A Case Study of the Interaction between a Pile and Soft Soil focusing on Negative Skin Friction using Finite Element Analysis

P. Belinchón
Aarhus University, Department of Engineering, Denmark, pabbeba@gmail.com

K. K. Sørensen
Aarhus University, Department of Engineering, Denmark

R. Christensen
Ramboll and Aarhus University, Aarhus School of Engineering, Denmark

ABSTRACT
This paper presents the findings of an investigation carried out with numerical methods in order to evaluate the interaction between a driven concrete pile and soft organic clay (gyttja) in the case where the soft soil is experiencing settlement relative to the pile. A numerical model was developed based on a full-scale field test that was carried out to investigate the effects of bitumen coating on the development of negative skin friction on driven concrete piles. The bitumen was modelled with interface elements and was controlled by the strength reduction factor $R_{\text{inter}}$. Relevant soil and pile properties were partly obtained from laboratory testing. Numerical and analytical solutions were compared with field test results. Two consecutive scenarios were considered. The first scenario assumed a slightly over consolidated soft soil loaded vertically through the addition of a fill layer at ground level. This scenario reflects the conditions under which the field test was carried out. For the second scenario, additional loading was applied to bring the soft soil into a normally consolidated state. No field results were available for comparison since this scenario will be the next step of the experimental project. Hence, the first part served as a check against the observed field behaviour whilst the second part is used to predict future behaviour.

Keywords: soft soil, finite element modelling, negative skin friction, soil-pile interaction.

1 INTRODUCTION

When loading a single pile driven into soft soil, shaft and base resistance will mobilise to carry the load. Subsequent surface loading of the soft soil is likely to result in settlement of the soil relative to the pile. As a consequence, down drag forces will appear along the pile shaft where the soil settles more than the pile (Fellenius, 1984). This effect is called negative skin friction and can be a problem since it increases the working load of the pile (Tomlinson and Woodward, 2008).

It is common practice in Denmark to apply bitumen coating to the top part of precast concrete piles in order to reduce negative skin friction (Møller et al. 2016). On the basis of the case study presented below, a numerical model is created and presented that investigates the interaction between a pile and soft soil, with emphasis on the development of negative skin friction.

2 CASE STUDY

In 2013, the Research Group of Geotechnical Engineering at Aarhus University in collaboration with Per Aarsleff A/S and Centrum Pæle A/S initiated a project to study the effects of bitumen coating on the development of negative skin friction for driven concrete piles in soft soils.

A full-scale pile test setup was established at Randers Harbour, Denmark. It consisted of four test piles (T) and five reaction piles (R) installed in a row with 3 m centre-centre spacing. A longitudinal section view of the
Modelling, analysis and design

test setup is illustrated in Figure 1. All piles are precast reinforced concrete piles with a square cross-section (0.25 m width). Bitumen coating was applied to test piles T1 and T3. Piles T2 and T4 are not coated. A HE 280 M beam is directly supported by the reaction piles. The test piles are connected to the beam through an anchor. A load cell is installed between the beam and the anchor, as shown in Figure 2, so the total down drag force on the test pile could be measured.

Other in situ instrumentation comprised four piezometers and four magnetic extensometers located at different depths approx. 1.5 m from R1 (away from the pile row). These have been used to monitor the pore water pressures and the settlements of the soil respectively.

The ground investigation (CPT and boreholes) carried out prior to pile installation showed the soil stratigraphy to consist of: 1.0-1.3 m of fill overlain by 6.5-9.5 m layer of soft organic clay (gyttja) over sand (with gravel). The water table (WT) was at the top of the soft clay (Ventzel and Jensen, 2013).

After pile installation and a period of rest, 0.8 m of fill was placed around the piles to initiate settlement in the soft soil below and hence to generate negative skin friction on the piles. Monitoring of the field test is still on-going, and the results are yet to be published. Preliminary results have been presented by Sorensen (2015).

3 FINITE ELEMENT MODEL

A numerical model has been developed to simulate and further analyse the full-scale field test. The Finite Element software PLAXIS 2D AE is used in the analysis.

A single pile is considered in an axisymmetric model to represent test piles T1 and T2 (pile length of 9.75 m with pile toe at a depth of 8.75 m below original ground level). The original square cross-section is
converted to an equivalent circular cross-section (radius = 16 cm) with the same perimeter (1 m), i.e. the same shaft surface. The model has a radius of 15 m, large enough to minimise boundary effects. The mesh is formed by 15-noded elements with “fine” global coarseness. WT is assumed at the top of the gyttja. The analysis is carried out as long-term drained. Figure 3 shows the FE model in Plaxis.

