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Numerical prediction of the installation of vibratory monopile foundations for offshore wind energy projects

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ABSTRACT: Despite the extensive research and advancements in installation methods and practises in the past few decades, there exist knowledge gaps regarding the numerical prediction of monopile installation in offshore settings. Efforts have been undertaken to develop and validate a computational tool that can effectively predict the installation behaviour of offshore monopiles. This study presents an enhancement to the classical Material Point Method (MPM), a numerical technique that can effectively simulate extensive material movements. The proposed upgrade involves the adoption of the Convected Particle Domain Interpolation (CPDI) method, which is capable of accurately capturing particle shear. The method has been extended to include pore fluid in order to simulate the saturated behaviour of the soil. The UBCSAND model, which incorporates both soil fluidisation behaviour and hardening/softening behaviour during dynamic loading, was utilised to simulate an offshore monopile. The evaluation of the model's performance is carried out through a comparison between the computed results and the field data gathered from the VISSKA project, which carried out in the North Sea.

Keywords: offshore monopiles; MPM; CPDI; large deformation simulation; monopile installation

1 INTRODUCTION

The offshore pile foundation installation technique of vibratory driving of monopiles is not comprehensively understood, leading engineers to exercise caution in applying it. Vibratory driving presents a considerable potential for cost and time savings when compared to impact driving. This study seeks to shed light on the vibratory installation of offshore monopiles by utilising the two-phase Convected Particle Domain Interpolation (CPDI) model in conjunction with the UBCSAND constitutive law.

Per reference (Madsen, Andersen, & Ibsen, 2012), it can be inferred that around 75% of offshore wind parks were established on monopile foundations. The utilisation of monopiles for offshore wind farms appears to be the most rational option for sandy soils. Nevertheless, the installation behaviour remains inadequately understood due to the absence of visual access and the incapacity to accurately measure and record alterations in the soil’s condition during installation. Modifications in the soil condition encompassing a pile have an impact on its ultimate strength, which can be either favourable or unfavourable. The study conducted by reference (Achmus, Schmoor, Herwig, & Matlock, 2020) aimed to differentiate the lateral bearing characteristics of large monopiles that are subjected to vibration and impact loading in dense sand.

During the vibratory installation of piles, conventional soil dynamics issues such as shear resistance of soils, local densification of soil body, and fluidisation of soil, to name a few, become relevant. When sand is sheared, the grains rearrange and reorient themselves in an effort to occupy a stable position with maximum inter-particle contact. Whether this rearrangement results in densification or loosening is dependent on soil-specific parameters and the vibration process.

In highly dynamic processes occurring in saturated granular soils, liquefaction or fluidisation is an additional factor that must be considered. When fully saturated soils that have been subjected to cyclic or monotonic loading undergo liquefaction, a considerable loss of shear strength occurs. Contractive loose sand attains its critical state, resulting in a condition of continuous flow with significant shear distortion and strength decrease. This state is maintained by the soil as long as the acting shear stresses were less than its reduced shear strength (Castro & Poulos, 1977). When the soil reaches the critical state, it flows with a constant void ratio, effective mean stress, and deviatoric stress.

In this study, efforts have been made to use the CPDI code to estimate the installation characteristics of a vibratory monopile and to compare the Class-A predictions with subsequent field data.
2 CONVETTED PARTICLE DOMAIN INTERPOLATION METHOD

In the traditional formulation of MPM, the continuum is represented by Lagrangian points, also known as particles or material points. Particles traverse a fixed Eulerian mesh, also known as a background grid or mesh. Particles store state variables and physical properties of the continuum, such as mass, momentum, stress, and strain, in addition to other state variables. At the start of a computational step, all pertinent information required for the solution is transferred from the particles to the computational grid in the background using the appropriate shape functions. The incremental solution is determined in a Lagrangian fashion on the grid. At the conclusion of each computational step, the solution is mapped back to the particles from the background in order to update the information associated with the particles, and the grid is reset for the subsequent computational step. The MPM approach utilizes the strengths of Lagrangian and Eulerian approaches while avoiding their weaknesses. Interested readers are directed to references (Sulsky & Schreyer, 1996), (Ceccato, 2015), (Bandara, 2013) and (Kafaji, 2013), which provides a comprehensive description of MPM.

2.1 Multi-Phase formulation of CPDI

Implementing the hydro-mechanical forces that capture the interaction between pore water (i.e., the liquid phase) and the solid skeleton is necessary for numerically simulating saturated soils. Reference (Kafaji, 2013) elaborate on the formulation on which the multi-phase implementation in this work is carried out. The readership is directed to that reference for a comprehensive understanding of the two-phase formulation. In the interest of completeness, only the most relevant equations describing two-phase media have been presented.

