NUMERICAL EVALUATION OF THE SOIL BEHAVIOR DURING IMPACT DRIVING OF PIPE-PILES

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ABSTRACT

During the impact driving of pipe-piles, the soil is influenced in different ways including the void ratio, stress distribution, and plugging formation. Such effects may play an important role in structural design criteria such as the pile's lateral support provided by the soil. Hence, this work is focused on investigating the change in the mechanical characteristics of the soil during impact driving using an advanced numerical analysis tool which is validated against an experiment. The investigation includes the pile penetration behavior, plugging formulation inside the pile, and the change of the lateral stress in the soil during the pile installation. The proposed numerical model is shown to provide similar results compared to experimental measurements. The void ratio of the soil is influenced due to pile driving up to a lateral and vertical distance of 2D and 1D, respectively, where D is the pile diameter. Compared to the initial void ratio, the soil inside the pile experienced loosening about 20% while the soil outside is densified about 30% during driving. Moreover, the induced lateral stress inside is more than the one outside the pile, indicating the formation of plugging. Compared to the initial lateral stress state, the pile installation increased the lateral stress up to four times inside and two times outside the pile. Based on the findings of this work, the effects of driving on soil mechanical properties are not minimal and may affect the pile performance including the lateral resistance of the pile. By using the numerical approaches such as one in this study, the evaluation of the various effects on the soil due to pile driving and gaining a better understanding of the such complex problems are possible.

Keywords: Pipe-piles, Impact driving, Soil-structure interaction, Multi-Material Arbitrary Lagrangian-Eulerian, Hypoplasticity for granular soil

1. INTRODUCTION

Pipe-piles are conventionally used as deep foundation systems, particularly for offshore applications. The choice of the pile installation method depends on various parameters, such as pile dimensions, soil strength, and their interaction. One of the conventional driving methods is the impact installation where the pile is pushed into the soil using a falling mass.

The numerical methods have shown great potential as an evaluation tool in geotechnical problems in recent decades. Yet, the installation processes are difficult to simulate using common numerical formulations due to significant and complex deformation [1]. In addition, the granular soils exhibit a non-linear behavior even during the initial loading which cannot be captured using simple elastoplastic-based constitutive models [2].

The motivation of this study is therefore to investigate the impact driving technique using an advanced numerical model in terms of several major parameters including the required force of the pile installation, pile penetration behavior, plugging formulation inside the pile, and the change of the soil state outside the pile during the installation. The outcome of this study shall provide a better outlook on the effect of the installation method on the soil characteristics.

The effect of installation methods on the soil and the neighboring structures have been studied in several works. In a work done by Hartung [3], the effect of the installation method on the bearing capacity for open-ended piles was investigated. It was concluded that in case of impact driving, the soil exhibits different behavior based on its initial density. Depending on its loose or dense state, the soil around the pile shaft contracts or dilates, respectively. Nevertheless, an increase in stress is always observed in areas near the pile tip [3].

Concerning the numerical simulation of pile driving, special care should be given since such problems are considered in the category of large deformation problems, where significant distortions and complex movement in soil encountered. This causes several issues when classical numerical methods based on the Lagrangian formulation are used such as solution divergence, inadequate soil-structure interaction, and accuracy loss. Therefore, a robust alternative numerical technique should be devised.

Recently, a set of robust numerical alternatives for capturing large soil deformation during pile installation are introduced which is originally borrowed from the field of fluid dynamics such as MMALE (Multi-Material Eulerian Arbitrary Lagrangian Eulerian) [4]. This method has been verified and validated for various geotechnical problems involving large deformation [5], [6] and therefore shows itself as a potential and cost-effective evaluation tool for such problems. A recent work done by Daryaei et al. [7], investigated the effect of frequency in achieved penetration depth in the vibratory installation of pipe piles.

The structure of this paper is as follows. In the following parts of section 2, the state of the art in the numerical modeling of pile installation and complex soil behavior is presented. The numerical model development including the validation is also discussed in section 2. Afterward, the simulation results are presented and discussed in section 3. Finally, the conclusion of this study and outlook on future works are presented.

2. METHODOLOGY

2.1. The MMALE numerical approach

The MMALE method can be considered as an advanced mesh-based numerical formulation which benefits from the advantages of both classical Lagrangian and Eulerian element formulation in the Finite Element Method (FEM) [1]. In the Lagrangian formulation, the computational grid moves and deforms in accordance with the material particles, as if grid points are fixed to the particles. This constraint causes considerable shortcomings when the soil significantly deforms, including large distortion, solution divergence, or unreliable results. On the contrary, the grid is fixed in the Eulerian formulation allowing the material to move independently through the grid. Therefore, after each calculation step, the solution must be transported to the initial grid. Despite solving the large deformations and vorticity issues, extra considerations for treating path-dependent material behavior and tracking material interfaces should be given in the Eulerian formulation [1].

