Numerical Evaluation on Consolidation Settlement of Suction Caissons in Marine Environment

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Abstract
Consolidation settlement analysis is a necessary part of the design process for suction caissons. However it is a complex task since soil volume important for settlement analyses is mainly affected by the installation process. Consolidation settlements have been found to be the critical design criterion in several particular subsea projects. Therefore performing adequate and comprehensive analysis seems inevitable. The finite element method (FEM) has been used to consider the consolidation process after completed installation of the suction caisson. Studying of shear strength increase with time along the skirts has been done. During consolidation settlement, dissipation of excess pore pressure result in higher effective stresses. Consequently the modeled caisson undrained shear strength increases. Adequate modeling of the changes in the interface zones adjacent to the caisson walls during consolidation is vital for correct prediction of long term settlements. Modeled undrained shear strength have huge impact on the analysis results due to different mobilization of the surrounding soil. Appropriate evaluation of soil structure interaction is essential to assess the reliability of the analysis. The results were found to be reasonable with respect to final consolidation settlements and development of mobilized shear strength with time.

Keywords: Suction Caisson, Consolidation Settlement, Undrained Shear Strength

Introduction
Suction caisson is an important foundation alternative in several offshore projects. Bucket foundations exhibit viable and attractive qualities with respect to capacity and cost efficiency. In recent years FEM analysis for design of suction caissons has become an essential and inevitable tool. Consolidation settlement analysis is an important part of this task. However evaluation of consolidation settlement is a complicated task. To underline the significance of adequate and accurate analysis it should be noted that consolidation settlements have been determined to be the critical design criterion in several particular subsea developments [1]. Throughout application of FEM code PLAXIS and procedures suggested in current paper, the principles of consolidation settlement analyses will be investigated. Emphasize will be put on modeling the increase in effective stresses and undrained shear strength with time along skirt wall and evaluation of final consolidation settlement.

Impact of installation outside caisson skirt
Skirt penetration during installation will reduce the shear strength of the clay along the outside of the caisson. The remolded undrained shear strength which is the original undrained shear strength (Su) divided by the sensitivity (Sγ) is believed to be a good approximation. For comparison, the final design undrained shear strength after installation and full regeneration of shear strength can be as high as 30% higher than remolded undrained shear strength [2].

The soil displacement pattern and therefore the effect on the stress distribution along the skirt is extremely dependent on penetration procedure. During penetration due to self-weight, there is remarkable soil displacement to the outside of the suction caisson. Displacement extends the furthest at the tip of the skirt. Along the upper parts of the skirt wall the mainly influenced zone has a thickness approximately equal to the skirt thickness. Moreover a series of centrifuge tests performed which suggested that an average around 50% of the soil displaced after complete installation (both by self-weight and suction) flows inward into the suction caisson [3]. The same tests also indicated an inward soil flow of approximately 20% during self-weight penetration to a depth equivalent of four diameters.

The outward flow of soil results in increased normal stresses outside the skirt tip. This ongoing process will also increase normal stresses along the skirt wall above the present skirt tip elevation. The stress change causes an increase in excess pore pressure. With dissipation of excess pore pressure increased effective normal stresses is expected. Furthermore it gives potential for increasing interface friction (set-up) with time.

When underpressure is applied in order to penetrate the caisson foundation further (after reaching equilibrium between self-weight and skirt friction resistance) the soil distribution changes character. Despite no outwards soil displacement the penetration of the skirt tip could leave some strains outside the skirt wall. Moreover, increased shear strength due to high strain rates as a result of the thin shear zone could contribute to the thickness of the remolded zone. Other centrifuge tests show that assumption of a remolded zone with a thickness of one skirt width seems reasonable.

Impact of installation inside caisson skirt
Soil displacement pattern and effect on shear strength inside the suction caisson are extremely influenced by geometry and design. For instance, inside stiffeners will affect both the remolded zone and the clay plug inside the caisson.
However in this study a simple design without inside stiffeners is considered. Hence the effect of inside stiffeners and geometry change will not be further investigated.

The clay sample inside a soil sampling tube is a good analogue to the clay plug inside a suction caisson after installation. It is evident that a soil sample may experience strong shear strains along the soil sampling tube wall. Additionally the clay plug within the thin shear zones seems to deform quite uniformly throughout the sample. This assumption is supported by experience from model test results [4]. The thickness of the remolded zone along the skirt wall have been studied and described comprehensively by scientists. The basic concept of a remolded zone with a thickness approximately equal to the thickness of the skirt wall seems incorporated in most studies.

