Canadian Geotechnical Journal

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Journal:	<i>Canadian Geotechnical Journal</i>
Manuscript ID	cgj-2020-0224.R2
Manuscript Type:	Article
Date Submitted by the Author:	30-Nov-2020
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Keyword:	Large-scale model test, Offshore platform, Pile group
Is the invited manuscript for consideration in a Special Issue? :	Not applicable (regular submission)

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Large-scale model tests of a single pile and two-pile groups for an offshore platform in sand

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Abstract

Estimating pile group efficiency for open-ended steel piles in small group arrangements is a challenging task for designers. This paper reports on the large-scale experimental campaign performed for the BorWin gamma offshore converter platform, which involved single piles and two-pile group systems on a scale of 1:10. The experimental works included installation, dynamic end-of-driving tests, dynamic restrike tests, and static load tests of a single pile and a pair of two-pile groups in densely compacted, artificially prepared homogeneous sand. The CPT profiles and the blow counts confirmed that the foundation systems are comparable to each other. The experimental results of the single pile system were compared with conventional design methods. Such comparison indicated that CPT-based methods and load-transfer methods are applicable at the considered model scale. The bearing capacity prediction obtained via the CAPWAP method is conservative with respect to the static capacity. A consistent setup effect can be detected by analyzing the complete dynamic loading session. The pile group efficiency for the given foundation system was found to be less than 1.0 at both very small and very large soil strains, while it equaled 1.0 at failure.

Keywords: Large-scale model tests, offshore platform, pile group

Introduction

Pile groups are geotechnical systems typically used to support massive structures such as tall buildings, silos, power plants, and bridge piers. In the offshore sector, pile groups with a limited number of individual piles (2 to 10) arranged around a jacket leg are used to support major oil and gas platforms. The stiffness as well as the ultimate capacity of a pile group may differ from the sum of its potential individual contributions due to the interaction between adjacent piles. It is well established that such interaction may be a function of both the installation and loading phase. During the installation of a new pile close to existing piles, the stress condition of the latter can be significantly influenced, leading to important changes in their bearing behavior. This aspect was notably investigated by Chow (1995), who found that, with closed-ended piles in dense sand, the radial stress on the existing pile would increase, whereas the base capacity would take more displacement to develop due to pile uplift. Furthermore, during the loading phase, when the piles of the pile group are simultaneously loaded, the stress fields may overlap, generating substantial changes in the pile loading behavior. This can result in additional soil strains and, for instance, in the displacement of the interlocked soil block in between the piles. To cope with these issues, simplified models based on linear elasticity (Randolph and Wroth 1979), empirical and analytical methods (Randolph, 2003; Sheil et al. 2019), and advanced numerical solutions (Jardine and Potts, 1988; Clancy and Randolph 1993; Han et al. 2019) have been developed since the 1970s but find scarce application in practice where simplistic methods are often employed (Tomlinson and Woodward 2015). Experimental evidence from large-scale test setups could decrease the uncertainty associated with some of the advanced analysis methods and, in turn, increase the confidence in using appropriate analysis tools. However, large-scale tests on pile groups are rather limited and mostly focused on non-displacement concrete piles with raft (Bai et al. 2006; Dai et al. 2012). The recommended practice from the American Petroleum Institute (API 2011), the German standards for offshore structures (DIN 2017), and the standard DNV GL (2016) state that pile group effects should be assessed if the center-to-center spacing, a , is smaller than eight times the pile diameter, D . The

German standard for structures for wind turbine and platforms (DIN 2019) recommends using analytical or numerical analysis to estimate pile group effects and refers to the Recommendations on Piling (EA Pfähle) of the German Geotechnical Society (DGGT 2012) for more details on the topic.

DGGT (2012) defines a resistance-related group effect G_R as:

$$G_R = \frac{R_G}{n_G Q_m} \quad (Eq. 1)$$

where R_G is the total resistance of the pile group, Q_m is the single pile resistance at the mean settlement of the pile group, and n_G is the number of piles in the group. In the international literature, G_R is often referred to as the pile group efficiency η_G (Fleming et al. 2009). In this article, pile group efficiency and group effect are used interchangeably.

DGGT (2012) states that the clearance at which piles do not mutually influence each other during loading can be assumed to be between 6 and 8 times the pile diameter and depends on the pile embedment length, L . In addition, DGGT (2012) recommends using the design charts developed by Rudolf and Kempfert (2006) to estimate the group effect in terms of settlements and bearing capacity of bored pile groups. For pile groups involving displacement piles in non-cohesive soils, the DGGT (2012) provides only some initial guidance specifying that for the ultimate limit state a value of $G_R \geq 1.0$ can be used for a/L ratios between 0.3 and 0.7. For the serviceability limit state, $G_R \leq 1.0$ should be used for $a/L \leq 0.5$ while $G_R \geq 1.0$ can be adopted if $a/L > 0.5$.

The offshore structures which are typically erected on pile groups rather than on single piles include converter platforms for wind energy. High voltage direct current (HVDC) platforms allow transmission of the energy generated by offshore wind turbines to the shore with negligible losses. They consist of a heavy topside, which can weigh up to 20,000 t for a 1 GW transmission capacity, supported by a lattice structure, i.e., a jacket structure. The BorWin gamma converter platform was erected in the German sector of the North Sea in 2018. At the corners of the platform jacket, open-ended piles were designed in pair arrangement (two-pile groups) to distribute the large loads coming from the platform's own weight and the external environment. Figure 1 shows a picture of the

platform's substructure during the pile installation operations, where the second pile of one pile group is being driven next to a pile which has already been installed through the jacket leg.

Figure 1: Pile driving operations for the BorWin gamma HVDC platform. Installation of the second pile of one pile group.

During the design process, the question presented itself of whether pile group effects should be considered. Due to the lack of existing consolidated knowledge and clear regulations on similar pile group arrangements, the electricity transmission system operator TenneT Offshore GmbH, the owner and operator of the BorWin gamma platform, decided to fund a research project and conduct a large-scale experimental campaign including numerical analyses to address the group effect issue as part of the authority approval process. An earlier paper on this research project was published in German and presented some of the experimental results (Wisotzki et al. 2018). This contribution represents a further development of Wisotzki et al. (2018) by presenting new and old experimental data and examining these in greater detail. The main aim of this work is to evaluate the pile group effect based on the experimental data collected. Secondary topics include the interpretation of cone penetration tests before and after pile driving, the analysis of the pile driving, the applicability of bearing behavior prediction methods in artificially prepared soil, and the estimation of setup effects by means of dynamic loading tests.

