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Numerical simulation of impact driven offshore monopiles using the material point method

Simulation numérique de monopiles offshore en utilisant la méthode des points matériels

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ABSTRACT: Impact driven monopiles are widely used in the construction of offshore wind turbines. The pile installation leads to soil disturbance in the vicinity of piles, which might affect their bearing capacity. A good understanding of the effect of pile installation on the bearing capacity of piles will lead to significant cost reduction in the construction of wind parks. However, due to the limitations in the common numerical methods (e.g. the finite element method) the effect of pile installation is not considered because it involves complex large deformation analyses, which are problematic. In this study, the Material Point Method (MPM) is used to simulate the installation of a real monopile, driven in the North Sea, to demonstrate the capability of MPM to simulate the installation of monopiles using results of the field test.

RÉSUMÉ: Les monopieux battu sont largement utilisés dans la construction des structures offshore. L'installation de ces pieux entraîne une perturbation importante du sol auprès des pieux, ce qui pourrait affecter leur capacité portante. Une bonne compréhension de l'effet de l'installation de pieux peut mener à une réduction significative des coûts de construction des parcs éoliens. Néanmoins, en raison des limitations des méthodes numériques courantes (par exemple, la méthode des éléments finis), l'effet d'installation des pieux n'est pas pris en compte parce qu'il implique des analyses des larges déformations lesquelles posent problème pour la méthode des éléments finis. Dans cette étude, la méthode de MPM (Material Point Method) est utilisée pour simuler l'installation d'un monopieux réel construite en mer du Nord. Les résultats de l'essai sur terrain sont utilisé afin de démontrer la capacité de MPM à simuler l'installation des monopieux.

Keywords: MPM; offshore wind; monopile; impact driven; pile installation

1 INTRODUCTION

The installation of offshore monopiles leads to significant soil disturbance in the vicinity of piles which might affect their axial and lateral bearing capacity. A better understanding of the process of pile installation will lead to cost and time reduction in the construction of wind farms. This process can be studied numerically in order to, for example, predict the probability of pile refusal. In addition, it can be used as tool to optimise the design of the piles and to estimate the required impact force and mass of hammer. However, the numerical simulation of the installation of offshore monopiles is challenging as it involves consideration of the highly nonlinear behaviour of soils in static and dynamic regimes, large deformation analyses around the pile, the contact between the pile and soil and wave propagation in both water and solid phases. This requires advanced modelling techniques to properly address the above mentioned mechanisms. Due to the limitations in the common numerical methods (e.g. the Finite Element Method, FEM) the effect of pile installation cannot be considered as it involves large deformation analyses which leads to severe mesh distortion that are problematic.

Recently, significant progress has been made in numerical analyses of geotechnical problems involving large deformations of soil by means of the Material Point Method (MPM). This numerical method has been developed specifically for the simulation of large deformation processes. It is closely related to the Updated Lagrangian Finite Element Method (UL-FEM). Similar to FEM, equilibrium equations are solved on a background finite element mesh. Large deformations of soil are modelled with MPM by a cloud of material points that moves through a background mesh, whereas with the FEM this can be done by updating the mesh according to soil deformations. The MPM avoids numerical problems related to severe mesh distortions which limit applicability of FEM with large deformation problems.

In this study, a 2D axisymmetric MPM (Galavi *et al.*, 2018) is used for modelling soil-structure and water-structure interaction. An improved contact formulation, originally proposed by Bardenhagen *et al.* (2000), defines the interaction between the pile and soil in which solid and water phases are treated separately. A coupled two-phase analysis (Van Esch, 2010) is utilised to model wave propagation in both water and solid phases.

The case presented in this paper is from a real impact driven monopile, installed in the North Sea. The pile has a total length of 77.4*m*, of which 33.8*m* is installed into the sea bed. The diameter of the pile is 7.8*m* with an average wall thickness of 7*cm*.

In this paper, installation of 7.3*m* of the pile length is numerically simulated. As the first 4.7*m* of penetration happened due to self-weight, the numerical simulation of the impact pile installation is presented for the depth of 4.7*m* up to 12*m*.

