Delft University of Technology

Master Thesis

Installation-effects of suction caissons in non-standard soil conditions

- Final report -

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Preface

This report is the final exercise of the Master program Geotechnical Engineering at the Delft University of Technology. The investigated subject "Installation effects of suction caissons in non-standard soil conditions" originates from the company SBM Offshore (SBM), a leading offshore engineering company in floating production and mooring systems. During the graduation progress I was located at the Geotechnical department of SBM Offshore in Monaco for a period of 3 months to obtain information from various practical experiences.

The report focusses on the installation-aspects of suction caissons in layered soil conditions, especially where sand is overlaid by clay. After literature review and experiments on plug uplift and cracking, an assessment of the plug uplift is presented.

Throughout the entire research project I have received lots of advice, support and help from many people. That is why I would like to seize the opportunity to express my sincere thanks to everyone who has contributed to the completion of this project.

In the first place I would like to express my gratitude to the members of my committee, who supported me throughout this entire research project. To my supervisor Sebastiaan Frankenmolen, for his patience, his enthusiasm and guidance. Numerous discussions helped me to get a good understanding of all relevant geotechnical phenomena. To Jelke Dijkstra, my supervisor at the Delft University of Technology, for his interest, constructive criticism and the effort he took to teach me the tricks of physical modeling. I have appreciated the feedback I got, as even in weekends my questions were answered. To Professor Frits van Tol and Klaas-Jan Bakker, for answering my questions whenever needed and have guided me in the right direction.

Besides my committee, I would like to thank my colleagues at SBM Offshore, for their interest in my research project and their varying contributions to the completion of my thesis. Furthermore my thanks go to Han de Visser, for his practical and cheerful assistance during my model tests. Last but not least, I would like to thank my family and friends, who had to listen for eight months to my undoubtedly very interesting, stories about suction caissons.

8th October 2013,

REMON HENDRIK ROMP
Abstract

Nowadays suction caissons are extensively used for anchoring large offshore floating facilities. Installation of suction caissons in coarse grained soils (high permeability) is achieved by induced seepage flow in the soil. This reduces the soil resistance at the caisson tip, which contributes to the installation process. Installation in fine grained materials with a low permeability (silt or clay), is achieved by suction (relative to seabed pressure) within the caisson, which forces the caisson to embed. As a result the installation in layered soils, especially sand overlaid by clay, is expected to be more challenging since the reduction of tip resistance is restrained from flow restrictions in the clay layer. It is unclear to what extent the upper low permeable layer will impact the seepage flow and thus the desired tip reduction.

During the installation in layered soils one of the main potential obstacles is excessive plug uplift, which could occur at certain applied suction pressures. The objective of the presented research is to assess the governing mechanisms of plug stability during installation of a suction caisson in layered soils where sand is overlaid by clay.

It was found from case studies on installation in layered soils that water-injection-devices (WID) were used. Subsequently, two general types of failure mechanisms of plug stability for installation of suction caissons in sand overlying by clay were identified; plug uplift and plug cracking. Further analysis showed that the failure mechanism depends on the dimensions of both the caisson (diameter $D$) and the clay plug (layer thickness $z$). Furthermore, basic experiments executed as part of this research showed that plug cracking is an unlikely failure mechanism for standard aspect ratios (i.e. $D/z < 6$)

Assessment of the plug stability was done by introducing two extreme cases; a fixed plug and a moving plug during installation. Stability of a fixed plug was assessed by introducing a fracture-ratio, which determines the crack-requirement for installation without uplift. When plug uplift can be accommodated to a certain extend (i.e. a moving plug), the relation between the plug uplift velocity and the installation-rate should be assessed. Based on analytical analyses, it is concluded that for low installation rates the total amount of uplift is higher compared to fast installation rates.

Finally, the contribution of reverse end bearing (REB) as part of the plug stability was investigated for layered soils. The uplift mechanism at the interface of the plug (i.e. the clay-sand interface) consists of the process of suction mobilization and suction release within the sand. It was found that the magnitude and duration of the temporary resistance (i.e. the reverse end bearing) depends on the dissipation of negative excess pore pressures in time. This shows that for a typical installation period (1 - 6 hours) and hydraulic conductivities lower than $1e^{-5}$ m/s, temporary capacity due to delayed inflow of water can be considered.

The analytical work was verified with the results of two analysed field cases and numerical analysis. In both field cases water injection devices were implemented as mitigating measure (i.e. to decrease tip resistance) to ensure penetration of the suction caisson in dense sands. However, this
system is at present still not a proven technology and back-calculations did not clearly indicate the benefit of these devices. The contribution (or the absence) of REB was recognized for both practical and numerical back-calculations. This indicates that the magnitude of REB is highly dependent on time-effects (i.e. the permeability of the sand and installation rates).
Samenvatting

Zuigpalen worden tegenwoordig veelal gebruikt als ankerpunt voor diverse drijvende voorzieningen in de offshore industrie. Installatie van een zuigpaal in zandige grond (met een hoge doorlatendheid) wordt gerealiseerd door de opgewekte lekstroom in de grond, wat tevens een reductie van de paalpuntweerstand genereert. Voor installatie in fijnere grond met een lagere doorlatendheid (klei of silt) wordt de relatieve onderdruk in het caisson gebruikt om een kracht te genereren die de paal de grond in drukt. Installaties in gelaagde gronden, waar zand bedekt is met een kleilaag, is complexer omdat de lekstroom verhinderd wordt door de slecht doorlatende kleilaag. Het is onduidelijk wat de invloed van deze slecht doorlatende toplaag is op de lekstroom en tevens op de vereiste reductie van de paalpuntweerstand.

Extreme opbarsting van de klei plug vormt een potentieel probleem bij hogere drukken gedurende installatie van de paal. Dit rapport heeft ten doel om de belangrijkste mechanismen (m.b.t. de stabiliteit van de plug) te onderzoeken tijdens installatie van een zuigpaal in de gelaagde grond conditie, waar zand bedekt is met een kleilaag.

Uit praktijkstudies is gebleken dat een water-injection-device (WID) wordt gebruikt om zuigpalen te installeren in gelaagde gronden. Daarnaast zijn er twee faalmechanismes beschreven voor een grondprofiel waar zand bedekt is met een kleilaag; opbarsten van de kleilaag & scheuring van de kleilaag. Toegepaste analyses tonen aan dat het faalmechanisme afhankelijk is van de gekozen dimensies, zoals paal diameter (D) en laagdikte van de klei (z). Daarnaast is door middel van eenvoudige experimenten aangetoond dat scheuring van de kleilaag optreedt voor dimensies met D/z > 6.

De stabiliteit van de kleilaag is beschouwd door twee condities te introduceren; een gefixeerd plug of een bewegende plug. In het eerste geval dient de doorlatendheid van de kleilaag vergroot te worden door scheuring van de laag, wat in een percentage is uitgedrukt als fracture-ratio. Voor een bewegende plug geldt dat de relatie tussen opbarsten van de klei en de snelheid van penetratie van belang zijn. Op basis van analytische vergelijkingen kan worden geconcludeerd dat voor lagere penetratiesnelheden de verwachte opwaartse verplaatsing van de kleilaag groter is.

Ten slotte is de bijdrage van reverse end bearing (REB) als deel van de stabiliteit van de plug onderzocht voor gelaagde gronden. Het opbarst mechanisme bij de interface van de kleilaag en het zand bestaat uit het mobiliseren van zuiging, gevolgd door het wegvloeien ervan in het onderliggende zand. De grootte en duur van deze tijdelijke weerstand tegen opbarsten hangt af van de dissipatie van negatieve wateroverspanningen in het onderliggende zand. Voor een gebruikelijke installatieduur (1 - 6 uur) en doorlatendheden lager dan 1e−5 m/s, kan een tijdelijke capaciteit door de vertraagde grondwaterstroming worden beschouwd.

De analytische resultaten zijn op basis van de gestelde randvoorwaarden geverifieerd door middel van twee praktijkvoorbeelden en numerieke analyses. Voor beide praktijkgevallen zijn WID’s als middel gebruikt om de paal in de dichtgepakte onderliggende zandlaag te installeren. Desondanks is de effectiviteit van deze WID’s is niet bewezen en ook met berekeningen niet aantoonbaar. De
bijdrage (of afwezigheid) van REB kon worden aangetoond voor beide praktijkvoorbeelden en numerieke berekeningen. Beide analyses tonen aan dat de grootte van REB sterk afhankelijk is van tijdseffecten, zoals doorlatendheid van het onderliggende zand en penetratiesnelheid.
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Chapter 1

Introduction

1.1 Background information

SBM Offshore (SBM) is a leading offshore engineering company in floating production and mooring systems. As part of the mooring systems suction caissons are frequently used as anchors for single-point-mooring systems and spread moored systems. These suction caissons are large diameters steel piles and installed by creating an underpressure inside the pile by suction. The underpressure generates a net downward force which contributes to the penetration of the caisson for either homogeneous clayey or sandy soils. For the latter condition, seepage flow is induced which contributes to tip resistance reduction of the caisson.

The main advantages of suction caissons compared to conventional offshore anchors are the relatively quick installation time and the possibility to remove the caisson by the reverse principle. Usually less equipment is required and smaller extraction spreads are mobilized for installation. Nowadays suction caissons are extensively used for FPSOs (Floating Production, Storage and Offloading) as presented in Figure 1.1 (Tjelta [2001]).

![Figure 1.1 FPSO Aseng prior to operation offshore in Equatorial Guinea, West Africa (SBM Glossary)](image)

1.2 Problem description

In general the installation of a suction caisson consist of two phases; initial self-weight penetration (SWP), followed by a suction-assisted penetration (SAP). The initial self-weight penetration depends on the effective weight of the suction caisson in relation to the soil resistance. Ballast weight can be used to increase the SWP. The second phase consists of a suction-assisted phase, which initiates
with a transitional phase where the pumping is started to remove the trapped water. Installation of suction caissons in coarse grained soils (high permeable soils) is achieved by the induced seepage flow through the soil (Figure 1.2a). This reduces the soil resistance at the caisson tip, which contributes to the installation process. The upward groundwater flow also reduces effective soil stresses and hence reduces the horizontal stress against the skirt (Erbrich and Tjelta [1999]).

For installation in clays (low permeable soils), the suction process is not creating significant seepage flow because of the lower permeability. In this case, the installation is achieved by the pressure difference over the top cap, i.e. the downward gradient of the water column provides the force to push in the caisson into the clay (Figure 1.2b).

Installation in layered soils, especially where sand overlaid by clay, is expected to be more challenging since the potential reduction of tip resistance is somehow restrained from flow restrictions (Figure 1.3a). It is not clear to what extent the lower permeable layer will impact the seepage and thus the tip reduction, which possibly is required to reach target depth.

The stability of the plug becomes an important aspect once the seepage flow is required to reduce the tip resistance and hence to install the caisson to target depth. When uplift of the clay plug occurs (Figure 1.3b) it is uncertain to what extent this generates a seepage flow. One of the main concerns is excessive plug uplift (i.e. when the plug contacts the top cap), which could occur when higher suction pressures are applied or maintained for a longer period than predicted. Plug uplift probably creates seepage flow within the underlying sand and thus reduces the tip resistance to some extent.
On the other hand, plug uplift will become problematic once the plug contacts the top cap and no suction (and thus no downward force) can be generated anymore.

This thesis focuses on the aspects of suction caisson installation in non-standard soil conditions, especially where sand is overlaid by clay. Installation aspects of suction caissons are extensively described by many authors for homogeneous clay (e.g. Houlsby and Byrne [2005]) or granular material (e.g. Andersen et al. [2008]). However, for installation aspects in layered soils limited documentation, references or guidelines are available (Senders [2008] and Cotter [2009]). Furthermore, at present safety measures are implemented in the design phase to overcome the uncertainties of excessive plug heave.

These uncertainties incorporate conservatisms, which could have a great impact on the feasibility of the suction caisson concept. A detailed research of the plug stability aspects will contribute to a better understanding of the governing aspects involved and hence reduce certain uncertainties at the design phase.

1.3 Purpose of research

The main purpose of this research is to increase understanding and gain insight into the governing mechanisms during the installation of a suction caisson in conditions where sand is overlaid by clay. Within this framework, the following objectives are formulated:

- Literature study on installation aspects of suction caissons in both homogeneous- and layered soils (sand, clay and/or silt strata);
- Formulation of an analytical model to describe the governing mechanisms associated with the installation of the caisson, i.e. cracking of the plug, variable installation-rate, plug heave;
- Gain insight in the mechanism associated with plug stability using basic experiments conducted at the Geotechnical laboratory of Delft University of Technology;
- Back-calculation of two representative field cases using the developed analytical model;
- Comparison of the analytical model with a commercially available geotechnical finite-element code (PLAXIS).

The research question to be answered is based on the objectives, available time and budget. The limitations of this research are as follows:

- Only installation aspects will be investigated (i.e. the holding capacity is not considered); This represents the penetration of the suction caisson until the design depth is reached. Neither influences of in-place conditions nor removal aspects of the suction caisson will be investigated. As well, no time-dependent effects during the lifetime of suction caissons are taken into account (e.g. set-up effects, creep in the subsoil, erosion).
- Only installation in layered soils is considered; The literature review discusses installation aspects of suction caissons in layered soil (i.e. sand, silt and/or clay strata). Further assessment of installation involves installation of sand profiles (silica sands) overlaid by clay.
- A simplified geometry for the suction caisson is considered; No effects of ring stiffeners or pad-eye stiffening will be taken into account within the analyses. Structural influences of buckling and/or radial expansion/compression will not be taken into account. Soil layers are assumed to be horizontally deposited and suction caissons penetrate vertically in the soil. Small deviations from these idealized conditions are not considered in this research.

M.Sc. Thesis R.H. Romp
1.4 Readers manual

This document includes the methods to calculate the resistances for installation of suction caissons in layered soil (Chapter 2). The next Chapter (3) discusses the plug response during installation of a caisson in layered soils. Chapter 4 focuses on the plug stability aspects, followed by a method to model the components of plug uplift in Chapter 5. Hereafter verification of the theory is presented by back-calculations of two cases and numerical calculations in Chapter 6. The conclusions and recommendations are presented in Chapter 7.
Chapter 2

Installation in layered soils

This Chapter discusses the geotechnical engineering aspects of a suction caisson installation in conditions where sand is overlaid by clay. First the general aspects of suction caisson installation are discussed in section 2.1, such as tip-reduction and seepage flow for installations in sandy soils. As well the effect of penetration-rate is presented.

In the following section (2.2) a description of the self-weight installation in a layered stratum is discussed considering conventional effective stress- and CPT-approaches as presented by the standards (DnV [1992]; API [2000]). These approaches are adjusted for layered soil conditions, which is not included in the standards. Once self-weight penetration has been completed, suction assisted penetration is applied. This condition is further discussed in section 2.3, where again the effective stress- and CPT-approaches are considered, together with the empirical approach of Senders [2008].

2.1 Installation of a suction caisson

In general the installation of a suction caisson consist of two phases; initial self-weight penetration, followed by a suction-assisted penetration (Figure 2.1). The initial self-weight penetration clearly depends on the effective weight of the suction caisson but can be increased by an additional weight on top of the caisson during penetration. In any case, it is recommended to ensure sufficient initial penetration prior to the suction assisted phase. For sands it holds that initial self-weight penetration should be deep enough to avoid piping effects. In case of homogeneous clays, the sealing is of great importance. From several practical cases it can be found that around 1 m of initial penetration is sufficient (Tjelta et al. [1986]).

![Figure 2.1](image-url)  
**Figure 2.1** Self-weight penetration and suction-assisted penetration during installation
**Installation in sand**

For sands the second phase of installation consists of; first a transitional phase followed by a suction-assisted phase. During the transitional phase, the pumping is started to pump out the trapped water. As a result, the relative pressure inside the caisson is lowered, which attracts the underlying pore water to flow inside the caisson in case of sand strata (Figure 2.2a). Around the skirt tip the induced water flow reduces the effective stress locally, which contributes to the penetration of the caisson. During the suction-assisted phase the pressure inside the caisson drops gradually, due to the combined effect of attracting pore water and pumping out water the caisson. In the next phase the rate of attracted pore water becomes constant, as well as the pumping-out rate. This phase remains constant until target penetration depth has been reached.

**Installation in clay**

In case of homogeneous clay, the relative pressure inside the caisson is lowered as well. In comparison to sand, it can be remarked that the permeability of the clay is much lower and therefore no seepage flow will be induced during a typical installation period (Figure 2.2b). Installation of caissons in clays is achieved by displacing the trapped water column, which provides the force to push the caisson into the clay. Apart from the seepage flow, this principle holds as well for sands and becomes very effective for deep-sea installations.

![Diagram](image_url)

**Figure 2.2** Schematization of suction caisson installation

### 2.1.1 Reduction of tip resistance in layered soils

Common installation of suction caissons in sandy soil is based on the principle of tip reduction due to seepage flow, induced by the applied suction. For installation in clays, the tip reduction is not applicable due to the impermeable soil. The soil resistance of clays in general is lower compared to caisson installations in sands. As mentioned in Chapter 1, installation in sand overlaid by clay could be problematic since the seepage is restricted by the clay layer.

However, according to several researchers (Tran et al. [2007], Senders [2008] and Cotter [2009]), it has been found that some reduction in underlying sand was monitored for installation in layered soil. A well-known full-scale experiment of installation in layered soil was performed by Tjelta et al. [1986] using a large diameter trial test. As well the installation of 9 suction caissons for the Curlew FPSO (Alhayari et al. [1999]) and installation of 12 suction caissons for the GORM field (Sempere and Auvergne [1982]) were successful. All installations were performed in layered soil conditions (either stratified clay/silt and sand) and successful installation could have been achieved by the use...
of water-injection devices near the caisson tip. Application of these devices contributes to reduction of the effective stresses, by injecting water in front of the caisson tip. As a result the tip-resistance is 'artificially' decreased, which contributes to successful installation in complex soil conditions.

**Water injection devices**

Usage of water-injection-devices (WID) seems to be an efficient mitigating measure to install suction caissons in complex soil conditions. However, this measure consists of both water injection and extraction (suction), which is not efficient from an engineering point of view (Figure 2.3). The effectiveness of WIDs is nowadays not well understood, what raises the question to the use of this measure. During installation with WIDs the soil around the skirt tip is remoulded along the skirt. When high pressures are applied (jetting) the soil is flushed and the effective stresses reduce to zero. When pressures are more gradual (injection) the effective stresses reduce to a lesser extent (Cotter [2009]). Both conditions could influence the holding capacity of the caisson during lifetime in a negative way. However, this aspect is not taken into account within this thesis. Quantification of the effectiveness is desired, which in this thesis will be assessed by considering the plug stability during installation. This thesis only focuses on the installation effects of suction caissons without any contribution of WIDs.

![Figure 2.3](image)

**2.1.2 Seepage flow in sand**

In case of homogeneous coarse grained soils; i.e. sands, the installation of a suction caisson is achieved by the reduction of the tip resistance. At the tip of the skirt the flow gradient will be high due to piping-effects. As a result the grains will (partly) liquefy around the tip and reduce the effective stresses. This mechanism of partly liquefaction will occur on a very small scale around the caisson tip. Because the induced flow is towards the inside of the suction caisson, the liquefied grains will flow inside the caisson, which result in some plug heave after installation. The theoretical minimum heave to be expected is equal to the volume of the installed skirt (Appendix D.2).

The required installation forces in layered soil are lower than for jacked installation according to several authors (Watson et al. [2006], Senders [2008] and Cotter [2009]). Experiments of Watson et al. [2006] show that installation of suction caissons in layered soil still can be achieved in an economical way. This was concluded from the fact that the required suction force was less than predicted for jacked installation. As result of this, reduction due to seepage flow is one of the possible mechanisms that caused the lower installation resistances. This could imply that a reduction of the tip resistance...
and thus seepage flow did occur during installation. Senders [2008] describes this principle in detail, whereas in general two mechanisms are identified which could have occurred (Figure 2.4).

Mechanism I:
Cracking of clay layer, which induces the seepage flow to develop through the underlying sand. Depending on the soil properties cracking is likely to occur in relatively thin top layers, in relation to the caisson diameter (see Figure 2.4a).

Mechanism II:
Uplift of the entire (intact) inner clay plug. This uplift transforms the under pressure in the trapped gap to just below the sealing clay layer. This way seepage flow is induced by the transferred ‘suction’ pressure and some reduction of tip resistance will occur.

Senders [2008] found that during the beginning of the suction installation the top layer can be disturbed or fractured (mechanism I). The seepage in the sand layer can thereafter occur similar as for the uniform sand mechanism. This effect is expected to occur for relative thin clay overlying sand profiles (see Figure 2.4a).

Especially in case of mechanism II it was found that the uplift is not dependent on the stratification of the soil, e.g. uplift occurs as well for clay over sand profiles as for interbedded clay in sand strata. According to Tran [2005] the influence of stratification has been investigated for silt layers with permeability roughly 2 orders of magnitude lower than for the underlying sand that was used. Variable top layer thicknesses and interbedded thicknesses were investigated, for the non-plastic silty material. From the results, it showed a rapid drop in suction pressure for silt interbedded in sand layers. Interesting is that the same effect occurs for silt at depth 0,8 m as well as for 2,0 m below surface. This implies that no depth-dependent effect was encountered.

Cotter [2009] investigated the installation process for inclined clay profiles, which results in gradually seepage restrictions. However, as presented by many other authors (Tran et al. [2007], Senders [2008] and Senpere and Auvergne [1982]), the installation was still successful.

From the work of several authors investigated, it can be remarked that the stratification is related to the mechanism to be expected. However, this only is valid if the thickness of the layer is considered, e.g. relative thick impermeable (clay) layers have the tendency to lift up. Considering the location of flow-restricted layers, there is no depth-dependant effect encountered.

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2.1.3 Effects of penetration-rate

Regarding the installation aspects, several researchers found that possible plug uplift is a function of the installation-time (Senders [2008]). The effect of elastic plug heave (presented in Appendix D.2) is not rate-dependent and is excluded for rate-dependency effects. Fast suction installation implies a greater pressure difference of inner- and outer pressure, which influences the plug response during installation. Cotter [2009] found that for lower suction pressured installation, plug heave was observed in all cases. Watson et al. [2006] observed that plug uplift accelerates towards the final stages of penetration, which also implies that the magnitude of plug uplift is dependent on the applied suction.