Description of the experimental campaign

Within the experimental campaign, one single pile (P1) and a pair of two-pile group systems (G1 and G2) were impact-driven, dynamically tested, and statically tested. The model pile dimensions reflected the foundations of the BorWin gamma platform geometry at a scale of 1:10: diameter $D = 0.254$ m, embedded length $L = 5.5$ m, wall thickness $t = 8$ mm, and total length $L_t = 6.3$ m. For the pile group systems, the center-to-center spacing, a , was 0.67 m ($2.6 \cdot D$). The pile steel grade was S355 with an estimated center-line average roughness typical of lightly rusted industrial steel ($\approx 10 \mu\text{m}$). The experimental campaign was run at the Test Center for Support Structures of the Leibniz University Hannover in the first half of 2016. The Test Center for Support Structures includes an indoor geotechnical testing pit measuring 14 m (length) by 9 m (width) by 10 m (depth), where large-scale model tests of foundations can be performed in a fully controlled, artificially prepared testing environment. Conducting large-scale pile tests in an indoor rigid soil tank with artificially prepared soil has advantages and disadvantages. The main benefit of this physical modeling technique is complete knowledge of the ground properties. The test pit can be prepared in such a way that its properties are consistent across the area of the sample and across different samples, enabling comparability and repeatability of the test results. Even though artificially prepared soil might behave differently than naturally stratified soils, a full understanding of its composition makes the generalization of experimental evidence less uncertain. Moreover, the possibility to run extensive in-situ test campaigns at various test stages significantly reduces the spatial variability, which is often a source of uncertainty in outdoor field tests. Other advantages concern logistics and are: a) Full control over the water table; b) The possibility to run tests independently of potential adverse weather conditions, which could prevent a test from being performed or could affect the material properties; c) Being able to perform tests on a large-scale with reduced logistical effort and no need for reaction piles.

One shortcoming of the presented experimental setup may be the presence of concrete walls around the test field. If disregarded, such rigid boundaries could interfere with the soil stress and

deformation, thereby influencing the soil-structure interaction and possibly affecting the experimental results. To address this issue, the specimens were installed at a distance of approximately ten pile diameters from the concrete walls. This complies with the minimum distances between load apparatus and test piles recommended in DGGT (2012). Another source of uncertainties caused by the concrete walls surrounding the test chamber is the reflection of stress waves induced when driving and dynamically testing the piles, which may affect the pile stiffness and capacity. The complete layout of the campaign is depicted in Figure 2. Since the presence of another specimen in the vicinity of a pile being tested could also trigger an unwanted pile-pile interaction, a distance of approximately ten pile diameters was maintained between the pile systems. Tests of piles under tensile loading were carried out on the left-hand side of the soil tank within the European IRPWind project and are reported in Schmoor et al. (2018). All the research activities described in this paper, including the cone penetration test (CPT) investigations, were performed after completion of the IRPWind campaign.

Figure 2: Complete layout of the experimental campaign presented in this work. Position of the test piles with respect to CPT soundings, boundary walls, and test piles of the IRPWind test campaign described in Schmoor et al. (2018).

The CPT campaigns were outsourced to Geo-Engineering.org GmbH, which used a cone with a tip cross section of 5 cm². Cone penetration tests were carried out very close to the pile positions before and after installation (see Figure 2). The CPTs after the pile installations for the systems G1 and G2 were performed in the middle of the pile group at 0.335 m from the piles axes (1.3-D). Accordingly,

the CPT sounding for the single pile system P1, after pile driving, was located at $1.3 \cdot D$ from the pile axis. Figure 3 shows a CPT being performed between the piles of pile group G1.

Figure 3: CPT K3-P1 performed after pile installation between the piles of the two-pile group system G1.

The described layout allowed detection of the potential change in the soil profile due to pile driving. Unless stated otherwise, the geometric average over a depth interval of 25 cm is used instead of the raw data whenever CPT results are used for quantitative analysis. This technique neutralizes, to some extent, the small but inevitable variability of the soil layering, allowing more straightforward, and yet realistic, identification of the salient aspects.

The pile installation was performed with a Menck SB 120 double-acting pile hammer. The theoretical striking energy per blow of the hammer is 5.8 kJ while the striking rate is 2.5 Hz.

The three piles of the pile systems P1 and G1 were subjected to a series of dynamic tests before being statically tested, whereas the two piles of pile system G2 were only statically tested. The dynamic load tests consisted of one end-of-driving (EOD) test and three restrike (RS) test sessions, all of which were carried out by GSP mbH. The three restrike test sessions are called RS1, RS2, and RS3 respectively. For the RS tests, a device consisting of a 1 t weight falling through a vertical guide was employed to induce the dynamic loading. Each restrike session included three to five tests with

increasing falling height ranging from 0.1 m to 0.8 m. The results from the dynamic loading tests were interpreted with the CAPWAP method to estimate the load-displacement curve and ultimate bearing capacity.

22 to 34 days after driving, the piles were statically tested by applying a compressive load until a vertical pile head displacement, w , of between 60 and 70 mm was reached. The static tests were performed in a load-controlled manner, as indicated by the German recommended practice DGGT (2012). Strictly speaking, the static tests were in reality quasi-static, in the sense that the load steps were applied at the slow rate of 3 kN/min. At the end of each load step, the load was kept constant for 15 minutes. Before unloading the system, the load was kept constant for 60 minutes. The piles were loaded through a hydraulic cylinder with a maximum capacity of 1 MN connected to a robust steel loading frame.

During the static loading, each pile was individually monitored with load cells and displacement transducers decoupled from the loading system, which measured the settlement and force of each pile head.

Figure 4: Static compressive capacity test of the pile group system G1.

Figure 4 shows pile group G1 during the static load test. Table 1 summarizes the relevant activities of the experimental campaign together with the specific days on which they were carried out, and,

more importantly, with the time intervals between the various tests and the last mobilization of the bearing capacity.

Soil characterization

The test environment consisted of compacted sand layers with a thickness, after compaction, of between 25 cm and 30 cm. For each layer, the sand was distributed equally across the surface, wetted to allow better compaction potential, and compacted with directional plate compactors. According to the result of the Proctor test, the optimal water content for compaction is 9%. Figure 5 shows the testing pit upon completion. The soil used for the experiments is a uniformly graded siliceous medium sand with fine content < 1% called Rohsand 3152. Its physical properties are listed in Table 2.