Figure 1 indicate the soil displacement path during installation with applied suction. For a simple suction caisson without inside stiffeners, point 1 and point 2 is relevant. Point 1 is indicating the in situ conditions before penetration, while point 2 shows the situation after caisson penetration of the soil element. It is presumed that the soil element is not subjected to any stress changes before it enters the caisson. This is a reasonable assumption since the caisson is penetrated by suction and there is no additional external load. Subsequently it is assumed that an intact clay plug deforms uniformly. This generates shear strains in the clay plug while deformed inside the suction caisson. Since there is no volumetric change in an undrained load situation the imposed horizontal displacement generates equivalent vertical displacement.

After completed penetration to required depth the underpressure will be turned off. Successively the total skirt friction will be reduced to equilibrium with the submerged weight of the suction caisson. Since this friction is rather small compared to the weight of the clay plug, the total stress relative to the seabed after completed skirt penetrations is supposed to be the equivalent effective stress [5].

Horizontal stress equilibrium is presumed between the clay plug and the remolded zone inside the skirt wall. With no external load this means that horizontal total stress is the same within the suction caisson. Due to the large shear strains and the remolding of the soil an isotropic stress condition is assumed for the remolded zone. Effectively the vertical total stress is assumed equal to the horizontal total stress inside the remolded zone after completed skirt penetration. The pore pressure in the remolded zone depends on the soil characteristics. For soft clays, the pore pressure is expected to be equal to the total stress after installation. This indicates that the initial effective stresses are zero. This assumption is supported by direct simple shear testing and field measurement on piles during installation in normally consolidated clays. For overconsolidated clays the inclination of dilatation will affect the generation of excess pore pressure. Normally smaller excess pore pressures are expected and sometimes even buildup of negative pore pressures can occur for large overconsolidation ratios (OCR). Figure 2 suggest a possible tendency that can be used to consider the change in pore pressure in the remolded zones. Despite of the limitation in data there is a clear tendency of high excess pore pressure for low OCR, and low as well as negative excess pore pressure for high OCR. These measurements are supported by experience from piles and laboratory tests. However it should be considered that the pore pressures from the CPTU and piles are measured outside the wall. Therefore the generation of excess pore pressure inside the skirt wall might differ, but the data clearly indicate the influence of OCR.
Set-up effect on skirts in soft clay
The increase in shear strength with time is often referred to as “set-up”. The phenomenon describes increase in shear strength due to a combination of dissipation of excess pore pressure, increased horizontal effective stress and thixotropy. Thixotropy is gain in shear strength with time in spite of no volume change. The individual contribution of the three factors is time dependent and closely related to soil properties presented in Figure 3.

Penetration by self-weight or additional suction will affect the soil displacement pattern as well as the relative significance of set-up mechanisms. Usually the soil displacement during penetration will cause significant increase in normal stress in the soil. During penetration by self-weight this will be applicable for both sides of the skirt wall. For the penetration with applied suction it will primarily be credible for the remolded zone inside the suction caisson. The increased normal stresses give potential for high effective normal stresses after dissipation of the excess pore pressure. However the soil displacement outside of the suction caisson also cause additional excess pore pressure further away from the skirt. This will extend time of dissipation and regeneration of shear strength. Set-up for suction caisson during self-weight penetration is comparable with set-up for piles. With applied suction it is very different as the interface friction may be smaller than the initial shear strength due to lack of increase in normal stress [6]. Practical instruction has proposed to estimate the set-up factors summarized in Table 1 unless more accurate data from site is available. The set-up factor values are lower bound estimates for skirts penetrated by applied underpressure. Note that for overconsolidated soils a correction factor ($\alpha_{OC}/\alpha_{NC}$) for the set-up factor along the outside skirt wall is required [7].

<table>
<thead>
<tr>
<th>Inside set-up factor (left) and outside set-up factor (right)</th>
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</thead>
<tbody>
<tr>
<td><strong>Set-up factor $\alpha = S_{\text{p}}/S_{\text{pc}}$ after 2 months</strong></td>
</tr>
<tr>
<td>$\phi_{pc}$</td>
</tr>
<tr>
<td>$S_{pc} &gt; 3$</td>
</tr>
<tr>
<td>$S_{pc} &lt; 3$</td>
</tr>
</tbody>
</table>

Exploring and understanding the set-up effect for suction caissons is crucial since the design holding capacity is strongly affected. For instance, a suction caisson with a $d/t$ ratio of 5 installed in typical subsea clay with a sensitivity of 4 may have a potential for somewhat 30% increase in capacity after full regeneration of the shear strength. Furthermore it is essential to understand these mechanisms in relation to consolidation settlement calculations. The modeling of the transition from undrained to drained design situation in the remolded zones will affect final consolidation settlements.

