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THE IMPACT OF INSTALLATION METHOD ON MONOTONIC LOADING OF MONOPILES IN SAND

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ABSTRACT

Monopiles are the most commonly used foundations for offshore wind turbines (OWTs). To date these are mostly impact driven into the seabed using high-capacity hammers. The noise and vibrations caused by driving create environmental concerns and the increasing size of these foundations has resulted in alternative installation methods being explored. In this paper a comparison is made between piles installed by vibration and driving. The results of full-scale field trials for pile pairs installed in dense sand by driving and vibration are presented. Lateral load tests were performed to determine the effect of installation method on the capacity and stiffness of the piles. In this paper 3D finite element model predictions of the pile response are reported. The soil model adopted was the HS small model in PLAXIS.

The measured pile responses are compared to predictions made using soil parameters derived using correlations with Cone Penetration Test, CPT end resistance q_c value (known herein as the direct approach) and through correlation with sand relative density. It is seen that the stiffness and capacity of the vibratory installed piles were lower than driven piles. The finite element model predictions using the input parameters based on relative density underestimate the stiffness response at low displacement levels whilst providing good predictions for displacements in excess of 2cm. The direct correlation approach provided good predictions of the initial response whilst over-estimating the stiffness for displacements above 2cm. The influence of sand creep on the field response of the piles and the performance of the numerical predictions is discussed.

Keywords: Monopile, CPT, sand, lateral loading, vibratory driving

INTRODUCTION

The offshore wind sector is growing rapidly across the globe as innovations driven by research and development drive cost reductions which have resulted in OWTs becoming cost competitive with energy sources. A significant driver for these savings has been the increase in OWT size and output which leads to the requirement for larger foundations. Large diameter monopiles remain the most efficient and cost-effective foundation system, with approximately 75% of OWTs founded on piles with diameters, D ranging from 4m to 10m and embedment depth L, leading to foundations with L/D ratios in the range 3 to 6. The installation process involves impact driving and as the pile size increases so does the energy required to install the pile causing concerns over fatigue and the environmental impact caused. Offshore pile driving raises concerns primarily with respect to hearing damage and habitat displacement for mammals, See De Jong and Ainslie (2008) and Snyder and Kaiser (2009) and allowable sound exposure levels have been set by many countries, See Meulendijk-de Mol et al. (2014). As a result, alternative methods of installing are of interest such as jacking, Igoe et al. (2011) and vibratory methods which are a focus of this paper.

For the design of offshore wind turbines it is important to consider a number of limit states including the ultimate limit state (capacity) controlled by the soil strength and the serviceability limit state, governed by the stiffness response during in-service loading. For the case of OWTs where dynamic interaction effects are significant conventional design approaches such as the API p-y approach (API 2010) are unable to capture the footing response (Doherty and Gavin 2011). Significant research effort notably work resulting from the industry sponsored PISA project, See Byrne et al. (2019), Zdravković et al. (2019) and others has resulted in improved understanding of behaviour of driven offshore piles, See Figure 1.

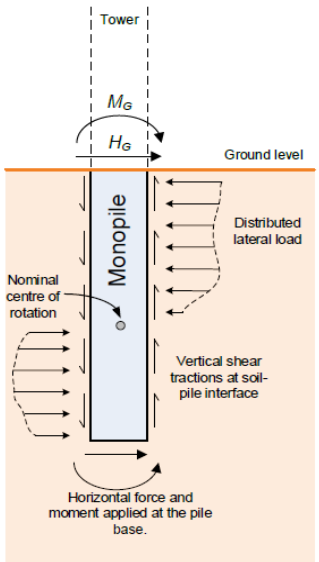

Figure 1 Soil reaction components for a monopile, after Byrne et al. (2015)

Key insights of the impact of vibratory installation have been determined from laboratory tests and large-displacement finite element (FE) analyses. Remspecher et al. (2018) used the Digital Image Correlation method (DIC) to identify density changes around a half model pile with a diameter of 20 cm, tested in a container with a glass panel. There results indicate a zone of loosened soil (-20% reduction in relatively density) in a shear zone with a thickness equal to the pile wall thickness. Outside of this interface shear zone, the soil within the pile (plug) generally loosened, whilst the soil outside the pile densified by up to 20% over an influence zone of approximately 10 times the pile wall thickness.