2 OVERVIEW OF THE FIELD TEST

2.1 Soil layers

The offshore test field is located in the North Sea. The basic mechanical properties of the soil layers in the area of the field test are listed in Table 1. It mostly consists of sand layers. The top two layers are composed of a loose sand layer with thickness of 1.7*m*, and a medium dense sand layer with thickness of 3*m*. The rest of the layers are characterised as dense and very dense sand layers with relative densities higher than 98%. The water table is around 40*m* above the ground surface.

Table 1. Basic parameters of soil layers

Depth [m]	Type	D_R [%]	γ' [kN/m3]	φ'_p [°]
0 - 1.3	Loose	12	8	22
1.3 – 4.7	Medium	47	9	34
4.7 – 7.2	Very dense	99	10.5	41
7.2 – 9.2	Dense	81	10	38
9.2 - 13.7	Very dense	98	10.5	41
13.7 – 16.2	Very dense	108	10.5	42
16.2 – 18.7	Very dense	96	10.5	41
18.7 – 27.2	Very dense	106	10.5	42
27.2 - 30	Very dense	127	11	42

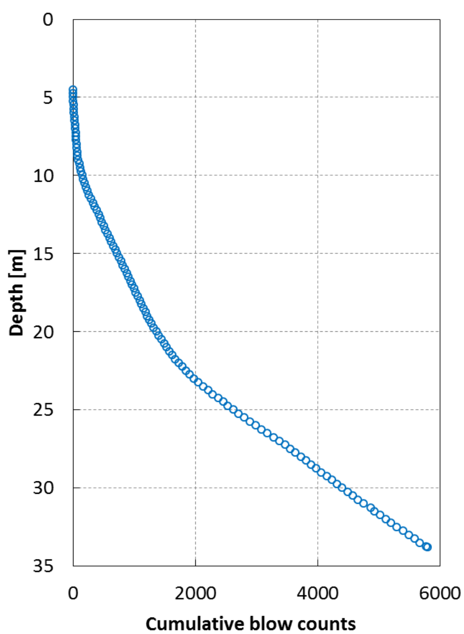

Figure 1. Cumulative blow counts versus depth

2.2 Pile and hammer properties

The total length of the monopile is 77.4*m* with a diameter of 7.8*m* and an average wall thickness of 7*cm*. The total mass of the pile is 1160*ton*.

The pile was installed using an impact hammer with a mass of 220*ton* and a falling height of 2*m*. The efficiency of the hammer is measured as 95%.

Figure 1 shows the recorded number of blows versus the penetration depth. The pile was penetrated due its self-weight until a depth of 4.7*m*. Pile driving was performed from that depth until the depth of 33.8*m*. The soil layering is given in Table 1.

3 NUMERICAL MODEL

Numerical simulations of the pile installation have been performed using a 2D axisymmetric MPM (Galavi *et al.*, 2018). An explicit dynamic two-phase formulation (Van Esch, 2010) was used to account for generation and dissipation of pore water pressure during the pile installation. This formulation enables us to consider the dynamic waves in both water phase and solid phase separately. The explicit scheme requires that the computational time steps are small enough to capture the wave propagation in the system. The time steps are function of element size, permeability and stiffness of material. A stiff or less permeable material requires lower time steps.

As the stiffness of steel pile is very high compared to the stiffness of soil, the pile is assumed to be rigid. The wave propagation inside the pile is much faster than the wave propagation inside the soil. Therefore, an impulse is assumed to simultaneously reach any point along the pile wall. This assumption increases the size of critical time step significantly and consequently decreases the computational time. Therefore, the wave propagation in the pile is neglected.

Figure 2 shows the geometry of the problem including the soil layers. The first 1.3*m* layer, which consists of a very soft material, is not considered in the numerical model, as the effect of this layer on the results is negligible. The finite element mesh consists of almost 2100 linear triangle elements.

A frictional contact formulation is defined between the pile and soil to prevent interpenetration between the two bodies. The original formulation is described in Bardenhagen *et al.* (2000). The formulation is further improved to account for sharp edges and gap/closure between pile and soil. The friction coefficient between the pile and the sand is assumed to be equal to 2/3 of the critical frication angle, *i.e.* 20 degrees.

In order to increase the accuracy, the mesh is refined in the vicinity of the pile. In addition, the moving mesh approach (Al-Kafaji, 2013) is utilised to ensure that contact nodes on the pile and soil are always on the same coordinates to reduce oscillations in contact forces.

