Assessing the small-strain soil stiffness for offshore wind turbines based on in situ seismic measurements

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ABSTRACT: In this contribution, in situ seismic measurements are used to derive the small-strain shear modulus of soil as input for two soil-structure interaction (SSI) models to assess the initial soil stiffness for offshore wind turbine foundations. This stiffness has a defining influence on the first natural frequency of offshore wind turbines (OWTs), which is one of the most important design parameters of these structures. The fundamental natural frequency as measured on installed OWTs is significantly higher than its designed value, and it is expected that the explanation for this can be found in the currently adopted modeling of soil-structure interaction. In this paper a method is suggested to improve the accuracy of estimating the small-strain soil stiffness.

The field data used in this study is measured with a seismic cone penetration test. A method is suggested to identify the shear moduli, which together with the measured in situ mass densities and estimated Poisson’s ratios are input for two static 3D SSI models. The first model is a linear elastic finite element model of a half-space of solids attached to a pile (shell). This is a straightforward and fast approach in which a static horizontal force and a bending moment are applied at the top of the pile. The second model extends the first one by introducing contact elements at the interface between pile and soil, where a contraction of the soil towards the pile is not allowed and maximum friction forces on the interface can be prescribed. Finally a method is suggested to translate the global response of a 3D model into an engineering model of a 1D beam laterally supported by uncoupled distributed springs.

When comparing the deflections, those derived with the global 3D models, are smaller than deflections derived with the p-y curve design code. The deflections of the global models behave more rigidly, and especially the upper 10m deflections are smaller. The presented work is a small part of the research that still needs to be done before any conclusion can be drawn.

KEY WORDS: Small-strain, initial soil stiffness; Offshore wind turbine monopile foundations; Natural frequency; Seismic measurements; FEM.

1 INTRODUCTION

In order to become an economically competitive energy source, offshore generated wind power needs to lower its levelized cost of energy. Less conservatism in design of the support structure (SStr) of offshore wind turbines (OWT) can contribute to this by decreasing the capex and/or increasing the energy yield. The latter is achieved by a longer certified lifetime of SStrs. Both can be attained by increased knowledge of soil-structure interaction (SSI): in depth understanding of both the stiffness and damping occurring in the interaction of the monopile and the soil. A secondary but possibly large economic effect is the increased bankability of a wind farm, because of more certainty in this defining SSI domain. SStrs and Financials (loans) are the main cost drivers in an offshore wind farm project.

Over the recent years, measured natural frequencies of installed OWTs have been found to be higher than designed for. Apart from being a waste of expensive steel—often larger diameter foundation piles are employed for reaching the desired fundamental natural frequency—also more fatigue damage is accumulated than anticipated, because of the higher amount of cycles during its lifetime. The industry is turning towards SSI for finding the cause of this discrepancy in frequency between design and real life. A higher measured frequency may suggest an under-estimation of the stiffness of the soil against the pile motion.

The currently available engineering approach which is prescribed in the standards [1] is called the p-y curve method. In this approach, the monopile is laterally supported by uncoupled distributed non-linear springs. The displacement (y)-dependent magnitude of these springs is based on empirical relations found for slender, small-diameter piles in both clay [2] and sand [3]. However, most probably the small ratio of embedded length L over diameter D that is currently employed in the industry, invokes a fundamentally different soil reaction than for which the p-y curves were originally calibrated. The typical monopile bending can be characterized by being more rigid than the flexible piles for which the p-y curves are to be used. The bending behavior of monopiles conjures different soil-reaction mechanisms than slender flexible piles. Especially the fact that the initial stiffness of these curves is dictated to increase linearly with depth, and its magnitude to be independent of the diameter of the pile, is erroneous [4].

The initial stiffness has a large effect on the natural frequency of the OWT: the soil responds linearly for most of the endured vibrational amplitudes of the foundation, and it is
this small-strain stiffness that defines the modal properties of the SStr. Therefore, this initial part of the reaction, often called initial subgrade modulus \( k_{s,0} \) or \( E_{py}^* \) [N/m²], has been subject of attention of several researchers [5], [6], [7]. The dimension of \( k_{s,0} \) can be understood as a discretized spring stiffness over a unit vertical length of the pile, so it is rather [N/m/m]. In the p-y curve formulation \( k_{s,0} \) is calculated by multiplying \( k \), the modulus of subgrade reaction [N/m³] with depth \( z \). \( k \) is determined with the angle of internal friction of sand \( \phi \).

