Offshore Wind Accelerator

Suction Installed Caisson Foundations for Offshore Wind: Design Guidelines

February 2019

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1. INTRODUCTION

1.1 Objectives

The main objectives of this document are to:

- Provide design guidelines for SICF for offshore wind applications;
- Increase confidence in the use of suction caissons in the offshore wind industry;
- Provide greater clarity to designers and developers on the key issues to consider when designing suction caissons.

The document provides guidance on how to undertake the geotechnical design of suction caisson foundations. Therefore, it is expected that these guidelines will be utilised by experienced geotechnical engineers.

1.2 Offshore Wind Accelerator background

The Offshore Wind Accelerator (OWA) is a world leading industry-led collaborative programme between the UK Government, Scottish Government and industry, designed and led by the Carbon Trust. The multi-million-pound programme has been running since 2008 with the aim of reducing the cost of low carbon electricity produced by offshore wind farms.

Suction Installed Caisson Foundations (SICF) have been identified as having the potential for reducing installation costs and offshore vessel time for offshore wind projects. Several developments have utilised suction caissons. However, there is still a lack of knowledge and some uncertainties regarding this foundation type. The Offshore Wind Accelerator sponsored the development of this design guidance document as a method of addressing this issue.

1.3 Suction Installed Caisson Foundations (SICF) background

Suction caissons, also referred to as suction anchors, suction piles, suction buckets or SICAs (Suction Installed Caisson Anchors) are an offshore foundation type developed in the 1990s for offshore oil and gas applications (Bye et al., 1995); (Andersen et al., 2005). A suction caisson foundation can be described as a large steel cylinder, open at the bottom and sealed at the top. After an initial penetration under its own weight, the caisson is embedded into the seabed to target depth by creating a negative (suction) pressure inside the caisson, and the resultant pressure differential across the top plate effectively pushes the caisson into the seabed.

Some potential benefits of Suction Installed Caisson Foundations (SICF) are:

- They can be integrated with the jacket/TP (Transition Piece) substructure and installed in a single operation, potentially reducing offshore time and the number of offshore lifts.
- The cost of the installation spread is reduced, as pile-driving equipment is not required. Pumps are required but these tend to be lighter and easier to handle than piling hammers.
- SICF installations are almost silent, meaning less disturbance for marine life and therefore no requirement for noise mitigation equipment which can add to the cost and timescales of pile installation.

- The weight and seabed footprint area of SICF structures is generally much less than Gravity Base Structures (GBS), meaning greater flexibility with installation vessels and potentially little or no seabed preparation.
- At the end of its service life, a SICF can be completely removed from the seabed by reversing the installation process, leaving no steel behind.

The main disadvantage is that SICF installation is particularly sensitive to ground conditions, and even relatively small variations in soil composition and/or strength could have a significant effect on installation design.

Further background information on SICFs is provided in Byrne and Houlsby, (2003); Ibsen and Brincker, (2004); Houlsby et al., (2005); Ibsen et al., (2005), (Sturm (2018).

1.4 Stakeholders

The various stakeholders involved in an offshore wind project approach SICFs from different perspectives:

- A developer's priority will be risk management (technical, temporal and financial), and delivering the project on time with costs as low as reasonably possible;
- Designers will focus on design parameters, calculation methods and SICF dimensioning;
- Certifiers will focus on design and operational/installation risks and ensure calculations are reasonable and prudent;
- Fabricators will focus on design requirements and SICF dimensioning;
- Installation contractors will focus on installation risks and mitigation methods;
- This document has been prepared to provide a balanced guideline addressing the needs of all stakeholders.

1.5 Extents and limitations

These design guidelines are for the **geotechnical** design of suction caissons for substructure foundations, assuming the work is undertaken by a suitably competent engineer and do not cover:

- Structural design of suction caissons (although some guidance for buckling design related to installation is presented);
- Design of suction anchors for floating offshore wind developments, although similar principles could be used.

Where appropriate these guidelines reproduce formulae and recommended values from other relevant design standards and publications to avoid the requirement to reference multiple documents. It is the responsibility of the end user to ensure that the references are still current, and the values detailed are still applicable.

These guidelines present the current state of knowledge and good industry practice for SICF design. They do not provide a detailed State-of-the-Art review, nor propose design methods that are still topics for research. Therefore, this document cannot provide definitive guidance on topics such as repeated loading into tension, permanent deformations due to cyclic loading, foundation damping, and installation in all soil conditions. However, where relevant, the reader is directed to recent publications for further guidance and to new methods which are under development, with the necessary caveats.

1.6 Document structure

Starting from Design Principles (Section 3), Site Conditions (Section 4) and SICF loading (Section 5) the document then outlines Installation Design (Sections 6 and 7), because the SICF must be installable, before moving to In-Service Design (Section 8) and Decommissioning (Section 9).

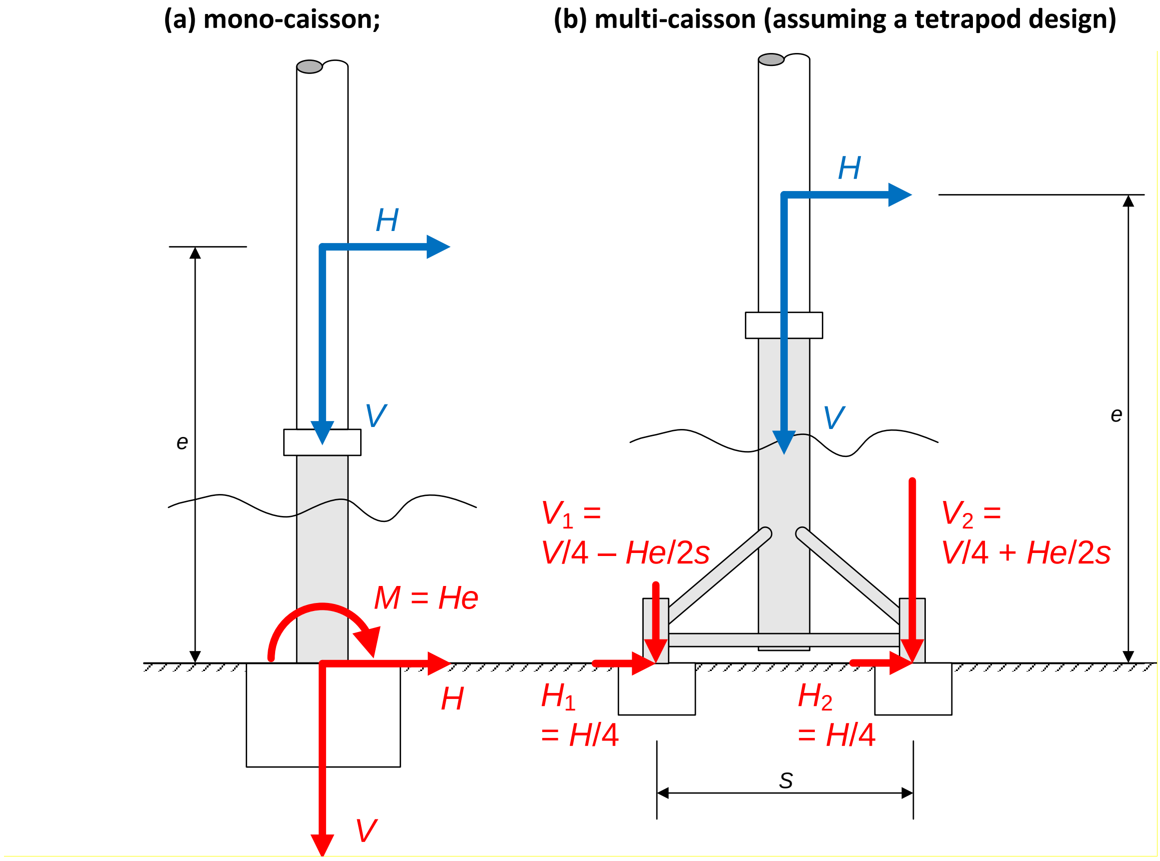
2. TERMINOLOGY, NOTATIONS AND ABBREVIATIONS

2.1 Terminology

Suction Installed Caisson Foundations are used with two main substructure configurations (see Figure 2-1) as detailed below with indicative loading regimes:

- Mono-caisson: where the sub-structure is supported by a single large diameter caisson. Typically, the response and capacity of the caisson under the influence of overturning moments is the key design factor.
- Multi-caisson: where the sub-structure is typically supported by 3 or 4 caissons. Figure 2-1(b) shows a tetrapod style sub-structure but tubular jackets are more common. Typically, the vertical response and capacity of the caissons is the key design factor, and specifically the capacity of the "upwind" caisson which may experience tension uplift.

Figure 2-1: Typical structural configurations



*Blue arrows show resultant applied structural loads

**Red arrows show the consequent loads on the foundation (for case b, the loads are for a 4-footed jacket assuming equal distribution of horizontal load and no moment transmitted to individual foundations)

There is a wide variety of terminology used for discussing suction caisson foundations. The terminology used in this document is presented in Figure 2-2 and Table 2-1. The table includes some widely used terms which may not fully describe the physical reality, but which have been retained for continuity and completeness.

Figure 2-2: Suction installed caisson components

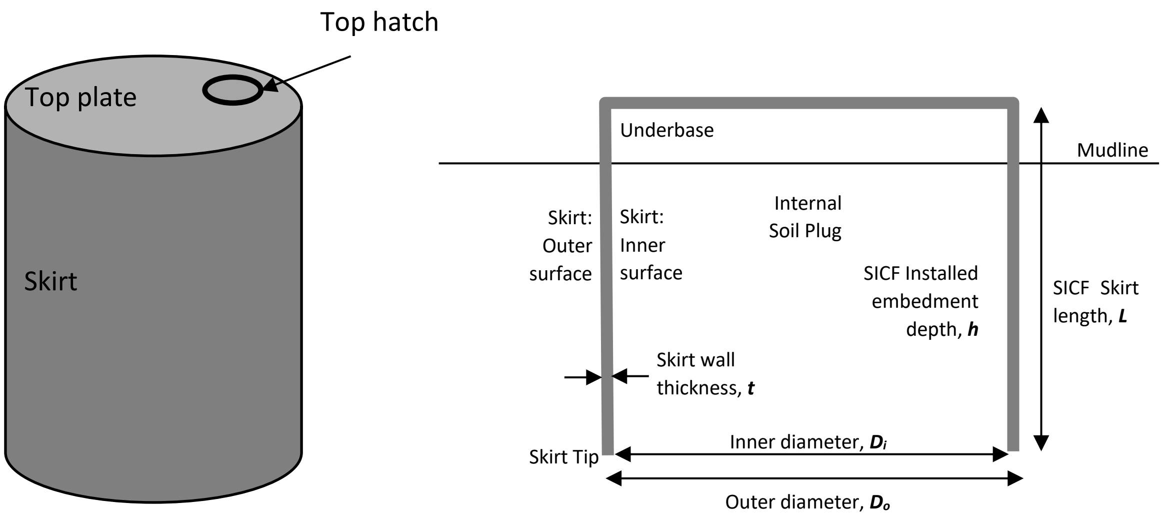


Table 2-1: Suction installed caisson foundation terminology

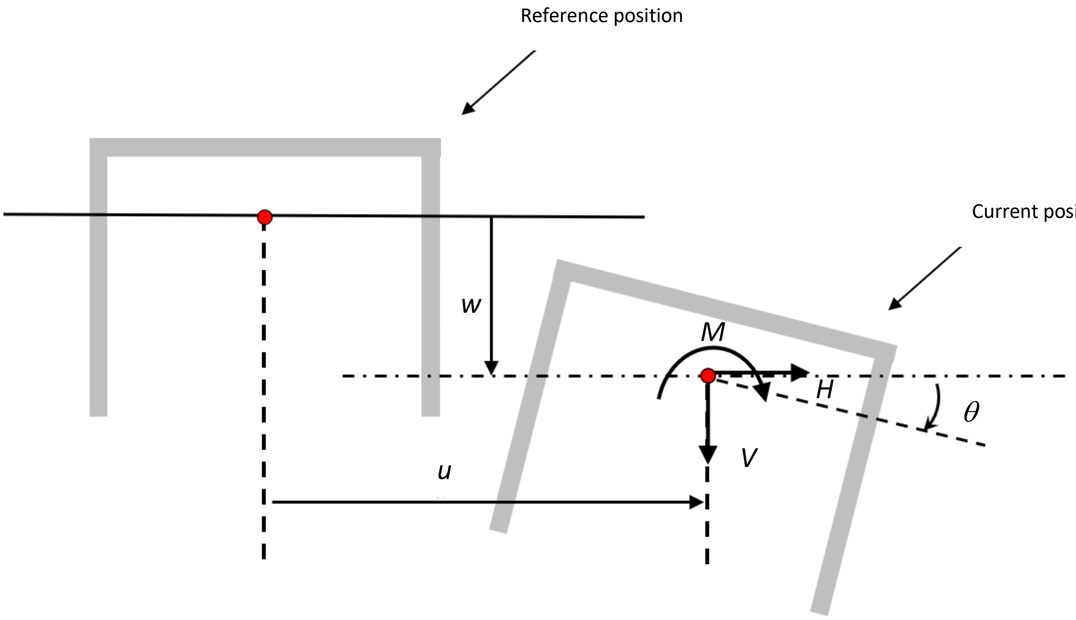
Term	Description	Alternatives
Suction Installed Caisson Foundation (SICF)	The entire structural foundation. This is preferred as it is applicable to all geometries and applications. Caisson is the proper engineering term for a large diameter embedded foundation while bucket is a more colloquial term.	Suction bucket Suction caisson Suction anchor Suction pile SICA (Suction Installed Caisson Anchor) Skirted foundation
Top plate	The structural plate or cover at the top of the SICF which provides the seal to allow the underpressure to be developed and distributes the load from the connection point to the skirt and seabed.	Base plate Lid
Top hatch	Vent(s) provided to allow water to escape during seabed landing often integrated with the pumping system	Vent Pump flange Suction interface

Term	Description	Alternatives
Skirt	The side wall of the SICF, with the majority embedded below seabed as shown in Figure 2-2, generally cylindrical, but other shapes such as square, triangular, or lotus shapes are also in use.	Shaft Shell Side Wall
Stiffener	Secondary sections or plates attached to the top plate and/or skirt to 'stiffen' them against buckling and/or out of plane deformations	Supports
Seabed	The initial undisturbed in-situ seabed level.	Mudline Seafloor
Underpressure	The negative pressure differential developed inside the SICF when pumping water out to cause the pressure differential required to penetrate the SICF. Note that an absolute "negative pressure" is never developed. Pumping leads to a pressure differential which is generally reported as a negative pressure relative to the ambient pressure, e.g. at top plate level.	Suction pressure Negative pressure
Overpressure	The positive pressure relative to ambient pressure developed inside the SICF when pumping water in to create a pressure differential to extract the foundation from the seabed. An overpressure can also be created during the self-weight penetration of the caisson if venting is not adequate to allow escape of water.	Positive pressure

2.2 Conventions

For these guidelines, the sign convention adopted is shown in Figure 2-3.

Figure 2-3: Sign convention for foundation loads and displacements



Where:

V	Vertical load
H	Horizontal load
M	Overturning moment
w	Vertical displacement
u	Horizontal displacement
θ	Rotation

Note: Displacements shown are much exaggerated for clarity. Loads are shown as the loads applied by the SICF to the ground.

