Effects of monopile installation on subsequent lateral response in sand, Part I: pile installation

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ABSTRACT

Monopiles are widely used as the foundation to support offshore wind turbines (OWTs). The response of monopile supported OWTs is strongly affected by the natural frequency of the system relative to the load spectrum, and in turn the natural frequency is governed by the initial stiffness of the foundation. Both fatigue limit state (FLS) and serviceability limit state (SLS) design are also influenced by the foundation stiffness. The stiffness of the foundation during operational loading is a function of the ‘initial’ soil state post-installation, which is influenced by the installation effect. This is the first of a pair of companion papers that investigates the effect of different installation methods on the subsequent response of monopile under lateral loading through extensive numerical analysis. The present paper focuses on quantification of the effect of pile installation on the soil states in sand for three different initial relative densities. The numerical model is first validated against purpose-designed centrifuge tests. Subsequent analyses demonstrate how the soil state, including the void ratio and stresses, is altered from in-situ conditions during pile installation, and are markedly different depending on whether the pile is jacked or impact driven. The results presented here and in the companion paper highlight the need to account for the effects of the installation process on the lateral pile response.

Key words: monopile; installation effect; lateral response; sand; offshore engineering

INTRODUCTION

Offshore wind is an established industry now in European waters, but still in its infancy in other parts of the world. The overall capacity of all wind turbines installed worldwide by the end of 2018 reached 597 GW, with 50,100 MW added in 2018 alone, with strong growth in many Asian countries, and also Brasil and the USA (WWEA 2019). Monopiles remain the most prevalent foundation option for offshore wind turbines (OWTs) representing 75% of all installed foundations in Europe in 2018 (Wind Europe, 2018).

Offshore wind turbines supported by monopiles are typically designed as ‘soft-stiff’ structures, requiring the target natural frequency of the structure to lie within the narrow band between the rotational frequency (1P) and the blade passing frequency (3P) (Arany et al. 2016). Consequently, accurate prediction of foundation stiffness is of great importance as it influences the natural frequency and also the accumulation of rotation under long term cyclic loading, which is limited by a tight serviceability limit state (SLS) criterion of 0.5° permanent rotation (DNV 2014). In practice, geotechnical site investigations aiming at characterising the soil in its original in-situ state are generally used as a design basis. However, the in-situ soil state will be changed due to the pile installation process. The subsequent lateral response depends to a large extent on the post-installation soil state, which becomes the ‘initial’ state for the lateral loading phase. This emphasises the need to understand the effect of pile installation on the surrounding soil.

Published research including numerical (Daryaei et al., 2020; Henke and Grabe, 2008; Labenski et al., 2016) and experimental (De Nicola and Randolph, 2009; Paik et al., 2003; Yu and Yang, 2012) studies have primarily focused on the effect of installation on the

axial – rather than lateral - response with respect to soil plug development and axial capacity of open-ended piles. Quantification of the changes in soil state (stresses and void ratio) caused by pile installation is very challenging using physical modelling techniques. Numerical investigations such as Daryaei et al. (2020), Henke and Grabe (2008), Labenski et al. (2016) and Phuong et al. (2016) demonstrate the possibility of revealing changes in soil states numerically. Numerical results indicate the soil is significantly affected by the installation process, with marked differences in the degree of disturbance from different installation methods. Consequently, the lateral response of monopiles installed using different methods will be different, as also revealed by centrifuge based studies (Dyson and Randolph, 2001; Fan et al., 2019; Klinkvort, 2013). Quantification of the installation effect on the soil state and hence the subsequent lateral response still holds significant uncertainty; this is the focus of this set of companion papers.

The findings discussed here are based on results from numerical simulation of monopile installation and subsequent lateral loading. Both the pile installation model and the lateral loading model are validated against purpose-designed centrifuge tests (Fan et al., 2019), where both the pile installation and lateral loading were conducted in-flight without stopping the centrifuge. The first of the two papers discusses the effect of pile installation on the changes in the surrounding soil with respect to the stress and void ratio. The effect of different installation methods, jacking and impact driving, is examined for three different initial relative densities of the sand. The second paper focuses on the effect of different installation methods on the subsequent response of monopiles under lateral loading. The implications of pile dimensions, relative density driving distance and load

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- 96 eccentricity on the lateral loading response are investigated. The findings highlight the
- 97 importance of the soil state on monopile response.

DEVELOPMENT OF NUMERICAL MODEL

Pile installation was simulated using the Coupled Eulerian-Lagrangian (CEL) technique available within Abaqus/Explicit (Dassault Systèmes, 2014). This has been shown to be an appropriate tool for the simulation of penetration problems in geotechnics where large deformations are involved (Qiu et al., 2011; Tho et al., 2010; Wang et al., 2015). To further validate the numerical model, simulation of cone penetration tests conducted alongside the pile installation experiments is also presented.

