

# Experimental Study on the Group Behaviour of Displacement Piles Driven into Medium-Dense Sand

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**ABSTRACT:** Pile foundations are widely used to transfer structural loads to deeper soil layers and are applied extensively in both cohesive and non-cohesive soils. Despite their long history of use, the group behaviour of piles, especially in sand, is not yet fully understood, and most design approaches continue to rely on empirical assumptions. Previous research has predominantly addressed single displacement piles, although in onshore practice they most often operate as part of a group. For pile groups in sand, many studies often focus on estimating a group factor at failure, without providing a full characterisation of the load-settlement behaviour of both single and group piles, which is essential for understanding differences in their bearing behaviour.

The present study examines the group behaviour of driven displacement pile groups under controlled laboratory conditions. Large-scale physical tests were performed on  $3 \times 3$  pile groups installed by hammering in medium-dense sand, with pile spacings of  $3 \cdot D$  and  $6 \cdot D$ . Single-pile tests were performed in parallel under identical conditions to allow direct comparison. Selected group piles were instrumented with strain gauges to investigate the mobilisation of shaft and base resistance, while pile head forces were measured to determine load distribution within the group. Dynamic PANDA<sup>®</sup> soundings were carried out inside the groups to evaluate compaction effects resulting from installation.

**KEYWORDS:** Displacement pile group, pile-pile-interaction, group behaviour, group efficiency

## 1 INTRODUCTION

The behaviour of piles within a group differs significantly from that of a single isolated pile. This is caused by the interaction of the individual piles through the surrounding soil, generally referred to as *group action*. This interaction affects the distribution of loads as well as the deformation behaviour of the group. The resulting group efficiency, defined as the ratio between the axial resistance of the pile group and the sum of the axial resistances of the individual piles, for pile groups in cohesive soils is often smaller than unity. The extent of this effect is governed by parameters such as pile spacing, group arrangement, installation procedure, and the properties of the soil.

The first systematic investigations into pile group behaviour concentrated on pile groups in clay, e.g. Schlitt (1952) or Whitaker (1957) and led to the development of empirical efficiency factors and design recommendations. In sands, i. e. the work of Kézdi (1955) and Vesić (1969) demonstrated the influence of overlapping stress zones and of soil densification on group performance. However, experimental data on displacement pile groups remained limited.

Later studies, including those by Hanna (1963), Kishida & Meyerhof (1965) and Tejchman (1973), showed that the installation of displacement piles in non-cohesive soils can considerably alter density and stress conditions in the surrounding soil. This may lead to improved performance and, under certain conditions, to group efficiencies even above unity. Nevertheless, most available work addresses efficiency only at failure, without providing a detailed description of the complete load-settlement behaviour or the respective contributions of shaft and base resistance within the group, especially at pile spacings bigger than  $6 \cdot D$  ( $D$ : pile diameter).

At the Institute of Geotechnical Engineering at the University of Stuttgart, model tests on displacement pile groups were carried out in a calibration chamber. The investigations focused on  $3 \times 3$  pile group configurations with particular attention to the influence of pile spacing. The following section describes the experimental set-up and the testing procedure, followed by selected results for three separately conducted tests with two pile groups with pile spacings of  $3 \cdot D$  and  $6 \cdot D$  and an unaffected single pile.

## 2 TESTING FACILITIES AND TESTING PROCEDURE

Firstly, the medium-dense sand specimen was prepared using the *upright sand column method* (Walz et al., 1975). The sand was subsequently brought to the desired stress state by applying vertical stress through a concrete slab, braced to the test tank with a system of hydraulic jacks, anchor rods and preload beams, cf. Mitlmeier & Moormann (2024). For the herein presented tests, the applied vertical stress on the sand surface was  $\sigma_v = 200 \text{ kN/m}^2$ . The achieved densities and the stress conditions for the conducted tests are listed in Table 1.