Solid phase mass conservation is described by the equation:

\[
\frac{d}{dt} [(1 - n) \rho_s] + \frac{\partial}{\partial x_j} [ (1 - n) \rho_s \hat{v}_j] = 0, \tag{1}
\]

where, \( \hat{v}_j \) is the velocity vector of the solid phase. The mass conservation relationship for the water phase is as follows:

\[
\frac{d}{dt} [(n \rho_w)] + \frac{\partial}{\partial x_j} [n \rho_w \bar{w}_j] = 0, \tag{2}
\]

where, \( \bar{w}_j \) is the vector of true velocity of the water phase. The variables \( n, \rho_w \) and \( \rho_s \) are the porosity, density of water and solid grains, respectively. Two reasonable assumptions will be made moving forward: i) the grains will be considered incompressible, and ii) the spatial variation in porosity and density of the control mass will be disregarded. Assuming water is linearly compressible and rearranging the above equations yields the storage equation, or the constitutive relation for pore fluid as follows:

\[
\frac{d p}{dt} = \frac{K_w}{n} \left[(1 - n) \frac{\partial \psi}{\partial x_j} + n \frac{\partial \bar{w}_j}{\partial x_j}\right], \tag{3}
\]

where, \( p \) is the pore pressure and \( K_w \) is the bulk modulus of water.

3 UBCSAND CONSTITUTIVE LAW

To simulate the coupled behaviour of soil and liquefaction during dynamic processes such as seismic excitation, the University of British Columbia developed the UBCSAND model. In this work, the saturated soil continuum is represented by the UBCSAND model together with the two-phase CPDI. UBCSAND is an effective stress model that simulates the elasto-plastic mechanical behaviour of the sand skeleton, and this work explores the possibility of applying this constitutive law to modelling the soil that is subjected to dynamic excitation due to the monopile vibration/impact hammering, as well as the multi-body contact that occurs during the installation. The code was implemented in-house and combined with CPDI code.

3.1 Elastic response

The elastic response of the model is assumed to be isotropic, in which the shear modulus \( G^e \), and the bulk modulus \( B^e \) are given by relations:

\[
G^e = K^e_G \cdot Pa \cdot \left(\frac{\sigma^t}{Pa}\right)^{n_e}, \tag{4}
\]

\[
B^e = K^e_B \cdot Pa \cdot \left(\frac{\sigma^t}{Pa}\right)^{m_e}, \tag{5}
\]

Here, \( Pa \) is the atmospheric pressure, chosen as 100 kPa for all simulations in this work, \( \sigma^t \) is the mean effective stress, and \( n_e, m_e \) are elastic exponents, that vary between 0.4 and 0.6. \( K^e_G \) and \( K^e_B \) are the shear and bulk modulus numbers, respectively. Unloading in this model is assumed to be purely elastic.

3.2 Plastic response

Plastic strains are controlled by the yield loci, which are assumed to be radial lines, starting at the origin of the stress space. For first-time loading, the yield locus is defined by the current stress state of the soil. As the shear stress increases, the stress-ratio (\( \eta \)), given by \( \eta = \tau / \sigma^t \), increases as well, activating the primary yield surface based upon isotropic hardening. Consequently, the yield surface is dragged to the new location,
expanding the elastic zone of the model. This results in plastic strains, both shear and volumetric. Unloading deactivates the primary yield surface, following an elastic path. A Mohr-Coulomb type failure function is assumed to determine the ultimate strength and state of stress achievable in this model. It is given by the relation:

\[ f_f = \sigma'_1 - \sigma'_3 N_{\phi_f} + 2c \sqrt{N_{\phi_f}}, \]  

(6)

where, \( \sigma'_1 \) and \( \sigma'_3 \) are the effective major and minor principal stresses, respectively. The parameter \( c \) is the cohesion and \( N_{\phi_f} \) is given by the relation:

\[ N_{\phi_f} = \frac{1}{1 - \sin(\phi_f)}, \]  

(7)

where, \( \phi_f \) is the peak friction angle. The flow rule is given by the relation:

\[ \frac{d\varepsilon^p_v}{d\gamma^p_s} = -\tan(\psi), \]  

(8)

where, \( \varepsilon^p_v \) and \( \gamma^p_s \) are the plastic volumetric and shear strains, respectively and \( \psi \) is the dilatation angle. The phase transformation friction angle, or stress ratio (\( \phi_{ctv} \)) and dilatation angle are related by the relation:

\[ -\sin(\psi) = \sin(\phi_{ctv}) - \eta, \]  

(9)

where, \( \eta \) is the developed stress ratio. Additionally, \( \eta \) is bound by the rule \( \eta \leq \sin(\phi_f) \). The reader is directed to reference (Naesgaard, 2011) for a detailed formulation of the model.

