Design Aspects of Suction Caissons for Offshore Wind Turbine Foundations

Aspects de conception des caissons d'aspiration pour les fondations de turbines éoliennes en mer

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ABSTRACT: This paper provides an introduction to the geotechnical design of suction caisson foundations for *Offshore Wind Turbine* (OWT) foundations. It summarizes the experience gained in a number of projects from across the world and proposes a guidance for the design of future projects. The paper is structured in a logical manner; the first section introduces the general design approach of suction caisson foundations, whereas the individual design aspects are discussed in detail in the subsequent sections. Therein, all relevant aspects are covered, including design basis, installation-, capacity- and serviceability-analysis, assessment of the foundation stiffness, and soil reactions. In the last section other aspects such a grouting, integrated analysis, and application of the presented approach to complete wind farms is briefly discussed.

RÉSUMÉ: Ce papier introduit la conception géotechnique de fondations de caissons de succion utilisés dans les fondations des turbines des éoliennes en mer. Cet article résume l'expérience acquise au cours de projets menés à travers le monde et propose quelques conseils pour l'élaboration de projets futurs. Ce papier est structuré en trois sections. Dans la première partie, différentes approches utilisées lors de la conception des caissons de succion des fondations sont présentées de manière générale. Les aspects individuels et particuliers de la construction sont expliqués en détails plus loin dans cette même section. Tous les aspects pertinents sont couverts allant de la conception à l'analyse de l'installation, de la capacité et de la maintenance à l'évaluation de la rigidité de la fondation et des réactions du sol. Dans la dernier section, d'autres aspects, tels que le ciment, la conception intégrée, et l'application de l'approche présentée à un parc éolien complet sont discutés.

KEYWORDS: suction caissons, offshore wind, design

MOTS-CLES: caissons de succion, éoliennes en mer, design

1 INTRODUCTION

All major offshore wind energy developers worldwide are currently investigating alternatives to the *Monopile* concept, which is widely used for the foundation of *Offshore Wind Turbines* (OWT). This effort is driven by technical considerations – mainly increasing turbine capacities and deeper waters at future wind parks – as well as environmental and economical considerations. A promising foundation concept is the so-called *Suction Caisson*; a hollow steel cylinder closed at the top and opened at the bottom. Suction caissons are installed by means of the self-weight of the structure and a suction pressure applied inside the caisson. Once installed, they resist environmental loads like an embedded shallow foundation, but can also temporarily mobilize considerable suction, which further increases the capacity and stiffness.

Though suction caissons are already used since several decades, practical experience with the short- and long-term behavior of these foundations used for OWTs is limited so far. Notwithstanding the lack of experience, a number of projects have been initiated where suction caissons have been or will be applied. The *Norwegian Geotechnical Institute* (NGI) has been involved in most of these projects, including *Borkum Riffgrund 1* (BKR01), *Borkum Riffgrund 2* (BKR02), *Hornsea 1* (HOW01), *Aberdeen Offshore Wind Farm* (EOWDC), *Hywind Scotland Pilot Park*, and *Southwest Offshore Demonstration Wind Farm* (SWK), providing various services such as laboratory testing, geotechnical design, suction installation support, and health monitoring systems. The experience gained in these and other projects forms the basis for the presented work.

The objective of this paper is to provide an overview of the particular design-requirements and -challenges of suction caissons for the foundation of OWTs, and should assist decision makers to consider this foundation concept in future wind farm projects. The presented design aspects and recommendations can be directly applied in ongoing and future projects, and provides a basis for currently developed standards and guidelines for certification and approval. Not included in this contribution are detailed descriptions of design methodologies as they are widely discussed in the many other publications. However, some references to relevant design methodologies are included. Main focus is to outline OWT-specific design aspects, for both caissons for jackets and mono-caissons.

1.1 General design approach

Suction caissons are used since the 1980s in the *Oil & Gas* (O&G) industry as the foundation of both bottom fixed and floating offshore structures. It is estimated that by the end of 2010 more than 1000 permanent offshore suction caissons and anchors were installed.

In the last decades a vast amount of articles and journal papers were published presenting results of research work and practical experience with suction caissons and anchors. Most of these are addressing particularly deep-water application cases. While in the early years mainly suction caissons in clayey soils were considered, also sandy and layered soils came into the focus in the more recent years. Most publications present theoretical and numerical studies as well as small-scale 1g or Ng model tests (e.g. Byrne 2000, Johansson et al. 2003, Kelly et al. 2006, Jostad et al. 2015a). Only limited measurement data is found from actually built structures. Some examples of installation data are report by Sparrevik (2002), Colliat et al. (2007), Aas et al. (2009), Langford et al. (2012), Solhjell et al. (2014), Saue et al. (2017), and in-place measurement data on prototypes by Schonberg et al. (2017), Svanø et al. (1997).

The experience gained in the last 30 years from the O&G industry provides a good basis for the design of suction caissons for OWTs. However, there are a number of important aspects, which are different, and which require particular consideration in the design of caissons for OWTs:

- Most offshore wind farms are located in relatively shallow waters where the sub-surface has been exposed in the more recent geological history to significant environmental changes such as glacial periods, dry periods and floods, yielding pronounced soil layering comprising a large range of different soil types and properties (e.g. Cotterill et al. 2017, Dove et al. 2016). As a result, soil profiles may vary significantly both in depth and horizontally.

- The loading conditions are different for OWT foundations. With increasing turbine size operational and other load cases can govern the geotechnical design, being potentially more severe than a conventional 50-, or 100-years storm event, which is typically used in the design of offshore O&G structures. In addition, these design-critical load cases may have considerably recurrence rates during the lifetime of an OWT.

- The response of the sub-structure of an OWT is very sensitive to the foundation behavior, i.e. stiffness and (differential) settlements. Although this can be an important design aspects for O&G structures, it is in general more important for OWTs due to the high-cyclic loading conditions during operation and the sensitivity of the turbine on a tilt.

To complicate matter, the supposed conservative assumptions made in the geotechnical design in order to cope with these and further challenges are not necessarily conservative for the structural design – and vise versa, for apparently conservative assumptions made in the structural design. Thus, input and assumptions in both the geotechnical and the the structural design need to be aligned and consistent.

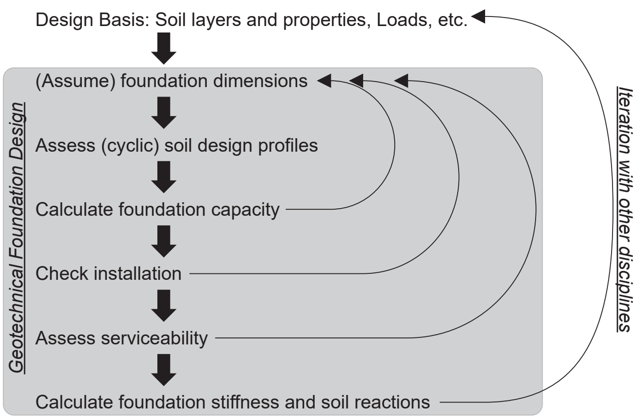

Figure 1: Schematic presentation of the iterative and interdependent workflow of suction caissons design

The consistency is achieved by an iterative design approach as illustrated in Figure 1. The geotechnical design of a suction caisson foundation comprises 5 main activities: 1) Assessment of the cyclic soil properties for the given boundary conditions, i.e. load conditions, foundation geometry, and soil layering and properties; 2) Foundation capacity assessment for short- and long-term loading; 3) Prediction of the installation resistance and corresponding required suction pressure; 4) Serviceability assessment, i.e. short- and long-term settlement, displacement and rotation; and 5) Calculation of the foundation stiffness including corresponding soil reactions. The activities are interdependent and typically need to be solved in an iterative manner in order to optimize the caisson geometry.

Furthermore the geotechnical design is embedded into a design loop interacting with other disciplines. The basis for the geotechnical design will be continuously updated based on the results of both the geotechnical analysis and other involved disciplines. The structural designer may update the properties of the caisson and the sub-structure, the turbine manufacturer may update the (cyclic) loads, and the soil layering and properties may be complemented by updated field and laboratory test data, to name a few.

The workflow of the (geotechnical) design approach illustrated in Figure 1 is not very much different to that of any other foundation. However, it is important to be aware of the interdependency, as this pose a natural limitation on the achievable optimization. A typical project comprises different phases; e.i. feasibility study, pre-FEED[1], FEED and Detailed Design. Each of these phases can comprise one or several iteration(s). Current research aims to solve some of the activities in an integrated manner (e.g. Krathe & Kaynia 2016, Page et al. 2016, Skau et al. 2017). That means it is tried to model the complete OWT in one analysis to capture the interdependency. However, all parts, and in particular the soil-foundation-system, is often represented in these analysis in a simplified way in order to limit the required calculation time. Thus an integrated analysis may not be suitable for an optimization, but can be very beneficial for other aspects, in particular for the assessment of loads.

1.2 Interface between disciplines

The iterative design approach illustrated in Figure 1 requires a physical interface between the different disciplines at which input, or output, respectively, is exchanged. There are in principal two types of information which need to be exchange between the geotechnical and structural designer:

- The geotechnical designer gets loads and delivers back the corresponding deformations, i.e. load-deformation curves. These curves are practically represented by lumped stiffness values describing the response of the soil-foundation-system in one point. The stiffness values are typically provided in matrix form and can comprise of linear secant stiffness values or non-linear tangential stiffness values.

- The structural designer requires for the caisson design distributed loads and/or deformations acting on the skirts and lid. These distributed loads/deformations are often denoted *Soil Reactions* as they describe the response of the soil. Soil reactions can be provided as unit loads, total loads or linear springs (i.e. Winkler-type springs).

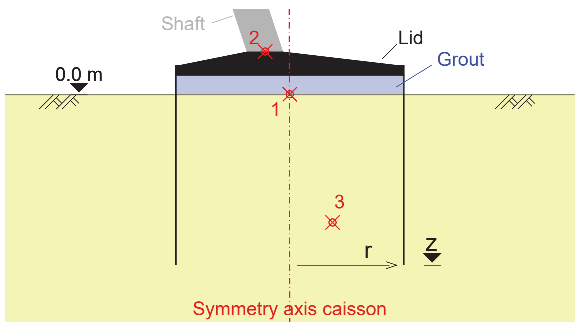

Figure 2: Possible interface points for the geotechnical and structural designer

Practically, three different points could be imagined for the load-stiffness exchange, as illustrated in Figure 2. Each point has advantages and disadvantages.

[1]Front End Engineering Design (FEED)

1. Traditionally, Point 1, located on the symmetry axis of the caisson at mudline, is very often used. However, the structural designer needs to establish loads at a point which is not connected to the structure. In order to do that, he needs to introduce a so-called *super-element*, connecting the structure with the ground in this point. Given that the structure – in this case the caisson lid and grout – is significantly stiffer than the soil for the considered load level, simplified, linear elastic properties can be assigned to the super element. If the flexibility of the structure is considerably larger and a interaction with the soil behavior may be expected, more complicated properties need to be assigned to the super element. However, these properties are very difficult to assess, which may not be possible. Experience from recent projects has shown, that both the lid and skirt flexibility is important and an optimization of the caisson geometry is difficult, for which reason, Point 1 is not recommended to be used in future projects.

2. Point 2, located at top of the caisson lid in the interface between the shaft of the sub-structure and the caisson, has been used in more recent projects. The advantage is, that Point 2 is also often an interface for the structural design, as the design of the caisson and sub-structure is often done separately. Loads are assessed by the load- or structural-designer using integrated analysis where only the sub-structure is modeled. The soil is therein often represented by set of springs in Point 2. That means no super-element is required, but the geotechnical designer needs to include the lid accurately in his analysis.

   The load-deformation response is complex, meaning that a reasonable stiffness matrix describing the load-deformation of the soil-caisson-system will have both diagonal and off-diagonal components. However, most programs used for integrated analysis cannot cope with a full stiffness matrix but can take only the positive diagonal terms. That means that the soil-foundation response can be only considered in a simplified manner when using Point 2.

