Plugging Effect of Open-Ended Displacement Piles

Prise en compte de l'effet de bouchon pour les pieux battus ouverts

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ABSTRACT: During jacking an open-ended displacement pile the soil is entering through the pile toe into the profile. This plug can close up the pile toe completely. Because of this the pile can be treated approximately as a fully closed-ended displacement pile and is able to mobilize an additional base resistance. Indeed the soil-mechanical processes and the different factors of influence on the plugging effect are mostly unknown. This report is based on research work and investigated the influence of different factors on the plugging effect and hence the change in the load-bearing behaviour mainly in non-cohesive soils using experimental, numerical and statistical methods. All investigations show that a fully plugged soil inside the pile could not be identified and disproved the classical model representation of a fully plugged pile toe. The load transfer in the plug takes place by compression arches, which are mainly influenced by the pile diameter and the soil density. Finally, based on these results a practical calculation method is suggested.

RÉSUMÉ : Lors de la mise en place d'un pieu battu ouvert, le terrain est susceptible de pénétrer dans le pieu par son pied de manière plus ou moins importante. Suivant le degré de pénétration du sol dans le pieu, celui-ci peut être considéré comme ouvert ou fermé et une résistance supplémentaire peut alors être mobilisée. Ce papier propose une étude des processus de pénétration du terrain dans les pieux battus ouverts pour des sols non cohésifs. La variation de capacité portante des pieux induite par ces processus est analysée selon des points de vue expérimentaux, numériques et statistiques. Toutes les investigations réalisées montrent que l'effet de bouchon complet n'existe pas et qu'un pieu battu ouvert ne peut pas être considéré comme véritablement fermé. L'effet de bouchon correspond à la formation de « voûtes » à l'intérieur du pieu. Enfin, une méthode de prévision de la capacité portante intégrant ces processus est proposée.

KEYWORDS: open-ended displacement pile, plugging effect, pile bearing capacity, pile foundation.

1 INTRODUCTION

Open-ended displacement piles are piles, which are open at the pile toe like pipe piles, H-profiles or composed of sheeting piles. During the piling process (jacking, impact driving, vibrating or pressing) the soil is entering into the pile tube. Between the opposite inner shaft areas a plug can occur, which is able to mobilize an additional toe resistance. This toe resistance depends on the soil parameters, the pile geometry and the stress distribution.

Open-ended displacement piles are often used in harbour constructions or as foundations for offshore wind plants (i.e. monopiles or jackets).

Technical standards like API or others assume a fully plugged open-ended displacement pile and treat this plug in a monolithic way. However the soil-mechanical process and the different factors of influence on the plugging effect are mostly unknown.

Starting with a short state of the art this paper summarizes laboratory tests, numerical and statistical calculations and recommends new experience values for the bearing capacity of open-ended displacement piles.

These research results are based on the works described in Lüking 2010 and also Lüking and Kempfert 2012.

2 STATE OF THE ART

The bearing capacity of the plug can be evaluated by the values IFR (Incremental Filling Ratio) after Brucy et al. 1991 or the PLR (Plug Length Ratio) after Paik and Lee 1993, see Eq.1 and Eq.2.

$$IFR = \Delta h_p / \Delta d_e \quad (1)$$

$$PLR = h_p / d_e \quad (2)$$

These values describe the incremental and the absolute ratio of the height of the plug h_p to the pile embedded depth d_e.

An IFR = 1 means that the surface of the plug does not penetrate into the soil during driving in comparison to the last measurement. Only the pile penetrates into the soil. This means that no plugging effect takes place.

In contrast an IFR = 0 means that the surface of the plug penetrates into the soil with the same value as the pile. In this case the pile is fully plugged and all the soil has to be displaced sideways.

The IFR will be measured during driving by a sounding line. The PLR is only measured after finishing the driving and gives only an average value for the plug development. This is problematic in layered soils.

The highest radial displacement u_R and radial stresses σ'_R occur by an IFR = 0. In this case the soil is fully plugged which means that the soil resistance is the same like the toe resistance of the profile. Then the plug could be treated like a monolith and is comparable to a closed-ended pile. With an increasing IFR the radial displacement and the radial stresses are decreasing. If the IFR lies between 0 and 1 the soil is partially plugged. The changeover from a full plug to a partial plug and no plugging is steady and the statuses cannot easily be distinguished. Figure 1 gives an overview of the described context after White et al. 2005.

