Fatigue life calculation of monopiles for offshore wind turbines using a kinematic hardening soil model

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ABSTRACT: A better assessment of the fatigue life for offshore wind turbines requires more realistic foundation response models, particularly incorporating a better representation of damping. This paper provides a description of how a kinematic hardening model can be applied to the current design method for wind turbine monopiles, installed in sand, to provide improved predictions of the amplitude-dependent material damping derived from the pile-soil response. The proposed method forms a suitable engineering tool for fatigue life calculation of offshore wind turbines.

1 INTRODUCTION

Offshore wind turbines are subjected to large dynamic loads derived from the wind and wave environment. These dynamic loads lead to cyclic stresses, which could induce fatigue failure if not adequately designed for. Minimising the possibility of fatigue failure is achieved by avoiding resonance and increasing the damping on the structure. Measurements from different wind farms have shown that the current design guidelines for monopiles in sand lead to an underprediction of the resonant frequencies as well as the magnitude of modal damping (Kallehave et al. 2012; 2014). To optimise the design of these structures a better prediction of the damping on the modes is needed.

The damping on the structure consists of aerodynamic damping, hydrodynamic damping, structural damping, tower oscillation dampers and soil damping. Of these aspects, the soil damping is the most uncertain. The soil damping can further be divided into radiation damping, material damping and a damping contribution from pore pressure dissipation. Radiation damping is only dominant at high frequency loading and is expected to be relatively low at the dominant loading frequencies (0.1-0.4 Hz) expected in operation. Pore pressure dissipation is not expected to lead to significant damping, making only minor contributions in very permeable sand and gravel soils (Leblanc Thilsted & Tarp-Johansen 2011). Material damping is therefore regarded as the most important damping component for monopile foundations. Material damping can further be subdivided into small-strain and large-strain material damping. At very low strain levels, the damping ratio has an approximately constant value. At higher strains, non-linearity in the stress-strain relationship leads to an increase in material damping at increasing strain amplitude (Zhang et al. 2005).

There are several commercial programs available for calculating the interaction of the wind turbine substructure, the foundation and the rotor dynamics. These programs can be divided into two categories based on the time integration scheme used to calculate the dynamic response. Direct time integration methods, including ADAMS and HAWC2, use an
implicit solver for the full system matrices. This is a rigorous approach, where the dynamic interaction between the different structural parts is approximated well. These methods are computationally expensive though they are becoming more feasible, as the available computing power increases. Modal-based methods perform an eigenvalue analysis to compute the principal modes of vibration of each structural part. The dynamics of the system is then determined using a superposition of these modes. This requires a simplification of the structure, making the methods more efficient computationally compared with the direct time integration methods.

In both methods, the soil reaction is typically modelled as non-linear elastic, leading to no energy dissipation in the foundation. Damping can be added to direct time integration methods using Rayleigh damping, where the damping matrix $\mathbf{C}$ is written as a linear combination of the mass $\mathbf{M}$ and stiffness $\mathbf{K}$ matrix ($\mathbf{C} = \alpha \mathbf{M} + \beta \mathbf{K}$). The coefficients $\alpha$ and $\beta$ are chosen so that the damping is equal to the expected value at two chosen frequencies. However, the damping value at other frequencies is then fixed and may not accurately represent the damping. Modal-based commercial programs add an overall damping value to the main resonance modes of the structural parts. Each mode acts as a single degree of freedom system where this damping value is specified.

To address these shortcomings this paper introduces a kinematic hardening model for the soil reaction, based on the soil reaction curves in the current offshore design methods (API 2010; DNV 2014). The kinematic hardening model can be included in both calculation approaches described above, and leads to the inclusion of large strain material damping from the soil, in the modelling. This removes the need to re-evaluate the soil damping for each load simulation and the consequent tuning of damping coefficients. The proposed method is applied to a simplified simulation of a rotor stop to illustrate the effect of the kinematic hardening model on the structural response.