2.3 Abbreviations

BH	Borehole
CPT	Cone Penetration Test
DLC	Design Load Case
FEA	Finite element analysis
GBS	Gravity Base Structures
LAT	Lowest Astronomical Tide

MSL Mean Sea Level

OSS Offshore substation

SICF Suction installed caisson foundation

WTG Wind turbine generator

VHM Vertical-Horizontal-Moment (see Figure 8-12)

V_{ult}, H_{ult}, M_{ult} Ultimate resistance for pure uniaxial loading (e.g. H_{ult} is the ultimate resistance when $V = M = 0$)

V_{max}, H_{max}, M_{max} Ultimate resistance for pure translation or rotation (e.g. H_{max} is the ultimate resistance for $w = \theta = 0$)

A Area

a Ratio of excess pore pressure at tip of caisson skirt to beneath caisson base

D Caisson diameter $\approx \frac{(D_i+D_o)}{2}$

G_0, G_{max} Small-strain shear modulus

h Installed embedment depth of caisson

h_w Water depth

K Factor relating horizontal stress to vertical stress

k Soil permeability

k_f Factor for the friction component in the CPT installation method

k_p Factor for the tip component in the CPT installation method

L Length of SICF skirt

m Multiple of diameter over which vertical stress is enhanced

N_c Bearing capacity factor (cohesion)

N_c^* Bearing capacity factor for uplift of a buried circular footing (cohesion)

N_q Bearing capacity factor (overburden)

N_γ Bearing capacity factor (dimension)

p_a Atmospheric pressure (100kPa approx.)

q_c End bearing pressure from the cone penetration test

R Resistance

s_u Undrained strength of the soil

s_{u1} Average undrained strength of the soil over the depth of the SICF skirt

s_{u2} Undrained strength of the soil at the SICF skirt tip

s Suction applied during SICF installation (pressure difference between outside and inside caisson)

S_t Soil sensitivity

t Caisson skirt wall thickness

t_{thix} Thixotropic soil strength increase with time (time dependent)

V, V' Vertical load, effective vertical load

z Vertical ordinate below mudline

Z Stress enhancement factor for installation in sand

α Adhesion factor

δ Interface friction angle

γ_f, γ_m Partial safety factors (load and material)

γ, γ' Unit weight of soil, effective (buoyant) unit weight of soil

γ_w Unit weight of (sea) water (10kN/m^3 approx.)

ϕ' Angle of friction of the soil

σ' Effective stress

Subscripts

a active

i inside caisson

o outside caisson

p passive

r remoulded

3. DESIGN PRINCIPLES

3.1 Safety philosophy

The selection of a safety philosophy and design standards to be used for SICF design should be made in accordance with local regulations. In European waters a Load and Resistance Factor Design (LRFD) approach (otherwise known as partial factor method) is recommended, as outlined in DNVGL-ST-0126 (2016), and other offshore standards such as ISO 19901-4:2016(E) (2016) and Eurocodes (BS EN 1997-1:2004, 2009). All local design standards should be reviewed and followed as required.

The SICF should be designed to meet all requirements with regards to installation, stability and serviceability (and decommissioning if required) during the design life of the structure. All applicable geotechnical aspects and risks, such as layered soils and potential obstructions, should be carefully accounted for during the design process.

3.2 Limit states

A limit state is a condition beyond which a structure or structural component no longer satisfies the design requirements. The following Limit States are considered:

- Ultimate Limit State (ULS)
- Accident Limit State (ALS)
- Fatigue Limit State (FLS)
- Serviceability Limit State (SLS)

These are discussed in more detail below.

When considering the different limit states allowance should be made for the possible effects of scour development, cyclic loading and any other factors which could influence the stability and stiffness as discussed in Section 8.

3.2.1 Ultimate Limit State (ULS)

Ultimate Limit States (ULS) correspond to geotechnical failure of the SICF affecting the stability and/or structural integrity of the substructure. The following load conditions are associated with ULS for SICF design:

- Operating and extreme environmental conditions, for example a 50-year design storm. For WTG foundations reference is made to DNVGL-ST-0126 (2016) and IEC 61400-3;
- Operational boat impact (e.g. by crew transfer vessel).
- Installation and decommissioning pertaining to the integrity of the SICF (for example the structural integrity under design installation underpressure as outlined in Section 6.4.1.3) and the substructure (the installation should be considered separately).

Examples of ultimate limit states pertaining to the geotechnical design of SICF are:

- Exceedance of the bearing capacity of the embedded caisson, i.e. development of a failure mechanism in the soil;

- Compromising the structural integrity or stability of the substructure by excessive SICF displacements/rotations under ULS conditions.

The ultimate resistance should account for the possible effects of scour development, cyclic loading and any other factors which could influence the stability and stiffness.

3.2.2 Accidental Limit State (ALS)

Accidental Limit States (ALS) correspond to the maximum load-carrying capacity for rare accidental loads, or post-accident integrity for damaged structures. The following load conditions are generally associated with ALS for SICF design:

- Accidental boat impact (large maintenance vessel);
- Earthquake loading (see DNVGL-ST-0437).

Examples of accidental limit states pertaining to the geotechnical design of SICFs are as for the ULS condition.

3.2.3 Fatigue Limit State (FLS)

Fatigue Limit States (FLS) correspond to structural failure caused by repeated/cyclic loading. Possible geotechnical effects such as degradation of strength due to repeated/cyclic loading should be assessed and appropriate allowance should be made as part of the ULS and SLS design analyses.

In a SICF context, FLS conditions are used to determine the foundation soil stiffness response for input into the structural FLS assessment, and for input into the integrated systems' natural frequency analysis.

3.2.4 Serviceability Limit State (SLS)

Serviceability Limit States (SLS) correspond to tolerance criteria applicable to normal use, generally the maximum allowable displacements and/or stiffness. The following conditions are associated with SLS conditions for SICF design:

- Differential penetration during penetration (i.e. tilt);
- Settlement, sliding and/or tilt due to operating and extreme loading conditions:
    - Permanent (and differential) loads causing long-term settlements, including effects of shakedown, and primary and secondary consolidation;
    - Variable and environmental (differential) loads and the foundation/structure deformation response.
- Effects of scour on settlements and stiffness.
- Effects of interaction with jack-up spudcans on settlements and stiffness.

Examples of SLS pertaining to the geotechnical design of SICF are:

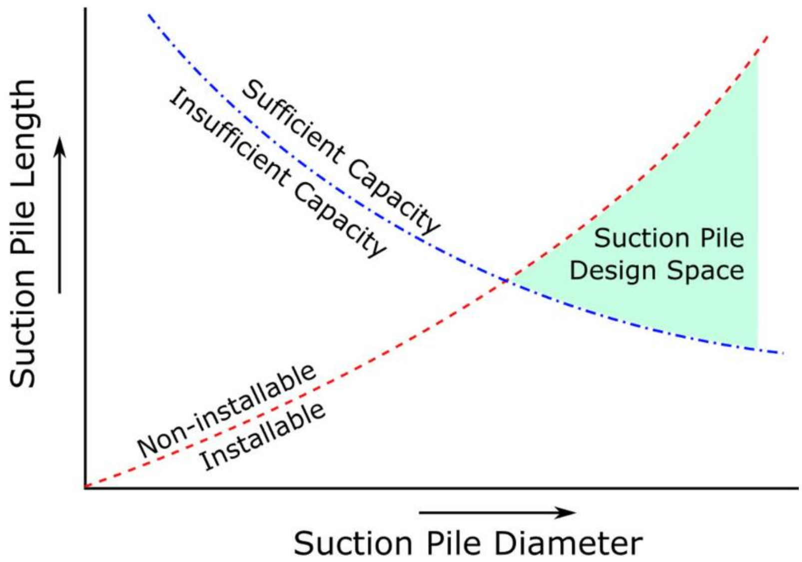
- Foundation exceeds the specified installation tolerances (i.e. tilt) during installation and especially at the end of installation (Section 6.4.2)
- Differential settlements of the foundation causing intolerable (permanent) tilt of the wind turbine;

- Reduction (or increase) in SICF soil response stiffness causing the system's natural frequency to fall outside the permissible range defined by the WTG provider.

3.3 Design Optimisation

Suction caisson foundation design differs significantly from the design of driven piles or gravity bases. Suction caisson foundations are classed as an intermediate foundation. For this type of foundation, the vertical, horizontal and moment (VHM) loads and resistances are coupled (Section 8.4). For the SICF design the installation feasibility (Section 6) is of equal importance to the in-place capacity. The key task for the geotechnical designer of a SICF is therefore to find a suitable balance between installation and in-place requirements, as illustrated in Figure 3-4.

Figure 3-4: Design Optimisation



3.4 Design by LRFD (partial safety factor) method

3.4.1 Approach

The LRFD or partial safety factor approach is a design method by which the target safety level is obtained by applying load and resistance factors to characteristic values of the governing variables and subsequently fulfilling the design criterion:

- Design load effect, $S_d \leq$ Design Resistance, R_d

Two approaches to establish the design load effect S_d or the design resistance R_d are identified in DNVGL-ST-0126 (2016). The most appropriate approach depends on the design situation, as specified below.

Table 3-2: DNVGL-ST-0126 LRFD Methodology

Approach	Design load effect	Design resistance
1 – assessing dynamic response	$S_d = \gamma_f * S_k$	$R_d = R_k / \gamma_m$
2 – assessing nonlinear behaviour	$F_d = \gamma_f * F_k$	$R_d = R(\sigma_k / \gamma_m)$

Where:

F_d, S_d – Factored design load

F_k, S_k – Characteristic design load (unfactored)

γ_f – Load factor

R_d – Factored design resistance

R_k – Characteristic (unfactored) resistance

γ_m – Resistance (or material) factor

R – Relationship between resistance and material strength

σ_k – Characteristic material strength

Approach 1 is suggested when determination of the dynamic response is the primary concern, whereas, approach 2 is suggested when proper representation of the nonlinear material behaviour is the primary concern.

The load factors are to account for potential deviations of load from characteristic values, the limited probability that different loads exceed the characteristic values simultaneously, and any uncertainties in the model and analysis used to derive the load effects.

The resistance factors are to account for potential deviations in material parameters from the characteristic values, and any uncertainties in the resistance calculation methodology.

The factors can also be modified to account for the potential consequences of failure. For example, the factors recommended for Wind Turbine Generators (WTG) and Offshore Substations (OSS) are often different due to the differences in loading regimes and the critical nature of OSS. Load and material factors to be considered for turbine and substation foundations are specified in Sections 3.4.2 and 3.4.3 below.

3.4.2 Load and material factors for WTG foundations

The load factors to be used for design of WTG foundations should be selected in accordance with the specified WTG design code. For reference, the factors recommended by DNVGL-ST-0437 are outlined below.

Table 3-3: Partial Safety Factors for WTG Loads (DNVGL-ST-0437)

ULS				FLS, ALS, SLS	
Permanent*		Variable and environmental		Permanent*	Variable
Favourable	Unfavourable	Normal	Abnormal****		
0.9**	1.1**	1.35***	1.1	1.0	1.0

* Permanent loads include dead loads and pretension loads for the support structure. A load is favourable when a reduced value of this load results in an increased load effect in the structure.

** Factors for permanent loads in ULS may be taken as 1.0 if appropriate measures are taken

*** May be reduced for specific design load cases

**** Abnormal denotes situations with serious failures and/or combinations of unlikely environmental conditions

Typical SICF dimensions lead to classifying them as intermediate foundations, between shallow and deep (see ISO 19901-4 (2016), DNVGL-RP-C212 (2017)). Nevertheless, for the assignment of material factors, they are often treated as shallow foundations. For reference the factors recommended by (DNVGL-ST-0126, 2016) are:

- γ_m = 1.15 for effective stress parameters (i.e. drained strength)
- γ_m = 1.25 for total stress parameters (i.e. undrained shear strength)

Note: The simulation of wind turbine loads involves analysis of a complex non-linear dynamic system. Therefore, the use of different partial load factors for permanent, functional and environmental loads is not always practicable. Instead, a single load factor can be applied (equal to the maximum partial load factor) on the generated load time series.

3.4.3 Load and material factors for offshore substation foundations

Substations are of critical importance and are often designed following dedicated design standards. For information DNVGL-ST-0145 provides two sets of load factors for when different types of load categories are combined as outlined below. Both combinations should be assessed.

Table 3-4: Partial Safety Factors for OSS Loads (Ref DNVGL-ST-0145)

Load factor set	Load Category			
	Permanent	Variable	Environmental	Deformation
Structural (a)	1.3*	1.3*	0.7	1.0
Environmental (b)	Ψ **	Ψ **	1.3	1.0

* When loads are well defined 1.2 can be used

** Ψ = 1.0 for unfavourable loads, 0.9 for favourable loads

DNVGL-ST-0145 recommends using the same material factors as for gravity base foundations. For reference the values specified in (DNVGL-OS-C101, 2016) are:

- γ_m = 1.2 for effective stress parameters (i.e. drained strength),
- γ_m = 1.3 for total stress parameters (i.e. undrained shear strength).

4. SITE GEOTECHNICAL CONDITIONS

Site conditions consist of all site-specific conditions which could influence the design of a SICF by influencing applied loading, installation, in-situ capacity, and/or decommissioning.

This section provides initial guidance regarding the information required for geotechnical design of SICF.

4.1 Site characterisation requirements

Geotechnical studies and investigations should provide all necessary data to allow detailed design and installation engineering to reduce uncertainties to an acceptable level (as determined by the project stakeholders). The suggested site characterisation stages are outlined in Table 4-5.

Table 4-5: Site characterisation stages

Stage	Description
Geological Desk Study	A geological study, based on the geological history and provenance of site-specific soils, should form the basis for scoping of survey operations and help select the most appropriate methods and extent of the geophysical and geotechnical investigations. A ground risk register should be included to document the uncertainties/risks and how they are addressed through the project lifecycle.
Geophysical survey (Bathymetry and sub-bottom)	A geophysical survey can be combined with the results from a geotechnical soil investigation to establish information about soil stratification, seabed topography and seabed features for an extended area, such as the area covered by a wind farm. The geophysical data can also give a valuable insight into heterogeneity across the SICF footprint.
Geotechnical survey	A geotechnical survey may consist of in-situ testing (such as CPT), borehole drilling and sampling, borehole logging, sample recovery and description, core logging, and laboratory testing on the recovered samples.
Ground model	A database of information that includes structural geology, geomorphology, sedimentology, geohazards and geotechnical properties throughout the site.

For further guidance and industry practice regarding requirements to scope, execution and reporting of offshore soil investigations, and to equipment, reference is made to SUT (2014), DNVGL-RP-C212 (2017), Norsok G-001 (2004), Norsok N-004 (2013) and ISO 19901-4:2016(E) (2016). National and international standards such as Eurocode (BS EN 1997-1:2004, 2009) and BSH 7004 (2014) may also be relevant depending upon the region.

Trial installations as discussed in Section 6.5 may be considered as part of the geotechnical investigations, such that results are available at the start of the design process.

4.2 Geotechnical hazards

SICF structures are sensitive to specific geological and geotechnical conditions and hazards, particularly for installation. Appropriate consideration of these aspects will ensure reliable and efficient design and installation. As a minimum the hazards detailed in Table 4-6 should be considered, however, it should be noted this list is not exhaustive.

The hazards identified should be thoroughly investigated with a combination of geophysical, geotechnical and other surveys. The survey data should be integrated to provide a holistic view of the site conditions. The identified hazards should be assessed to estimate the potential impact on the foundation/structure and accounted for in the design or with appropriate mitigation methods. Is should also be noted that hazards may change over time, therefore, future hazards should also be considered.

Table 4-6: Examples of geotechnical and geological hazards

Hazard	Description and potential impact
Seabed features	Irregular topography such as sand waves, megaripples, ridges, jack-up footprints etc, which could provide uneven seabed
Seabed obstructions	Any potential obstructions which could hinder installation operations i.e. wrecks, existing infrastructure, boulders, debris
UXO	UneXploded Ordnance (UXO)
Lateral variability	Lateral soil variability across the SICF diameter could be the source of installation difficulties or result in tilt. Examples would be siting the SICF on the edge of a relic channel or localised soil lenses.
Seabed mobility / Scour	Site characterisation should include an assessment of the potential for global sediment mobility and/or localised scour. Loss of soil around the SICF can compromise load capacity
Seabed slopes	Steep seabed slopes could result in installation difficulties
Slope stability	Slope failure could result in movement or failure of the SICF
Gravel / cobbles / boulders	Gravel and/or cobbles and/or boulders could result in installation difficulties through increased skirt tip resistance or lack of self-penetration to create a seal allowing suction pressures to be generated
High permeability	Soil layers with high permeability can hinder installation by allowing high flow rates and inability to achieve required underpressure. Examples would be gravel or shell beds or sand in layered soils.