The maximum pile diameter in which a plugging effect could occur is about 1.5 m, see Jardine et al. 2005.

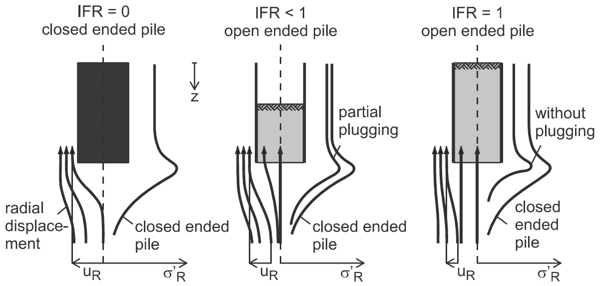

Figure 1. Distribution of the radial displacement u_R and the radial stress σ'_R on the pile shaft depending on different IFR after White et al. 2005

3 EXPERIMENTAL INVESTIGATIONS

3.1 *General*

Different experimental investigations were carried out. The next section gives a short overview of the laboratory test program before the results are discussed. A detailed description and a documentation of all test results is given in Lüking 2010.

3.2 *Model Tests and Particle Image Velocimetry Tests*

In the first test series a test pile of two pipe piles was constructed. Both piles were only connected at the top. In all the test pile had the following geometry: outer pile diameter 19 cm and inner pile diameter 16 cm. The pile embedded depth after driving the test pile into a sand box was about 140 cm. After this a static pile test loading was carried out.

This test pile was equipped with different strain gauges. Based on the measured strain ε the inner shaft friction q_{is}, the outer shaft friction q_s and the pile toe pressure q_b could be calculated. By means of a special constructed cone-penetration-test (lab-CPT) the change in density and the displacement effect of the pile installation could be examined.

In the second test series Particle Image Velocimetry (PIV) tests were carried out. The PIV method is a contact free measurement, in which displacement vectors can be identified. Basics to this method can be found in Raffel et al. 2007.

The test pile in the second test series had an outer pile diameter of 60 mm and a wall thickness of 2 mm. It was driven behind an acrylic glass to an embedded depth of 50 cm. Figure 2 gives a perspective view of both test series which were mainly carried out in non-cohesive soils.

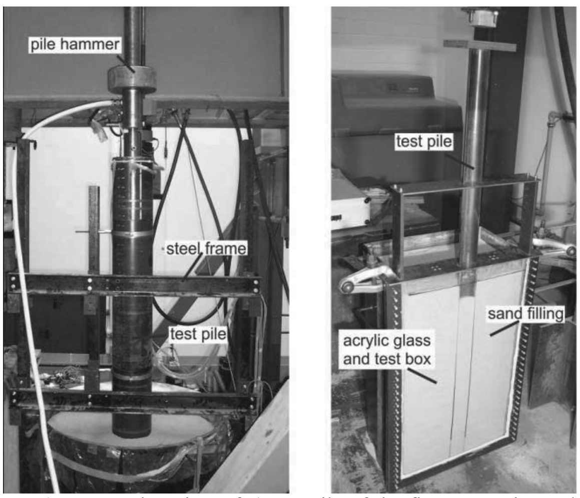

Figure 2. Perspective view of a) test pile of the first test series and b) test pile of the second test series (PIV)

3.3 *Results of the experimental test series*

In general the experience on the pile bearing behaviour regarding different influence factors could be confirmed. With an increasing relative density and increasing stress level the pile bearing capacity is also increasing.

The change in density around the test pile was lower in dense sands than in loose sands, which could be identified by different tests with the lab-CPT. The base resistance of the lab-CPT inside the soil plug was up to 80 MPa. A higher density of the soil tends to a higher IFR. Nevertheless the IFR does not converge to a fixed value. It was increasing and also decreasing during driving which means that the soil inside was plugging and loosening again. This phenomenon was also identified during the static pile test loading. However during both test series the value never reached IFR = 0. The minimum was IFR = 0.2. This means that only a partially plugged soil could occur and based on this the concept of a monolithical soil plug should be analyzed critically.