Calculation procedure
The penetration analysis also referred to as installation analysis of a suction caisson is divided in three different analyses. These evaluations comprised of calculation of penetration resistance, necessary underpressure to complete the installation to required depth and maximum allowable suction to avoid critical soil heave or cavitation inside the suction caisson. The total penetration resistance \( Q_{\text{tot}} \) for skirts without stiffeners is calculated as the sum of the shear force along the skirt wall \( Q_{\text{side}} \) and the point end bearing capacity at the skirt tip \( Q_{\text{tip}} \).

\[
Q_{\text{tot}} = Q_{\text{side}} + Q_{\text{tip}} = A_{\text{wall}} \cdot \alpha \cdot S_u^{av} + (N_c \cdot S_u^{av} + \gamma' \cdot Z) \cdot A_{\text{tip}}
\]  

(1)

Where \( A_{\text{wall}} \) is skirt wall area (sum of inside and outside contribution), \( A_{\text{tip}} \) is skirt tip area, \( \alpha \) is shear strength factor (normally presumed equal to the inverse of the sensitivity), \( S_u^{av} \) is average direct shear strength test over penetration depth, \( S_u^{av}_{\text{tip}} \) is average undrained shear strength at skirt tip level (average of triaxial compression, triaxial extension and DSS shear strengths), \( \gamma' \) is effective unit weight of soil, \( N_c \) is bearing capacity factor in plane strain condition and \( Z \) is skirt penetration depth.

At the end of self-weight penetration (equilibrium between suction caisson weight and penetration resistance) the necessary suction for further required penetration is given by:

\[
\Delta U_n = \frac{Q_{\text{tot}} - W'}{A_{\text{in}}}
\]

(2)

Where \( W' \) is submerged weight of caisson and \( A_{\text{in}} \) is plan view inside area where suction is applied.

The installation procedure is relevant to the settlement calculation since it strongly affects the shear strength and displacement pattern of the soil. Effect of self-weight penetration linearly reduces to zero from the depth of suction penetration equilibrium to a depth of one diameter below this point. In the transition zone between self-weight penetration and penetration by suction the solution for self-weight penetration should be used to the depth where it gives the most favorable results. Given that all the displaced soil moves into the caisson during penetration with applied suction and that the clay plug deforms uniformly, the strains in the inner clay plug can be derived from:

\[
\varepsilon_r = \frac{t}{r} \quad \varepsilon_v = -2. \varepsilon_r \quad \gamma = (\varepsilon_v - \varepsilon_r) = 1.5. \varepsilon_v = -3. \varepsilon_r
\]  

(3)

Where \( t \) is skirt wall thickness, \( r \) is radius of skirt compartment (inner radius suction caisson), \( \varepsilon_v \) is vertical strain and \( \varepsilon_r \) (horizontal strain).

During skirt penetration the average vertical total stress at depth \( z \) in the clay plug calculated from:

\[
\sigma_v = \gamma' \cdot z + 2 \cdot z \cdot \frac{S_{DSS}}{S_t} - \Delta U_{\text{top}}
\]

(4)

Where \( \gamma' \) is effective weight of soil, \( z \) is depth in clay plug, \( S_{DSS} \) is average undrained direct simple shear strength over the penetration depth, \( S_t \) is sensitivity of clay and \( \Delta U_{\text{top}} \) is applied suction at the top of clay plug.

After installation of the caisson the applied underpressure is turned off. It is a conventional assumption that the top of the suction caisson then is totally sealed and the underpressure at the top of the clay plug will be zero. Subsequently the mobilized friction along the skirt wall will be reduced to equilibrium with the submerged weight of the suction caisson. Since the friction usually is very small compared to the weight of the clay plug, the vertical total stress relative to seabed after installation and the resulting horizontal total stress in the clay plug is assumed to be:

\[
\sigma_v = \gamma' \cdot z + u
\]

(5)

\[
\sigma_h = \sigma_v - 2\tau
\]

(6)

Another important aspect during installation of the suction caisson is the generation of excess pore pressures. For the inner clay plug the excess pore pressure is given by:

\[
\Delta u = \Delta \sigma_{\text{oct}} + \Delta u
\]

(7)

Where \( \Delta \sigma_{\text{oct}} \) is change in octahedral total stress in clay plug and \( \Delta u \) is generated pore pressure due to shear strains.