Figure 5: Geotechnical sand pit of the Test Center for Support Structures upon completion of the sand preparation procedures. Dimensions: Length 14 m, width 9 m, depth 10 m.

Core samples were collected throughout the filling procedure to establish the water content and compaction state of each layer. The samples were extracted using a standard steel cylinder conventionally used for non-cohesive soils with a length of 12 cm and an internal diameter of 9.6 cm (DIN 2011). The cylindrical sections of soil were taken approximately 5 cm below the sand surface of the compacted layer. Figure 6 shows the results of the relative density D_R with the depth of the geotechnical testing pit. These were obtained by collecting and directly interpreting six core samples from each compacted layer. The core samples indicated consistent relative densities along with the depth, z , with an average D_R of 0.705 and a coefficient of variation, CV, of 2.9%. After completion of the test environment, water was added to the geotechnical testing pit through a base flooding/drainage system at a very slow rate, around 30 cm/day. Throughout all the phases of the

experimental campaign, the water table was set to 0.5 m below sand level. As already mentioned in the previous section, CPT soundings were performed before and after installation of the test piles. Figure 6 also shows the D_R prediction according to the CPT correlation proposed by Jamiolkowski et al. (2003):

$$D_R = 100 \cdot \left[0.268 \cdot \ln \left(\frac{\frac{q_c}{\sigma_{atm}}}{\sqrt{\frac{\sigma'_{v0}}{\sigma_{atm}}}}} \right) - 0.675 \right] \quad (Eq. 2)$$

where q_c is the CPT cone tip resistance, σ_{atm} is the atmospheric pressure, and σ'_{v0} is the effective lithostatic pressure. All inputs are to be entered in kPa. The CPT profile used for the correlation was that performed before the pile installation close to P1, K2-P2 (see Figure 2). The correlation returned most values within the dense sand range, lending an acceptable prediction of D_R with respect to the direct measurements. Turning now to the CPT soundings, Figure 7 shows the raw data of cone resistance, q_c , and sleeve resistance, f_s , for the pile system P1 before and after driving: K2-P2 and K3-P3, respectively.

Figure 6: Relative density of the model sand. Direct core sample measurements and CPT correlation as per Jamiolkowski et al. (2003).

Figure 7: CPT profile of P1 before and after pile installation. The distance of the sounding from the pile axis after the pile installation is $1.3 \cdot D$.

Focusing on the q_c and f_s profiles before driving (dotted lines in Figure 7), a zig-zag trend is evident throughout the depth. The maxima correspond to the top of each compacted layer, whereas the minima represent approximately the middle of each layer. Between $z = 6.5$ m and $z = 7$ m, the profiles show unexpectedly low minima (q_c of approximately 7 MPa and f_s of approximately 20 kPa) that can be presumably ascribed to an approximately 40-cm thick layer prepared and compacted in correspondence to that depth interval. This thicker, and, therefore, on average more loosely compacted sand layer, was not intentional and was only discovered once the CPT data had been analyzed. Results consistent with this finding can also be seen in Figure 6, where the only value of D_R as per Jamiolkowski et al. (2003) outside the dense sand range is that at $z = 6.625$ m (emphasized with a dashed ellipse in the figure). The reason why the direct measurements of D_R do not show any relevant reduction at the same depth is that the core samples were taken at approximately 5 cm from the layer surface, down to a depth of approximately 20 cm. While this sounding depth seems to be enough to characterize layers with thickness of between 25 cm and 30 cm, it is most probably not sufficient to provide realistic D_R values for a presumptive 40-cm thick layer. It is also worth mentioning that the looser sand layer was not localized, but rather extended across the test area and was at least four pile diameters below the pile tip. For these reasons, the looser sand layer should by

no means compromise the comparability among the test piles and, if at all, should only be marginally involved in the pile failure mechanism.

As far as the profiles after pile driving are concerned (solid lines in Figure 7), the following qualitative observations can be drawn: 1) The difference between low and high peaks within a given sand layer is reduced, especially for f_s values down to a depth of 4 m; 2) The post-installation CPT results are generally smaller than the pre-installation ones down to a depth of 4 m, remain rather similar between $z = 4$ m and $z = 5.5$ m, and tend to be larger than the pre-installation ones in correspondence to the more loosely compacted layer ($z = \text{approximately } 6.625$).

Figure 8: CPT profile of P1 before pile installation and after pile installation. The distance of the sounding from the pile axis after the pile installation is $1.3 \cdot D$ (middle of the pile group).

Post- and pre-installation CPT profiles of the pile group systems show the same pattern and Figure 8 shows those referring to G1 as an example. At $z = \text{approximately } 4$ m, the sleeve friction shows values exceeding 200 kPa. This is most probably an artifact of the measurement and will not be considered further here. As observed for the single pile system, the disparity between low and high peaks within one compacted layer reduced after the piles had been driven. In correspondence to the looser sand layer, both cone and sleeve resistances increase conspicuously after pile driving. Moreover, the aforementioned cone and sleeve resistance decreases identified for the single pile system (after driving, down to $z = \text{approximately } 4$ m) seem to be less pronounced for the pile group. One conventional method to quantify the change in CPT profile provoked by soil compaction or pile

driving is to plot the sleeve and cone resistance ratios (Massarsch et al. 2020; Spill et al. 2019). These are defined as the cone and sleeve resistance values after pile installation, q_{c1} and f_{s1} , divided by the cone and sleeve resistance values before the pile installation, q_{c0} and f_{s0} . Sleeve resistance and cone resistance ratios for all three pile systems are shown in Figure 9. As a reference, the values indicating no change in sleeve and cone resistance ($q_{c1} / q_{c0} = 1$ and $f_{s1} / f_{s0} = 1$) are plotted as a dash-dotted line.

Figure 9: Sleeve resistance and cone resistance ratios for the three pile systems P1, G1, and G2.

As already qualitatively observed, the values of the cone resistance ratio for pile group systems are generally somewhat larger than those for the single pile system. The pattern is, however, less clear for the sleeve resistance ratios, where G2 presents values less than unity down to a depth of 3 m.