3. In order to overcome the shortcoming of using a simplified stiffness matrix in the structural analysis, the stiffness matrix could be provided for the so-called *decoupling point*, which is illustrated in Figure 2 by Point 3. The decoupling point can be assessed in the stiffness analysis as described in Section 7, and is characterized by the fact that incremental horizontal, vertical or moment loads yield only displacements or rotations in the corresponding loading direction. That means that the stiffness matrix comprises only positive diagonal terms. If the geotechnical designer includes in the stiffness analysis the caisson with its correct properties, and applies the loads in Point 2, the structural designer can use a rigid super element connecting Point 2 with Point 3 and apply the stiffness matrix in the integrated analysis in Point 3. Though Point 3 seems to be the most appropriate point for the interface, the problem is, that the location of the decoupling point is not constant but depends on the load-level, combination of load components and load-deformation response.

Based on experience from recent projects, it is recommended that the structural designer provides the caisson model and the loads in Point 2, and the geotechnical designer delivers back a stiffness matrix in Point 2 and Point 3 as well as the coordinates of Point 3.

In Section 7 is introduced the concepts of a global model. Though this model is a considerable improvement as both the sub-structure, caisson and soil is modeled, it does not overcome the above described problem of finding an appropriate interface point. The structural designer will still need stiffness values at the bottom of the sub-structure.

In principal, stiffness values and soil reactions could be established from the same analysis as they are actually describing the same response. However, the extraction of soil reactions from FE analysis is difficult and very sensitive to the modeling technique, element-type and -size. As the soil reactions are only used for the caisson design, but neither for the load assessment nor the design of the sub-structure, it has been found most appropriate to establish reasonable ranges for the distributed loads acting on the skirts and lid based on empirical considerations.

2 DESIGN BASIS

The design basis is the input to the geotechnical design before any interpretation or processing is done. It comprises soil properties, loads, structural properties, guideline requirements, and other relevant boundary conditions such as weight- and size-limitations due to logistical considerations.

2.1 Site and soil parameters

The loading regime acting on a suction caisson requires special attention with respect to the soil parameters used in the design. The impact of cyclic loading on the soil strength and stress-strain-behavior needs to be quantified by a thoroughly planned laboratory testing program of all relevant soil layers. The following list outlines the recommended minimum site- and soil-investigation program to establish the required soil-profiles and -parameters:

- From a geotechnical perspective, a geophysical survey is recommended to identify the number and depth of the soil layers at the OWT location(s). The geophysical survey should provide an overview of the soil profile variability at a location, which is in particular relevant for multi-legged sub-structures having three or more caissons. In some recent projects, two surveys have been conducted. In a first survey the complete offshore wind farm was screened, whereas in a second survey high resolution 3d seismic scans of the shallow soil has been performed. The advantage of the latter survey is, that it allows to find also small boulders, which can be critical for the installation.

- Minimum one seabed *Cone Penetration Test* (CPT) per location with a minimum investigation depth z_a, measured from the skirt tip, where z_a is the maximum of

  - the depth below the caisson where the additional stresses $\Delta\sigma'_v$ due to the permanent weight of the structure does not exceed 15% - 25% of the in-situ stress prior to the installation of the caisson. Assuming a load spread angle of 1:3, a submerged foundation weight between 5 to 7MN, a caisson diameter between $D = 8$ and 10m, a submerged unit weight of the soil of $10\,\frac{\mathrm{kN}}{\mathrm{m}^3}$, and a skirt depth of $s = 0.6 \cdot D$, the required depth $s + z_a$ (measured from mudline) varies between 15 and 20m.

  - the depth of the governing failure mechanism in a bearing capacity analysis, which is a function of the caisson diameter D, the number of footings and distance of the legs, and the loading regime. A rotational failure is expected for mono-caissons, whereas a compression failure is expected for caissons supporting a jacket. In both cases, the depth measured from the skirt tip level is less than the caisson diameter, given that there is no interaction between the footings of multi-legged structures. For the dimensions indicated above, the required depth $s + z_a$ (measured from mudline) varies between 10 and 14m.

Even though, neither combined deep failure mechanisms of multi-legged structures, nor exceptionally high weights, have been observed in past projects, it is recommended to check in the FEED study, whether the values given above are not exceeded. That means, it needs to be ensured that the additional stresses are not larger, nor the actual failure mode giving the lowest foundation capacity reaches deeper than assumed. If the required investigation depth cannot be achieved by the seabed CPT, complementary downhole CPT should be performed.

At sites and turbine locations where highly variable soil conditions are expected, several CPTs should be conducted.

In general, it is recommended to perform the CPTs outside the actual caisson location, to avoid open holes which will potentially affect the caisson installation and may even prevent the caisson to reach the target penetration depth.

- Sufficient boreholes at the site in order to extract samples of all relevant soil units. Number and locations of the boreholes should be selected based on the review and interpretation of the geophysical and CPT data, preferable on basis of a ground model (e.g. Forsberg et al. 2017)
- Laboratory tests of all relevant soil layers within the CPT depth. Andersen et al. (2013) provide a comprehensive list of required parameters for various foundation concepts. A summary of parameters for suction caissons is listed in Table 1. The crosses in brackets indicate parameters, which are, according to the author's experience, somewhat less relevant.

In order to determine the required parameters, drained and undrained, monotonic and cyclic DSS, triaxial compression and triaxial extension tests need to be performed. Further, oedometer tests, bender element tests, and interface tests should be included in the testing program. For layers with few decimeter thickness, triaxial tests may be omitted. The number of tests depends on the loading conditions, available data from previous investigations at similar material, and the applied design methodologies. A representative set of laboratory tests per soil layer may comprise

- 2 oedometer tests,
- 1 monotonic undrained DSS test and 1 monotonic undrained triaxial compression test, as well as corresponding drained tests when testing sands,
- 3-5 cyclic undrained DSS tests,
- 4-6 cyclic undrained triaxial tests

In addition, other tests such monotonic as drained triaxial extension, or resonant column tests may be conducted where necessary. Of particular importance is the soil-skirt interface strength. It may be best represented by a remolded DSS test consolidated to a stress equivalent to the lateral in-situ stress after installation. The stress level needs to be estimated. Reasonable stress ratios may be 0.5 and 1.0 times the vertical in-situ stress $\sigma'_v = \gamma'_{\text{soil}} \cdot z$. Larger values may be less likely due to set-up effects and arching, but may need to be decided project specific.

Table 1: Recommended soil data for suction caisson design (after Andersen et al. 2013)

Soil parameter	Clay	Sand
Frictional characteristics		
Peak drained friction angle, φ'		x
Residual / critical drained friction angle, φ'_c		x
Undrained friction angle, φ'_u		x
Dilatancy angle, ψ		(x)
Slope of DSS drained failure line, α'		x
Slope of DSS undrained failure line, α_u		x
Interface friction angle, δ_{peak} and δ_{residual}		x
Monotonic data		
Undrained shear strength, s_u^{C}, s_u^{DSS}, s_u^{E}	x	x
Initial shear modulus, $G_{\max}$	x	x
Cyclic data (triaxial and DSS)		
Undr. shear strength, $\tau_{f,cy} = f(\tau_a, \tau_{cy}, N)$	x	x
Pore pressure, $u_p = f(\tau_a, \tau_{cy}, N)$	(x)	x
$u_p = f(\tau_{cy}, \log N)$ for $\tau_a = \tau_0$,	(x)	x
Stress strain data, γ_a, γ_p, $\gamma_{cy} = f(\tau_a, \tau_{cy}, N)$	x	x
$\gamma_{cy} = f(\tau_{cy}, \log N)$ for $\tau_a = \tau_0$	x	x
Damping	x	x
Consolidation characteristics, intact soil		
Preconsolidation stress (and OCR)	x	x
Un- and reloading constrained moduli	x	x
Permeability, k	(x)	x
Remoulded soil data		x
Sensitivity, S_t	x	
Undrained shear strength, s_u^{DSS}	x	
Cyclic undrained shear strength, $\tau_{f,cy}$	x	
Constrained modulus	(x)	
Permeability	(x)	
Thixotropy	(x)	

It is important to perform the tests at a stress and density or OCR, respectively, representative for the expected in-situ conditions before and after installation. Three zones need to be distinguished; inside the caisson, outside the caisson, and below the caisson. While the soil state outside the caisson will be less affected by the installation, the soil at the inside may undergo considerable shearing, which will affect the density and stresses. The soil below the caisson will be less affected by the installation, but the weight of the OWT will yield an increase of the vertical effective stresses (with time).

In addition, index parameters such as relative density D_r, plasticity coefficient I_p, water content w, and grain size distribution should be determined. These are in particular relevant in an early stage of the project for the feasibility study and preliminary sizing, where not all laboratory tests have been initiated yet, and where strength and stress-strain-behaviour has to be assessed based on correlations using index data and CPT soundings. Andersen (2015) proposes a comprehensive set of correlations, which can be used as a first estimate of the expected soil parameters.

In addition, information of scour development and/or scour protection is required. Type, thickness, submerged weight, and information on the stability of the planned scour protection need to be considered in the geotechnical analysis.

2.2 Loads

The geotechnical designer needs to consider two different load sets. One set is required for the actual geotechnical design, i.e. capacity and serviceability analysis. The other set is used in the load-stiffness iteration (outer loop in Figure 1). Some load cases may be included in both sets. But in general, the loads cases are different in both sets, since the governing design-loads and -criteria are typically different in the structural and the geotechnical design. That means each discipline has to identify the relevant load cases, and need to define them such that everyone involved in the design process has a common understanding. Since this is a very critical aspect of a successful project, a load document should be prepared, which is continuously updated. This has been proven beneficial in many projects.

Most design guidelines distinguish between loads for the *Ultimate Limit State* (ULS), *Serviceability Limit State* (SLS), and *Fatigue Limit State* (FLS)[2]. ULS loads are required by both the geotechnical and the structural designer. However, SLS loads are mainly relevant for the geotechnical analysis, whereas FLS loads are mainly relevant in the structural analysis. All load cases are assessed by the load or structural designer, and the geotechnical designer need to provide input to these.

Identifying or defining the required loads needs an experienced designer. A reasonable starting point for the capacity analysis is to look at the load cases comprising the maximum amplitudes; that means maximum compression, tension, moment, etc. The maximum load amplitudes often adhere a load event which is embedded into a cyclic load history, which can be a storm for example. The German *Bundesamt für Seeschifffahrt und Hydrographie* (BSH) introduced in the standard BSH (2015) a 35-hrs design storm based on a composition of the *Design Load Case* (DLC) 6.1 proposed in the IEC standard IEC (2009). This cyclic event shall be applied to assess the cyclically (degraded) soil strength, which is to be used in the (subsequent) geotechnical analysis. Practically, this event has also been also applied outside Germany, due to the lack of alternatives, since the DLC's defined in the IEC standard are 10 or 60 minute long load-time series, which cannot be directly used in a geotechnical design.

In more recent projects, where turbines with larger capacity were considered, it has been found that also other events can be critical, such as an (emergency) shut-down at relative high wind-speeds. In the event of an (emergency) shut-down, the OWT swings and the load spectrum corresponds to a damped vibration. Depending on the degree of damping, which affects the decay rate, subsequent load cycles with smaller amplitudes can be critical due to the cyclic degradation of the soil, induced by the previous larger load cycles. Another event found critical for the foundation capacity analysis of multi-legged structures is the prolonged tension load case, which typically occurs during operation of the turbine at high wind speeds.

In addition to the in-place loads, there may be further situations which needs to be considered in the design. These can be load cases during installation, maintenance, and decommissioning of the OWT.

[2]The author questions the appropriateness of the expression *limit state* in this context. However, since it is widely used, it is – due to convenience reasons – also adopted in this contribution.

More complicated is the identification of the load cases which should be used for the serviceability analysis. Two scenarios have to be distinguished; a maximum deflection and rotation during a severe load event, and accumulated average long-term deformation and rotation. The peak deflection may be assessed using the loads used in the capacity analysis. For assessment of the long term deformations and rotations, cyclic loads are required. Ideally, all loads during the lifetime of the OWT should be considered in chronological order. However, as this cannot be applied in a geotechnical analysis, simplified load histories are required.