CPT model

Figure 1a shows the numerical model of the cone penetration test (CPT). The accuracy of the solution was validated against measured cone resistance from the centrifuge test by Fan et al. (2019), where a 7 mm cone was tested at 100g. Prototype dimensions were used in the analysis. The cone was modelled as a Lagrangian rigid body and the soil was modelled as an Eulerian domain. The dimensions of the soil domain are similar to those used in other 2D Arbitrary Lagrangian-Eulerian (ALE) analyses (Fan et al., 2018; Kouretzis et al., 2014; Liyanapathirana, 2009), uniform mesh with approximately 10 elements beneath the cone radius was defined in the region close to the cone tip.

Pile installation model

Figure 1b shows the numerical model of pile installation. To facilitate comparison with measured data, all input is taken from purpose-designed centrifuge tests by Fan et al. (2019), where a model monopile with an outer diameter of 52.2 mm and a total length of 500 mm was tested at 100g. The wall thickness of the steel tube was 1.0 mm, but epoxy coating to protect the strain gauges brought the overall wall thickness to 2.1 mm. The

corresponding prototype monopile has a diameter (D_{pile}) of 5.22 m and an overall wall thickness of 0.21 m. The pile tip has a wall thickness of 0.1 m. The transition from the epoxy to steel was modelled as shown in Figure 1b. Prototype dimensions were used in the analysis and a penetration depth of 15.9 m ($3.1D_{\text{pile}}$), the same as test conditions, was targeted. The pile was modelled as rigid, with total length of $7.6D_{\text{pile}}$. A second pile geometry of an 8 m diameter pile with a constant 0.1 m wall thickness was also included. The soil was modelled as an Eulerian domain that is $5.4D_{\text{pile}}$ wide and $7.6D_{\text{pile}}$ deep. A void region with a depth of $1D_{\text{pile}}$ was defined at the top (not shown in Figure 1b).

Due to symmetry, one-quarter of the pile and soil domain was modelled. Velocity boundary conditions were imposed at the soil base and side in the vertical and lateral direction respectively. A relatively fine mesh was defined in the vicinity of the pile wall, with approximately 2 elements beneath the pile tip. Considering the size of the problem, it would be computationally unaffordable to increase the mesh density further at the very thin pile tip, as a penetration depth of around 15.9 m (76 times the wall thickness) is required and the stable time increment size in an explicit solver is controlled by the minimum characteristic dimension, i.e. the smallest element size. A mesh convergence study was undertaken to minimise computational cost whilst maintaining the accuracy of the analysis, with validation against experimental results provided below. The size of the soil domain was proven sufficiently large to avoid boundary effects as results reported are consistent with an extra study with a larger soil domain with a width of $10.8D_{\text{pile}}$.

The pile installation analyses were performed in CEL rather than, for instance, ALE due to the open-ended pile geometry, the requirement to model the resulting flow of soil and the mesh distortion associated with modelling large penetration depths.

Soil characteristics and constitutive model

Dry sand conditions are considered in this study. The properties of very fine UWA silica sand used in the centrifuge test are summarised in Table 1. In order to capture the installation effect on the void ratio and stresses in the surrounding soil, a constitutive model that is capable of capturing the stress- and state-dependent behaviour of sand is required. A rate-independent hypoplastic constitutive law (Kolymbas, 1991, 1985) was used, in the form proposed by von Wolffersdorff (1996) with enhancement of intergranular strains according to Niemunis and Herle (1997). The user subroutine used here combines the UMAT implementation (Gudehus et al., 2008) with an interface code (Bienen et al., 2014) to a VUMAT as required for Abaqus/Explicit.

The hypoplastic parameters of very fine UWA silica sand are taken from Pucker et al. (2013) as summarised in Table 2. The parameter ϕ_c is the critical friction angle of sand. The parameters h_s and n control the shape of the limiting void ratio curves. The parameters e_{c0} , e_{d0} and e_{i0} are reference void ratios specifying the positions of limiting void ratio curves. Exponents α and β control dependencies of peak friction angle and soil stiffness on relative density respectively. The parameters m_R , m_T , R , β_R and χ control the stiffness at small strains and load reversals. Documentation and the model implementation are freely available on www.soilmodels.com. This set of hypoplastic parameters was calibrated against oedometer and triaxial tests (Pucker et al. 2013). Numerical simulation

of cone penetration (Fan et al., 2018) shows this set of parameters to be appropriate for loose and medium dense sand ($D_R = 37$ and 47%). Although the hypoplastic constitutive model was developed to capture sand at any relative density using the same set of parameters and merely varying the initial void ratio, in practice limitations are well-known (e.g. Wichtmann and Triantafyllidis, 2016a, 2016b who include useful discussion of the model parameters as well as the strengths and limitations of the model). Here also, the numerical analysis results of oedometer tests on dense sand do not match the test results well (Pucker et al., 2013). It was therefore decided to modify the existing parameters to achieve better agreement for an oedometer element test and also in a boundary value problem for dense sand. The adjusted parameters h_s and n are similar to those proposed by Heins and Grabe (2019) as given in Table 2 and were used for very dense ($D_R = 88\%$) sand.