The authors describe the uplift mechanism being dependant on the installation time, e.g. short installation time implies lower plug uplift. Mechanism I (cracking of the plug) is not considered in relation to the installation-time. It is expected that erosion will determine the time-dependent behaviour for this condition. From practical experiences it has been observed that longer installation time involves higher risks. In order to control the risks involved during installation (e.g. installation refusal), a high flow rate for the pumps is therefore desired.

2.1.4 Conclusions

From practical experiences about installation of suction caissons in layered soils, it was found that successful installation is achieved by the use of water-injection-devices. The theory about installation in sand overlaid by clay identified two mechanisms; cracking- or uplift of the clay plug. From both practical experiences by several researchers it was concluded that fast installation limits the plug uplift and is therefore desired to control the risks involved, such as installation refusal.

2.2 Initial phase: Self-weight penetration

This section presents the self-weight penetration phase for a suction caisson installation in layered soils. First homogeneous sand and clayey soils are considered by available prediction methods. The self-weight penetration depth \( h \) can be calculated by the resistance of skirt skin friction, end bearing and submerged weight of the caisson. The latter acts as the driving force for initial penetration. The depth of penetration can be calculated by solving the equation, which is dependent on the unknown parameter \( h \). The solution for this parameter equals the downward forces (submerged weight) with the encountered resistances (skirt skin friction and tip end bearing).

If the downward force of self-weight is higher than the sum of resistance forces, the caisson will penetrate. The existing procedures for predictions of penetration resistances can be divided into the following two categories;

**Effective stress-approach:** based on tip- and shaft resistance in proportion to the effective stresses.

Houlsby and Byrne [2005] and API [2000] provide common methods which calculate the resistances based on effective stresses. Only the Houlsby and Byrne-method presents an approach to predict the soil resistances in the suction-assisted phase. These calculations are rather extensive and therefore a simplified Houlsby and Byrne-method is adopted to calculate the soil resistances during penetration.

**CPT-approach:** based on tip and shaft resistance in proportion to cone resistance \( q_c \).

The DnV [1992] method is the most conventional method which calculates the soil resistances during self-weight penetration. However, the potential reduction during suction-assisted penetration is not
included. Andersen et al. [2008] presented a procedure to calculate the reduction considering seepage flow for dense sands, which also uses the CPT-data for predictions. A combination of both methods is used to describe the installation process of the suction caisson. In addition Senders [2008] described an approach for layered soil profiles, especially where sand is overlaid by clay.

Table 2.1 presents an overview of the methods and their applicability. A simplified Houlsby & Byrne-method as effective stress-approach and the DnV-method as CPT-approach are discussed in the next paragraphs.

<table>
<thead>
<tr>
<th>Approach</th>
<th>Self-weight penetration</th>
<th>Suction-assisted penetration</th>
<th>Sand/Clay/Both Layered soils</th>
<th>CPT/σ′v</th>
</tr>
</thead>
<tbody>
<tr>
<td>Houlsby and Byrne [2005]</td>
<td>yes</td>
<td>yes</td>
<td>Sand + Clay no</td>
<td>σ′v</td>
</tr>
<tr>
<td>API [2000]</td>
<td>yes</td>
<td>no</td>
<td>Sand + Clay no</td>
<td>CPT</td>
</tr>
<tr>
<td>DnV [1992]</td>
<td>yes</td>
<td>no</td>
<td>Sand + Clay no</td>
<td>CPT</td>
</tr>
<tr>
<td>Andersen et al. [2008]</td>
<td>yes</td>
<td>yes</td>
<td>Sand no</td>
<td>Both</td>
</tr>
<tr>
<td>Senders [2008]</td>
<td>yes</td>
<td>yes</td>
<td>Both yes</td>
<td>Both</td>
</tr>
<tr>
<td>Simplified H &amp; B</td>
<td>yes</td>
<td>yes</td>
<td>Both yes</td>
<td>σ′v</td>
</tr>
</tbody>
</table>

In case of layered soils the self-weight penetration depth can be calculated by extrapolating this principle for multiple layers. However, a transition in friction and end bearing resistance shall be considered for penetration through the interface of different layers.

### 2.2.1 Effective stress-approach

As stated previously, the Houlsby & Byrne-method is rather extensive to predict the soil resistances during penetration. The self-weight penetration phase of the suction caisson in the clay layer is calculated as the sum of skirt friction and the end bearing on the tip (see Figure 2.5).

\[ Q_{tot} = Q_{inside} + Q_{outside} + Q_{tip} \]  

If the undrained shear strength of the clay is considered to be uniform and adhesion factors of inside and outside are the same, the equation becomes less extensive (equation 2.2).

\[ Q_{tot} = Q_{inside} + Q_{outside} \]

The self-weight penetration depth in the clay \((h_c)\) can be calculated by assuming the submerged
weight \((W')\) equal to the penetration resistance \((Q_{\text{tot}})\). Undrained behaviour is applicable since no significant consolidation will occur during a typical installation time period.

\[
W' = (A_i + A_o)\alpha s_u h_c + (s_u N_c + \gamma_s' h_c)A_{\text{tip}}
\]  

From equation 2.2 it can be noted that the dimensionless adhesion factor is taken equal for inside and outside of the caisson in order to simplify the equation. The end bearing parameter is based on DnV [1992] and equal to 6.2 for shallow penetration and will increase up to 9 for deeper levels of penetrations (see Appendix B). If the submerged weight exceeds the soil resistance in the clay, the penetration will continue in the underlying sand (see Figure 2.5b). The self-weight penetration phase in sand can be calculated by the same approach, i.e. the penetration resistance consists of inner and outer skirt friction plus end bearing at the tip. However, drained behaviour is considered for the sandy material. The skirt friction can be calculated by a lateral earth pressure coefficient of \(K = 0.8\), which is used for the prototype cases in Andersen et al. [2008]. The value of \(K = 0.8\) is also recommended by API [2000] for the calculation of the drained shaft friction of open-ended unplugged piles.

A simplification of this method (Simplified H & B) ignores the difference in vertical effective stress for inside and outside the caisson, due to differences in area of influence \(\sigma_v' = \sigma_{vi}' = \sigma_{vo}'\), from Houlsby and Byrne [2005]). The angle of friction is estimated by \(\phi_c = 0.9 \phi_i\), where the internal friction angle is based on triaxial tests and drained conditions are considered for installation in sand. The resistances are calculated by;

\[
Q_{\text{inside}} = Q_{\text{outside}} = Q_{\text{skirt}} = \frac{1}{2} \pi DK \tan(\phi_c)\sigma_v' h_s
\]

\[
Q_{\text{tip}} = (\sigma_v' N_q + \gamma_s' \frac{t}{2} N_g)A_{\text{tip}}
\]

\[
W' = (D_i + D_o) \frac{1}{2} \pi K \tan(\phi_c)\sigma_v' h_s + (\sigma_v' N_q + \gamma_s' \frac{t}{2} N_g)A_{\text{tip}}
\]

2.2.2 CPT-approach

According to Senders and Randolph [2009] the cone penetration resistance is closely related to the measured resistance during self-weight penetration of a suction caisson. This relation relies on the similarities in penetration speed and dimension of the cone compared to the thickness of the caisson. Since the shape of the cone is different compared with the strip-shape of the caisson skirt, shape factors are introduced. This shape factor relates the cone resistance to the caisson resistance. For this reason the CPT-based method is considered to be reliable to predict self-weight penetration. The principle of this method is given by the DnV [1992], which is defined by;

\[
Q_{\text{tot}} = Q_{\text{inside}} + Q_{\text{outside}} + Q_{\text{tip}}
\]

\[
Q_{\text{outside}} = \pi D_o k_f \int_0^L q_c(z)dz
\]
Installation effects of suction caissons in non-standard soil conditions

\[ Q_{\text{inside}} = \pi D_i k_f \int_0^L q_c(z)dz \]  

(2.8)

\[ Q_{\text{tip}} = A_{\text{tip}} k_p q_c(L) \]  

(2.9)

Where the coefficient \( k_f \) varies between 0.001 (most probable) and 0.003 (highest expected), for \( k_p \), a factor of 0.3 to 0.6 is suggested. These values are based on North Sea silica sand, which is typical dense sand.

2.2.3 Penetration in layered soils

The self-weight penetration in layered soil can be calculated using both approaches. In case of the effective stress-approach, the equations are adjusted to estimate the total resistance in layered soils. The calculation principle of soil resistance in homogeneous sand is extended by the friction of the upper clay layer, if it is assumed that penetration through the top clay layer occurs.

\[ W' = (\alpha_o A_o + \alpha_i A_i) s_u h_c + (D_o + D_i) \frac{1}{2} \pi K tan(\phi_e) \sigma'_v h_s + (s_u N_c + \sigma'_v N_q + \gamma' s_\gamma N_\gamma) A_{\text{tip}} \]  

(2.10)

The total penetration depth \( h \) will be the depending variable to be calculated and is defined by; \( h = h_c + h_s \) (according to Figure 2.5b). A distinction between the penetration in clay or underlying sand can made, with the end bearing conditions:

\[ h \leq h_c : \quad N_c = 6.2 - 9 \quad N_q = 1 \quad N_\gamma = 0 \]

\[ h > h_c : \quad N_c = 0 \quad N_q > 1 \quad N_\gamma > 0 \]

In case of the CPT-approach, the penetration resistance can directly be related to the measured cone resistance. This is advantageous from an engineering point of view, since the measured soil resistance is related to a single factor, i.e. \( k_p \) and \( k_f \). Although these factors are empirical factors and site dependent, this is considered to be the most robust method.

2.2.4 Comparison of approaches

The differences between both approaches generally rely on the difference in input parameters. The effective stress-approach uses the effective soil stress and friction angle to calculate the soil resistance, by considering drained behaviour during installation. The CPT-approach directly relates the cone resistance to the predicted soil resistance. Disregarding the limited time consumption for interpretation and soil data needed, the main advantage of the CPT-approach is based on the penetration-rate. Similar soil response is obtained in terms of (un-)drained or partially drained behaviour. Additionally, limiting input parameters are required to predict the soil resistance for tip and skin friction (\( k_p \) or \( k_f \)).

In order to compare both approaches properly, similar input conditions are required. Thus the in-situ soil strength should be correlated to CPT-data, which is done according to Verrijt and van Baars [2007] in equation 2.11 and Chen and Juang [1996] in equation 2.12;

\[ q_{c,\text{clay}} = \sigma'_v + N_c s_u \]  

(2.11)

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\[ q_{c,\text{sand}} = \sigma'_v 0.260 e^{6.82 \tan(\phi')} \]  

(2.12)

These equations are used to compare the effective stress-approach and CPT-approach for a typical case. The ratios of friction and end bearing relative to the total resistance for increasing depth are plotted in Figure 2.6, where sand with \( \phi' = 40^\circ \) is overlaid by 3 m of clay with \( s_u = 25 \) kPa (uniform with depth). For the CPT-approach the \( k_f = 0.0015 \), \( k_p = 0.45 \) (most probable) as best-fit and \( k_f = 0.003 \), \( k_p = 0.6 \) (highest expected) are taken according to Senders and Randolph [2009]. The \( N_c \) parameter for the clay is calculated according to DnV [1992] (see Appendix B). From Figure 2.6 it can be seen that the used correlations agree reasonably well, for the two different approaches. Further details of the calculations are presented in Appendix A.3.

![Figure 2.6 Normalised contributions of shaft- and tip resistances for effective stress- and CPT approaches for a typical layered soil profile](image)

It can be noted in Figure 2.6 that at the initial stage of penetration (i.e. within the clay layer) the total resistance is governed by the shaft resistance (\( \text{shaft} \)). With increasing penetration in the underlying sand it can be seen that end bearing becomes the governing resistance (\( \text{tip} \)). Tip reduction due to seepage flow would then become an effective measure to reduce the total soil resistance in the underlying sand. Further discussion about seepage flow in the underlying sand is presented in the subsequent section (2.3) and Chapter 4.

### 2.2.5 Conclusions

In order to predict the soil resistance during the self-weight penetration phase limit state methods are applied. In general there are two options; effective stress- or CPT-approach. The difference between the methods relies on key-input parameters, such as friction angle and effective unit weight (effective stress-approach) or cone resistance and shape-factors (CPT-approach).

The Houlsby & Byrne method is based on effective stresses, which are calculated iteratively. In addition, this describes the stress changes (reductions) for suction-assisted penetration, which makes this approach useful. On the other hand the reduction of effective stresses are not applicable for layered soil conditions, thus some adjustments should be made. This is done by introducing a simplified Houlsby & Byrne method which is less extensive.

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From an engineering point of view, the CPT-approach is preferred because the relation to the measured resistance is directly related by a single factor. As well the CPT should encounter the same soil response as for the caisson during installation. Considering the penetration-rate, similar soil response is obtained in terms of (un-)drained or partially drained behaviour. Only the shape factor is different and should therefore be chosen properly. According to Senders [2008] and Andersen et al. [2008] it is possible to estimate the reduced stresses during the suction phase.

2.3 Second phase: Suction-assisted penetration

The suction-assisted penetration phase can be characterized by considering the amount of reduction in soil resistance from seepage flow in the underlying sand layer. In order to predict the tip (and inner-) reduction, an upper boundary for the applied underpressure is introduced, known as the 'critical suction'. This principle is explained below for clay, sand and layered soil profiles. Hereafter the reduction in soil resistance is assessed by the effective stress- and CPT-approaches.

2.3.1 Critical suction in layered soils

As the suction below the caisson lid increases, the pressure difference on the soil plug will increase. This gradient is limited to the 'critical suction', from which the effective stress becomes zero. For suction pressures higher than the critical suction, liquefaction is expected for sandy soils, while for homogeneous clays intact uplift of the plug ultimately occur. The critical suction represents the maximum suction that can be applied to ensure plug stability. This critical suction is well documented for homogeneous clays (Houlsby and Byrne [2005]) or sands (Andersen et al. [2008]; Senders [2008]), however not extensively for layered soils. Because of this, the critical suction is determined on the basis of relations for the critical suction for clay and sand. By combining these calculations, an expression for layered soils is obtained.

In clay

In homogeneous clay the critical suction is based on the reverse end bearing ($P_{\text{bearing}}$), shaft resistance ($P_{\text{friction}}$) and submerged weight of the clay plug ($P_{\text{weight}}$). Reverse end bearing is defined as the mobilized uplift resistance from end bearing in the short to medium term (Randolph and Gourvenec [2011]). Figure 2.7 presents the contributions of each component for a caisson with $D = 10$ m and $\alpha s_u = 8$ kPa. The parameter $z$ represents the thickness of the clay plug.

![Figure 2.7 Contributions of plug uplift in homogeneous clay](image-url)
In absolute terms the contribution of reverse end bearing is independent of the penetration depth for constant $s_u$ (uniform profile). Considering the total resistance for plug uplift, it can be seen that for increasing depth the relative contribution of reverse end bearing decreases for increasing penetration in the clay, by assuming a constant $N_c$-factor.

For layered soil profiles, it is assumed that the self-weight installation completely penetrated through the top clay layer and the contribution of reverse end bearing capacity cannot be taken into account (Senders [2008]). From Figure 2.7 it can be found that the frictional component will be lower than the self-weight component in that case. The self-weight component is not dependent on caisson dimensions and will be the governing component for uplift when $\frac{\gamma'c D}{\alpha_s u s} > 21$. The theory of Senders [2008] assumes effective unit weight of the clay plug, by considering low uplift rates and hence the submerged weight of the plug can be taken. The critical pressure for the clay plug is defined by the uplift criteria in equation 2.13:

$$S_{crit, clay} = \gamma'c z_{clay} + \frac{D_i}{4} \alpha_s u s_{clay}$$  \hspace{1cm} (2.13)

### In sand

In case of homogeneous sand, the critical pressure is found by the 'exit' gradient and seepage length. The limiting factor is the gradient across the upper surface of the plug (bed level), despite higher hydraulic gradients are expected at the pile tip. However, the soil around the pile tip is confined by non-liquefied soil and thus liquefaction will intervene first at bed level. Therefore the gradient at pile tip level is not the limiting factor, but the exit-gradient at bed level. The critical pressure depends on the effective weight of the soil and empirical relations for the seepage length:

$$S_{crit, sand} = s \gamma_w i_{crit} = \gamma'c s$$  \hspace{1cm} (2.14)

**Figure 2.8**  Equi-potential lines for homogeneous sands (Figure 6 from Senders [2008])

For the underlying sand in a layered soil profile, this seepage length is described by Senders [2008]. Figure 2.8 presents the development of pore pressures when suction is applied in the caisson. For layered soil conditions the seepage length will increase due to the impermeable top layer of clay as can be noted in Figure 2.8. Senders [2008] presented an empirical correlation for the seepage length, based on the condition with an impermeable top layer by;

$$\frac{s}{h_s} = 1 + 0.3 \left( \frac{h_s}{D} \right)^{-0.85}$$  \hspace{1cm} (2.15)
If equations 2.14 and 2.15 are combined, an expression for the critical pressure is given by:

\[
S_{\text{crit, sand}} = \gamma_s' h_s \left[ 1 + 0.3 \left( \frac{h_s}{D} \right)^{-0.85} \right]
\] (2.16)

The methods by Andersen et al. [2008] and Houlsby and Byrne [2005] also use a critical suction pressure for homogeneous sands. The normalized critical suction according to the researchers for homogeneous sands is presented in Figure 2.9. The example presents a 3 m clay layer overlying sand at \(z/D = 0.3\). It can be remarked that for penetration in the sand, the critical suction estimate for homogeneous sand (dashed lines) tend to zero. This is due to 'shallow' penetration in the sand and in theory is more favourable to piping failure. However, since the sand is overlaid by clay, piping failure is less likely to occur. A correction for the Houlsby and Andersen-methods is therefore desired, which accounts for the depth effects of penetration. This is done by the author by taking the Senders-solution for low penetration as a starting point. In fact the Houlsby and Andersen-methods are shifted horizontally towards the Senders-solution for the initial penetration in the sand.

![Figure 2.9 Critical suction for sand overlaid by 3 m clay](image)

From Figures 2.8 and 2.9 it can be noted that the critical suction for layered soils is higher than homogeneous sands, due to the increased seepage length. It can also be remarked that the corrections for critical suction of the Houlsby and Andersen-methods show a close relation with the Senders-method for layered soils.

### 2.3.2 Reduction of soil resistance

In order to ensure tip reduction at the caisson tip a seepage flow should be induced. This can be achieved by applying suction within the caisson, which will eventually induce a seepage flow (due to plug cracking or uplift). The seepage flow around the tip will reduce the soil resistance and thus penetration of the caisson will occur. This required suction reduces the tip resistance and is defined by \(S_{\text{req}}\). The relation between the applied suction and the reduced tip resistance is described by several authors.

The previously discussed methods of Houlsby & Byrne and NGI both assume installation in homogeneous sandy material. Only Senders [2008] has presented a calculation method for layered soils. The latter method is appropriate for sand overlaid by clay, however all methods are evaluated...
to estimate the reduction in tip resistance due to suction. This is done by implementing a corrected
critical suction for the methods which are used for homogeneous sand conditions (Figure 2.9). In
order to calculate the required suction for installation, the critical suction is taken as upper bound
since the effective stress becomes zero in that case. Calculation methods to determine the reduced soil
resistances are presented in Appendix A.1. The soil resistances without reduction for the previously
discussed methods are presented in Appendix A.3.

Houlsby & Byrne (2005)

According to the simplified method (see Appendix A.3), the tip reduction due to suction is calculated
by taking a factor for the pore pressure into account. The reduction factor for the pore pressures
at the caisson tip for various L/D-ratios takes into account 3D-effects (Houlsby and Byrne [2005]).
For the underlying sand the penetration is given by $h_s$. The dimensionless pore pressure ($a_1$) is
calculated as follows;

$$a_1 = c_0 - c_1 \left[ 1 - \exp \left( - \frac{h_s}{c_2 D} \right) \right] \quad (2.17)$$

Where $c_0 = 0.45$, $c_1 = 0.36$ and $c_2 = 0.48$. As well the effect of inner and outer permeability can
be taken into account, by an adjusted pore pressure factor $a(z)$.

$$a(z) = \frac{k_f a_1}{(1 - a_1) + a_1 k_f}; \quad \text{with} \quad k_f = \frac{k_i}{k_o} \quad (2.18)$$

The final equation is given by equation 2.19, where each component is reduced by a single factor,
varying with penetration depth. When $S_{crit}$ is filled in as required suction, the remaining variable is
$S_{red}$. The equation can then be solved and the minimal required suction can be determined.

$$W' + S_{req} A_{base} = \alpha_{out}(z) Q_{outside} + \alpha_{in}(z) Q_{inside} + \alpha_{tip}(z) Q_{tip} \quad (2.19)$$

$$\alpha_{out}(z) = [1 + a(z)] S_{red} A_o \quad (2.20)$$

$$\alpha_{in}(z) = [1 - a(z)] S_{red} A_i \quad (2.21)$$

$$\alpha_{tip}(z) = f(\alpha_{out}, \alpha_{in}) \quad (2.22)$$

Andersen et al. (2008)

The NGI-method for homogeneous dense sands takes into account a ’reduction factor’ for the total
measured resistance by the CPT. This reduction factor depends on the caisson diameter, penetration
depth, soil permeability and critical suction. The normalized critical suction is presented as critical
suction number by $S_{crit,NGI} = S_{crit}/\gamma z$, which is calculated according to equations in Appendix A.2.
The NGI-method includes a parameter $a(z)$, which varies with the penetration depth. However, calculating the required suction ($S_{req}$) is rather complex. Andersen assumes that the reduction-factor is applied to all soil resistances (inner-, outer- and tip resistance). Based on the Senders’-method only the inner- and tip resistance will linearly reduce for layered soil conditions. Therefore the Andersen-method is adjusted by taking $a(z) = 1$ for all $z$ and no reduction of the outer soil resistance is considered.