4 MONOPILE INSTALLATION SIMULATION

The CPDI code has previously been utilised to simulate both the solid and open steel pipe pile installation processes. Reference (Hamad, 2016) extended the CPDI code to its axi-symmetric formulation for improved computational efficiency in pile installation simulations. The penalty function method, as described in reference (Hamad, Giridharan, & Moormann, 2017) has been used in this work. References (Moormann, Gowda, & Giridharan, 2018) and (Moormann, Gowda, & Giridharan, 2019) extended the CPDI method to simulate model piles in saturated sand. References (Giridharan, Stolle, & Moormann, 2019) and (Giridharan, Gowda, Stolle, & Moormann, 2020) implemented the UBCSAND model to account for the liquefaction effects that occur during the dynamic installation process and applied the code to simulate pile installation.

Here, results of the Class-A (pre-event) prediction of the pile installation depth is presented. In this study, efforts have been made to predict the installation behaviour of monopile installation using vibratory methods prior to offshore installation in order to provide an estimate of the time required to install the monopile. Later, the results of the Class-A predictions are compared to field data to determine whether the developed tool is indeed reliable for use in simulations of offshore pile installation.

In this paper, an effort was also made to employ a model capable of capturing the liquefaction effects, so that the liquefaction phenomena described in the literatures are accounted for by the model. In addition, the capability of the code to model the extremely dynamic and complex pile installation is evaluated.

4.1 Dimensions and mesh

![Background computational grid discretisation (Left), and Particle discretisation of pile and soil (Right)](image-url)
For the numerical model, an axially symmetric boundary condition and the penalty contact algorithm, together with the two-phase CPDI formulation and the UBCSAND model have been considered. The UBCSAND model, which was previously employed for both model and actual scale tests of pile installation presented in reference (Giridharan, Gowda, Stolle, & Moormann, 2020) that yielded good results for back-calculation or Class-C analysis of the experiments, is employed once again, but this time for Class-A predictions. Class-A predictions are especially challenging because the CPT data must be used as an input to to estimate the soil parameters in order to perform the numerical simulations. To develop a model that can simulate the dynamic process with reasonable accuracy and complete the task in a reasonable amount of time, reasonable assumptions must be made. In order to achieve this, the following assumptions are made about the model: i) it is assumed that the soil is completely saturated, ii) it is assumed that the pile has a constant cross section, and iii) it is assumed that the soil is homogenous and isotropic. The behaviour of the soil is smeared across the entire continuum to generate a single parameter set for the numerical simulation. Aside from CPT data, there are very little high-quality data available for a rigorous calibration of the parameters, which justifies the assumption of a homogeneous soil continuum. Due to a paucity of soil test data, the additional effort required to model the soil's many layers cannot be justified.

The axially-symmetric continuum depicted in Figure 1, which consists of the soil and pile continuum, was discretised with approximately 27,000 particles and 13,000 interface elements, which simulate pile-soil contact. The background computational grid consisted of a tartan grid, with the smallest rectangular grid measuring 0.1 m by 0.01 m. The pile was embedded five metres into the ground prior to start of simulation. Due to the fact that the amount of pile penetration due to gravity cannot be accurately estimated in advance, this initial depth was chosen in part to improve the numerical stability of the model.

4.2 Initial Conditions

The vibration process in the field began at a pile height of 34.8 metres, which translates to a self-weight penetration of 2.2 metres. The numerical simulation, however assumed a self-penetrating depth of 5 metres. This assumption was made in part to improve the numerical robustness of the CPDI code, as instability in the calculation process has been observed when applying dynamic load at lower effective stress regimes. However, this decision was deliberate and not the result of calculations. The effective stresses were assigned to the particles using the $K_0$ procedure, and a 5-second null step, in which no dynamic load was applied to achieve a steady-state condition, was performed on the model. As the soil characteristics and geological history of Cuxhaven are comparable to those of the proposed offshore windpark site, the UBCSAND parameters for the Cuxhaven sand, shown in Table 1 were assumed to simulate the soil continuum. In addition, the model had access to an abundance of high-quality laboratory data. The parameters used in the simulation are listed in the table below.

The following vibrator parameters listed in the Table 2 were used for the installation process. The vibrating load was applied to the pile's head. Throughout the calculation, the static load remained constant, whereas the dynamic load varied sinusoidally at a frequency specified in the table. The pile is assigned the properties of steel, viz.: Young’s Modulus : 200 GPa, Poisson’s ratio : 0.3. The contact elements are assigned a coefficient of friction of 0.4.