Figure 3 shows the distribution of the inner and outer shaft friction at different load levels from the first test series. The outer shaft friction is increasing with higher pile length as expected. In contrary the inner shaft friction is very high on a length which approximate two pile diameters. Above this the inner shaft friction in section 1 and 2 is very low and it looks approximately independent of the load level. The increasing of the inner shaft friction in section 3 is an indication for a (partial) plugging effect of the soil.

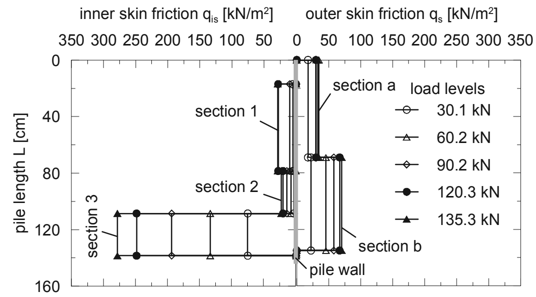

Figure 3. Distribution of the inner and outer shaft friction q_{is} and q_s for different load levels in non-cohesive soils.

Figure 4 shows the vertical displacement of the soil on the lowest two pile diameters exemplary for the second test series.

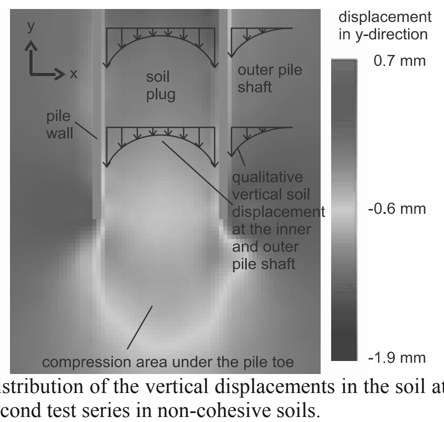

Figure 4. Distribution of the vertical displacements in the soil at the pile toe in the second test series in non-cohesive soils.

There an inhomogeneous distribution could be identified. Near the inner pile shaft the vertical displacement is much higher than in the middle of the soil plug. This distribution occurs during driving independently of all investigated boundary conditions in the second test series. It is another indication that the load transfer takes place by the inner shaft friction and not by an additional base resistance underneath the soil plug. This assumption can also be supported by the comparable distribution of the inner and outer shaft frictions, see also in Figure 3. For a monolithic approach the vertical displacement had to be more constant which could not be observed. Furthermore the tests show that these results in non-cohesive soils cannot be transferred easily to cohesive soils. It looks like that two different mechanisms are active which are not comparable.

4 NUMERICAL INVESTIGATIONS

4.1 *General*

The experimental works were further investigated by finite element calculations. The experimental test loadings as well as test loadings with bigger pile diameter were recalculated. The numerical calculation software PLAXIS 2D - Version 9.0 was used. A rotation-symmetric, 2-dimensional FE-model was built. The simulation of the soil displacement during jacking was considered by the method of Dijkstra et al. 2006. A detailed calculation description and the verification of the numerical model is given in Lüking 2010.

Finally the numerical results confirmed the results of the experimental tests quantitatively and qualitatively.

4.2 *Results of the numerical calculations*

Figure 5 shows the distribution of the inner skin friction for different inner pile diameters (D_i = 0.45 m up to D_i = 3.95 m) of a pile which is embedded in the soil of about d_e=10 m. The soil is non-cohesive and has a "dense" relative density (cone penetration resistance of about $q_c \approx 20$ MPa). The settlement for the mobilization of the skin friction was about s = 4.2 cm.

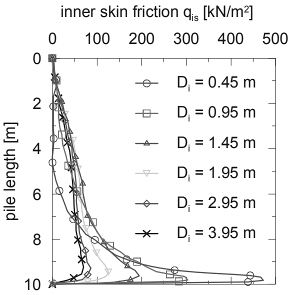

Figure 5. Distribution of the inner skin friction q_{is} under variation of the inner pile diameter D_i at a pile embedded length of d_e = 10 m and a pile settlement of about s = 4.2 cm

At low pile diameter (D_i = 0.45 m) the results show a good agreement in the distribution to the experimental works, compare Figure 3 with Figure 5.