Senders (2008)

As assumed by Senders [2008] the inner friction and tip resistance will reduce linearly from the ‘initial’ resistance due to push-in installation at zero suction, to zero when suction reaches $S_{crit, sand}$. The critical suction of Senders [2008] is similar to equation 2.16.

$$W' + S_{req} A_{base} = Q_{outside} \left(1 - \frac{S_{req}}{S_{crit, sand}}\right) [Q_{inside} + Q_{tip}]$$ (2.24)

2.3.3 Comparison of methods

All methods are compared in Figure 2.10a, where it can be noted that the suction requirement reduces for the flow condition. The suction requirement is calculated by the critical suction for layered soils. The CPT-approaches of Andersen et al. [2008] and Senders [2008] both incorporate parameters for ‘most probable’ (MP) and ‘highest expected’ (HE). These are also presented in Figure 2.10a, where it can be noted that for the MP-condition the self-weight penetration is approximate 2 m.
For the remaining calculation methods it can be found that the self-weight penetration is lower (i.e. 1.5 m) due to more conservative input-parameters (higher $k_p$ and $k_f$ - values). From Figure 2.10b it can be noted that the CPT-based methods show a reduction in the range of 25 - 50% during the initial penetration in the sand. The Houlsby-method shows a gradual reduction for initial penetration, but the results approaches the CPT-based results after 2 m penetration. The difference with the Houlsby-method is likely caused by the limitations of the iterative calculation-procedure and the adopted simplification of the approach. Within the first meters of penetration the reduction is higher than for deeper levels of penetration for CPT-based approaches. This is caused by the relative low critical suction that is applicable for shallow depths.

A remark about the feasibility of both approaches can be made, regarding the limitations of the chosen calculation methods. The reduced penetration resistances are based on boundary value level (critical suction), however it is uncertain whether the seepage flow is generated due to plug cracking or uplift. Further investigation into the mechanisms of plug cracking and plug uplift are further investigated both experimentally (Chapter 3) and analytically (Chapter 4).

### 2.3.4 Conclusions

It can be concluded that the approaches for homogeneous sands (Houlsby and Byrne [2005] and Andersen et al. [2008]) can be adjusted in order to calculate the resistances in layered soils, where sand is overlaid by clay. The most important adjustment was to correct the critical suction for the homogeneous sandy material. The critical suction was horizontally translated for shallow penetration in the sand, according to the solution of Senders [2008].

In the analysis (see Figure 2.10) it was found that during the suction-assisted penetration the total soil resistance is reduced, but that this reduction gradually decreases with increasing penetration depth. The reduction at greater depths is roughly 25 - 50% compared with the initial resistance in the sand. It should be remarked that the reduced soil resistances are based on the assumption of generated seepage flow, but the exact mechanism should be further assessed (either plug cracking or uplift).

### 2.4 Conclusions

From practical experiences about installation of suction caissons in layered soils, it was found that installation is achieved by the use of water-injection-devices. The theory about installation in sand overlaid by clay identified two mechanisms; cracking- or uplift of the clay plug. From both practical experiences and theory from literature it can be concluded that fast installation is desired to limit plug uplift and control the risks involved.

In order to predict the soil resistance during the self-weight penetration phase, limit state methods are applied. In general there are two options; effective stress- or CPT-approach. The difference between the methods relies on key-input parameters, such as friction angle and effective unit weight (effective stress-approach) or cone resistance (CPT-approach).

From an engineering point of view, the CPT-approach is preferred because the relation to the measured resistance is directly related by a single factor. As well the CPT should encounter the same soil response (drained, partially drained) as for the caisson during installation. Only the shape factor is different and should therefore be chosen properly. According to Senders [2008] and Andersen et al. [2008] it is possible to estimate the reduced stresses during the suction assisted phase.

The Houlsby & Byrne method is based on effective stresses, which are calculated in an iteratively way. In addition, this describes the stress changes (reductions) for suction-assisted penetration, which makes this approach useful. On the other hand the effective stresses are not applicable for layered.
soil conditions, thus some adjustments should be made. This is done by introducing a simplified Houlsby & Byrne method which is less extensive.

Furthermore, it can be concluded that the approaches for homogeneous sands (Houlsby and Byrne [2005] and Andersen et al. [2008]) can be adjusted in order to calculate the resistances in layered soils, where sand is overlaid by clay. The most important adjustment was to correct the critical suction for the homogeneous sandy material. The critical suction was horizontally translated for shallow penetration in the sand, according to the solution of Senders [2008].

It was found that during the suction-assisted penetration the total soil resistance is reduced, but that this reduction gradually decreases with increasing penetration depth. The reduction at greater depths is roughly 25 - 50% compared with the initial resistance in the sand. It should be remarked that the reduced soil resistances are based on the assumption of generated seepage flow, but the exact mechanism should be further assessed (either plug cracking or plug uplift). The subsequent Chapter briefly discusses the plug response during installation (Chapter 3).
Chapter 3

Plug response during installation

This Chapter discusses the literature about plug response during installation of suction caissons (section 3.1). Since there is limited literature about layered soils (especially sand overlaid by clay) references about installations in homogeneous sands and clays are considered as well. Based on the literature review on the plug response during installation, some experiments on plug stability have been executed at the Geotechnical Laboratory of Delft University of Technology. Information about the experiments on plug stability can be found in section 3.2. Finally the conclusions of plug response during installation are presented in section 3.3.

3.1 Theory from literature

Senders [2008] describes that seepage flow can be generated in sand overlaid by clay, by cracking or plug uplift of the clay layer. The exact failure mechanism is not fully understood, therefore both mechanisms are considered for further research. In the subsequent analysis plug uplift and plug cracking is described.

3.1.1 Theory of plug uplift

The potential for plug uplift failure is dependant on the applied differential pressure acting on the plug in relation to the resistance of the plug. Several side-effects could influence the pressure development, which will also influence the plug uplift behaviour. For instance, the restricted seepage flow in case of sand overlaid by clay will be influenced by the surrounding overlying clay properties. In order to avoid complex correlations, the uplift failure is considered to be primarily dependent on the pressure acting on the plug. Figure 3.1 presents a schematization of the parameters that determine the plug uplift criteria.

![Figure 3.1 Schematization of plug uplift parameters](image-url)
From the theory of Senders [2008] the plug uplift mechanism is described with equation 3.1. This equation describes the limiting differential pressure for intact plug uplift, which takes into account self-weight and skirt friction to overcome, prior to uplift. Hence it can be noted that contribution of reverse end bearing is not included.

\[ p_{\text{plug}} = \left( \gamma' c + \frac{4}{D_i} \alpha s_u \right) z_{\text{clay}} \]  

(3.1)

Cotter [2009] describes that plug uplift is also dependant on the intact shear resistance of the 'underlying' clay. In that case the plug will lift up prior to full penetration of the clay layer. The required uplift pressure depends on the depth of penetration into the clay layer (equation 3.2) and properties of the clay. In case of full penetration into the clay layer \((h = z_{\text{clay}})\), the equation becomes equal to equation 3.1 and intact uplift is assumed.

\[ p_{\text{plug}} = \frac{4}{D_i} \left( s_u D_i \left( h (\alpha - 1) + z_{\text{clay}} \right) \gamma' c z_{\text{clay}} + \frac{D_i^2}{4} \right) \]  

(3.2)

The driven values to resist plug uplift are the self-weight and inner skirt friction of the clay in case of full penetration. From equation 3.1 it can be seen that uplift resistance will be higher when the variables on the right side of the equation increase. Except for the inner diameter \((D_i)\) it holds that higher uplift resistance will be achieved by a smaller \(D_i\). A comparison of both equations is presented in Figure 3.2. It can be remarked that for relative thick plugs, the ratio \(D/z\) is small and the normalised suction for uplift will be higher.

![Figure 3.2 Plug failure according to Senders [2008] and Cotter [2009]](image-url)

**3.1.2 Theory of plug cracking**

The cracking mechanism is described by Senders [2008] by assuming seepage flow along the skirt of the suction caisson (piping) or cracking in the middle of the clay. Failure of the piping principle is described by shear failure at the edges of the clay plug. Piping along the clay skirt will occur when the applied pressure will be higher than the reduced shear strength. The research of Senders [2008] does not focus on the failure mechanisms for plug cracking, therefore an estimate of plug cracking is developed within the framework of this thesis. A schematization of plug cracking is presented in Figure 3.3.

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The phenomenon of cracking can be estimated by calculating the bending moment of the clay plug. The calculation is simplified by assuming linear elastic behaviour of the clay plug and plate bending with clamped- or hinged edges. The difference in fixation provides a range of solutions for cracking failure. The resistance for cracking is calculated using the undrained shear strength as tensile strength for clay according to Thusyanthan et al. [2007]. Considering the applied cracking pressure as uniform distributed load, the required cracking pressure range is presented in equation 3.3. A detailed elaboration is presented in Appendix C.

\[
\frac{64s_u z_{clay}^2}{\pi D_i^3(3 + \nu)} < p_{crack} < \frac{64s_u z_{clay}^2}{\pi D_i^3(1 + \nu)}
\]  

(3.3)

Similar to equation 3.1, it can be seen that increasing self-weight and undrained shear strength lead to higher pressures for cracking failure. However, higher pressures will be achieved by smaller inner diameter to the power 3. The relation with inner diameter is much higher in this case, which makes the driving failure pressure be dependent on the dimensions of the suction caisson. A comparison of plug uplift and plug cracking is presented in Figure 3.4.

From both mechanisms it can be found that the dimensions of the plug (i.e. the suction caisson) determine the driving failure mechanism. Within this framework some series of tests were executed in the laboratory of Delft University of Technology. The results are presented in section 3.2.
3.1.3 Reference data of plug response

Considering the two mechanisms described, it can be remarked that the dimensions depend whether the plug will lift or crack during the suction process. According to several references (see Figure 3.5) it has been found that plug uplift did occur in cases for relative thick plugs. From a practical point of view the range of reference data is limited to $D/z < 6$, since thinner clay layers are not considered in practice. In order to analyse the data, an upper- and lower boundary of the undrained shear strength is chosen; i.e. lower boundary of 5 kPa and upper boundary of 50 kPa.

![Figure 3.5 Plug failure for several reference-data (data-labels represent $s_u$ [kPa])](image)

Figure 3.5 shows a reasonable bandwidth about reference-data from literature. Some comments about the data-point can be made:

- The test performed by Allersma et al. [2001] showed that plug uplift occurred for a Kaolin clay with $s_u = 1.7$ kPa.

- Cotter [2009] performed several tests on layered soil strata which consist of sand overlaid by clay. The shear strength was briefly determined (5.5 kPa) and model-dimensions are presented as well. The left orange data-point didn’t show any plug uplift, which corresponds to plug failure boundary.

- The blue data point (Tran et al. [2007]) is well reported, but these tests were performed with silt materials. Uplift failure did occur, but since silt material is more favourable for scouring-effects compared with clay, this data point is less guiding.

- The grey data point (Watson et al. [2006]) presents the data reported in Senders [2008], which briefly described the plug uplift mechanism. However, the undrained shear strength of the Kaolin-clay used for the centrifuge modelling test was not presented. If a $s_u$ of 2 - 4 kPa is assumed (Burns et al. [2010]), the data point is in accordance to the given boundaries.

From all reference data considered, it can be concluded that plug uplift failure is in accordance with the theory of Senders [2008]. On the other hand, there’s no reference data present about possible cracking failure, since $D/z > 6$ is not found in practice. In the framework of this, better understanding about the failure mechanisms involved was desired. Therefore some small scale tests were set-up in the laboratory of Delft University of Technology, whether to check this hypothesis. A description about the test is presented in section 3.2.
3.1.4 Conclusions

From literature it can be found that plug uplift is the major failure mechanism to be expected in all cases. Failure mechanism of cracking has been described by Senders [2008], but has not been investigated for varying dimensions of suction caissons. By considering clamped and hinged edges of the plug-disk, a range of possible cracking failure mechanisms can be found. However, verification of this theory is desired which can be done by some basic experiments on plug cracking. The purpose of the tests is to investigate the failure mechanisms that occur for different dimensions. As a result of this, it is possible to determine the transition point or range of failure behaviour, which might be interesting for the industry. More details about the experimental analysis are presented in the subsequent section.

3.2 Experiments on plug response

As described in the previous chapters some basic tests are recommended to check the hypothesis about cracking failure for relative thin plugs. First the possibility of scaled modelling is investigated, followed by a test set-up. In total 4 series of tests were conducted, whereas each series consists of two tests.

3.2.1 Scaling laws

Considering available time and budget it was chosen to perform basic 1G-experiments in the laboratory of Delft University of Technology. The purpose of the tests is to verify the predictions of the model and to check whether cracking or plug uplift will occur. Both mechanisms could behave differently in scaled circumstances; therefore the scaling factors have to be implemented (Table 3.1).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Scaling factor (model/prototype)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear dimension ((z_{\text{clay}}) and (D_i))</td>
<td>(1/n)</td>
</tr>
<tr>
<td>Undrained shear strength ((s_u))</td>
<td>1</td>
</tr>
<tr>
<td>Submerged unit weight of clay ((\gamma'_c))</td>
<td>1</td>
</tr>
<tr>
<td>Suction pressure ((p))</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 3.1 Scaling laws for 1G testing

Adding these scaling factors into the equations found in previous section, results in:

\[
p_{\text{plug}} = \gamma'_c n z_{\text{clay}} + \frac{4}{n D_i} \alpha s_u n z_{\text{clay}} \tag{3.4}
\]

\[
\frac{64 s_u n^2 z_{\text{clay}}^2}{\pi n^3 D_i^3 (3 + \nu)} < p_{\text{crack}} < \frac{64 s_u n^2 z_{\text{clay}}^2}{\pi n^3 D_i^3 (1 + \nu)} \tag{3.5}
\]

It can be seen that both mechanisms are not scaled by pressure or undrained shear strength (equation 3.4 and 3.5). This implies that the scaling is dependent on the dimensions of the caisson (or plug) and therefore a fundamental difference in failure mechanism can be investigated, i.e. plug uplift or cracking. The difference in failure mechanism will be described by a dimension analysis, depending on the mobilized shear strength and suction pressure. As a result of this, it might be concluded that scaled model tests can be executed at 1G for these mechanisms in clay. One should note that for the underlying sand, the calculations of effective stresses don’t scale with the same rates and thus are not considered.

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Parameter determination

Allersma et al. [2001] tested a layered soil profile, consisting of sand overlying by clay and performed installation of a suction caisson using an oscillating pressure inside the caisson. In order to verify this theory, similar material properties were taken into account. Taking the prototype-data of Allersma et al. [2001] with the same Kaolin-clay, the desired undrained shear strength is 10 kPa. A distinction of uplift or cracking failure was made by taking $D/z$-ratios far off the transition range $6 < D/z < 10$ (see Figure 3.4). Plug uplift failure was tested at lower ratios ($D/z \approx 6$) and cracking failure at higher ratios ($D/z > 20$). The latter ratio (cracking) is an extreme value from a practical point of view and will be discussed later. The test apparatus has an inner diameter ($D_i$) = 190 mm and therefore determines the $D/z$-ratio for crack or uplift failure (Table 3.2).

<table>
<thead>
<tr>
<th>Table 3.2</th>
<th>Model properties</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Allersma et al. [2001]</strong></td>
<td><strong>Model tests</strong></td>
</tr>
<tr>
<td><strong>Prototype</strong></td>
<td><strong>Ng scale (50 g)</strong></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Diameter ($D_i$)</td>
<td>3 m</td>
</tr>
<tr>
<td>$s_u$ [kPa]</td>
<td>23 (K122)</td>
</tr>
<tr>
<td></td>
<td>10 (K147)</td>
</tr>
<tr>
<td></td>
<td>1.7 (Kaolin)</td>
</tr>
<tr>
<td>Thickness ($z_{clay}$)</td>
<td>1.7 m*</td>
</tr>
<tr>
<td>Ratio $D/z$ [-]</td>
<td>1.76*</td>
</tr>
</tbody>
</table>

* = Back-calculated

Using sensitivity analyses for $s_u$, $\gamma_c'$, $\alpha$ and $\nu$, the upper and lower boundaries were determined for layer thickness of the clay. The values of layer thickness in Table 3.2 are used as boundaries for the experiments.

Clay preparation

From Table 3.2 it can be remarked that the undrained shear strength of clay is 6.1 kPa, which is rather lower than the desired 10 kPa. Assuming sufficient time to obtain $U > 99\%$ as degree of consolidation, the shear strength of the clay only depends on the preloading stress prior to consolidation. The limiting factor on the shear strength was given by the weight capacity of the consolidation apparatus (limited to 75 kg max). Therefore the maximum shear strength to be created was limited. The actual undrained shear strength was determined by the Vane Test according to BS 1377:part 7:1990. A series of 3 tests (within a range of 5%) showed that the average peak strength of the clay is 6.1 kPa. Since for all tests the same conditions were adopted (preloading stress and sufficient consolidation time), it can be assumed that the undrained shear strength is the same for each test.

3.2.2 Test set-up

The purpose of the tests is to identify the possible transition point or range of the cracking failure for high $D/z$-ratios. This is done by taking $D/z$-ratios far off the predicted transition point. First a series of observational tests is executed to verify the zone of $D/z$ for plug uplift or cracking. Thereafter the testing-procedure is optimized and data has been collected for further analysis.

Execution of tests

In the first phase the Kaolin-clay slurry was consolidated by two-way drainage and sufficient pre-consolidation surcharge. Two-way drainage is established by opening the valve to the water column (see Figure 3.6). Prior to testing, the water on top of the clay is extracted to perform observational tests. The level of the water column is kept equal to the level of the saturated clay layer.
Each transparent box filled with sand/clay material is connected to a water column of 1 m height, which is filled with deoxidized water. As well this water column can be lowered by opening a valve, to equal both water levels within the soil-box and water column. The principle for testing the stability of the clay plug for uplift or cracking failure consist of increasing the hydraulic pressure below the clay layer, by increasing the height of the water column.

Optionally, the valve in between soil-box and water column can be closed, to ensure a larger differential pressure that initiates more sudden. Once the clay-slurry was consolidated, the preloading was removed and increasing differential pressure made the clay samples crack or uplift.

A schematization of the test-apparatus is presented in Figure 3.6b, including dimensions. In Figure 3.6a the schematization of the the soil-box is shown, which consist of a permeable material overlaid by the consolidated clay with height $z_{clay}$. The hydraulic gradient within the soil-box is concentrated in the centre of the testing cylinder. The porous filling material will spread this concentrated flow, ensuring a uniform distributed pressure below the clay layer.
Pressure distributions

Pressurization of the clay layer is achieved by increasing the hydraulic head acting below the clay, which is done by filling the water column. The difference in water table of the soil-box and water column is defined by the water table difference ($\Delta h_{wt}$). Depending on the magnitude of $\Delta h_{wt}$, there can be high or low pressures subjected to the clay sample. Depending on the conditions of a large or small $\Delta h_{wt}$, the pressures on the clay are calculated differently;

**Large difference condition**

A large difference in water table causes a high 'potential' pressure on the clay sample. However, the water inflow into the soil-box is limited by the dimensions of the connecting tube. This means that the pressurization of the clay is limited by the inflow-rate of the tube. Thus the pressure-rate on the clay is not given by the water column, but depends on the maximum inflow-rate of the tube (for large $\Delta h_{wc}$). In order to apply a large $\Delta h_{wc}$ on the clay sample, the valve is closed while the water table of the water column is filled.

**Small difference condition**

In case of a small difference between the water tables, the differential pressure on the clay is low. There is more time required to level both water tables, but the flow-restriction of the connecting tube is not applicable anymore. The pressure on the clay is now given by the actual height of the water column. In order to make a distinction between the two conditions, the large difference holds for the case when $\Delta h_{wt} > 30$ centimetres. Below this value is the condition considered to be a small difference (see Figure 3.7).

![Figure 3.7 System-permeability test without clay](image-url)
3.2.3 Results

This section presents the results of the tests performed at the Delft University of Technology laboratory of Geo-Engineering. The first test series represent the observational tests. In the subsequent analysis the results of series 2 - 4 are presented.

Test 1 Observational test

For both thin and thick clay plugs the pressure of the water column was increased stepwise. In both test series the rate of pressure increase was low. For the thick clay plug (Figure 3.8 - left series) it was clearly observed that plug uplift occurs, while the clay plug stays intact during uplift. During uplift there was no leakage observed on top of the clay plug.

![Initial phase](image1)

![Initial phase](image2)

![Uplift of 2 mm](image3)

![Uplift of 0.5 mm](image4)

![Uplift of 6 mm](image5)

![Uplift of 1 mm](image6)

![Uplift of 12 mm](image7)

![Uplift of 2 mm](image8)

**Figure 3.8** Uplift of clay for relative thick (left series) and thin (right series) clay

In the case of a thin clay layer (Figure 3.8 - right series) uplift of the clay layer was observed as well due to gradual increasing pressure. However, after further increase of the water table in the water column (high $\Delta h_{\text{wat}}$), the plug starts to bend with approximate 5 mm deflection in the centre of the plug. The convex-shaped plug moved further upwards until cracking occurred due to shear forces (see Figure 3.9). The cracking causes a minor leakage, since it didn’t cause an increased pressure drop of the water column.

The first test-series showed plug uplift for both high and low D/z-ratios, but only for gradual increasing pressure. The locations of the cracks indicate local shear failure of the clay layer due to...
the mobilized pressure. Despite the cracking observed, still the initial failure mechanism for both clay thicknesses was uplift.

**Test 2**

In the second test series, a plastic membrane was used in order to create a high permeable base. By the consolidation of the clay-slurry, it showed that the plastic membrane was compressed more than 50%. Once the top-load was removed, the elastic de-compression of the plastic membrane made the clay lift up. Once the membrane was in equilibrium again, small pressure increments lead to uplift of the clay plug, possibly by the mobilized remoulded shear strength along the skirt. In the end no reliable data could be implemented by this test.

Despite there was no reliable data obtained, still some observations were made during testing. For a gradual increasing water table, it showed that the clay plug lifted upwards. While the water table of the water column was further increased, the clay plug cracked when \( \Delta h_{\text{wt}} \) was large. This shows that the rate-effect of pressurization also depends which failure mechanism could be expected.

**Test 3 and 4**

These test series consist of a more solid, but still highly permeably underlying material (gravel), overlaid by clay with the same properties as mentioned before. In order to avoid the clay penetrating into the gravel during consolidation, a geo-membrane is used to separate the clay from the gravel. For these test series the valve to the soil-box was closed and the water column was filled till \( \Delta h_{\text{wt}} \approx 70 \) cm (large difference condition). During the second test (test 4a), it turned out that the geo-membrane contributes to entire uplift of the clay layer. Simultaneously, leakage started along the skirts, due to uneven uplift of the plug.