Fissures and voids	Fissures and voids can dramatically increase mass permeability and reduce the achievable underpressure. Such phenomena can also be detrimental to soil mass strength
Shallow gas	Shallow gas could result in disturbance of shallow soils reducing foundation capacity (and stiffness) or potentially fill the caisson with gas resulting in an internal overpressure
Liquefaction	Soils may 'liquefy' under cyclic loading (seismic or wave loading) reducing foundation capacity
Seismic loading	Seismic loads are transmitted to the structure through the foundations and vice versa. This is generally not an issue in Northern Europe but may be a consideration in other geographic areas.
Shallow rock / cemented soils	Shallow rock and cemented layers could result in premature refusal during installation
Unusual soils	Special or unusual soils such as peat could pose an installation risk due to the potentially fibrous nature of plant remains. Standard design factors could also vary for organic clays, micaceous sands, etc.

4.3 Geophysical survey

The nature and extent of geophysical surveys should be determined by an experienced geophysicist in conjunction with a geotechnical engineer experienced in the design of SICF and aware of the attendant data requirements.

The basic scope should address the following:

- Multibeam echo sounder and side scan sonar should provide complete data coverage to determine the depth and nature of the seabed.
- Sub-bottom data should be acquired at sufficient resolution to characterise the stratigraphy and to investigate all the sub-bottom hazards discussed in Table 4-6.
- The orientation and spacing of survey lines should be chosen to ensure that the soil conditions are properly characterised without excessive interpolation between sub-bottom profiles.

A good quality geophysical survey provides essential input for a site ground model (Section 4.5). It should also be noted that the water depth (particularly in shallow water depths) can influence cavitation for stiff clays and dense sands.

The reference datum should be clearly detailed, potentially in the Design Basis / Interface Handbook discussed in Section 5.2; reference datum is usually taken as Lowest Astronomical Tide (LAT), however, occasionally alternatives such as Mean Sea Level (MSL) are used.

4.4 Geotechnical survey

The extent of the survey, and the methods to be deployed, should consider the type, size and importance of the wind turbine structure, the site geology, the anticipated variability and complexity of soil and seabed conditions, and the nature of the soil deposits. The scope of the soil investigations and the choice of soil investigation methods should be determined by a geotechnical engineer experienced in suction caisson design, familiar with the anticipated ground conditions across the site under consideration and aware of the associated data requirements for design and to allow proper evaluation of potential risks.

A geotechnical survey should normally comprise a combination of boreholes and CPTs.

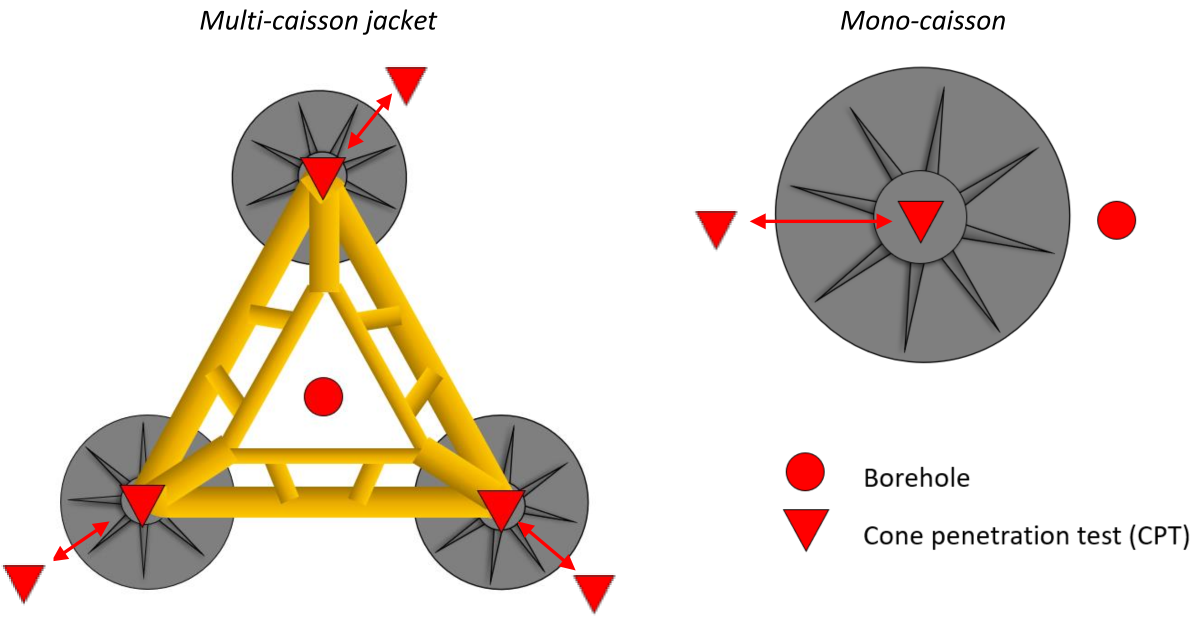
Boreholes provide an accurate indication of the soil type and physical samples for laboratory testing to provide design parameters. Sufficient boreholes should be performed, and sufficient samples collected, to ensure the requirements of the laboratory testing for soil parameters can be achieved. This should have regard to any layering structures that may exist.

CPTs provide a continuous profile with information regarding soil type and strength parameters. CPT interpretation should be validated using borehole data from the same site. Seabed CPTs are generally preferred to downhole CPTs as they provide a continuous measurement with no potential data gaps between pushes.

The scope of the survey is site and project stage dependent. However, by the detailed design stage the following guidelines for the extent of the geotechnical data coverage are proposed for complex geotechnical conditions (see Figure 4-5):

- Multi-SICF jackets - CPT at the centre of each SICF, and borehole at the centre of the structure or at the centre of or near the SICF positions to evaluate potential variability;
- Mono-SICF structure - CPT at the centre or outside the footprint of the SICF, and borehole outside the anticipated footprint area and installation tolerance to confirm lateral variability and validate CPT interpretation.

Figure 4-5: Basic geotechnical survey recommendations



The position of the tests should be carefully planned based upon the anticipated soil types and variability.

The number of tests at each WTG position could be reduced if the ground conditions are anticipated to be very homogenous and confirmed by a ground model (Section 4.5). This decision should be made by an experienced engineer based upon the anticipated variability in, and results from, the initial offshore surveys.

Boreholes could disturb the seabed, leaving a void or disturbed area which could cause piping/ratholing during installation or potentially result in preferential flow paths that could influence installability and/or reduce long-term capacity of the SICF. Therefore, preferably boreholes should be located at the centre of the overall structure footprint or a suitable distance outside the footprint on a mono-caisson and should not be close to the edge of the SICF footprint. Compared to boreholes, CPTs have a much smaller diameter and cause less disturbance. Where there is the potential to create preferential flow paths it is recommended that the tests are located outside the potential SICF footprint areas. Where there is potential soil variability it is recommended that the test is as representative as possible of the conditions at the SICF location.

The depth of the geotechnical testing should be sufficient to ensure that engineering analyses and risk assessments are not constrained by borehole depth. Therefore, the depth should extend to at least to the depth of any critical shear surfaces, and the zone of influence of the foundation from a settlement perspective. The minimum borehole depth will depend on the subsoil conditions but is likely to exceed one caisson diameter below the skirt tip penetration, or to the depth to bedrock if this is less.

The boreholes should provide sufficient samples in each soil unit, or at each location in variable soil conditions, to confirm the CPT interpretation and allow appropriate laboratory testing (see Appendix A) to be undertaken.

4.5 Ground model

Development of a ground model which integrates geophysical and geotechnical data is recommended, particularly for sites with complex layering and substantial soil variability. A ground model provides a coherent basis for characterising the engineering properties of a site and developing a robust SICF design. It is also important for identifying data gaps, hazards or other uncertainties which need to be addressed in subsequent surveys. Ground models are preferably initiated at an early stage of a project and should evolve as additional survey data is gathered. A ground model is particularly important for structures founded on SICFs because of their sensitivity to localised ground conditions during installation.

4.6 Geotechnical parameters for design and installation

The characteristic strength and deformation properties required at each SICF location should be determined. For installation engineering, the permeability of cohesionless soils is also very important, particularly when layers of permeable and impermeable soil are present.

The main geotechnical parameters relevant for foundation design are indicated in Appendix A along with in-situ and laboratory tests which may be used for their determination. The list is not exhaustive and is not mandatory - the actual testing to be undertaken should be specified by an experienced geotechnical engineer on the basis of the anticipated soil conditions and an initial inspection of the offshore CPT logs and samples.

The results of both in-situ and laboratory tests should be evaluated, correlated and compared to existing published data where possible, and this process should be documented. The process needs to account for possible differences between properties measured in the tests and those soil properties that govern the behaviour of the in-situ soil for the limit state in question. Such differences could be due to:

- Soil disturbance due to sampling;
- Difficulty of reconstituting samples to a representative in-situ state with the corresponding stress history;
- Presence of fissures;
- Different loading rate between test and limit state in question;
- Simplified representation in laboratory tests of certain complex load histories;
- Soil anisotropy effects giving results which are dependent on the type of test;
- Possible effects of installation activities on the soil properties (Section 4.7 below).

When the characteristic value of a soil property is estimated from limited data, the estimate should be a cautious estimate of the value that affects the occurrence of the limit state.

Relevant statistical methods may be used to assist the selection of characteristic values of soil properties, for example as described in DNVGL-RP-C207 (2017).

4.7 Special considerations for selection of soil parameters

This section provides a short introduction to specific considerations for soil parameter selection for in-service analysis and design of SICFs (Section 8). General methods for the selection of soil parameters is outside the scope of this document, some guidance is contained in DNVGL-RP-C207 (2017).

Soil properties and parameters are affected by suction installation and the following non-exhaustive list should be considered:

- Disturbance of soil due to installation - a zone of disturbed or remoulded soil around the caisson walls immediately after installation will be present and may affect SICF behaviour. This zone may be looser or weaker than the in-situ soil and may respond differently to cyclic loading in the initial stages of operation. However, this zone may also experience relatively rapid densification and/or consolidation. In clays, the interface factor, α (detailed in Sections 6.2.1.1 and Appendix C) attempts to account for the effect of installation and should be selected carefully accounting for consolidation effects.
- Disturbance of the soil plug during installation – the application of suction pressures and upward hydraulic gradients may loosen sand layers inside the caisson. In addition to contributing to soil heave, looser sand may be more susceptible to increased pore water pressures during a storm event and may exhibit higher compressibility and greater volume reduction than the in-situ soil. The degree of loosening should be assessed on a case by case basis as well as the possible associated settlement as load transfer to the soil plug occurs.
- Load transfer between skirt walls and top cap - after installation, it is likely that the structure weight is shared between the skirt walls and the top cap, however in some scenario's SICF may be designed to rely only on skirt capacity. Depending on the methods used for ensuring good top plate contact (e.g. using underbase filling as discussed in Section 7.7.1), load transfer between the skirts and top cap is likely to occur rapidly with some settlement. Once full top cap contact and full load transfer has occurred the in-place analysis discussed in Section 8 can be applied. However, the potential for shakedown should be considered as discussed in Section 8.8.2.

Shortly after installation, consolidation and strength regain of any remoulded clay near the caisson walls will occur. Cyclic loading due to operating and extreme metocean conditions followed by consolidation will also improve the stiffness and strength of most soil conditions. Ageing effects in soil are not well understood but also result in increases in stiffness and possibly strength (Schmertmann, 1991). Thus, soil parameters should not be considered as fixed in time but should be selected conservatively for the analysis to be performed. In most cases, the local areas of disturbed soil will not dominate the overall SICF response, and in-situ stiffness and strength parameters will be appropriate for the overall response. However, it is recommended this is evaluated for each site and loading condition by an experienced geotechnical engineer.

The effects of long-term stiffening of the soil due to cyclic densification and ageing may be used when considering lifetime fatigue of the structure.

4.8 Clustering

Clustering is the process of categorising locations according to water depth, soil conditions, hazards, etc such that structures with similar design conditions can be assessed together to optimise the design and fabrication processes.

Clustering is often applied for monopile and driven jacket pile designs to achieve design and fabrication efficiencies. Although this approach may be feasible for in-place design of SICF's it is not recommended for installation engineering due to the sensitivity of the installation procedure to ground conditions unless it can be demonstrated that a clustering approach does not increase installation risk.

5. FOUNDATION LOADING

5.1 General

The design of a suction caisson foundation is site-specific. Verification of the structural integrity of its load-carrying components is based on limit states (see Section 3.2) valid for the geotechnical and environmental conditions (wind, sea state, etc.) at the site under consideration. The applicable site conditions and loads are determined according to a suitable design code such as DNVGL-ST-0437.

A global load analysis is required to determine the loads on the SICF due to the dynamic interaction and non-linear effects between the wind turbine, tower, substructure and SICF. Due to the non-linearities, a dynamic analysis in the time domain is normally used and therefore load-time histories are computed for the SICFs. These time-histories are processed as discussed later in this section to extract the loads required for ULS and SLS geotechnical design of the SICFs and to establish the overal structural integrity.

Analysis of the structure and foundation, and calculation of loads on any element, may be performed using either a coupled or uncoupled approach (see Section 5.5):

- In the coupled, or integrated approach, the WTG, support structure and SICFs are integrated in a single structural model with a proper distribution of masses and stiffnesses;
- In the uncoupled, or iterative approach, the different elements are analysed in separate models, and stiffnesses and loads are shared at the interfaces. The system non-linearities lead to requiring iteration to achieve consistent loads and stiffnesses.

Both approaches normally involve dynamic analyses in the time domain using a (global) load calculation model of the offshore wind turbine plus support structure subject to wind/wave loading and turbine states. Advantages and disadvantages of the two approaches are discussed in Section 5.5.

Load calculation models may be built-up from sub-models of rotor-nacelle-assembly, tower, substructure and foundation/soil stiffness. The SICFs may be modelled as soil springs or stiffness matrices at a pre-defined interface point (the Load Reference or Load Application Point(s), see Section 5.2.2). Output of the calculation model includes the SICF reactions at the Load Reference Point providing the input for geotechnical design.

5.2 Interfaces between model elements

5.2.1 Sign convention

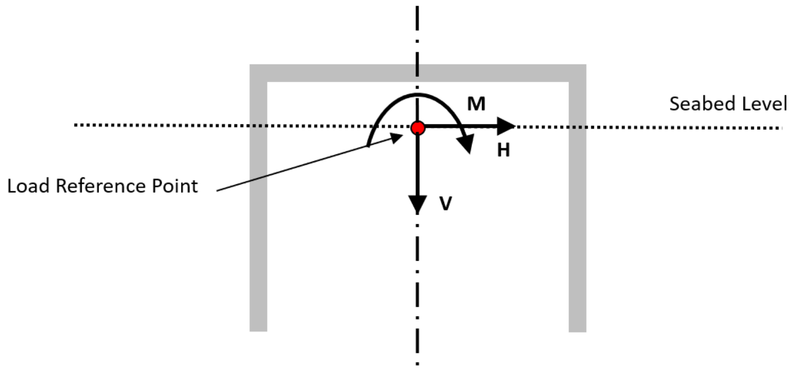
A sign convention for the global coordinate system, to be used for all load and displacement components, should be agreed between all parties in the design process and specifically between the geotechnical team (for the SICF), and the substructure team. The local SICF convention should be agreed and clearly detailed in comparison to global structural sign convention in the Design Basis or Interface Handbook for the project.

5.2.2 Load reference point (interface point)

The Load Reference or Application Point should be agreed and defined in the Design Basis or Interface Handbook. In an uncoupled approach, the substructure team provides all foundation loads acting at the Load Reference Point. The geotechnical team provides the foundation stiffness elements at the Load Reference Point.