However, seemingly contradictory to what is mentioned above, these referred studies find that the p-y curve method actually over-predicts the initial stiffness of the soil for larger depths. If this were generally the case, it would not explain the higher natural frequencies that are measured. The researchers suggest that a more realistic – taking 3D, global effects into account - variation of the stiffness with depth might rather follow a power law form with a power exponent smaller than 1 [4].

Most often geotechnical tests like the cone penetration tests (CPT) are being performed and borehole samples for laboratory testing are taken. A balanced approach with geophysical (seismic) measurements, with a focus on the low frequency response, would be beneficial. Whereas the CPT measures local resistance and friction along the shaft, a recorded soil wave carries information of the soil characteristics along its entire path, yielding more global parameters of a certain location. This global parameterization of the soil can best be used to predict the response of the foundation. By assessing the wave velocities in the profile, the in situ stiffness is determined, which incorporates for instance oedometric stiffening and saturation effects. This can be considered an advantage over most laboratory tests in which these in situ characters are partially lost.

This paper is aimed at assessing this initial stiffness aspect. An alternative approach is suggested, in which a 3D model is used to calibrate the initial part of the p-y curves which are fit for fast simulation design purposes. The 3D model incorporates global effects and interface friction. The novelty of the approach lies in using in situ seismic measurements to extract the small-strain stiffness of the design location. The material properties identified with the seismic measurements are input to the 3D model. Finally, a translation method is suggested for finding the 1D equivalent stiffness of the 3D response.

2 SEISMIC IN SITU DATA

An in situ soil measurement campaign was carried out at a near shore wind farm. This campaign comprised a geotechnical part including a CPT at each OWT location and 10 boreholes for lab test sampling, followed by a geophysical campaign. At 6 locations around the farm, seismic measurements including SCPTs and a newly developed test designed to capture the low-frequency dynamic response of the soil: the Low Frequency Cone Penetration Tests (LFCPT) were performed. This paper will discuss a frequency independent analysis of the SCPT data.

Here we focus on the seismic data of a location closest to one of the design locations (deepest & softest soil). At this location a depth of 25m was reached, measuring each meter with a dual-phone cone with an interval distance of 0.5m. A hydraulic shear wave hammer was used as excitation device at mudline. Stacking responses of multiple hits for each depth rendered clear shear-wave patterns. To enable automatic picking, the wave arrival time was defined as the maximum peak. An example of the response measured at the cone can be seen in Figure 1.

![Figure 1: Cone response at 4m depth. The wave form is interpreted as a shear wave. The maximum peak (red dot) was defined as the arrival time.](image)

The difference in arrival time between two adjacent receivers – the interval time - is obtained by simply subtracting the arrival time of the upper geophone from the lower geophone. For example, \( \Delta t = t_{32} - t_{31} \) for the third layer in Figure 2. An inverse problem was set up to find the shear wave velocities. Assuming a stratification with 1m layer thickness, the successive layer shear wave velocities can be computed by minimizing an objective function for the observed arrival time. The objective function incorporates the effect of wave refraction through Snell's law.

Confidence about finding the global minimum can be reached by visual inspection of the objective function for the first layers, and is aided by a realistic initial guess of the solution.

A schematic view of the solution method is shown in Figure 2, in which an example is given for the third layer. Geometric relations with their unknowns form the constraining equations of the optimization problem. The amount of equations and unknowns equals 4n+1, where 'n' denotes the layer number.
The determined shear wave velocities are displayed in Figure 3. Next to the velocity profile, the laboratory soil classification is given, which is based on liners retrieved from a borehole at the same location. It can be seen that the velocity increases with depth, which is expected. There are some outliers, and the weaker soils between approximately 15m and 23m depth are reflected in the measured velocities.

![Figure 3: Shear wave velocity profile and Laboratory soil classification from borehole](image)

The in situ soil densities, \( \rho \), were also measured. According to the type of soil, a Poisson's ratio, \( \nu \), was estimated: 0.3 for sand and 0.45 for cohesive material. The Young's modulus, \( E \), was calculated according to

\[
V_s^2 = \frac{E}{2(1+\nu)\rho} = \frac{G}{\rho}
\]

in which \( V_s \) is the shear wave velocity, and \( G \) the shear modulus.

3 MODELLING APPROACH

Two FE models were developed using ANSYS, to try and capture the global (non-local) stiffness effects of the interaction between pile and soil. The models are similar in terms of the soil and the pile, but differ in the way the interface between these two media is modeled. For both models, the pile was modeled with Shell elements, and the soil was modeled using Solid elements. The models are described in the next 2 paragraphs. In paragraph 4.2 a method is proposed to find a 1D equivalent lateral stiffness to the one found with a 3D model.