The initial geostatic stress in the soil was established by applying gravity on the soil domain. A lateral earth pressure coefficient of $K_0 = 1 - \sin\phi = 0.5$ was considered, where ϕ is the critical friction angle of sand. Three initial void ratios were investigated: $e_0 = 0.66$ ($D_R = 38\%$), $e_0 = 0.60$ ($D_R = 60\%$) and $e_0 = 0.52$ ($D_R = 88\%$).

Contact formulations

A general contact formulation was used to describe the interface between the rigid Lagrangian body and the Eulerian soil domain. The shaft surface of the cone penetrometer, the pile internal wall and the pile tip were modelled as frictionless. An intermediate roughness of $\tan\delta/\tan\phi_c = 0.5$ was assumed for the cone tip and the pile external wall, where δ is the interface friction angle between the penetrometer and sand.

The contact properties were selected to reflect the test conditions described in Fan et al. (2019), where the pile external wall surface was coated with epoxy and the pile internal wall surface was relatively smooth (extruded stainless steel).

Installation methods

Two different pile installation methods were investigated in the current study: jacking and impact driving. In the centrifuge experiments, quasi-static pile jacking was simulated by a displacement-controlled method, applying a constant penetration rate of 0.5 mm/s. In the numerical analysis, an increased penetration rate of 5 m/s was applied to minimise computational cost. This will not affect the accuracy of the solution, as demonstrated by Fan et al. (2018) for modelling of cone penetration tests using the ALE technique; Qiu and Grabe (2012), and Tho et al. (2012) for modelling of spudcan penetration using the CEL technique. Increasing the penetration rate would lead to a reduced computational cost while maintaining the accuracy of the solution, but an energy balance check is required to ensure the response still reflects a quasi-static solution. Although jacked piles are not used in practice for offshore monopiles installation, jacking is included in this study as it provides a contrast relative to driven installation and allows direct comparison with the centrifuge model testing where jacked installation was widely used in physical modelling research (Dyson and Randolph, 2001; Klinkvort, 2013)

Dynamic pile impact driving was simulated by a force-controlled method. The peak impact driving force used for validation was taken directly from the measurement in the centrifuge test (Fan et al. 2019). Figure 2 shows the measured impact driving force and the simplified driving force used in the numerical analysis for validation. The magnitude

206 of the impact driving force used in the case study was adjusted for each soil density to
207 ensure the same pile penetration depth is achieved at reasonable computational cost
208 (Table 3).
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VALIDATION OF THE NUMERICAL MODEL

The centrifuge experiments used for validation were conducted in a dry medium dense sand with a relative density of around 38%. Numerical simulation of cone penetration, pile jacking and pile impact driving in the same soil conditions was conducted. In the following discussion, the penetration resistance and impact driving force are normalised by the total cross-sectional area of the penetrometer (A_c for cone penetrometer; A_{pile} for monopile). The penetration depth is normalised by the pile diameter (D_{pile}).

Cone penetration test

Figure 3 compares the cone penetration resistance obtained from the numerical analysis with the experimental data from Fan et al. (2019) for $D_R = 38\%$. Also included are the results from ALE (Arbitrary Lagrangian-Eulerian) analyses, to tie in with the study on numerical stability and efficiency for cone penetrometer tests in sand (Fan et al. 2018). The smoothed cone penetration resistance from the CEL analysis matches well with the lower-bound of the ALE analysis result and the experimental data (Figure 3). The difference in the results of these two types of analysis may be caused by the contact formulation used. A kinematic contact method was used in the ALE analysis, while a penalty contact method was used in the CEL analysis.

Cone penetration resistance for $D_R = 60, 88\%$ is also included in Figure 3 for comparison. The result for medium dense sand ($D_R = 60\%$) is based on the ‘original’ constitutive model parameters. The adjusted constitutive model parameters (Table 3) lead to results for dense sand ($D_R = 88\%$) that agree well with experimental data.

Pile installation

Pile jacking

Figure 4 compares the jacked penetration resistance obtained from numerical analysis with the experimental data from Fan et al. (2019) for $D_R = 38\%$. Overall, good agreement was achieved between the numerical simulation and centrifuge test results. A slight overestimation is visible up to a pile penetration depth of around two pile diameters, but at depths greater than two pile diameters the simulation matches the experimental data well.

Pile impact driving

A peak impact ‘force’ (F/A_{pile}) of 1.2 MPa and a driving frequency of 0.05 Hz (at prototype scale) were used in the centrifuge test. For validation purposes, the same peak impact driving force was used in the numerical analysis. The driving frequency used in the centrifuge test was much lower than in typical field conditions due to the constraint of the maximum achievable driving frequency of the miniature driving hammer; however, since the tests involved dry sand, assumed rate independent, the actual frequency is not relevant. For the numerical simulations, the driving frequency was increased to 4 Hz in the analysis to minimise computational costs. Given the short duration of the force impulse of around 10 ms (Figure 2), the total time per blow of 250 ms (equivalent to 16 return times for stress-waves in the pile) is sufficiently long for the analysis to stabilise before the next blow is initiated.