2 CURRENT DESIGN METHOD

The current offshore pile design method (API, 2010; DNV, 2014) characterises the lateral soil reaction by a series of non-linear ‘$p - y$’ springs along the length of the pile. In this paper, the lateral displacement is denoted $y$ instead of $y$ (for consistency with the current usage in numerical analysis), though the term ‘$p - y$’ is maintained for the method. For sandy soils, these ‘$p - y$’ curves are (Murchison & O’Neill 1983):

$$p(v) = Ap_u \tanh\left(\frac{kz}{Ap_u}v\right)$$

where $A = 0.9$ for cyclic loading and $A = 3.0 - 0.8 \frac{z}{D}$ for static loading, with $z$ the depth below mudline and $D$ the diameter of the monopile. The initial modulus of the soil reaction, $k$, depends on the friction angle, where a distinction is made between saturated and unsaturated sand. The ultimate soil resistance $p_u$ is calculated by:

$$p_u = \min\left\{(C_1 z + C_2 D)yz, C_3 Dyz\right\}$$

where the coefficients $C_1$ to $C_3$ depend on the friction angle and $y$ is the effective unit soil weight.

The contribution of soil damping is typically calculated as described in Cook (1982) and Cook et al. (1982), where the sources of damping on different mode shapes of a single piled platform in the Gulf of Mexico were assessed. The Rayleigh damping coefficients or modal damping values are then calculated, based on the sum of the hydrodynamic damping, structural damping, tower oscillation dampers and soil damping. Aerodynamic damping is implicitly calculated as part of the aerodynamic computation. More recent studies have shown that the method for soil damping results in an underestimation of the actual soil damping (Versteijlen et al. 2011).

3 KINEMATIC HARDENING

Kinematic hardening can be described by the extended Masing rules (Masing 1926). This response is known to be representative of the hysteretic soil response observed in experimental work including cyclic element tests (Pyke 1979). It has also been found to apply to the cyclic response for suction caissons of up to 1 diameter embedment (e.g. Byrne and Houls-
by, 2004) and slender pile foundations (e.g. Hajiali-lue-Bonab et al. 2013) with embedded length of 20 diameters. Further experimental work is needed to fully validate this type of behaviour for the case of monopiles in sand, where typical embedded lengths are 4 to 6 diameters. However, given that this L/D ratio falls within the two cases described above the expectation is that Masing rules will be suitable. Of course there may be complex phenomena that are not yet captured by such rules, including gapping, rattching and stiffness changes. Some of these phenomena can be captured using different foundation models (e.g. Gerolymos & Gazetas 2006; Taciroglu et al. 2006; Boulanger et al. 1999).

When applied to the current design method, the initial loading curve is specified by the ‘$p - y$’ curves. In unloading and reloading branches of the steady-state hysteretic response, the soil reaction is specified by:

$$\frac{p - p_0}{2} = A p_u \tanh \left( \frac{k z - v - v_0}{A p_u} \right)$$

This curve is geometrically similar to the initial loading curve, but factored by 2 and translated to the reversal pressure $p_0$ and displacement $v_0$. To fully describe the full range of loading and unloading behaviour the following two rules also apply (Pyke 1979):

1. The unloading and reloading curves should follow the initial loading curve (backbone curve) if the previous maximum pressure is exceeded.
2. If the current loading or unloading curve intersects the curve described by a previous loading or unloading curve, the pressure-displacement relationship follows the previous curve.

The Masing rules for the non-linear ‘$p - y$’ curves can be reproduced by a series of spring-slider elements as shown in Figure 1. The spring stiffness of an element is denoted $k_i$, the maximum pressure of the slider is denoted $h_i$, with its plastic displacement $v_i^p$. At a given displacement, the soil reaction is given by $p = \sum_{i=1}^{n} k_i(v - v_i^p)$. If the initial estimate of the element reaction, based on the plastic displacement from the previous load step $v_i^{p0}$, exceeds the sliding force $(k_i|v - v_i^{p0}| > h_i)$, the plastic displacement is updated to $v_i^p = v - \text{sgn}(v - v_i^{p0})h_i/k_i$.

![Figure 1. Illustration of the kinematic hardening model.](image)

The shape of the resulting load-displacement response is multi-linear. However, the approximation becomes smoother as the number of elements increases. The spring stiffness and sliding pressure for each element is calibrated to approximate the ‘$p - y$’ curve in the initial loading stage. In Figure 2, the soil reaction is illustrated for a normalised displacement $v^{A p_u/kz}$ going from $0 \rightarrow 1 \rightarrow -1 \rightarrow 2$. The hysteresis loop satisfies the Masing rules and results in energy dissipation from the pile-soil response.