There are numerous options for the definition of the Load Reference Point. These guidelines suggest that it is taken at the centre of the suction caisson at seabed level, as indicated in Figure 5-6. Another common Load Reference Point is the connection point between the structure and the top plate.

Figure 5-6: Load Reference Point



Note: Loads shown as applied by the substructure to the SICF

The characteristics of the connection between the Load Reference Point and the substructure should be agreed between the parties and defined in the Design Basis or Interface Handbook. For example, it should be agreed how to represent the stiffness of the suction caisson top plate and the underbase filling or voids present under the suction caisson top plate. The stiffness response is discussed in Section 8.7. Appropriate transformations must be made if there is any change in location of the Load Reference Point.

5.3 Foundation load components

The substructure design team should provide foundation loads to the SICF geotechnical design team as a 6-component load vector $\{F_x, F_y, F_z, M_x, M_y, M_z\}$ applicable at the Load Reference Point. For SICF geotechnical design it is often convenient to simplify this vector to a 4-component load vector $\{V, H, M, T\}$ applying any sign changes required by the agreed convention.

5.4 Load cases

The IEC 61400-3 (2009) and DNVGL-ST-0437 (2016) standards provide principles, technical requirements and guidance for loads and site conditions of wind turbines. These codes specify a set of Design Load Cases which cover the most significant conditions anticipated for the offshore wind turbines and their support structures.

The following load cases are specifically relevant to geotechnical aspects of SICF design for offshore wind applications:

- Suction caisson installation and removal (temporary design conditions for seabed landing, underpressure, overpressure). The potential structural loads that could be imposed on the substructure due to non-uniform soil reactions during installation should also be considered;
- Operational conditions for in-place analysis (ULS, SLS and FLS). Assessment of these loads require SICF stiffness matrices at the appropriate load level (and including potential cyclic loading / degradation effects) for structural analysis, as well as for ULS stability and SLS deformations.
- Special cases such as caisson-spudcan interaction (in case of WTG installation using a jack-up vessel).

5.5 Load generation and modelling

5.5.1 Design process

The design process of the full offshore wind turbine (or substation) structure generally involves multiple design organisations (turbine designer, substructure designer and foundation geotechnical designer), each with their own engineering expertise and perspectives. Industry standard offshore wind turbine load design and optimisation software, and offshore structural code checking software are employed by the respective organisations in their specific engineering processes.

The specialist design tools are referred to as 'WTG software' and 'structural design software':

- WTG software - A software package for offshore wind turbine load design and optimisation can be set up for fully coupled analysis of the wind turbine and support structure, capturing the interactions of simultaneous aerodynamic and hydrodynamic loading and control system actions. A SICF would typically be modelled as a linear stiffness matrix or equivalent spring set depending on the capability of the software. It should be noted that a linear stiffness matrix cannot capture the load-dependence of the stiffness response.
- Structural design software for support structure - These software packages performs ULS and FLS structural code checking of spatial frames subjected to loads including gravity, buoyancy, waves and wind turbine interface loads at the tower base. The FE-model typically consists of beam elements, but complex parts may also be modelled by super-elements from detailed 3D shell or solid FE-models. The SICF will then be part of a super-element. This inherently requires linearization of the non-linear suction caisson - soil interaction. Note that the use of super-element techniques involves simplification of the wave loading regime as well.

These tools can be deployed in the load generation process for the design of the suction caissons. The load generation process involves dynamic time integration analysis in one of the following two ways:

- Integrated Load Analysis (ILA), or coupled analysis;
- Sequential Load Analysis (SLA), or uncoupled (iterative) analysis.

The two approaches are discussed below and illustrated in Figure 5-7 .

5.5.2 Integrated Load Analysis (ILA) / Coupled approach

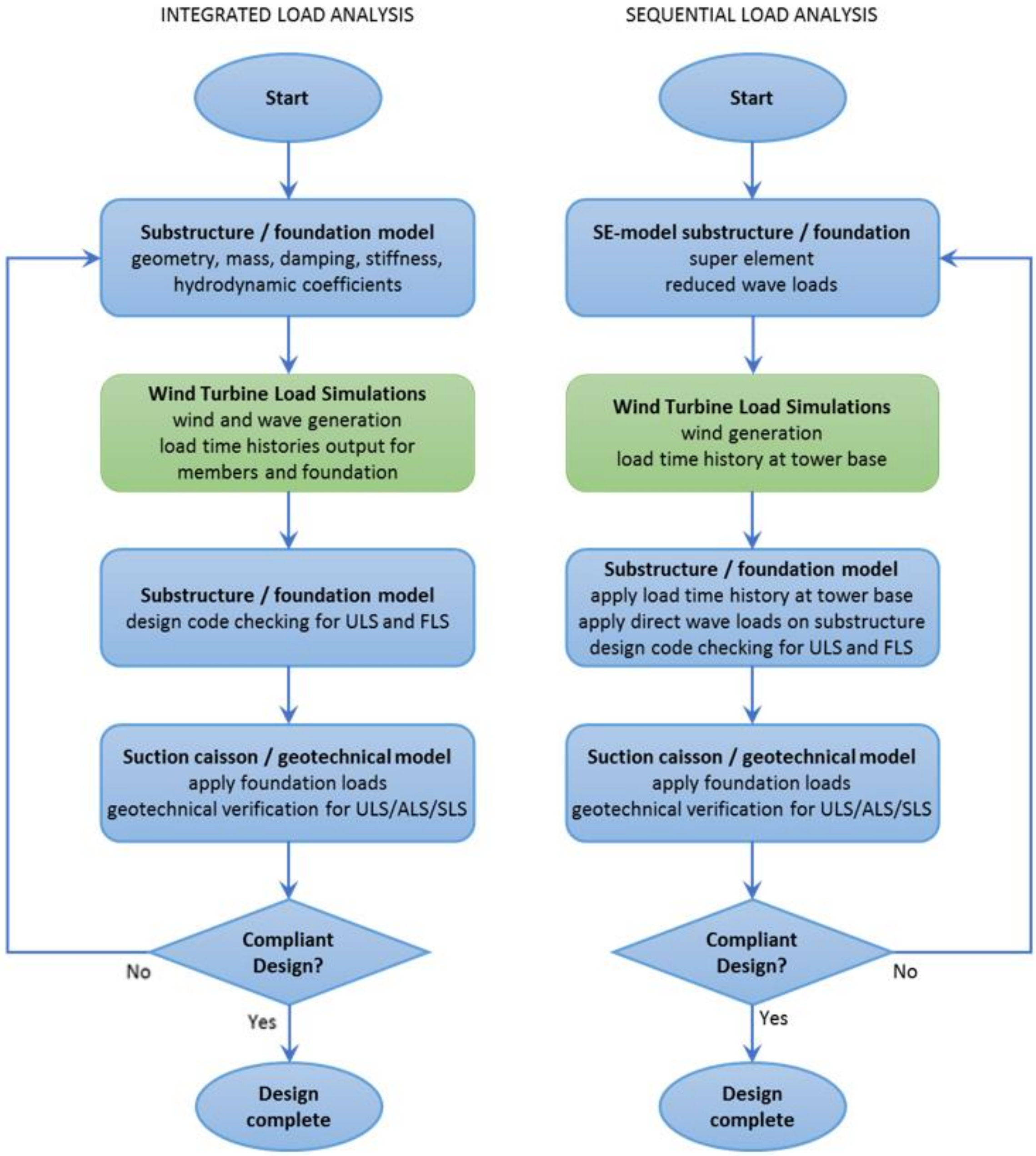
By modelling the wind turbine and support structure in a single aero-hydro-servo-elastic tool, the structural dynamics of the entire structure are fully coupled, and wind and wave loads are applied simultaneously. The aero-hydro-servo-elastic tool models the structure including aerodynamic and hydrodynamic loading meaning damping can be properly accounted for, achieving more structurally efficient designs with fewer design iterations. The structural design software is engaged in FE-modelling and ULS/FLS code checking of the substructure/SICF designs. This code checking involves post-processing of load time histories of members and foundation reactions, all generated as output by the software.

It should be noted that ILA could also be used to refer to an Independent Load Analysis during the certification process. It is therefore recommended that the abbreviations are agreed at the outset and clearly detailed in the Design Basis / Interface Handbook.

5.5.3 Sequential Load Analysis (SLA) / Uncoupled approach

This approach is typically employed by support structure or SICF designers. The wind turbine and offshore support structure are modelled and analysed in separate design tools, requiring repeated data exchange, and may not fully account for the dynamic interaction between wind turbine and substructure. In the load generation process, the WTG software computes and yields the interface loads at the tower base, and the structural design software applies them together with direct wave loads on the substructure model for subsequent ULS/FLS code checking of the substructure and SICF design.

Figure 5-7: Design Process for Integrated and Sequential Load Analysis



The initial stiffnesses would be estimated as discussed in Section 8.7.

5.5.4 Comparison of Integrated and Sequential Load Analysis

A comparison of the two load modelling approaches is given in Table 5-7. The table aims to provide an initial comparison between the two modelling approaches detailed above and may not be representative of more advanced tools.

Table 5-7: Comparison of Integrated (ILA) and Sequential Load Analysis (SLA)

Feature	ILA	SLA
Fully coupled load analysis?	Yes	No
Hydro-elastic coupling between substructure and waves?	Yes	No
Non-linear soil springs?	Yes*	No
Allow complex (i.e. non-beam-element) substructure designs?	No	Yes
Code checking for ULS and FLS in structural design software?	Yes	Yes
Substructure/foundation load time histories?	Yes	Yes
Foundation reaction loads available?	Yes	Yes
Allow for geometric stiffness corrections of substructure?	Yes	No
Substructure/foundation vs. WTG design kept confidential?	No	Yes

*System dependant - some systems require linear springs as discussed in Section 5.5.2 above.

The most appropriate design tool should be selected depending upon the design stage and experience of the design team.

5.6 Foundation damping

Since WTG support structures show a highly dynamic load response, the loads on the WTG and the support structure, including the foundation reaction loads, are dependent on the damping of the system components. A common approach is to define an overall damping (for example Rayleigh damping) for the structure. This damping is then considered to represent multiple damping sources, such as material and foundation damping.

There is limited literature regarding the damping of offshore foundations in general, and SICF in particular. The foundation damping of SICFs subjected to moment loading has been investigated by Houlsby et al. (2005) as discussed in Section 8.7.4 .

If the foundation damping is included explicitly in the modelling of the foundation reaction loads, the assumed foundation damping (matrix) should be backed up by experiments or in-situ measurements on installed structures with SICFs whenever possible. Due consideration should be given to the contribution of hysteresis (material) and radiation (geometrical) damping to the overall foundation damping for various load frequencies.

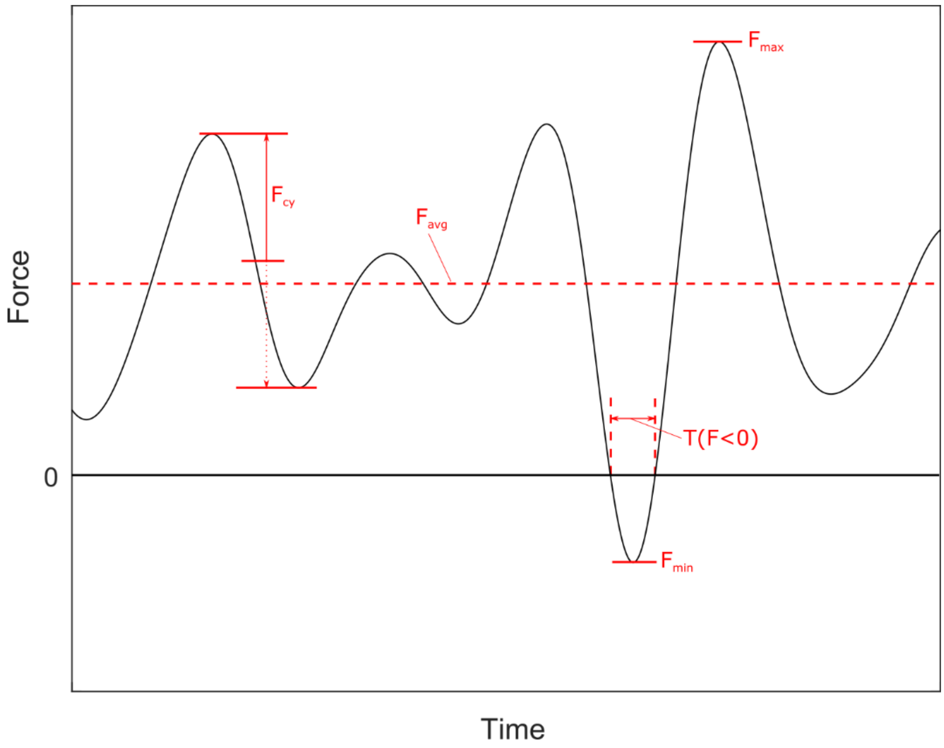
5.7 Extraction of loads acting on SICFs

5.7.1 Relevant load characteristics

Especially for the design of WTG support structures, foundation reaction loads are typically generated as a time series. A separate time series should be available for each load component. The following characteristics of the time series, as visualized in Figure 5-8, are needed as input for the design of suction caissons:

- Loading period - the time, T, that a certain load level is sustained ($T(F \geq X)$ or $T(F \leq X)$) is needed to distinguish between short-term (undrained) and quasi-static (drained behaviour) loading. This is especially relevant for tensile loads.
- Extreme load peaks (F_{min}, F_{max}). Load peaks are relevant for verification of load effects from short-term loading. The use of group averaged extreme load peaks may also be appropriate.
- Average load over a longer period (F_{avg}). Average load is relevant for verification of load effects from quasi-static loading.
- Cyclic load amplitude (F_{cy}). Cyclic load amplitude is relevant for verification of effects from cyclic loading.

Figure 5-8: Load characteristics relevant for suction caisson foundation design



5.7.2 Reporting

For the extreme load peaks, the following load cases should be reported and provided to the geotechnical design team as a minimum:

- Maximum Compression (V_{max}) with all associated load components
- Maximum Tension (V_{min}) with all associated load components
- Maximum Shear (H_{max}) with all associated load components
- Maximum Moment (M_{max}) with all associated load components
- Maximum total resultant force with all associated load components

All load combinations should be mapped relative to the VHM failure surface to ensure there are no load combinations which fall outside the VHM envelope, which would mean that design criteria are not satisfied.

Each load case should be reported as a 4-component or 6-component load vector (see Section 5.3). The vector should consist of the load peak for the relevant load component, together with all other load components that act simultaneously. The combination of maximum load peaks for multiple load components in a single load case (so-called max-max approach) is not recommended.

For multiple caisson foundations, each load case of an extreme load peak should also contain the load vectors that act simultaneously on the other suction caissons.

For the load characteristics other than the load peaks, more complex post-processing of the load time series is required. The parties involved should agree which party performs this processing, and whether processed load characteristics or relevant time series are provided to the geotechnical design team.

In all cases, relevant load case parameters, such as Design Load Case (DLC), wind and wave conditions and orientation, and load factors used, should be reported and provided together with the foundation loads in accordance with Appendix C of DNVGL-ST-0437.

6. INSTALLATION DESIGN

6.1 Installation design and risk management

The design of the installation phase for a SICF is as important as the design for its in-service performance. Some aspects of installation design can be supported by design calculations, whilst others can only be treated qualitatively.

The entire installation should be treated as a managed process in which the expected performance is predicted, and measured behaviour is compared with expected performance. Risks should be anticipated, and the installation procedures designed to mitigate them. Contingency plans should be in place to deal with residual risks. The SICF installation procedure is outlined as follows:

1) Initial crane lift and overboarding
2) Lowering through splash zone (see Section 7.4)
3) Lowering through the water column
4) Touch-down (see Section 7.4)
5) Self-Weight Penetration (SWP) as discussed in Section 6.3
6) Suction penetration to target depth as discussed in Section 6.4

In this document, the basic design calculation methods are described in this section and installation planning matters are discussed in Section 7. Both sections form part of the overall installation design and risk management process which is recommended in these guidelines.