In MMALE, the grid deforms as in the classical Lagrangian formulation. Then, a new less distorted grid is generated. Subsequently, the solution variables are transported to the new mesh as in the Eulerian formulation [8]. In MMALE, multiple materials inside one element can be considered which significantly improves the formulation in case of extremely large deformations. Usually, a material-free or void zone is defined within the grid without mass nor strength, so that other materials can flow into these regions of the physical space. For a more detailed theoretical background of the method, the reader is referred to [8], [9].

2.2. The Hypoplastic constitutive equation

The mechanical behavior of soils, especially the granular soils are very complex and require special treatments. The distinctive feature of granular materials such as sand is its non-linear behavior which can be observed from the simultaneous elastic and plastic deformation even at the initial loading states. [2].

Hence, an alternative concept called the hypoplasticity is introduced which captures the inelastic deformation from the beginning of the loading and does not distinguish between elastic and plastic deformation. The hypoplastic constitutive model is popular for its simplicity where loading and unloading steps are considered in a single incrementally nonlinear equation. The stress rate of the granular material, $\dot{T}$, is determined by the effective stress, T, intergranular strain, δ, and the void ratio, e [10]:

$$\dot{\mathbf{T}} = \mathcal{M}(\mathbf{T}, \mathrm{e}, \delta) : \mathbf{D} \tag{1}$$

Besides the stress rate, the void ratio in the Eq. (1) is governed by the minimum and maximum void ratio, e_i and e_d, respectively as well as the critical void ratio, e_c.

Since the introduction of the hypoplasticity, numerous material models were developed to adapt the model to various loading conditions. In this study, the hypoplastic equation presented by Niemunis and Herle [10] is used which is the improved version of the hypoplastic equation developed by von Wolffersdorff [11]. The improvement addresses the previous issues of the model in reflecting the accurate strain accumulation during the cyclic loading [10]. Previous application of this version of the hypoplastic material model in large deformation geotechnical problems showed good agreement with experimental results [12].

2.3. Soil-structure interaction

Here, a brief description of the employed contact scheme in MMALE for soil-structure interaction, the penalty contact scheme, is presented. The contact force is measured based on the arbitrary penetration of the parts. This is considered by adding an additional term (also called a penalty term) to the energy equation as follows [14]:

$$\Pi = \mathrm{E_p} + E_k + \frac{1}{2} k \Delta u^2 \tag{2}$$

Where E_p and E_k are potential and kinetic energy, respectively, k is the spring stiffness resembling the contact interface, and u is the arbitrary penetration of two contact parts.

2.4. General remarks about the numerical model

In this section, a description of modeling considerations using MMALE technique is presented where a pile is installed in the soil using impact load. An axisymmetric model is developed to reduce the computational costs. The model configuration is shown in Figure 1.

The rigid pile has 1.5 m height, 0.2 m diameter, and 0.004 m thickness which is modeled using the conventional 2D Lagrangian shell element formulation with reduced integration point and a uniform element size of 0.004 m.

For the soil and void, a mesh with 1.6 m height and 0.85 m radius with the one-point integration MMALE shell element formulation is generated. A structured mesh, ranging from 0.004 – 0.04 m element width is used. The mesh contains the soil up to a height of 1.4 m. A void domain with 0.2 m height, which has neither mass nor strength, is defined above the soil material to enable the soil to move to this domain after penetration starts. The hypoplastic material model is adopted, whose corresponding material constants for Berlin sand are listed in Table 1. The relative density of the soil is Dr=75% ($e_{initial}$=0.465). The initial stress in the soil is defined with assigning the gravity acceleration as 10 m/s2 and using the lateral earth pressure, K_0=0.5.

Table 1. *Hypoplastic material constants for Berlin sand*

φ_c (°)	h_s (Mpa)	n	e_{d0}	e_{c0}	e_{i0}	α	β	m_R	m_T	R	χ	β_r
31.5	230*	0.3	0.391	0.688	0.791	0.13	1	4.4	2.2	1×10^{-4}	6.0	0.2

*The actual value of granular hardness, h_s, is 2300 MPa. This value is reduced by 10% due to low-stress soil state

To define the coupling between pile and soil, penalty contact is defined. A tangential friction coefficient of 0.38 is assigned ($2/3 \tan\phi$). The pile is fixed against horizontal movements while the lateral sides of the soil are constrained against movements in directions perpendicular to their faces, with fixity applied in all directions at the bottom of the soil.