![Table 1. UBCSAND model parameters for Cuxhaven Sand, $I_d = 100\%$](image)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frequency of Vibrator</td>
<td>20 Hz</td>
</tr>
<tr>
<td>Total mass, with mounting elements</td>
<td>435 ton</td>
</tr>
<tr>
<td>Static load</td>
<td>4,266 kN</td>
</tr>
<tr>
<td>Dynamic load</td>
<td>30,319 kN</td>
</tr>
</tbody>
</table>

4.3 Results and Discussion

Figure 2 depicts the outcomes of the pile penetration, which include experimentally measured data and numerical results. In comparison to the experimental results, the numerical results demonstrate a good correlation with the penetration. There is a reasonable correlation between the field and the calculated numerical data, despite the assumption of a highly homogenised and idealised model setup. This demonstrates that the CPDI+UBCSAND tool performed admirably when applied to the numerical Class-A prediction of pile installation. Two successes of the numerical tool are highlighted here: i) a reasonable match with the field data, wherein the numerical model reaches the embedment depth reached by the offshore vibration installation; and ii) the achievement of a stable solution simulating the
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vibratory process of an offshore monopile. Notably, a progressively slower installation rate, which is also attributed to the frictional effects of the soil on the pile, indicates that the numerical model incorporated the soil-pile interaction during the installation process.

![Figure 2. Vertical penetration of the pile - Numerical vs. Field Measurement](image)

During the first 50 seconds of simulation, the installation rates derived from field data and numerical simulation are comparable and nearly identical. In comparison to field data, the results of the numerical simulation indicate a more rigid response after 150 seconds of installation.

![Figure 3. Measured CPT data at planned location of Pile installation](image)

Figure 3 confirms the results from the CPT data for the location, which explains the softer response from the field data. The relative density is assumed to be homogeneous and constant in the numerical model. A zone of increased cone resistance is measured in the field between 15 m and 20 m below sea bed, which will result in a relative density greater than the model's assumption of a density of 100 percent, which is overlooked by the homogenised numerical model.

Cone resistance tends to increase at depths exceeding 15 metres below sea bed. This pattern persists until a depth of 20 metres below sea bed. This layer of denser soil explains why the installation rate of piles has declined. Again, the homogenisation soil continuum assumed by the numerical model cannot accommodate this localised extremely dense soil zone. At a depth of 17 metres, the cone resistance increases further, resulting in a further decrease in the installation rate of piles. At this point, the numerical model overlaps and exceeds the field data. The pile in the field, however, encounters a zone of densely compacted soil. For approximately a hundred seconds, the pile vibrates with minimal vertical movement. In contrast, the installation rate in the numerical model begins to decrease. This decrease in the numerical model is due to the increased friction acting on the pile shaft as it approaches greater depths, as well as the increased stress-dependent stiffness developed by the UBCSAND model. After the vibration reached a depth of 17 metres below sea level in the field, refusal was reported. The depth where refusal occurs correlates well with the CPT test results where 125 MPa of cone resistance is encountered. At this point, the simulation was terminated after approximately 10 minutes of real time.

![Figure 4. Effective stress of soil, 10 m below Sea Bed](image)

The diagram presented in Figure 4 illustrates the effective stress measurements extracted at a depth of 10 metres below the ocean floor. As anticipated, the change in the effective radial stress attains its greatest magnitude in proximity to the pile's jacket, at a
distance of 0.51 times the diameter from the axis of symmetry, and is roughly four times greater than its corresponding geostatic value. The radial stress zone experienced a reduction in elevation as the control plane moved away from the pile within the range of 0.75 D to 1.5 D. However, it still maintained an elevated state. Furthermore, it is possible to extract the stress distribution in the vicinity of the pile and transfer it to another software programme that is capable of modelling the long-term cyclic behaviour under lateral or axial loading. The CPDI code utilised in this investigation is presently inadequate in precisely emulating lateral behaviour due to its two-dimensional implementation.

5 CONCLUSIONS
The present study showcases a Class-A simulation of monopile installation. In general, the forecast proved to be precise. Furthermore, the algorithm demonstrated competence in accurately replicating the rate of installation, while also exhibiting the ability to simulate highly dynamic vibratory pile vibration for a duration exceeding 10 minutes. The simulation outlined in this study was executed on an AMD Ryzen Threadripper Pro processor operating at a frequency of 4.2 GHz and equipped with 32 threads. The simulation of the problem required a duration of approximately 10 days. The prevailing approach in modelling involves the assumption that the soil is a uniform and undifferentiated continuum. Although the aforementioned simplification yielded satisfactory outcomes within a feasible timeframe, current research endeavours are centred on the development of multiple soil layers to replicate real ground conditions.

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7 REFERENCES