The material point discretisation is depicted in Figure 3. The total number of material points in the mesh is around 10000. More material points are placed initially in the region where the pile is going to pass to increase accuracy and decrease grid crossing noises. A uniform distribution of material points is considered in the vicinity of the pile to ensure that all material points in an element have the similar integration weights, which results in a more accurate material point integration. In addition, the uniform distribution of material points stabilises the simulations by preventing the creation of non-physical empty elements. An empty element is created when an unstructured mesh is moved over the material points. In this case a small element might be located between some material points and becomes empty.

3.1 Hammer-Pile system

A hammered pile is installed by dropping a mass from a free fall height. The resulting impact forces are applied on the pile pushing the pile to penetrate into the ground. The impact forces are function of the mass of the hammer, *m*, the drop height, *h*, and the impulse time, t_i. The impulse time is the time that the hammer and the pile are in contact and is a function of the stiffness of the coupled system, *i.e.* the stiffness of the hammer, the pile and the soil.

For simplicity, an uncoupled hammer-pile system is used to transfer the impact forces from the hammer to the pile as illustrated in Figure 4. Using the method, described in Al-Kafaji (2013), the maximum impact force can be obtained from:

$$f_{max} = \frac{\pi \eta m \sqrt{2gh}}{2t_i} \tag{1}$$

where η is the efficiency of the hammer and g is the gravitational acceleration.

The used hammer in the field has a total mass (m) of 220*ton*, a drop height (h) of 2*m* and an efficiency of 95%. The average impulse time was measured as 20*ms*. Therefore, the maximum impact force is calculated as 102825*kN* in 3D geometry or 16365 *kN*/*rad* in 2D axisymmetric geometry. The average inter-blow time, *t*, is 1.25*s*.

