Investigations of Soil Plugging in Open-Ended Piles with Respect to the Long-Term Behaviour of the Plug

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Abstract. Soil plug formation in open-ended piles can lead to a significant increase in bearing capacity as well as in driving resistance. Therefore, it is of main interest to predict the tendency of soil plug formation with respect to the pile geometry, the installation method as well as the penetration depth. In the present paper, the results from in-site testing as well as centrifuge testing are presented. The results are discussed mainly focussing on the tendency of soil plug formation with respect to the above mentioned factors. Furthermore, the results of long term in-site measurements of the earth-pressure at a tubular pile’s toe are evaluated showing that the soil plug remains stable over a period of more than one year although the pile was subjected to loading due to traffic, waves as well as tidal water changes.

1. Introduction
Open-ended steel piles are widely used on- as well as offshore mainly as foundation elements. Typical cross-sections are tubular piles but also U-, H- and sheet piles are often used. All these piles have in common that inside the profile arching might occur such that internal bracing leads to a soil plug.

Soil plugging on the one hand leads to an increase in bearing capacity as in this case the plugged base consisting of steel area and plugged soil together contributes to vertical base resistance. On the other hand, the driving resistance will also increase in the case of soil plug formation. Therefore, it is of main interest to predict the tendency of soil plug formation with respect to the influencing parameters.

Factors affecting the soil plug formation for example are:

- Pile geometry,
- penetration depth,
- soil density and
- installation method.

It is evident that it is rather complex to find an approach for soil plug prediction considering all the afore mentioned parameters such that a closed solution is not available up to now.

In the present paper, results from in-site measurements as well as from centrifuge tests are presented showing the influence of the above-mentioned parameters on soil plug formation. Beside the general discussion about soil plug formation results from long-term measurements of arching stresses inside an open-ended pile are discussed focusing on the long-term stability of a plug under cyclic loading.
2. Short literature review

Many researchers investigated the phenomenon of soil plug formation using different models. Following, exemplary research is shortly summarized giving an overview of different methods used for soil plug investigation.

- 1g-model tests were used by [1] who investigated the soil plug development in 17 model scale tests by means of Incremental Filling Ratio IFR which is defined as ratio between the measured height of the soil column inside a tubular pile and the penetration depth. It has to be commented that the IFR is not suitable for plug characterization in the case that the soil inside the pile will be highly compacted due to the pile installation process.
- [4] and [7] used the pressure chamber for soil plug investigation to better consider the stress state during pile installation.
- Centrifuge tests were for example used in the work of [2] and [3] who developed the pile driving hammer for the geotechnical centrifuge which was also used in this research.
- To overcome the weakness of model testing several researchers carried out tests in prototype scale. As an example, the results of [6] are shown in figure 1. In this figure, it is evident that the plug formation tendency inside a tubular pile is mainly influenced by soil density and pile diameter.

![Figure 1. Limiting diameter for soil plugging $d_{\text{limit}}$ with respect to soil density after [1]](image)

3. In-site testing programme

3.1. General information

During the construction of two railway bridges and two roadway bridges in-site measurements were carried out. The tests were carried out on tubular piles with an outer diameter $d_a = 0.71$ m and a total length of 15.1 m. The general layout of the testing site and the location of the two tested piles is shown in figure 2.

The piles were installed by vibratory driving with a maximum frequency of $f_{\text{max}} = 38$ Hz up to a depth of -7 mNN. Afterwards, the piles were impact driven up to a final penetration level of approx. -13 mNN. For impact driving, a hydraulic hammer IHC S70 with a maximum driving energy of $E_{\text{max}} = 70$ kNm/blow was used.

The soil conditions at the test location are presented in figure 3. Furthermore, the pile penetration depth is compared directly with the soil conditions encountered during driving in this figure. As it can
be seen, vibratory driving is mainly carried out in clay. Afterwards, impact driving is mainly done in sand up to the final penetration depth.

Figure 2. In-situ testing site showing the test piles as well as locations of soil site investigations

Figure 3. Soil conditions at the test site compared with the installed pile
3.2. Pile instrumentation

A general layout of an instrumented pile is shown in figure 4. Photographs of the instrumented piles are summarized in figure 5. The following parameters were measured during pile installation:

- Accelerations (triaxial) and strains at the pile head to be able to predict driving resistance during installation via CAPWAP.
- Total and pore water pressure development at the pile toe (internal and external) to investigate the development of arching inside the pile in comparison with the external stresses that are not influenced by arching effects. As the piles were installed in fully saturated soil, the pore water pressures were also recorded to calculate effective stresses. For this reason, three internal as well as three external combined earth and pore water pressure sensors were installed in each pile tested, see figure 5.
- In pile B12 (see figure 3) a CPT was installed in the center of the pile to investigate the evolution of base resistance directly throughout the installation.