In correspondence to the more loosely packed layer below the pile tip, sleeve and cone resistance ratios present values between 1.5 and 3, with the ratios for the pile group systems being notably greater. This soil region indeed had more compaction potential and, accordingly, it was markedly affected by the energy induced to the soil during the driving operations. Figures 11 and 12 present the q_c values of the three pile systems together with their coefficient of variation over the depth, before and after pile driving respectively. After pile driving, the disparity among the soil profiles reduces slightly. The average CV, calculated up to $z = L + 3 \cdot D$, is 9.9% before driving and 8.3% after driving. Even though installation of the piles seems to have influenced the pile groups more strongly, the comparability among the pile systems does not appear to be compromised, at least in terms of q_c values.

Figure 10: CPT profile for the three geotechnical systems before driving and their coefficient of variation as a function of depth.

Figure 11: CPT profile for the three geotechnical systems after pile driving and their coefficient of variation as a function of depth.

Pile installation

All the piles were installed in accordance with the following seven steps: 1) Penetration of the pile due to deadweight down to approximately 0.45 m; 2) Driving of the pile down to a penetration depth of approximately 2.3 m and temporary installation stop of approximately 30 minutes to allow for plug measurements; 3) Resuming of pile driving down to a penetration depth of approximately 3.5 m and temporary installation stop of approximately 30 minutes to allow for plug measurements; 4) Resuming of pile driving down to a penetration depth of approximately 4.5 m and temporary installation stop of approximately 30 minutes to allow for plug measurements; 5) Resuming of pile driving until reaching approximately 5 m; 6) Stop-and-go driving in order to reach the designated final installation depth ($L = 5.5$ m) as accurately as possible; 7) Measuring of the plug length at the end-of-driving. The piles which underwent dynamic tests were stopped approximately 10 cm before reaching 5.5 m, with the final centimeters of penetration being achieved in the restrike sessions. Before the static tests, all the piles reached the designated pile stick-up length of 0.8 m. Millimetric differences between the stick-up pile lengths were minimized but could not be entirely avoided. The blow count of the five installed impact-driven piles is displayed in Figure 12 along with their coefficient of variation. The five curves show very similar trends, with the CV values tending to decrease with depth. Since a few hammer blows characterize the installation in the first few

centimeters of depth, the blow counts at those shallow depths were unstable and produced a peak in CV values of approximately 35%. The aforementioned steps to measure the plug can also be consistently recognized and are characterized by an increase in installation resistance after a pause in the operation (marked with ellipses in Figure 12). This effect could be explained by considering the very high blowing frequency of the hammer, which could lead to pore pressure building up and a consequent decrease in soil strength. The pore pressure has time to equalize during the plug measurements and, when the pile driving resumes, its resistance tends to increase temporarily until the building up of pore pressure is triggered again. Similar observations were made by Klingmüller et al. (2015), where a real offshore pile installation was monitored and an evident increase in energy needed and blow count were measured when restarting the installation after a pause. As already mentioned, from approximately 5 m of penetration depth, installation continued with different stop-and-go fashion approaches, which were also challenging to monitor and eventually resulted in higher CV values (marked with dashed ellipse in Fig 12). Even though the blow count is uncertain from $z = 5$ m, it remains constant or decreases with depth. Regardless of the uncertainty related to the blow count measurements in this phase, it cannot be excluded that the presence of the less dense sand layer at approx. $z = 6.625$ could have contributed to this apparent blow count decrease. Except for the initial phase of the curve, the CV values range between 5% and 20%.

Figure 12: Blow count of all five piles installed in this experimental campaign together with their global coefficient of variation as a function of depth.

It is well known that the installation process has substantial effects on the pile bearing behavior as it influences the volume of soil displaced during driving and the consequent stress fields around the pile. Given their geometry, typical offshore open-ended pile foundations are not prone to plugging during the installation process and are therefore usually considered non-displacement piles (Jardine et al. 2005). The piles of the BorWin gamma platform are no different and have most likely been installed in an unplugged manner. The geometry of the test piles presented here is such that a plug formation can be triggered. In an attempt to reduce the plug formation as much as possible and thereby retain similarity between model test and field conditions, the tip of all model piles was modified slightly with an internal shoe. Based on the findings of Byrne and Randolph (2003), 40-cm long and 6-mm thick sleeves were welded at the internal pile tip. Detailed technical drawings of the pile shoe can be found in Wisotzki et al. (2018).

One parameter which quantifies the degree of soil plugging is the incremental filling ratio (IFR), which is defined as the ratio between the soil plug increment and the pile embedment increment over a given time interval. To have consistent IFR values, the installation process should be continuously monitored. However, this is not an easy task, especially when the pile is driven into the soil with high-frequency impact hammers. An alternative parameter reflecting the forming of plugs within open-ended piles is the plug length ratio (PLR). The PLR is the ratio between the plug length and the pile embedded length, calculated at the end-of-driving. PLR values equal to 1 identify an unplugged installation, i.e., non-displacement piles or coring penetration, whereas PLR values equal to 0 imply plugged installation, i.e., full displacement piles. The results of the plug measurements in terms of PLR as a function of the internal diameter, D_i , are plotted in Figure 13 together with: 1) The empirical PLR prediction formula of Yu and Yang (2012); 2) The empirical PLR prediction formula of Gudavalli et al. (2013); 3) The PLR results of the experimental campaign on tensile loaded piles reported in Schmoor et al. (2018). The latter was performed in the same artificially prepared soil of the experiments presented here (see Figure 2). The piles tested in Schmoor et al. (2018) had D_i from 0.263 m to 0.342 m, L from 5.7 to 6.7, and were installed without a pile shoe.

Figure 13: Plug length ratio (PLR) of this work compared with similar piles installed in the same experimental environment (Schmoor et al. 2018) and with prediction formulas from the literature.

The experimental PLR values of this work are rather consistent ($CV \approx 2\%$) and present an average of 0.86. This means that the piles penetrated in a partially plugged manner, allowing most of the soil to flow inside their cross section. Some soil was, however, displaced and flew outside the pile cross section, potentially increasing the radial stresses acting on the shaft. While this is not the ideal condition, a $PLR = 0.86$ is deemed to be rather high and not capable of compromising the resemblance to the in-situ situation. To some extent, this statement is supported by means of the CPT soundings performed after pile installation, which, down to L , do not show relevant changes to the pre-installation CPT profiles (see Figure 9).

The direct comparison with the piles from Schmoor et al. (2018) reveals that the pile shoe most likely did not have any beneficial effect on reducing the plug during the installation.

The experimental results of this work, considered together with those of Schmoor et al. (2018), follow the trend outlined by the prediction formulas by Yu and Yang (2012) and Gudavalli et al. (2013) and yet are underestimated by these. This could have been expected since the experimental database cast in the empirical estimations includes longer piles, which are clearly likely to show smaller PLR values at a given internal diameter.