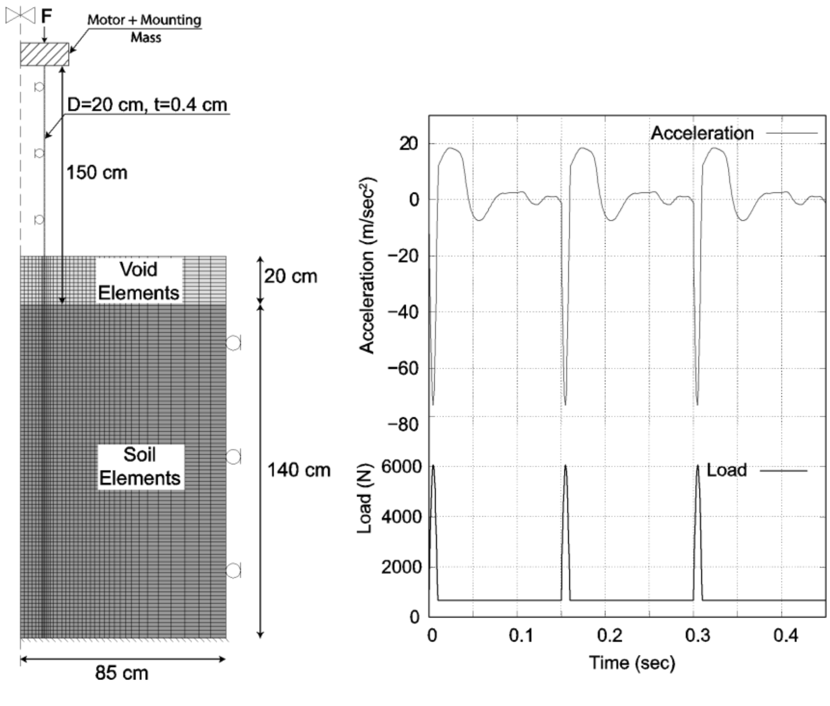

Figure 1. *Numerical model configuration of the pile driving experiment (left) Load application curve and the induced pile acceleration during the impact driving (right)*

The numerical simulation consists of two phases. The pile is embedded at the depth of 0.1 m due to numerical stability considerations. Initially, the pile is let to penetrate by its own weight as was done in the experiment. Subsequently, the driving force is applied.

In the case of the impact driving, the weight and the height of the falling mass is often available, which should be converted to the applied load on the pile head. According to Al-Kafaji [15], one can alternatively use the following equation:

$$f_{max} = \frac{\pi\eta m\sqrt{2gh}}{2t} = \frac{\pi \times 0.765 \times 22.1 \times \sqrt{2 \times 9.81 \times 0.28}}{2 \times 0.01} = 6.224\ kN \quad (3)$$

Where η is the reduction factor due to energy dissipation during impact, m and h are the mass and falling height of the drop weight, t is the impact duration which represents the duration in which the falling mass and the pile are in contact, and g is the gravity acceleration.

The imposed dead load on the pile due to self-weight, motor and mounting is about $F_s = 0.674$ kN. The total duration of the experiment was 354 seconds which corresponds to an impact rate of 0.5 Hz (177 blows). The underlying reason for the long duration was the long intervals between each blow because the mass had to be lifted up and prepared for the next blow. In the numerical model, however, this duration is computationally expensive. The intervals between each blow should be decreased to reach a suitable computation cost. However, the blow intervals should not be placed too close either. In principle, the optimum interval should be determined by evaluating the duration, in which the pile acceleration varies significantly. In other words, the next blow should be applied at the time, at which the pile is at a steady state. Figure 1 shows the acceleration history of the pile due to one impact. In this case, the interval of 0.15 is adequately large enough to avoid overlapping in the acceleration of the pile. The loading history curves of both pile driving methods are shown in Figure 1.

3. RESULTS AND DISCUSSION

The numerical model is validated against an experiment conducted at the laboratory of the Chair of Soil Mechanics and Geotechnical Engineering at Technische Universität Berlin (TU Berlin). Details regarding the test set-up are available in the work done by Le et al. [16]. The experiment includes a half-cylindrical pile with 1.5 m length, 0.004 m thickness, and 0.2 m outer diameters placed in a container filled with the Berlin sand and consists of three rigid steel walls and a glass panel. The pile movement is constrained in the horizontal direction using pile guides.