Figure 5 shows the cumulative hammer blows recorded versus the penetration depth from the numerical analysis. Centrifuge test data from by Fan et al. (2019) is included for

comparison. Two numerical simulations with different starting points of impact driving were conducted. Simulation 1 replicated the test conditions, where impact driving was initiated after a settlement of 0.9 pile diameter under self-weight. A good match of the blowcount of around 27 blows/m may be observed between the numerical analysis and the experimental data. It was not feasible to model the entire driving history of around 1,800 blows considering the extremely high computational cost. Approximately 80 hammer blows were simulated in simulation 1. The agreement between the numerical and experimental results provides confidence in the simulation at relatively shallow penetration. Simulation 2 was carried out to gain insight into the performance of the model at deeper penetration. The pile was jacked to a depth of $2.7D_{\text{pile}}$ before the impact driving was initiated. Approximately 130 hammer blows were simulated in simulation 2. The blowcount from the numerical simulation is around 185 blows/m, which is lower than, but still comparable to, that from the experimental data (300 blows/m). This is likely a result of the jacked pre-embedment, which does not induce the same reduction in void ratio and hence increase in soil strength in the area below the pile tip compared with impact driving. This will be clarified further in the following.

NUMERICAL ANALYSIS RESULTS

The validated numerical model was used to simulate pile installation in sand at three different relative densities. The same pile dimensions and penetration depth as the test conditions were modelled. In addition to the tested condition, sand with a higher relative density of $D_R = 60\%$ and $D_R = 88\%$ was considered. This is because monopiles are generally installed in medium dense and dense sand in the field, as reported for soil conditions at ten offshore wind farms (Arany et al., 2016).

The peak impact driving force and the jacking velocity are summarized in Table 3. Increased peak transient driving force with relative density was applied to drive the pile to the same target penetration depth. A total of 151-229 hammer blows were required to drive the pile to a depth of 15.9 m ($3.1D_{\text{pile}}$) using the impact force given in Table 3. However, it still takes about 3 weeks using 15 logical cores of a computer with 3.10 GHz CPU to simulate a penetration of $3.1D_{\text{pile}}$.

Figure 6 shows the penetration resistance during jacking in sand at different initial relative density. The jacking force required to jack the pile to a penetration of 15.9 m ($3.1D_{\text{pile}}$) is summarized in Table 4. As expected, the jacking force required increases with relative density in order to achieve the same embedment length. The force required to jack the pile to a penetration of $3.1D_{\text{pile}}$ is larger than the peak impact driving force required as shown in Table 3 and Table 4.

Relative density, $D_R = 38\%$

The void ratio distribution and horizontal stress distribution along several vertical paths inside and outside the pile following pile jacking and impact driving in sand with an initial relative density of 38% are given in Figure 7 and Figure 8 respectively. The upper subplots (a, b) and lower subplots (c, d) show the soil state following jacking and impact driving respectively. The subplots on the left (a, c) and subplots on the right (b, d) show the soil state outside and inside the pile, respectively. The same methodology is applied for discussions of similar figures for other relative densities. The legend in the figure denotes the distance to the external or internal pile wall. The initial state is also provided to illustrate the changes. The void ratio contours and horizontal stress contours are shown in Figure 13a and Figure 14a respectively.

Pile jacking resulted in dilation at a distance of $0.04D_{\text{pile}}$ from the pile external wall and $0.05D_{\text{pile}}$ from the internal wall, as shown in Figure 7a, b, due to the monotonic shearing at the pile-soil interface. Dilation was seen up to a depth of $\sim 2.6D_{\text{pile}}$ outside the pile, and to a depth of $\sim 1.5D_{\text{pile}}$ inside the pile. Less dilation was observed inside the pile as the pile internal wall is frictionless. Densification was observed in the lower part inside the pile ($1.5\text{--}3.7D_{\text{pile}}$). The void ratio in the region around the pile tip decreased significantly, from an initial void ratio of 0.66 to around 0.53, due to the displacement of soil caused by the penetrating pile body. At a distance of $0.26\text{--}2.72D_{\text{pile}}$ from the pile external wall, a decrease in the void ratio (to a minimum void ratio of ~ 0.63) was observed. The degree of densification decreases as the distance from the pile wall increases, as expected. A significant increase in the horizontal stress was observed at a depth of $2.9\text{--}3.5D_{\text{pile}}$ inside

and outside the pile (Figure 8a, b). Peak horizontal stresses of 1830 kPa and 3160 kPa were generated inside and outside the pile respectively.

Impact driving resulted in densification both inside and outside the pile as shown in Figure 7c, d. Outside the pile, the greatest densification, to an average void ratio of ~0.48, was observed in the region adjacent to the pile wall. The degree of densification decreases significantly as the distance from the pile wall increases. Densification near the soil surface is significantly higher than at depth. Inside the pile, the soil was found to be densified to a greater extent, to a minimum void ratio of ~0.33. No significant increase in the horizontal stress was observed inside and outside the driven pile (Figure 8c, d). However, an increase in the horizontal stress was observed below the pile tip. Peak horizontal stresses of 360 kPa and 510 kPa were generated, respectively, inside and outside the pile near the tip. A higher peak horizontal stress of around 700 kPa in the area below the pile tip (at a depth of $3.1\text{--}5.5D_{\text{pile}}$) was generated on both sides of the pile wall. Impact driving leads to an increase in the strength of the soil below the pile tip due to the combined effect of an increase in stress and soil densification (a decrease in void ratio). This could explain the relatively lower blowcount reported in simulation 2 in the validation section, as this effect was not reflected by the jacked pre-embedment simulated in the numerical analysis.