Labenski et al. (2016) used a finite element formulation implemented in Abaqus with the results being validated using centrifuge tests to investigate the effect of installation method on the formation of the internal soil plug in open-ended piles. The initial void ratio of the sand they modelled was set at e_0=0.7 and they found that vibratory installation caused loosening of soil at the pile-soil interface. In contrast the soil outside this interface shear zone was densified in a region extending up to 5.5 times the pile diameter. For an impact driven pile, the numerical simulations also show loosening at the pile-soil interface, however, outside this interface, the extent of the zone of slightly densified was much smaller, extending to only 1.5 times the pile diameter.

FIELD TESTS

In order to investigate the impact of vibratory installation when compared to traditional impact installation, a joint industry funded project known as the VIBRO project was undertaken at the Cuxhaven site in Germany in 2014, See Gattermann et al. (2015), Hebig and Ossig (2016) and Moorman et al. (2016). Full-scale monopiles with external diameter, D of 4.3m, penetration length, L of 18.2m (L/D = 4.2) and wall thickness of 40mm to 45 mm were installed using vibration or driving in a sand quarry. The site consists of a deep deposit of glacial, over-consolidated dense sand, with a 1m thick clay layer evident at 5m below ground level, See Figure 2.

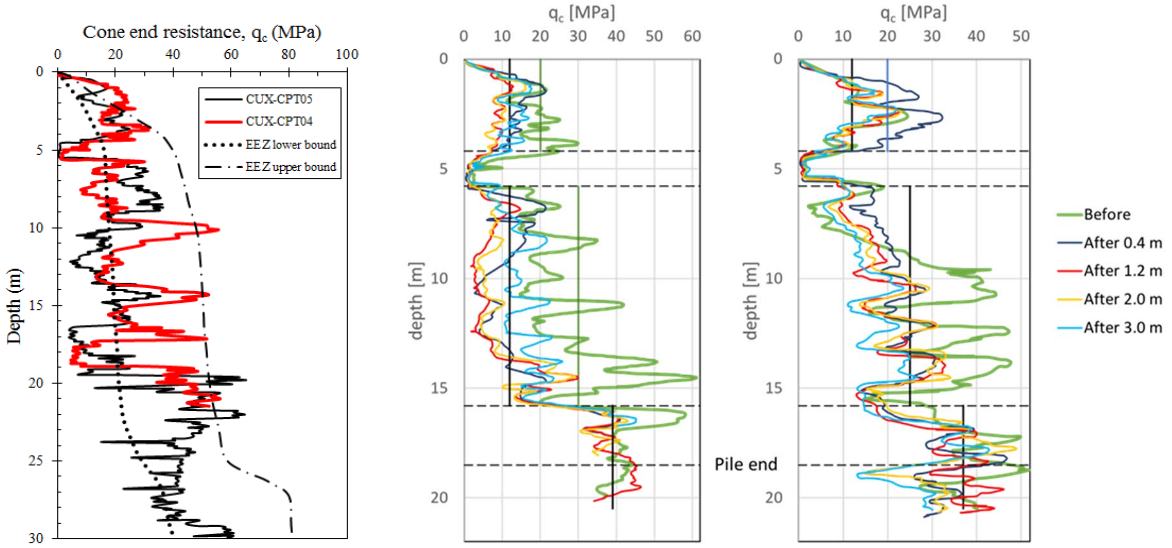

Figure 2 (a) CPT results from Cuxhaven compared to representative wind farm locations in the EEZ, modified from Hebig and Ossig (2016) (b) effect of installation on q_c for vibrated pile (c) driven pile.