Furthermore the results show that the inner skin friction for lower pile diameters is significantly higher at a length of approximately two pile diameters. On the upper part of the pile length no skin friction was mobilized. With an increasing pile diameter the peak value of the skin friction is reduced and is transferred to the upper part of the pile. At pile diameters of about 3 m or 4 m the distribution of the inner skin friction is comparable to the outer skin friction. The changeover from a raised inner skin friction to a more constant inner skin friction is continuous. Calculations show that this changeover depends mainly on the pile diameter and the relative density of the soil, see Lüking 2010. The distribution of the inner skin friction is also valid at "loose" relative density ($q_c \approx 10$ MPa).

Figure 6 shows the numerical results for the orientation of the stress trajectories and for the load transfer depending on the pile diameter in a "dense" relative density of a non-cohesive soil. The left part of each pile shows the derived load transfer based on the stress trajectories which are shown on the right part. In general all results show a rotation of the stress trajectories near the pile toe and also at the pile wall. With increasing distance from the pile wall to the middle of the soil plug the rotation is reducing. Also this depends mainly on the pile diameter.

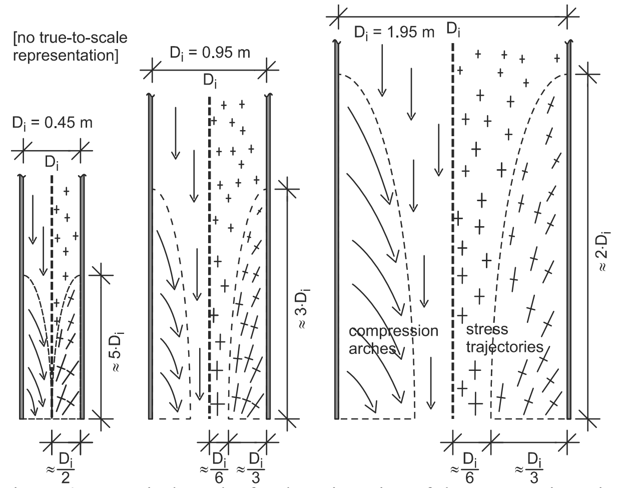

Figure 6. Numerical results for the orientation of the stress trajectories (right part of each pile) and the derived load transfer (left part of each pile) for different pile diameters in a "dense" relative density of a non-cohesive soil

The orientation of the stress trajectories suggests a compression arch, which is in analogy to the load transfer mechanism of the outer skin friction, see Kempfert 2009.

At low pile diameters these compression arches can be overlapped and results in another support. Because of this the inner skin friction can increase significantly which is also shown in the numerical and experimental results, compare Figure 5 and Figure 3. With increasing pile diameter the height of the compression arches is also increasing. This load transfer could also be identified in "loose" relative density.

Finally the results suggest that the load transfer takes place over an inner skin friction which is based on compression arches inside the soil. No fully plugged soil inside an open-ended displacement pile could be identified which would legitimate to treat the plug in a monolithic way.

5 CALCULATION METHODS

5.1 *General*

Based on the new knowledge two feasable methods for calculating the bearing capacity of open-ended displacement piles are suggested. The values were verified statistically to a large extend with calculation method 1 up to a pile diameter of D = 1.6 m in cohesive and non cohesive soils and with calculation method 2 up to a pile diameter of D = 1.2 m in non-cohesive soils. All histograms of the statistical verifications can be found in Lüking 2010.

5.2 *Calculation Method 1*

Calculation method 1 is based on an analysis of 28 static and 59 dynamic pile loading tests with pile diameters up to D = 1.6 m. This method derived new adaptation factors which are linked to the values of experience of the EA-Pfähle 2012. The basic equation for calculating the pile resistance is given in Eq.3.

$$R_k = \eta_b \cdot q_{b,k} \cdot A_b + \eta_s \cdot q_{s,k} \cdot A_s \qquad (3)$$

R_k: characteristic pile resistance
η_b: adaptation factor for the pile toe, see Eq.4
$q_{b,k}$: characteristic pile toe pressure after EA-Pfähle 2012
A_b: pile base area (contact area of the pile and the bottom area of the soil plug)
η_s: adaptation factor for the pile skin, see Eq.5
$q_{s,k}$: characteristic pile skin friction after EA-Pfähle 2012
A_s: outer shaft area of the pile

The best compliance for the adaption factors was found by a hyperbolic correlation, see Eq.4 and 5.