Table 1. Density Index and  $K_0$ -values of the conducted experiments

Test	$I_D$	$K_0$
3·D Group	37 %	0.42
6·D Group	38 %	0.46
Single Pile	39 %	0.46

To control the homogeneity of the artificially poured soil specimen, dynamic soundings with the PANDA<sup>®</sup> probe were conducted before installation (S1 and S2, Figures 8 and 9) at a distance of  $12 \cdot D$  to the later installed pile group. The model piles, with a diameter of  $D = 40 \text{ mm}$ , were instrumented with strain gauges to distinguish between shaft and base resistance, see Mitlmeier & Moormann (2024). Installation was carried out hammering with a weight of 4.5 kg and a falling height of 0.50 m, resulting in a theoretical driving energy of 22.5 N·m, in a conventional installation order (Fig. 1) through holes through the concrete slab, until the piles were embedded 750 mm in the soil.

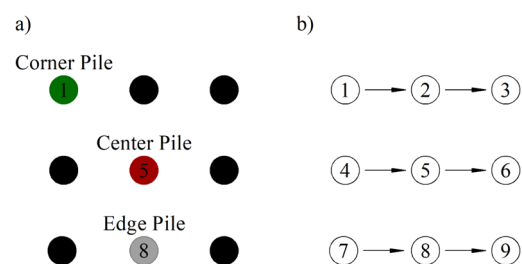


Figure 1. a) Group pile definition and b) installation order

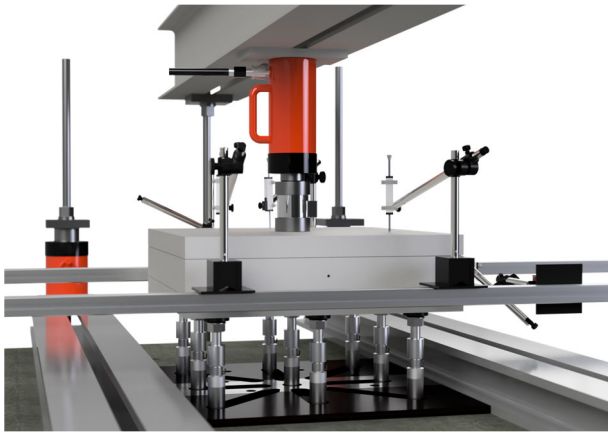


Figure 2. Rendering of the static axial load test of the 3 x 3 pile group with 6-D pile spacing

After the pile group with at least nine piles was installed, the static axial pile group test was set up (Fig. 2). Pile heads of the center, edge and corner piles were equipped with force transducers and the rigid pile cap was hingedly connected to the pile heads. Vertical displacement sensors were used to monitor the settlement of the pile cap during the axial load test and to back-calculate the individual group pile settlement. Prior to testing, all strain gauge levels were set to zero. The axial load test was performed with two load cycles as recommended for pile-testing, cf. DGGT (2013), see Fig.3. The reference single pile test had a maximum test load of  $1/9 \cdot P_p$ .

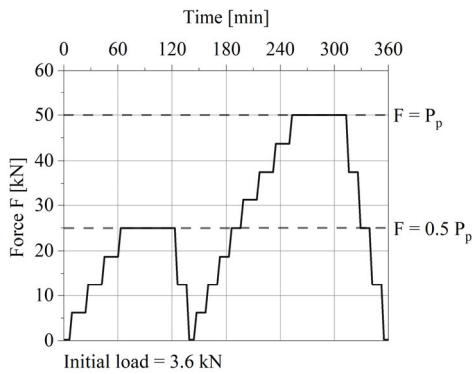


Figure 3. Loading program for the pile group tests with a test load of  $P_p = 50$  kN