It can be supposed that large cyclic load amplitudes will contribute most to the accumulated deformations and rotation. Thus focusing on a series of storm events may be a reasonable simplification. One option could be to use the 35-hrs design storm and assuming a Gumble distribution to extrapolate the peak amplitudes of other storms with different return periods. The accumulated average displacements and rotations can be calculated for each scaled 35-hrs design storm separately and then superimposed depending on the expected number of occurrences of each storm during the lifetime of the OWT.

The main challenge is to derive from the load-time-series the actual load amplitudes and corresponding mean values, and number of occurrences, both of the maximum- and the cyclic-load events. Most commonly the so-called *rainflow-counting-algorithm* is applied. Though this algorithm is widely used in structural fatigue analysis, it is important to be aware of its limitations:

- It is assumed that the loads are independent, meaning that the order of load cycles is not important.
- The information of the load frequency, that means the cyclic period, gets lost.
- Since only the peak values are counted (that means actually half-cycles are counted), no information can be directly derived of the actual corresponding mean load.

Depending on the soil type, drainage properties and boundary conditions, these information can be crucial. Thus, if these information would need to be considered, other counting methods may be applied where possible; for example the method proposed by Norén-Cosgriff et al. (2015). They apply high- and low-pass filters and determine the amplitude of each half-cycle from adjacent maxima and minima, which belong to the same load cycle. In addition, the proposed method keeps track of the corresponding average load and may also keep the information of the load period (frequency). The authors compared their method with the rainflow-counting-algorithm and showed that the calculated cyclically degraded soil strength using the example of a normally consolidated clay can be significantly different.

Cyclic load histories are often provided in from of a *Markov Matrix* comprising cyclic load amplitudes and corresponding mean load value as well as number of occurrences. Since these are often established using the rainflow-counting-algorithm, it is recommended that the geotechnical designer reviews also the original load-time-series from which the Markov Matrix has been established. This in particular applies to the load-time-series comprising the maximum load values used in the geotechnical capacity analysis. The load cycle yielding the maximum load values may sometimes appear to have a considerable offset from the rest of the cyclic loads history and it requires geotechnical judgment to decide on the load cycle which the soil actually experience. But also a critical review of the mean load value is important, as the soil behaves essentially different symmetric and asymmetric cyclic loads.

It is recommended that permanent and environmental loads are provided separately, and both as characteristic values, as occasionally, different partial safety factors need to be applied to the different load components in the geotechnical and structural analysis.

2.3 Structural properties

As outlined in Figure 2, it may be important to include structural components in the geotechnical analysis. With increasing complexity of the structural model, the stability and accuracy of numerical analysis may be quickly challenged. Thus, if structural models shall be included in a geotechnical analysis, they may be simplified as appropriate. Beam and plate elements should be preferred over continuum elements. Structural components such as stiffeners and stays may be omitted where possible.

For capacity analysis, a rigid structure may be assumed, as the the strength and stiffness of the soil at failure is several magnitudes smaller than the strength and stiffness of the structure, given that the yield stress of the caissons material is not exceeded at any time.

For installation purposes, the properties of the skirts are of fundamental importance and need to be considered in the penetration analysis as accurate as possible. In general, the skirt tip resistance increases with increasing wall thickness. If stepped skirts are considered, i.e. where the skirt wall thickness varies over the height, the skirt friction may be affected considerably, which also will affect the in-place behavior. It is also important to consider compartments[3] and stiffeners in the penetration analysis if present.

2.4 Guidelines and safety factors

A dedicated standard or guideline for the design of suction caissons for OWT applications does not exist. In the absence of such a document, other non-dedicated standards and guidelines need to be applied in the design. This requires to define a code hierarchy, where in general national standards rank highest, followed by offshore wind related standards as well general offshore standards, and finally other standards, guidelines and publications, which rank lowest. Some examples are presented in the following.

The IEC has proposed a series of documents addressing the particular design aspects of onshore and offshore wind turbines. For the load assessment and corresponding partially load factors, typically IEC standard 61400-3 is applied (IEC 2009). Other standards published by the IEC consider structural and geotechnical design aspects. However, these documents are so generally formulated, with respect to geotechnical requirements – and in particular suction caisson design – that other standards need to considered.

To the author's knowledge, all countries where OWTs are considered, have own national standards for geotechnical design. However, since these standards originate form onshore design requirements, the application of the recommended methods and procedures to offshore structures can be critical. Thus, some countries are in the process of establishing national standards particularly for OWTs. This has been done by the German BSH for example. The US *Bureau of Ocean Energy Management* (BOEM) and the German DIN are also working on corresponding documents.

As most OWTs need to be certified due to financial and insurance reasons, some certifiers have published their own guidelines, which are frequently used in the design. Most relevant is the DNV GL standard 0126 (DNV-GL 2016). This document provides valuable recommendations and includes also a section on suction caissons. However, it is very generally formulated and neither particular methods nor procedures are proposed.

Selecting appropriate safety factors for the design is difficult. Solely the DNV standard proposes a consistent safety concept for capacity analysis considering the particular offshore conditions. In general, the strength of the soil shall be reduced or carefully estimated for capacity and serviceability analysis. However, for the installation analysis, a higher strength is more critical, which is not considered in any standard. Sturm et al. (2015) proposes safety factors for installation analysis of suction caissons in sand, which were established based on probabilistic analysis. Similar type of analysis may be performed for other design aspects. No safety factors should be applied in the serviceability-, stiffness-, and soil reaction-analysis as detailed in the corresponding sections.

Due to the lack of long-term experience, it is recommended to consider a comprehensive monitoring system as part of the so-called *observational method*.

3 CYCLIC STRESS-STRAIN BEHAVIOR

The loading condition of an OWT is of inherent cyclic nature. Thus, all components including the soil, need to be designed accordingly. The general supposition is, that cyclic loading yields a decrease of strength and stiffness, often denoted as *cyclic degradation*. This applies to all soil types and foundation concepts.

A number of authors have proposed methods for assessing the effect of cyclic loading on the suction caisson foundation response. Therein two main approaches are followed; an *empirical approach* and an *analytical/numerical approach*.

- The ***empirical approach*** is typically based on model test where the soil-foundation system is considered as one entity. The caisson is subjected to cyclic loading and the response in the loading point is measured. The actual behavior of the structure and soil is not considered separately, hence it is a phenomenological approach. The results can be presented in interaction diagrams[4] or failure envelopes in the HVM space, where HVM is the horizontal, vertical, or moment load component, respectively. Failure envelopes allow a more detailed description of the foundation response compared to interaction diagrams. In addition, a failure envelope diagram can be extended to describe the actual load-displacement behavior by introducing a stack of HVM envelopes to which the corresponding displacement components are assigned. Since these diagrams are based on interpolation of some few data points, they are essentially *empirical*. Many, so-called *macro-elements*, are based the empirical approach. Some Macro elements are mathematical complex and can describe very detailed the load-deformation behavior of a caisson subjected to general cyclic loading. A number of authors have developed macro-elements for suction caissons, (e.g. Nguyen-Sy 2005, Nguyen-Sy & Houlsby 2005, Salciarini & Tamagnini 2009, Salciarini et al. 2011, Foglia et al. 2014, Skau et al. 2017). Macro-elements are well suited in integrated analysis for structural design and load assessment.

- In the ***analytical/numerical approach*** the response of the soil-foundation system is assessed by modeling the actual soil-structure interaction under consideration of the structural flexibility and stress-strain-behavior of the soil. This requires a detailed description of the skirt-soil- and lid-soil-interface behavior. In an analytical approach, the distribution of average and cyclic loads – or actually stresses – along the skirts need to be assumed, whereas the distribution is automatically calculated in a numerical approach. The assessment of the

[3] Compartment mean that the caisson lid area is divided into different cells

[4] Similar to diagrams used for cyclic axially loaded piles

cyclic stress-strain behavior and strength of the soil needs to be described by using appropriate soil models. The analytical/numerical approach is well suited for the geotechnical sizing of the caisson, but may also be used for assessment for the serviceability and calibrating of the input parameters to a macro-element.

NGI has developed a method for describing the behavior of cyclically loaded soil elements using so-called *cyclic contour diagrams*. The method, originally proposed in the early 70$^{\text{th}}$, which was continuously developed further, has been presented in a numerous publications; the most recent and comprehensive one is the article by Andersen (2015). Cyclic contour diagrams span a 3-dimensional space and provide a general relation between average and cyclic shear stresses and corresponding average and cyclic shear strains as function of number of applied cycles. Diagrams are established for one soil type and density or OCR, respectively. One complete set of 3d-diagrams for one soil unit comprises typically of 4 diagrams; 1 strain and 1 pore pressure diagrams for triaxial and DSS conditions, respectively. In many practical application cases, only some representative 2-dimensional cross-sections of the 3-dimensional space are required. This simplifies the approach and reduces the number of cyclically laboratory tests. The selection of appropriate cross-sections requires some experience and assumptions.

In combination with a cyclic load history, the cyclic contour diagrams can be used in the so-called *cyclic accumulation procedure*. The cyclic degradation due to the cyclic loading is calculated and the effect can be expressed by the so-called *Equivalent number of cycles* ($\mathrm{N_{eq}}$).

As cyclic contour diagrams provide a relationship between stresses and strains, but the cyclic loads are given as forces, assumptions on the load transfer and stress distribution has to be made, which is best done using the *Finite Element Method* (FEM). This is in particular the case where complicated boundary conditions, soil layering and drainage conditions are analyzed, which is in general the case for suction caissons for OWTs. NGI has implemented the cyclic accumulation procedure using cyclic contour diagrams in an FE code. Jostad et al. (2014) present the procedure for fully undrained conditions during the considered cyclic load history (UDCAM)[5], whereas the procedure for partially drained conditions (PDCAM)[6] is presented by Jostad et al. (2015b). The cyclic accumulation is done for each integration point. The advantage of using the FEM is, that the stress redistribution is considered accurately and continuously updated if relevant, and that strain continuity is ensured. Furthermore, a output of such an analysis is not only the cyclic stress-strain behavior and degraded strength and stiffness, but also the accumulated displacements and rotations, which are required for the serviceability analysis.

Though the soil-structure interaction is modeled in detail (numerical approach), the description of the soil behavior using cyclic contour diagrams is an empirical approach.

An example of a PDCAM analysis of a suction caisson subjected to a combination of vertical and horizontal cyclic loading is shown in Figure 3. A suction caisson with 8m diameter and 6m skirt length in a homogeneous soil deposit with an average soil permeability of $k = 1 \cdot 10^{-5}\,\frac{\mathrm{m}}{\mathrm{s}}$ is modeled. At the peak phase of an 35-hrs design storm according to BSH (2015), the soil at skirt tip level accumulates considerable excess pore pressure. Due to the symmetric soil and load conditions the predicted pore pressure field is also almost completely symmetric.

[5]UnDrained Cyclic Accumulation Model
[6]Partially Drained Cyclic Accumulation Model

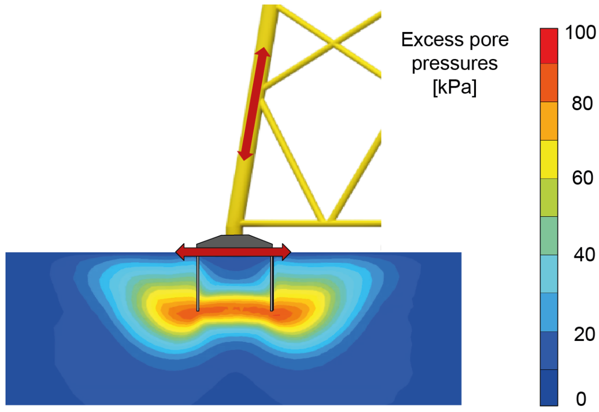

Figure 3: Finite element analysis of a suction caisson subjected to combined vertical and horizontal cyclic loading using the NGI soil model PDCAM. The contour plot shows the excess pore pressure at the end of the peak phase during a 35-hrs design storm.

4 FOUNDATION CAPACITY

The foundation capacity needs to be ensured for all possible load combinations. Two main load scenarios should be distinguished, which are detailed in the following.