3.1 Linear elastic FE model

The first model is linear elastic. The soil elements in and outside the pile, are attached to the pile elements, creating the possibility of high shear forces on the interface, and tensile forces between the soil and the pile. However, as the focus is laid upon small-amplitude vibrations, such kinematic - physically incorrect - constraints might not contaminate the results too much. A unit horizontal force of 1 MN and bending moment of 1 MNm was applied to the top of the pile at 5m above mudline. The soil stratification was given a 1m discretization and each layer was assigned the material properties derived from the seismic data with the method described in paragraph 2. Because of the limited depth of the seismic data, the deepest 7m along the pile (from -25 to -32m) was assumed to be one layer, with the same properties as the layer above (-24 to -25m). Geotechnical data indicates that indeed a quite uniform sand layer is present at this depth, however the assumption of constant Young's modulus is clearly conservative, as it is most probably higher due to a larger effective overburden pressure.

![Figure 4: Impression of the FE halfspace model: a pile in stratified soil](image)

<table>
<thead>
<tr>
<th>Table 1. Model Dimensions</th>
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<tbody>
<tr>
<td>Radius [m]</td>
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<td>Pile</td>
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<tr>
<td>Halfspace</td>
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3.2 Elastic FE model including contact condition

The second model is almost identical to the linear elastic model, but includes the use of Contact Elements (CE) at the interface of pile and soil. Gaps between the soil and the pile can now occur, and a maximum friction coefficient, $\mu$ can be defined. If the maximum friction force is exceeded, sliding will occur.

The interface friction angle $\delta$ was taken to be 2/3 of the internal angle of friction of the soil. Then $\mu$ was calculated as $\tan(\delta)$. Averaging this over the depth results in $\mu=0.5$. Three cases were analyzed: firstly a homogeneous soil was assumed with average soil properties and a friction coefficient of 0.5. Then the same friction coefficient was taken, but with the 'real' stratified soil. Thirdly a stratified case with friction coefficient $\mu=0.4$ was calculated for comparison purposes.

Summing up the three cases presented are:
- A homogeneous case in which an average density, Young's modulus and Poisson's ratio was used, and $\mu=0.5$
- A stratified ('realistic') case with $\mu=0.5$
- A stratified case with $\mu=0.4$

With the limited forcing that is applied, the soil 'sticks' to the pile nearly everywhere, except for some sliding in and outside the pile near mudline. This is generally so for the three cases.

4 RESULTS

The deflection shapes and lateral soil reactions from the 3D models are presented in the next paragraph. A method is suggested for finding a local, uncoupled equivalent spring stiffness distribution in paragraph 4.2.

4.1 Deflections and horizontal reactions derived from 3D model

Figure 5 displays the deflection shapes and zoom of the pile-tip displacements for the linear elastic model, the design (PY) code model and the 3 CE cases. The purple line reflects the shape derived when applying the design code [1], and applying double the load which was used for the halfspace models (a horizontal force of 2MN and moment of 2MNm), as this 1D model does not consider half of the symmetric problem. The blue line, 'FE model', is the shape of the linear elastic model described in paragraph 3.1. The other 3 shapes are the cases described in paragraph 3.2 including interface friction (variation) and an average homogeneous soil case.

It can be seen that the p-y curve approach seems to be conservative in estimating the displacements at mudline. The pile modeled in this way behaves more flexibly, and especially the upper 10m (30% of the depth) deflections are significantly larger than the global models predict, resulting in up to 50% larger deflections at mudline. From the stiffness distribution displayed in Figure 7, we can see that the purple line from the p-y curve approach would indeed follow a linearly increasing initial stiffness with depth, despite the soft soil layer between -14m and -23m. This creates high stiffness at greater depth, explaining the rather small 'toe-kick' for this design approach.

The deflection shape of the linear FE model has a larger mudline displacement than the design code, and its toe-kick is the greatest of all models. As previously mentioned, because of the attachment of the soil Solid elements to the Shells of the pile, this model allows higher interface friction forces than the models including a slip possibility.

Further, we see that the shapes of the two cases of contact element FE models with friction coefficients $\mu$ of 0.4 and 0.5 are quite similar. In the zoom of the pile-tip displacements (right panel of Figure 5) it can be seen that the green line with higher allowed interface friction force has a slightly higher toe-kick. Taking into account the shape of the linear model, we observe a trend of higher tip-displacements with higher interface friction. The point where these curves cross zero-displacement is shifted upwards with respect to the blue shape of the linear model. Because of smaller soil pressures near mudline, the models with contact element allow for some slip in the upper region, which might explain the larger mudline deflections.