![Figure 3 FE model in PLAXIS.](image)

The initial conditions of the soil are obtained by the \( K_0 \)-procedure followed by a plastic calculation. A subsequent plastic calculation phase is applied to activate the pile, anchor and interface elements. At this stage all displacements are reset to zero.

Two scenarios are considered. In Scenario 1, a surface load is applied by placing an additional 0.8 m of fill at ground level \( y = 0 \) m with a radius \( R = 5.8 \) m. The full-scale field setup reflected this scenario, thus numerical and field results can be compared. In Scenario 2, further loading of the ground level is simulated. The additional applied load ranges from 15 to 50 kPa, and the results are used to predict future behaviour (field test results are presently not available for comparison with the FE model).

The main limitation of the 2D numerical model of a single pile compared to the field setup is that group effects cannot be studied. However, for one row with 3 m centre-centre spacing between the piles (approximately 10 times the diameter) group effects can be neglected according to previous research (Comodromos and Bareka, 2005), (Jeong et al., 1997). Furthermore, as the reaction piles surrounding the test piles have been coated with bitumen in the settling soil layers, this significantly reduces their influence on the settlement of the soil around the test piles.

### 3.1 Soil models and parameters

The stratification and soil properties assumed in the model are presented in Table 1. Drained (D) parameters are specified in all cases. The properties of the existing and additional fill (soft organic sandy silty clay) and sand layers have been roughly estimated based on the soil description and ground investigation. Simple Mohr-Coulomb (MC) model parameters are used to represent these layers.

Since the settlement and soil-pile interaction is mainly controlled by the properties of the soft organic layer, these properties have in contrast been determined based on laboratory testing (direct shear tests and oedometer tests) carried out by students from Aarhus School of Engineering (Brandt et al., 2015).

#### 3.1.1 Soft organic clay (gyttja)

A soft soil model is chosen to represent the behaviour of the soft organic soil layer, as it is important to reflect accurately the changes in stiffness and strength of the soil with compression. The soft soil model has a logarithmic compression behaviour controlled by the modified compression index \( \lambda^* \) and the modified swelling index \( \kappa^* \) as defined in Figure 4.

The results from oedometer tests have shown the soft organic soil to be slightly overconsolidated. The assumed value of the pre-overburden pressure (POP) is given in Table 1.

In scenario 1, the soft soil experience reloading from a slightly over consolidated state up to around the preconsolidation pressure, i.e. settlement depends primarily on \( \kappa^* \). While, in scenario 2 where the soil is further loaded, the settlement is primarily governed by the compression of a soil in a normally consolidated state and the parameter \( \lambda^* \).
Figure 4 Logarithmic relation between volumetric strain and mean stress (Brinkgreve et al., 2014).

3.2 Pile

A linear elastic model is used for the concrete pile. It is a simple constitutive model but it represents adequately the behaviour of the pile for the stress levels applied. Parameters are given in Table 2.

Table 2 Pile parameters.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Pile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material model</td>
<td>Linear elastic</td>
</tr>
<tr>
<td>Drainage type</td>
<td>Non-porous</td>
</tr>
<tr>
<td>Density $\gamma$ [kN/m$^3$]</td>
<td>24</td>
</tr>
<tr>
<td>Young’s modulus $E'$ [kN/m$^2$]</td>
<td>26</td>
</tr>
<tr>
<td>Poisson ratio $\nu'$ [-]</td>
<td>0.2</td>
</tr>
</tbody>
</table>

3.3 Interface

Interface elements are used to model the pile-soil interaction. They are placed along the vertical limit surface between the pile and soil, from the pile top to 0.5 m below the bottom of the pile (see Figure 5). This extra length is very important to avoid non-physical stress oscillations in the corner. Interface elements are also placed at the base of the pile. The roughness between the pile and the soil is defined by a strength reduction factor $R_{\text{inter}}$.

Figure 5 Detail of the interface at the pile toe.

3.4 Anchor

The pile is assumed vertically restrained at the top. This is modelled with a fixed-end anchor element. A linear elastic material model is used to define the anchor, where the value of $EA = 160,850$ kN is determined based on the stiffness of four M16 steel bolts (shown in Figure 2) with a total area $A = 256$ mm$^2$ and a Young’s modulus $E = 200,000$ MPa.