4.1 Short-term loading

Short-term loading is characterized by a loading duration being so short that the soil behaves essentially undrained, meaning that the soil response depends on the undrained shear strength only. In sandy soils, the caisson may mobilize considerable suction below the lid and negative pore pressure in the soil, causing an increase in mean stresses and hence higher shear strength. Due to the shallow water depth at typically OWT sites, particular attention requires the cavitation limit. The cavitation limit cannot be exceeded by the suction or negative pore pressure, respectively. That is in particular important to consider when deriving the shear strength from laboratory tests where considerable back-pressures may have been applied, as these tests can potentially exceed the maximum achievable pore pressure and hence strength compared to the actual in-situ conditions. The theoretical cavitation limit $p_{\mathrm{cav,max}}$ in a soil element is the sum of, the depth z of that element below mudline plus the water depth w_s, multiplied with the unit weight of water $\gamma_w = 10\,\frac{\mathrm{kN}}{\mathrm{m}^3}$, and the atmospheric pressure $p_{\mathrm{atm}} = 100\mathrm{kPa}$, viz.

$$p_{\mathrm{cav,max}} = (z + w_s) \cdot \gamma_w' + p_{\mathrm{atm}} \tag{1}$$

At NGI, the short-term capacity analysis is often done using a total stress approach. Figure 4 shows a potential failure mechanism of a suction caisson under combined compression and moment loading. The undrained strength in the failure zone is described by the strength measured in undrained DSS tests, or in a trixial tests where different *Total Stress Paths* (TSP) are followed. Cyclic contour diagrams can be used for assessing corresponding cyclic shear strength values.

Figure 5 illustrates the four main different total – and corresponding effective – stress paths, using the example of a medium dense to dense sand specimen consolidated to a stress state of $k = \frac{\sigma_h'}{\sigma_v'} = 0.5$ at a vertical effective stress of $\sigma_v' = 200\mathrm{kPa}$. The difference between the TSPs is the way the shear strength has been applied. For path 1 and 6 the cell pressure in a triaxial test has been decreased or increased, respectively, whereas for path 4 and 2 the vertical pressure has been increased or decreased, respectively.

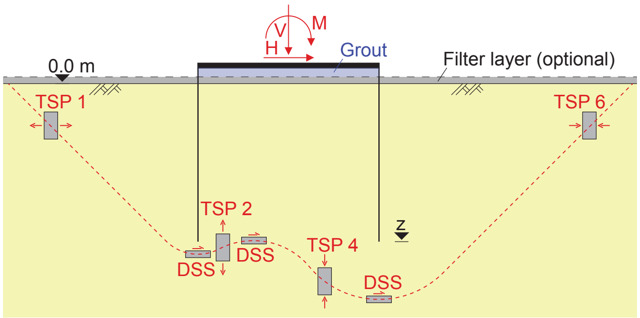

Figure 4: Possible failure mode of a caissons subjected to combined compression and moment loading

In addition, the total and effective stress path in direction 4 for a specimen consolidated to $\sigma'_v = 20\text{kPa}$ is shown.

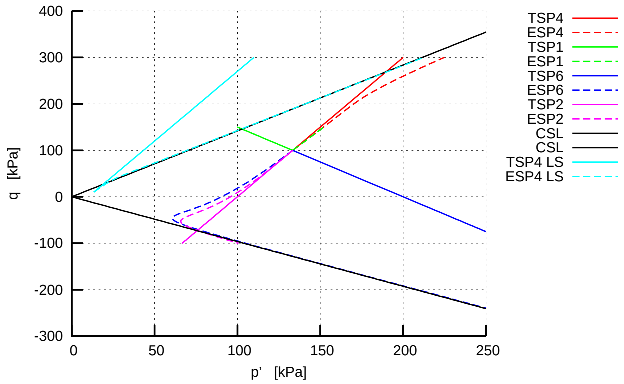

Figure 5: Total and Effective stress path in trixial tests where the shear stress is applied in different ways.

From Figure 5 becomes apparent that the soil strength of a sand specimen for a given initial density and stress state is depending on the loading path. The difference between the total and effective stress for the different paths equates the corresponding pore pressure. The maximum negative pore pressure cannot exceed the cavitation limit. Whether the NGI method or any other method is applied, it is important that the dependency of the stress path and the cavitation limit is considered accurately when assessing the soil strength profile.

The stress path dependency is equally relevant for clay specimens. In additon, due to the viscosity of clays, the dependency of the shear strength on the shear rate needs to be considered. The shear rate in laboratory tests may be different compared to insitu loading rate for short-term loading, meaning the shear strength may need to be corrected accordingly.

The capacity of suction caissons to short-term loading is essentially governed by the load combination, that means horizontal, vertical and moment loading. As illustrated in Figure 1, the design basis, including the loads, is continuously updated. Figure 6 shows the dependency of the ULS loads on the rotational stiffness of a suction caisson at the example of a multi-legged substructure. The loads of a leg in compressions, are normalized with the reference loads provided in the 1^{st} iteration. The predicted corresponding rotational stiffness – also normalized – is shown at the abscissa where all load components are crossing. Though the global loads acting on the OWT are constant, the local loads can vary considerably depending on the response of the caisson. The higher the rotational stiffness, the lower the vertical and torsional loads. Similar effects, but less pronounced is found for the other stiffness components.

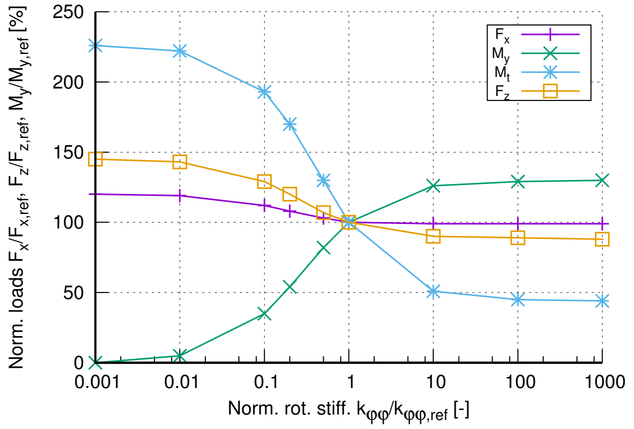

Figure 6: ULS loads as function of the rotational stiffness of caisson supporting a three-legged jacket.

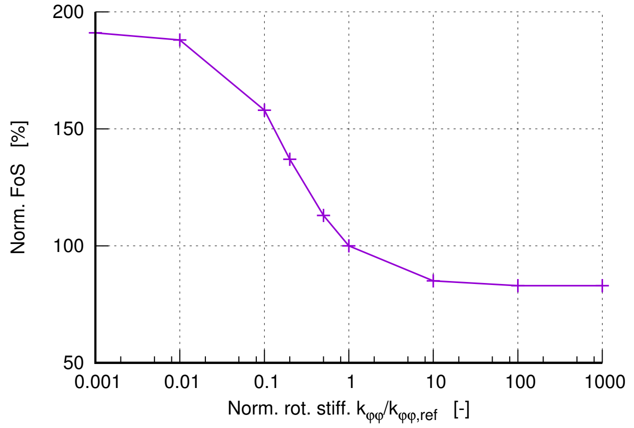

Figure 7: Normalised *Factor of Safety* (FoS) as function of the applied loads shown in Figure 6

The effect of the load combination shown in Figure 6 on the caisson capacity is shown in Figure 7, where the normalized *Factor of Safety* (FoS) is plotted on the abscissa. As it may be expected from conventional bearing capacity analysis, the normalized FoS is lower for larger moments, that means for a rotational failure mode. That applies also to a mono-caisson foundation, which is essentially subjected to environmental horizontal and moment loading only.

In offshore foundation design of multi-legged jacket structures, it is often assumed that the rotational stiffness of a foundation at ULS loading is considerably lower than the rotational stiffness of the corresponding leg of the sub-structure. Hence, the local moment loading at failure may be omitted in the capacity analysis. However, the relatively high jackets stiffness can be an issue for the fatigue design of an OWT, as the goal is, that the first eigenmode shall be in the range between 1P and 3P; e.g. typically between 0.25 and 0.35 Hz for turbines with 6 to 8MW. Thus, the structural designer tries to make the jacket more flexible, meaning, that omitting the local moment may be too optimistic.

To complicate matters, the local load components at a leg of a jacket do not scale proportionally with the global load amplitude, even though the global loads may be applied linearly increasing. Thus, the ULS load components provided in the design basis may not be scaled proportionally with a load factor. However, as the soil will be always softer than the jacket leg in rotation when being at failure, overestimating the local moment may yield lower FoS as shown in Figure 7. Nevertheless, it is recommended to check the FoS for differently scaled local loads, that means lower load factor applied to the local moment and a larger load factors to the vertical, horizontal and torsional load components. For mono-caissons, a redistribution of the local loads is not expected and the same load factor should be applied to all load components.

For suction caissons subjected to tension loading, the same considerations discussed above apply. The TSP strength used in the analysis need to account for the different loading and hence stress conditions.

Gapping at the outside of the caisson may need to be considered in the capacity analysis, if previous load conditions or stepped skirts may have generated a gap. Due to the short-term loading, the drainage time may not be sufficient to generate a new gap during the considered load event. This depends of course on the load combination and soil type and may need to be checked.

Of particular importance is the scour development and scour protection. The stress and density state of the soil can be considerably affected, which can have an impact on the foundation capacity. Whether to include or omit the effect of a scour and scour protection should be discussed with the operator, as the presumption of a permanent scour protection may require more frequent on-site inspections, which can have an impact on the *Operational and Maintenance* (O&M) costs.

4.2 Long-term loading

Suction caissons have considerable capacity under short-term loading conditions. However, the resistance to long-term loading, can be very low, as the possibly mobilized suction may dissipate. This is in particular relevant for suction caissons supporting a jacket structure. During operational load cases the caisson(s) may experience considerable tension loading, which can last for hours or even days. The tension capacity of suction caissons is a function of the skirt wall friction and the soil permeability.

For caissons in clay the soil permeability will be low, meaning that the capacity can be calculated similar to the long-term capacity, but the shear strength needs to be reduced to account for the slow loading rate. In the absence of suitable tests, the decrease in shear strength may be estimated using

$$s_{u,\mathrm{slow}} = s_{u,\mathrm{ref}} \cdot \left(\frac{\dot{\gamma}_{\mathrm{slow}}}{\dot{\gamma}_{\mathrm{ref}}} \right)^{I_v} \tag{2}$$

where the $s_{u,\mathrm{ref}}$ is the shear strength measured in the laboratory at a shear rate of $\dot{\gamma}_{\mathrm{ref}}$. $\dot{\gamma}_{\mathrm{slow}}$ is the shear rate representative for the considered load case. I_v is a viscosity coefficient which typically varies between 0.03 and 0.07 for a silty or fat clay, respectively (Leinenkugel 1976). I_v can be determined with Equation 2 from an undrained static laboratory test, where the shear rate is varied.

If previous load cases, structural boundary condition or any other causes may have generated channels or gaps at the outside and inside of the caisson in the clay, only the skirt wall friction can be considered in the tension capacity analysis.

For caissons in sand, the soil permeability is considerably higher, meaning that a continuous flow of water from the outside to the inside can be expected, given that the tension load exceeds the resistance calculated by integrating the fully drained skirt wall friction over the skirt area at inside and outside of the caisson. In this case, the capacity is the sum of the drained skirt wall friction at the outside, a reduced drained friction at the inside – due to the upward flow reducing the effective vertical stresses – and a small suction pressure below the lid, which is required to maintain a constant flow. The friction capacity needs to be further reduced to account for the relative vertical movement of the caisson, which reduces the vertical stresses in the soil and hence the shear stresses in the soil-skirt-interface.

The difficulty is to decide upon the load and resistance factors which shall be applied. If a load case can potentially cause a failure of the structure, the full load and resistance factors according to the considered standard should be applied. However, if the loads for a considered load case can be controlled, for example by the turbine operation, the load factors may be reduced somewhat to acknowledge for the reduced uncertainty in the actual load amplitude. But also the failure mechanism may justify to apply somewhat lower safety factors. In case of a suction caisson in sand subjected to long-term tension loading, the structure may not experience a sudden failure, but may be pulled out gradually. If reduced load and resistance factors are applied, the serviceability needs to be ensured at any time, and an appropriate monitoring system should be installed, in order to apply the observational method. In addition, mitigation measures need to be prepared.