Relative density, $D_R = 60\%$

The void ratio distribution and horizontal stress distribution following pile installation for sand with an initial relative density of 60% are given in Figure 9 and Figure 10 respectively.

Pile jacking resulted in dilation at a distance of $0.04D_{\text{pile}}$ from pile external wall and $0.05D_{\text{pile}}$ from pile internal wall as shown in Figure 9a, b. The dilation was extended to a depth of $2.7D_{\text{pile}}$ and $2.1D_{\text{pile}}$ outside and inside the pile respectively (Figure 9a, b). In the region around the pile tip, the soil was densified from an initial void ratio of 0.60 to a void ratio of 0.56 and 0.49 inside and outside the pile respectively. Outside the pile, dilation was observed at a distance of $0.26D_{\text{pile}}$ from the pile external wall at shallow depth up to $\sim 0.3D_{\text{pile}}$ from the soil surface (Figure 9a). At a distance of $0.52\text{--}2.72D_{\text{pile}}$ from the pile wall, the soil was densified to a minimum void ratio of 0.58 and the degree of densification decreases as the distance from the pile wall increases. A significant increase in the horizontal stress was observed at a depth of $\sim 2.9\text{--}3.5D_{\text{pile}}$ inside and outside the pile as shown in Figure 10a, b. Peak horizontal stresses of 2540 kPa and 4630 kPa were generated inside and outside the pile respectively.

Impact driving resulted in densification inside and outside the pile, as shown in Figure 9c, d. Outside the pile, the void ratio next to the pile external wall was 0.48 on average. Inside the pile, the void ratio was around 0.46 on average. The void ratio at the relatively low-stress region near the soil surface was slightly higher. An increase in the horizontal stress was observed at a depth of $2.7\text{--}5.5D_{\text{pile}}$ (Figure 10c, d). Nearly identical peak horizontal stresses of 830 kPa were generated on both sides of the pile wall in the region below the pile tip. Peak horizontal stresses of 1000 kPa and 1250 kPa were generated inside and outside the pile in the region around the pile tip.

Relative density, $D_R = 88\%$

The void ratio distribution and horizontal stress distribution following pile installation for sand with an initial relative density of 88% are given in Figure 11 and Figure 12 respectively.

Pile jacking resulted in dilation at $0.26-2.72D_{\text{pile}}$ from the pile wall outside the pile (Figure 11a), in contrast to the densification observed in medium dense sand. The degree of dilation decreases as the distance to the pile wall increases. The most significant dilation was observed in the region around the pile-soil interface inside and outside the pile. Densification was observed in the deeper region ($1.6-3.2D_{\text{pile}}$) outside the pile due to soil displacement caused by the penetrating pile body. Inside the pile, a greater dilation was observed up to a depth of $\sim 3.2D_{\text{pile}}$ (Figure 11b). A more significant increase in the horizontal stress outside the pile was observed at a depth of $\sim 3.0-3.3D_{\text{pile}}$ (Figure 12a, b). The increase in the horizontal stress inside the pile was much less owing to the frictionless pile internal wall. Peak stresses of 3870 kPa and 10280 kPa were generated inside and outside the pile respectively.

Impact driving resulted in densification inside and outside driven piles (Figure 11c, d). Outside the pile, the void ratio following impact driving was ~ 0.48 on average, very close to the minimum void ratio reported in medium dense sand ($D_R = 38\%$ and $D_R = 60\%$). The maximum densification was reported at the distance of $0.26D_{\text{pile}}$ from the pile external wall, instead of next to the pile-wall interface as reported in medium dense sand. This could be ascribed to the more significant dilation effect at the soil-pile interface in this relatively dense sand. Inside the pile, greater densification was observed. Similarly.

375 the densification in the region around the pile-soil interface inside the pile is less due to
376 the dilation effect. The distribution of the horizontal stress inside and outside in dense
377 sand (Figure 12c, d) is more complex as the dilation effect becomes more dominant. Peak
378 horizontal stress was observed in the region around the pile tip and an increase was also
379 observed in the area below the pile tip. Overall, peak horizontal stress in driven piles was
380 significantly lower than for jacked piles. Peak stresses of ~2280 and ~1450 kPa were
381 reported inside and outside the pile respectively.

DISCUSSION OF THE RESULTS

The soil state, characterised by void ratio and horizontal stress, is significantly affected by the pile installation. The soil state following different installation methods - pile jacking and impact driving are substantially different and also depends on the initial sand relative density. This is further explored in the following based on the results from the numerical analysis results.