Hebig and Ossig (2016) note the CPT profile of the deposit See Figure 2 is consistent with development sites for offshore wind projects in the exclusive economical, EEZ zone located in the Southern part of the North Sea which has a similar geological history. The cone end resistance, q_c is typically in the range 10 to 30 MPa over the depth of the test piles. The water table was 3m below piling platform level. The piles were allowed to age after installation and during this set-up period CPT tests were performed at radial distances of 0.4 m, 1.2 m, 2.0 m and 3.0 m from the pile shaft. All tests showed that the CPT resistance changed after installation, See Figure 2b and 2c. For the vibrated pile, Figure 2b the q_c values reduced along the embedded length, with the smallest reductions evident near the pile tip and the largest being in high strength sand layers. For the driven pile the CPT value mostly increased over the first 8m, and decreased over the remainder of the pile length, See Figure 2c. Again the largest reductions were evident in high-strength sand layers.

FINITE ELEMENT ANALYSES

The FE analyses described in the paper were conducted using Plaxis 3D. The mesh was comprised of 10-node tetrahedral soil elements, See Figure 3a. A circular volume extending to 4 m from the pile shaft was created to achieve better mesh refinement, See Brinkgreve et al. (2017). The pile was modelled as a hollow cylinder, the wall consisted of steel plate with Young's modulus E_{pile} = 210GPa and a Poisson's ratio of 0.3. Rough interfaces, R_{inter} = 1.0 were implemented and the option 'gap closure' option was activated. Lateral loads were applied 1m above ground level to mimic the field test.

To account for the clay layer present around 5m below ground level and to model variability the variability of q_c with installation method, depth and radial distance from the pile evident in Figure 2, four soil layers were considered, an upper sand, layer 1, clay, layer 2 and two lower sand layers (3 and 4 respectively). To investigate the impact of installation method vertical layers were considered, See Figure 3b where the post-installation soil properties could be varied radially from cluster A where the largest reduction of strength and stiffness was imposed, to cluster D where the free-field, initial conditions are assumed to apply.

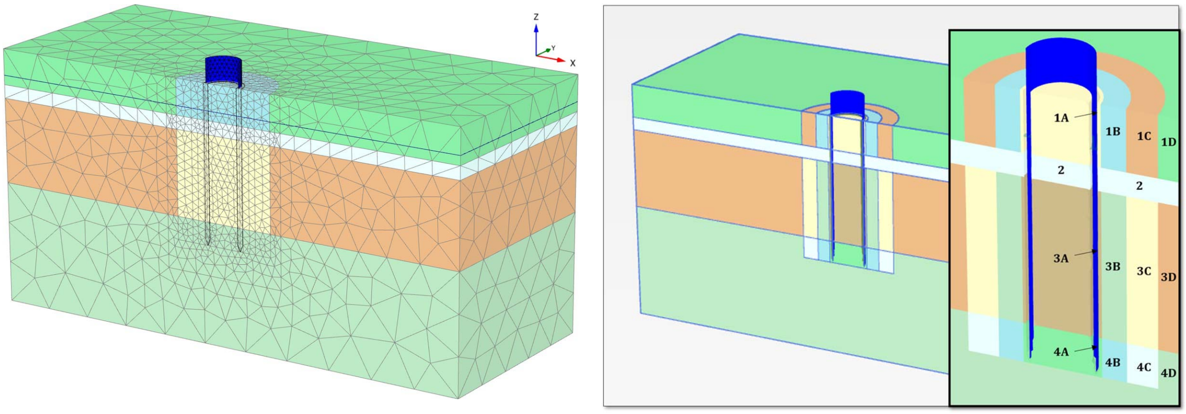

Figure 3 (a) 3D FE mesh used in the analyses (b) details of the horizontal and vertical layering assumed