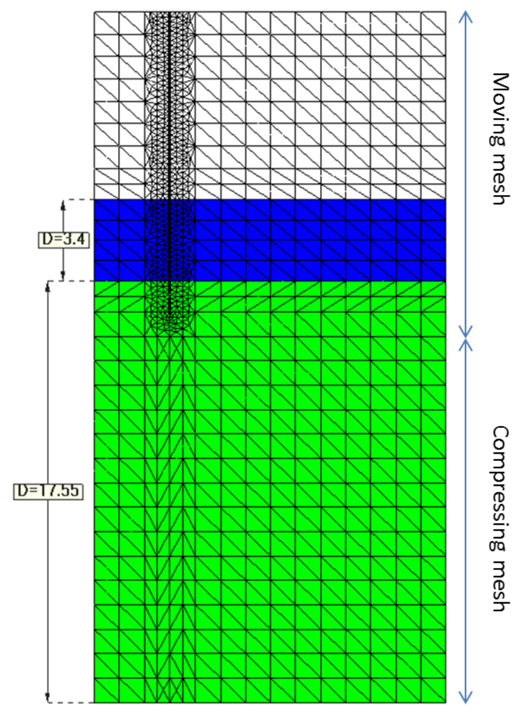

Figure 2. Geometry, soil layers and FE mesh

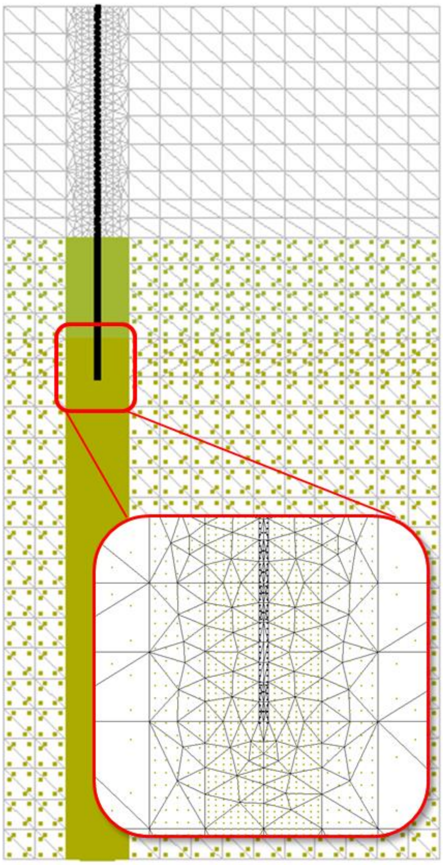

Figure 3. Material point discretisation

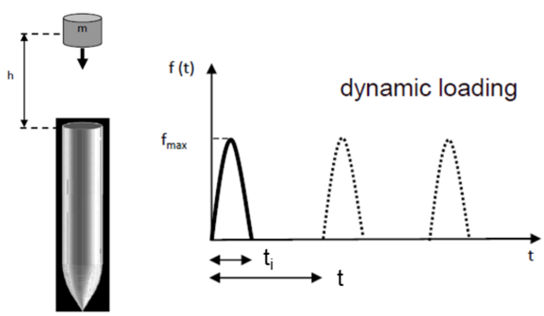

Figure 4. Hammer-Pile system and the corresponding impact force

3.2 Material properties

A double hardening constitutive model was used to describe the mechanical behaviour of the sand layers. The full description of the model can be found in Benz (2007). The model requires nine material parameters which can be determined from conventional laboratory tests. For in-situ cases, when only the relative density of sand is known, Brinkgreve *et al.* (2010) proposed some correlations between the material parameters and the relative density. Table 2 gives the material parameters estimated using these correlations. The double hardening model does not consider state dependency in its formulation. Therefore, there is no threshold for the maximum dilatancy during shearing. In order to prevent unrealistic volumetric expansion at very large deformation regime, which happens along the shaft, the maximum dilation angle is reduced from 4 and 8 degrees to 0.5 and 1 degrees in the sand layers with relative densities of 47% and 99%, respectively.

Table 2. Material parameters of the double hardening model

Parameters	**Unit**	**D_R = 47%**	**D_R = 99%**
E_{50}^{ref}	[kPa]	28.2×10^3	48×10^3
E_{oed}^{ref}	[kPa]	28.2×10^3	48×10^3
E_{ur}^{ref}	[kPa]	84.6×10^3	144×10^3
ν_{ur}	[-]	0.2	0.2
G_0^{ref}	[kPa]	91.6×10^3	114.4×10^3
$\gamma_{0.7}$	[-]	1.53×10^{-4}	1.2×10^{-4}
m	[-]	0.55	0.45
φ'	[°]	34	38
c'	[kPa]	1.0	1.0
ψ	[°]	0.5	1.0
R_f	[-]	0.9	0.9

3.3 Calculation and results

The pile sank into the top soft layers until 4.7 *m* depth by its self-weight and after that it is driven into the ground by impact-driven technique. For the sake of brevity, the first part of the pile installation, *i.e.* penetration due to self-weight, is not studied here. The installation process is simulated from 4.7*m* up to 12*m* depth. The pile is initially embedded at the depth of 4.7*m*.

The stresses are initialised using a K_0-procedure, in which the pile is fixed and the weight of the pile is not considered. Figure 5 shows the effective stresses initialised by the K_0-procedure. To bring the system of pile-soil to equilibrium, a drained nil-step analysis is performed after the K_0-procedure. Here, the pile is released and the interaction between the pile and the soil is modelled using the contact formulation. The resulting vertical effective stresses are depicted in Figure 6. As can be seen, the vertical stresses are considerably increased close to the pile toe.

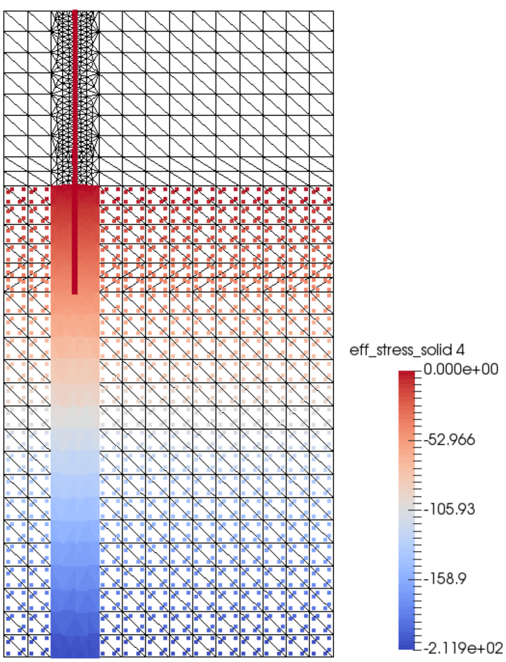

Figure 5. Initialised vertical effective stresses by K_0-procedure

The pile driving process is simulated using a load-controlled approach where the impact force is applied to the rigid pile. The impact time and the blow time are kept constant throughout the simulations, as 0.02*s* and 1.25*s*, respectively. In total 285 blows are simulated. The computational time is around 45*min* for each blow.