6.2 Penetration resistance

The main purposes of the installation design calculations are to:

a) Predict the vertical load-penetration response without the application of suction when the self-weight of the caisson and structure are applied i.e. the "self-weight penetration".
b) Predict the suction-penetration response, i.e. the suction pressures required to reach target penetration, and compare the predicted suctions against the limits established in Section 6.4.1.
c) Anticipate specific risks to successful installation and put in place appropriate measures to mitigate those risks.

In making the predictions for (a) and (b) above, normal practice would include presenting both "best estimate" and "high estimate" calculations. "High estimate" in this context is likely to represent a high resistance, low penetration situation.

The 'high estimate' calculation should ensure the SICFs are installable to the target penetration depth (determined from the in-service design) in the most onerous soil conditions anticipated, whilst the 'best estimate' calculations provides the base case for the engineers controlling installation pressure and penetration rate.

The calculations should account for the presence of any stiffeners as appropriate.

The basic equation for the installation calculations for (a) and (b) is as follows:

$$V' + s \cdot A_{caisson} = R_{inside} + R_{outside} + R_{tip}$$ Equation 6-1

Where:

$A_{caisson}$	Internal plan area of caisson (internal top plate area)
R	Resistance
s	Suction applied during SICF installation (pressure outside caisson minus pressure inside caisson)
V'	Effective vertical load (accounting for buoyancy)

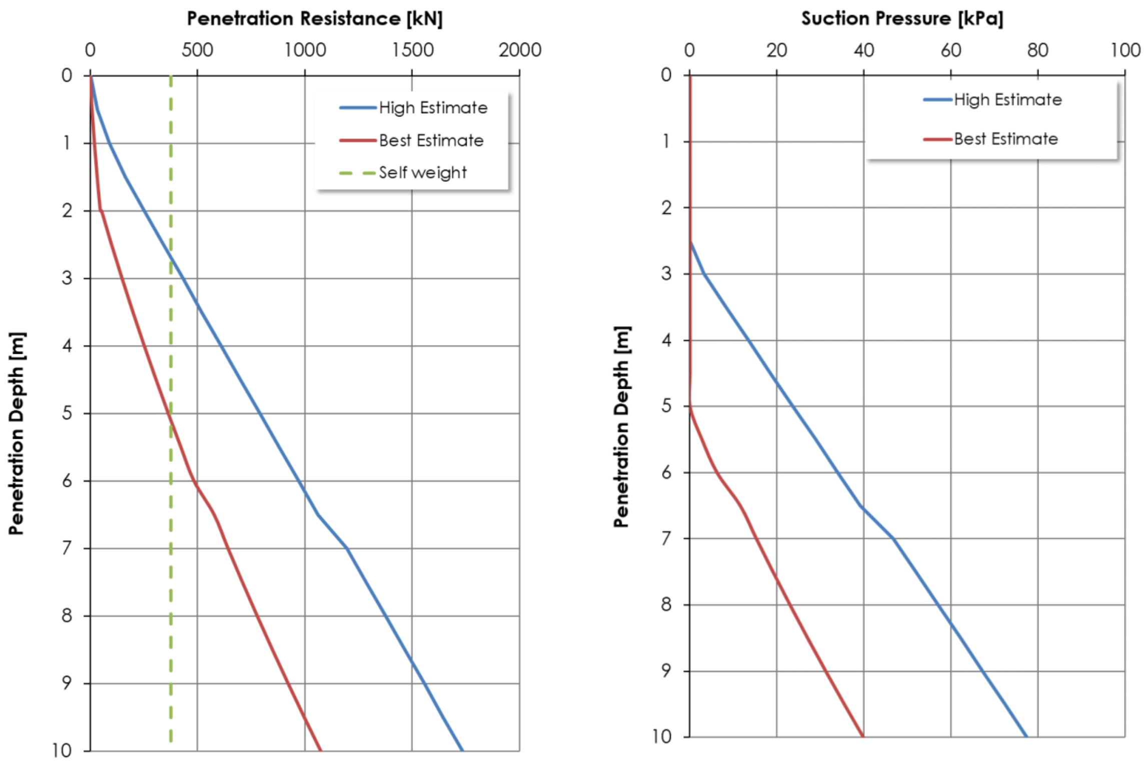
The standard method is to present to results of the installation calculations in graphical format, including the following:

- plots of penetration resistance against depth
- plot of required suction against penetration depth

Examples are shown in Figure 6-9 below.

Figure 6-9: Example penetration resistance plots

Penetration Resistance *Required underpressure*



Note: any limits that may be applicable should also be plotted

There are two main types of calculation method for caisson installation. The first (Section 6.2.1) are based on using the geotechnical properties of the soil, determined either by *in-situ* tests or laboratory tests, together with an approximate mechanism-based calculation of the relevant loads on the caisson.

The second group of methods (Section 6.2.2) are based on using CPT data. Empirical factors are applied directly to CPT measurements to convert these directly to estimates of loads on the caisson.

Experience indicates that although the two approaches are very different, they can often lead to rather similar predictions.

The methods described below represent the basic form of the calculations, but they may readily be adapted to accommodate (a) soils with varying properties with depth, (b) structural features such as internal stiffeners.

In cases involving complex soil profiles or other special considerations, although the basic approaches are still applicable, more extensive soil data is likely to be required, more sophisticated modelling methods such as Finite Element Analysis may be employed, and trial installations may be considered.

6.2.1 Mechanism-based methods

The mechanism-based methods make use of standard geotechnical parameters determined by in situ or laboratory testing:

In clay: the undrained strength s_u as a function of depth to below the skirt tip;

the soil sensitivity S_t or remoulded undrained shear strength s_{ur}

the adhesion factor α

the unit weight of the soil γ (which has only a very minor effect on the main calculations).

In sand: the angle of friction ϕ';

the effective unit weight of the soil γ';

the soil permeability k (for flow rate calculations).

The selected undrained shear strength should account for the type of shearing (compression, direct simple shear, extension, etc). The adhesion factor can be calculated as a function of the soil sensitivity or remoulded shear strength.

In addition to the above, several dimensionless factors need to be specified, for instance friction factors on the side of the caisson and bearing capacity factors at the tip.

The design methods are based on broadly the same principles as those used for estimating the capacity of driven piles: the vertical load on the caisson is resisted by friction on the sides (inside and outside) together with end bearing on the annular rim. There are several mechanism based methods available including Houlsby and Byrne, (2005b), Andersen et al. (2008) for cohesionless soils and Houlsby and Byrne, (2005a), DNVGL-RP-E303 (2017), for cohesive soils.

Any of the available mechanism-based methods should be appropriate for the design of SICF when applied by an experienced geotechnical engineer. However, it is recommended the design methodology is discussed and agreed by the key stakeholders including the developer, structural design engineer and project certifier.

6.2.1.1 Mechanism based method for clays

In clays the calculation for a basic case gives Houlsby and Byrne, (2005a) Eq. 3 and DNVGL-RP-E303 (2017), Appendix A:

$$V' + s\left(\frac{\pi D_o^2}{4}\right) = \alpha_o \pi D_o h s_{u1} + \alpha_i \pi D_i h s_{u1} + (\gamma' h + N_c s_{u2})(\pi D t) \qquad \text{Equation 6-2}$$

Where:

D Caisson diameter $\approx \frac{(D_i + D_o)}{2}$

h Installed depth of caisson

N_c Bearing capacity factor (cohesion)

s_u Undrained strength of the soil

s_{u1} Average undrained strength of the soil over the depth of the SICF skirt

s_{u2} Undrained strength of the soil at the SICF skirt tip

t Caisson skirt wall thickness

α Adhesion factor (sometimes estimated as $\alpha = \frac{1}{S_t}$, where S_t is the clay sensitivity)

γ' Effective unit weight of soil

Substituting zero suction pressure ($s = 0$) gives the relationship between vertical load and penetration before suction is applied (self-weight penetration), and positive values of suction (s) for known vertical load (V') give the subsequent relationship between applied suction and depth.

6.2.1.2 Mechanism based method for sands

In sands the situation is more complex due to (a) the contribution of friction enhancing the vertical stress further down the caisson, and, (b) the change in effective stresses around the caisson skirt given the seepage flow regime induced by the applied suction. For a simplified case accounting for these effects Houlsby and Byrne (2005b) , Eq. 16 the result is:

$$\begin{aligned} V' + s\left(\frac{\pi D_i^2}{4}\right) &= \left(\gamma' + \frac{as}{h}\right) Z_o^2 \left(\exp\left(\frac{h}{Z_o}\right) - 1 - \frac{h}{Z_o}\right)(K \tan\delta)_o \pi D_o \\ &+ \left(\gamma' - \frac{(1-a)s}{h}\right) Z_i^2 \left(\exp\left(\frac{h}{Z_i}\right) - 1 - \frac{h}{Z_i}\right)(K \tan\delta)_i \pi D_i \\ &+ \left\{\left(\gamma' - \frac{(1-a)s}{h}\right) Z_i \left(\exp\left(\frac{h}{Z_i}\right) - 1\right) N_q + \gamma' t N_\gamma\right\}(\pi D t) \end{aligned} \qquad \text{Equation 6-3}$$

Where:

a Ratio of excess pore pressure at tip of caisson skirt to beneath caisson top plate

N_q Bearing capacity factor (overburden)

Z Parameter controlling stress enhancement for installation in sand

δ Interface friction angle

Again substituting $s = 0$ gives (approximately) the relationship between vertical load and penetration before suction is applied, and positive values of s (for known V') give the subsequent relationship between applied suction and depth.

The factor α in the above calculation determines the fraction of the suction transmitted to the caisson tip and is itself a function of the caisson penetration and of assumptions about soil permeability. For very shallow caisson penetrations in soil of uniform permeability this factor would be 0.5, reducing to about 0.15 for $\frac{h}{D} = 1$. The factor can be modified to account for increased permeability within the caisson due to soil loosening as the installation proceeds. The variable Z accounts for the stress enhancement and depends on the area of soil over which the friction forces developed on the caisson are carried. For the basic case (as given above), assuming the internal friction is carried uniformly across the soil plug within the caisson leads to Equation 6-4 and the external friction is carried uniformly across a zone between diameters D_o and $D_m = mD_o$ to give Equation 6-5 below

$$Z_i = \frac{D_i}{4(K \tan\delta)_i} \qquad \text{Equation 6-4}$$

$$Z_o = \frac{D_o(m^2-1)}{4(K \tan\delta)_o}. \qquad \text{Equation 6-5}$$

More complex variations can be considered but cannot be solved analytically. The predictions depend significantly on the choice of the factor m, and values of approximately 1.5 have been found to be appropriate.

6.2.2 CPT-based methods

The only point of similarity between the CPT-based methods and the mechanism-based methods is that the resistance to vertical load is again considered as the sum of an end bearing term and a frictional term. Different versions of the method are available, as described by Andersen et al. (2008) and Senders and Randolph (2009).

A common approach for penetration resistance follows the method outlined in DNVGL-RP-C212 (2017) section 7.3:

$$V' = \pi D_o \int_0^h k_f(z) q_c(z) dz + \pi D_i \int_0^h k_f(z) q_c(z) dz + (\pi D t) k_p(L) q_c(L) \qquad \text{Equation 6-6}$$

Where:

k_f Factor for the friction component in the CPT installation method

k_p Factor for the tip component in the CPT installation method

q_c Tip resistance from the cone penetration test

Guidance on the factors, k_f and k_p, can be obtained in DNVGL-RP-C212 (2017) as well as in Andersen *et al.* (2008) based on field case records. Suggested values for both sets of factors are detailed in Table 6-8 below. For other common soil types: Colliard and Wallerand (2008) recommend factors for normally consolidated clays and Lehane et al (2015) for carbonate soils.

Table 6-8: Indicative CPT method factors

Type of soil	Ref	Most probable		Highest expected	
		k_p	k_f	k_p	k_f
Clay	(DNVGL-RP-C212, 2017)	0.4	0.03	0.6	0.05
Sand	(DNVGL-RP-C212, 2017)	0.3	0.001	0.6	0.003
	(Andersen et al., 2008)	0.01-0.55	0.001	0.03-0.6	0.0015

Andersen *et al.* (2008) also suggest a hybrid method for estimating penetration resistance where the skirt resistance is estimated by the mechanism-based approach, whilst the CPT approach is used for the tip resistance.

The penetration resistance in clays can be straightforwardly modified, as for the mechanism-based approach, to account for the effect of an applied suction. However, there is no common approach for addressing the effect of suction on the penetration resistance in sands. For example, Senders and Randolph (2009) make the assumptions that the external friction remains unchanged in the presence of the suction whilst the internal friction and the tip resistance reduce in proportion to the degree of critical suction, s_{crit}, mobilised. The critical suction at a given penetration is defined as that for which piping would occur, and so the critical suction varies with depth. This yields the result:

$$V' + s\frac{\pi D_i^2}{4} = \pi D_o \int_0^h k_f(z)q_c(z)dz + \left(\pi D_i \int_0^h k_f(z)q_c(z)dz + (\pi Dt)k_p(L)q_c(L)\right)\left(1 - \frac{s}{s_{crit}}\right) \quad \text{Equation 6-7}$$

Andersen et al. (2008) describes a variation of this method. There are several suggestions for calculating s_{crit}; for example in Senders and Randolph (2009): $s_{crit} = 1.32\,\gamma' D\left(\frac{L}{D}\right)^{0.75}$.

Installation considerations typically lead to caissons of $L/D < 1$ in sands (drained installation) whereas $L/D > 1$ (but less than 5) can typically be installed in soil profiles consisting of clays (undrained installation) if cavitation is not limiting.

6.3 Touch down and self-weight penetration

During the initial penetration phase the SICF penetrates the seabed under self-weight, the caisson vents are generally open, so that water can freely escape as the caisson is lowered and weight is transferred from the crane to the soil. The rate at which self-weight penetration occurs and water flows through the hatches must be assessed and the hatch size and/or penetration velocity selected to ensure that overpressures are kept within acceptable limits to avoid piping or hydraulic fracture in the soil (additional discussion in Section 7.8.1). Note that this is particularly important if water inflow is limited in any way for reasons such as controlling verticality.

The potential rate of flow through the hatches is likely to be an important factor and the number and size of hatches should be designed to allow for sufficient flow at the anticipated touch down speed.

The water flow out of the hatch(es) can be evaluated using the standard nozzle flow methods such as detailed by Rajaratnam (1982), or DNVGL-RP-N103 (2017).

6.4 Suction installation

For installation planning, it is important to estimate the flow of water during the suction phase, to enable the sizing of pumping equipment, pipework, and to generally plan the installation process.

During suction installation in clays the volume of water removed is the water displaced by the caisson internal volume, as the caisson is installed. The flow rate required will simply depend on the time allowed for the suction installation process.

For sands the process is more complex as flow is induced within the soil which increases the volume of water that must be removed over that due simply to the caisson internal volume. Simplified calculations for the flow due to seepage alone are described in Houlsby and Byrne, (2005b). For the assumption of uniform excess pore pressure across the base of the caisson and uniform permeability the seepage flow is approximated by $q = \frac{kDs}{\gamma_w}\frac{(1-a)\pi}{\frac{4h}{D}}$. The calculation can be modified to account for increased permeability within the caisson due to soil plug loosening.

6.4.1 Limits to suction pressures

The required suction during installation should not exceed certain limits, defined in the following sections. The designer should assess the required suction pressure relative to the limiting pressures detailed below. The required suction pressure should be calculated for the best estimate and upper bound soil parameters with no partial factors. However, the designer may consider it appropriate to include a safety factor in the calculation of the limiting pressures for plug heave, piping, buckling, etc.