![Figure 2. Soil reaction using the ‘$p - y$’ curves (black) and the kinematic hardening model (grey) for $v^{A p_u/kz} = 0 \rightarrow 1 \rightarrow -1 \rightarrow 2$.](image)
4 Rotor Stop Simulation

The kinematic hardening model is applied to a simplified model of an offshore wind turbine, as shown in Figure 3. The structure consists of a monopile, a transition piece and a tower of which the properties vary linearly over the length. At the top of the substructure, the turbine is modelled as a 4 × 10^5 kg mass with a moment of inertia equal to 7.8 × 10^7 kgm^2. These figures are representative of a 5 MW wind turbine.

The estimated structural parameters (bending stiffness EI, shear stiffness kAG, distributed mass ρA and distributed moment of inertia ρI) are given in Table 1. The model assumes a medium dense sand with a friction angle of 40° and an effective unit weight of 9 kN/m^3. The rotor stop is simulated by slowly building up the load at the nacelle, after which the load is quickly released.

A finite element model is set up for the structure using Timoshenko beam elements (Astley 1992). These elements have 5 degrees of freedom: lateral displacement and rotation at the nodes and a constant shear deformation over the element. The integration over each element is done using 4 Gauss points. The mesh of the structure is given in Figure 3. Below mudline, the elements are laterally supported by the soil reaction as described in the previous sections. This is done rigorously by evaluating the soil stiffness and reaction at the Gauss points. This method requires only a small number of foundation elements to achieve a good approximation of the soil reaction.

Table 1. Properties of the simplified structure.

<table>
<thead>
<tr>
<th>Section</th>
<th>EI [Nm²]</th>
<th>kAG [N]</th>
<th>ρA [kg/m]</th>
<th>ρI [kgm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monopile</td>
<td>7.22 × 10¹¹</td>
<td>5.02 × 10¹⁰</td>
<td>9.63 × 10³</td>
<td>2.70 × 10⁴</td>
</tr>
<tr>
<td>Transition</td>
<td>1.74 × 10¹²</td>
<td>1.12 × 10¹¹</td>
<td>1.92 × 10⁴</td>
<td>6.50 × 10⁴</td>
</tr>
<tr>
<td>piece</td>
<td>20 m</td>
<td>10 m</td>
<td>50 m</td>
<td></td>
</tr>
<tr>
<td>Tower</td>
<td>5.47 × 10¹¹</td>
<td>4.13 × 10¹¹</td>
<td>7.10 × 10⁴</td>
<td>2.05 × 10⁴</td>
</tr>
<tr>
<td>bottom</td>
<td>7.79 × 10¹¹</td>
<td>1.51 × 10¹¹</td>
<td>5.29 × 10³</td>
<td>2.91 × 10³</td>
</tr>
<tr>
<td>Tower top</td>
<td>10 m</td>
<td>10 m</td>
<td>50 m</td>
<td></td>
</tr>
</tbody>
</table>

The dynamics of the structure are evaluated using a non-iterative implicit computing scheme described in Sakai & Sawanda 1996. This solution scheme provides an unconditionally stable algorithm without having to iterate for each time-step, as is needed in the implicit Newmark method.

![Figure 3. Illustration of the simplified structure (left) and the mesh (right).](image1)

![Figure 4. Applied loading and resulting displacement at the nacelle using the 'p – y' curves (black) and the kinematic hardening model (grey).](image2)
Two calculations are conducted: one with the nonlinear elastic ‘p – y’ curves and one with the kinematic hardening model. For these calculations, no other sources of damping are included. Figure 4 gives the loading at the nacelle and the resulting horizontal displacement. This figure illustrates that the nonlinear ‘p – y’ curves do not generate damping in the system as opposed to the kinematic hardening model which includes damping. This is illustrated by the decay of the vibrational response.

The kinematic hardening method provides an advance on current codified methods in providing more realistic foundation modelling for the analysis of monopiles. This method generates amplitude dependent damping arising from the foundation response and therefore removes the need to re-evaluate the damping coefficients for different load simulations.

5 CONCLUSION

The proposed kinematic hardening method forms a suitable basis for time-domain calculations which are used for the fatigue life calculation of offshore wind turbines. The resulting soil damping is inherent to the method as opposed to the Rayleigh damping for direct time integration methods and the overall modal damping values for modal-based methods which are currently used.

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REFERENCES