### 3 EXPERIMENTAL RESULTS

#### 3.1 Installation data

During the installation of the piles the hammer blows per 5 cm of penetration were recorded, see Fig. 4 a), c) and the corresponding theoretical energy sum  $\sum W_{\text{theor.}}$ , cf. Fig. 4 b), d). The sum of blow counts for selected group piles is also listed in Table 2. As all of the performed tests have been performed in Berlin Sand under almost identical density and stress state conditions (Table 1), the blow count and the corresponding energy sum can be compared between the tests. The first installed pile in these series was the corner pile P1. It is noticeable that for the subsequent piles in the 3-D test an increase in the blow count can be asserted, resulting from densification of the medium-dense soil stratum due to group pile installation. Rather for the center and the edge pile of the 6-D test a conspicuous decrease in blow count is detectable, coming to the decision, that in this distance loosening effects of the medium-dense soil from pile installation are occurring.

Table 2. Sum of blow counts  $\sum N_5$

Test	Corner Pile (P1)	Center Pile (P5)	Edge Pile (P9)	Single Pile
3-D Group	122	128	128	
6-D Group	125	88	94	
Single Pile				127

#### 3.2 Axial load tests

To analyze the axial load tests axial resistances in different states of settlement are regarded. In this study two design states are defined. The ultimate limit state (ULS) is defined at the common known  $0.1 \cdot D$  settlement, which correlates to 4 mm in this study. The serviceability limit state (SLS) is set at a smaller vertical pile head displacement defined at  $s = 0.025 \cdot D = 1$  mm. In this context, settlement related pile group efficiencies defined as

$$\eta(s) = \frac{Q_g(s)}{n Q_s(s)}, \quad (1)$$

with  $Q_g(s)$  as the axial resistance of the pile group at mean settlement,  $n$  as the number of piles in the group, and  $Q_s(s)$  as the axial resistance of a single pile at mean settlement.

Fig. 5 a) shows the load-displacement curve for the 3-D and 6-D group and nine unaffected single piles. Under service load conditions, the 3-D group with  $\eta(s = 0.025 \cdot D) = 1.16$  already shows a positive group effect, the 6-D group shows even a stiffer load-settlement behaviour with  $\eta(s = 0.025 \cdot D) = 1.23$ . At a settlement  $s = 0.1 \cdot D$  in ULS the 3-D group efficiency is 1.18, and also the 6-D group efficiency with 1.24 demonstrates a higher mobilised resistance in this limit state in comparison to the same amount of unaffected single piles, see also Table 3. These results are significant because they contradict the common assumption in geotechnical engineering that the load-displacement behaviour of displacement piles in a 6-D group is identical to that of a single pile. Although not the main focus of his study, Vesić (1969) had already observed similar behaviour in his tests on four-pile groups.

Table 3. Pile group efficiencies  $\eta$  at  $s = 0.025 \cdot D$  (SLS) and  $s = 0.1 \cdot D$  (ULS)

Test	$\eta(s=0.025 \cdot D)$	$\eta(s=0.1 \cdot D)$
3-D Group	1.16	1.18
6-D Group	1.23	1.24

Regarding their axial behaviour, the 3-D and the 6-D group piles behave different, cf. Fig. 5 b) and 5 c). In the 3D- group, the center pile mobilises especially in the ultimate limit state the highest resistance, followed by the edge and the corner pile, which load-settlement behavior is almost alike the single piles' one. When the piles are spaced in 3-D distance, the installation order and the pile position inside the group seem to have an influence on the resistance mobilisation of the group piles. The center pile in 3 x 3 displacement pile groups with 3-D spacing mobilises independent from the installation order always the highest resistance in the ultimate limit state, see i. e. Vesić (1969). In this experiment the edge pile was driven later with a higher amount of energy and mobilises a higher resistance than the first driven corner pile. An experiment where the corner piles were driven in the end, the load-settlement behaviour was stiffer than that of the earlier driven edge piles, cf. Mitlmeier & Moormann (2025).