Figures 7 and 8 show the vertical and horizontal effective stresses after 100 blows, respectively. A slight increase in horizontal effective stress can be observed inside the pile in comparison with the exterior of the pile. This results in an increase in the mean effective stresses in the interior of the pile, as plotted in Figure 9.

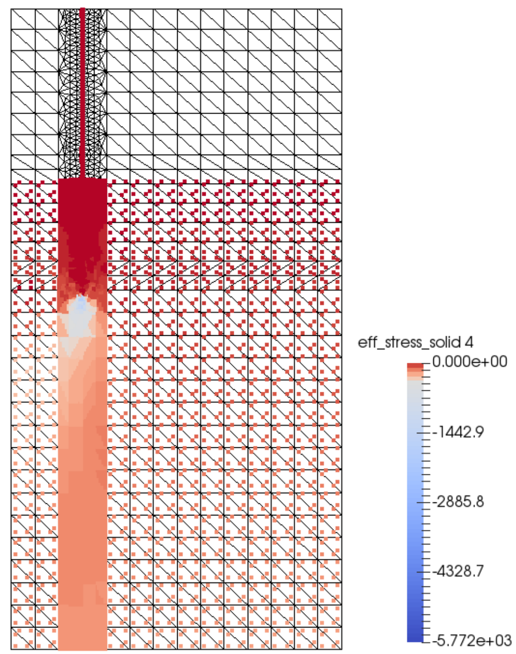

Figure 6. Vertical effective stress after nil-step

The deviatoric strains are plotted in Figure 10, which are mainly due to the shearing around the toe of the pile. It should be noted that most of the shearing at the shaft is happening at the interface between the pile and the soil, as the friction between the pile and soil is lower than the internal friction angle of the soil.