6.4.1.1 Pump capacity

The pumping system should have the capacity to achieve the required maximum pressure differential at the required flow rate. The pumping equipment should be specified by an experienced engineer, who should review the detailed pump specifications including:

- Pump characteristic curves which relate the power supplied to the pressure difference and the flow rate.
- Potential losses between the pump and the caisson.
- Potential cavitation issues (discussed below)

6.4.1.2 Cavitation

When the absolute pressure of the water in the caisson is equal to the vapour pressure of water (which is about 1kPa to 2kPa at relevant temperatures), the water would be expected to cavitate in whichever part of the system is subjected to the lowest pressure. Any attempts to lower the pressure further will fail. Note that at a depth of sea water $h_w$and with a suction $s$the absolute pressure will be $p_a + \gamma_w h_w - s$.

Less than 100% of the atmospheric pressure may be used to provide a safety margin (e.g. 80%).

Cavitation is an important limiting factor particularly in shallow water. Where cavitation is limiting (i.e. in shallow water and stiff clays) the caisson diameter is an important design driver.

6.4.1.3 Structural failure: buckling (structural integrity)

The structural integrity of both the top plate and the shell should be verified for the design underpressure to be applied.

A shell structure subjected to an internal underpressure is susceptible to buckling. The critical under-pressure is dependent upon the skirt strength, thickness and curvature and the effective buckling length of the cylinder. The structural integrity of steel shells is covered in dedicated guidance documents such as (DNV-RP-C202, 2013), (Eurocode 1993-1-6, 2007); see also Bakmar (2009) and Bakmar et al (2010). To allow for understanding of the process DNV-RP-C202 (2013) details the following for unstiffened circular shells:

Elastic buckling strength, $f_E = C\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{L}\right)^2$ Equation 6-8

Where: E = Young's modulus (for standard steel ~210 GPa)

ν = Poisson's ratio (0.3)

L = unsupported length

t = thickness

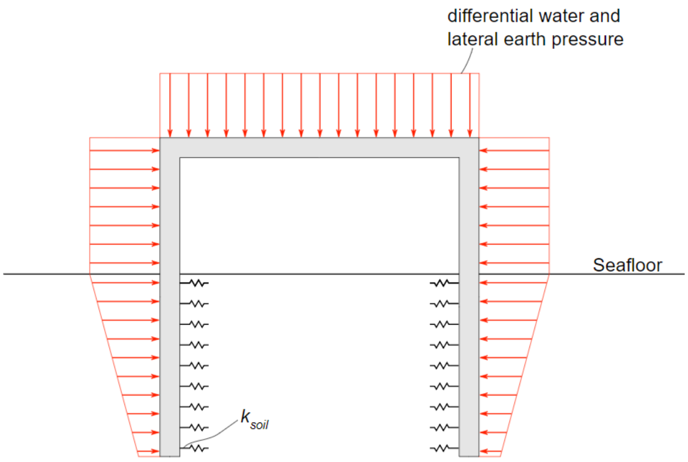
C = reduced buckling coefficient, $C = \Psi\sqrt{1+\left(\frac{\rho\varepsilon}{\phi}\right)^2}$

For hydrostatic pressure: $\psi = 2,\ \rho = 0.6,\ \xi = 1.04\sqrt{\frac{l^2}{r*t}\sqrt{1-\upsilon^2}}$

l = unsupported length, r = shell radius, t = shell thickness

Noting that the embedded soil provides partial support to the shell, solving Equation 6-8 essentially requires the determination of an equivalent unsupported length of the shell. The equivalent unsupported length of the shell is a function of the non-penetrated length of skirt and the effective stiffness of the embedded section (see Figure 6-10).

Figure 6-10: Structural integrity assessment for installation case



The stiffness of the embedded section depends on the soil type, soil/strength stiffness and potentially the flow regime which could influence the effective stresses and resultant stiffness in granular soils.

Loose or soft soils (or sands with a low effective stress due to water flow) will provide less stiffness and therefore the effective unsupported length will be greater than for dense/high strength soils. For an initial buckling risk evaluation an equivalent point of fixity could be assumed at a given depth below seabed to give an equivalent unsupported length. The calculation of effective unsupported length should entail close communication between the geotechnical engineer and the structural engineer and consider an appropriate stiffness response corresponding to anticipated flow conditions. For more detailed buckling calculations linear soil stiffness springs could be used to model the soil.

The detailed calculations should consider any stiffeners and/or internal structure to the caisson if present. Therefore, more complex non-circular shapes require more detailed assessment.

As an alternative to the methods detailed above Finite Element Analysis may be used to evaluate buckling with the use of appropriate soil springs. This may be especially appropriate during detailed design and for more complex structures.

6.4.1.4 Soil failure: piping and ratholing

Piping is a specific type of hydraulic failure (see EN 1997-1:2004, 2009) in which internal erosion begins at the low-pressure boundary and regresses until a pipe-shaped discharge tunnel is formed between the hydraulic boundaries. The pipe-shaped tunnel once formed is also called a rathole. A rathole is also terminology used for a pre-existing hole, for example, between the skirt tip and the soil due to limited initial penetration and incomplete skirt-soil contact.

Piping may be diffuse or localised. Especially in freely draining soils (sands and coarser materials) the possibility of soil failure by diffuse piping or of localized “ratholing” is a primary consideration during installation.

Piping occurs when the upward hydraulic gradient in the soil becomes sufficient to reduce the effective stresses to near zero. As penetration of a suction caisson into sand may depend on the action of upward hydraulic gradients to reduce effective stresses at the caisson tip, piping conditions may be approached in sand. SICF designers should consider this and optimise the design to reduce the risk of piping; empirical evidence would indicate piping is not common for properly designed and installed SICF in sand.

If piping does occur then it may have various adverse effects, including (a) the inability to apply further suction, (b) excessive loosening of the soil within the caisson and (c) development into ratholing (see below).

An initial estimate (Houlsby and Byrne, 2005b, Eq. 25) of the penetration at which this may be of most concern is given by $h \approx \frac{D}{2(K\,tan\,\delta)_o}$ (however, it is a concern at all depths). While it is acknowledged that estimating of K after penetration of the caisson is difficult, values for Ktanδ ranging from 0.45 to 0.8 were found through back-calculation of several case studies. For typical parameters this indicates that the limit of suction-assisted penetration in sand is on the order of one diameter. Further experimental data on critical suction pressures and piping is provided in Panagoulias et al. (2017).

Piping conditions are intrinsically unstable; any region of slightly higher permeability will attract greater flow which in turn will loosen the soil and increase the permeability. Thus, piping can rapidly lead to the form of localized “ratholes” or channels beneath the caisson. Ratholes may also form if excessive suction is applied early in the suction-assisted penetration, leading to localized flow at points of weakness. Once ratholes have formed further application of suction is likely simply to lead to their further development and not to satisfactory penetration of the caisson. The conditions for the possibility of ratholing are similar for those controlling suction, except that account needs to be taken of possible non-uniformities or irregularities in both geometry and soil properties.

In addition to the conditions described above, other conditions are required for piping to fully develop: (1) time,(2) a higher permeability soil zone or (3) a grading that allows for individual particles to be washed out. These effects may be less well understood, or difficult to quantify.

Equilibrium methods can be useful to assess the risk for onset of piping, whereas experience indicates that steadily continuing penetration of a caisson can prevent the actual development of piping channels (Erbrich and Tjelta, 1999). Further details are available in Vandenboer et al. (2017) and Ibsen and Thilsted, (2010).

6.4.1.5 Soil failure: punch-through or squeezing

A 'strong' soil overlying a 'weaker' soil could result in a significant reduction in penetration resistance at the interface between the layers. Resulting in the potential for potentially uncontrolled penetrations (punch-through) when the SICF mobilises the resistance of the soft layers. Where soil layering has been identified, the potential for punch-through should be considered and the pressure/penetration rate controlled to prevent sudden uncontrolled penetrations.

The potential for squeezing of soft impermeable clay layers should also be considered where necessary.

6.4.1.6 Soil failure: plug uplift

In certain circumstances in layered soil with a layer of impermeable material (clay) over a freely draining material (sand) the suction may be sufficient to lift a "plug" of clay within the caisson, creating a void beneath the plug. Such a scenario is clearly undesirable and needs to be addressed by appropriate bespoke design calculations, such as described by Cotter (2009).

6.4.1.7 Soil failure: reverse bearing failure (for clays)

For a caisson installed into clay soils it is possible for a "reverse" bearing capacity failure to occur, if the differences in vertical stress between inside and outside the caisson is such that plastic failure occurs, so that soil flows into the caisson. An initial estimate of the geometry for which this condition will become critical was given by Houlsby and Byrne (2005a), modified Eq. 8) as $\frac{h}{D} \approx \frac{N_c^*}{4\alpha_o}\frac{s_{u2}}{s_{u1}}$. For a range of parameters this indicates limits of $\frac{h}{D}$ between 3 and 6 but with considerable variability.

6.4.2 Installation tolerances – penetration and verticality

Defining the installation tolerance on SICF final elevation is part of installation design. An assessment of stick-up length (caisson top elevation above natural seabed level) should take account of the following effects:

a) Soil plug heave: This will be, approximately, the displaced soil due to the caisson skirt, plus, for sand, an allowance for the loosening of the soil plug.

b) Fluctuations of the seabed level across individual caissons and across a system of caissons for a multi-caisson structure.

c) Other effects such verticality/tilt across individual caissons as well as across a system of caissons.

There could be multiple inclination tolerances during installation, which must not be exceeded during installation (for structural integrity or for toppling) and which must not be exceeded at the end of installation.

6.5 Trial installations

Because of the uncertainties in relation to suction caisson installation calculations and the current state of knowledge, especially in layered or heterogeneous materials, trial installations may have an important role to play in the process of de-risking and design optimisation for major projects. They can be used to:

a) validate design methods by comparing predictions (for the trial) against measurements;

b) refine design methods, for instance by development of site-specific values of factors such as k_f and k_p in the CPT-based design methods, or the adhesion factor, α, in clays;

c) develop installation procedures and protocols including for measurements;

d) identify/confirm or discount risks.

The potential benefits of field trials should be considered especially with regards to cost savings due to design optimisation and risk reduction/mitigation. A cost-benefit assessment can then be undertaken to ascertain whether trials are worthwhile.

In designing a field trial, the following should be considered:

a) Scale and geometry: a trial caisson will usually have a smaller diameter than the prototype but required to penetrate to the same depth. The L/D ratio will therefore be larger for the trial. However, if cavitation is likely to be a limiting issue then careful consideration should be given to the diameter of the test caisson. Furthermore, the wall thickness may be different in trial, and it may not include relevant details such as stiffeners.

b) Loading: dead loading available in a trial may be significantly lower than that of the prototype.

c) Fixity: a single caisson trial is likely to be unrestrained against rotation and may not be fully representative of a (restrained) multi-caisson design.

Specific tests may also be devised to complement field trials and de-risk installation or reduce costs. For example, a vertical plate penetration test to directly measure the adhesion factor for penetration in stiff cohesive soils could be considered.

To ensure relevant and useful data are obtained from a trial, a range of measurements should be taken during the trial, as a minimum those described in Section 7.2.

6.6 Non-homogenous soils for installation

Installation into homogeneous deposits of sands or clays is reasonably well understood, and can be described by calculations, as in Section 6.2.

Soils for which further assessment would be required are discussed in this section.

6.6.1 Installation in layered or heterogeneous soils

Layered soils can result in seepage conditions which are not uniform and which in turn affect the penetration resistance. Such conditions require special consideration for caisson installation, but many successful installations have been performed in complex sand-clay stratigraphy at a variety of locations (e.g. Tjelta et al., 1986; van den Heuvel and Riemers (2005); Alderlieste and van Blaaderen (2015)). Recent experience of installation in layered soils is described in Saue et al (2017) and Panayides et al (2017).

Too many combinations of layering are possible for them to be treated comprehensively, but the following general considerations apply:

a) Installation into layers of clay of different strengths should generally be possible if each individual layer can be penetrated.

b) Installation into layers of sand of different density/strength should be possible if each individual layer can be penetrated.

c) Where a clay layer occurs beneath sand, penetration should be possible. However, note the necessity to check for "plug uplift" if there is further sand beneath the clay layer (even below the final design penetration depth), see Section 6.4.1.5.

d) Where a sand layer occurs below clay, the clay may prevent the formation of the seepage flow pattern in the sand which reduces the tip resistance. The ability to penetrate the sand layer without the beneficial reduction in effective stress due to water flow should be checked. Note also the necessity to check for "plug uplift" of the clay layer, see Section 6.4.1.5.

Guidance for assessing installation feasibility and installation design/operations are provided in the literature cited above.

The above comments relate principally to soils which consist of relatively thick layers of different types. Deposits consisting of very finely interbedded layers are unlikely to pose a problem for installation as long as penetration could be achieved in a uniform material of equivalent strength, although note that seepage patterns in such materials are likely to be very different from those on more uniform materials.

Note that all design methods in current use assume that the soil properties are uniform in the horizontal direction. Horizontal non-uniformity may clearly present an obstacle to penetration, and (especially for mono-caisson structures) could result in tilting. Such non-uniformity may for instance occur in materials that have been deposited under glacial, peri-glacial or fluvial conditions, in which pockets or lenses of different materials may occur. Differences of permeability may be as important as differences of strength.

6.6.2 Installation in other materials

The following cases will require special consideration and installation may not be feasible at all depending on the proportion of these specific soils in the overall soil profile. If considered, trial installations are likely to be needed to confirm feasibility:

a) **Stiff highly fissured clays:** Fissures may not allow the formation of a seal around the rim of the caisson for suction-assisted penetration to proceed. One possibility is that fracturing may occur, with water simply flowing through the fissures. This problem may be exacerbated by the fact that the penetration resistance in very stiff clays may be high.

b) **Coarse materials:** Extremely heterogeneous coarse materials, and materials with a significant fraction of coarse gravel or larger sizes would almost certainly present an obstacle to installation. Very permeable gravels, even if not particularly coarse, would require very high flows to generate significant suction pressures.

c) **Boulders:** Individual boulders may be sufficient to obstruct penetration. Micro-seismic site-investigation methods may be necessary at sites where the geological conditions indicate a significant possibility of encountering boulders.

d) **Silts:** Calculations for silts are challenging, as simplified analyses require that drained or undrained conditions are applicable, and this may not be the case. Partially drained calculations for caisson penetration have not yet been formulated. However, given that penetration in clays and sands is relatively straightforward, it would be expected that installation in reasonably homogeneous silts should be possible.

e) **Shell deposits**: Whilst shell material is unlikely to pose a problem due to its strength, significant deposits of coarse or open shell material at the surface could prevent the formation of an effective seal around the caisson.

f) **Carbonate soils:** There is experience of successful installation in carbonate soils, particularly for deep water anchor type applications. The installation calculations need to take appropriate account of the potential crushability of carbonate materials, particularly for the friction component. The potential for cemented layers also requires careful consideration, it may be possible to model these layers as high strength cohesive bands (based upon Uniaxial Compressive Strength of cemented soils) with an associated increase in end bearing resistance but this should be confirmed on a project specific basis.

g) **Special conditions:** The effect of other special conditions (e.g. shallow gas deposits within the depth of the caisson, organic material etc.) is poorly understood and would have to be dealt with on an *ad hoc* basis.

6.7 Multi-chamber designs

Some caissons employ multi-chamber designs in which the suction pressures in different chambers can be controlled independently. The main purpose is to use individual control of pressure in the chambers to control tilt of a mono-caisson. Such methods can be effective in clays and have been successfully used for numerous Gravity Base Structures in the Oil and Gas sector. Note that the end bearing and friction on the internal walls also needs to be considered in calculating resistance to penetration.

7. SITE INSTALLATION OPERATIONS

7.1 Criticality

Once a suction caisson has been successfully installed there is little evidence that any caisson has not performed satisfactorily for in-service and ultimate loading conditions. However, in the past, difficulties have arisen at several sites during caisson installation, and some of these are documented in the literature. The installation phase is therefore critical and must be carried out with considerable care. The key to successful installation is good installation engineering studies, access to high quality soil and monitoring data, and the appropriate use of that data.

Successful installation requires adequate preparation for the offshore operations, reliable, high performance and tested suction equipment, and experienced installation management (Tjelta, 2014).