The sand was modelled with the Hardening Soil Small strain model (HSS). As high quality lab test data with which to determine the input parameters were not available, two approaches to derive these values were taken. The first relates the soil properties to relative density using correlations proposed by Brinkgreve et al. (2010), known herein as Method 1. The equations are summarised In Table 1 below where: RD values obtained by testing or correlation are expressed as percentages and the references pressure, p_{ref} = 100 kN/m^2.

$$\gamma_{unsat} = 15 + 4.0RD/100 \ [kN/m^3] \qquad [1]$$

$$\gamma_{sat} = 19 + 1.6RD/100 \ [kN/m^3] \qquad [2]$$

$$E_{50}^{ref} = 60RD/100 \ [MN/m^2] \qquad [3]$$

$$E_{oed}^{ref} = 60RD/100 \ [MN/m^2] \qquad [4]$$

$$E_{ur}^{ref} = 180RD/100 \ [MN/m^2] \qquad [5]$$

$$G_0^{ref} = 60 + 68RD/100 \ [MN/m^2] \qquad [6]$$

$$m = 0.7 - RD/320 \ [-] \qquad [7]$$

$$\gamma_{0.7} = (2 - RD/100) \cdot 10^{-4} \ [-] \qquad [8]$$

$$\varphi' = 28 + 12.5RD/100 \ [°] \qquad [9]$$

$$\psi = -2 + 12.5RD/100 \ [°] \qquad [10]$$

$$R_f = 1 - RD/800 \ [-] \qquad [11]$$

The second approach known as Method 2 or the direct method adopted the correlations suggested by Murphy et al. (2018) which uses direct correlations to q_c where possible to determine the input properties.

$$\varphi' = 17.6 + 11\log\left[\frac{q_t}{p_a}\left(\frac{p_a}{\sigma'_{v0}}\right)\right]^{0.5} \ [°] \qquad [12]$$

$$\psi = \varphi' - 30[°] \quad [13]$$

$$E_{oed} = q_c{}^{1.78-0.0122RD} \;\; [kN/m^2] \quad [14]$$

$$E_{50} = \frac{(1-2\nu)(1+\nu)}{1-\nu} E_{oed} \;\; [kN/m^2] \quad [15]$$

$$E_{ur} = 3 \cdot E_{50} \;\; [kN/m^2] \quad [16]$$

$$G_0 = \frac{E_0}{2(1+\nu)} \;\; [kN/m^2] \quad [17]$$

$$OCR = \left[\frac{1.33 q_t{}^{0.22}}{K_{0NC} \cdot \sigma'_{v0}}\right]^{\frac{1}{\sin\phi' - 0.27}} \quad [18]$$

$$K_{0NC} = 1 - \sin\phi' \quad [19]$$

$$K_0 = K_{0NC} \cdot OCR^{\sin\phi'} \quad [20]$$

The reference moduli that serve as input into Plaxis 3D are calculated with the following equations, where the exponent m defines the stress dependent stiffness and c is the effective cohesion.

$$E_{50}^{ref} = E_{50} / \left(\frac{c\cos\phi - \sigma'_3 \sin\phi}{c\cos\phi + p_{ref}\sin\phi}\right)^m \quad [21]$$

$$E_{oed}^{ref} = E_{oed} / \left(\frac{c\cos\phi - \sigma'_1 \sin\phi}{c\cos\phi + p_{ref}\sin\phi}\right)^m \quad [22]$$

$$E_{ur}^{ref} = E_{ur} / \left(\frac{c\cos\phi - \sigma'_3 \sin\phi}{c\cos\phi + p_{ref}\sin\phi}\right)^m \quad [23]$$

The relative density for both methods was derived using the approach suggested by Jamiolkowski et al. (2003)

$$RD = \frac{1}{C_2} \ln\left(\frac{q_c/p_a}{C_0(\sigma'/p_a)^{C_1}}\right) \quad [24]$$

Where C_0, C_1 and C_2 have values of 24.94, 0.46 and 2.96 respectively.