Void ratio

Both pile jacking and impact driving lead to significant changes in void ratio, as also noted by a number of researchers (Heins et al. 2018; Henke and Grabe, 2008; Labenski et al. 2016; Lundberg, 2015; Phuong et al. 2016). The differences in the changes in void ratio are significant between pile jacking and impact driving as shown in Figure 7, Figure 9 and Figure 11 for sand at different relative densities. In general, pile jacking leads to densification in medium dense sand and dilation in dense sand outside the pile. Dilation is observed inside jacked piles and the degree of dilation increase as the initial relative density of sand increases. In contrast, impact driving leads to densification inside and outside the pile for both medium dense and dense sand. Sand tends to reach the same void ratio outside the pile following impact driving regardless of the initial relative density (Figure 7c, Figure 9c and Figure 11c).

Figure 15 shows the void ratio distribution at different depths from the soil surface after installation of 15.9 m ($3.1D_{\text{pile}}$). The horizontal axis denotes the normalised distance (y/D_{pile}) to the centre of the pile and the vertical axis is the void ratio (e) after pile installation. It is clear that the area of densification caused by impact driving is much

greater than that resulting from jacking in both the radial and vertical direction. The degree of densification due to impact driving is also much greater than following jacking.

Horizontal stress

Both pile jacking and impact driving lead to an increase in the horizontal stress inside and outside the pile. Figure 16 shows the maximum horizontal stress reported inside and outside the pile following pile jacking and impact driving in sand at different relative densities. However, the degree of the increase in the horizontal stress in jacked piles is significantly higher than in impact-driven piles, as also noted by Henke and Grabe (2008) and Labenski et al. (2016). Impact driving leads to an increase in the horizontal stress in the region below the pile tip. The combined effect of the increase in the stress and the decrease in void ratio (soil compaction) leads to the increase of the soil strength. The magnitude of the peak horizontal stress increases as the initial relative density of sand increases. In the current study, higher stress is generally reported outside jacked piles as the external pile wall has a higher friction coefficient. The magnitude of the peak stress is affected by the contact properties between the pile and soil. Figure 17 shows the evolution of horizontal stress at $0.26D_{\text{pile}}$ from the external pile wall surface for $D_R = 38\%$ as the pile penetrates. Results for impact driving at $z/D_{\text{pile}} = 0.6$ and 1.2 are identical as the driven pile was pre-jacked to $z/D_{\text{pile}} = 1.2$. For greater penetrations, pile jacking leads to a significant build-up in horizontal stress while for impact driving the magnitude of horizontal stress increase is less marked, although the zone of increased stress extends deeper.

Soil plugging

Jacked piles exhibit a greater tendency for soil plugging than impact-driven piles. This is supported by the observation of a more significant stress increase inside jacked piles and is consistent with findings in published research (De Nicola and Randolph, 2009; Henke and Bienen, 2014; Henke and Grabe, 2008). Full plugging is judged unlikely inside jacked piles due to the relatively small penetration depth to diameter ratio, although arching is thought to have occurred (Figure 13, Figure 14). The ratio of pile penetration resistance to cone tip resistance can also indicate the likelihood of plugging. In pile penetration prediction, the end bearing component under static loading rises gradually with the degree of soil plugging (so-called incremental filling ratio) from around $0.2q_c$ for piles installed unplugged, to $0.6q_c$ for closed ended (or fully plugged) conditions (Xu et al., 2008). The maximum ratio between pile penetration resistance and the cone tip resistance at the final penetration of $3.1D_{\text{pile}}$ is only 0.38 (Table 4), even without deducting the contribution from friction on the external wall, indicating that only partial plugging, rather than a fully plugged condition, has occurred.

Soil settlement

The dynamic impact driving leads to soil settlement outside the pile. This is in line with the experimental observation of a settlement of the sample during the centrifuge test (Fan et al. 2019). The degree of settlement decreases as the relative density of sand increases (Figure 13, Figure 14). The soil movement inside impact-driven piles is dependent on the initial relative density of sand. Soil settlement inside impact-driven piles is observed for medium dense sand case. This is supported by the observed densification of sand inside

the pile regardless of relative density (Figure 7d, Figure 9d and Figure 11d). However, soil displacement due to the penetration of pile body becomes dominant and results in soil heave inside the pile for dense sand. Pile jacking leads to soil heave inside and outside the pile due to the combined effect of dilation of sand and displacement of soil caused by the penetrating pile body. The degree of soil heave increases as the initial relative density of sand increases.

IMPLICATION OF PILE GEOMETRY ON THE SOIL STATE CHANGE

All discussions in the section above are based on the pile dimensions (5.22 m pile) modelled experimentally. Monopiles used in the field generally have a significantly higher diameter to wall thickness ratio of around 80 to 100 (Velarde and Bachynski, 2017). Additional pile installation analyses considering a second pile with a diameter of 8 m and a wall thickness of 0.1 m were conducted to investigate the effect of pile geometry on the degree of soil state changes.

While the entire pile jacking process was simulated, a reduced impact driving distance of $1.2D_{\text{pile}}$ was simulated for the 8 m pile. Piles were pre-jacked to $1.9D_{\text{pile}}$ before impact driving was initiated in order to limit the extremely high computational costs for pile installation analysis, especially for the 8 m diameter pile with its significantly higher diameter-to-wall thickness ratio. This requires a very fine mesh relative to the pile diameter, and hence a small stable time increment for the pile installation analysis since that is governed by the characteristic element size. In addition the absolute penetration depth for the large diameter pile is increased in order to match the target embedment

length-to-diameter ratio. The results presented for the 8 m pile are based on a pre-jacking of $1.9D_{\text{pile}}$ and impact driving of $1.2D_{\text{pile}}$.