Finally, the red line reflects the deflection shape when assuming a homogeneous soil with the averaged properties of the true stratified soils, and a friction coefficient of $\mu=0.5$. The shape is quite different which indicates that stratification has a large influence on the response. This can be expected: an averaged stiffness gives significantly smaller displacements at both mudline as at the pile-tip. The presence of a stiffer soil layer (because of averaging) at a shallow depth, and possibly also around the point of rotation, has a large influence on the displacements at mudline and consequently also at the pile-tip. The zero-displacement crossing is identical to the stratified contact models.

![Figure 5: Comparison of embedded pile horizontal deflections, y](image)
From Figure 7 it can be concluded that the initial stiffness from the global 3D models is not the equivalent initial stiffness that is to be used for the 1D engineering models. Due to the singularity effects when dividing the force over (near-) zero displacements, non-physical high stiffnesses are found, and also negative stiffness can occur above the zero-crossing. The latter might be caused by a global effect: soil laterally displaced below the zero-crossing, can increase the lateral stress in the soil above this point due to the Poisson’s effect. A similar high stiffness due to the zero-crossing was also found by Sørensen et al [7]. In the next paragraph, a method is suggested to translate the results found with these – possibly more realistic – 3D global models in a 1D engineering model.

\[
\begin{align*}
\min \left( \sum_{n=1}^{N} (u_{b,n} - u_{t,n})^2 \right) \\
\end{align*}
\]

in which \(n\) is the node number of the beam, and \(N\) the amount of nodes, \(u_b\) is the horizontal displacement of the Timoshenko beam model, and \(u_t\) the target displacement from one of the 3D models. As fitting a deflection shape with a certain stiffness distribution does not have a unique solution, this optimization is performed in a stepwise approach in order to steer the solution to be physically acceptable and within the bounds of the classic engineering beam-on-Winkler foundation model, which does not allow negative stiffness. To this end, the first step is to assume a constant stiffness distribution with depth, and finding the initial stiffness which gives the best fit to the target deflection.

\[
\begin{align*}
k_{x,0}(z) &= K_{11} \\
k_{x,0}(z) &= K_{21} + K_{22} \cdot z \\
k_{x,0}(z) &= K_{31} + K_{32} \cdot z + K_{33} \cdot z^2 
\end{align*}
\]

Since it is a one dimensional minimization problem (only \(K_{11}\) in eq. 4-2 is varied), the solution space can be plotted and it can be visually verified that the global minimum has been found. The resulting deflection and stiffness distribution can be seen in Figure 9, in which also the deflection and initial stiffness distribution is given which is obtained by following the design code. It is clear that a good fit cannot be obtained when assuming constant stiffness with depth.

The second step is assuming a linearly increasing initial stiffness with depth, see eq. 4-3 (partly similar as is done in the DNV p-y curve design code). The (selected) solution space of this 2 dimensional optimization problem is shown in Figure 8. Note that \(K_{11} \neq K_{21} \neq K_{31}\) nor \(K_{22} \neq K_{32}\). The resulting deflection fit and stiffness distribution are displayed in Figure 10. The quality of the fit clearly increases with the order of the polynomial.

In this contribution we aim to only show the principle, by going up to the 2\(^{nd}\) polynomial order (eq. 4-4). The second order deflection fit and stiffness distribution can be seen in Figure 11. Again, the quality of the fit is improved.
CONCLUSIONS AND DISCUSSION

When using the in situ measured small-strain shear modulus in a global 3D model, a first run of analyses indicated smaller mudline displacements than the ‘p-y curve’ approach dictated by the industry design code [1]. This result may be interpreted as being according to expectation. Although researchers’ opinions on this subject deviate [8], the presented method of using in situ determined small-strain properties in a 3D global model was expected to generate higher stiffness. This would explain the higher measured natural frequencies than designed for.

A method is suggested for finding a 1D equivalent stiffness distribution of the obtained global responses. Distributions of higher polynomial order will be approximated, and natural frequency analyses of these cases will be compared with those derived with the design code. Some researchers [4] find lower initial stiffness than the p-y curve approach for greater depths, and higher stiffness for shallow depths. The location of the stiff layers in the depth profile is expected to have a high influence on the natural frequency. This is confirmed by the different deflection shape when considering a homogeneous soil as is done in Figure 6. Obviously, many other causes are not yet examined here. Phenomena like soil stiffening due to pile driving, cyclic stiffening, and possible un-drained soil effects might also cause the higher stiffness.

The presented work is a small part of the investigations that need to be performed. Further research is needed to be able to draw any conclusions.

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REFERENCES