RESULTS

The load tests were performed using a maintained load procedure with the progression of load-steps controlled by creep displacement criteria and included two unload steps. The load-displacement response of the piles is compared in Figure 4, where it is clear that the pile installed using the vibratory installation method had a lower stiffness and resistance than the impact driven piles.

In the finite element analyses summarised in Figure 4b, the method of choosing the soil properties, e.g. using Method 1 or Method 2, had a much more significant impact on the predicted response than modelling the installation effects (by reduction of q_c) in the zones 1 to 4. Using parameters derived using Method 1 resulted in significantly lower pile stiffness and capacity than Method 2. For a given method, the FE models predicted much smaller differences due to installation method than those measured, compare Figure 4a and Figure 4b.

Comparing the measured and predicted response of the driven pile, See Figure 4c, Method 1 under-predicts the stiffness over the entire test and under-predicts the peak measured capacity by around 33%. Method 2 provides an excellent prediction of the initial stiffness and of the pile load resistance for applied loads up to 8MN. Thereafter, the FE prediction over-estimates the pile resistance. Comparing the measured and predicted response of the vibro pile, See Figure 4cd, Method 1 again under-predicts the stiffness over the entire test. However, given the softer measured response of the pile, the FE predictions seems more reasonable, though the final measured capacity is under-predicted by 33%. Method 2 again provides an excellent prediction of the initial stiffness however, for loads above 5MN, the approach over-predicts the pile resistance.

It is of interest to note from the field test results that the effects of creep influence the measured pile response, See Figure 4a, particularly at higher load levels. Similar effects were visible in field tests reported by Murphy et al. (2018) and Mc Adam et al. (2019) when the applied lateral load exceeded approximately 30 to 50% of the maximum applied load. Given that the HSS model adopted does not account for creep, it would seem that Method 2 provided realistic estimates of the pile response when creep effects were not significant in the tests.