In sum, the analysis of blow counts and PLRs confirms the substantial homogeneity of the installation resistance and therefore supports the hypothesis of comparability among the test piles.

Single pile and dynamic pile load tests

Figure 14 visualizes the essential plots of the experimental raw data for the pile system P1.

The typical features of piles subjected to load-controlled compressive load can be recognized: 1) The secant stiffness of the system decreases with increasing settlement; 2) Creep events appear after each load step; 3) The tangent stiffness recovers after each creep stage and is temporarily larger than the one before the creep stage; 4) When unloading the system at large soil strains, it is evident that the induced displacement is largely irreversible (indicated in Figure 14 as “Second unloading branch”). In each of the plots, the second unloading branch as well as all creep stages can be recognized. The first unloading branch features a hysteresis loop that is barely visible in Figures 14a and 14b. This is shown within the magnified view in Figure 14c for clarity.

Figure 14: Raw data of the pile system P1. a) Vertical pile head displacement against time; b) Vertical force against time; c) Load-displacement curve.

Failure criteria for piled foundations under axial load abound in the international literature. These can be based on elasticity theory, on simple geometrical frameworks, on creep analysis, or on pile geometry proportions (DGGT 2012; Al-Khazaali et al. 2019). In this work, the failure criterion is taken as the vertical force correspondent to a vertical pile head displacement equal to 10% of the pile diameter. The measured static ultimate capacity of P1 is $Q_m = 314$ kN.

The experimental results of the dynamic loading tests are now considered, and Table 3 lists the measured pile ultimate capacity from the static and dynamic tests for all five piles. The ultimate capacity from the dynamic loading tests were obtained using CAPWAP signal matching, which is an extended procedure to determine the integrity and ultimate axial capacity of a given pile from acceleration and strain measurements of a pile subjected to dynamic loading (DGGT 2012, Rausche et al. 2010). When processing the dynamic test measurements with the CAPWAP method, no attempt was made to model the soil plug. The complete CAPWAP outputs in terms of damping and quake values are given in Table 4.

The restrike capacities reported in Table 3 are those recorded with the last test of each restrike session. They featured an average falling height of the 1 t weight of 0.74 m (CV = 9.2%) and an average induced pile settlement of 15 mm (CV = 5.3%). Three main experimental observations can be drawn from the table: 1) The results from dynamic load tests, $Q_{m,EOD}$, $Q_{m,RS1}$, $Q_{m,RS2}$, and $Q_{m,RS3}$ are very consistent among the three piles (CV $\leq$ 5.6%); 2) The capacities measured with the dynamic test are lower than the capacity measured with the static load tests; 3) The restrike capacity estimations are larger than the end-of-driving capacity estimations and increase consistently according to the time between test and last mobilization of the bearing capacity.

Besides confirming the comparability between the pile systems once more, the first observation demonstrates that the piles of G1 do not seem to be influenced by their adjacent pile when singularly dynamically loaded. The second observation indicates that bearing capacity estimations by means of dynamic load test tend to be conservative in the present experimental campaign. It is remarkable that even RS3, which featured a time interval of between 25.8 and 27.5 days and had about 6 cm more embedded length through the static test, underestimated the static capacity by approximately 10%. Turning now to the third observation, it seems plausible to state that end-of-driving capacity, $Q_{m,EOD}$, is significantly lower than the restrike capacities $Q_{m,RS1}$, $Q_{m,RS2}$, and $Q_{m,RS3}$. This is most certainly due to the interference with the potential pore pressure building up as already outlined in the previous section and shown in Figure 12. Another possible concurrent cause of the extremely

small value of $Q_{m,EOD}$ could be the rather uncertain efficiency of the hammer used for the driving operations, i.e., also for the EOD test. Against a theoretical striking energy per blow of 5.8 kJ, the real energy transferred from the hammer to the pile could have been well below the nominal value. On the contrary, the rudimental but efficacious device operated for the restrike tests ensured conveyed energies of approx. 6.8 kJ. With reference to the third experimental observation from Table 3 again, it is argued that the consistent growing of measured capacity with increasing time interval is an expression of the setup effect. This is now a well-known and yet still not fully explored phenomenon, which can be briefly defined as the time-dependency of pile bearing behavior. The causes and consequences of this complex and peculiar aspect of foundation engineering are treated extensively in contributions such as Jardine et al. (2006), Lim and Lehane (2015), Anusic et al. (2019), and Carrol et al. (2020) among others. In this work, the restrike test results indicate an average increase in capacity of 6.8% after 7.0 to 7.9 days (RS1 against RS2) and of 20.0% after 25.8 to 27.5 days (RS1 against RS3). It should be stated, however, that the quantitative assessment of the setup effect is rather uncertain since, as Jardine et al. (2006) reported, restrike or static tests may partly or totally disrupt aging processes. This is particularly true for the value pertaining to RS1 against RS3, as the static test run before RS3 could have significantly and irreversibly changed the stress field around the piles.

Typical design methods included in standards and recommended practices are now compared to the experimental results of P1. The scope of the comparison includes the plausibility check of the test results as well as the applicability of the design methods in non-natural, artificially prepared soil.

The simplistic and yet well-established API Main Text method in accordance with API (2011) is perhaps the most widely used in industry for the calculation of pile compressive bearing capacity.

This method relates both pile skin friction and pile base resistance to the effective vertical stress of the soil through relative density-dependent parameters. The CPT-based methods are more sophisticated approaches to the estimation of the pile axial bearing capacity. These relate both shaft friction and base resistance to the CPT profile and their formulation takes phenomena such as

friction fatigue, dilation, and the difference between tensile and compressive skin friction into account. Recent comparative studies can be found in Yang et al. (2015), Yang et al. (2017), and Lehane et al. (2017). API (2011) includes four CPT-based methods: the simplified ICP-05, the simplified UWA-05 (also called Offshore UWA-05), the FUGRO-05, and the NGI-05. The full versions of ICP-05 and UWA-05 are described in Jardine et al. (2005) and Lehane et al. (2005) respectively. It is worth mentioning that the capacities calculated with these methods are usually achieved ten or more days after installation due to the inherent setup effect in their formulations (Jardine et al. 2015; Yang et al. 2015; Lehane et al. 2017). As stated in Table 1, the static test of P1 was carried out 22 days after driving while the last capacity mobilization occurred 14 days after the installation, by means of RS2. The static test results of P1 should thus be compatible with the CPT-based methods predictions. To calculate the capacity estimations with the CPT-based methods, the cone resistance of the CPT profile after driving, K3-P3, was averaged over a depth interval of 0.5 m and used as an input. For the Main Text method, the input relative density was the one calculated as the average of all direct measurements ($D_R = 0.705$). The experimental backbone curve is plotted in Figure 15 together with the predictions of all methods shortly presented above. The red dashed line in the figure indicates the above-defined failure criterion. The ratios of measured capacity to the calculated capacities, Q_c / Q_m , are consistent with the Zhejiang University – Imperial College London database described by Yang et al. (2017), with UWA-05 and ICP-05 delivering the best predictions among the full CPT-methods, with Simplified ICP-05 and Offshore UWA-05 returning more conservative predictions than their full counterparts and with Main Text method underestimating the experimental result.