As the loading conditions of OWTs is of essentially cyclic nature, also the long-term tension loading is actually a cyclic load case. Thus, an appropriate cyclically degraded shear strength profile and corresponding stress-strain response need to be used. For that purpose assumptions need to be made on the distribution of the average long-term tension load and the cyclic amplitude. Depending on the considered load case, it may be assumed that the skirt-soil-interface at the outside of a caisson in clay may take the cyclic component and the soil below and inside the caisson may take the average component. Where this distinction should not be possible, an equally degraded strength profile may need to be assumed.

As the cyclic load components have relatively short period, the soil response of a caisson in sand will be essentially undrained to this component only. Thus, for a caisson in sand, the capacity needs to be checked for at least two cases; the resistance to the average tension load, and the resistance to combined cyclic and average load using an appropriate cyclic shear strength profile. When using the NGI framework based on cyclic contour diagrams, the strength and stress-strain response can be derived from diagrams where the average shear stress was applied drained in the corresponding laboratory test. Further information can be found in Andersen (2015).

The same considerations made for the short-term bearing capacity analysis on whether to include or to omit the effect of scour or scour protection, applies to the long-term bearing capacity analysis as well.

5 INSTALLATION

The installation is considered by many as one of the most challenging aspects of suction caisson application. However, experience from actual installations has demonstrated that installation in many different soil types and profiles is feasible. Moreover, the predicted penetration resistance and hence the required suction pressure agrees often reasonably well with the actual measured values (e.g. Sparrevik 2002, Colliat et al. 2007, Aas et al. 2009, Langford et al. 2012, Solhjell et al. 2014, Saue et al. 2017).

The governing mechanisms are well understood and several authors have developed calculation methods. Most methods can be applied in uniform and homogeneous soil conditions or soil profiles with perfectly horizontal layering. A general discussion of the installation process and calculation methods is presented in Subsection 5.1.

All existing calculation procedures have limitation, and there are a number of aspects which need particular attention during the actual installation, since they cannot be considered by the existing calculation models. Some of the most relevant aspects are presented in Subsection 5.2. Possible mitigation measures are discussed in Subsection 5.3.

5.1 Calculation methods

The often reasonably accurate predictions of the penetration resistance and hence required suction pressures is a result of extensive research in this field. A number of authors have proposed methods for calculating the penetration resistance and required suction pressure in both clay, silt and sand layers; particularly noteworthy are the models proposed by Houlsby & Byrne (2005a,b), Andersen et al. (2008) and Senders & Randolph (2009). These are based on model tests, field tests and prototype installations.

The penetration resistance is a function of the skirt tip resistance Q_{tip} and the skirt wall friction Q_{wall}. Q_{tip} may be estimated using a bearing capacity based approach or correlations with measured CPT resistances. Q_{wall} is a function of the skirt-soil-interface strength τ_{fric} and the effective skirt wall area. τ_{fric} can be assessed by means of laboratory tests, such as DSS tests or ring shear tests. Alternatively, τ_{fric} can be estimated using correlations with measured CPT resistances.

If the total penetration resistance $Q = Q_{\text{tip}} + Q_{\text{wall}}$ exceeds the submerged weight of the caisson and sub-structure $W' = W'_{\text{cais.}} + W'_{\text{substr.}}$, an additional driving force needs to be applied in order to penetrate the caisson to the required *Target Penetration Depth* (TPD). This is done by applying a relative under- / suction-pressure p_{suc} at in the inside of the caisson. The additional driving force is calculated by integrating the applied suction pressure over the horizontally projected area A_{suc} to which the pressure is applied. The maximum achievable penetration depth is reached when the total resistance Q exceeds the driving forces $W' + p_{\text{suc}} \cdot A_{\text{suc}}$.

Two main scenarios need to be distinguished; an *undrained* penetration and a *drained* penetration. A penetration is *undrained* if the soil permeability k of the penetrated layer is so low, that no significant amounts of pore pressure will dissipated during the actual installation process. In contrast to an undrained penetration is the pore pressure dissipation considerably in a *drained* penetration, which will affect the the stress regime in the soil. Due to the applied suction pressure, a seepage flow through the soil from the outside to the inside will develop in a high permeable soil layer. The upward flow in the soil plug inside the caisson causes a decrease of the vertical effective stresses σ'_v and hence a decrease of the inside side friction τ_{fric}. Furthermore, also the tip resistance will decrease due to the potentially high gradient around the skirt tip. Both yield a considerable reduction of the penetration resistance, meaning that a suction pressure has a twofold effect in a drained penetration; it increases the driving force and reduces the resistance in high permeable soils. Figure 8 illustrates the driving forces (top), stresses in the soil (left bottom) and resulting reaction forces (right bottom) acting on a suction caisson during installation in a high permeable soil.

The maximum possible suction pressure $p_{\text{suc,cav}}(z)$, which can be applied inside the caisson, is limited by the cavitation pressure. As detailed in Section 4, the cavitation pressure depends on the pump configuration, and is given by the sum of the atmospheric pressure $p_{\text{atm}} = 100kPa$ and the unit weight of water $\gamma'_w = 10\frac{\text{kN}}{\text{m}^3}$ times the depth of either

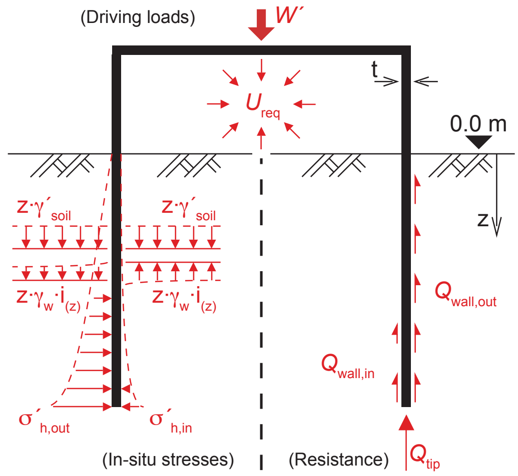

Figure 8: Forces and stresses acting on a caisson during suction installation in a high permeable soil; from Sturm et al. (2015)

- the submersion depth of the pump, given that the pump sits on top of the caisson lid, or
- the mudline depth, given that a closed system is established, where one hose is connecting the caisson with the pump and another hose returns the water from the outlet of the pump back to the mudline.

Though the pressure is theoretically higher for the latter case, it is technically more challenging. Furthermore, a considerable head loss can be expected due to the length of the hoses, which reduces the efficiency of the second solution.

The actual maximum achievable pressure $p'_{\text{suc,cav}}(z)$ is practically somewhat less than the calculated value $p_{\text{suc,cav}}(z)$, since the pump may not be able to go as low as to the theoretical pressure. Thus, a reduction of 20 to 50kPa of $p_{\text{suc,cav}}(z)$ may be considered in the design, where the reduction should be adjusted based on the pump specifications.

The actual allowable suction pressure $p_{\text{suc,all}}(z) \leq p'_{\text{suc,cav}}(z)$ may be limited by geotechnical and structural stability considerations. The skirt needs to take the load without to buckle. In the initial phase when applying the first time a suction pressure right after the self-weight penetration phase, the caisson is exposed to buckling failure due to the lack of any soil support above mudline. This is in particular critical for penetration in stiff clays at shallow depths. But also in the course of further penetration when the required suction pressure $p'_{\text{suc,req}}(z)$ increases with depth, the caisson may be exposed to buckling failure, if the inside soil support is low. This is typically the case for penetration in high permeable soils due to the upward flow of pore water in the soil plug reducing the stresses and hence strength.

Geotechnical limitations which can potentially affect $p_{\text{suc,all}}(z)$ are reverse bearing failure, primarily when penetrating in low permeable soils, and hydraulic heave failure, primarily when penetrating in high permeable soils. Some authors have included in their calculation models criteria and functions to ensure that these failures are avoided.

Somewhat more complicated is the penetration in layered soil profiles. Two scenarios need to be distinguished; sand over clay and clay over sand, where sand is a high permeable layer and clay a low permeable layer. Sand over clay is a common profile in many areas of the North and Baltic Sea, and the penetration through these do not pose a particular challenge. However, clay over sand is subject of ongoing discussion. Some authors have found in centrifuge

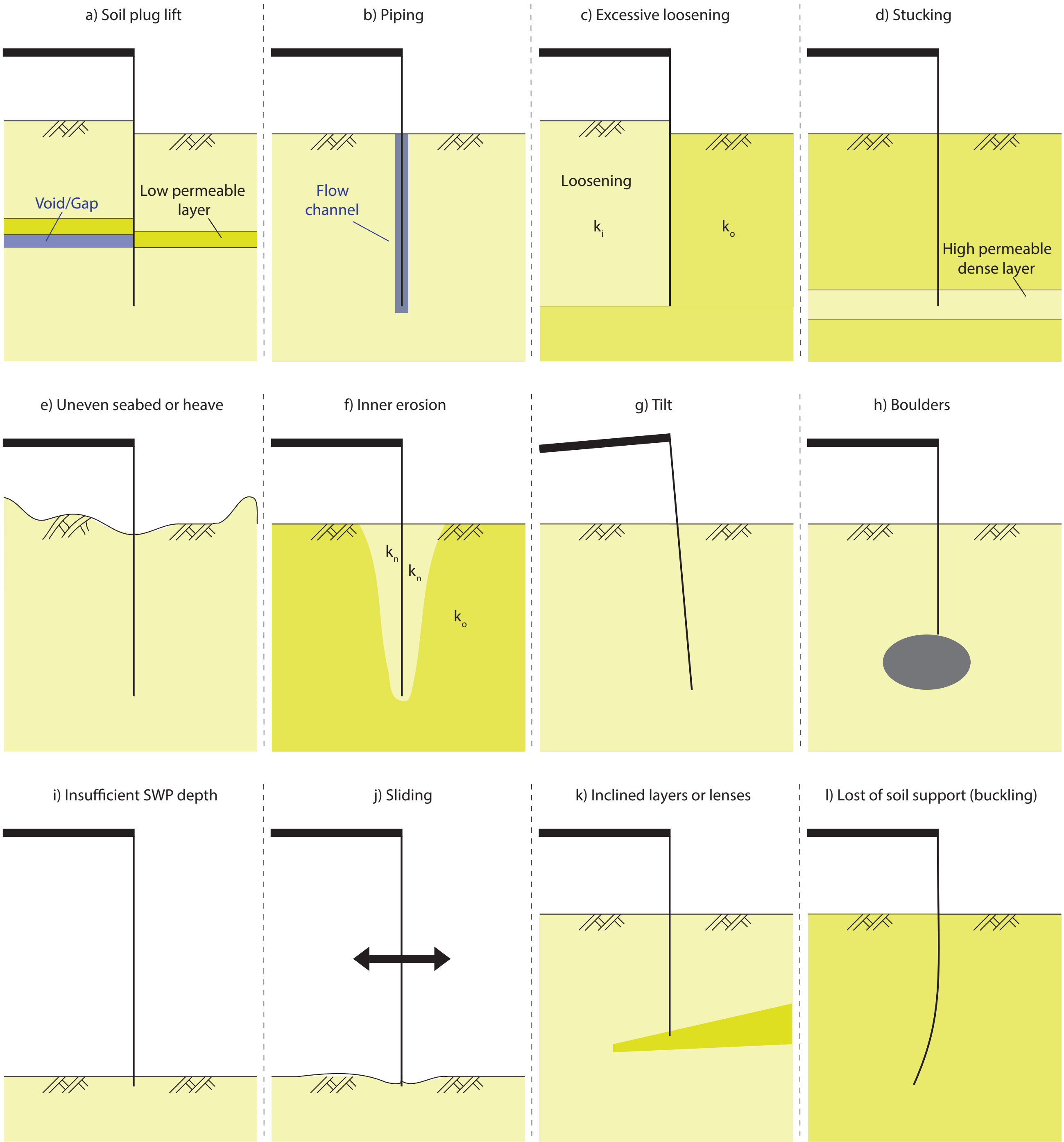

Figure 9: Some possible failure during installation, which cannot be predicted or insufficiently predicted with the available installation analysis models.

tests and/or small scale model tests, that penetration in the underlaying sand layer may not be possible without triggering a plug-lift failure (e.g. Cotter 2009). They recommend to stop the penetration above the sand layer, where the maximum allowable penetration depth into the clay is given by the shear strength of that layer below the skirt tip and the caisson geometry. However, installations of suction caissons in such layered soil profiles have demonstrated, that a penetration is in principal possible without a measurable soil plug-lift. In installations, where pore pressure sensors were placed at the in- and outside of the skirt walls above tip, it was found that the pressure gradient in the sand layer around the skirt tip, equates the gradient measured in installations in homogeneous clean sand deposits. That supports the assumption that a plug lift failure is not necessary. However, to generate a gradient in the sand layer covered by the clay, a seepage flow must have been developed. As the water cannot flow out through the soil plug in the caisson, the sand layer below the clay layer needs to take the water volume, meaning that the sand will reduce its density. Thus despite the fact, that the trial installations demonstrated that a penetration in layered soils is possible, it is recommended to penetrate relatively fast to avoid excessive loosening (soil plug heave) or eventually a soil plug lift.