Real-time or near-real installation time data reviewed by a geotechnical specialist, including comparison with expected behaviour is recommended to ensure that anomalous situations are identified and managed correctly.

7.2 Instrumentation

The installation parameters typically requiring monitoring during installation are detailed in Table 7-9, with an indication of potential instrumentation methods. The table gives an indicative list which is not exhaustive and is not mandatory. The parameters for monitoring and instrumentation should be specified by an experienced engineer.

The parameters detailed should be recorded electronically with the agreed measurement units, datum and a common time stamp. This should be sufficient to ensure appropriate reconciliation between different data logging systems as well as with any handwritten notes and other relevant details.

Table 7-9: Installation parameters and instrumentation

Parameter	Instrumentation / measurement
Effective structure / SICF weight	Crane load
Vertical penetration	Depth (pressure) sensor Echosounder (internal and/or external) External penetration markings and ROV monitor
Tilt (both directions)	Inclinometers mounted on the top plate and the topside structure ROV monitor
Suction pressure*	Internal and external pressure recordings Pump measurements

Parameter	Instrumentation / measurement
Flow rate*	Volumetric pumping rate, which should also be integrated to give total volume pumped
Plug heave	Echosounder to monitor internal plug level
Tip injection	Injection pressure for each section Flow rate for each section
Seabed level (disturbance)	ROV camera monitoring surrounding seabed for evidence of ratholing Geophysical survey equipment mounted to installation vessel

*Note: if multi-chambered SICF are utilised then these parameters should be monitored for each chamber.

The placement of the sensors should be carefully considered to ensure they provide a representative measure of the required parameter to the required level of accuracy, without being influenced by anticipated installation conditions such as deformation of the SICF skirt due to under-pressure (can affect inclinometers), lack of visibility due to soil disturbance (can affect ROV operations), etc.

Appropriate calibrations should be completed, including the recording of known benchmarks (or "zeroes") at key points through the installation process. Sensor calibrations should also be checked following completion of the installation. Time should be allowed during the installation to ensure that monitoring systems have been checked and are working robustly.

7.3 Seabed preparation and scour protection

Seabed preparation is generally not necessary for suction caisson installation. However, in some cases seabed preparation such as dredging and/or levelling may be considered to form a level seabed or for the purposes of scour protection.

Scour may be a significant concern at some sites and scour protection may need to be placed around the foundations. Scour protection typically comprises rock dump or an alternative such as fronded mattresses or geotextiles. Rock dump protection is the most common option and can be placed before or after SICF installation. The advantages and disadvantages are outlined below.

Table 7-10: Scour protection installation

Placement	Advantages	Disadvantages
Before installation	Easier to get complete, even coverage around the caissons	Installation through rock dump needs to be carefully considered and may increase risk (e.g. of penetration punch-through)
After installation	Simplified installation assessment and potentially less installation risk.	Potential difficulties placing rock protection evenly around the complete structure, especially for multi-caisson jackets

The need for scour protection should be assessed, and if it is considered necessary then the preferred placement option needs to be carefully considered. If it is considered appropriate to place a filter layer for scour protection, the ability of the caisson(s) to penetrate through this layer under self-weight should be confirmed. Several SICFs have been successfully installed through layers of scour protection. Analysis usually employs modelling these layers using mechanism-based approaches for cohesionless soils (Section 6.2.1.2) and appropriate geotechnical parameters to model the rock dump. It is necessary to ensure that self-weight penetration is sufficient to give a good seal for suction penetration.

Penetration through a scour protection layer into lower resistance soil below is a potential punch-through situation which needs to be considered and managed. Rapid penetration due to punch-through could lead to overpressures being applied to the soils below, with resulting damage (washout).

Consideration could also be given to allowing scour to develop (no initial scour protection) combined with regularly monitoring the structures and planning remedial works/backfill campaigns to address any scour which is outside design limits. However, the potential for scour development should be well understood, an adequate provision for scour included in the detailed in-place design, and appropriate backfill campaigns planned.

7.4 Lowering through splash zone (venting) and touchdown

Lowering the caisson to the seabed and the subsequent touchdown are critical to successful installation, especially in the case of an erodible or very soft seabed.

Lowering or deballasting should be carefully controlled as discussed below. The caisson should be vented when passing through the splash zone and in the touchdown phase so that air and water can escape through the vent as appropriate, thus minimizing overpressure within the caisson. As the caisson approaches the seabed, water between the caisson and the seabed is displaced and high velocity flows may occur due to the motion of the caisson. The effectiveness of the venting is critical at this stage. If wash-out/erosion occurs from beneath the rim, a successful seal for the application of suction may be more difficult. Assessment of the critical lowering and landing speed can be based on DNVGL-RP-N103 (2017). The required vent area should be evaluated to limit pressure differences inside and outside the caisson accounting for SICF motions at touchdown.

7.5 Self-weight penetration

Once touchdown has been achieved, the crane load should be reduced in a controlled manner (or deballasting performed), to allow the slow penetration of the caisson into the seabed, under the self-weight of the caisson and structure (as discussed in Section 6.3). It is good practice to slowly release the crane load until the end of self-weight penetration to allow for a degree of control in the event of any sudden reduction in resistance or tilt development. After self-weight penetration, limited slack should be maintained. During self-weight penetration the caisson should be vented to allow free escape of water. The rate of penetration should be controlled in such a way that the overpressure within the caisson is kept to a minimum, and certainly not sufficient to cause pressure gradients that would result in piping (on the outside of the caisson) or ratholing. Calculations should be made as part of the installation design to determine limits to the amount of overpressure that can be tolerated.

During this phase it is essential that the following should be monitored:

a) Crane load (and hence vertical load on caisson)

b) Vertical penetration

c) Tilt (in both directions)

d) Overpressure within caisson (internal pressure minus external pressure)

The intention is that self-weight penetration will be sufficient to create an adequate seal between the caisson and ground so that suction-assisted penetration can proceed.

7.6 Suction-assisted penetration

Once self-weight penetration is complete, vents in the caisson should be closed and pumping commenced.

Initially, to correct any off-verticality after self-weight penetration for a multi-caisson foundation pumping will commence at the highest caisson(s), if required.

The suction penetration phase should be carefully monitored to avoid sudden increases in pumping rates, at least in the initial phases where minor changes in pumping rates could lead to critical hydraulic gradients being developed and possible piping. Once the installation is conforming to expected performance then more tolerance on increasing the flow rates can be envisaged.

During this phase it is essential that the following should be monitored:

a) Crane load (and hence vertical load on caisson)

b) Vertical penetration

c) Tilt (in both directions)

d) Suction within caisson (external pressure minus internal pressure)

e) Volumetric pumping rate

These monitored quantities should be compared with expected performance in real-time, to allow identification at an early stage of any deviations which may be indicative of a problem developing.

7.7 Underbase filling

When penetration of the caisson ceases, some voids remain between the seabed inside the caisson and the caisson top-plate. These voids may be due to many factors: an uneven seabed, a sloping seabed, settlement of the soil plug, non-vertical installation of the substructure, a designed stick-up and/or due to not reaching the target penetration. The voids will naturally be filled with water. This can freely exchange with the ambient seawater, unless all openings, for example for suction and venting, are closed after the installation.

7.7.1 Considerations for underbase filling

Filling of these voids is normally required to ensure the design load-transfer between the top-plate and the soil, and to regulate the contact stresses, except in specific circumstances:

- If sufficient stiffness and vertical capacity can be demonstrated through caisson wall resistance alone. This may be the case if sustained tensile loads are governing.
- If sufficient (differential) settlement of the substructure can be accepted until the necessary load-transfer capability is achieved, and the stiffness is acceptable in the meantime. This may affect both the foundation tolerances and the structural design of the caissons and substructure.
- If contact between top-plate and soil can be ensured by alternative means. For example, two methods proposed by the industry consist of (1) using a double top-plate, and (2) using a water jetting and slurry suction system.
- If load-transfer through the water-filled voids can be demonstrated for short duration loads while not compromising any other load transfer or structural requirement. This may be possible for low permeability soils. Reliable sealing of all openings in the top-plate after the suction installation would be required for the lifetime of the structure.

If underbase filling is omitted, or the voids only partially or inadequately filled to achieve full load-transfer, then the structure and foundation must be designed to accommodate the caisson response to the applied loads. All relevant support conditions for the top-plate should be considered in the structural design of the caisson and in the analysis of the foundation stiffness: fully supported, unsupported, and partially supported, along with the associated settlements.

7.7.2 Functional specification

The material to fill voids under the top-plate of suction caissons should have, at the time of loading, a strength and stiffness similar to or greater than the surface soil. It should also have sufficient strength so that it is not extruded through any open valves. However, at the time of filling, the material should have sufficiently low viscosity for pumping into the caisson.

The curing time, when using a cement-based material such as grout, should therefore be specified in relation to the time between installation of the foundation and installation of the wind turbine or substation topside.

The filling material should retain sufficient strength and stiffness during the lifetime of the structure. Possible deterioration should be considered: chemical, mechanical, shrinkage, and installation-related problems such as incomplete mixing and dilution.

The filling system should be designed in such a way as to ensure adequate filling of the void, while avoiding uplift of the suction caissons when applying the filling material (e.g. grout) under pressure. Moreover, the system should not limit the under- or overpressures to be applied during the suction installation. The piping system for delivery of the filling material to the underbase area should be designed and tested for appropriate pressures.

To ensure efficient completion of the filling operation, the system should include redundancy and back-up equipment.

Test procedures should be defined for both the filling material and equipment. Tests should be performed, and the results documented accordingly.

7.7.3 Standard grouting procedures and equipment

The recommendations given below are based on best practice as used within the industry where underbase filling of caissons has been applied.

Generally, a cement-based grout is used, consisting of cement, sodium silicate, and seawater. Other additives may be considered on a case-by-case basis e.g. to accelerate or retard the curing time. However, other materials may be considered if they meet the required functional specifications.

The following non-exhaustive list outlines the equipment that has been used for underbase grouting operations:

- Grout silos, mixing and pumping systems and testing equipment on deck of heavy lift or installation support vessel
- Grout flow lines, flexible hoses or pipes, from deck level to suction caisson, running along or through the legs of the substructure; connectors may be situated on top of the substructure or subsea on top of the suction caisson
- Multiple grout inlets/outlets, associated piping, valves and an ROV control panel on the caisson top-plate; the valves and ROV panel allow for opening and closing of the grout inlets and outlets, and for monitoring grout overflow
- An additional separation valve below the suction pump may be considered; closing this valve would allow for commencing the filling process while avoiding grout spilling into the suction pump; alternatively, the suction pump would be removed prior to the grouting
- A back-up system should be considered for all items deemed critical.

The grouting procedure can be summarised as follows:

- Prior to the suction installation, all piping protruding from the suction caisson top-plate is closed by valves
- After reaching the final penetration the separation valve below the suction pump is closed, or the pump is retrieved
- The respective grout inlet and outlet valves are opened, grout supply connected, and the filling through one or multiple inlets starts
- The pumped-in grout volumes are monitored and the grout outflow at the subsequent outlets at the caisson are observed to establish if the filling is complete. Measuring the properties of

the filling material at the outflow may be considered to determine when non-diluted grout is being returned.

- Finally, the grout valves are generally closed and (optionally) the flow lines removed.

7.8 Installation hazards

There are a number of hazards which could result in installation difficulties. Several are outlined below with potential indicators.

The potential installation risks should be evaluated at an early project stage in a ground risk register which monitors uncertainties and risks though the project and recommends appropriate mitigation strategies. Before proceeding with mitigation measures attempts should be made to understand the cause so that appropriate mitigation techniques may be adopted.

7.8.1 Instability during touchdown

Causes:

The water entrained within the caisson during fast touchdown or seabed impact can result in:

- local or global soil plug failure as water tries to 'escape' around the skirt tips
- lateral hydroplaning and possibly seabed gouging as the caisson is supported on a cushion of water
- vertical or 'pumping' scour below the caissons

Indicators:

Lateral movement of the structure, seabed damage and/or erosion of seabed soils.

7.8.2 Piping or ratholing

Causes:

Piping or ratholing is a hazard in freely draining soils (e.g. sand, gravels, shelly seabed) – see Section 6.4.1.4. It is most likely to occur early in the suction-assisted penetration phase and may be due to a variety of causes, such as excessive applied suction, limited self-weight penetration and/or uneven seabed, disturbance of top-soil during set-down, (pre-existing open) hole due to CPT, borehole, pockmark, or obstacle.

Indicators:

Onset of piping or ratholing may be indicated by any or all the following:

a) Increased pump flow whilst pressure maintained constant or decreasing.

b) Rapidly fluctuating pump flow and/or pressure. This is particularly indicative of ratholing as holes may form and collapse, resealing the hole, and then reform.

c) No further penetration.

7.8.3 Shallow obstructions

The presence of obstructions such as cobbles, boulders, debris or even dense/hard soils could result in shallow refusal during installation operations.

Causes:

Inability to penetrate to the required depth using suction may be due to a variety of causes, such as boulders or other obstructions below seabed.

Indicators:

Self-weight penetration does not exceed minimum depth to provide a sufficient seal (to be determined based upon site specific conditions). When monitoring of suction pressures and vertical penetration, any significant obstruction is likely to show as a sudden increase in pressure for no additional penetration.

7.8.4 Structural issues

The structural integrity of the SICF could halt installation operations.

Causes:

- required installation pressure exceeds allowable limits, with respects to skirt buckling / top-plate failure
- buckling of the skirt tip results in an increase in penetration resistance

Indicators:

The measured suction pressure exceeds the specified pressure limit for buckling (or top plate failure). Penetration resistance and therefore required under-pressure increases if a buckle forms and propagates, which could result in early refusal.

7.8.5 Tilting

Maintaining the tilt of the structure to within tolerances is part of normal installation operations. Tilting of multi-caisson structures (tripod, tetrapod) can generally be corrected by adjusting the penetration velocity of each caisson using differential flow rates (and therefore suction pressures). Tilting of mono-caisson structures is more difficult to correct and, for this reason, some mono-caisson designs include different skirt compartments to allow different flow rates to be applied.

Causes:

In ideal conditions caissons should be essentially self-levelling, but tilt may occur due to a variety of causes, including:

a) **Eccentric loading**: During installation, especially in the early stages, caissons are remarkably sensitive to eccentric loads. Care should therefore be taken to minimise eccentric loads as far as possible due to, for instance, slewing of the crane or loads applied by connections to ancillary equipment (hoses *etc*).

b) **Inhomogeneous soils**: Inhomogeneity either of strength or of permeability could cause tilting. Such inhomogeneity could be due to local pockets of stronger/weaker or more/less permeable material or could be due to non-horizontal horizons between differing materials.

c) **Local obstructions**: Man-made (metallic) obstructions such as anchors, UXOs etc. should be detected during the site investigation phase. Natural obstructions such as boulders may occur in some geologies. The site investigation should reveal whether these are a possibility.

Indicators:

Tilt monitoring. For a mono-caisson, the tilt measurements may be combined with vertical penetration to define the elevation of the perimeter of the skirt tip and the possible position of a localized obstruction. For a multi-caisson, tilt should be measured at the caissons and the jacket and the relative inclination monitored. Correcting the tilt by penetrating or extracting one caisson could result in a significant bending moment at the jacket-caisson connection which should be assessed.

7.8.6 Soil plug failure

Causes:

Soil plug is displaced up into the SICF rather than the under-pressure penetrating the SICF below seabed level:

- Excessive plug heave or soil loosening
- Plug failure

Indicators:

Internal plug is displaced upwards into the SICF – requires monitoring of relative levels of seabed and inner plug possibly using echosounder.

7.9 Potential mitigation measures

Several of the more common hazards to installation are detailed above. The effectiveness of the mitigation strategies is dependent upon the specific circumstances of a project and in some situations certain proposed mitigation strategies may make the problem worse. Therefore, the most effective strategy for a given project should be determined by an experienced geotechnical engineer in conjunction with the installation team.