However, the load-settlement curves of the 6-D group piles are comparable; the installation order and the position inside the group do not seem to have a decisive influence on their

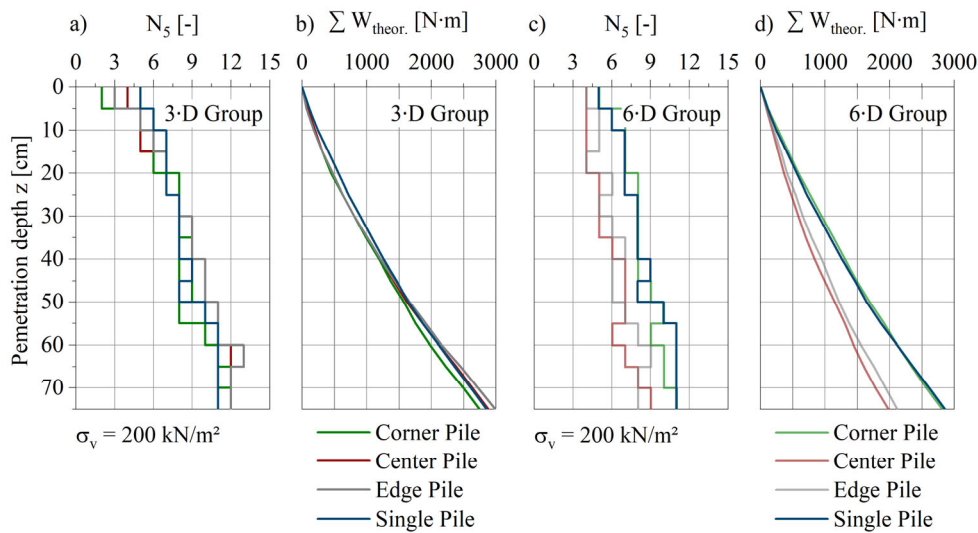


Figure 4. a) Number of blows  $N_5$  and b) energy sum  $\sum W_{theor.}$  for group piles of the 3-D pile group. c) Number of blows  $N_5$  and d) energy sum  $\sum W_{theor.}$  for group piles of the 6-D pile group

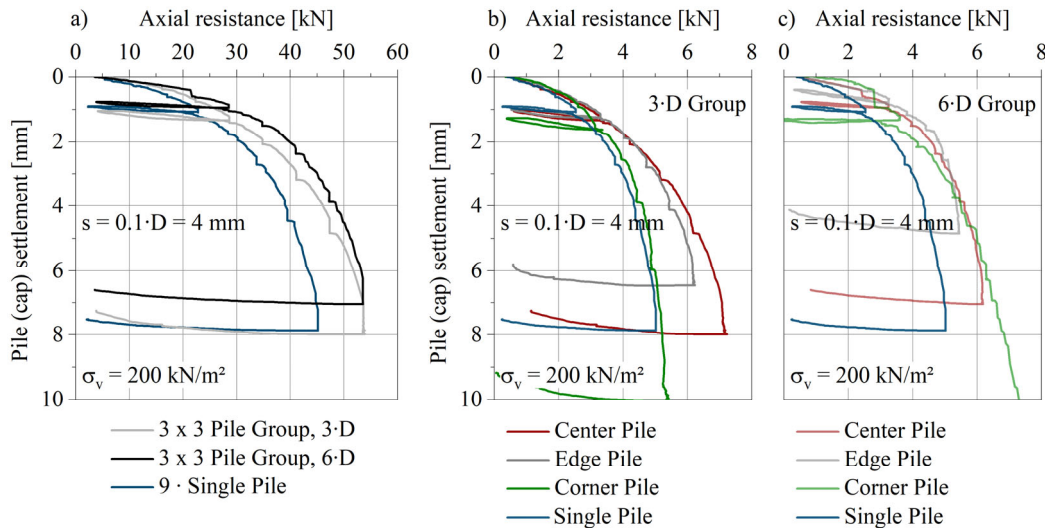


Figure 5. a) 3-D and 6-D pile group resistance compared to the resistance of an equivalent number of unaffected single piles, b) Group pile resistances at a spacing of 3-D versus single pile resistance, c) Group pile resistances at a spacing of 6-D versus single-pile resistance

mobilised resistances. Ekström (1989) also stated that, in his tests on 9-pile groups in sand with a pile spacing of  $6.5 \cdot B$ , the influence of installation order especially on the load distribution decreases distinctly.