5.2 Challenges

The methods mentioned in Subsection 5.1 are applicable for idealized conditions, i.e. uniform and homogeneous soil conditions or perfectly horizontal layering, vertical and parallel skirts, and no structural imperfections, to name but a few. However, there are a number of situations which are not covered. Some of the most common ones are illustrated in Figure 9.

Soil plug lift is a failure often discussed in connection with penetration in layered soils. In contrast to soil plug heave, soil plug lift will generate a water filled void or gap in the ground. That needs to be avoided in order to not negatively affect the in-place behavior of the suction caisson. Furthermore soil plug lift may prevent the caisson from penetrating to the TPD as the caisson will be filled up with soil. Practical experience form installations in layered soil profiles suggest to apply a minimum penetration rate in order to reduce the amount of water flowing into the soil plug and potential void.

Piping is a critical failure, as the volume of water per time flowing from the outside to the inside will increase considerably. If the water volume exceeds a certain amount, the pump may not be able to apply the required suction pressure and the TPD may not be reached. Furthermore, piping channels generated during installation can negatively affect the in-place performance, as the temporary suction during short-term loading will dissipate much faster which can potentially decreases the capacity significantly. Piping can be triggered by obstacles below the skirt tip which are dragged down while penetrating the caisson. These obstacles can leave a highly disturbed zone along the skirt wall. But also locally varying soil properties in combination with penetration at high suction pressures and hence penetration rate can trigger the generation of piping.

Excessive loosening may occur in installation in permeable soils. Due to the reduced vertical stresses and additional shearing of the material inside the caisson, the soil will dilate. That will affect the soil permeability and hence the seepage flow pattern, which can prevent the caisson to reach the TPD, since the required flow gradient in the soil cannot be achieved. Experience from installations in homogeneous sand deposits indicate that the degree of loosening correlates positively with the installation time, meaning that penetration at higher rate may potentially avoid excessive loosening. Sturm et al. (2015) proposes safety factors for the penetration analysis capturing the uncertainty of an excessive loosening.

Embedded and thin granular but relatively low permeable soil layers and lenses may cause the caisson to ***stuck***, if the required suction pressure exceeds an allowable value and if no seepage flow can be mobilized in that layer, which would reduce the tip resistance considerably.

An ***uneven mudline*** may prevent the caisson to reach the TPD, if not considered in the design of the so-called *free height*, which is the skirt length in addition to the calculated required penetration depth. The free height is typically measured from the original mudline and need to accommodate the soil plug heave, grout, and pre-installed filter material if applied, and seabed elevation. An uneven mudline can be also critical for the self-weight penetration phase, if the penetration resistance is locally too high preventing the whole caisson circumferences to penetrate and to establish a sealing, which is required to apply a suction pressure.

Soil layers with a gap graded grain size distribution curve, where the large diameter grains can form a stable matrix, are sensitive to ***inner erosion***. Fine grained particles are washed out of the soil due to the applied suction, and a very high permeable grain skeleton remains in the ground. Since the amount of water volume flowing into the caisson per time increases, the pump may not be able to apply the required suction pressure, meaning that the TPD cannot be reached.

Tilt of the caisson can be critical, as the penetration resistance increases. Installations with single caissons and anchors showed that a caisson is a self-stabilizing system, meaning that it rectifies due to the lateral soil resistance. However, if the caisson is constrained – for example when attached to a jacket – the loads can become critical for the sub-structure. Thus it is important to ensure a minimum degree of verticality of all caisson of a multi-legged sub-structure during the fabrication.

Boulders and other large obstacles can prevent the caisson to reach the TPD as the penetration resistance will increase considerably. If not identified in due time by the pump operator, the caisson skirts may be damaged or buckled. Small boulders may flip or pushed to the inside due to the suction pressure. Boulders can be detected by means of suitable geophysical site investigations. If boulders are met, the caisson may be retrieved and relocated, given that the structure has not been damaged.

If the submerged weight of the caisson and substructure is too low, the ***self-weight penetration*** may not be sufficient to ensure a seal at skirt tip level, which is necessary to apply a suction pressure.

Sliding during the lowering and touch-down phase of the caisson may remove soil in the vicinity of the skirt tip, preventing sufficient seal, which is necessary to apply a suction pressure. Hence, allowable sea states for the installation should be assessed in the design.

Particularly challenging is the penetration of profiles comprising ***inclined layers and lenses***. In case of an inclined clay layer or lens below or in a sand layer, respectively, the pore pressure gradient at skirt tip level may become critically high, since the changed drainage conditions will affect the seepage flow pattern. That can potentially trigger a local failure or piping along the skirt at the side of the caissons which is still in the sand. In case of an inclined sand layer or lens below or in a clay layer, respectively, the penetration resistance may considerably increase since a seepage flow, as described for perfectly horizontally layered profiles, may not be established. Furthermore, the soil resistance will be asymmetric and potentially causing a tilt of the caisson or local moment in the leg of the sub-structure, respectively. However, the deeper the caisson has penetrated the more soil support at the outside of the caisson is available, which can compensate for the asymmetric penetration resistance.

Imperfections or ***buckling*** at skirt tip level can increase the penetration resistance considerably and also affect negatively the in-place behavior of the suction caisson. Thus the allowable suction pressure should not be exceeded and a maximum tolerance for imperfections and misalignments shall be considered in the fabrication.

5.3 Mitigation measures

In case that the penetration resistance is higher than predicted, the required suction pressure to penetrate the caisson will be higher as well. Where it is not possible to apply the required suction pressure due to geotechnical, structural or technical limitations, one may consider to abort the penetration or apply mitigation measure in order to try to penetrate further until reaching the TPD. The decision should depend on the achieved penetration depth as well as on the course of the penetration process. If for example, the caisson has penetrated 80 or 90% of the TPD and the penetration resistance had been continuously higher than predicted in the design, it may indicate that the foundation has already sufficient capacity for the actual reached penetration depth. More challenging is the impact of the stiffness for a lower penetration depth.Sturm & Mirdamadi (2017) propose a reliability based method for assessing foundation stiffness, which can be used during installation, on which basis a decision can be made if the caisson(s) need to be penetrated further

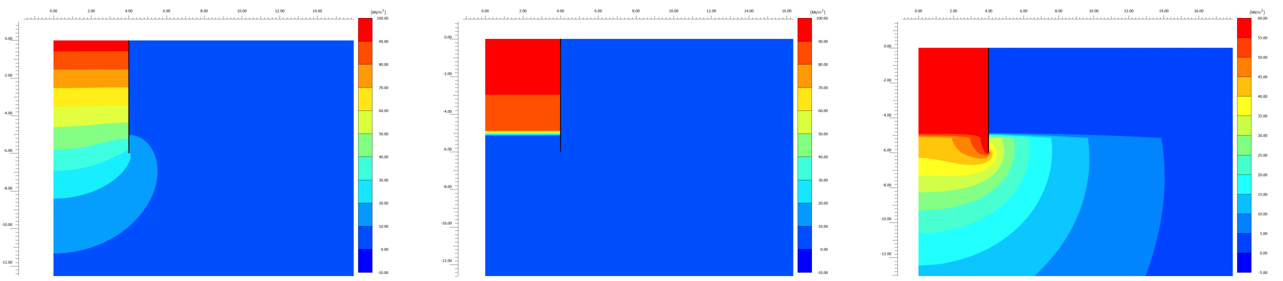

Figure 10: Pressure gradient in the soil for a clean sand profile (left), a sand profile with a clay layer a skirt tip (center), and sand profile with a clay layer a skirt tip with a stepped skirt (right)

by means of applying mitigation measure.

Two categories of mitigation measures need to be distinguished; preemptive and reactive mitigation measures. Preemptive methods are those which have been considered before the actual installation. Reactive methods are applied during the actual installation and do not require any particular structural considerations.

A simple but often effective reactive mitigation is to ***ballast the structure*** to increase its weight. This can help in many situations discussed in Subsection 5.2, for example in case of piping, inner erosion, stucking, and insufficient self-weight penetration.

Another reactive mitigation measure is to ***cycle the suction pressure***, which is illustrated in Figure 11. Cycling has been applied in many installations to successfully penetrate to the TPD.

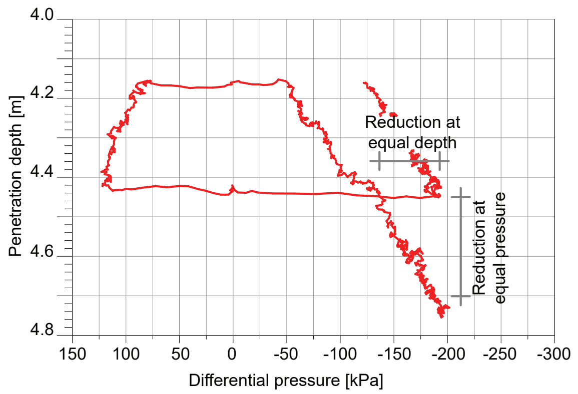

Figure 11: Suction pressure versus vertical displacement during cycling as a reactive mitigation measure

Due to the cycling of suction pressure, the caisson will move somewhat up and down, which will affect the soil in the vicinity of the skirts. Over-consolidated clays will be remolded due to the cycling and the shear strength will decrease. This will mainly affect the skirt wall friction Q_{wall}. Cycling when penetrating in sand layers can be beneficial as well, as the soil below the skirt tip will dilate due to the unloading, which will decrease the tip resistance (see Cudmani & Sturm 2006). The effectiveness of cycling can be described by considering the reduced suction pressure at equal penetration depth or the achieved additional penetration depth at equal suction pressure; both illustrated in Figure 11.

A systematic evaluation of NGI in-house installation data, where the suction pressure was cycled, showed that both measures are equivalent, though more practical relevance has the increase in depth at equal pressure. Further, a general tendency can be observed that the effectiveness of cycling increases with increasing penetration resistance. This may be expected as the decrease in strength due to remolding is higher for over-consolidated material than for normal consolidated material. In fact cycling may have a negative affect on the resistance in normal and low consolidated clays, as the soil may partially drain and by that increases its strength. The soil sensitivity may provide an indication of the expected efficiency of cycling.

The effectiveness of cycling depends further on the cyclic displacement amplitude which is also indicated by the results presented by (Cudmani & Sturm 2006). The larger the displacement amplitude the more effective the cycling, which can be explained by an increased shearing of the soil. In addition, the cycling rate may have an effect on the effectiveness as it allows the soil to drain somewhat.

A preemptive mitigation measure is a ***stepped skirt***. A stepped skirt has different wall thicknesses over the height. Similar to driving shoes used for piles, a stepped skirt, as considered herein, will be thicker at the tip compared to the rest of the skirt wall. This will generate a thin gap or disturbed zone along the skirt, which needs to be at the inside of the caisson, in order to be effective for the penetration. The stepped skirt functions as a *friction breaker*.