Figure 18 shows the post-installation soil states following different installation methods for the 8 m pile in sand with an initial relative density of 38%. Overall, the area of densification is significantly smaller in both radial and vertical directions, and the degree of densification is significantly lower, compared with the 5.22 m pile (Figure 18c). The horizontal stress increase along the pile length is more significant for jacked piles than for impact-driven piles, and some arching occurs inside the jacked piles (Figure 18b, d). The post-installation soil state is mapped as the initial soil state for the lateral loading model. The effect of pile geometry on the subsequent response was further investigated in the companion paper (Fan et al., 2020).

CONCLUSIONS

This paper has discussed findings from an investigation of the effect of different pile installation methods on the surrounding soil for dry sand conditions. Reasonably good agreement between the calculated pile jacking resistance and blowcounts during impact driving, relative to data from centrifuge model tests, was obtained, as well as for related cone penetrometer test data. These comparisons validated the performance of the numerical model. Subsequently, the effect of pile installation by jacking or impact driving on the state of the surrounding sand was investigated, including quantification of changes in stresses and void ratio in the sand at three different initial relative densities.

The following key conclusions are drawn based on the numerical results.

- Jacked piles exhibit a greater tendency for soil arching than impact-driven piles for the simulated embedment of 3.1 pile diameters, confirming previous findings.
- Impact driving analysis commencing from pile pre-embedment achieved through jacking agreed well with experimental results of fully impact-driven pile installation. This technique results in significant computational savings while retaining representation of the entire pile installation process. Compared with the widely used wished-in-place assumption, the effect of soil displacement due to the penetrating pile body is accounted for using this technique.
- The effect of the pile installation process on the surrounding soil can be substantial. The soil state resulting from jacked and impact-driven pile installation differ significantly in terms of both stresses and void ratio. The dynamic impact driving process leads to much greater soil densification inside and outside the pile.

Conversely, the quasi-static pile jacking leads to higher horizontal stresses inside and outside the pile.

- The results illustrate how the initial sand relative density affects the soil state changes resulting from the pile installation process.

The findings highlight the differences between the original in situ soil state and that at the end of pile installation. These differences will have an important influence on the in-service lateral pile response as discussed in the companion paper (Fan et al., 2020).

The findings discussed here are based on a 5.22 m diameter test pile with a diameter to wall thickness ratio of around 25, which enabled experimental validation. Monopiles used in the field generally have a significantly higher diameter to wall thickness ratio, resulting in a significantly lower effective area ratio. Pile installation leads to less marked changes in soil state than those reported here. This was demonstrated through pile installation analysis using a second pile geometry with $D = 8$ m and $D/t = 80$, with the effect on the lateral response discussed in the companion paper (Fan et al., 2020).

LIMITATIONS

This study provides insights into the effects of monopile installation on the changes in soil state, but a number of limitations exist, including:

- Limited range of pile geometries have been investigated in this study.
- The pile was modelled as rigid, therefore the pile deformation during pile installation has not been accounted for in this study.
- Soil ageing observed in field conditions has not been considered in this study.

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- 522 • Loss of soil fabric due to the pile installation process and grain crushing possible
523 at high stresses has not been considered in this study.

524

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532

533 NOTATIONS

A_c	[m ²]	Cone penetrometer cross sectional area, $\pi D_c^2/4$
A_{pile}	[m ²]	Pile total (gross) cross sectional area, $\pi D_{pile}^2/4$
d_{50}	[mm]	Mean grain/particle size
D_c	[m]	Pile outer diameter (steel)
D_{pile}	[m]	Overall pile diameter (including epoxy)
D_R	[-]	Relative density of sand
e	[-]	Void ratio
F	[N]	Pile impact driving force, or penetration resistance
G_s	[-]	Specific gravity
T	[m]	Overall pile wall thickness (including epoxy)
U	[-]	Index of uniformity
y	[m]	Distance to the centre of pile
δ	[°]	Interface friction angle between pile and sand
γ	[kN/m ³]	Sample dry density
$\gamma_{d\ min}$	[kN/m ³]	Minimum dry density
$\gamma_{d\ max}$	[kN/m ³]	Maximum dry density
σ_{11}	[kPa]	Horizontal stress

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- 647

648 **TABLES**

649 Table 1 Properties of very fine UWA silica sand (Chow et al., 2018)

Parameter	Value	Description
d_{50}	0.18	Mean particle size (mm)
U	1.67	Index of uniformity
$\gamma_{d\ min}$	14.7	Minimum dry density (kN/m ³)
$\gamma_{d\ max}$	17.4	Maximum dry density (kN/m ³)