The change in octahedral total stress in the clay plug can be expressed as:

\[
\Delta \sigma_{\text{oct}} = \frac{1}{3} (\Delta \sigma_v + 2. \Delta \sigma_h)
\]

(8)

\[
\Delta \sigma_v = \sigma_v - \sigma_{v,c}'
\]

(9)

\[
\Delta \sigma_h = \sigma_h - K_p, \sigma_{v,c}'
\]

(10)
Where \( K_0 \) is lateral earth pressure coefficient and \( \sigma_{v,c}' \) is initial vertical in situ effective stress.

**FEM model of suction caisson**

FEM code PLAXIS 2D was used for the numerical consolidation analysis. The soft soil material model using stress dependent stiffness and failure criterion according to Mohr-Coulomb was selected. Two dimensional axis symmetry analyses were utilized in the FEM model. Figure 4 represents the connectivity plot and model boundaries, eight times (8d) and twenty-four times (24d) the suction caisson diameter in width and depth respectively. Several square clusters surrounding the suction caisson tip was added to improve soil element geometry. Plate elements were used to model the suction caisson top lid and skirt wall. The material properties were determined by choosing a structural stiffness considerable higher than the soil stiffness and being rigid enough to avoid large deflections of the steel structure. Interface elements were added to make the skirt wall impermeable. However they were not switched on in the “staged construction interface” in order to avoid too slender elements in the relatively thin interface zones. The modeled interface zones next to the skirt wall had a thickness of 0.04 m. The load was applied on the suction caisson using a distributed load across the top lid. The total load was assumed to be 2500 kN.

![Figure 4: Connectivity plot, FEM model and detailed cut of the suction caisson](image_url)

<table>
<thead>
<tr>
<th>Material parameters suction caisson</th>
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</thead>
<tbody>
<tr>
<td>Design load, L [kN]</td>
<td>2500</td>
</tr>
<tr>
<td>Caisson diameter, D [m]</td>
<td>5.0</td>
</tr>
<tr>
<td>Skirt wall thickness, t [m]</td>
<td>0.03</td>
</tr>
<tr>
<td>Axial modulus, EA [kN/m]</td>
<td>3.19E+12</td>
</tr>
<tr>
<td>Rigidity modulus, Ei [kN/m²/m]</td>
<td>2.58E+08</td>
</tr>
<tr>
<td>Distributed weight plate elements, w [kN/m²/m]</td>
<td>0</td>
</tr>
<tr>
<td>Distributed load, A:A [kN/m²]</td>
<td>127.3</td>
</tr>
</tbody>
</table>

From the soil investigation data and correlation with original test data and soil test in PLAXIS, the general clay parameters utilized in the soft soil material model are given in Table 3. The unloading-reloading Poisson ratio (\( \nu_{pr} \)) is often assumed to be between 0.10-0.20 for soft lightly overconsolidated clays, and a \( \nu_{pr} = 0.15 \) was used for all of the FEM analyses. The lateral earth pressure coefficient for normally consolidated clay is then calculated.

**Table 3: Input clay properties for soft soil material model**
Results and discussions
Initially a series of FEM analyses was carried out to put final consolidation settlement estimates in perspective with different presumptions (Table 4). Figure 5 shows the results of the FEM analyses. Base case 1 is a reference analysis assuming no initial stress changes or material parameter updates prior to the loading and consolidation of the suction caisson. This situation is not realistic with respect to a normal installation process. However it gives a lower bound value and emphasize the effect of adding the design load to a similar preinstalled and completely reconsolidated foundation structure. Evaluated settlements of approximately 10 times less than the upper bound estimate (Case 5) highlight the effect of installation and initial changes of stresses and soil properties.