Figure 6 a) shows that the center piles of both groups indicate an improved load-displacement response compared to the single pile. Figures 6 b) and 6 c) illustrate the distribution of shaft and base resistance and their development with increasing settlement. The results reveal that the shaft resistance of both center piles, independent of pile spacing, is at least 1.45 times higher than that of the single pile in both, the serviceability limit state (SLS) and the ultimate limit state (ULS), see Table 4. For base resistance, the single pile and the center pile in the 6-D group show similar behaviour, whereas the center pile in the 3-D group exhibits increasing base resistance with settlement, reaching  $\eta_{base}(s=0.1 \cdot D) = 1.28$  in the ULS. This behaviour has also been observed in comparable group tests with a pile spacing of 3-D in medium-dense sand (Mitlmeier & Moormann, 2025).

Furthermore, the results of the edge piles of both groups are shown in Figure 7 a). Figure 7 b) shows the development of the shaft resistance, and Figure 7 c) that of the base resistance.

Table 4. Center pile efficiencies  $\eta$  at  $s = 0.025 \cdot D$  (SLS) and  $s = 0.1 \cdot D$  (ULS)

Efficiency $\eta$ (s)		3-D Center Pile	6-D Center Pile
$\eta$	( $s=0.025 \cdot D$ )	1.18	1.28
$\eta_{shaft}$	( $s=0.025 \cdot D$ )	1.53	1.67
$\eta_{base}$	( $s=0.025 \cdot D$ )	0.87	0.94
$\eta$	( $s=0.1 \cdot D$ )	1.36	1.29
$\eta_{shaft}$	( $s=0.1 \cdot D$ )	1.45	1.54
$\eta_{base}$	( $s=0.1 \cdot D$ )	1.28	1.09

Table 5. Edge pile efficiencies  $\eta$  at  $s = 0.025 \cdot D$  (SLS) and  $s = 0.1 \cdot D$  (ULS)

Efficiency $\eta$ (s)		3-D Edge Pile	6-D Edge Pile
$\eta$	( $s=0.025 \cdot D$ )	1.24	1.46
$\eta_{shaft}$	( $s=0.025 \cdot D$ )	1.32	1.68
$\eta_{base}$	( $s=0.025 \cdot D$ )	1.16	1.28
$\eta$	( $s=0.1 \cdot D$ )	1.25	1.24
$\eta_{shaft}$	( $s=0.1 \cdot D$ )	1.10	1.33
$\eta_{base}$	( $s=0.1 \cdot D$ )	1.37	1.16