4 ANALYSIS OF SCENARIO 1

4.1 Settlement of the soil

During the field test, the settlement of the soil was measured using four extensometers placed at different depths and, approximately, 1.5 m away from reaction pile R1. The measured settlements 315 days after loading (the additional fill layer of 0.8 m) are shown in Figure 6. At this point the pore water pressure and time-settlement curves indicated nearly full consolidation of the soft soil layer (Sorensen, 2015).
Two analytical solutions and the FE PLAXIS solution (at x=1.5 m, using $\lambda^* = 0.1013$ and $\kappa^* = 0.0231$) are also shown in Figure 6 for comparison.

In Analytical Approach A (AA-A), a simple linear relation between settlement (s) and variation of vertical effective stress ($\sigma_v^\prime$) is assumed using a constant value of stiffness $E_{oed}$.

$$s = \frac{1}{E_{oed}} \sum_i \Delta\sigma_v^\prime H_{0,i}$$  \hspace{1cm} (1)

An initial thickness of sublayers $H_{0,i} = 0.5$ m is considered. An oedometer modulus $E_{oed} = 4000$ kPa was obtained directly from oedometer tests at an appropriate stress interval.

$$E_{oed} = \frac{\Delta\sigma}{\Delta\varepsilon}$$  \hspace{1cm} (2)

The linear variation of $E_{oed}$ relative to stress was investigated but over the small stress range, the variation in $E_{oed}$ values obtained from the oedometer testing showed little effect on the results.

In Analytical Approach B (AA-B) settlement of the soft soil is based on the logarithmic relation (in reloading) between volumetric strain ($\varepsilon_v$) and mean stress ($p^\prime$) (Brinkgreve et al., 2014).

$$\varepsilon_v^\prime - \varepsilon_v^0 = -\kappa^* \cdot \ln\frac{p^\prime}{p_0^\prime}$$  \hspace{1cm} (3)

If the vertical strain is assumed equal to the volumetric strain (no horizontal strain) the settlement can be estimated from Eq. (4).

$$s = \kappa^* \cdot \sum_i \Delta \ln p^\prime_i H_{0,i}$$  \hspace{1cm} (4)

As seen from Figure 6 the measured settlements at levels $y = -7.1$ m and $y = -10.1$ m are identical. This suggests that the stiffness of the soil at these depths is very large, something not anticipated for this level of stresses. It is probable that measurements from the extensometer at $y = -7.1$ m are erroneous due to relative movement between the soil and the extensometer.

Generally, it can be observed that the Plaxis model fairly accurately predicts the measured settlements. AA-A gives results that are fully comparable with the Plaxis results up to a level approximately 4 meters below ground level. Near the ground level, where the stresses are small, AA-A underpredicts the settlement compared to Plaxis. This may be explained by the differences in the logarithmic (PLAXIS) and linear (AA-A) strain-stress relation used.

AA-B is seen to result in a significant over estimation of the settlements compared to the other solutions and the field results. The main reason for this discrepancy is the assumption of zero horizontal strain, which is not likely to be true. An analytical solution using $\kappa^*$ (and $\lambda^*$) requires further investigation.

### 4.2 Effect of $R_{\text{inter}}$ on the relative settlement between soil and pile

The relative settlement between soil and pile from the PLAXIS model is shown in Figure 7. No neutral point (defined as the point at which the settlement of the pile equals the settlement of the soil) is found since the soil settles more than the pile at all depths. A decrease of the strength reduction factor $R_{\text{inter}}$ leads to an increase in relative settlement due to both larger settlement of the soil and smaller settlement of the pile. As discussed in section 4.3, this results in less negative skin friction. Figure 7 also shows that none or very small relative settlement occurs at the pile top. This is due to the low confining

![Figure 6 Settlement profile.](image)
pressure of the fill at the boundary zone. In addition, the cohesion of the fill (6 kPa) results in the soil adhering to the pile top. If \( c' = 0 \), the relative settlement would be greater at the top. However, this effect only occurs for a small length (a few centimetres) and it does not affect the results significantly.

![Figure 7](image)

*Figure 7 Relative settlement soil-pile.*

Figure 8 shows the settlement of the original ground level at distances \( x \) from the centre of the pile for different values of \( R_{inter} \). As the movement of the pile is restrained by the anchor it hinders the settlement of the surrounding soil for high values of \( R_{inter} \). For low values of \( R_{inter} \) the soil is barely affected by the presence of the pile.