**Figure 3.9** Top view of cracks with minor leakage (≈ 1 % of cracks)

**Figure 3.10** Top view of cracks for test 3 (left) and 4 (right)
In the first and third test the clay didn’t lift up, but starts leaking once the high differential pressure was initiated. In all test the clay cracked just before the water column dropped for 5 centimetres. Prior to the cracking process bulging of the clay was observed in test 3 and 4b. For all tests it was observed that leakage starts at the skirts of the clay, followed by some cracks in the middle of the clay as indicated in Figure 3.10. This can be justified by the reduced adhesion of the clay-perspex interface and the maximum bending moments, which indicates the weakest points of the clay plug.

**Back-calculations**

In case of the third test series the water table of the water column and time are accurately measured. The data from the measurements (time and water table heights) are used to back-calculate the pressure on the clay during failure. To do this accurately, a lower-/upper boundary approach has been chosen, which is presented below;

- **Lower boundary: System permeability**

Prior to the tests on the clay layers, the time required to fill the soil-box by opening the valve of the water column was measured (Empty in Figure 3.11). This could be regarded as the 'system-permeability'. This system-permeability is used as a reference, to compare with the system permeability during testing (With clay in Figure 3.11). For both situations (reference and testing) it holds that the water table of the water column was set to 0.90 cm, prior to opening of the valve.

![Figure 3.11 Reference permeability for test 3](image)

In case of the soil-box being empty (water only), the increase of the water table in time is measured. As mentioned in previous section, the inflow-rate depends on the dimensions of the tube (large difference condition). Without any resistance, the increase of water table is given by Empty in Figure 3.11. During testing, the increase of water table is 'blocked' by the clay layer, which causes more resistance. As a result the pressure drop in the water column is 'delayed'. However, the maximum pressure-rate stays similar to the empty situation, so the equivalent pressure of cracking can found by taking the reference $\Delta h_{box}$ at time of failure. The value of $\Delta h_{box}$ at failure can then be back-calculated to the actual pressure at failure. The time required to cracking failure is recorded (Video in Figure 3.11) and an intermediate data point has been added when cracking started.
• Upper boundary: *Volume continuity*

The pressure increment on the clay can also be calculated by taking volume continuity into account. All tests have been executed with deoxidized water and air capture was avoided, which contributes to straightforward continuity calculations. Therefore the extracted volume of water in the water column ($V_{wc}$) will be equal to the increased volume of water in the soil-box ($V_{box}$). The effect of evaporation is assumed to be disregarded. While the water table in the water column is lowered, the equivalent displaced volume needs to be transferred to the soil-box, which causes a pressure increment below the clay layer. In case of a 5 cm drop an increment of 0.07 kPa in the soil-box is to be expected (0.68 cm). Since all tests failed (i.e. uplifted or cracked) just before the first measurement-point at 0.85 cm, this estimate will be the upper boundary of the test results.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Lower boundary [kPa]</th>
<th>Upper boundary [kPa]</th>
<th>Range [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.055</td>
<td>0.068</td>
<td>0.013</td>
</tr>
<tr>
<td>4a</td>
<td>0.095</td>
<td>0.068</td>
<td>0.027</td>
</tr>
<tr>
<td>4b</td>
<td>0.120</td>
<td>0.068</td>
<td>0.052</td>
</tr>
</tbody>
</table>

As can be found in Table 3.3, the ranges of the lower and upper boundaries are very small. For this reason only the range of test 4 is taken into account for this approach. From Figure 3.12 it can be found that the test-data are in accordance with the cracking predictions. It should be noted that the horizontal axis is bounded by $D/z > 20$. Further discussion about the results is presented in the next section.

![Figure 3.12](image)

**Figure 3.12** Test results of high D/z-ratio compared with model (including bandwith)

### 3.2.4 Conclusions & discussion

From estimations of video-recordings, a back-calculation of the observational tests (test 1) can be made. From the 3 tests that were implemented for relative high D/z-ratios, the measurements are back-calculated to compare with the model. All test results are presented in Figure 3.13 with a full range of D/z-ratio.

It can be concluded that the failure mechanism is indeed dependant on the dimensions, since a clear distinction in required pressures was measured. For relative thin clay plugs the tendency of failure is cracking, while for thicker plugs uplift occurred. The first test series (test 1) encountered plug uplift for both D/z-ratios, which fit to the plug failure prediction in Figure 3.13. In case of test
1a and 1b-plug, the differential pressure was low (small difference condition), which made the plug lift upwards in both cases. In test 1b-crack, 3 and 4 the large difference condition was applied, which resulted in cracking failure in all cases. As the transition range is $6 < D/z < 10$, it showed that for relative high $D/z$-ratios cracking was the governing failure mechanism, which is in accordance with the predictions.

However, in case of test 1b it can be seen that for a high $D/z$-ratio (test 1b) still uplift occurred which is close to the prediction of plug uplift failure. This data was obtained while the pressure-rate on the clay was low (small difference condition). But when the condition on the same plug was adjusted to a large difference condition, the failure mechanism changed as cracking was observed. This change of failure mechanism was also observed for test 2, which did not provide reliable data due to elastic deformations of the underlying material during consolidation.

From this it can be concluded that the rate-effect of pressurization also depends on which failure mechanism could be expected. Even for high $D/z$-ratios it was found that uplift occurred, but only in case of small pressure differences.

**Idealized boundary conditions**

In general it can be remarked that the calculation model is strongly idealised, which also includes idealised boundary conditions (i.e. horizontal layering, uniform shear strength of clay, uniform thickness of clay layer and fully saturated clay and underlying material). In case of plug cracking, the assumption about clamped or hinged edges was made to provide a range of solutions. The calculation is based on static mechanical loading on a circular plate. Because the clay is a cohesive material, the assumption of plate bending could be made. From observations it was found that the bulging indeed developed and therefore this schematization was in accordance with the observations. However, the behaviour of the clay material is rather complex in case of failure at large deformations. Some plastic deformations are expected, which are not taken into account within the calculations.

**Pressure calculations**

The calculations to indicate the pressure below the clay layer are based upon static back-calculations of the pressures. As a measure of back-calculations, the pressures are captured by using a lower-/upper boundary approach. More detailed experiments are required with monitoring equipment to measure the failure pressures more accurately.
**Pressure instead of suction**

In order to obtain a differential pressure on the clay plug, the hydraulic pressure below the clay plug is increased, while the atmospheric pressure acts on the top of the clay layer. In both situations the differential pressure makes the clay move upwards. Pressurization of the clay is applied during testing, while in practice suction acts on the clay. This adjustment excludes generation of negative excess pore pressures below the clay plug (principle of reverse end bearing), which is therefore not considered. As well, stress-dependency of clay in (deep) sea conditions are expected in practice. The limitations of these aspects on the experimental results should be therefore be recognized.

**Time dependency**

From the test results it has been found that the failure mechanism also depends on the pressure-rate. Senders [2008]] also described a rate-dependency for potential plug uplift by equation 3.6.

\[
L_{plug, lift} = \left( \frac{k_{sand}}{\gamma_w s_{ave}} - \frac{k_{plug}}{\gamma_w H_{plug}} \right) \Delta t \text{ for } p_2 \leq p_1 - p_{plug}
\]  

(3.6)

For \( k_{sand} \gg k_{clay} \) and large \( \Delta t \) is shows that the plug heave (\( L_{plug, lift} \)) increases while the pressures are kept constant. This rate-dependency was also observed during centrifuge experiments by Watson et al. [2006]. For these tests the pressure-rate has a similar effect on the clay layer, as the suction-rate has during installation in practice. From this point of view, it is interesting to further investigate to rate-dependent behaviour of plug stability (see Chapter 4).

**Practical perspective**

From a practical perspective it can be remarked that the encountered range of interest when clay tends to crack (\( D/z \approx 15 \)) is rather thin. In case of an inner diameter of 10 m, the tendency of cracking will occur in layers with thickness lower than 0.7 m. Plug uplift would therefore be expected for industry practice values. However, the limitations of the theory shall be considered (e.g. contribution of reverse end bearing). From the data which has been collected, both the quality and quantity are not sufficient to present adjustments for the current design method. However, still some conclusions could be drawn, regarding rate- and dimension dependency. In this framework, it is proposed to further investigate the rate-dependency of the plug uplift behaviour (see Chapter 4).

### 3.3 Conclusions

According to the literature it has been found that in general two types of failure mechanisms are involved for installation of suction caissons in sand overlying by clay; plug uplift or plug cracking. Comparing these two failure mechanisms it showed that the failure mechanism depends on the dimension of both the caisson (\( D \)) and clay thickness (\( z_{clay} \)), clay properties (\( s_u \) and \( \gamma'_c \)) and rate-effects. The latter is observed from several experiments (Senders [2008] and Watson et al. [2006]), however not well documented in the guidelines.

From literature research it can be concluded that cracking of the clay plug is a non-existing failure mechanism in practice nowadays. Considering typical cases with \( D/z \approx 6 \), it was assumed that plug uplift occurred during installation, however there’s an uncertainty at the transition point regarding failure mechanism, i.e. cracking or uplift. This indicates a limitation of the theory and is further investigated within the framework of this research.
In order to implement these two mechanisms into a model to predict plug uplift or cracking failure, the required failure pressures were calculated for different D/z-ratios. The uplift calculation of Senders [2008] was used to predict the uplift failure. With the use of schematization of circular plate bending, an estimation of the cracking failure could be made. From the model a transition of plug uplift or cracking was noticed for $6 < \frac{D}{z} < 10$, where uplift was expected for lower D/z-ratios. However, from a practical perspective it can be remarked that the encountered range of interest when clay tends to crack is rather thin.

Within this framework some basic experiments were conducted at the Delft University of Technology to investigate the failure mechanisms for varying D/z-ratios. The D/z-ratios of the tested clay samples were far off the transition range of plug or cracking failure, to ensure a distinction in failure mechanism. After back-calculation of the test results, it can be concluded that the model predictions are in range of the failure pressures. Cracking failure did occur for high D/z-ratios, which actually is in accordance with the theory. However, from observations of the same clay plug with high D/z-ratio there also was plug uplift encountered. This occurred for a low differential pressure condition, which indicates a pressure-rate dependency of the failure mechanism. This can also be found in literature (Senders [2008] and Watson et al. [2006]), where slow penetration (low differential pressure) contributes highly to uplift of the clay. In addition to this it was observed that high differential pressures contribute to cracking of the clay layer during testing. The relation of installation-rate and uplift of the clay plug is further discussed in Chapter 4.

The test results on plug stability exclude the possible contribution of reverse end bearing in the underlying sand, prior to uplift or cracking of the plug. The theory of reverse end bearing in layered soils is presented into detail in Chapter 5.
Chapter 4

Assessment of plug stability

In the considered framework an assessment of the plug stability is realised by introducing two extreme cases; penetration in layered soil with a stable plug (Figure 4.1a) or a moving plug (Figure 4.1b). It is assumed that ‘some’ seepage is required to install the suction caisson to target depth. The red-arrows indicate the considered reduced soil resistance due to the seepage flow. The first case is described in section 4.1, the second case is discussed in section 4.2. Finally the conclusions of both cases are presented in section 4.3.

![Figure 4.1](image)

(a) Seepage through stable clay plug with cracks  
(b) Uplift of intact clay plug

**Figure 4.1** stable and moving plug

**Case 1: Stable plug**

If it is assumed that reduction of tip resistance is required, the seepage through the clay should be sufficient to induce a seepage flow in the underlying sand. For an intact plug this means that the permeability has to increase. The permeability of the clay plug can increase due to cracks in the clay plug. The critical seepage is described by a 'critical permeability', which can be related to a fracture-area of the clay.

**Case 2: Moving plug**

In case of a moving plug, the seepage flow is induced by the uplift of the clay plug. The suction below the clay plug will attract water (generating seepage) and thus reduction of the tip resistance should occur.
### 4.1 Installation with a stable plug

When uplift of the plug should be prevented, the (boundary) condition of a stable plug is considered. From the theory obtained in Chapter 2 it was found that cracking of the plug is not considered as failure mechanism during installation. According to basic experiments on plug response it was back-calculated that a small amount of cracks (≈ 1 %) resulted in substantial seepage flow. The principle discussed in this section is to assess plug instability by increasing the plug permeability. This increased permeability (critical permeability) can then be rewritten by introducing a fracture-area; the clay plug will be fractured and the plug permeability has to increase. Because of the upward seepage flow it is assumed that the cracks are filled with the underlying sand material. As a result the permeability of fractured clay plug will increase, because of the more permeable sand (compared to clay).

#### 4.1.1 Seepage through clay plug

The seepage through the clay plug is similar to a 1D-permeability problem. There will be relative negative pore pressures due to the applied suction. With regards to the tip reduction due to seepage flow, the question arises to what extent the suction applied above the clay plug will also result in suction just below the plug (Figure 4.2). For impermeable clay layers with permeability of $k < 10^{-5}$ m/s it was found that there is negligible pore pressure reduction below the clay plug within the typical time span of caisson installations (see Appendix D.1).

**Figure 4.2** Suction inside caisson with idealized pressure drop

---

**Critical permeability**

In order to estimate the selected suction for fractured clay, an approach considering the critical permeability is chosen. This critical permeability is based on the permeability of the intact clay, but will be higher in case of cracks. It is assumed that the cracks are filled with the underlying sand material, so the critical permeability can be calculated with:

$$ Q_{\text{crit}} = Q_{\text{clay}} + Q_{\text{sand}} = \left( k_{\text{clay}} A_{\text{clay}} + k_{\text{sand}} A_{\text{sand}} \right) \frac{S}{h_c \gamma_w} $$

(4.1)

$$ Q_{\text{crit}} = k_{\text{crit}} A_{\text{base}} \frac{S}{h_c \gamma_w} $$

(4.2)
\[ k_{\text{crit}} = \frac{k_{\text{clay}}A_{\text{clay}} + k_{\text{sand}}(A_{\text{base}} - A_{\text{sand}})}{A_{\text{base}}} \] (4.3)

For simplified calculation purposes it is recommended to assume a single permeability \(k_{\text{crit}}\). For more advanced calculations it is recommended to use appropriate finite element software-tools (see Chapter 6). Based on back-calculations the fracture-ratio \(A_{\text{clay}}/A_{\text{base}}\) can be determined which is required to reduce the tip resistance. The fracture-ratio is defined by \(A_{\text{clay}}/A_{\text{base}}\), if this ratio equals 1, there is no crack requirement.

### 4.1.2 Suction below the plug

As stated previously, the suction and thus the amount of seepage flow just below the clay plug determines the amount of the reduction of tip resistance and inner friction in the underlying sand. This reduction is of interest to estimate the installation feasibility. It is expected that the suction just below the clay plug will be a fraction of the applied suction within the caisson. Considering volume continuity of the seepage through the plug, while the plug is not allowed to move, the following equation is valid;

\[ q_{\text{sand}} = q_{\text{plug}} \quad \text{thus:} \quad k_{\text{sand}} \frac{S_{\text{red}}}{\gamma_w s} = k_{\text{crit}} \frac{S}{\gamma_w z_{\text{plug}}} \] (4.4)

The boundary conditions of the 1D seepage problem is now defined, thus for each depth the parameter \(k_{\text{crit}}\) can be solved according to equation 4.5.

\[ \frac{S_{\text{crit}}}{S_{\text{crit}} + S_{\text{plug}}} = \left( \frac{k_{\text{crit}}}{k_{\text{sand}}} \right) \left( \frac{s}{z_{\text{plug}}} \right) \] (4.5)

The subsequent subsections will be based on equation 4.5, whereas the critical permeability \(k_{\text{crit}}\) can be back-calculated to the fracture-ratio \(A_{\text{clay}}/A_{\text{base}}\).

### 4.1.3 Introducing the fracture ratio

Based on the calculations as presented in Chapter 2 the required suction is calculated. This suction is required to install the suction caisson by inducing seepage flow in the underlying sand. The suction should be applied to the underlying sand just below the clay plug and thus equals \(S_{\text{red}}\). The true suction in the caisson \(S\) is limited by the uplift criteria for clay plug (equation 2.13) and thus the critical permeability can be back-calculated according to equations 4.4 and 4.5. The calculated critical permeability is then rewritten into a fracture-ratio, which is presented in Figure 4.3. A step-wise procedure to determine the fracture ratio can be found in Appendix D.3.
From Figure 4.3 it can be noted that with increasing depth the ‘required’ amount of fractures in the clay is constant. When $A_c/A_b$ equals 1, this implies that the clay plug is intact, which is the case during the initial 3 meters of penetration. The permeability of the sand was taken as $k = 1 \times 10^{-4} \text{ m/s}$. The Houlsby-method shows some deviation compared with the CPT-based methods, which is likely caused by the iterative calculation method of the soil resistances (see Chapter 2). For increasing depth, all methods show approximately the same fracture requirements.

As can be found in Figure 4.3a, there is a limited change of fracture ratio for permeabilities in the range of $1 \times 10^{-10} \text{ m/s}$ up to $1 \times 10^{-6} \text{ m/s}$. These values represent a range of clayey soils and therefore it can be concluded that the permeability of the clay is not governing for plug stability. This can be validated by changing the sand permeability, which is presented in Figure 4.3b. It shows that for less permeable sand the fracture requirement is lower, which indicates that the sand permeability governs the fracture requirement.

### 4.1.4 Effect of layer thickness

As can be found in Figure 4.3, the Houlsby-method deviates for shallow penetration in the sand. Therefore the Houlsby-method is not considered for investigation of the effect of layer thickness. In order to evaluate the effect of layer thickness on the fracture ratio, the thickness of the overlying clay layer has been varied for four cases (Figure 4.4). It can be noted that for increasing clay layer thicknesses, the fracture requirement increases (the ratio $A_c/A_b$ decreases). For the condition where 7 m clay overlies the sand, at least 15 - 20 % of cracks are required.

These values for fracture-ratio are not realistic in practice. In accordance to the previous experimental results (Chapter 3) it was found that for thin clay layers the fracture ratio was approximate 1%. In order to indicate a fracture ratio of 15 %, an example of 10 m diameter can be taken. For a fracture ratio of 15 % it holds that a circular strip-shaped crack of 0.78 m is required. From a practical point of view this is not realistic for the installation condition of required seepage flow. It can therefore be concluded that due to cracking only, it is highly unlikely that plug stability can be maintained during installation.
4.1.5 Conclusions

Assessment of the plug stability can be done by introducing a case with increased permeability of the clay plug and hence preventing uplift. The requirement of additional cracks can be expressed by a fracture ratio, which equals unity for an intact clay plug. From volume continuity it is found that the fracture ratio should be 5 - 20% for respectively a clay layer thickness of 3 m to 7 m. For the condition where seepage flows are required to install the caisson it can be concluded that these values are considered to be unrealistic in practice to maintain a stable plug.

4.2 Installation with a moving plug

From the theory within the framework of this thesis it was found in Chapter 2 that the residual plug heave will be lower for higher installation-rates. This conclusion was supported by basic experiments on the plug response of layered soils (Chapter 3), which indicated a rate-dependency on the plug response. For installation conditions when uplift can be accommodated (e.g. by a permanent stick-up height) the rate-effects of the clay plug should be assessed. This section discusses the condition of a moving plug, based on volume continuity.

If the resultant suction over the clay plug becomes higher than the uplift criteria for the clay plug, the plug tends to move upwards. This upward movement will be dependent on the inflow of water through the underlying sand to fill the water gap. With volume continuity the uplift-rate of the plug can be quantified.

4.2.1 Principle of volume continuity

Based on volume continuity, the equations 4.6 - 4.7 are valid for the suction assisted penetration phase according to Senders [2008] (see Figure 4.5). The author also mentions a system-volume, which is related to the bulk-modulus of water and takes the compressibility of the de-pressured water into account. However, this contribution is negligible and is therefore not considered here.

\[
\Delta V_{\text{pump}} = \Delta V_{\text{displaced}} + \Delta V_{\text{seep,clay}} + \Delta V_{\text{seep,sand}}
\]

(4.6)

\[
\Delta V_{\text{displaced}} = A_i v \Delta t
\]

(4.7)
Figure 4.5  Volumes during suction-assisted penetration

The displaced volume ($\Delta V_{\text{displaced}}$) is related to the installation speed of the suction caisson ($v$). The seepage volume consists of two components:

**Seepage through the clay plug (1D);**

$$\Delta V_{\text{seep,clay}} = k_{\text{crit}} \frac{S - S_{\text{red}}}{\gamma_w h_c} A_i \Delta t$$  \hspace{1cm} (4.8)

**Seepage through the underlying sand (3D);**

$$\Delta V_{\text{seep,sand}} = k_{\text{sand}} \frac{S_{\text{red}}}{\gamma_w s} A_i \Delta t$$  \hspace{1cm} (4.9)

The seepage length for the clay plug equals the height of the plug (1D). For the underlying sand, the seepage length is determined by equation 4.13 (Senders [2008]) is taken into account, since it is assumed that penetration into the underlying sand has taken place. If the clay plug is intact (i.e. not fractured), the permeability of the clay is low compared to the sand and can be neglected in the volume continuity equations. For simplicity it is therefore assumed that there will be no seepage through the (lower permeable) clay plug during uplift.

If the plug is assumed to be a solid seal, a relation with the installation-rate can be made. Equations 4.10 - 4.13 are valid for Figure 4.6, where an upward movement is considered to be positive, i.e. the velocity of the caisson ($v_{\text{can}}$) is negative and the plug ($v_{\text{plug}}$) is taken positive. It can be seen that the uplift-rate of the plug ($v_{\text{plug}}$) depends on the permeability of the underlying sand, installation-rate ($v_{\text{can}}$) and the level of suction within the caisson.

$$\Delta V_{\text{pump}} = \Delta V_{\text{displ,can}} + \Delta V_{\text{displ,plug}} + \Delta V_{\text{seep,sand}}$$ \hspace{1cm} (4.10)

$$\Delta V_{\text{displ,can}} = A_{\text{plug}} (-v_{\text{can}} + v_{\text{plug}}) \Delta t$$ \hspace{1cm} (4.11)

$$\Delta V_{\text{displ,plug}} = A_{\text{plug}} (q_{\text{sand}} + v_{\text{plug}}) \Delta t$$ \hspace{1cm} (4.12)

$$\Delta V_{\text{seep,sand}} = k_{\text{sand}} \frac{S_{\text{red}}}{\gamma_w s} A_{\text{base}} \Delta t$$ \hspace{1cm} (4.13)
4.2.2 Suction below the plug

It is assumed that when the plug starts to move upwards, the pressure drop over the plug stays constant (no seepage through clay plug). Since the pressure stays constant over the clay plug, the applied pressure above the plug will be transferred just below the plug as the suction further increases. As a result the plug moves upwards and suction is applied on the sand below the clay plug.