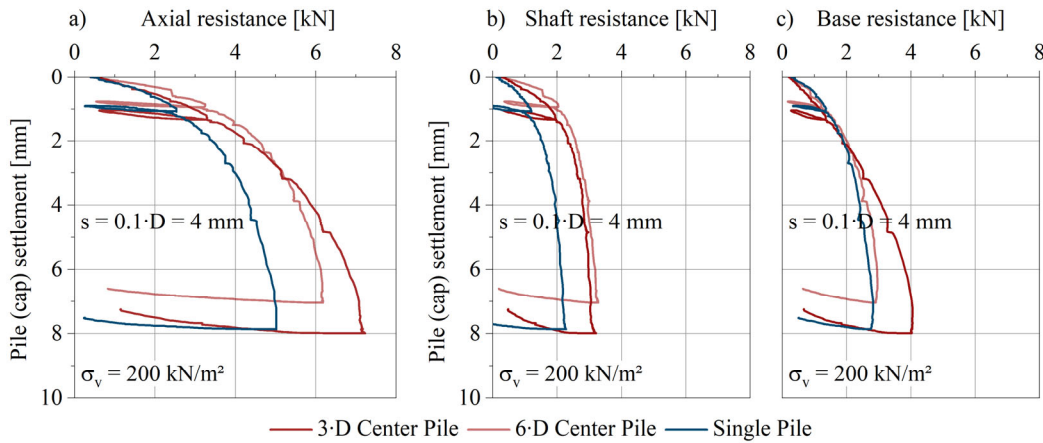


Figure 3. Center pile resistances for the pile groups spaced at 3·D and 6·D compared with single pile resistance: a) Total resistance, b) Shaft resistance, c) Base resistance.

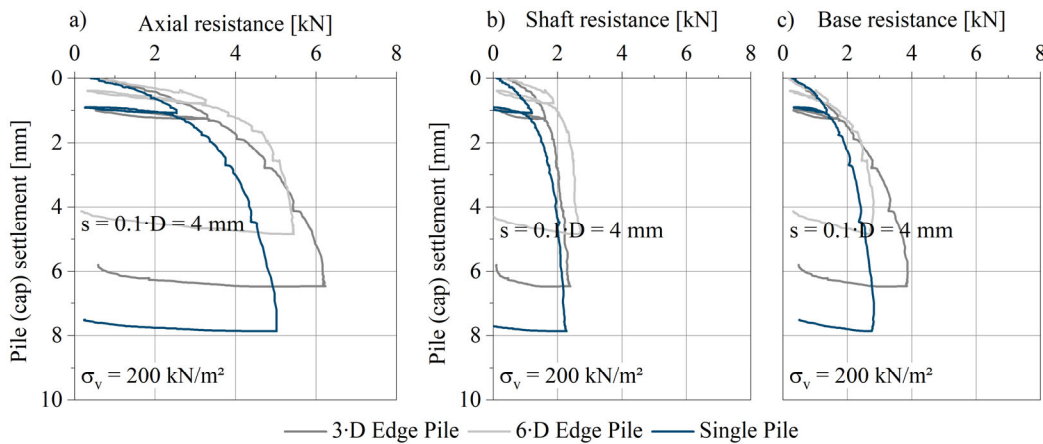


Figure 7. Edge pile resistances for the pile groups spaced at 3·D and 6·D compared with single pile resistance: a) Total resistance, b) Shaft resistance, c) Base resistance.

For the edge piles, it is also observed that the resistances in both the SLS and ULS are higher compared with the single pile; in the ULS the efficiency  $\eta$  ( $s = 0.1 \cdot D$ ) is at least 1.24 (see Table 5). The development of shaft resistance differs, however: the 6·D edge pile shows a stiffer load-displacement response and a higher maximum mobilisable shaft resistance than the 3·D edge pile. In terms of base resistance, a behaviour similar to that of the 3·D center pile is observed: the 3·D edge pile exhibits a significant increase in base resistance with settlement, which in the ULS exceeds that of the 6·D edge pile.

Based on the present test results, two effects can be identified: in the group tests, both the center and edge piles mobilise higher shaft resistances at 3·D and 6·D pile spacing than the single pile, in both the serviceability limit state (SLS) and the ultimate limit state (ULS). Although only minor load increases are observed for settlements greater than 4 mm, the limiting shaft friction appears to be nearly reached for the center, edge, and single piles in the ULS.

In contrast, the base resistance of the center and edge piles in the 3·D group shows a continuous increase, which continues even at a settlement of 4 mm. This additional mobilisation is not observed to the same extent in the single pile or in the 6·D group, suggesting an additional arching effect in the 3·D group. This finding still requires verification through further tests.