![Figure 4](image-url)

**Figure 4.** a) General layout of an instrumented tubular pile and b) combined earth pressure and pore water pressure sensor

3.3. Exemplary results

In the present paper, the focus is laid on internal and external stress development as indicator for soil plug formation. In figure 6 the total stress and pore water pressure measurements during vibratory driving are shown separately for internal and external sensors. In figure 7 the results for the following impact driving process are summarized.

Regarding vibratory pile driving it is remarkable that only a slight pore water pressure increase during driving in clay is visible. Obviously, the pore water is not significantly influenced as a result of the rather small thickness of the piles. By comparing effective stress development during pile installation, it becomes clear that there are no significant differences between internal and external values. This indicates that no significant arching occurs such that no soil plug is formed.
Figure 5. Exemplary pictures of the instrumented piles showing the installed measurement devices at the pile toe

Regarding the results during impact driving the behaviour is completely different. Directly after start of driving the internal stresses increase much more than the external values. At final penetration depth, the ratio between internal and external effective stresses is about 4-5 which indicates that a soil plug formed inside the pile.

In the case that the internal stresses are normalized on the initial vertical stresses a value of $K_{pf} = \frac{\sigma_{meas}}{\sigma_{geo}} \approx 9$ is calculated whereas the external normalized stresses do not show a significant increase.

3.4. Long term behaviour of the soil plug

The test piles are directly influenced by tidal water changes of the River Elbe for example. Furthermore, after finalization of bridge construction the piles were subject to dynamic traffic loads. Therefore, it has to be investigated whether or not this cyclic pile loading influences the stability of the plug. Thus, the total pressures inside the piles were monitored over a period of approx. 700 days. Therefore, the effective stresses during the whole construction period as well as about one year after finalization were recorded.

In figure 8 the effective stress development over time inside and outside pile B13 (see figure 2) is shown. It is evident that the internal stresses as well as the external ones remain constant over the whole investigation period. Thus, the cyclic loading of the pile does not influence soil plug stability even after rather long time.
4. Centrifuge testing programme

4.1. General information

All tests were carried out in the geotechnical beam centrifuge at the University of Western Australia, see figure 9.

All tests were executed at an acceleration level of 50g. Thus, the conversion from model to prototype scale is done using a factor of $N = 50$. The acceleration filed was adjusted that the acceleration of 50g acted at 1/3 of the pile’s final penetration depth.

All tests were carried out in very fine dry silica sand, a pure sand with a mean grain diameter of $d_{so} = 0.19$ mm. The maximum and minimum void ratios are determined as $e_{\text{max}} = 0.79$ and $e_{\text{min}} = 0.49$.

In one test box in general six or seven piles were installed. The distance between two piles was set to at least six times the pile diameter to avoid influence between different piles. The tests were carried out in loose as well as dense silica sand. The resulting void ratio was calculated out of the soil volume and the weight. To verify homogenous soil conditions core penetration tests were carried out in advance of the tests. The relative soil density $I_{\text{0}}$ for loose packings ranged between 0.25 and 0.35. The dense soil showed relative densities $I_{\text{0}}$ between 0.92 and 0.95.
4.2. Piles

Piles with different cross-section were investigated, see figure 10:

- Four tubular piles with different diameter,
- three U-profiles with different height and
• three sheet-piles with different opening angles.

4.3. Installation method

All piles are installed by quasi-static jacking with constant velocity. Furthermore, the tubular pile with 0.83 m diameter in prototype scale as well as the U-pile with a 0.45 m height in prototype scale are installed by cyclic jacking as well as impact driving.

The installation by jacking is done displacement controlled with a velocity of 0.2 mm/s (1 cm/s in prototype scale). Cyclic jacking was also done displacement controlled with similar jacking velocity but with additional sinusoidal movement of the actuator. The frequency was set to \( f = 0.25 \) Hz with an amplitude of 1 mm. Impact driving was modelled using the pile driving hammer developed by [2] and [3]. The installation using the pile driving hammer is done force-controlled with a driving frequency of 10 Hz. The falling height is varied from 10 mm for penetration into loose soil and 20 mm for penetration into dense soil. The falling mass is chosen as 50 g.