$$\eta_b = 0.95 \cdot e^{-1.2 \cdot D_a} \quad (4)$$

$$\eta_s = 1.1 \cdot e^{-0.63 \cdot D_a} \quad (5)$$

D_a: outer pile diameter

5.3 *Calculation Method 2*

Calculation method 2 is based on an analysis of 28 static pile loading tests with pile diameters up to D = 1.2 m. In contrary to calculation method 1 this method derived new values of experience for each part of pile resistance for the 10 % and the 50 % quantile. Eq.6 gives the basic equation.

$$R_k = q_{is,k} \cdot A_{is} + q_{a,k} \cdot A_a + q_{s,k} \cdot A_s \quad (6)$$

R_k: characteristic pile resistance
$q_{is,k}$: characteristic inner pile skin friction after Table 1
$q_{a,k}$: characteristic pile toe pressure of the pile contact area after Table 2
$q_{s,k}$: characteristic outer pile skin friction after Table 3
A_{is}: inner shaft area of the pile
A_a: contact area of the pile
A_s: outer shaft area of the pile

This method is valid for pile diameters from 0.3 m up to 1.2 m only in non-cohesive soils. The first values of the experiences in the following tables are the 10 % quantile and the second are the 50 % quantile.

Table 1. Values of experience for the characteristic inner shaft friction $q_{is,k}$ depending on the pile settlement and the resistance of the CPT

Settlement s	*Characteristic inner shaft friction $q_{is,k}$ [kN/m²] at a cone penetration resistance q_c [MN/m²]*		
	7.5	15	≥ 25
s = 0.035•D_a	15/κ ÷ 35/κ	35/κ ÷ 55/κ	50/κ ÷ 67.5/κ
s = 0.1•D_a	30/κ ÷ 50/κ	60/κ ÷ 80/κ	90/κ ÷ 100/κ
with κ = 2•PLR, see Eq. 2			

Table 2. Values of experience for the characteristic pile toe pressure $q_{a,k}$ depending on the pile settlement and the resistance of the CPT

Settlement s	*Characteristic pile toe pressure $q_{a,k}$ [kN/m²] at a cone penetration resistance q_c [MN/m²]*		
	7.5	15	≥ 25
s = 0.035•D_a	650 ÷ 1.200	1.300 ÷ 1.750	1.750 ÷ 2.800
s = 0.1•D_a	1.100 ÷ 2.000	2.000 ÷ 3.000	2.800 ÷ 4.800

Table 3. Values of experience for the characteristic outer shaft friction $q_{s,k}$ depending on the pile settlement and the resistance of the CPT

Settlement s	*Characteristic outer shaft friction $q_{s,k}$ [kN/m²] at a cone penetration resistance q_c [MN/m²]*		
	7.5	15	≥ 25
s_{g*}	15 ÷ 25	30 ÷ 50	50 ÷ 70
s = 0.1•D_a	20 ÷ 30	35 ÷ 60	55 ÷ 75
with s_{g*} [cm] = 0.5•$R_{s,k}$ [MN] ≤ 1 [cm]			

5.4 *Comparable Calculations*

Figure 7 gives an overview of the caclulation results of both methods compared with the results of the pile load tests.

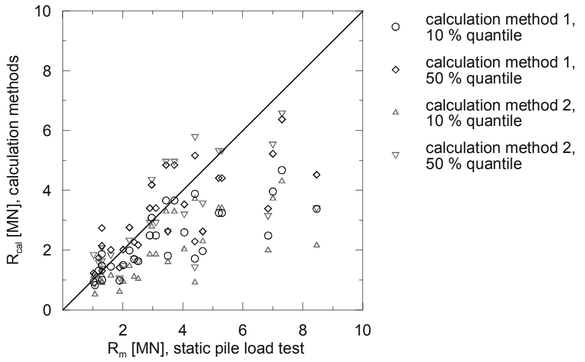

Figure 7. Calculation results for the characteristic pile resistance R_{cal} of both calculation methods in comparison to results R_m of static pile load tests

Figure 7 shows that the requirements of the calculation methods for the 10 % and 50 % quantile are fully accomplished. Further calculations and variations of parameters are given in Lüking 2010.

6 SUMMARY

The load transfer inside a plug of an open-ended displacement pile was investigated by experimental, numerical and statistical methods. It was shown that the load transfer takes place by compression arches. A fully plugged soil could not be identified.

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