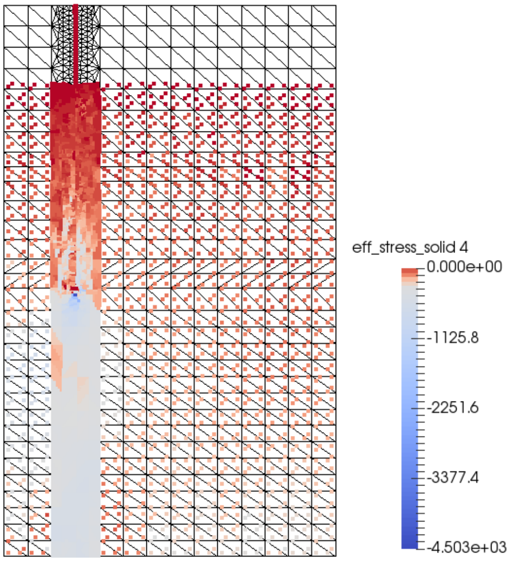

Figure 7. Vertical effective stress after 100 blows

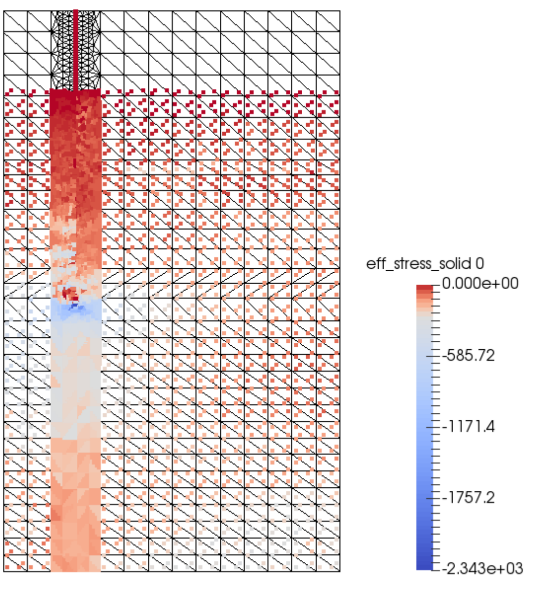

Figure 8. Horizontal effective stress after 100 blows

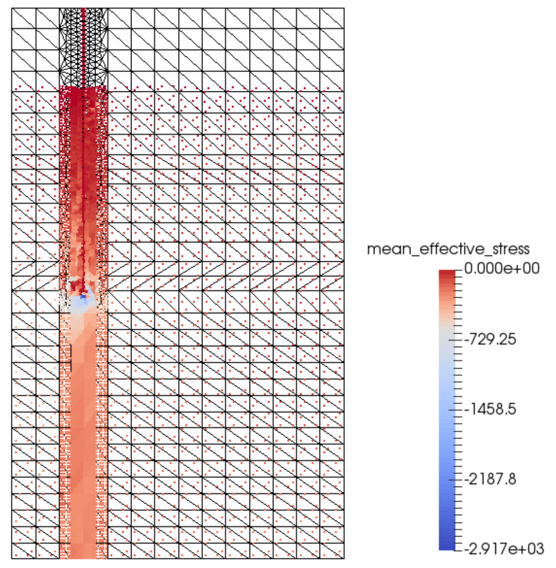

Figure 9. Mean effective stress after 100 blows

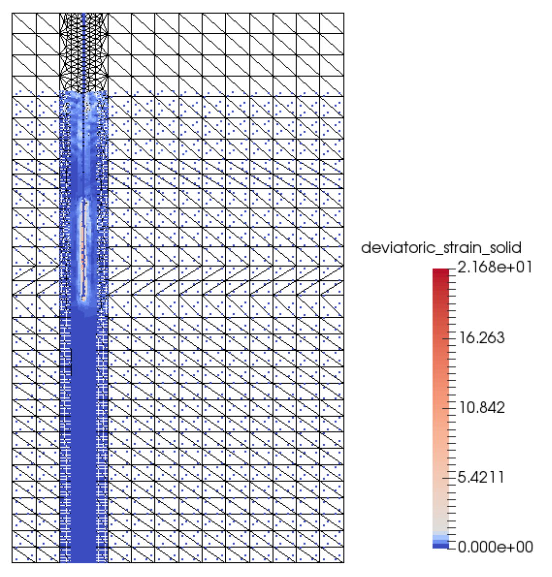

Figure 10. Deviatoric strain after 100 blows

Figure 11 shows the cumulative blow counts versus depth for the upper and lower bounds of the permeability in the sand layers in the field. It can be seen that the simulated number of blows versus depth is very close to the measurements for both values of permeability. However, lower permeability results in a slightly stiffer behaviour close to the toe of the pile, because water flows slower after each blow which results in higher bulk stiffness in the soil in the vicinity of the pile toe.

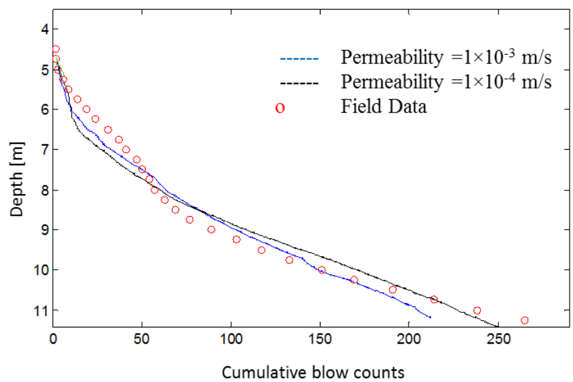

Figure 11. Simulated number of blows versus depth for two different permeability

4 CONCLUSIONS

The material point method has been used to numerically study a real case impact driven monopile installation in the North Sea. In order to speed up the simulations, a 2D axisymmetric MPM has been used. A dynamic two phase formulation was used to properly simulate wave propagation in both solid and water phases. The double hardening material model is used to predict the mechanical behaviour of the sand layers. The numerical results revealed that the developed MPM code can be used as a promising tool to predict the pile installation process for impact driven monopiles.

5 ACKNOWLEDGEMENTS

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