Figure 15: Load-displacement behavior of pile P1. Experimental data against various prediction methods.

The somewhat surprisingly significant discrepancy between Offshore UWA-05 and UWA-05 can be ascribed to the role played by PLR and IFR in the formulation of the latter. In Figure 15, the CAPWAP prediction of the dynamic loading test RS2 also seems to give a reasonably good representation of the pile initial stiffness but, as already reported above, it underestimates the pile capacity measured with the static test.

The model piles were not instrumented along the shaft and the distinct contribution of shaft and base resistance could not be experimentally obtained. Nevertheless, this can be estimated by means of the prediction methods discussed above. The contribution of the shaft resistance with respect to the entire bearing capacity is 33% for the Main Text method, 44% for the Simplified ICP-05 method, 48% for the Offshore UWA-05 method, 46% for the ICP-05 method, 51% for the UWA-05 method, 36% for the Fugro-05 method, 64% for the NGI-05 method, and 75% for the CAPWAP method.

Interestingly, the Simplified ICP-05, Offshore UWA-05, ICP-05, Fugro-05, and CAPWAP methods give very similar absolute values of shaft resistances (average 173 kN, CV = 5.7%). This may indicate that, when testing the piles dynamically, the shaft resistance was entirely mobilized while the end bearing resistance could only partly be expressed.

When running structural simulations of offshore platforms, load-transfer methods (LTMs) are commonly used to describe the pile-soil interaction. Two relevant LTMs are normally used for axially loaded pile foundations in compression: the t - z curves and the Q - z curves. While the t - z curves

characterize the pile shaft response, the Q-z curves represent the pile base behavior. Scientific literature and offshore standards provide a wide range of different t-z and Q-z curves. Most of them are non-linear and each has its own set of input parameters. Among others, pile diameter, soil shear modulus, pressiometer modulus, limit shaft resistance, and limit base resistance are some of the most employed input parameters. A recent overview of the different LTM for axially loaded piles present in the literature is given by Bohn et al. (2016), where new curves are also proposed based on experimental results. In this study, the results of the simulations with LTMs for axially loaded pile foundations proposed by API (2011), Fleming (1992), and Bohn et al. (2016) are compared with the experimental results from test P1. The input parameters needed for the selected formulations are the limit base resistance, $q_{\max}$, and the limit shaft resistance, $\tau_{\max}$. These values were estimated with the CPT-based Offshore UWA-05 method, as this semi-empirical method gives the best prediction for the pile bearing capacity. For the LTM as per Fleming (1992), the Young's modulus of the soil, E , was estimated with the correlation proposed by Lunne and Christoffersen (1983) by averaging the E values over the pile length ($E = 27$ MPa). The coefficient M was set equal to 0.005. Figure 16 shows the comparison of the experimental load-displacement curve of P1 together with the LTM predictions.

Figure 16: Load-displacement behavior of pile P1. Experimental data against LTMs.

Overall, the LTMs give a fairly good approximation of the experimental curve, which was expected, since the main inputs of all three load transfer methods adopted are, in principle, the outputs of the

empirical method which best fit the experimental results, i.e., Offshore UWA-05. The initial stiffness, however, is rather poorly predicted by API (2011) and Bohn et al. (2016), whereas it is reasonably well represented by Fleming (1992).

Pile groups and pile group efficiency

The essential plots of the experimental raw data for the pile systems G1 and G2 are shown in Figure 17 and Figure 18.

Figure 17: Raw data of the pile system G1. a) Vertical pile head displacement against time; b) Vertical force against time; c) Load-displacement curve of the group piles.

Figure 18: Raw data of the pile system G2. a) Vertical pile head displacement against time; b) Vertical force against time; c) Load-displacement curve of the group piles.

Technical problems prevented the second unloading branch from being conventionally performed and, at the end of the test, both G1 and G2 were suddenly unloaded (indicated in Figure 17 and Figure 18 as “Abrupt unloading”). Test G2 lasted longer than Test G1 as a larger force was designated within the first loading phase. This difference can be easily noted in the magnified views in Figure 17c and 18c, where the unloading branch occurs at different loading steps. One technical challenge of the project was to make sure that the applied force could be equally transferred to the two piles of the pile group systems. Progression of the applied forces against time is shown in Figure 17b for the pile system G1 and in Figure 18b for the pile system G2. During most of test G1, the force applied to pile G1-2 was slightly greater than the one applied to pile G1-1. Afterwards, pile G1-1 was more loaded for the last loading steps (from approximately 32,500 s). Similarly, pile G2-1 carried more load than G2-2 until approximately 23,500 s. The maximum divergences between the forces distinctly applied to the two piles occurred at very small strains for both pile systems. During the very first load step ($V < 30$ kN, $w < 0.1$ mm), the divergence reached values in excess of 20%. Such relatively large divergences are most likely caused by the small initial adjustments between the pile heads to neutralize the millimetric, but still present, difference in the stick-up length of the piles within the

pile group systems. After this initial phase, the divergence reduces significantly and does not exceed 10%.

The ultimate capacity of all four piles involved is given in Table 3. The total ultimate capacity of the pile group adds up to 627 kN for G1 and to 639 kN for G2. The divergence in capacity is smaller than 2% and can therefore be considered within the experimental uncertainty. Nevertheless, it cannot be disregarded that the pile system G2 consisted of two fresh piles tested 34 days after their installation. Conversely, G1 was twice subjected to restrike before being statically tested 16.1 days after the last restrike session. Assuming the indication of the restrike sessions described in the previous section to be true, it could be argued that both pile systems featured a performance increase of approximately 7% due to setup within the first 7 days after the last capacity mobilization, i.e., after installation for G2 and after RS2 for G1. Afterwards, G1 had still approximately 9 days and G2 approximately 27 days before being statically tested. The 18-day time divergence between the two pile systems may have been contributed to the 2% larger capacity displayed by G2.