Regarding the previous section, Figure 8 also shows that the settlement of the soil at \( x = 1.5 \text{ m} \) varies a few mm depending on \( R_{inter} \). However, the difference is very small compared to results from AA-B (Fig. 6).

![Figure 8](image)

*Figure 8 Settlement of the original ground level (\( y = 0 \text{ m} \)) at \( x \) distance from pile centre.*

### 4.3 Effect of \( R_{inter} \) on shear stresses along the pile shaft

As discussed, the soil settles more than the pile at all depths. Hence, negative skin friction develops along the entire length of the pile. However, the maximum shear stresses are not mobilised over the full length of the pile as seen in Figure 9.

![Figure 9](image)

*Figure 9 Shear stresses along the pile shaft (black line for mobilised shear stress and grey line for maximum shear stress).*

Table 3 shows the total down drag force \( F_{neg} \) developed along the pile shaft and the relative settlement \( s_{rel} \) required to mobilise full skin friction for the different \( R_{inter} \) values.

Table 3 shows that a reduction in \( R_{inter} \) leads to less \( F_{neg} \) and that the relative settlement required for full skin friction to develop increases. Higher settlement normally gives higher mobilised friction, but since \( R_{inter} \) is reduced, mobilised friction is less.

![Table 3](image)

*Table 3 Total down drag forces and relative settlement to mobilise full skin friction in the FE model.*

Based on the results of direct shear interface tests \( R_{inter} = 1 \) is considered for the uncoated piles and \( R_{inter} = 0.12 \) for the coated piles. \( R_{inter} = 1 \) might be too high since a rigid interface implies that the soil is able to transfer the full shear stress to the pile which
results in a very large $F_{\text{neg}}$ (111 kN). The actual $R_{\text{inter}}$ values might be lower since piles are precast and have a fairly smooth concrete surface. For bitumen coated piles $R_{\text{inter}} = 0.12$ is considered and numerical results show that $F_{\text{neg}} = 6$ kN. This suggests a reduction of 95% in $F_{\text{neg}}$ due to the effect of the bitumen coating. The field test results show that 1 year after surcharging, $F_{\text{neg}} = 4$ kN on the coated test pile T1. 41 kN are registered on uncoated test pile T2 with values further increasing.

Greater $F_{\text{neg}}$ values are obtained from the numerical FE model compared to the field test results. This may be caused by an over estimation of $R_{\text{inter}}$ values, especially in the case of the uncoated piles. The reduction in negative skin friction as a function of $R_{\text{inter}}$ is in good agreement with results from similar studies (El-Mossallamy et al., 2012) but using $R_{\text{inter}} = 1$ leads to excessively large $F_{\text{neg}}$. Finally, field test results do not assume any resistance force at the pile base whilst in the numerical model this force is approx. 3 kN.

4.4 Effect of anchor stiffness and position
In order to understand better the effect of the anchor the scenario of a floating pile is presented. The same model is used but the anchor is not activated. As the only boundary condition of the pile is the surrounding soil (shaft and base) equilibrium of forces impede negative skin friction to develop along the whole length of the pile.

Figure 10 shows the relative settlement between the soil and the pile for this scenario. The soil settles more than the pile in the upper part (above the neutral point) leading to the development of negative skin friction. Below the neutral point the pile settles more than the soil and positive (upwards) shaft resistance is mobilised. The position of the neutral point is found to be affected by $R_{\text{inter}}$.

If the anchor is activated, then the movement of the pile is restrained and larger negative shear stresses are developed along the pile-soil interface. The movement of the pile depends on the stiffness of the anchor. This has an influence on $F_{\text{neg}}$ as shown in Figure 11. Different values of $E_A$ are considered. One can identify three trends in the series. For low values of $E_A$, $F_{\text{neg}}$ is not affected significantly by the presence of the anchor since the movement restriction is not high enough to allow negative skin friction along the full length of the pile. However, when greater $E_A$ is adopted, negative skin friction is mobilised along the full length and $F_{\text{neg}}$ increases. For $E_A > 10^5$ kN an increase of stiffness does not significantly affect $F_{\text{neg}}$.