Figure 10 shows the required pore pressure field to penetrate a caisson in a clean sand deposit (left figure). When penetrating through a thin clay layer embedded in the sand, the seepage flow is prevented and the required gradient around the skirt tip cannot be achieved (center figure). The caisson cannot be penetrated further. However, when using a friction breaker, a gap or disturbed zone along the skirts, and in particular in the thin clay layer, may be generated, which allows to establish a seepage flow from the outside to the inside. Due to the different seepage flow pattern, the actual required suction pressure to achieve the same pressure gradient at skirt tip is less than the required suction pressure in a clean sand profile (right figure). This indicates that a friction breaker can be a very effective mitigation. However, due to the disturbed zone, the in-place performance of the caisson may be negatively affected, since the suction generated during short-term loading will dissipate faster. And also the resistance to long term loading may be reduced compared to a caisson with constant wall thickness.

Another preemptive mitigation measure is the ***water injection*** system, where at a pipe with nozzles through which water can be injected into the soil is arranged at the skirt tip. Purpose of the water injection system is to reduce the penetration resistance. This is achieved in sand by a loosening the soil at skirt tip, and in clay by remolding the soil along the skirts. Injection of water appears to be most effective in combination with cycling, where the amount of injected water is adjusted to the void generated by the skirt when moving upwards. This will have a minimal effect on the soil state after installation. Water should be injected in any case at low pressure to avoid excessive soil disturbance, which can potentially negatively affect the in-place behavior of the caisson. Aas et al. (2009) reports results of a water injection system used in layered profiles.

6 FOUNDATION SERVICEABILITY

The foundation serviceability is probably one of the most imprecisely predictable aspects in geotechnical engineering. Serviceability in this context means settlements, lateral displacements, and rotation or tilt, respectively. Most critical is the tilt of an OWT as it affects the operation of the turbine. Pure settlements are typically less critical, though some secondary steel components such as the J-tube or the boat lander may be affected. The lateral displacements are typically small, and have practically no relevance in the projects considered so far. Thus main focus is given in the following on differential settlements or tilt of multi-legged substructures or mono-caissons, respectively.

In order to assess the *Serviceability Limit State* (SLS), corresponding limit values need to be defined. These are typically given by the turbine supplier. In addition, the maximum tilt may be limited in order to reduce operational loads, which is in particular relevant for multi-legged OWTs; increased average tilt yield typically an increased average tension load.

Three different types of settlement/tilt components need to be distinguished:

- ***Static*** settlement/tilt due to the submerged weight of the OWT.
- ***Peak*** settlement/tilt due to a ULS loads.
- ***Accumulated (average)*** settlement/tilt due to cyclic loading from wind, wave and operation loads.

Following traditional geomechanics, the *static* settlement/tilt can be further distinguished into immediate-, consolidation- and creep-settlements/tilt. The corresponding values can be computed using well established geotechnical calculations procedures.

The *peak* settlement/tilt can be assessed by means of a monotonic pushover FE analysis. The soil model needs to be calibrated in order to reproduce the correct stress-strain-behavior of the soil. Where necessary, the decrease of strength and stiffness due to cyclic loading needs to be included. This may be done for example by using a total-stress-based model with adjusted stress-strain curves based on cyclic contour diagrams, or an effective-stress-based model to which a pore pressure field is superimposed; see also Section 3. The peak settlement/tilt represents actually the maximum expected value, meaning that the load case considered is in general the same used in the ULS capacity analysis, but without applying load and resistance factors. Practically, this value is less relevant, as the settlement – and more important the tilt – will immediately decrease again in the subsequent unloading. Further, the OWT may not be in operation during the ULS event, for which reason the allowable serviceability limit criteria may not apply.

Most relevant is the assessment of the *accumulated average* settlement/tilt, which, however, is also one of the most challenging components. Thereto, different strategies can be applied. One of the most conservative assumptions is to take all load cycles which occur during the lifetime of an OWT and sort them in ascending order. This sorted cyclic load history can be applied in a calculation procedure, for example in the NGI method (Jostad et al. 2014, 2015b), or in an FE analysis using the high-cyclic accumulation model (Niemunis et al. 2005, Wichtmann et al. 2010).

Since small load cycles will typically not contribute significantly to the accumulated total displacements, a different approach has been followed in more recent projects. The design storm used in the ULS analysis, which is based on a 50 years wind wave event, has been extrapolated to other storm events with different recurrences using a Gumble distribution. That enables to calculate the displacements for a given cyclic history, but at different scaling factors. The accumulated total displacements can be than determined by summing up the the calculated displacements for the different storm events multiplied with the number of occurrences of the corresponding event.

However, both approaches miss out important aspects. Different to engineering materials such as steel or concrete, soils are sensitive to the order of cyclic loading. While large cyclic load amplitudes can cause a degradation of the soil strength and stiffness, can the soil regain strength and stiffness when subjected to lower cyclic load amplitudes, which can be described as *self-healing*. The influence of varying strength and stiffness of the soil on the settlement and tilt depending on the cyclic loading conditions is described in Sturm (2009) and Sturm (2011) at the example of skirted shallow foundations. It is introduced the concept of the so-called *cyclic attractor*, which is a value being asymptotically approached by a given cyclic load history with constant amplitude. Given that the foundation is stable for all relevant cyclic load histories, the value of the cyclic attractor is proportionally to the composition and intensity of the cyclic load history. Thus, for the assessment of the cyclic accumulated average tilt of a stable OWT, only the cyclic attractor for the largest cyclic load event needs to be determined, meaning that only one cyclic load history needs to be considered in the design. Cyclic attractors can be found for the accumulated average tilt of shallow foundations. However, no attractors exist for vertical settlements of shallow foundations.

7 FOUNDATION STIFFNESS

The local foundation stiffness is the link between the geotechnical and structural designer. Foundation stiffness is an output of the geotechnical analysis, but is not part of the actual sizing, i.e. capacity serviceability and installation analysis. However, the results of the stiffness analysis will affect the design basis as illustrated in Figure 2. As detailed in Subsection 1.2, foundation stiffness can be provided as single secant stiffness values, nonlinear tangential stiffness values, or full linear or non-linear stiffness matrices including coupling terms if necessary. This needs to be agreed in upfront with the involved disciplines and may be included in the load document. Further, it need to be agreed on the load cases for which the foundation stiffness shall be assessed.

Foundation stiffness can be established using simplified analytical methods or advanced FEM based methods. Gazetas (1991) has proposed a large number of closed form equations for assessing the stiffness of different foundation types and ground conditions. In contrast to the simplified methods, which consider linear soil properties, the FEM allows to capture the non-linearity of the soil and the flexibility of the structure, i.e. the soil-structure-interaction. The methods used for assessing the foundation stiffness should be adjusted based on the stage of a project and anticipated degree of optimization. In an early stage of a project, i.e. feasibility and concept study, simplified analytical methods may be used, whereas in a FEED and Detailed Design the FEM may be more appropriate.

Typically the foundation stiffness is provided as a range with high-, best- and low-estimate. The width of the range should be narrowed down during the project and every design iteration. No attempts should be made by the geotechnical designer to assume any particular soil profile which may be conservative for the structural design. The selected soil profiles should rather reflect the inherent uncertainties of the soil state after installation and load conditions.

Two different type of stiffness values need to be distinguished; stiffness values for the structural utilization (denoted in the following ULS load case) and stiffness values for the load assessment,

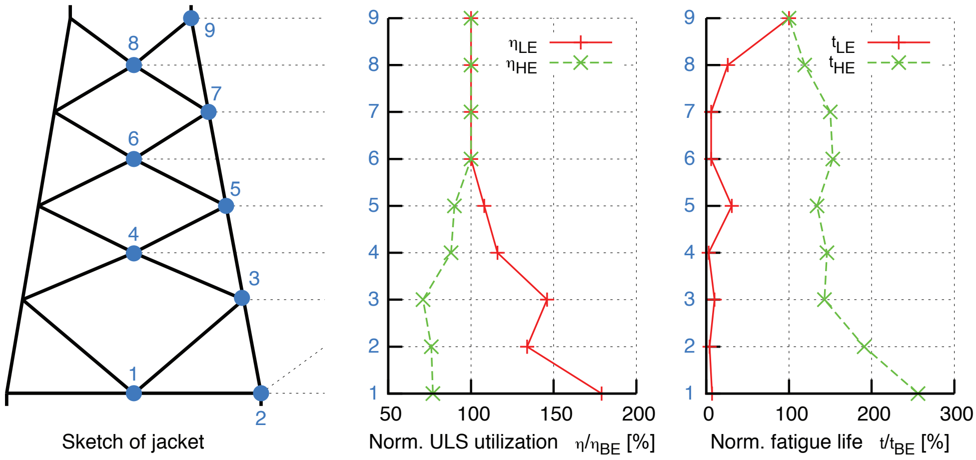

Figure 12: Impact of the HE and LE ULS and FLS stiffness on the jacket response

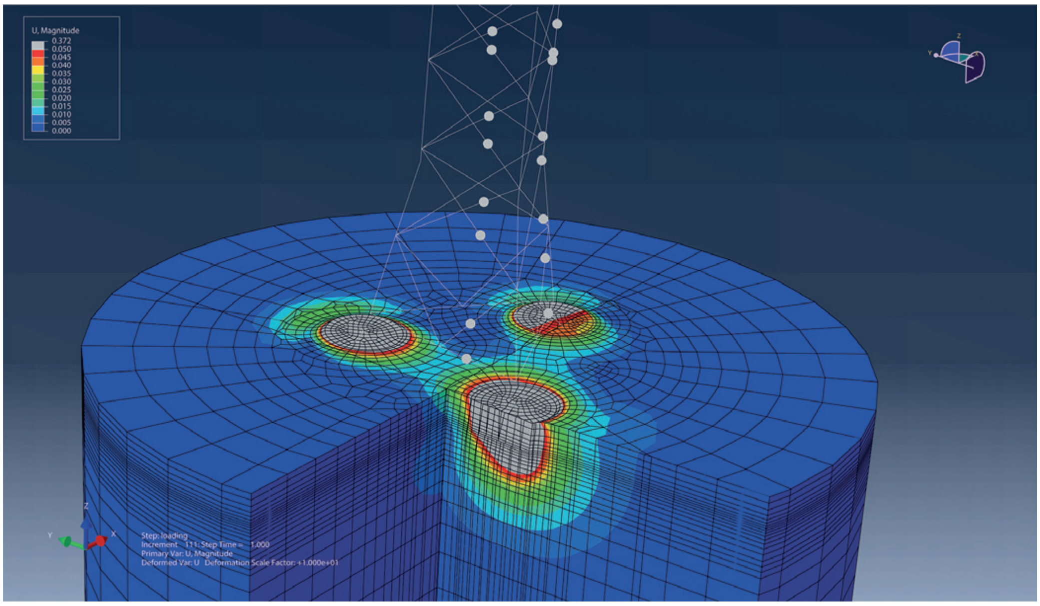

Figure 13: Example of a global model used for assessing local foundation stiffness

structural fatigue analysis and eigenmode analysis (denoted in the following FLS load case). In general the ULS stiffness is non-linear due to the high mobilization of the foundation, wheres the FLS stiffness appears often to be linear due to the significant lower load amplitudes. The soil profiles used in both the simplified and the advanced analysis need to reflect the different loading conditions. The FLS load case is governed by the cyclic amplitude, where the average or mean load is of less important.

Table 2: Variation of the HE and LE ULS and FLS stiffness normalized by the corresponding Best Estimate (BE) stiffness

	ULS	FLS
High Estimate (HE)	288%	189%
Low Estimate (LE)	37 %	69%

Figure 12 shows the impact of the high-, best- and low-estimate foundation stiffness on both the fatigue life and the structural utilization of a jacket supported by three suction caissons. Basis for this analysis are the structural and geotechnical properties at the end of the first iteration of a generic FEED study. The corresponding values are normalised by the best-estimate values. The corresponding normalized stiffness values are listed in Table 2.

It becomes apparent that the structural utilization scales approximately proportional with the ULS foundation stiffness. The variation in utilization, however, is not reflecting the relatively large range in ULS foundation stiffness values, meaning that the jacket is less sensitive to variations in foundation stiffness. In contrast to that, is the impact of the high- and low-estimate FLS stiffness on the fatigue life very pronounced. Even though, the low estimate FLS stiffness is 69% of the best-estimate FLS stiffness, the fatigue life decreases to less than 10%.