Unfortunately, the current experimental database does not provide enough information to prove or disprove the described scenario and further research would be required to corroborate this hypothesis. In particular, there are two main aspects of the problem that would need special consideration. The first is the fact that the setup effect could be mitigated or even exacerbated by the interaction between two adjacent piles. The second is the uncertainty related to the dynamic test when estimating the bearing capacity and, in turn, the setup effect.

Moving now to the estimation of the pile group efficiency, the load-displacement curves of all five statically tested piles are shown in Figure 19.

Figure 19: Load-displacement curves of test piles P1, G1-1, G1-2, G2-1, and G2-2.

The magnified portion of the load-displacement curve shows the initial stiffness and the unloading reloading branches in more detail. In the initial portion of the load-displacement curve ($w < 1$ mm), the single piles tend to be stiffer than the piles of the pile group systems. The piles of G2 are notably stiffer than those of G1 in this test stage, which is most likely attributable to the different loading histories of the two systems, i.e., G1 undergoing restrike sessions and G2 being fresh. Between 1 mm and 15 mm of pile head displacement, w , the piles of the pile group approach the single pile exceeding its capacity. At large strains ($w > 0.1 \cdot D$), the pile group systems appear gradually to lose the lead they showed at smaller strains. A quantitative estimation of what has been qualitatively described is given in Figure 20, where the pile group efficiency as defined in Eq. 1, is plotted as a function of the normalized pile head displacement.

Figure 20: Numerical and experimental pile group efficiency for the pile groups G1 and G2 as a function of the pile head displacement.

As already qualitatively explained above and shown in Figure 19, the experimental pile group efficiency is smaller than 1.0 at very small strains, exceeds 1.0 for most of the static loading test and tends to decrease at very large strains. It is remarkable that the trends of G1 vs. P1 and G2 vs. P1 are very similar, with G1 vs. P1 clearly showing smaller G_R values probably due to the restrike sessions performed before the static tests. The disadvantageous pile group effect found at small strains is partly to be ascribed to the initial adjustment phase described above. More importantly, however, the pile response is dominated by the shaft friction at such small strain fields, implicating perhaps that a significant contribution to the negative group effect could be due to overlapping of the concurrent stress fields caused by the shear strength mobilized on the contact interface. Within the first millimeters of settlements, the skin friction reaches its maximum value along the entire shaft and, at the same time, the base resistance becomes relevant. However, before reaching failure, the stress fields generated by the base resistance are not large enough to cause overlapping with that of the adjacent pile. This might be the reason why the G_R values exceed 1.0 in the transition between small strain and failure. Eventually, at very large strains (after pile failure), it is likely that the base resistances also begin to interact with one another, triggering a reduction in pile group efficiency again.

The diagram in Figure 20 also includes numerically estimated pile group efficiencies. The numerical analysis represented by the dashed line was performed within this research project and consists of a 3D FEM model of both single pile and two-pile group systems with the software ABAQUS. For the simulations, the Mohr-Coulomb failure criterion was used to describe the sand behavior while the penalty contact formulation was used for the interface between pile and soil. The soil properties were based on CPT measurements after the pile installation applying the correlations of Mayne (2007) for the angle of friction, ϕ' , and Lunne and Christoffersen (1983) for the Young's modulus of the soil. In addition, the soil cohesion was set equal to 1 kPa, the Poisson's ratio of the soil equal to 0.3, the angle of soil dilation $\psi = \phi' - \phi_{crit}$, assuming $\phi_{crit} = 30^\circ$, while the friction coefficient was $\tan(2/3 \phi')$. Even though the piled foundations are modeled as wished-in-place, some experimental measurements taken after driving were used to modify the soil parameters: 1) The CPT performed after pile installation were used for the input soil profiles; 2) The effect of partial plugging during driving was taken into account by increasing the radial stresses as per Gudavalli et al. (2013). These two countermeasures were taken in an attempt to replicate the influence of pile driving on the surrounding soil, but do not necessarily cover the full range of potential installation effects. Indeed, further soil state modifications due to pile driving could have taken place and gone undetected. Since the soil constitutive model and the contact definition employed in the numerical analysis are rather simple, it is not expected that the experimental data can be accurately replicated. Nevertheless, such simplified models are not uncommon in practice, and it is believed that the results of the simulations in terms of pile group efficiency can be of interest. The numerical results of this simplified FEM analysis are able to give an acceptable representation of the group efficiency, which marginally underestimates the experimental results.

The second numerical group efficiency estimation is that of Han et al. (2019), who adopted an advanced bounding-surface-plasticity constitutive model to simulate small non-displacement pile groups in sand, which had slightly different geometry ratios from the piles presented here.

Remarkably, the exact same trend retrieved experimentally in the experimental tests presented here can be observed.

Conclusions

Standard procedures and solid experimental evidence are lacking for the prediction of the bearing behavior of open-ended steel piles arranged in a group of two piles. During the design phases of the offshore converter platform BorWin gamma, the response of its foundation systems, which included single piles and two-pile groups, were investigated in large-scale physical tests. The experimental campaign was conducted at a scale of 1:10 and included a pair of two-pile group tests and one single pile test. As expected, the analysis of the CPT results before and after pile driving revealed that the cone resistance of a sounding performed in the middle of the soil portion between the two piles increases more than that performed adjacent to a single pile. The blow count and plug measurements confirmed the hypothesis of comparability among the model piles. Given their geometry, the model piles could not be installed in a fully unplugged manner. However, the measured plug length ratio was very close to the coring situation, with the result that the model piles maintained the resemblance to the real structure to a large extent. Relevant bearing capacity methods for axially loaded pile foundations were used to predict the ultimate bearing capacity of the single pile. The errors of the various methods were found to be consistent with an existing pile test database from the literature. Load-transfer methods for the bearing performance were successfully compared to the experimental curve of the single pile. The dynamic loading tests were interpreted with the CAPWAP method and showed increasing capacity at increasing time between testing (setup effect). However, the CAPWAP predictions consistently fall short with respect to the static capacity, underestimating it by 20% to 25%. The experimental pile group efficiency is not constant throughout the test but rather changes with increasing pile head displacement. At very small and at very large soil strains, the group effect amounts to values smaller than 1.0, whereas the group efficiency is larger than 1.0 for most of the load path until failure. While a simplified FEM model performed by

the authors tended to underestimate the experimental pile group efficiency, an advanced FEM model of a comparable geotechnical system from the literature seems to capture the trend and magnitude of it better.