### 3.3 Dynamic Soundings

Compaction effects within the installed pile group were examined by means of dynamic cone penetration tests using the PANDA<sup>®</sup> probe in predefined interaction zones. These zones

characterise areas inside the pile group where the soil is expected to be affected by varying degrees of densification depending on the proximity to adjacent piles. The zones were defined according to the number of surrounding piles and their spacing, with a maximum distance of 3·D. Figure 8 shows the profiles of dynamic cone resistance  $q_d$  with depth obtained from the soundings in the 3·D pile group, while Figure 9 presents the corresponding profiles for the 6·D pile group.

For a quantitative analysis of the soundings and the changes in the soil stratum, the sounding profiles are integrated as follows:

$$Q_d = \int_{0.10 \text{ m}}^{1.0 \text{ m}} q_d dz \left[ \frac{\text{MN} \cdot \text{m}}{\text{m}^2} \right]. \quad (2)$$

Soundings S1 and S2, performed prior to pile installation at a distance of 12·D from both pile groups, provide the initial reference values of the dynamic cone resistance. Soundings S3 and S4, carried out at the same distance after the pile group test, show no significant change compared with S1 and S2, see also Table 6. In contrast, soundings conducted inside the 3·D pile group after testing (S5 to S7) clearly demonstrate that the medium-dense sand has been compacted. Beneath the pile tips, at depths between 0.8 m and 1.0 m, the dynamic cone resistance returns to the initial values observed in S1 to S4.

In contrast, no comparable compaction effects were observed in the 6·D pile group (see Figure 9 and Table 6), which can be attributed to the larger pile spacing. The soundings conducted after testing show dynamic cone resistance values that suggest a loosening of the sand compared with the initial

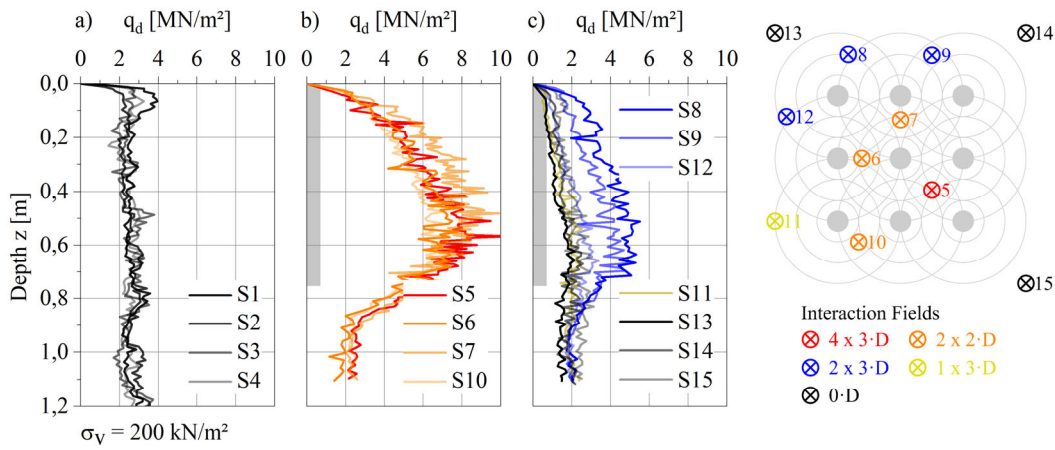


Figure 8. Soundings with the Dynamic Cone Penetrometer PANDA® in the area of the 3-D pile group. Initial soil density was indirectly determined at a distance 12·D from the group (S1 and S2 before installation; S3 and S4 after the pile group test). Additional soundings within the 3 x 3 pile group (S5-S15, all after the pile group test) were performed to quantify compaction and stress-increase effects.