The plug can be schematized as a solid seal with downward pressure equal to $S_{crit,clay}$. If the suction pressure $\Delta S_1$ is applied, the clay plug will move when $\Delta S_1 > S_{crit,clay}$. The pressure below the clay plug is equal to the difference between the applied suction and critical uplift suction; $\Delta S_2 = \Delta S_1 - S_{crit,clay}$ (Figure 4.7a). Figure 4.7b presents the development of suction within the caisson, starting from initial phase with hydrostatic pressures (black line). If the water is extracted within the caisson, the pressure drops with magnitude $S$, but with a reduced magnitude $S_{red}$ just below the plug.

The clay plug will move once the suction pressure exceeds the submerged weight and frictional resistances of the clay plug (critical suction). From Figure 4.8 it can be noted that the components of plug resistance consist of a self-weight and frictional-term. Once $\Delta S_1$ becomes lower than $S_{crit}$, a suction pressure below the clay plug develops ($\Delta S_2$) which will induce a seepage flow. In the subsequent description no contribution of reverse end bearing is incorporated. The reverse end bearing is further discussed in Chapter 5.

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4.2.3 Uplift-rate of the plug

The uplift rate can be determined based on the volume continuity. In order to investigate the effects of uplift-rate, a variation of penetration-rate is considered. From equations 4.10 - 4.13 it can be seen that the uplift-rate of the plug ($v_{plug}$) depends on the permeability of the underlying sand and the installation-rate of the caisson. This indicates the bilateral dependency of plug uplift, which involves a coupled differential equation to be solved. However, by analytical elaboration it can be seen that for higher installation-rates, the increment of displaced volume of the caisson ($\Delta V_{disp,can}$) reduces faster if time-step $\Delta t$ is kept constant (equation 4.11). As a result the total pumped volume in time will increase faster and thus installation is achieved quicker. Considering the components that contribute to plug uplift, it can be found that a smaller time period corresponds to lower plug velocity. This is in accordance to the theory found in Chapter 2 and experimental results in Chapter 3.

The total uplift can be determined from the total volumes as presented in the previous sections. The difference in total volume of $V_{disp,plug}$ and $V_{seep}$ is equal to the volume which represents the uplift of the plug. The plug uplift is then calculated by dividing this volume by the base area. As well there will be a contribution of elastic plug heave, due to displaced soil of the installed caisson. Further details about elastic plug heave are presented in Appendix D.2.

4.2.4 Conclusions

Based on analytical analysis it is concluded that for slow installation (i.e. relatively low pump flow rates) the total uplift is higher compared with fast installation. This is in accordance to the theory found in Chapter 2 and experimental results in Chapter 3. The governing component which determines the speed of uplift of the plug is the seepage flow of the underlying sand. It can therefore be concluded that the permeability and applied suction are key parameters which influence the rate of plug uplift. From a practical point of view, it can be remarked that when the caisson is halted during installation, the under pressure below the clay plug is still applied and thus seepage flow is induced, i.e. the plug will lift upwards. Once the pressures equate, the plug will finally move back downwards due to self-weight.

4.3 Conclusions

If plug uplift has to be minimized, the plug stability can be assessed by introducing two extreme cases; a stable plug and a moving plug during installation. In the first case, the permeability (and thus seepage) is increased to some extent by cracking of the clay plug. Stability of a stable plug can
be assessed by introducing a fracture-ratio, which determines the crack-requirement for installation without uplift. From Figure 4.9a it can be concluded that for clayey soils \(k \approx 1 \times 10^{-7} \text{ m/s}\), at least 10% of cracking is required to install the caisson (if tip reduction is required) in the underlying sand with a stable plug. From a practical point of view this not realistic for the installation condition of required seepage flow. It can therefore be concluded that due to cracking only it is highly unlikely that plug stability can be maintained during installation.

![Fracture-ratio for 3 m clay](image)

**Figure 4.9** Fracture ratios

The effect of layer thickness of the overlying clay layer shows that for increasing thicknesses of the clay layer the required amount of cracks also increases (Figure 4.9b). A maximum of 20 % was calculated for a 7 m thick clay layer, which approximate remains constant for increasing depths. Again a remark about the practical applicability can be made here, since tip reduction is required and plug uplift cannot be accommodated.

When uplift can be accommodated by designing a permanent stick-up height of the caisson, the relation between the plug velocity and the installation-rate should be assessed properly to prevent excessive plug heave. Based on analytical analysis it is concluded that for slow installation the total uplift is higher compared with fast installation. This is in accordance to the theory found in Chapter 2 and experimental results in Chapter 3. The governing component which determines the amount of uplift of the plug is the permeability of the underlying sand and applied suction. From a practical point of view, it can be remarked that when the caisson is halted during installation, the under pressure below the clay plug is still applied and thus seepage flow is induced, i.e. the plug will lift upwards.

Furthermore it can be remarked that possible contributions of reverse end bearing are not implemented in the analysis. This implies more conservative results, which should be recognized. The effect of capacity due to reverse end bearing is discussed in Chapter 5.
Chapter 5

Modelling components of plug uplift

This Chapter presents a method to model the plug uplift mechanism for the condition where sand is overlaid by clay. In section 5.1 the mechanism for uplift of the clay plug is discussed. The next section (5.2) focuses on the possibility of passive suction contributions (reverse end bearing) in layered soil profiles. A generic model to describe plug uplift is presented in section 5.3. Finally the conclusions are presented in section 5.4.

5.1 Mechanisms at the interface

A brief description of the breakout time related to the bottom in-situ time of an object (pipeline) is presented by Roderick and Lubbad [1975]. This paper describes the bottom breakout mechanism in detail, which can also be considered for the plug uplift mechanism. The mobilization of reverse end bearing during 'breakout' of the clay plug is shown in Figure 5.1b.

As soon as suction in the caisson is mobilized, the resultant force on the plug causes a motion of the plug. This leads to a reduction in pore water pressures in the seabed beneath the plug relative to the pressures in the surrounding sand. This gives rise to the suction force mobilized by the plug, which could possibly be considered temporarily as extra capacity to withstand uplift (suction mobilization in Figure 5.2b).

![Figure 5.1 Components of plug uplift](image-url)
Suction mobilization

If the uplift force is less than required to cause immediate uplift ($Q_{s,max}$), the time required for uplift depends on the force magnitude and seabed properties. The reduction of pore pressures in the seabed under the object causes a flow of water from the surrounding material towards the low pressure area. The rate of flow depends on the coefficient of permeability of the soil and the magnitude of pressure difference. The suction force increases from zero to a maximum value.

Suction plateau

The suction plateau in Figure 5.2b is presented as plateau distance, where the displacement of the pipe is shown on the horizontal axis. For breakout of the pipe (or plug), this distance represents a force equilibrium phase where the downward suction force is maintained for a certain upward movement. For constant installation-rates, the horizontal axis can be replaced by a time axis while maintaining the suction curve. The plateau distance will then become a time period where the suction remains constant. This is interesting for installation practice, because this time period can be regarded as extra safety to 'delay' uplift of the plug.

Suction release

As water moves to the low pressure zone, the pressure difference decreases and there is an accompanying upward movement of the object. As the object movement becomes greater, the dissipation of pressure differences and seabed deformations cause a loss of strength and uplift occurs (Suction release in Figure 5.2b). Under further upward movement of the plug the suction force reduces from its maximum to zero. If the uplift force is large or is increased due to high suction in the caisson, plastic deformations might occur. This condition is not expected for suction caisson installation, since the mobilized suction is gradually decreased during the installation phase.

![Figure 5.2](image-url)  

Figure 5.2  Pipe/soil interaction after Bridge et al. [2004]

Considering a layered soil profile, where sand is overlaid by clay, the theory for object breakout is applicable to the underlying sand if the clay plug is considered to be impermeable. From literature it is found that the assessment of suction capacity is based on dissipation of negative excess pore pressures and thus is related to the permeability of the seabed (Gourvenec et al. [2009]). An assessment based on the permeability of the underlying sand is therefore proposed to investigate a possible contribution of passive suction capacity in sand. This is presented in the next section (section 5.2).
5.2 Reverse end bearing in sand

Within the framework of this thesis it was found that reverse end bearing is not considered as potential contribution for installation layered soils (Chapter 2). Additionally, the experimental test results (Chapter 3) and analytical analysis (Chapter 4) excluded possible contribution of passive suction capacity. This section discusses the components of reverse end bearing in layered soils, especially for conditions where sand is overlaid by clay. For homogeneous clays the benefit of caisson installation with regards to plug stability, lies in their ability to resist uplift by generating significant resistance due to passive suction capacity (in literature this phenomenon is defined by reverse end bearing). This capacity is governed by the development of negative excess pore pressures (suction) within the confined soil plug. If these passive suctions can be maintained, a significant uplift resistance is given by reverse end bearing. Especially for large diameter caissons, the capacity due to reverse end bearing could be governing, since the passive suction capacity is mobilized over the entire cross sectional base.

Figure 5.3 presents a simplified overview of the mobilization of reverse end bearing in case of penetration in layered soil profiles. The schematization indicates that for penetration through the clay (Figure 5.3d), the contribution of reverse end bearing relies on the properties of the underlying sand (e.g. permeability). For layered soils, when for example sand is overlaid by clay, it is uncertain what the (remaining) capacity of reverse end bearing is and how long this can be considered. In order to assess this uncertainty, first the reverse end bearing is evaluated for homogenous clay conditions.

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Reverse end bearing capacity in clay

The generated capacity due to reverse end bearing in clay is calculated in a similar way as for 'regular' end bearing capacity (Verruijt and van Baars [2007]), i.e.,

\[ P_{REB} = N_c s_u \]  \hspace{1cm} (5.1)

Where \( N_c \) is estimated by many authors and guidelines (DnV [1992]) and is assumed to be 9 for greater depths (see Appendix B). This value is based on Prandtl’s-theory for a strip loading \( N_c = 5.14 \) and extrapolated for circular footings and depth dependency. The factor is based on a critical stress state and therefore neglects any consolidation effects. For the condition of layered soils this is a critical aspect (for the underlying sand) and thus the consolidation time should be assessed. Therefore the next subsection discusses the time dependency of reverse end bearing in the underlying sand.

5.2.1 Time dependency of reverse end bearing

The reverse end bearing capacity in time relies on the dissipation of the negative excess pore pressures in the seabed. For homogeneous clays the dissipation of pore pressures develops slowly and is similar to a consolidation problem, as indicated in previous section. The time-dependency of the dissipation during consolidation (or reverse end bearing) depends on the permeability of the clay and drainage path, i.e. suction will dissipate faster if the flow through the seabed can be higher or the distance is shorter. This is also stated by Huang et al. [2003], were the magnitude of suction to be sustained greatly depends on the permeability of the subsurface soil and drainage paths. Higher permeable soils will tend to decrease the suction faster under sustained loads.

![Figure 5.4 Schematization of reverse end bearing during installation](image)

From this it can be concluded that the reverse end bearing capacity is clearly related to the permeability of the soil and drainage path. However, for clays, a dimensionless \( N_c \)-factor is taken to estimate the capacity. The factor is based on total stress failure and should be treated with great care, since it does not incorporate soil properties (e.g. permeability). The properties which determine the capacity of reverse end bearing are the undrained shear strength \( s_u \) (magnitude of capacity) and permeability of the clay (amount of time to maintain capacity). A schematization can be found in Figure 5.4. Since the permeability of clays is very low, the total stress failure will intervene first according to Prandtl’s-theory.

For layered soils (as presented in Figure 5.3d), the generation of negative excess pore pressures will develop in the underlying sand. Therefore the permeability and seepage length of the underlying sand should be considered in order to assess the time-effects.
Introducing the time lag

For clays, with permeabilities of at least $10^3$ times lower than sand, the dissipation of the suction takes substantially more time than for sand. The effects of permeability can be explained by introducing the time lag \(^1\). The elaboration of the differential equation is presented in Appendix E.1 and a definition is given by:

$$T = \frac{s}{k} = \frac{\text{seepage length}}{\text{permeability}} \quad [s] \quad (5.2)$$

From this equation it can be concluded that for lower permeabilities the time lag increases, thus the dissipation of the suction takes longer. In order to put this in perspective, a simple example is evaluated with regards to the typical sand and clay permeabilities. The time lag for this example is shown in Figure 5.5, the range of normalized suction is typical for suction caisson installation.

![Figure 5.5 Time lag for range of permeabilities](image)

The horizontal time - axis in Figure 5.5 is an extended range compared to practical conditions as the installation of suction caissons roughly is done in 1 - 6 hours, depending on geometry and soil conditions. Each line represents a range of time - lag $T$ and is depending on caisson dimensions and penetration depth. It can be noted that for $k = 1e^{-3}$ [m/s] the time-lag is in the range of suction caisson installation, thus the pressures will equalize relatively quick. Reverse end bearing is mobilized, but is maintained very short and capacity due to passive suction should not be considered.

However, for a permeability of $k = 1e^{-5}$ [m/s], it can be concluded that the time required for equalization is more than one day for the chosen range of dimensions. For a typical installation period of 6 hours, the passive suction can be considered, since the time required for water inflow is lower than a typical installation period.

For clayey soils (3 right curves) it can be seen that the range of time lag exceeds one week. Considering typically installation time periods, this implies that the time for dissipation of the negative excess pore pressures (reverse end bearing) in these soils is more than the time period of installation.

---

\(^1\)The time required for water to flow to or from a point in a subsurface, until a desired degree of pressure equalization is attained (Hvorslev [1951]). For this approach, the definition of flow lag would be more appropriate, since the delayed inflow of water is considered.

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Practical perspective

It should be remarked that this theoretical approach is based on purely homogeneous soils, which are rarely found in practice. However, a distinction in the permeabilities of sand which might be favorable to passive suction generation can be recognized in Figure 5.5. These values can be implemented for the design phase in engineering practice by carefully determination of the permeability of the sandy soil. This can be achieved by CPT-tests which include a dissipation test on the pore pressures. In a dissipation test the porewater pressure change is obtained by recording the values of the pressure against time during a pause in pushing and while the cone penetrometer is held stationary. As a result the dissipation (contribution of reverse end bearing) can be quantified, which is of interest to assess the plug stability. Additionally, oedometer tests or correlations with particle size distributions can be used to determine (a range of) the in-situ hydraulic conductivity properties.

5.2.2 Conclusions

The plug uplift mechanism is schematized by using the theory for object breakout from the seafloor. This theory is based on recommendations (DnV [1992]) for clay properties only and hence not applicable for sandy soils or layered soil profiles. However, from literature it was found that the assessment of suction capacity is based on dissipation of negative excess pore pressures and thus is related to the permeability of the seabed. An assessment based on seabed permeability is therefore proposed to investigate a possible contribution of passive suction capacity in sand.

Reverse end bearing capacity in time relies on the dissipations of negative excess pore pressures in time and is related to the soil permeability and drainage length. For clays, the permeability is low and therefore total stress failure will intervene first during installations.

For sands, the possible capacity in time of pore pressure dissipation depends on the time required to equalize the pressure to its initial value. This time requirement is defined by the 'time lag' and can be calculated for a range of caisson dimensions, pressures and permeabilities. This shows that for a typical installation period of 1 - 6 hours, for permeabilities lower than $1e^{-5}$ m/s some reverse and bearing capacity due to delayed inflow of water can be build up, which is interesting for installation practice.

5.3 Modelling the plug uplift mechanism

Previous section described the time-effects related to contribution of reverse end bearing capacity in layered soils. In order to further assess the contribution of reverse end bearing is these conditions, a simplified model based on individual components is adopted to describe the uplift behaviour. This section describes the plug uplift aspects by dividing the contributions two components; uplift by suction acting on top of the clay plug and uplift due to seepage inflow below the clay plug. These components are described in the first two paragraphs, followed by a combination of the components in the third paragraph. This analysis presents a first approximation of the governing components which determine plug uplift. It should be noted that this model is strongly simplified and is a set-up to indicate the effects of individual components during uplift of the plug.
5.3.1 Uplift by suction only

In theory the plug uplift is linearly proportional to the applied suction, according to a spring model (Figure 5.6). Similar to Hooke’s Law, elastic potential energy is stored as a result of deformation of an elastic object, in this case plug uplift. The model assumes no boundary effects of underlying sand and thus the plug movement depends only on the applied resultant force and spring stiffness.

\[ \Delta s_{plug} = \frac{SA_{base}}{k} \]  \hspace{1cm} (5.3)

\[ k = f(s_u, D, \gamma', z) \]  \hspace{1cm} (5.4)

Once the plug is moving upwards due to the resultant upward force, the volume above the plug will decrease. From Figure 5.7 it can be noted that \( V_{3a} \) is lower than \( V_2 \) due to the uplift of the plug. Since a negative pressure is applied (suction) above the plug, a volume decrease of \( V_2 \) to \( V_{3a} \) will increase the pressure above the plug. An increased pressure lowers the suction on the plug, thus the underpressure is brought back to its initial value. This can be observed by a drop in suction pressure during sudden uplift of the plug. As a result the resultant force (or suction) lowers and the upward movement will reduce. In the next time-step the suction can be built up again, according to the spring-model. A detailed flow-chart is presented in Appendix E.2.

In order to account for this iterative process, the movement of the clay plug is defined by the mechanical ‘work’ definition; a force is said to do work when it acts on a body so that there is a displacement of the point of application, in the direction of the force. For the application of a clay

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plug, the force is obtained by the applied suction multiplied by cross area;

$$\Delta W(t) = S(t)A_{base}\Delta s_{plug}$$

(5.5)

The iterative process is then implemented by taking the increments in suction and volume to calculate the generated work ($W$). A small time-step should be taken to avoid large deviations of the calculations as indicated in Figure 5.8;

$$\Delta W(t) = \Delta P(t_{-1})\Delta V(t_{-1})$$

(5.6)

![Figure 5.8 Effect of large time-step for calculations](image)

It can be noted that the general trend for uplift due to suction only, consists of an initial increase in plug uplift. During installation (increasing time and depth) the increments for uplift deceases because the incremental work applied to the plug decreases. Combining both equations give the equation to describe the uplift of the plug by;

$$\Delta s_{plug} = \frac{\Delta P(t_{-1})\Delta V(t_{-1})}{S(t)A_{base}}$$

(5.7)

### 5.3.2 Uplift by seepage inflow

During uplift of the clay plug, the gap below the plug is filled with extracted water from the underlying sand. As stated in the previous section, the plug uplift due to seepage is time-dependent. If the plug is assumed to be impermeable, the total volume of water inflow during installation equals the residual plug heave.

$$\frac{\Delta s_{plug}(t)}{dt} = q_{sand} = -k_{sand}\frac{S(t)}{\gamma_{w}s}$$

(5.8)

Figure 5.9 shows a schematization of the plug uplift due to seepage inflow by a dashpot. Dashpots are elements to model time-dependent behavior and are therefore applicable for uplift due to seepage inflow. In contrary to a spring model, energy will dissipate as a result of deformation of the object. This implies that there will be residual heave depending on the installation time.
Modelling components of plug uplift

As indicated in section 5.1, the amount of plug heave is highly dependent on the soil permeability and thus a range of permeabilities is chosen to assess the plug heave in Figure 5.10. The installation time for this example was set to 2.5 hr with a constant pumping rate of 300 m$^3$/hr. It can be noted that the plug uplift accelerates for increasing penetration in sand (and time). This is due to increasing suction and seepage length, which results in a wide range of plug uplift for different hydraulic conductivities. This indicates that the permeability of the underlying sand has great impact on the residual plug heave.

![Figure 5.9 Dashpot-model for plug uplift](image)

![Figure 5.10 Plug heave due to seepage flow for a range of permeabilities](image)

5.3.3 Combination of suction and seepage inflow

The total uplift of the plug can be described by combining both components of the uplift mechanism; uplift by suction and uplift by seepage inflow. The question arises whether the components should be placed in series or in parallel. Regarding the orientation of the plug, (suction on top of plug and seepage below) the composition of the components seems to be in series according to Figure 5.11.

If it is assumed that the plug stays intact during uplift (sealing effect and rigid plug), the displacements of both components have to be equal according to Figure 5.11. This also implies that a possible higher suction on top of the plug (which will cause a certain uplift of the plug) is restricted by the seepage inflow below the plug. This principle relies on dissipation of pore pressures in time and is considered to be similar to the reverse end bearing principle. However, as indicated in previous sections, it should be remarked that this schematization is strongly idealized. The model considered a residual plug heave due to the deformed dashpot, which is not realistic in practice. Once the pressures equalize, the plug will settle due to self-weight of the plug.
Modelling reverse end bearing

The reverse end bearing capacity can be schematized by the approach in which the components of suction only and seepage inflow are included. If it is assumed that the clay plug will be an impermeable sealed plug and not subjected to elastic deformations, a theoretical assessment of the reverse end bearing can be made. As stated in the previous section, the displacement of the plug depends on both suction (applied on top of the plug) and seepage flow (below the plug). This is presented in Figure 5.12, where it can be noted that for suction only the uplift is more or less constant for increasing depth, whereas the contribution of seepage inflow below the plug slowly increases for increasing depth.

It should be remarked that the chosen value of permeability is highly dependent on the calculated plug uplift (as indicated in Chapter 4) in Figure 5.12. The amount of uplift can be considered as indication of the contributions. The contour-line shows an indication of the maximum expected plug heave.

The starting point of both 'contributions' is around 2 m penetration. The self-weight and shaft resistance of the plug are incorporated for the calculations, by calculating the suction requirement for plug uplift according to Senders [2008]. It can be remarked that the schematized contribution

---

**Figure 5.11** Combination in series to model plug uplift

**Figure 5.12** Components of plug uplift
of reverse end bearing is mobilized when plug uplift initiates. This is accordance to the break-out resistance of Bridge et al. [2004] and the uplift-resistance of Gourvenec et al. [2009], where a small displacement is required to mobilize resistance against uplift. Considering both contributions in Figure 5.12 of plug uplift, a distinction between 2 phases can be made;

### Phase A
The theoretical uplift due to suction only is higher than that for seepage inflow. Since both contributions are coupled (impermeable and rigid plug), the actual uplift depends on the contribution of the seepage inflow, which is lower.