Consider that in the model, $E_A = 160,850$ kN. Therefore, a higher stiffness of the anchor would not increase $F_{\text{neg}}$ significantly while a lower stiffness would lead to a lower $F_{\text{neg}}$. In other words, an under estimation of $E_A$ is not a problem in this model while an over estimation of $E_A$ might lead to excessive $F_{\text{neg}}$. Nevertheless, it is important to say that an equivalent length of 1 m has been used in the model while in the field
setup the anchor length is approximately 70 cm resulting in a greater stiffness (the actual stiffness is \( \frac{E A}{L} \) but for practical reasons \( E A \) is used in the text as the anchor stiffness, since \( L = 1 \) m is considered). Also, the anchor was moved to other positions \((0 < x < 0.16 \text{ m})\) but no significant effects were noticed.

4.5 Effect of stiffness parameters of soft soil

The overburden pressure due to the addition of 0.8 m of fill is equal to 14.4 kPa, slightly lower than the pre-overburden pressure \((\text{POP} = 15 \text{ kPa})\). Settlement of the soil should be only affected by \( \kappa^* \). Different values from laboratory testing were used \((0.0174-0.0285)\). The best fit between field and numerical settlement results was obtained for \( \kappa^* = 0.0231 \) - the value given in Table 1.

On the contrary, \( \lambda^* \) is not expected to have an influence on the results for scenario 1. Oedometer tests suggest that \( \lambda^* \) values range from 0.1013 to 0.1377 \((0.1013 \text{ has been assumed in the previous analysis})\). It has been checked that both values result in the same settlement profile at \( x = 1.5 \) m. However, small variations have been found regarding the relative settlement and shear stresses along the pile shaft, which affects \( F_{\text{neg}} \) as seen in Table 4.

<table>
<thead>
<tr>
<th>( R_{\text{inter}} )</th>
<th>( \lambda^* )</th>
<th>( \phi' )</th>
<th>( c' )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>111</td>
<td>77</td>
<td>40</td>
</tr>
<tr>
<td>0.7</td>
<td>104</td>
<td>71</td>
<td>35</td>
</tr>
<tr>
<td>0.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.12</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Contrary to the anticipated results, an increase of \( \lambda^* \) (softer soil in the normally consolidated state) leads to a decrease in shear stresses at the lower part of the pile shaft resulting in less \( F_{\text{neg}} \). It has been checked that no point of the soil has reached its pre-consolidation pressure, confirming that soil behaviour should not depend on \( \lambda^* \).

4.6 Effect of strength parameters

The strength of gytta is stress dependent. Figure 12 and Figure 13 show the total down drag force obtained for different values of \( \phi' \) \((20^\circ - 35^\circ)\) and \( c' \) \((0 - 4 \text{ kPa})\) respectively.

It can be seen that \( F_{\text{neg}} \) increases with \( \phi' \) for high values of \( R_{\text{inter}} \), especially from 20\(^\circ\) to 30\(^\circ\). The shear stress increases with \( \phi' \) within that range. However, the increase is lower from 30\(^\circ\) to 35\(^\circ\) and, for low \( R_{\text{inter}} \) values \( F_{\text{neg}} \) decreases relatively little with increasing \( \phi' \).

According to the Mohr-Coulomb failure criterion shear stress \((\tau)\) can be expressed as:

\[
\tau = c' + \sigma'_N \cdot \tan \phi'
\]

(5)

An increase of \( \phi' \) results in a greater \( \tan \phi' \) but conversely \( K_0 \) decreases resulting in a decrease of the normal stress \((\sigma'_N)\).

On the contrary, an increase of cohesion leads to an increase in negative skin friction independently of the stress level. Figure 13 shows that even for low values of \( R_{\text{inter}} \) the...
total down drag force increases with \( c' \). Hence, the effect of cohesion is greater compared to the effect of friction angle for the investigated range.

5 ANÁLISIS DE SCENARIO 2

5.1 Settlement of soil

Figure 14 shows the settlement profile at \( x = 1.5 \) m for different load values (q) and range of \( \lambda^* \) obtained from laboratory testing.

If in scenario 1 the maximum settlement was about 30 mm, then in scenario 2 the soil experiences a much greater settlement for a similar increase in the overburden pressure (109 mm of total settlement for \( q = 15 \) kPa). This is due to the soil being loaded to a normally consolidated state. For higher q the difference in settlement due to variation of \( \lambda^* \) is higher since more soil is normally consolidated and, hence, settlement is governed by \( \lambda^* \).

5.2 Negative skin friction

In Table 5 numerical values for down drag forces are shown. Scenario 1 corresponds to \( q = 0 \) kPa so all results can be compared.