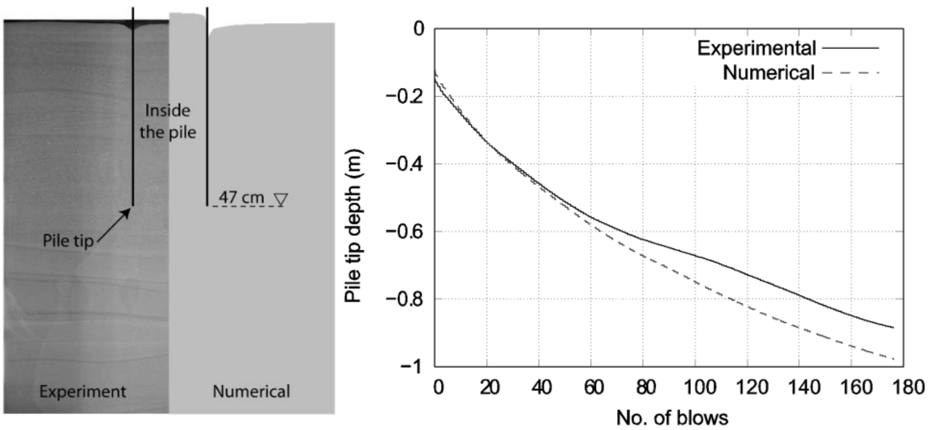

Figure 2. *Comparison of the soil deformation (left), and pile tip depth from the numerical models and the experiments (right)*

The deformed shape of the soil inside and outside the pile is compared with the one captured from the experiment in Figure 2. The height of the entrapped soil inside the pile is at this stage overestimated. This can be attributed to the shortcoming of the hypoplastic model on predicting the soil behavior in low-stress regions. On the other hand, the outer part of the soil surface calculated in the numerical model is in good agreement with what is observed in the experiment.

The curves of the pile tip depth from the numerical model against the experiment is also compared (Figure 2 right). At the early stages of driving, the numerical model is close to the experiment. After 60 blows, the numerical model starts to overestimate the penetration. After 120 blows, the penetration rate of both curves become equal.

Figure 3 shows the void ratio distribution at the pile tip depth of 0.47 m. The entrapped soil inside the pile exhibits loosening, unlike the outer part. The lateral and vertical influence distance of the soil is 2.5D and 1D, respectively, where D corresponds to pile diameter. Moreover, a thin area around the pile shaft with a width of 0.028 m has also experienced loosening due to the impact force. This loosening may facilitate driving performance by reducing the frictional resistance.

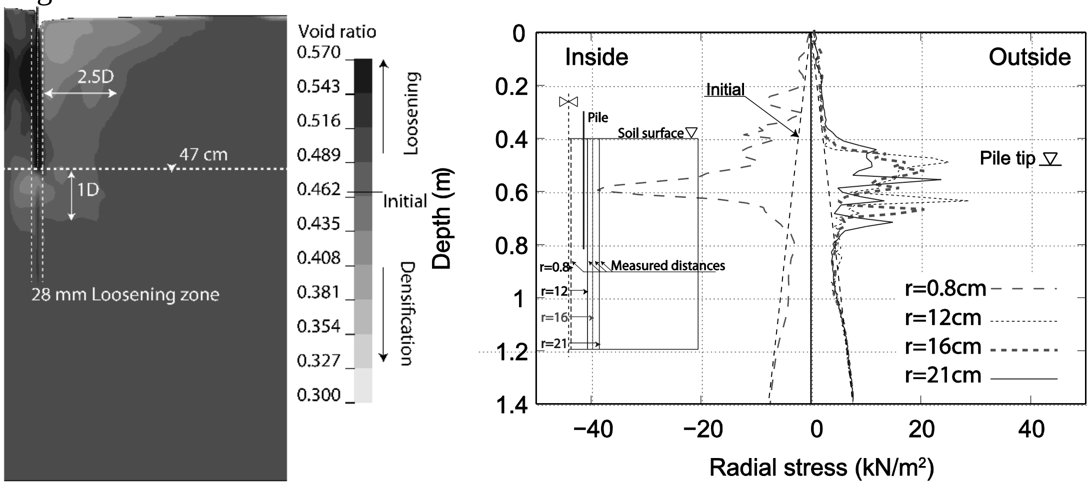

***Figure 3**. Void ratio distribution (left) and Horizontal stress distribution over the soil depth inside and outside the pile at the pile tip depth of 0.46 m (right)*