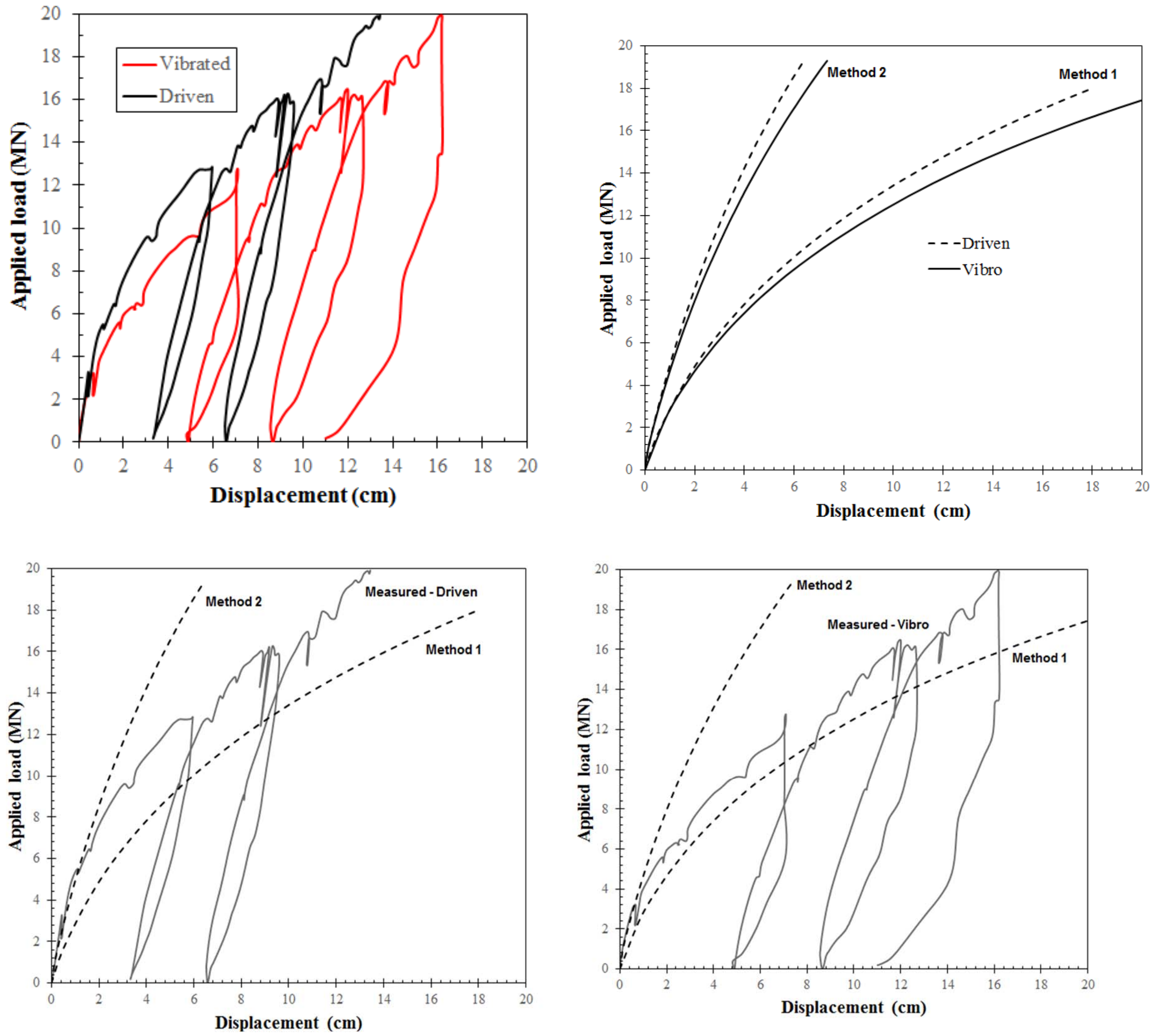

Figure 4 (a) Field measurement of the lateral load-displacement response of a driven and vibrated pile at Cuxhaven (adapted from Moorman et al. 2016) (b) Summary of FE analyses (c) FE predictions of the driven pile (d) FE predictions of the vibro pile

DISCUSSION AND CONCLUSION

The paper presents finite element analyses, FE predictions of the response of laterally loaded piles in sand. Specifically the effect of the installation method, driven or vibration on the stiffness and capacity of the pile was investigated. Data from field tests from the literature were analysed. The field tests revealed that a driven pile had a higher stiffness and larger resistance at a given displacement than a pile installed using vibration. The field test data included CPT tests performed before and the pile tests. These allowed the impacts of installation method to be modelled directly in the FE predictions using zones of disturbance.

Two methods of determining the soil properties in the constitutive soil model were compared. Method 1 is based sand relative density, whilst method 2 used largely direct correlation between soil properties and q_c. It was apparent that Method 2 provided a better match to the initial stiffness response of the piles regardless of the installation method. Method 2 over-estimated the maximum lateral pile resistance developed and Method 1 underestimated the value for both installation methods. The field tests showed significant creep effects that were not modelled in the FE analyses. Both methods of deriving soil properties suggested relatedly small impact of installation method (or in fact the extent of the zone of disturbance revealed by the post-installation CPT values) on the initial stiffness response of the piles. The principal impact suggested was that the displacement required to mobilise the maximum lateral load increased by ≈10%. In the field the load-displacement response of the driven and vibrated piles were quite similar until the lateral load exceeded 3 MN.

Given the field test results suggest that lateral load response of the pile is affected by installation method, and the impact seems to be sensitive to sand strength and installation energy, frequency etc. adopted, further work on the impact of installation on the pile-soil stiffness is required. The authors suggest that until robust models are available that the approach of performing periodic CPT tests to quantify aging effects around piles combined with model parameters derived using direct CPT correlations should be adopted in design.

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