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TABLES

Table 1: Chronology of the relevant activities within the experimental campaign of the research project. Interval in days and hours between different pile testing. The experimental campaign took place in the first half of 2016.

Activity	Pile P1	Piles G1-1 and G1-2	Piles G2-1 and G2-2
	Test P1	Test G1	Test G2
CPTs before pile driving		03/30/16	
Pile driving	04/19/16	04/20/16	04/22/16
EOD + RS1	04/19/16	04/20/16	-
RS2	04/27/16	04/27/16	-
CPTs after pile driving		04/28/16 and 04/29/16	
Static compression test	05/11/16	05/13/16	05/26/16
RS 3	06/08/16	06/08/16	-
Hours between driving and RS1 (days)	1.5 (0.06)	1.6 (0.07)	-
Hours between RS1 and RS2 (days)	190 (7.9)	169 (7.0)	-
Hours between RS2 and static test (days)	347 (14.5)	387 (16.1)	-
Hours between static test and RS3 (days)	660 (27.5)	618 (25.8)	-
Hours between installation and static test (days)	538.5 (22.4)	557.6 (23.2)	816 (34)

Table 2: Physical properties of Rohsand 3152 sand.

Property	Definition	Unit	Value
D_{10}	Grain diameter at 10% passing	mm	0.22
D_{30}	Grain diameter at 30% passing	mm	0.29
D_{50}	Grain diameter at 50% passing	mm	0.36
D_{60}	Grain diameter at 60% passing	mm	0.4
C_u	Coefficient of uniformity	-	1.82
C_c	Coefficient of curvature	-	0.96
G_s	Grain density	g/cm ³	2.65
n_{min}	Minimum porosity	-	0.31
n_{max}	Maximum porosity	-	0.46

Table 3: Capacity of the piles according to the various experimental techniques: Q_m static test, $Q_{m,EOD}$ dynamic test end-of-driving, $Q_{m,RS1}$ dynamic test first restrike, $Q_{m,RS2}$ dynamic test second restrike and $Q_{m,RS3}$ dynamic test third restrike.

Pile	Pile capacity [kN]				
	Q_m	$Q_{m,EOD}$	$Q_{m,RS1}$	$Q_{m,RS2}$	$Q_{m,RS3}$
P1	314	151	233	249	278
G1-1	315	158	234	253	281
G1-2	312	138	232	245	280
G2-1	308	-	-	-	-
G2-2	331	-	-	-	-

Table 4: CAPWAP signal matching outputs.

Pile	Test	Avg. shaft quake	Toe quake	Avg. shaft Smith damping	Toe Smith damping
		[mm]	[mm]	[s/m]	[s/m]
P1	$Q_{m,EOD}$	1.10	3.60	0.85	1.31
	$Q_{m,RS1}$	2.10	6.90	0.41	0.43
	$Q_{m,RS2}$	1.20	5.90	0.40	0.31
	$Q_{m,RS3}$	2.70	3.90	0.50	0.08
G1-1	$Q_{m,EOD}$	1.70	5.00	0.90	1.34
	$Q_{m,RS1}$	5.20	8.30	0.51	0.35
	$Q_{m,RS2}$	3.60	17.60	0.82	0.08
	$Q_{m,RS3}$	1.60	4.00	0.52	0.08
G1-2	$Q_{m,EOD}$	3.60	3.70	1.42	1.19
	$Q_{m,RS1}$	6.30	9.50	0.48	0.11
	$Q_{m,RS2}$	2.00	7.10	0.52	0.20
	$Q_{m,RS3}$	3.30	6.10	0.44	0.29

FIGURE CAPTIONS

Figure 1: Pile driving operations for the BorWin gamma HVDC platform. Installation of the second pile of one pile group.

Figure 2: Complete layout of the experimental campaign presented in this work. Position of the test piles with respect to CPT soundings, boundary walls, and test piles of the IRPWind test campaign described in Schmoor et al. (2018).

Figure 3: CPT K3-P1 performed after pile installation between the piles of the two-pile group system

G1.

Figure 4: Static compressive capacity test of the pile group system G1.

Figure 5: Geotechnical sand pit of the Test Center for Support Structures upon completion of the sand preparation procedures. Dimensions: Length 14 m, width 9 m, depth 10 m.

Figure 6: Relative density of the model sand. Direct core sample measurements and CPT correlation as per Jamiolkowski et al. (2003).

Figure 7: CPT profile of P1 before and after pile installation. The distance of the sounding from the pile axis after the pile installation is $1.3 \cdot D$.

Figure 8: CPT profile of P1 before pile installation and after pile installation. The distance of the sounding from the pile axis after the pile installation is $1.3 \cdot D$ (middle of the pile group).

Figure 9: Sleeve resistance and cone resistance ratios for the three pile systems P1, G1, and G2.

Figure 10: CPT profile for the three geotechnical systems before driving and their coefficient of variation as a function of depth.

Figure 11: CPT profile for the three geotechnical systems after pile driving and their coefficient of variation as a function of depth.

Figure 12: Blow count of all five piles installed in this experimental campaign together with their global coefficient of variation as a function of depth.

Figure 13: Plug length ratio (PLR) of this work compared with similar piles installed in the same experimental environment (Schmoor et al. 2018) and with prediction formulas from the literature.

Figure 14: Raw data of the pile system P1. a) Vertical pile head displacement against time; b) Vertical force against time; c) Load-displacement curve.

Figure 15: Load-displacement behavior of pile P1. Experimental data against various prediction methods.

Figure 16: Load-displacement behavior of pile P1. Experimental data against LTMs.

Figure 17: Raw data of the pile system G1. a) Vertical pile head displacement against time; b) Vertical force against time; c) Load-displacement curve of the group piles.

Figure 18: Raw data of the pile system G2. a) Vertical pile head displacement against time; b) Vertical force against time; c) Load-displacement curve of the group piles.

Figure 19: Load-displacement curves of test piles P1, G1-1, G1-2, G2-1, and G2-2.

Figure 20: Numerical and experimental pile group efficiency for the pile groups G1 and G2 as a function of the pile head displacement.

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