This example illustrates, that an optimization of an OWT can be challenging, if the range of foundation stiffness values is too large. Furthermore the implications of assumptions in the geotechnical design on the structural design can be hardly estimated without performing corresponding structural analysis.

When assessing the foundation stiffness for a mono-caisson using the FEM, the loads provided in the design basis can be directly applied to the caisson. When using the calculated foundation stiffness values in the subsequent structural analysis the updated loads are typically of similar order and ratio. That means the load-stiffness iteration – outer loop in Figure 1 – converge relatively fast.

This is somewhat more complicated for multi-legged sub-structures. Depending on the footprint width, caisson dimensions, load conditions, and ground conditions, the loads can be redistributed between the different legs due to both the flexibility of the sub-structure – i.e. the jacket – and the interaction of the caissons in the ground. Using FE models of single caissons only will not capture the redistribution correctly. More accurate would be to model both the caissons, the sub-structure and the soil. This is denoted *global FE model* and is shown in Figure 13. The difference between a global model and an integrated model is the type of analysis. A global model is typically used in monotonic push-over analysis, whereas an integrated model is used in a time domain analysis.

Advantage of a global model is, that the loads and foundation stiffness values can converge relatively fast in just some few load-stiffness iterations. However, such analysis are time consuming, and – depending on the stage of a project – single caisson models may be used instead, though the accuracy is less good. Based on recent experience, it is recommended to use global FE model in FEED and detailed design at some representative locations of an offshore wind farm. The identification of relevant locations can be reasonably well done using the simplified methods or single FE models, as the *error* is in general proportional.

It may be noted that global FE models are particular relevant for assessing ULS foundation stiffness due to the large mobilization. For FLS load cases, single caissons models are sufficient. An exception is the assessment of foundation damping, both for ULS and FLS. If FE analysis is used for determine foundation damping, the complete soil may be modeled to capture the interaction and larger soil mass.

In addition should be mentioned that attempts are undertaken to use macro elements in structural analysis. However, the macro elements require a calibration of the particular site and caisson geometry, which can be done for some models using the above described methods.

8 SOIL REACTIONS

Soil reactions are, like foundation stiffness, an output of the geotechnical design, but are not considered in the geotechnical sizing of the caisson. However, the results of the stiffness analysis can affect the design basis as illustrated in Figure 2. Soil reactions are typically provided as loads distributed over the skirt(s) and lid which is in contact with the soil. The assessment of load reactions is difficult and depends on many factors, such as the flexibility of the caisson, the soil layering, the recent cyclic load history, and the actual applied load for which the soil reactions shall be provided.

Soil reactions are used for the structural design of the caisson, and need to be provided for two different cases; for installation and for in-place conditions. The soil reactions during installation will typically govern the required thickness and shape of the skirt wall – assessed in buckling analysis – whereas the in-place soil reactions will primarily govern the design of the caisson lid. Of particular importance is the distribution of the loads carried by the lid and the skirts.

Figures 14 and 15 show the result of FE analysis of five suction caissons with different geometries and soil conditions, subjected to short-term compression or tension loading, respectively. The load conditions are representative for a compression or tension leg of a multi-legged sub-structure. From Figure 14 becomes apparent, that the load taken by the lid scales proportionally with the

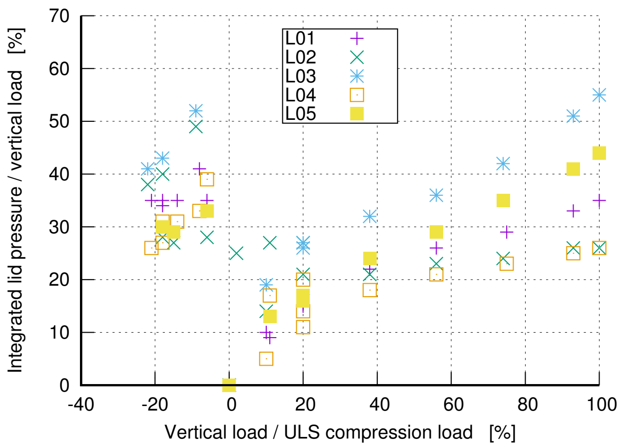

Figure 14: Ration between the loads carried by the lid and the skirts as function of the load amplitude

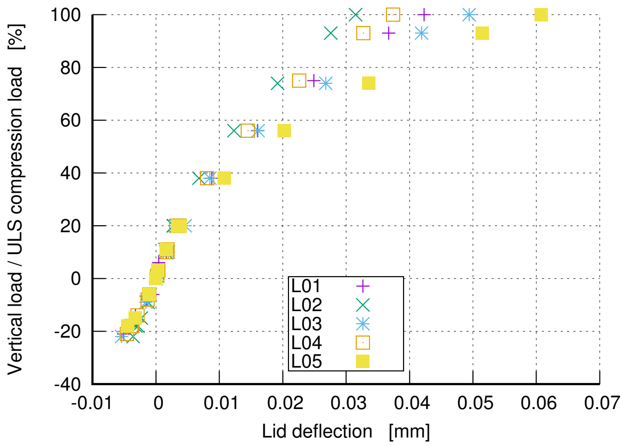

Figure 15: Deflection of the lid as function of the load amplitude

applied load. Small load amplitudes are taken solely by the skirts. The increase in lid pressure is linear and larger for tension loading, though a considerable scatter can be observed on the tension side.

Figure 15 shows the displacements of the caisson at the loading point, which is in this case the top of the lid. It appears to be very linear for low and medium sized load amplitudes, whereas it becomes pronounced non-linear for larger loads. Variations of the lid and caisson stiffness showed, that the linear response at low and medium sized loads is directly proportionally to the caisson stiffness. The soil non-linearity becomes first visible at larger loads. This demonstrates the importance to select the correct geotechnical-structural interface point discussed in Section 1.2.

Thought the results are encouraging, the calculation of the actual values using the FEM is very time consuming and sensitive to the modeling. The stresses depend on the size, number, shape, and type of continuum element used in the analysis. Further, the tip resistance is difficult to assess due to the ratio of wall thickness to caisson diameter, which requires exceptionally small elements along the skirt and below the tip. Thus, the FEM may be used as a complementary method for assessing the soil reactions.

In conventional offshore geotechnical engineering, soil reactions are established based on engineering judgement and are provided as so-called *unit loads*. Unit loads scale proportionally with the applied total load. Assumptions are made on the distribution; similar to the one shown in Figure 14. Upper and lower estimates of the distribution need to be provided.

Soil reactions for the installation can be derived from the actual penetration analysis, where both skirt wall friction, skirt tip resistance, and required suction pressure are calculated. More complicated is to assess the soil support during installation. It may be assumed that almost no support is provided for penetration in sand where significant flow gradients in the soil plug is expected (drained penetration). Some support may be assumed for undrained penetration, which may be estimated based on suitable laboratory tests.

9 OTHER ASPECTS

9.1 Grouting

An issue often discussed is the necessity of grouting. Grout is used to fill the void between the lid and the soil at the inside of the caisson. Most suction caissons and anchors installed so far were grouted with only some few exception. Main reason of using grout for suction caissons of bottom fixed OWTs is to reduce or avoid potential (differential) settlements, and *pumping*-effects. Due to cyclic vertical loading, the water cushion below the lid is exposed to continuous pressure pulses, which can trigger a local piping failure along the skirts. In addition, a lack of soil/grout support below the lid will cause large local stresses and moments in the lid. All loads need to be transferred trough the lid into the skirts. This requires a thick, massive lid, to avoid large deflections and fatigue issues.

In order to improve the bearing behavior of the lid and and to avoid the afore mentioned negative effects when omitting grout, structural components may be applied to replace the grout. Stopper-pods, which are elements made of steel, hard rubber, or composite materials, can be attached under the lid. The caisson need to penetrate until the pods are in contact with the mudline enabling to transfer loads from the lid into the soil. Alternatively, small ribs or T-beams may be welded under the lid dividing the base into compartments. The structural elements may be slightly cone-shaped to allow partially penetration into the ground in order to compensate for inclined mudline or uneven soil heave. If, an uneven soil surface is expected, a jetting system may be used to flush the upper soil and by that generating a slurry mixture which slowly consolidated during the final phase of the installation.

Disadvantage with using structural and jetting systems is, that the soil which is in contact with the structural components is soft, and the stresses in the lid may be concentrated to some few points only. Based on current experience, the use of grout seems to be appropriate to optimize the lid geometry. However, the cost savings due to an optimized lid geometry, needs to be compared to the costs of the additional offshore work for the grouting.

9.2 Integrated design approach

As mentioned in the several sections, so-called *integrated analysis* are performed in OWT design (e.g. Krathe & Kaynia 2016, Page et al. 2016, Skau et al. 2017). Such analysis are particularity suited for structural analysis, such as in the load or eigenmode assessment. Integrated analysis are not appropriate for the foundation sizing, though some macro-elements may indicate this possibility. The foundation response can be very sensitive to soil layering and size of the caisson and skirt, which cannot be considered by the macro-elements. Furthermore, also other aspects than capacity and serviceability may be design driving as detailed in the corresponding sections in this contribution.

Until today, the geotechnical sizing is uncoupled from the structural analysis and it is not expected that this may change in the near future without compromising an optimization of the caisson geometry.

9.3 Earthquake loading

In some parts of the world, earthquake loading and earthquake induced liquefaction needs to be considered in the design. Both can be considered in the design using existing methods. The loads from the earthquake represent just another load case to which particular soil conditions need to be assigned in the corresponding analysis. Kaynia (2017) provides a comprehensive introduction to the design of OWTs subjected to earthquake loading.

9.4 Observational method

OWTs supported by caissons are a relatively new concept and long-term experience does not exist yet. Thus, the observational method may be considered in current projects. It can be applied during both installation and operation. In order to use the observational method, it is important that the failure is ductile, which allows to initiate mitigation measures(s) in time.

The observational method is a combination of predictions and measurements. The behavior of the OWT is calculated using existing methods. Further, ranges of allowable values need to be defined. If exceeded, mitigation measures need to be initiated, which need to be planned in the forehand.

Examples of mitigation measures during installation are presented in Subsection 5.3. The decision value is typically the required suction pressure, which shall be provided as a range with high and low estimate. If the high estimate value is exceeded, the mitigation measures may need to be applied. The same concept can be applied for the serviceability. When a maximum tilt is exceeded the OWT may need to be rectified.

For a successful application, it is important to plan both an appropriate health monitoring system and mitigation measures. Aspects of monitoring systems are presented by Sparrevik & Strout (2015). The usefulness of such systems is presented by Schonberg et al. (2017) at the example of the Borkum Riffgrund 1 Suction Bucket Jacket.

9.5 Wind farm design

So far, only single OWT foundations were considered herein. An iterative approach as outlined in Section 1.1 at each turbine location of an *Offshore Wind Farm* (OWF) would require considerable time, which may not be possible in the given project time frame. Thus, a clustering may be introduced. Typically, the clustering is a based on the water depth, since the loads are expected to be very similar for a given depth. Foundation capacity and installation analysis can be relatively quickly performed. The results of these can be used for the sizing. If FEM is used for the capacity analysis, as described by Jostad & Andersen (2015), the foundation stiffness can be qualitatively estimated. Based on that, the softest and stiffest location within a cluster can be identified. These two can then used in the stiffness and soil reaction analysis, representing the parameters used in the iteration process, given that the same sub-structure will be applied in the cluster.

10 OUTLOOK

In this article a general overview of the geotechnical design of suction caissons for OWTs has been provided. The different phases of the design were detailed, and the relevant aspects were outlined. The general design of suction caissons is reasonably well understood and many authors have proposed numerous methodologies for specific geotechnical calculations, such as capacity, installation and stiffness analysis. Only some few were mentioned in this article; mainly those which are familiar to the author from personal experience. The reader is encouraged to get himself an overview of the numerous methods proposed in the literature. The article at hand may serve as a guideline to evaluate the suitability of a method for the particular design aspect.

Due to the increased interest in suction caissons for OWTs, a number of researchers and practitioners are currently working to continuously advance the knowledge. Several of the currently designed and installed OWTs are equipped with comprehensive health monitoring systems, which will provide further inside into the short- and long-term behavior of suction caissons.

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