It is important to note that, in contradiction to expected results, the low values in the intervals correspond to \( \lambda^* = 0.1377 \) which lead to higher settlements of the soil. It is not quite clear why an increase of \( \lambda^* \) produces a decrease in \( F_{\text{neg}} \) and, why this effect is greater for \( q = 0 \) (when the soil is over consolidated). Hence, this point requires further investigation.

Table 5 Down drag forces.

<table>
<thead>
<tr>
<th>( R_{\text{inter}} )</th>
<th>1</th>
<th>0.7</th>
<th>0.4</th>
<th>0.12</th>
</tr>
</thead>
<tbody>
<tr>
<td>( q ) (kPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>104-111</td>
<td>71-77</td>
<td>35-40</td>
<td>5-6</td>
</tr>
<tr>
<td>15</td>
<td>130-135</td>
<td>92-96</td>
<td>47-50</td>
<td>8-9</td>
</tr>
<tr>
<td>30</td>
<td>153-157</td>
<td>113-115</td>
<td>60-61</td>
<td>12-13</td>
</tr>
<tr>
<td>50</td>
<td>187-187</td>
<td>138-138</td>
<td>75-75</td>
<td>16-16</td>
</tr>
</tbody>
</table>

Figure 15 shows the results tabulated in Table 5. In agreement with previous studies (Comodromos and Boreka, 2005), (Liu et al., 2012), an increase of the applied loads results in greater down drag forces transmitted to the pile. Also, the effect of soil stiffness on \( F_{\text{neg}} \) disappears for \( q = 50 \) kPa, since full negative skin friction is mobilised along the full length of the pile. Hence, for the same level of stresses, larger settlement does not give higher \( F_{\text{neg}} \).

6 CONCLUSIONS

Based on a full-scale field test, a numerical model was developed to investigate the soil-structure interaction between a driven concrete pile and soft organic clay (gyttja) with emphasis on the development of negative skin friction due to settlement of the soil relative to the pile.

In scenario 1, relative settlement between the soil and the pile was investigated by
adding 0.8 m of fill at ground level. The calculated total settlement of the soil was similar to field test results (30 mm). A simple analytical solution (AA-A) that considered a constant stiffness gave comparable results to the FE model at a depth greater than 4 m below the ground surface. A more complex analytical analysis (AA-B) that considered similar parameters to those adopted in PLAXIS requires further investigation.

The presence of bitumen was modelled with interface elements and controlled by the interface reduction factor $R_{\text{inter}}$. Based on the results of direct shear interface tests, $R_{\text{inter}} = 1$ was considered for the uncoated piles and $R_{\text{inter}} = 0.12$ for the coated piles. Numerical results showed that a reduction of $R_{\text{inter}}$ leads to greater relative settlement. The presence of a bitumen coating also reduced the length of the pile that experienced full mobilised negative skin friction.

Total down drag forces $F_{\text{neg}}$ from the numerical model (104-111 kN) were found greater than the field test results (41 kN) for the uncoated pile. It was assumed that a rigid interface mainly caused this. Group effects were not expected to have a big influence due to the large centre-centre interspacing and since the reaction piles are coated with bitumen within the settling soil layers.

The anchor (stiffness) was found to have an influence on the relative settlement between soil and pile and, hence, on $F_{\text{neg}}$.

Conversely to the anticipated results, the modified compression index $\lambda^*$ had an effect on numerical results in scenario 1, although the analysis did not indicate any part of the soil matrix to have moved into a normally consolidated state. An increase of $\lambda^*$ resulted in a reduction of $F_{\text{neg}}$, which could not be explained and requires further investigation.

Numerical results were not significantly sensitive to the effective friction angle $\phi'$, but an increase of the effective cohesion $c'$ increased the negative skin friction.

In scenario 2, an additional load $q$ was applied on top of the fill. For a similar overburden pressure applied as in scenario 1, the settlement was approximately three times greater (most of the soil was expected to behave in a normally consolidated state). As expected, the value of $\lambda^*$ had a greater impact on the settlement with increasing values of $q$. However, numerical results for $q = 50$ kPa showed that the stiffness had no influence on $F_{\text{neg}}$ (the load was large enough to mobilise full negative skin friction along the full length of the pile).

7 ACKNOWLEDGEMENTS

The authors want to thank Per Aarsleff A/S and Centrum Pæle A/S for their contribution to the project, as well as previous students at Aarhus School of Engineering, who have been involved in the project and contributed with field and lab results.

8 REFERENCES