<table>
<thead>
<tr>
<th>Simulation case</th>
<th>Aberration from soil parameter updates</th>
<th>Interface zones</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Clay plug</td>
<td>Inner interface zone</td>
</tr>
<tr>
<td>Base case 1</td>
<td>No updates applied</td>
<td>No updates applied</td>
</tr>
<tr>
<td>Base case 2</td>
<td>-</td>
<td>Updated according to the clay plug</td>
</tr>
<tr>
<td>Case 3</td>
<td>Combined penetration analysis (self-weight and underpressure)*</td>
<td>Combined penetration analysis (self-weight and underpressure)</td>
</tr>
<tr>
<td>Case 4</td>
<td>-</td>
<td>( \varphi = 39.5^\circ )</td>
</tr>
<tr>
<td>Case 5</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Comparing base case two with case four and five indicate how the remolded interface zones affect consolidation settlement. The effect of only updating initial soil parameters and stress properties in the clay plug (inner interface zone embedded as the clay plug) is barely noticeable when comparing base case one and two. However adding interface zones (Case 4 and 5) have a significant effect on the final consolidation settlements. Although the installation procedure in general was simplified (assumed applied underpressure during the complete installation) a more advanced FEM analyses incorporating both self-weight and suction penetration was performed (Case 3). In short the simplified installation procedure is somewhat conservative. Self-weight penetration would generate different excess pore pressure distribution due to installation and allow for buildup of more horizontal stresses along parts of the skirt wall, subsequently reducing final consolidation settlement. Initial vertical displacement during undrained loading between simulation case four assuming intact undrained shear strength in the interface zones \( S_{u,\text{int}} \) and case five utilizing remolded undrained shear strength \( S_{u,\text{rem}} \) exhibit an expected pattern. Case four experiences lower initial vertical displacement than case five due to higher undrained shear strength in the remolded zones, hence mobilization of more surrounding soil is observed. Development of final consolidation settlement also compares very well with initial expectations with case five being the most conservative.

During undrained loading of the installed suction caisson excess pore pressure is generated. The buildup of excess pore pressure is largest at the bottom of the clay plug (inside the suction caisson). Normally the excess pore pressure is larger along the inner interface zone. Below the clay plug excess pore pressure generation is decreased accordingly to the shape of a point end bearing capacity failure pattern. Figure 6 indicates how the excess pore pressures are reduced.
within the clay plug and interface zone through global dissipation, hence the excess pore pressure just below the suction caisson lid is reduced the slowest.

Figure 6: Excess pore pressure after undrained loading and at the end of consolidation

Figure 7 indicates how the vertical effective stress increases inside the clay plug during dissipation of excess pore pressure after loading. The clay plug and appurtenant zone below experience the largest increase, while the effect decreases with increasing radial distance to the suction caisson.

Figure 7: Vertical effective stress ($\sigma_v'$) after undrained loading and at the end of consolidation

During dissipation of excess pore pressure increasing effective stresses are expected. Figure 8 represents radial effective stress after undrained loading and at the end of consolidation interval. After being notably reduced due to high excess pore pressures in the remolded zone during loading, radial effective stress along the outside of the skirt wall increase with time during consolidation. The initial high radial stress inside the suction caisson due to installation is reduced with time, however the final pattern also show higher radial effective stress along inside of the skirt wall. At the clay plug and skirt tip a final radial stress concentration develops. Below the clay plug the increase in radial effective stress can be explained by load transfer (dissipation of excess pore pressure) from clay plug to soil below the caisson.
Due to the high excess pore pressures in the remolded zones (consequently low initial effective stress) the load is primarily carried by pile tip resistance during undrained loading. Dissipation of excess pore pressures and increase in effective stresses give potential for increased shear stress. Accordingly the mobilized friction increases with time and the load is distributed more to skirt wall friction. Figure 9 obviously shows how the mobilized friction is very low in the clay plug after undrained loading and significantly increasing during consolidation. The increase is largest along the inner parts of the skirt wall, while similar effect is primarily seen at the lower parts of the outer skirt wall.

Conclusion
A series of FEM analyses established the basis for evaluating the effect of consolidation of suction caissons installed with applied suction. During consolidation dissipation of excess pore pressures and increase in effective stresses result in an increase in undrained shear strength. However the increase is smaller than expected compared with set-up factors in literature review. Despite buildup of effective stresses the model is not able to account for the total set-up contribution. An adjusted simulation procedure including incremental increase of friction angle in the interface zones could be used as an alternative. However further correlation of in-situ records and monitored settlements is recommended for validation of the analyses, the results were found to be reasonable with respect to final consolidation settlements and development of mobilized shear strength with time.

Modeled undrained shear strength with time demonstrated to have huge impact on final consolidation settlements. Mobilization of soil around the suction caisson is closely related to shear strength. Higher shear strength leads to reducing final consolidation settlement for the suction caisson. However the settlement of the surrounding seabed simultaneously increases. Appropriate evaluation of soil structure interaction is important to assess the reliability of the analysis. The FEM model was found to be surprisingly sensitive to small adjustments to soil properties, emphasizing the importance of sufficient evaluation of the model behavior. Taking into account the soil volume (strength and stiffness) changes in the interface zones along the skirt wall is necessary for determining long term settlements.

References