### Phase B
The theoretical uplift caused by the seepage inflow is larger than that for suction only. The former will be the governing contribution to the plug heave. The plug is 'pushed' upwards due to the inflow of water just below the plug.

The most interesting condition is Phase A, where a downward resistance (due to restrictions) is generated as long as the uplift due to seepage is lower than that for suction only. Figure 5.13 presents a plot of the differential uplift of both components for Phase A. It can be noted that Figure 5.13 shows similarities with the passive suction model according to Bridge et al. [2004] in Figure 5.2b. The suction plateau is negligible in this example, thus the capacity consists of mobilization followed by release of the suction capacity. The total time required to mobilize the capacity in this example is 40 minutes.

![Figure 5.13 Difference in uplift with penetration depth](image)

It should be remarked that Figure 5.13 only indicates duration of negative excess pore pressure generation (reverse end bearing), based on the difference in uplift by two components acting on the plug. The magnitude of capacity of reverse end bearing is not indicated by this figure. The model assumes constant penetration-rate, thus effect of halted penetration are not considered. If the penetration is stopped, uplift will continue due to seepage inflow below the plug until equilibrium of pressures is reached. Hereafter the plug will settle due to self-weight of the plug.

### 5.3.4 Conclusions
With the implementation of a spring-dashpot model the components which determine uplift of the plug can be described. The applied suction above the plug will initiate movement of the plug, but this will be restricted by static components (e.g. skin friction and self-weight) and inflow of water below the plug. For the condition when the applied suction on top of the clay plug is higher than seepage inflow below the plug, an indication of negative excess pore pressures is presented.
5.4 Conclusions

Considering the passive suction capacity successively, the phases of *Suction mobilization*, - *plateau* and - *release* are distinguished. Disregarding the soil type, it was found that the magnitude of resistance (Suction mobilization) and duration (Suction plateau) depends on the dissipation of negative excess pore pressures in time. This can be related to the permeability of the soil and shows that for clayey/silty soils, the time for dissipation exceeds a typical installation time period by a large extent. The magnitude of the maximum uplift resistance and the duration of the resistance (time period of suction plateau) are believed to be most important for assessment of plug uplift failure. Significant contributions of plug uplift resistance can be considered for installations of caissons in clayey soils. In case of installations in layered soils, where sand is overlaid by clay, the time to maintain reverse end bearing capacity is expected to be lower since the permeability of sand is higher.

The possible capacity of excess pore pressure dissipation depends on the time required to equalize the pressure to its initial value. This time requirement is defined by the *time lag* and can be calculated for a range of caisson dimension, pressures, permeabilities and drainage length. This shows that for a typical installation period of 1 - 6 hours and for permeabilities higher than $1e^{-5}$ m/s some capacity due to delayed inflow of water can be considered. It should be remarked that the chosen range of permeability has a great impact on the time requirement for dissipation.

With the implementation of a spring-dashpot model the components which determine uplift of the plug can be indicated. If the applied suction above the plug will initiate upward movement of the plug, this will be restricted by the inflow of water below the plug. Besides the skin friction and self-weight of the plug are static components, which will settle the plug after equalization of the pressures. Mobilization of reverse end bearing can be schematized for cases when the uplift by suction only exceeds that of seepage inflow (phase A in Figure 5.12). The time range of the generated capacity is based on the condition where the theoretical uplift due to only suction is higher than that for seepage inflow.

The time requirement to maintain reverse end bearing capacity is greatly dependent on the chosen permeability and drainage length. For more permeable soils the seepage flow increases rapidly, which limits the time-period for reverse end bearing generation. As a result of this, it can be concluded that plug heave will be higher for more permeable soils.
Chapter 6

Verification of the theory

This Chapter presents an analysis of two practical cases which are favourable to plug uplift, accompanied by a numerical calculation on plug uplift. The first case, Case A, is published by Alhayari et al. [1999] and discusses the anchors of the Curlew FPSO-project in section 6.1. The second case, Case B, was published by Tjelta et al. [1986] and discusses the experimental program for installation of the Gullfaks C-platform in section 6.2. Both cases include sufficient key-data to do back-analysis for plug uplift assessment. Section 6.3 presents numerical verification of the theory on plug uplift using finite element software. Finally the conclusions are presented in 6.4.

6.1 Case A: Curlew FPSO

For the Case A-site a group of three suction caissons were installed in the North Sea, where the soil conditions are typically characterized by dense sand overlaid by less permeable soils. In this case the dense sand is overlaid by clayey, silty sands, which is assumed to prevent seepage flow to develop during the suction phase. The key-data of the three anchors are presented in Table 6.1 and Appendix F.1.

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Diameter [m]</th>
<th>Wall thickness [m]</th>
<th>Penetration depth [m]</th>
<th>Total submerged weight [kN]</th>
<th>Depth dense sand [m]</th>
<th>Depth very dense sand [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7</td>
<td>0.03</td>
<td>9</td>
<td>600</td>
<td>4.0</td>
<td>6.8</td>
</tr>
<tr>
<td>2</td>
<td>7</td>
<td>0.03</td>
<td>9</td>
<td>600</td>
<td>4.0</td>
<td>6.8</td>
</tr>
<tr>
<td>3</td>
<td>7</td>
<td>0.03</td>
<td>9</td>
<td>600</td>
<td>4.7</td>
<td>8.1</td>
</tr>
</tbody>
</table>

In the design, no reduction in penetration resistance due to seepage is accounted due to high concentrations of silt and clay. These layers may prevent the flow of water around the skirts during the suction phase. In case of clean sands, this flow of water significantly reduces the skirt penetration during suction. As can be found in Table 6.1, the soil consists of a loose sand up to 4 m (4.7 for anchor 3), which overlies more dense sand up to 6.8 m (6.8 m for anchor 3). The very dense sand is found below 6.8 m (8.1 m for anchor 3). An interpolated plot of the presented cone resistances can be found in Figure 6.1.

Estimated resistances

In order to estimate the penetration resistances for the soil conditions of anchors 1 - 3, some reference-calculations were done of historical data with similar soil conditions, e.g. dense sands in the North Sea. The purpose of these reference calculations is to estimate the frictional terms ($K_f$) for installation.
of anchors 1 - 3. The total penetration resistance ($Q_{\text{tot}}$) consists of tip bearing and side friction and is calculated according to:

$$Q_{\text{tot}} = Q_{\text{tip}} + Q_{\text{skirt}}$$  \hspace{1cm} (6.1)

In the sand the tip resistance and side resistances are calculated as follows:

$$q_{\text{tip}} = K_t q_c \quad \text{and} \quad q_{\text{skirt}} = K_f \sigma'_v \tan(\phi')$$  \hspace{1cm} (6.2)

From the reference-data provided a good fit was found by using a $K_t$ value of 0.2 and a lateral earth pressure coefficient, $K_f$, varying from 0.5 to 2.0 dependent if the soil is layered with loose silty sand (0.5) or dense sand (2.0). These coefficients are based on a best-fit approach of measured suction and predicted suction of the reference-data. The skirt resistances (inside and outside) have been adjusted to meet the best-fit solution for different layering of the soil. Based on these reference calculations, a set of empirical coefficient, $K_t$ and $K_f$, are used for calculation of the skirt penetration resistance of anchors 1-3.

<table>
<thead>
<tr>
<th>Soil Profile</th>
<th>Anchor 1 &amp; 2</th>
<th>Anchor 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$q_c$ [MPa]</td>
<td>Depth [m]</td>
</tr>
<tr>
<td>Loose silty sand</td>
<td>&lt; 5</td>
<td>0 - 4</td>
</tr>
<tr>
<td>Dense sand</td>
<td>5 - 20</td>
<td>4 - 6.8</td>
</tr>
<tr>
<td>Very dense sand</td>
<td>&gt; 20</td>
<td>6.8 - 9</td>
</tr>
</tbody>
</table>

**Estimated suction**

The estimated suction is based on the reference parameters and assumes no tip reduction due to seepage flow. Thus the estimated suction is calculated in a similar way as calculations for installation in homogeneous clay, where the suction causes a pressure differential across the top plate of the caisson. This results effectively in an additional vertical load equal to the suction times the plan area of the caisson.

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Figure 6.2 presents the estimated suction requirement for installation of anchors 1 - 3. It can be noted that the suction requirement increases for the deeper dense sands \((z/D > 0.95\) for anchor 1 & 2 and \(z/D > 1.15\) for anchor 3). The normalized suction of \(P/\gamma wD = 8\) corresponds to a suction of 515 kPa.

6.1.1 Measured suction

From the three anchors considered only a single suction measurement plot was available to compare the predicted suction with the measured suction; i.e. only for anchor 3 the suction measurement was available. From cone resistance data (Figure 6.3a) it can be noted that the input for the \(q_c\)-values are well determined. These input-values are required to estimate the resistances for penetration.

Figure 6.3 presents the measured suction from the installation of anchor 3 by the black solid line. The red line represents the estimated suction according to the \(q_c\)-input in Figure 6.3a and \(K_t, K_f\)-values from Table 6.2. It can be noted that the measured suction is higher than the estimated suction (red curve). This deviation is explained in the following paragraphs, where the water injection...
devices and cross ties are discussed. First a best-fit back calculation of the suction is obtained for further analysis.

**Best-fit back calculated suction**

Disregarding both effects stated above, a best-fit back-calculation can be done based on the $K_t$ and $K_f$-values (Figure 6.4). Figure 6.4a show that a reasonable approximation of the measured suction can be obtained. The corresponding values for $K_t$ and $K_f$ are shown in Figure 6.4b. From the latter figure it can be concluded that the input frictional term is underestimated for layers with high $q_c$-values.

![Figure 6.4](image-url)  
(a) Measured and back-calculated suction  
(b) Back-calculated shaft- and tip parameters  

At depth 8 m the water injection devices might have reduced the tip resistance and thus $K_p$ could have equal zero. Even without tip resistance, still an overestimation of the suction is calculated at a depth of 8 m, as indicated in Figure 6.5a. This could imply that plug uplift occurred, since 1 m of penetration was achieved with constant 300 kPa suction. Verification of the possible plug uplift is presented in subsection 6.1.2.

**Cross ties penetration**

The geometry of the anchors includes a 3 m high cross tie (lug stiffening), which is connected just above halfway of the skirt. Once the anchor penetrated the soil at a depth of 4.5 m, the cross tie contacts the soil and the penetration resistance will increase. Thus an increase of resistance is expected at 4.5 m penetration, which can also be seen in Figure 6.4a and Figure 6.4b.

**Water injection device**

As the required and allowable suction were quite high and it was expected that the top silty sand layer would act as a seal, it was decided to install water injection devices at the skirt tip. The main advantage of these devices is that the tip resistance could be reduced by the outflow of water at the skirt tip, which will liquefy the very dense sand and hence reduce the tip resistance during installation. Since no data is available regarding the time or penetration depth of the injection, it is difficult to determine the impact of the water injection.

However from Figure 6.4a it can be found that the measured suction slightly reduces, while the estimated suction (and thus soil resistances) supposed to increase for penetration higher than

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It is therefore assumed that the tip resistance reduces towards 0 from a penetration of 7 m (Figure 6.4b). This shows good agreement up to a penetration of 8 m, however beyond this depth the prediction and back-calculation both overestimated the measurements. The reduction of tip reduction could be caused by plug uplift or by the water injection devices at the skirt tip. These effect are further discussed in subsection 6.1.2.

### 6.1.2 Plug uplift back-calculations

From the suction predictions it was assumed that no seepage flow will develop due to the less permeable top layers of silty, clayey sand. According to the plug uplift theory presented by Senders [2008], a back-calculation can be done to check if this assumption is valid. For a clay plug the equation is given by equation 6.3 and excludes contribution of reverse end bearing.

\[ P_{plug} = \left( \gamma' + \frac{4}{D} O s_u \right) z_{plug} \quad (6.3) \]

The shear term is valid for (remoulded) clay, but for this case replaced by shear resistance in sand, i.e.;

\[ f_{skirt} = K_f \sigma'_{vtan(\phi')} \quad (6.4) \]

In order to determine the uplift potential in a conservative way, the parameter \( K_f \) is taken as 2.0, which implies an overestimation of the shear resistance. Additionally, no contribution of reverse end bearing is considered. The uplift potential is thus conservative since a higher uplift pressure is required. A conservative back-calculation is chosen to ensure the most reliable back-calculation. The exact plug thickness is arbitrary and therefore three interface possibilities for plug lift off are shown in Figure 6.5. Only the top layer (0 - 4.7 m) consists of silty clayey sand and is therefore assumed to be most favourable to plug uplift.

Possible plugs 1 and 2 will generate resistance against plug uplift due to the underlying cohesive soil. In general the total resistance against uplift consists of self-weight, shear resistance and passive suction capacity (reverse end bearing). The latter contribution depends on the hydraulic conductivity of the underlying soil and is higher for low-permeable soils. Considering the plug which is the most
favourable to uplift, the third plug (thickness 4.7 m) is taken for back-calculations.

**Table 6.3** Potential plug dimensions for lift off

<table>
<thead>
<tr>
<th>Plug No.</th>
<th>Thickness [m]</th>
<th>$P_{plug}$ [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0 - 2</td>
<td>30.6</td>
</tr>
<tr>
<td>2</td>
<td>0 - 2.5</td>
<td>42.1</td>
</tr>
<tr>
<td>3</td>
<td>0 - 4.7</td>
<td>110.7</td>
</tr>
</tbody>
</table>

Figure 6.6 presents the criteria for plug uplift according to Senders [2008]. It can be seen that for penetration more than 4.7 m the plug is favorable for uplift. The exceeding of the plug uplift criteria is substantial, a further assessment of this possibility is presented in the subsequent paragraphs.

**Figure 6.6** Implementation of plug uplift for plug 3 ($z = 4.7$ m), without contribution of reverse end bearing

**Analysing parameters**

Soil parameters are determined prior to the design of the suction caisson. The most interesting parameter for plug stability analyses is the permeability of the layered soil. With the use of this parameter, the fracture requirement and possible plug heave (Chapter 4) and contribution of reverse end bearing (Chapter 5) can be analysed. From the provided report it was found that an estimate of the permeabilities was found by correlations to the grain size distributions. Based upon the report, the maximum recommended soil permeabilities are:

- Loose to medium dense silty sand: \( k = 1 \times 10^{-5} \) [m/s]
- Dense silty sand: \( k = 1 \times 10^{-6} \) [m/s]

**Fracture requirement**

From the theory in the previous chapters, a theoretical fracture-ratio can be back-calculated for this case. With rough estimations, it can be found that the fracture requirement varies between 25% for shallow penetrations, which decreases towards 15% for target depth (Figure 6.7a). In order to put this in perspective, a circular strip fracture of 0.9 m to 0.6 m is required to install without uplift. This is supposed to be not realistic in practice, thus it can be concluded that cracking likely did not occur.

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Plug heave

An estimate of the possible plug heave can be given by the total inflow of water below the sealing plug, once the differential pressure over this plug exceeded the self-weight, shaft resistance and reverse end bearing of the plug. For this example, no contribution of reverse end bearing is taken into account, since this will give more conservative results for the heave estimate. With the given permeabilities, suction requirement and installation time, an estimate of the plug heave can be made according to;

\[ q = k_{\text{sand}} \frac{S}{\gamma_w} \]  

(6.5)

\[ \Delta s_{\text{plug}} = q\Delta t \]  

(6.6)

Where the time-step (\(\Delta t\)) depends on the installation time of the suction caisson. Since the permeability of the soil is relatively low (for sandy soils) due to some silt fractions, it can be found that the plug heave estimate is negligible (order of centimeters) in Figure 6.7b. From the published data it was found that installation of the caisson was executed with a rate of 10 m/hr, thus the total installation time is approximate 1 hour. Back-calculations of the estimated plug heave indicate that the plug heave is very limited.

Reverse end bearing

The theory on reverse end bearing relies generally on the dissipation of negative excess pore pressures. Since it is found that the permeability of the sand is relative low (for sandy soils), there might be a contribution of reverse end bearing against plug uplift. This contribution is based on the delay of water inflow, once the plug tends to lift off. The time requirement of pressure equalization depends the time frame of the additional ‘safety’.

With the estimated permeabilities of the soil strata, the time lag for this case can be determined. Figure 6.8 presents the design permeabilities at a penetration depth which is favorable for plug uplift. It can be seen that within the range of design permeabilities a time lag of 1 day is exceeded.

From the published data it was found that installation of the caisson was executed with a rate of 10 m/hr, thus the total installation time is approximate 1 hour. Thus it can be concluded that a contribution of reverse end bearing (delay of water backflow) theoretically could be relied upon.
Additionally, the water-injection-device was connected to the vessel’s fire hose system. Therefore limited effects on pore pressure dissipation (reverse end bearing) are expected due to the relatively low pressure. However it is unknown what the effects or influence of the water injection devices was on the reverse end bearing development. Since the latter relies on dissipation of pore pressures, while near the tip ‘extra’ water is foreseen, the reverse end bearing might have been decreased relatively quickly.

Explanation of pressure drop

From the case data provided, no uplift was monitored and/or mentioned, thus another phenomenon might have occurred which clarifies the pressure drop at penetration of 8 m. Figure 6.9 presents the contributions of plug uplift according to Senders [2008], which are self-weight ($P_{\text{weight}}$) and shaft friction ($P_{\text{friction}}$). In addition the contribution of reverse end bearing ($P_{\text{REB}}$) is presented, based on an $N_c$-value of 7.5. This value is based on best-fit analysis of the total plug uplift criteria ($P_{\text{plug}}$).

Since the installation time of the caisson was relative short (1 hour), a contribution of reverse end bearing could be applicable. It can be noted that the total plug uplift resistance now increases to a value where the pressure drop in the caisson was encountered. This would imply that at the point of pressure drop, the seepage flow was induced by the uplift of the plug. Initially this causes seepage flow and thus reduction of the soil resistance near the tip. Simultaneously it causes limited uplift, but this will tend to increase in time. It should be remarked that this principle is only valid when it is assumed that the water injection devices near the tip have limited impact on the dissipation of negative excess pore pressures; i.e. suction within the centre of the clay plug can still be maintained.

Limitations in practice

From a practical point of view some limitations to contribution of reverse end bearing can be mentioned. In the first place it can be remarked that during installation the soil skeleton close to the skirt slightly changes due to penetration of the skirt. It should be noted that this may change the density of the sand and thus affect the flow characteristics. Secondly, it can be remarked that in cases of local shallow gas reservoirs, the dissipation of negative pore pressures could develop quicker and thus no contribution should be taken into account.

6.1.3 Conclusions

In order to install the anchors in challenging conditions with very dense sands overlaid by a less permeable layer. Since limited seepage flow was expected, water injection devices were implemented as mitigating measure for successful installation. The installation resistances are estimated according to reference-data for similar soil conditions. From the measured data it can be concluded that the
measured suction is partly in accordance with the estimated suction. Because of the implementation of water injection devices and cross tie penetration, the measured suction deviates somewhat from the predictions.

Disregarding these effects, still the potential for plug uplift failure can be assessed by calculations according to Senders [2008]. This showed that for conservative calculations, the estimated plug heave was limited (negligible), which is in accordance to the observations. However, interpretation of the measured suction indicated a pressure drop at 8 m penetration depth, which can be clarified by the contribution of reverse end bearing. This shows a reasonable agreement based on an $N_{c}$-value of 7.5. The contribution of reverse end bearing was justified by the high installation-rate of 10 m/hr and assuming limited effect of the water injection devices.

**Figure 6.9** Implementation of plug uplift, divided by components
6.2 Case B: Gullfaks C

This case is based on the historical large-scale penetration test at a deep water-site by Tjelta et al. [1986]. The Gullfaks C platform installation was scheduled in 1989, being the largest and heaviest offshore concrete structure ever installed. During the foundation design phase it became evident that geotechnical challenges were expected. Therefore it was decided to execute a large scale-test to improve the confidence in the predicted soil response and to clarify uncertain aspects. Key input-data to perform back-calculations are presented in Table 6.4 and Appendix F.2.

Table 6.4 Key-data from Case B anchor installation

<table>
<thead>
<tr>
<th>Diameter [m]</th>
<th>Wall thickness [m]</th>
<th>Penetration depth [m]</th>
<th>Total submerged weight [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.5</td>
<td>0.035</td>
<td>22</td>
<td>3532</td>
</tr>
</tbody>
</table>

Water injection devices were implemented to ensure tip reduction in the dense sands, which were expected at 18 m depth (Figure 6.10a). Figure 6.10b presents the measured tip resistance, which are compared with back-calculated resistances of the soil profile presented in Figure 6.10a. A similar approach as for Case A is used for the calculations, which shows good agreement with the measured resistance.

6.2.1 Measured suction

Based on the resistances for penetration, one can estimate the suction requirement for installation. If no tip- and shaft reduction due to seepage can be considered, the suction requirement is calculated according to:

\[ S_{\text{req}} = \frac{Q_{\text{tot}} - W'}{A_{\text{in}}} \]  

Figure 6.10 Back-calculations with best-fit approach
However, due to the use of water injection devices the tip resistance should be reduced to a certain extent because of the increased pore pressures (and hence reduces effective stresses). Figure 6.11 presents the measured suction during installation of the caisson up to 22 m penetration depth. The red curve shows the required suction if there is no reduction due to seepage and/or injection flow. The dashed curve shows the required suction for reduced tip- and inner resistance (best-fit curve). This reduction of resistance can be caused by the water injection devices, seepage flow through or along the plug or increased permeability due to deformed soil or uplift of the clay plug.

![Figure 6.11](image-url)

**Figure 6.11** Measured and back-calculated suction (with and without tip reduction)

At a penetration of 9.8 m the water injection devices were switched on and from this point the back-calculated suction is based on reduced tip and inner resistances. A best-fit suction requirement was obtained where the tip and inner resistances were assumed to reduce 50%, which is in accordance to the theory presented in Figure 2.10a. The reduction of the inner resistance might be justified by penetration of the skirt, which might affect the flow characteristics. From Figure 6.11 it can be noted that the best-fit curve shows good agreement with the measured suction.