650

651 Table 2 Hypoplastic model parameters for UWA silica sand (Pucker et al., 2013)

Parameter	Value	Description
φ_c	30	Critical state friction angle (°)
h_s	1354 (16248)	Granular hardness (MPa)
n	0.34 (0.306)	Exponent
e_{d0}	0.49	Minimum void ratio
e_{c0}	0.76	Critical void ratio
e_{i0}	0.86	Maximum void ratio
α	0.18	Exponent
β	1.27	Exponent
m_R	5.16	Stiffness ratio at a change of direction of 180°
m_T	3.07	Stiffness ratio at a change of direction of 90°
R	1×10^{-4}	Max. value of intergranular strain
β_R	0.58	Exponent
χ	5.74	Exponent

652 Note:

653 The values given in the parentheses are parameters used for dense sand.

654

655 Table 3 Numerical analyses summary

Case	Relative density	Installation method	Normalised peak driving force [MPa]	Number of hammer blows
1	38%	Impact driving	2.8	199
2	60%		3.7	152
3	88%		5.6	227
Jacking velocity [m/s]				
4	38%	Jacking	5.0 m/s	
5	60%			
6	88%			

656 Note:
657 The same penetration depth of 15.9 m ($3.1D_{pile}$) was targeted.

Table 4 Summary of penetration resistance following pile jacking to 15.9 m and
corresponding cone tip resistance

D _R	Penetration resistance	Cone tip resistance	Ratio
[-]	[MPa]	[MPa]	[-]
38%	2.9	7.6	0.38
60%	3.9	11.4	0.34
88%	12.4	53.4	0.23

FIGURE

Figure 1 (a) CPT model; (b) Pile installation model

Figure 2 Time history of impact driving force

Figure 3 Cone resistance profile from experiments and numerical simulations

Figure 4 Validation of pile jacking model, penetration resistance during jacking

Figure 5 Validation of pile driving model, cumulative hammer blowcount versus the penetration depth

Figure 6 Penetration resistance during pile jacking and cone resistance profile

Figure 7 Void ratio distribution along vertical path at different distance from pile wall (a, b: after jacking; c, d: after impact driving), $D_R = 38\%$

Figure 8 Horizontal stress distribution along vertical path at different distance from pile wall (a, b: after jacking; c, d: after impact driving), $D_R = 38\%$

Figure 9 Void ratio distribution along vertical path at different distance from pile wall (a, b: after jacking; c, d: after impact driving), $D_R = 60\%$

Figure 10 Horizontal stress distribution along vertical path at different distance from pile wall (a, b: after jacking; c, d: after impact driving), $D_R = 60\%$

Figure 11 Void ratio distribution along vertical path at different distance from pile wall (a, b: after jacking; c, d: after impact driving), $D_R = 88\%$

Figure 12 Horizontal stress distribution along vertical path at different distance from pile wall (a, b: after jacking; c, d: after impact driving), $D_R = 88\%$

Figure 13 Void ratio contour after installation (left: jacking; right: impact driving)

Figure 14 Horizontal stress contour after installation (left: jacking; right: impact driving)

684 Figure 15 Void ratio distribution at different depth after installation of 15.9 m
685 (3.1D_{pile}), a) D_R = 38%, b) D_R = 60%, c) D_R = 88%

686 Figure 16 Peak horizontal stress inside and outside pile after installation

687 Figure 17 Evolution of horizontal stress at 0.26D_{pile} from pile external wall surface
688 for D_R = 38%

689 Figure 18 Post-installation, initial state for lateral loading for 8 m pile, void ratio,
690 e (LHS), horizontal stress (kPa), σ_{11} (RHS), D_R = 38%.

691

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Figure 11 Void ratio distribution along vertical path at different distance from pile wall
(a, b: after jacking; c, d: after impact driving), $D_R = 88\%$

Figure 12 Horizontal stress distribution along vertical path at different distance from pile wall (a, b: after jacking; c, d: after impact driving), $D_R = 88\%$

a) $D_R = 38\%$ ($e_0 = 0.66$)

b) $D_R = 60\%$ ($e_0 = 0.60$)

c) $D_R = 88\%$ ($e_0 = 0.52$)

Figure 13 Void ratio contour after installation (left: jacking; right: impact driving)

a) $D_R = 38\%$

b) $D_R = 60\%$

c) $D_R = 88\%$

Figure 14 Horizontal stress contour after installation (left: jacking; right: impact driving)

Figure 15 Void ratio distribution at different depth after installation of 15.9 m ($3.1D_{pile}$),
a) $D_R = 38\%$, b) $D_R = 60\%$, c) $D_R = 88\%$

Figure 16 Peak horizontal stress inside and outside pile after installation

Figure 17 Evolution of horizontal stress at $0.26D_{pile}$ from pile external wall surface for

$$D_R = 38\%$$

a) Jacking, e , $D_R = 38\%$

b) Jacking, σ_{11} , $D_R = 38\%$

c) Impact driving, e , $D_R = 38\%$

d) Impact driving, σ_{11} , $D_R = 38\%$

Figure 18 Post-installation, initial state for lateral loading for 8 m pile, void ratio, e (LHS), horizontal stress (kPa), σ_{11} (RHS), $D_R = 38\%$.