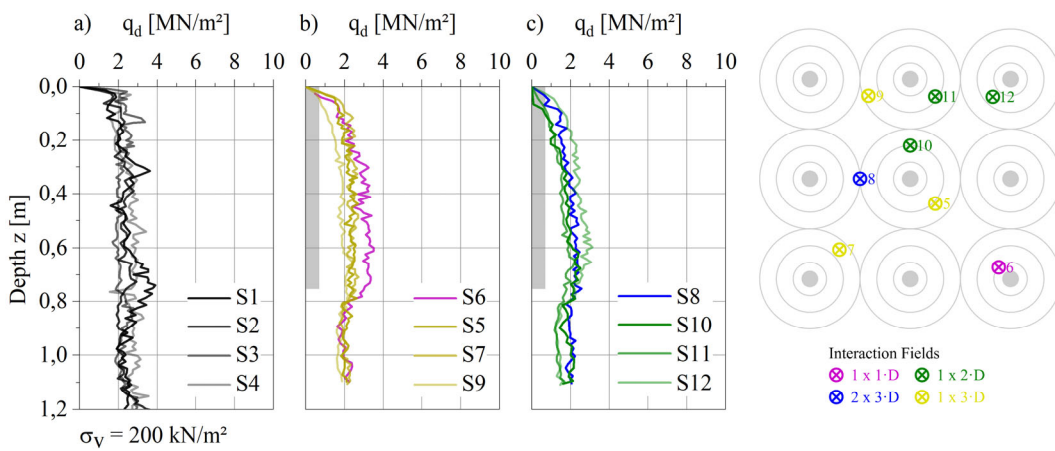


Figure 9. Soundings with the Dynamic Cone Penetrometer PANDA® in the area of the 6-D pile group. Initial soil density was indirectly determined at a distance 12·D from the group (S1 and S2 before installation; S3 and S4 after the pile group test). Additional soundings within the 3 x 3 pile group (S5-S12, all after the pile group test) were performed to quantify compaction and stress-increase effects.

soundings, indicating that additional soil densification due to pile installation did not occur in this case.

Table 6. Dynamic sounding integral  $Q_d$  from soundings within the 3·D and the 6·D pile group

Sounding	3·D Group	6·D Group
S1	2.36	2.33
S2	2.33	2.03
S3	2.28	1.98
S4	2.31	2.33
S5	5.12	2.02
S6	4.61	2.40
S7	5.47	2.04
S8	3.30	1.82
S9	2.76	1.64
S10	4.35	1.59
S11	1.50	1.37
S12	1.90	1.99
S13	1.22	-
S14	1.54	-
S15	1.92	-

Placing the sounding results in the context of the pile group load tests leads to the conclusion that the load-settlement behaviour of the 3-D group benefits significantly from compaction effects induced by pile installation through impact driving. In contrast, no substantial compaction effects between the group piles are detectable in the 6-D group. Consequently, the favourable load-displacement behaviour observed for the 6-D group, compared with an equal number of unaffected single piles, must be attributed entirely to group action. This conclusion, however, requires further experimental verification and validation.

#### 4 CONCLUSIONS

This paper presents the results of 3 x 3 displacement pile group tests in medium-dense sand with pile spacings of 3·D and 6·D. Both groups exhibit efficiencies greater than unity in serviceability limit state (SLS) and ultimate limit state (ULS), confirming a positive load-displacement behaviour compared with the same number of unaffected single piles. For the 3·D group, an efficiency of 1.18 was estimated, while for the 6·D group an efficiency of  $\eta = 1.24$  was obtained at a mean settlement of 0.1·D. The results for the 6·D group are particularly significant, as they contradict the common assumption in geotechnical engineering that the load-displacement behaviour of displacement piles in a 6·D group is identical to that of a single pile. Soundings conducted within

the 3-D group clearly reveal densification effects resulting from pile installation. In contrast, no densification effects were detected in the 6-D group, suggesting that the positive load-settlement behaviour in this case is explained largely by group action. All tests and assumptions presented herein require repetition and confirmation through further experiments, addressing additional key aspects such as pile spacing, initial soil density and the influence of the installation method.

## 5 ACKNOWLEDGEMENTS

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