6.2.2 Plug uplift back-calculations

Uplift is expected when the applied suction exceeds the plug uplift resistance, which is the case at penetration depth of 20 m (Figure 6.12). Plug uplift (or heave) was also mentioned in the corresponding paper of Tjelta et al. [1986] and estimated to be 1 m; 0.6 m due to displaced soil and 0.4 m due to applied suction. In order to assess the plug uplift potential a back-calculation of the plug heave can be made with the use of installation time which is presented in the paper.

The confined sand layer (9.8 - 17 m) is very silty, clayey with a clay content of about 10% (Appendix F.2) and is therefore assumed to be rather cohesive and less permeable. In fact the dense sand at 18 m depth is overlaid by a cohesive plug of thickness 17 meter. This assumption is incorporated for the plug contour line in Figure 6.12.

Considering the stratification of the soil conditions, there’s a potential for plug uplift. In order to assess the plug uplift criteria, the plug uplift contour is plotted together with the measured and estimated suctions in Figure 6.12. The contour line in Figure 6.12a includes the possible component
of reverse end bearing ($S_{upl,REB}$). From this it can be noted that the contour line does not cross the measured suction and therefore might be neglected. This observation is in accordance with the relative low installation-rate (1 m/hr), which is not beneficial for passive suction capacity.

![Graph](image)

(a) With reverse end bearing
(b) Without reverse end bearing

**Figure 6.12** Contributions of uplift components

This is confirmed by Figure 6.12b, where the contour line according to Senders [2008] shows good agreement with the encountered uplift. The contribution of reverse end bearing is excluded in Figure 6.12b, where only the contributions of self-weight ($S_{upl,weight}$) and shaft friction ($S_{upl,fric}$) according to the theory of Senders [2008] are presented.

**Fracture requirement**

From the subsequent theory in the previous chapters, a theoretical fracture-ratio can be back-calculated for this case. With rough estimations, it can be found that the fracture requirement is around 40%. In order to put this in perspective, a circular strip fracture of 1.5 m is required to install without uplift. This is supposed to be not realistic in practice, thus it can be concluded that cracking likely did not occur.

**Plug heave**

Back-calculations rely on the inflow of water in a defined time period and depend on permeability of the underlying sand and the applied suction. The latter can be directly related to the estimated suction and the suction to lift off the plug. However, the permeability of the sand is not defined and therefore an accepted range of sand permeability has been chosen. Since the underlying sand (20 - 24 m) is more silty, clayey, the permeability was chosen to be half of the permeability of the overlying dense sand (18 - 20 m).

An estimate of possible plug heave can be made according to the uplift theory of Senders [2008]. The paper of Tjelta et al. [1986] presented a plug heave of 0.4 m at target depth of 22.1 m, which is also in the range of solutions in Figure 6.13. The paper stated that plug heave was observed (by bottom clearance devices) in the last penetration stage, which is in accordance to the assumption.

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that plug heave occurs when the caisson penetrates the sand at 18 m. It can be remarked that the variation in sand permeability leads to substantial differences in estimated plug heave. A range of permeabilities for sand \((5 \times 10^{-4} - 1 \times 10^{-4} \text{ [m/s]})\) is chosen to determine possible plug heave and showed that the range of heave is 0.15 - 0.75 m for this case.

![Figure 6.13 Estimated heave for range of permeabilities](image)

However, if the range of permeabilities is chosen to decrease by a factor of 10, the plug heave estimate will also decrease by a factor of 10 and becomes 0.02 - 0.07 m. This clearly indicates the dependency of permeability and drainage length for plug heave. The installation time (close to penetration) also has influence on the plug heave estimate. If the time requirement for installation is increased by a factor of 2, the estimated plug heave will also increase by a factor of 2 and thus implies a linear dependency. Both examples demonstrate that plug heave is highly dependent on time-effects, i.e. permeability of the underlying sand and installation time.

### 6.2.3 Conclusions

Case B was analyzed by back-calculations of the suction requirement for reduced inner and tip resistances. From the measured suction it can be concluded that the tip- and inner resistance were reduced during installation, however it is uncertain whether this is caused by the water injection devices or seepage flow due to plug uplift.

Considering the presented data of the paper Tjelta et al. [1986] plug heave of 0.4 m was encountered during installation. Regarding the soil profile, a potential plug for uplift failure was determined and back-calculated for plug heave. The plug uplift contour line corresponds well with the recorded drop in suction pressure at penetration of 20 m, if no contribution of reverse end bearing was taken into account. In addition the plug heave was back-calculated by assuming a range of permeabilities to assess the amount of heave. This showed again good agreement with the measured heave; however it can be remarked that the variation in sand permeability leads to substantial differences in plug heave predictions.
6.3 Numerical calculations

In order to verify the theory obtained within previous analyses, numerical verification is done using finite element software. Plaxis is a commonly used finite element-software tool for geotechnical engineering, which provides a platform to assess complex conditions by solving them numerically. Within the previous analyses, it turned out that reverse end bearing might be mobilized prior to plug uplift. However, it is unknown to what extent and how long this additional ‘capacity’ contributes, which keeps the plug on its place during installation. From a practical point of view, it is interesting to assess the plug stability, by giving a prediction of the mobilized reverse end bearing capacity. In order to verify the theory in the most reliable way, the reference case of Chapter 2 has been taken, with the specification as presented in Appendix F.3 and Table 6.5.

<table>
<thead>
<tr>
<th>Table 6.5 Input-data for Plaxis calculations</th>
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<tbody>
<tr>
<td>Diameter [m]</td>
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<td>----------------</td>
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<td>10</td>
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</table>

According to the theory of Senders [2008], plug uplift for this case is likely to occur for pressure differences higher than 36 kPa. This value is based on an adhesion factor ($\alpha$) of 1 and excludes possible contributions of reverse end bearing. This is also implemented in Plaxis by setting the rigidity of the clay interface to 1. It is expected that for higher pressure differences, the effective stresses below the clay plug become zero and uplift should occur. In the subsequent paragraphs a further explanation of the uplift-mechanism in Plaxis is discussed.

**Purpose of calculation**

The purpose of the numerical verification is to gain insight in the time-dependent contribution of reverse end bearing. Prior to draw conclusions regarding this topic, first the uplift-mechanism should be identified with the use of Plaxis. This can be done by considering the vertical effective stresses just below the clay plug. If the uplift mechanism is recognized with the use of Plaxis, the contribution of reverse end bearing can be investigated.

6.3.1 Calculation method

Since Plaxis is based on continuum mechanics calculation procedures, it is not possible to model a gap-filling problem in time, which is applicable during plug uplift. It is possible to model the uplift-principle properly up to the moment that the plug will lift off. Beyond this point it is expected that plastic points or dilatancy effects will intervene, because in that case a continuum medium is maintained. The input-model is presented in Figure 6.14, with horizontal dimensions of 40 m and vertical dimensions of 20 m.

![Figure 6.14 Generated mesh of model](image-url)
The strategy of the modelling is to apply an increasing suction on top of the clay layer and monitor the dissipation of pore pressures in time just below the clay plug. Therefore the stress-points (see Figure 6.15) are chosen just below the clay plug as data-points. In addition data-points in the top of the clay plug are taken to compare the pressure difference of the clay plug. Further details about the calculation procedure are presented in Appendix F.3. The nodes $A - F$ are defined to back-calculate the deformation and pore pressures, nodes $K,L$ and $M$ are defined to back-calculate the (volumetric) strains. For this reason a double set of data-points just below the clay plug is implemented.

![Figure 6.15](image)

**Figure 6.15** Data-points to investigate behaviour of the plug

### 6.3.2 Modelling plug uplift

The chosen input model consist of an axisymmetric mesh which is refined close to the caisson skirt and clay plug. Inside the caisson an artificial soil-material has been added, which has no effective volumetric weight and therefore can be regarded as 'water'. This material is added to ensure a continuum medium.

The principle of applying suction on top of the clay plug is complex using Plaxis. Therefore it is chosen to apply a back-pressure on the surrounding soil, instead of applying suction inside the caisson. This alternative has already been proven to model plug uplift, by the experiments that were conducted in Chapter 3. It showed that for sufficient pressure difference the clay plug lifted off the sand and uplift occurred due to the 'back-pressure'. For this reason the similar principle is adopted for simulations using Plaxis.

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<tbody>
<tr>
<td>0</td>
<td>21 (initial)</td>
<td>21</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
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<td>20</td>
<td>40</td>
<td>200</td>
<td>-100</td>
<td>200</td>
</tr>
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</table>

Within the Plaxis software, the differential pressure is obtained by setting the inner 'water'-cluster of the caisson to *dry cluster*, while the surrounding soil is subjected to an phreatic head. In the subsequent calculations, the phreatic head is increased to ensure an increasing pressure difference.
over the clay plug. As a result, the differential pressure on the clay plug is gradually increased, which is similar to installation-practice.

Table 6.6 presents an overview of the pressure increments with time. These values are based on installation-data in practice according to Audibert et al. [2003]. The ‘penetration-rate’ is kept constant, by taking a suction increment for a fixed time period, as indicated in the right column of Table 6.6. The penetration-rate is defined as suction/time, which is found to be around -1 kPa/min.

It should be noted that the increase of phreatic head is also applied to the underlying sand of the surrounding soil, to avoid long-term consolidation over the top clay layer. In addition to this generic example, three cases are considered to investigate the influence of installation-rate and cracking of the clay plug. In order to investigate the time-dependency of installation, the time-period to equalize the pressure is doubled, which makes the installation-rate be halved. In the subsequent analysis this case is described by Case - 2t.

![Figure 6.16](image_url) Cases to model cracking of clay plug

The remaining two cases on plug cracking are divided in cases with a sand-filled crack on different locations. One case considers a crack along the skirt (Case - crack), while the other case considers a crack more in the centre of the clay plug (Case - crack-mid). As can be noted in Figure 6.16, the crack in Figure 6.16b is wider than the crack in Figure 6.16a. This is because the crack is modelled to be an circular strip and therefore a smaller radius implies a wider gap to ensure the same fracture-ratio. Both cases are based on a fracture-ratio of 10 %, which is in accordance to the theory of Chapter 4.

### 6.3.3 Results

This subsection discusses the results on several components, which are involved during plug uplift. Subsequently the results of plug heave, reverse end bearing, installation-rate and cracking of the clay plug are presented.

#### Plug heave

As stated in the introduction of this chapter, first the plug uplift mechanism should be identified. This is done by considering the effective stresses just below the clay plug. Once the effective stresses become zero just below the clay plug, it is considered that the resistances against uplift (self-weight and skin friction) are overcome. The determination of the time when the effective stresses become zero is shown for the initial case in Figure 6.17.

From Figure 6.17 it can be noted that the effective stresses just below the clay plug become zero after 55 minutes, which corresponds with an applied suction of 55 kPa. In addition, the volumetric strains are analyzed just below the clay plug. From Figure 6.17b it can be noted that from the moment when the effective stress becomes zero, the volumetric strain of the underlying sand increases.
From the findings of Senders [2008] it was found that the expected suction for plug uplift is around 36 kPa. However, from the calculations with Plaxis it is found that at a suction of 55 kPa, the uplift initiates. This is based on the fact that after 55 minutes the effective stresses become zero, in combination with an increasing volumetric strain just below the plug at the same moment. This is clearly an overestimation of the prediction according to Senders [2008]. However, this excludes contributions of reverse end bearing. Further discussion about this is presented in the next paragraph.

Reverse end bearing

The second analysis is done on the generated capacity of reverse end bearing prior to uplift of the clay plug. The magnitude of reverse end bearing is highly time-dependent due to dissipation of negative excess pore pressures. The dissipation in time is interesting for installation-practice, since this will describe the loss of an additional resistance against plug uplift. Figure 6.18 shows that for data-points D and E, some negative excess pore pressures are found during installation. The vertical axis presents the normalized excess pore pressure and the black dotted-lines indicate the applied pressure difference over the clay plug.

As presented in section 5.1, the mechanism of reverse end bearing relies on suction mobilization and suction release. This mechanism is also indicated in Figure 6.18 once an increase of the differential...
pressure is applied at $t = 0$, $10$, $20$, $50$ and $100$ minutes. For data-points D and E it can be seen that the excess pore pressures rapidly decrease due to the applied ‘suction’ (the model assumes a back-pressure). After this mobilization of suction, it can be noted that the pressure increases to the applied suction condition (suction release) which are presented by the black dotted-lines. For both data-points it can be remarked that some negative excess pore pressures are generated due to the pressure difference after 50 minutes and 100 minutes. After 55 minutes the generated mobilization and release of suction has a duration of about 12 minutes, as can be noted in Figure 6.18. In this time period the increment of suction equals -12 kPa, since the installation-rate is -1 kPa/min. This implies that the applied suction is increased with 12 kPa, which results in a suction of 48 kPa. This value for plug uplift is more or less found with the Plaxis results (55 kPa), which indicate that the theory of Senders [2008] is an underestimation of the actual plug-stability.

By varying the permeability of the underlying sand, the contribution of the reverse end bearing in time can be assessed (Figure 6.19). The permeability of the sand is lowered by a factor 2, which makes the permeability equal to $k = 5 \times 10^{-6}$ m/s. It shows that for a decrease of the permeability by a factor of 2, the time period of suction mobilization and release is approximate twice the duration compared to the higher permeable condition. Figure 6.19 also shows that for lowering the permeability, the time period of reverse end bearing capacity increased. Figure 6.19b illustrates that for the case with lower permeable sand the suction mobilization and release takes approximate 47 minutes, while for the initial permeability it takes 20 minutes. The permeability of the underlying sand clearly influences the dissipation of excess pore pressure during pressure differences acting over the clay plug. Therefore the development of reverse end bearing in time can be related to the hydraulic characteristics of the underlying sand.

**Installation-rate**

The theory can be verified by assuming an installation-time which is doubled, if no contribution of reverse end bearing is considered. In theory this will also double the amount of plug heave, since this is linear according to Chapters 4 and 5. The theory describes that the estimated plug heave will be higher for lower installation-rates. This case assumes that the installation-time is doubled, which in theory will also double the amount of plug heave.

From Figure 6.20 it can be seen that for the Case - 2$t$ the heave is approximate 40 % higher for a slower installation-rate. If the downward displacements of the caisson are also taken into account it can be concluded that for the initial case an additional displacement of 7 cm was calculated. For the for Case - 2$t$ the displacement of the caisson was 6 cm, which makes the relative displacements...
Verification of the theory

Figure 6.20  Calculated plug heave for lower installation-rate

of the plug 10 cm and 6 cm. This shows that the difference in relative displacement of the plug is around 50 % and thus in accordance to the theory.

Cracking

The influence of cracking of the clay plug is investigated by implementing two cases where a small vertical gap of underlying sand material is considered (see Figure 6.16). From experimental research is was found that a cracked plug will limit the plug heave due to the increased permeability of the clay plug. As a result seepage flow can develop through the underlying sand and succesfull installation can be achieved. However, according to the theory in section 4.1 it also shows that the amount of cracking (fracture-ratio) is highly unlikely and uplift of the plug will intervene first.

Figure 6.21  Calculated plug heave for cracked plugs

From Figure 6.21 it can be noted that calculated plug heave is lower for cases with a cracked plug (10 % of cracks). Despite the small amount of uplift, it can be noted that uplift of data-points D and E is lowered with 20 %. This reduction is lower than expected from the theory, but is likely being caused by the limitations of the used software.
6.3.4 Discussion & conclusions

Plug stability is modelled by considering vertical effective stresses that become zero for sufficient pressure difference acting over the plug. From data-points just below the clay plug, it turned out that volumetric strains developed once uplift of the clay plug developed. On the one hand this is a indication of plug uplift, on the other hand it is a limitation of the used software, since it based on continuum mechanics and therefore does not model a water gap below the plug.

If this limitation is recognized, uplift of the clay plug can be identified by considering the vertical effective soil stresses just below the clay plug. Once these become equal to zero, it can be assumed that lift-off of the plug (i.e. plug instability) has occurred. In addition, dilatancy of the underlying sand was encountered at the same moment, which also indicates plug uplift. From experiments in Chapter 3, it was already found that the condition with back-pressure can be used to generate plug uplift.

The principle of reverse end bearing relies on the dissipation of excess pore pressure in time, which are identified for this Plaxis model. The suction mobilization and suction release are identified during increasing pressure difference for a certain time period. As presented in the theory, the dissipation of the reverse end bearing capacity is highly dependent on the permeability of underlying sand and the pressure difference acting over the plug (applied suction). Quantification of the reverse end bearing capacity is desired for assessment of the plug stability in time.

Both from theory and experiments it was found that a higher installation-rate will reduce the amount of possible plug heave. The results of Plaxis show that a reduction of installation-rate by a factor 2 results in increase of residual heave by a factor 1.7. This dependency is nearly linear, which is in accordance with the theory. It is therefore strongly recommended to consider this fact for installation practice.

Cracking of the clay plug was investigated within the framework of this research by analytical calculations and experimental tests. Both showed that a cracked plug will increase the permeability of the plug, which will result in a pressure drop inside the caisson and increased seepage flow in the underlying sand. The former contribution will limit the plug heave since the driving force to lift-off the plug is lowered. The seepage flow will induce reduction of tip resistance, if the skirt is penetrated into the underlying sand. Cracking of the plug will therefore reduce the residual plug heave. From Plaxis results it is found that indeed the amount of plug heave is lower than for a solid plug, however substantial reduction is not recognized. It is therefore recommended to perform more research to the effects of cracking on the plug stability. Quantification can be achieved by laboratory tests with suction cells or extension triaxial tests on layered soil samples.
6.4 Conclusions

For both field cases water injection devices were implemented as mitigating measure to decrease the tip resistance in dense sands overlaid by less permeable material to a certain extent. This is not a proven technology, and back-calculations did not clearly indicate the benefit of these devices. The reduction of soil resistance and hence lowering of suction requirement could also be caused by the seepage flow which was induced by uplift of the plug.

Plug uplift prediction

Plug uplift was observed physically in Case B, where the uplift suction requirement according to the uplift theory of Senders [2008] was calculated. Considering the presented data of the paper (Tjelta et al. [1986]) plug heave of 0.4 m was encountered during installation. Regarding the soil profile, a potential plug for uplift failure was determined and back-calculated for plug heave. The plug uplift contour line corresponds well with the recorded drop in suction pressure at penetration of 20 m.

From finite element calculations it was found that uplift of the plug resistance was underestimated and lift-off initiated for higher suction (or later when the reverse end bearing was dissipated). The threshold of plug stability was modelled by considering vertical effective stresses that become zero for sufficient pressure difference acting over the plug. From data-points just below the clay plug, it turned out that volumetric strains developed once uplift of the clay plug developed. Considering the experiments in Chapter 3, it was already found that plug uplift can be generated by using back-pressure instead of suction.

Cracking of the plug

For both cases a theoretical fracture-ratio can be back-calculated, where a circular strip as 'fracture-gap' is implemented. The surface of this gap is according to the theory of Chapter 4 and theoretically ensures plug stability by increasing the permeability of the clay plug. For both cases it can be concluded that plug stability due to cracking is highly unlikely, since the required fractures were unrealistic high (i.e. 10 - 20 %). However, possible contribution of reverse end bearing capacity was not incorporated.

Cracking of the clay plug was investigated within the framework of this thesis by analytical calculations and experimental tests. Both showed that a cracked plug will increase the permeability of the plug and hence reduce the amount of plug heave. From the Plaxis results it is found that indeed the amount of heave is lower than for a solid plug, however substantial reduction is not recognized. It is therefore recommended to perform more research on the effects of cracking on the plug stability. Quantification can be achieved by laboratory tests with suction cells or extension triaxial tests on layered soil samples.

Estimation of plug heave

The amount of plug heave was back-calculated by taken into account a range of permeabilities. This showed good agreement with the measured heave, however it can be remarked that the variation in sand permeability leads to substantial differences in plug heave predictions. As well the installation time is linearly dependent on the plug heave estimation.

The results of Plaxis show that a reduction of installation-rate by a factor 2 results in an increase of residual heave by a factor of 1.7. Despite the difference in linearity, it still can be concluded that the plug heave estimate is highly dependent on time-effects, i.e. permeability of the underlying sand and
Installation effects of suction caissons in non-standard soil conditions

Installation time. It is therefore strongly recommended for installation practice to determine these parameters properly and to assess the installation time carefully.

**Contribution of reverse end bearing**

The contribution of reverse end bearing as additional resistance against plug uplift relies on dissipation of negative excess pore pressures in time. Considering Case A it was found that the installation time is relatively short (i.e., approximate 1 hour) and the permeability of the underlying silty sand was relatively low \((1 \times 10^{-6} \text{ m/s})\). Therefore additional capacity of reverse end bearing was justified in the back-calculations, which shows agreement with the drop in measured suction. The installation time of Case B was relative slow (installation rate of 1 m/hr) and thus no capacity of reverse end bearing was incorporated. Back-calculations showed that a good approximation of the uplift was obtained by excluding the contribution of reverse end bearing.

The principle of reverse end bearing was also recognized for the Plaxis model, where the suction mobilization and suction release were identified during increasing pressure difference for a certain time period. As presented in the theory, the dissipation of the reverse end bearing capacity is highly dependent on the permeability of the underlying sand. Predictions of the magnitude of reverse end bearing are to be further researched.
Chapter 7

Conclusions & Recommendations

This Chapter presents the conclusions and recommendations on installation effects of suction caissons in non-standard soil conditions, especially where sand is overlaid by clay. Section 7.1 presents the conclusions of the research performed. In addition the recommendations for further research and practical implementation are presented in Section 7.2.

7.1 Conclusions

Nowadays suction caissons are extensively used for anchoring large offshore installations at great depths to the seafloor. Installation of suction caissons in coarse grained soils (with a high permeability) is achieved by induced seepage flow in the soil. On the other hand for installation in fine grained materials with a low permeability (silt or clay), no seepage flow will occur instead a differential pressure used for installation will be mobilized. As a result the installation in layered soils, especially sand overlaid by clay, is expected to be more challenging since the reduction of tip resistance is restrained from flow restrictions in the clay layer. It is unclear to what extent the upper low permeability layer will impact the seepage flow and thus the desired tip resistance reduction. In addition to this, the plug stability should be assessed to avoid excessive plug heave.

Installation in layered soils

1. The favourable effect of water injection devices as mitigation measure to decrease the tip resistance of the suction caisson in dense sands is not fully supported by back-calculations of two field cases. The apparent reduction of soil resistance and hence lowering of suction requirement could also be caused other mechanisms (i.e. plug uplift, cracking of the plug or reverse end bearing contributions) rather than only injection.

