scott G. 2009. *Definition of a 5-MW reference wind turbine for offshore system development.* National Renewable Energy Laboratory, Colorado, USA.

Kaynia A.M. 2019. Seismic considerations in design of offshore wind turbines. *Soil Dynamics and Earthquake Engineering*, 124, 399-407.

Masing G. 1926. Eiganspannungen und Verfestigung beim Messing. *2nd International Congress of Applied Mechanics*, Zurich, 332-335.

Papadopoulou K. and Vallianatou D. 2020. *Dynamic Interaction of Offshore Wind Turbine with Monopile Foundation*, Diploma Thesis, NTUA, Athens (in Greek).

Schnabel P.B., Lysmer J. and Seed H.B. 1972. *SHAKE: A computer program for earthquake response analysis of horizontally layered sites*. Rep. No. UCB/EERC-72/12, Earthquake Engineering Research Center, University of California at Berkeley.

Tarp-Johansen N.J., Energy D., Meyers A., Andersen L., and Damgaard Christensen E. 2009. Comparing Sources of Damping of Cross-Wind Motion. *Proc of the European Wind Energy Conference & Exhibition*. Stockholm, Sweden.