The lateral stress distribution over the depth inside and outside the pile at one time stamp is also shown in Figure 3 where the r is the lateral distance of the measuring stations from the axis of symmetry. One station is chosen inside the pile close to the axis of symmetry, r = 0.008 m, and three stations outside the pile at distances of r = 0.12, 0.16, and 0.21 m. The outer wall of the pile is located at a lateral distance of r = 0.1 m. Significant variations are noticed at the depth of 0.4-0.8 m. These variations are along with notable oscillations which can be attributed to the dynamic nature of the impact pile driving. The maximum lateral stress is observed at almost 0.1 m under the pile tip which is two times and four times more than the initial stress outside and inside the pile, respectively. Afterward, the stress decrease until the depth of 0.8 m where the initial stress state of the soil is reached again. In addition, the induced stress due to driving is decreased by 50% from r = 0.12 to 0.21 m.

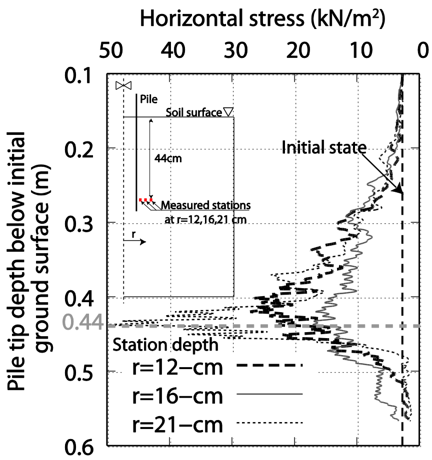

***Figure 4.** Horizontal stress history at three stations outside the pile, at the depth of 47 cm during impact (positive value indicates compression).*

The induced stress inside the pile is about two times greater than the outside. Therefore, it may be argued that plugging occurs. This cannot be initially concluded from the soil surface in

Figure 2 as the surface of the entrapped soil is higher than the surface outside the pile. This can be justified with the fact that at initial stages of the penetration soil heaving was observed. Afterward, this heaving decreases with further driving. In addition, by evaluating Figure 3 it is clearly seen that the plug is formed inside the pile at the final stage of the model.

Figure 4 depicts the change in lateral stress during the driving at three stations located at the depth of 0.44 m and at the same lateral distance of those in Figure 3. Initially, a limited increase in lateral stress is observed up to a depth of 0.25 m. Then, the stress starts to increase rapidly until the pile reaches the depth of the 0.44 m. Afterward, the lateral stress decreases drastically as the pile penetrates further until it reaches the initial stress state at the depth of 0.6 m. The rate of decrease is more than the rate of increase.

4. CONCLUSION

In this study, the effects of impact driving during pipe-pile installation in granular soil is numerically investigated, using the advanced numerical tool MMALE in conjunction with an advanced hypoplastic constitutive equation. The investigation criteria include the induced horizontal stress inside and outside the pile, change in void ratio, and pile penetration.

The penetration curve was initially in good agreement with the experiment. After reaching a time corresponding to one-third of the blows, the numerical model started to overestimate the penetration. After applying two third of the blows, the penetration rate of the numerical model became identical to the one in the experiment and hence the difference was maintained. The final difference in the pile tip depth compared to the experiment was about 10% more.

By evaluating the void ratio, loosening inside the pile and densification outside the pile was observed. The influence area corresponded to 2.5D in lateral and 1D in the vertical direction. Also, a loosening area was formed around the pile shaft caused by the disturbance from the impact driving.

Using the lateral stress, the formation of a soil plug was confirmed. Moreover, a limited depth of the soil was affected during the driving, up to the depth of 2D under the pile tip. At a lateral distance of 2D outside the pile shaft, the values of the induced lateral stress reduced to almost 50% of that in distance of 1D. The maximum lateral stress is two times and four times more than the initial stress outside and inside the pile, respectively.

During the penetration, the lateral stress at a stationary point increases as the pile penetrates until it reaches the same depth as the station. Afterward, it starts to decrease at a significantly faster rate than it increased previously.

Based on the presented results, it can be argued that the presented model is capable of simulating the complex problem of impact driving with reasonable accuracy.

For future works, a robust coupled formulation can be implemented which captures the pore water effects which can be used for offshore installation problems. There are also several considerations whose effects are still to be studied including the assumption of rigidity against an elasto-plastic pile or possible effects of wave reflection due to the employed boundary conditions.

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