2. The design approaches for homogeneous sands (Houlsby and Byrne [2005] and Andersen et al. [2008]) were successfully adjusted for calculation of the resistances in layered soils, where sand is overlaid by clay. The most important adjustment was to correct the critical suction for the homogeneous sandy material. The critical suction was horizontally translated for shallow penetration in the sand, according to the solution of Senders [2008]. It should be noted, with regards to the plug stability, that possible contribution of reverse end bearing is not incorporated.

Plug response during installation

1. Two different types of plug failure mechanisms are postulated for installation of suction caissons in sand overlaid by clay; plug uplift and plug cracking. Considering several cases with D/z ≤ 6, it was shown that plug uplift is more likely to occur during installation. However, there is an uncertainty at the transition point regarding failure mechanism (i.e. cracking or uplift). From
schematization of a bending circular plate, an estimation of the cracking failure could be made and showed that plug cracking is expected for $D/z > 10$, whereas uplift was expected for lower $D/z$-ratios. This was supported by a practical case where uplift was observed at a lower $D/z$-ratio (Case B). Comparing these two failure mechanisms it can be concluded that the failure mechanism depends on the dimensions of the caisson and clay layer.

2. Experiments on plug cracking showed failure for high $D/z$-ratios, but also showed a pressure-rate dependency. This corroborates data from the literature (Senders [2008] and Watson et al. [2006]), where slow penetration rates highly contributes to uplift of the clay plug. Additionally, high pressure-rates (fast installation) contributed to cracking of the clay layer during testing. Therefore, the failure mechanism of the clay plug, i.e. plug uplift or plug cracking, depends on the build up of suction pressure as a function of the desired installation-rate.

**Assessment of plug stability**

1. One of the scenarios to maintain a stationary plug during penetration of the suction caisson is to increase the permeability in the upper clay layer (i.e. cracking). This results in adequate tip resistance reduction in the sand and a reduced stress gradient across the clay-layer from seepage flow such that plug uplift is prevented. The cracking-requirement for installation can be calculated, by introducing a fracture-ratio. However, the resulting calculated values for cracking are not realistic for typical installation practice, since at least 10 % of cracking area is required for clayey soils. This conclusion was supported by two field cases (Chapter 6). It can therefore be concluded that due to cracking only, it is highly unlikely that plug stability can be maintained during installation. However, it should be remarked that no contribution of reverse end bearing was considered.

2. When uplift can be accommodated, the relation between the plug velocity and the installation rate should be assessed. Analytical analysis shows that for slow installation the total amount of uplift is higher than for fast installation. The governing component that determines the amount of uplift of the plug is the magnitude of seepage flow in the underlying sand layer (i.e. permeability). Two case studies and numerical analysis show that the reduction of installation-rate results in more plug heave. Furthermore, the plug heave estimate is strongly dependent on time-effects, i.e. permeability of the underlying sand and installation time.

**Contribution of reverse end bearing**

1. For sands, the permissible capacity from reverse end bearing depends on the time required to dissipate the negative excess pore pressures. This time requirement, defined as the time lag, shows that for a typical installation period of 1 - 6 hours and hydraulic conductivities in the sand layer lower than $1 \times 10^{-5}$ m/s some capacity due to delayed inflow of water can be considered. It should be remarked that the chosen range of permeability has a great impact on the time requirement for dissipation.

2. The underlying components for uplift of the plug were modelled using a spring-dashpot rheological model. If the applied suction above the plug will initiate upward movement of the plug, this movement will be restricted by the inflow of water below the plug. The difference in upward movement between both components show similar response compared with mobilization and release of reverse end bearing (i.e. dissipation of negative pore pressures). The model indicates that seepage flow increases rapidly for more permeable soils and therefore the time-period for generating reverse end bearing capacity is significantly reduced. The mechanism of reverse end bearing and the time dependency of this mechanism was identified in the installation data of two field cases. A lower installation-rate seemed detrimental for the reverse end bearing
capacity. The principle of reverse end bearing was successfully modelled by the finite element package PLAXIS. In the latter the suction mobilization and suction release are identified during increasing pressure difference over a certain time period. As shown by the analytical model, the dissipation of the reverse end bearing capacity is highly dependent on the hydraulic properties of the underlying sand.
7.2 Recommendations

The theoretical analysis and the experimental work have shown the geotechnical mechanisms involved for suction caisson installation in layered soils. This section presents recommendations for further research and practical suggestions for suction caisson installations in sand overlaid by clay.

1. Preferred approach

Commonly two approaches are used to predict the soil resistance during penetration: (1) the effective stress approach and (2) the CPT-approach. The first is an indirect method relying on the friction angle and the effective unit weight of the soil, the second is a direct correlation with the cone resistance. From an engineering point of view, the CPT-approach is preferred, because the measured resistance is directly related to penetration resistance by a single factor. This reduces the uncertainty in multiple input-parameters and avoids unnecessary elaborate calculations. In addition, the stress-path of a CPT is comparable to the penetration of the caisson skirt, which incorporates similar soil response in terms of (un-)drained behaviour.

2. Cracking of the plug

Cracking of the clay plug was investigated within the framework of this thesis by analytical calculations and experimental tests. Cracks increase the permeability of the plug and hence reduce the risk of plug heave. The analyses indicated that for less than 10% cracks sufficient seepage was achieved to limit plug uplift. Additionally, FE analysis showed that the plug heave of a cracked plug is lower than for a solid plug. However, substantial reduction was not shown. It is therefore recommended to perform additional research on the effect(s) of cracking on the plug stability.

3. High installation-rate

When uplift can be accommodated to a certain extent, the relation between the plug velocity and the installation rate showed that a higher installation-rate results in lower plug uplift velocity and thus less plug heave in the same time period. The plug heave is highly dependent on time-effects (permeability of the underlying sand and installation time). In order to minimize plug heave, minimization of installation time (e.g. high pump capacity) and accurate (in-situ) determination of the hydraulic properties is highly recommended.

4. Minimize the risk of installation-stop

When the caisson is halted during installation (e.g. due to breakdown of the pump), the under pressure below the clay plug is maintained and thus seepage flow will continue. A sudden installation-stop (once plug uplift has initiated) should therefore be prevented, since this will cause ‘maximal’ uplift of the plug. However, the plug will settle due to self-weight of the plug once the pressure difference is decreased.

5. Contribution of reverse end bearing

Nowadays contribution of reverse end bearing is not often taken into account for layered soils. The research presented shows that reverse end bearing capacity generally depends on the dissipation of negative excess pore pressure in time. Since there is a time-effect involved, the capacity of reverse end bearing greatly depends on the permeability of the underlying sand, drainage length and installation time. For design purposes it is therefore recommended to maximize the installation-rate and accurately determine the permeability of the underlying sand layer (e.g. by dissipation tests). In cases of local shallow gas reservoirs, the dissipation of negative pore pressures will develop quicker and thus the contribution in time will decrease.

6. Quantification of reverse end bearing

The theoretical analysis and numerical calculations within this thesis highlighted the possible contribution of reverse end bearing in layered soil conditions. Further quantification can be achieved by laboratory tests with suction cells or extension triaxial tests on layered soil samples.
Bibliography


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Appendices
Appendix A

Implementation of effective stress and CPT-approaches

A.1 Calculations according to effective stress-approach

For the effective stress-approach (Houlsby and Byrne [2005]) the method assumes a linear reduction in internal friction and end bearing and an increase in outside friction when suction is applied. A pore pressure factor \( a \) is calculated which accounts for the 3D-effect of the suction caisson.

\[
\frac{d\sigma'_v}{dz} = \gamma'_s + \frac{\sigma'_w}{Z_o} - \left(1 - a(z)\right)\frac{S}{z} \tag{A.1}
\]

\[
\frac{d\sigma'_i}{dz} = \gamma'_s + \sigma'_w + a(z)\frac{S}{z} \tag{A.2}
\]

In static conditions the internal pressure is typically larger, but with increasing pressure the external stress level will become larger. The stress level at the tip is therefore calculated by taking the difference into account;

\[
\sigma_{\text{red}}' = \begin{cases} 
\sigma'_w N_q + \gamma' w_i N_f, & \text{in case } (\sigma'_w - \sigma'_n) \geq 2 w_i N_f / N_q \\
\sigma'_w N_q + \gamma' w_i N_f, & \text{in case } (\sigma'_w - \sigma'_n) \geq 2 w_i N_f / N_q \\
\sigma'_w N_q + \gamma' \left( w_i - \frac{2x^2}{w_i} \right) N_f, & \text{in other cases}
\end{cases}
\]

with

\[
x = \frac{w_j}{2} + \frac{(\sigma'_w - \sigma'_n)N_q}{4 \gamma' N_f}
\]

The reduced stress is presented in the equation below; as well the reduced force can be determined.

\[
W_{\text{tot,Houlsby,FLOW}}(z) = \alpha_{\text{out}}(z)Q_{\text{outside}} + \alpha_{\text{in}}(z)Q_{\text{inside}} + \alpha_{\text{tip}}(z)Q_{\text{tip}} \tag{A.3}
\]

\[
\alpha_{\text{out}}(z) = [1 + a(z)] S_{\text{red}} A_i \tag{A.4}
\]
\[
\alpha_{\text{in}}(z) = [1 - a(z)]S_{\text{red}}A_i
\] \hspace{1cm} (A.5)

\[
\alpha_{\text{tip}}(z) = f(\alpha_{\text{out}}, \alpha_{\text{in}})
\] \hspace{1cm} (A.6)

### A.2 Calculations according to CPT-approach

The CPT-approach of DnV is not used to predict the changes in soil resistance during suction assisted penetration. However, Andersen et al. [2008] presented a calculation model based on critical pressure, suction pressure, self-weight, penetration depth and wall thickness. Senders [2008] developed a best-fit equation to describe the dependence of the critical suction number \((S_{N,cr} = p_{\text{crit}}/z\gamma')\).

\[
S_{N,cr} \approx 0.16 \left( \frac{z}{D} \right)^{0.5} + 3 \left( \frac{k_{\text{fac, thin}}}{k_i} \right) \left( \frac{z}{D} \right)^{0.5} \text{ for } \frac{z}{D} > 0
\]

The permeability-ratio \((k_{\text{fac, thin}} = k_i/k_a)\) is calculated by taking a thin slice directly next to the caisson wall, which is assumed to have a different permeability during suction. With the ratio \(R_{c,\text{Flow}}/R_c\) the following equation can be used to calculate the reduction in overall soil resistance:

\[
Q_{\text{tot,NGI,FLOW}}(z) = \left( 1 - \frac{S_{\text{req}}}{S_{\text{crit,NGI}}} \right)^{\frac{1}{\pi\gamma}} \left[ Q_{\text{outside}}(z) + Q_{\text{inside}}(z) + Q_{\text{tip}}(z) \right]
\] \hspace{1cm} (A.7)

### A.3 Correlation of effective stress- and CPT-approaches

For the model the following parameters where used; \(k_f = 0.0015\), \(k_p = 0.45\) (most probable) and \(k_f = 0.003\), \(k_p = 0.6\) (highest expected) \(q_c = 20\) MPa, \(s_u = 50\) kPa, \(\alpha = 0.4\). From Figures A.1 and A.1 it can be seen that the correlations of the Houlsby-method are close to the highest expected estimates.
Figure A.1  Estimated inner-, outer, tip- and total resistances according to Houlsby and Byrne [2005] and Andersen et al. [2008]
Appendix B

Determination of end-bearing coefficient

For pure vertical loading the inverse bearing capacity below skirt tip can be calculated with a bearing capacity factor ranging from $N_c = 6.2$ at the surface to $N_c = 9$ at depths greater than 4.5 times the diameter, see Figure B.1. The expression for the $N_c$-factor in Figure B.1 is;

$$N_c = 6.2 \left( 1 + 0.34 \arctan \left( \frac{z_i}{D} \right) \right) \quad \text{valid for} \quad \frac{z_i}{D} = 4.5$$

(B.1)

which includes a shape factor $s_c = 1.2$.

\[\text{Figure B.1} \quad \text{Bearing capacity factor $N_c$ vs normalised depth $z_i/D$ for circular foundation with pure vertical loading (Fig 4-4 from DnV [1992])}\]
Appendix C

Prediction method to determine cracking of the plug

Calculation of plug cracking due to bending moment;

\[ T_{\text{max,clamped}} = \frac{(1 + \nu)p_{\text{crack}}D^2}{64} [kNm/m] \] (C.1)

\[ T_{\text{max,hinged}} = \frac{(3 + \nu)p_{\text{crack}}D^2}{64} [kNm/m] \] (C.2)

Source: [http://www.me.ust.hk/~meqpsun/Notes/Chapter3.pdf](http://www.me.ust.hk/~meqpsun/Notes/Chapter3.pdf)

Soil resistance per unit width;

\[ T_{\text{max,clay}} = s_u z_{\text{clay}} A = s_u z_{\text{clay}}^2 [kNm] \] (C.3)

Equation C.3 is according to Thusyanthan et al. [2007], with \( A = z_{\text{clay}} B \) and \( B = 1 \) m.

Range of solutions; (i.e. first clamped then hinged)

\[ \frac{64s_u z_{\text{clay}}^2}{\pi D^3(3 + \nu)} < p_{\text{crack}} < \frac{64s_u z_{\text{clay}}^2}{\pi D^3(1 + \nu)} \] (C.4)
INSTALLATION EFFECTS OF SUCTION CAISSONS IN NON-STANDARD SOIL CONDITIONS

R.H. Romp

M.Sc. Thesis
Appendix D

Installation of a suction caisson

D.1 Steady-state suction

Depending on the permeability of the clay plug, steady-state suction can be reached in that case. The pressure increment due to seepage flow equals the pressure decrease due to suction as can be seen in Figure D.1a.

![Diagram of steady-state suction](image)

(a) Schematization of steady-state suction

![Graph of suction pressure vs. time](image)

(b) Reduction of underpressure due to seepage flow (z = 2 m)

**Figure D.1** Illustration of steady-state suction during installation

Figure D.1b shows that steady-state suction is reached within 1 hour for soil with permeability of $10^{-3}$ m/s (no volumetric strain or dilatancy effects). It can also be remarked that for permeabilities lower than $10^{-5}$ m/s (clays) steady state suction will not be reached for common installation periods.
D.2 Elastic plug heave

The proportion of the caisson wall that is accommodated by the flow of soil inwards into the caisson or outwards is discussed Randolph and Gourvenec [2011]. He presented that a 50:50 split during self-weight penetration (Figure D.2 - left), but a 100:0 split during suction assisted penetration (Figure D.2 - right).

Despite the stage of installation (self-weight or suction assisted), it can be remarked that there will be a contribution of internal plug heave due to the installed caisson. In case of sand overlaid by clay, there will be limited reduction at the tip and therefore limited inward flow of the sand. A 50:50 split is taken for layered soil, both for self-weight penetration as for suction assisted penetration. By assuming elastic behavior of the soil, the amount of plug heave depends on the penetrated volume of caisson.

\[
\Delta V_{\text{caisson}} = \Delta V_{\text{heave}} = 0.5A_{\text{tip}}\Delta z = \frac{h_{\text{heave}}\pi D^2}{4} \tag{D.1}
\]

\[
h_{\text{heave}} = \frac{2A_{\text{tip}}\Delta z}{\pi D^2} \tag{D.2}
\]

From equation D.2 it can be seen that the amount of plug heave linearly relates to the penetration depth (z). If the tip-surface is estimated as the circumference times the thickness (t), the heave can be estimated by;

\[
\frac{h_{\text{heave}}}{z} \approx \frac{2t}{D} \tag{D.3}
\]
D.3 Procedure to determine fracture-ratio

1. **Determine soil-resistances**
   Modified Houlsby & Byrne and DnV-methods are implemented, according to respectively effective stress- and CPT-approaches.

2. **Correlate effective stress- and CPT-approaches**
   For the model the following parameters were used; \( k_f = 0.0015 \) and \( k_f = 0.003 \), \( k_p = 0.45 \) (most probable) and \( k_f = 0.6 \) (highest expected) \( q_c = 20 \) MPa, \( s_u = 50 \) kPa, \( \alpha = 0.4 \).

3. **Calculate \( S_{crit} \)**
   According to the methods presented, a critical suction pressure should be determined. This critical suction will be the upper bound of the applied suction. The difference between the approaches to calculate the critical suction can be found in Figure 2.9.

4. **Calculate \( S_{red} \)**
   According to the methods discussed in previous section, the required suction for installation can be calculated. This suction is sufficient to induce a seepage flow in the underlying sand, and thus should be applied just below the clay plug. The true suction in the can (\( S \)) is calculated in the next step.

5. **Back-calculate \( k_{crit} \)**
   The true suction \( S \) (which is induced by the pump) is limited by the critical suction in case of plug uplift failure. This upper boundary for the pressure-ratio \( S_{crit}/S_{red} \) is then back-calculated to the critical permeability (\( k_{crit} \)) according volume continuity, which implies that the specific flow through the sand have to be equal of that through the clay plug. The permeability for the underlying sand is assumed to be \( k_{sand} = 1.0 \times 10^{-4} \) m/s.

6. **Determine \( A_o/A_{base} \)**
   The critical permeability can be expressed in terms of a fracture-ratio. This presents the ratio of cracks in the clay, which is required for installation in layered soil.
Appendix E

Modelling on plug stability

E.1 Determination of time lag

Rate of inflow at time t, based on theory of Hvorslev [1951]

\[ Q_{in} = k \frac{S}{\gamma_w} A_i \quad (E.1) \]

Volume of inflow in time \( dt \);

\[ Q_{in} dt = A_{in} dz \quad (E.2) \]

And;

\[ \frac{\gamma_w dz}{S} = k \frac{A_i}{s} dt \quad (E.3) \]

The total flow for equalization of pressure difference is defined by;

\[ V = A_{in} \frac{S}{\gamma_w} \quad (E.4) \]

And;

\[ T = \frac{A_{in} S}{Q_{in} \gamma_w} = \frac{s}{\bar{k}} \quad (E.5) \]
E.2 Schematization of plug uplift

Pressures and volumes in time:

\[ S_1 = S_{\text{plug,uplift}} + \Delta S \]

- **Suction** \( S_1 \) applied during installation
- **Differential pressure on plug increases**
- **Attraction of water due to** \( S_2 \)
- **Inflow of water due to** \( S_2 \)

\[ \Delta s = \frac{1}{2}a\Delta t^2 \]

- **Reduction of** \( S_1 \)
- **Reduction of** \( S_2 \)

\[ F_{\text{res}} = ma \]

\[ V_1 \text{ decreases} \]

\[ V_2 \text{ decreases} \]

\[ S_1 > S_{\text{plug,uplift}} \]

\[ S_1 < S_{\text{plug,uplift}} \]

\[ S_2 > S_{\text{plug,uplift}} \]

\[ S_2 = S_{\text{plug,uplift}} \]

\[ S_2 > 0 \]

\[ S_1 > 0 \]
Appendix F

Parameters for verification

F.1 Case A: Curlew FPSO

Figure F.1 Parameters for calculations of Curlew case (Case A)
F.2 Case B: Gullfaks C

Figure F.2 Soil data of Gullfaks C (Case B)

Figure F.3 Clay content of Gullfaks C (Case B)
F.3 Plaxis calculations

Material model

For both soil types (clay and sand) the Mohr-Coulomb model is taken as first approximation. According to the Plaxis 2D manual it can be found that the Mohr Coulomb model is based on a first order approximation of soil behaviour and includes a limited number of features. Care must be taken in undrained conditions, since the effective stress path may not be realistic. Alternatively, the model may be used with friction angle $\phi$ set to 0 and the cohesion set to $s_u$, to enable a direct control of undrained shear strength.

Drainage type

Drained analysis are appropriate for conditions when the permeability is high, the rate of loading is low and short term behaviour is not of interest. Undrained analysis are appropriate for low permeability soils, fast rate of loading and long term behaviour conditions. A suggestion by Vermeer and Meier [1998] for deep excavations presents undrained analysis for $T < 0.10$ and drained analysis for $T > 0.40$ according to equation F.1.

$$T = \frac{k E_{oed} t}{\gamma_w D^2}$$  \hspace{1cm} \text{(F.1)}

$k = \text{Permeability \ [m/s]}$

$E_{oed} = \text{Oedometric modulus \ [kPa]}$

$\gamma_w = \text{Unit weight of water \ [kN/m^3]}$

$D = \text{Drainage length \ [m]}$

$t = \text{Installation time \ [s]}$

$T = \text{Dimensionless time factor \ [-]}$

The parameter D represents the drainage length and is calculated according to the solution of Senders [2008], which is presented in equation 2.15. Within the framework of this thesis it can be found that drained analysis are applicable for the underlying sand, whereas undrained analysis for the clay material is appropriate. Since the sand material is considered to behave drained, build-up of excess pore pressures are not considered. However, the negative excess pore pressures are of interest, since reverse end bearing is considered. Using the Mohr-Coulomb model with the undrained (A) method overestimates the undrained shear strength of soft clays. In order to use $s_u$ as an input parameter it is preferred to use the undrained (B) method.

Calculation procedure

Classical mode is based on Terzaghi, advanced mode base on Bisshop, which takes into account suction for unsaturated soils. The latter is not applicable since fully saturated soils are considered, therefore classical mode has been implemented for calculations.

Furthermore the Consolidation (EPP)-calculation procedure is adopted to analyse the development of dissipation of excess pore pressures in a saturated clay-type soil as function of time.

Pore pressures

Pore pressures in stress points and external water pressures at model boundaries are updated during the calculation according to the deformed model boundaries and the displaces position of stress points. Basis for the update of water pressures is the general phreatic level and the cluster phreatic levels. In this way, the buoyancy effect of soil that is submerged below the phreatic level is taken into account.
Material properties

![Material properties of Sand (general)](image)

**Figure F.4** Material properties of Sand (general)
**Figure F.5** Material properties of Sand (parameters)
Figure F.6  Material properties of Sand (flow parameters)
Figure F.7  Material properties of Clay (general)
Figure F.8 Material properties of Clay (parameters)
Figure F.9  Material properties of Clay (flow parameters)
Printscreens of groundwater head due to applied suction

(a) Step 1

(b) Step 2

Figure F.10 Calculation steps 1 and 2 for initial case
Figure F.11 Calculation steps 3 - 5 for initial case