![Figure 9. Schematic setup of the geotechnical beam centrifuge](image1)

All tests are carried out up to a final penetration of 150 mm in model scale. The installation was done in-flight and stopped at 75 mm and 100 mm to carry out a static load test. After reaching final penetration depth a further load test was carried out.

4.4. Instrumentation

Main focus was laid on the development of total stresses inside and outside the piles. Stress sensors were located inside and outside the piles 5 mm and 40 mm above the pile toe. The maximum capacity of the sensors was approx. 1 MPa. Only the small tubular pile was not instrumented due to space reasons.

Furthermore, the penetration resistance was continuously measured at the pile head during installation (jacking, cyclic jacking). As impact driving was modelled force-controlled the penetration resistance was only measured during the static pile load tests.

4.5. Results

4.5.1. Influence of installation method

Exemplary, the penetration resistance of a U-profile with a prototype height of 0.45 m during jacking as well as cyclic jacking is compared in figure 11.

It is evident that the penetration resistance is higher during jacking. Furthermore, it can be seen that due to the cyclic motion of the U-profile the penetration resistance continuously changes during cyclic jacking.
For better understanding of soil plugging, the normalized total stresses $K_{Pf}$ at the pile toe are compared in figure 12. The normalization is done by dividing the measured total stresses by the initial vertical stress acting at the level corresponding to the current penetration depth. Therefore, $K_{Pf}$ is interpreted as effective earth pressure coefficient which indicates soil plugging inside a profile. If $K_{Pf}$ is significantly higher inside a profile compared to the outer value arching leads to soil plug formation. Furthermore, a constant value of $K_{Pf}$ over depth indicates formation of a stable plug.

In the present case, it can be seen that after approx. 2 m penetration, a constant $K_{Pf} = 9…10$ is measured inside the jacked U-profile. For the impact driven pile the development of $K_{Pf}$ is similar during the first 2 m but afterwards significantly decreases. This is due to dynamic effects as the plug which starts to form continuously is loosened due to the dynamic forces.

4.5.2. Influence of pile geometry

The influence of pile geometry is discussed for tubular piles. The main influencing factor is the pile diameter. To investigate the effect of pile diameter, the bearing capacity at different penetration depths (3.5 m, 5 m and 7.5 m in prototype scale) received out of the static load tests is compared in figure 13. In this figure, the bearing capacity is normalized on the pile area for reasons of comparability.

It can be seen that the normalized resistance is the highest for the pile with diameter $D_{ip} = 0.83$ m whereas the bigger pile ($D_{ip} = 1.44$ m) has significantly lower normalized resistance at 3.5 m and 5 m depth. At 7.5 m depth the normalized resistance of the biggest pile increases over-proportionally. This indicates that soil plugging starts to develop at this depth for the biggest pile whereas the other piles developed a soil plug at shallower penetration depths.

For justification, the total stresses for the piles with diameter 0.83 m and 1.44 m are depicted in figure 14. It is evident that at about 5 m penetration depth the internal stresses inside the bigger pile increase significantly such that due to this high arching a soil plug formed which lead to the increase in bearing capacity observed in figure 13.

5. Conclusion

The present paper presented extensive investigations of soil plugging by means of ng-model tests as well as in-site measurements. The following results could be drawn out of the research presented:
Pile geometry significantly influences soil plugging. As shown in the centrifuge, the pile diameter is the main influencing geometric factor for tubular piles. But for piles with different cross-section further geometric factors could be identified (refer to [5]).

The installation method significantly influences the soil plug formation process. During quasi-static jacking the internal stresses are much higher compared to dynamic installation methods. Furthermore, the in-site test shows that during vibratory pile driving arching inside the pile is unlikely to occur.

As expected, further influence can be seen regarding soil density, see [5].

With respect to all results received out of the model as well as in-site testing, it is evident that one important value to predict plug capacity is the plugging coefficient $K_{PF}$ which has to be investigated in more detail in further research work.

**Figure 13.** Normalized resistance of jacked tubular piles with different diameter at different penetration depths (loose sand $I_D \approx 0.3$)

**Figure 14.** Total stress at the pile toe over penetration depth for different tubular piles during jacking (loose sand $I_D \approx 0.3$)

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