7.9.1 Pre-emptive mitigation measures

The most efficient way to mitigate against potential hazards is to identify them at an early project stage (during the site characterisation discussed in Section 4) and subsequently avoiding them or including appropriate allowances in the design. Common pre-emptive mitigation strategies are:

- ***Hazard identification and mapping:*** Adequate seabed survey including geophysical and geotechnical to accurately identify potential hazards and allow SICF to be safely micro-sited. An example would be avoiding areas with a significant gravel and cobble content at seabed where possible.
- ***Geotechnical understanding:*** Adequate seabed survey data to accurately characterise the geotechnical profile and parameters, providing a good understanding of lateral and vertical variability and a bespoke and optimised SICF geotechnical design accounting for that variability.
- ***Seabed preparation*** such as dredging or rock armour should be suitable designed and allowed for in the design of the SICF installation procedures.

- ***Tip injection:*** Application of these systems can be effective in achieving further penetration especially in cohesionless materials. It may also be effective in cohesive soils but could reduce the tensile capacity of the SICF. Therefore, any potential influence on long term capacity should be evaluated if tip injection is planned.
- ***Multi-chamber designs for mono-caissons:*** Some SICFs incorporate multiple internal chambers, with the facility to apply differential pressures within each chamber. In that case differential flow rates and suctions may be applied with a view to controlling the tilt of the caisson (see Section 7.8.5). The robustness of multi-chamber designs should be demonstrated considering the complex seepage paths which are generated between chambers.

7.9.2 Reactive mitigation measures

All reactive mitigation measures require active monitoring of the instrumentation data (Section 7.2), appropriate engineering interpretation, and informed decision making. For the right decisions to be taken in a timely manner, preparatory engineering studies are required, and they must be available on-board to assist the geotechnical specialist supervising the installation. If any issues are encountered during installation it is important that all available data is reviewed to determine the most likely cause of the issue and the most appropriate mitigation method is selected, this is likely to necessitate halting operations to allow all monitoring data to be properly reviewed and evaluated.

- ***Pause installation:*** Stopping pumping for some time will cause piping to cease and may allow ratholes to heal naturally as loose material falls into the hole. Pumping should then be restarted cautiously.
- ***Additional ballast:*** Weight may be applied either directly to the caisson or to the structure, thus increasing the self-weight penetration and possibly closing the piping channel. This is likely to be an effective but expensive remedy early in the penetration phase.
- ***Remedial seabed works:*** Ratholes may be observed from an ROV and then sealed with sandbags or similar material.
- ***Pressure cycling:*** There is some empirical evidence that cycling of pressure within a caisson, especially in overconsolidated clays, may promote further movement in cases where a constant suction has been ineffective. In some cases, applying an overpressure sufficient to cause a small upward movement before reapplying suction has enabled penetration to be continued. This procedure may only be effective if there is sufficient reverse movement to cause some degradation of soil strength on the sides and/or at the tip of the caisson. The mechanism which causes this procedure to be effective in some conditions is not well understood, nor scientifically validated. It should, therefore, be employed with caution and only with an understanding of the potential consequences of cycling. Overpressure cycling may have a detrimental effect on the in-place capacity.
- ***Pressure shock:*** Some practitioners have advocated the use of a “pressure shock” to promote movement. This involves a sudden, brief increase in pumping power and hence suction. The mechanism is not understood, nor demonstrated in which conditions it may provide a benefit. In some circumstances it may have a detrimental effect on in-place capacity. Installations involving this technique require site-specific engineering, further research and field trials before use.

- ***Eccentric loading:*** Installation systems for multi-caisson foundations permit the deliberate application of eccentric loading by means of individual control of the suction pressures in each caisson to correct tilt.
- ***Retrieval and relocation:*** Ultimately the measure of last resort would be retrieval and relocation.

7.9.3 Selection of appropriate mitigation methods

Several of the more common hazards to installation and mitigation options are discussed in detail above. The Table below outlines mitigation strategies that may be appropriate for given hazards, the lists are not extensive or exhaustive.

Table 7-11: Installation hazards and suggested mitigation methods

Hazard	Mitigation
Piping or ratholing	Pause operations Remedial seabed works Backfilling holes Ballasting
Shallow obstructions	Tip injection Ballasting Pressure cycling / Pressure shock
Tilting	Tip injection Eccentric loading Multi-chamber design
Soil plug failure	Pause operations (no underpressure) Retrieval and reinstallation Pressure cycling

This list provides an indication of mitigation options that may be suitable, however, in some situations certain proposed mitigation strategies may make the problem worse. Therefore, if there are difficulties during installation all available data should be reviewed and evaluated as discussed in Section 7.9.2 and the cause should be determined by an experienced engineer and the most appropriate mitigation methods applied.

7.9.4 Water injection systems

Some caissons, especially mono-caisson designs, are fitted with systems that allow water to be injected under pressure at a number of points very close to the rim of the caisson. The purpose is to increase pore water pressure locally within the soil, thus reducing the effective stress and reducing the resistance of the soil to penetration. In extreme circumstances, the effective stresses could be reduced to near zero, causing the soil to liquefy locally. Application of these systems can be effective in achieving further penetration in free-draining materials but is unlikely to be effective in clays.

The use of water injection systems should not be confused with “jetting” systems which are sometimes used to install pipe piles or conductors. Jetting systems are designed to physically remove soil from beneath the pile tip and would require high flow rates and pressures. Caisson injection systems are designed to control the water pressure in the soil and should not require high flow rates or pressures. Excessive flow, causing removal of material, is likely to be detrimental to the subsequent in-service performance.

Water injection systems can be used for two purposes:

a) For either mono-caissons or multiple caisson systems, injection systems may be used to enable further penetration if this is no longer possible under the action of applied suction alone.

b) For mono-caissons water injection has been successfully used to control the tilt of the caisson. To achieve this the system needs to be segmented to allow separate injection into multiple sectors around the caisson rim. Water should be injected to the sectors where the penetration is smallest to correct tilt. Such a procedure is most effective at relatively small penetrations; once tilt becomes established at deeper penetrations it is difficult to correct.

The water injection system should be designed such that the nozzles do not get blocked during the penetration process and are in effect self-cleaning.

During water injection, the injection pressure and flow rate should be monitored. However, pressure measurements are likely to be well “upstream” of the actual injection points. Losses in the pipework and more specifically at the nozzles (which are usually of small diameter) means that the measured injection pressures bear little resemblance to the actual pore pressure in the soil. The measured pressures will be substantially higher. As a result, the actual pore pressures in the soil are poorly known and so control of the system is based largely on empirical experience. Such systems require adequate testing and experience and should be used with caution.

7.10 Non-circular caissons

Almost all caissons are either circular or near-circular in plan (some current designs involve a series of circular segments forming a slightly scalloped outline). The procedures described here are appropriate for these caissons. Other designs, square or rectangular, sometimes involving multiple compartments and even internal voids, have been used in applications outside the wind industry. Such designs which involve corners are beyond the scope of this guideline. Note, however, that some work has been done on the influence of corners on the installation process, see Tapper et al (2017).

8. IN-SERVICE DESIGN

8.1 General background

Soil response is non-linear, with a continuous transition from high stiffness at very small strains, through reducing tangent stiffness at intermediate strains through to ultimate failure.

Design approaches will consider the foundation response at operational load levels which will correspond to relatively small strains or "elastic" response (the Serviceability Limit State, SLS and Fatigue Limit State, FLS), and the "ultimate resistance" at large strain levels (Ultimate Limit State, ULS). Satisfactory performance under serviceability loads is often (but not necessarily) assured by a design adopting an appropriate factor of safety on a "failure" calculation.

Therefore, after setting out important background considerations, this Section presents the ULS assessment first (Section 8.6). SLS design is then addressed (Section 8.7) and methods for estimating the stiffness and permanent deformations under operational load levels are discussed.

For ULS design, the analysis must capture the ultimate resistance which is often modelled by considering only plastic deformations (i.e. high strain levels) using monotonic loading. In contrast, for SLS design, the effect of repeated loading is accounted for by selecting foundation stiffnesses at appropriate (relatively low) strain levels and giving due consideration to other potential effects of cyclic loading (such as excess pore water pressures), if any.

The design process is likely to include the following stages:

- Conceptual level design: Simplified calculations for sizing of the caisson foundations. These would follow well established bearing capacity formulations, including simplified VHM formulations (described further in Appendix C and below), and address the critical failure modes.
- FEED level design: More advanced or refined assessment, including numerical approaches. These would build on the conceptual design through more rigorous assessment of foundation stiffness and hence foundation loads for different load cases, as well as more accurate resistance calculations and sizing (diameter and penetration).
- Detailed design: Likely to involve bespoke numerical analysis approaches such as FEA for both resistance and stiffness verification. Site- or location-specific VHM envelopes may be assessed or verified and the analysis may also include coupled foundation-structure analyses.

Typically, mono-caisson structures will adopt caissons with $L/D < 1$, as the moment capacity scales with LD^3, whereas for multi-caisson structures, where vertical tensile capacity may be critical, it may be beneficial to use longer skirt length caissons, providing they can be installed.

8.2 Loading regime

The loading on the caisson foundation must be sufficiently defined so that the in-service assessment captures appropriate foundation failure modes:

a) For a multi-caisson structure (see Figure 2-1), the applied loading will be translated into variable vertical loading on the opposing foundations. The magnitude of the footing loads will depend upon the jacket footprint area, structural weight and the environmental loading. The downwind footing will experience the most significant compression load and the upwind footing reduced compression loads or possibly tension uplift. Both the compression capacity of the most heavily loaded downwind footing and, where relevant, the tension capacity mobilised on the upwind footing must be verified. Horizontal loads will be shared across the foundations. The load distribution can only be assessed accurately once the foundation stiffness has been determined (see Section 8.7). The moment loading on each SICF will be a function of the stiffness of the structure and structure-foundation connection, which is likely to result in low applied moment loads.

b) For a mono-caisson (see Figure 2-1), the applied load translates to a combined vertical, horizontal and moment loading on the foundation. The applied vertical load is equal to the structure weight and is likely to be a small fraction of the anticipated vertical capacity of the foundation. The ratio of moment to horizontal load, M/HD, is a convenient dimensionless variable for load characterisation.

Foundation clustering strategies may lead to design efficiencies, however, the potential implications on design risk should be carefully considered as discussed in Section 4.7.

8.3 Geotechnical analysis

Plausibility checks and sensitivity studies using simplified calculation methods as outlined in Sections 8.6 and 8.7 are recommended in parallel to FEA to allow bench-marking / sense checking of the more advanced modelling and to ensure the soil layers and parameters which dominate the response are adequately understood and characterized for detailed design.

8.3.1 Soil layering

Soil conditions at a site are rarely uniform and simplified design soil layering will have to be assessed by the geotechnical specialists who will also assign geotechnical parameters to each layer for analysis and design (see Sections 4.6 and 4.7).

At conceptual design stages, simplified profiles may be sufficient to characterize the foundation response. In this case, conventional bearing capacity or stiffness solutions for uniform conditions, or conditions with stiffness and strength increasing with depth may be adequate. When this is not the case, or when it is difficult to be confident that a simplified soil model adequately captures all aspects of the foundation response (either for ultimate resistance or for stiffness), FEA methods may be adopted to model the effects of soil layering.

Modelling complex soil conditions with FEA (see Section 8.9) and obtaining results for ultimate resistance and stiffness can lead to an inadequate understanding of the most important soil parameters or soil layers. In addition, it can mask the limitations associated with geotechnical characterization of the site conditions, particularly at an early stage in the design process.

8.3.2 Drained versus undrained analyses

The designer should assess whether the soils surrounding the caisson are to be considered drained, undrained or partially drained:

a) for the loading cases to be studied;

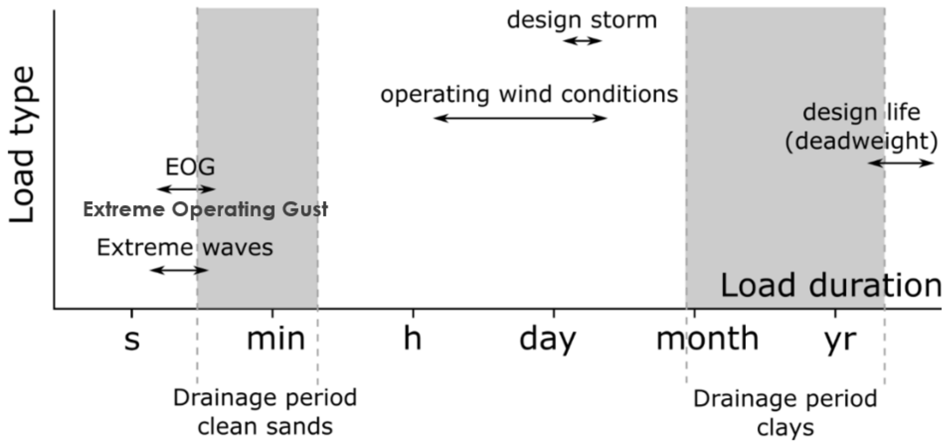
b) throughout the design storm, and

c) during a peak ULS load case.

Figure 8-11 compares load duration and drainage rates for typical soils.

Simplified vertical or radial drainage assessments are recommended at an early stage as illustrated below (or for example Verruit (2006) for more detailed radial drainage solutions).

Figure 8-11: Comparison typical load duration – drainage comparison



A useful concept for simplified assessment is the characteristic drainage period, $T_{\text{char.drain}}$, (de Groot et al., 2006) defined as :

$$T_{char,drain} = A\frac{d^2}{c_v} \qquad \text{Equation 8-9}$$

Where:

c_v = vertical coefficient of consolidation (m^2/s).

A = factor in formula for characteristic drainage period, often taken as 1.0 (-)

d = the drainage path length, (m)

The drainage path length should represent the distance from the zone with excess pore pressure (e.g. SICF tip) to the nearest effective drainage boundary (e.g. seabed or permeable layer), either in vertical or radial direction.

If $T_{char,drain}$ is very large compared to the loading duration, very little drainage will occur but if $T_{char,drain}$ is of the order of the loading duration at least some drainage will occur. $T_{char,drain}$ may be compared to the duration of the peak loading (e.g. 1 wave period) to determine drainage conditions applicable for a peak load analysis. For assessing whether pore pressures due to cyclic loading could accumulate during the most intense period of a storm, $T_{char,drain}$ should be compared to the length of the intense cyclic loading period. Further guidance is provided in de Groot et al. (2006) and Zienkiewicz et al., (1980). Monitoring of a full-scale SICF jacket described by Shonberg et al. (2017) has demonstrated the applicability of assessing drainage rates. Whilst monitoring of a full-scale SICF jacket by Tjelta (2014) indicated that no significant pore pressure accumulated in that case during cyclic loading in sand.

The outcome of these preliminary studies should indicate which type of analysis (undrained or drained) is most appropriate for the loading condition being considered. Calculation methods for undrained or drained conditions are readily available; partially drained approaches would require bespoke assessment.

8.4 Conventional bearing capacity and VHM envelopes

Caisson foundations, given their geometry, are not easily treated as shallow foundations nor as deep (piled) foundations (see ISO 19901-4 (2016), DNVGL-RP-C212 (2017)); they are classified as intermediate foundations.

For conceptual design, classical bearing capacity methods for shallow foundations (e.g. DNVGL-RP-C212 (2017), Vesic (1975)) may be adapted. However, the effect of embedment (typically $L/D > 0.5$) and the strong coupling between the horizontal and moment resistance makes their application not straightforward. Nevertheless, ways by which these methods may be adapted to the determination of ultimate capacity of caissons are outlined in Appendix C.

Another approach is to define the ultimate resistance of a SICF directly in the form of **VHM failure envelopes**. Conceptual VHM envelopes for shallow skirted foundations under drained and undrained loading are shown in Figure 8-12 (a), in which, typically, a rotated parabolic ellipsoid represents the locus of VHM loads at failure. In the *MH* plane this manifests as a rotated ellipse, whilst a parabola is applicable to planes along the *V* axis at ratios of constant M/H.

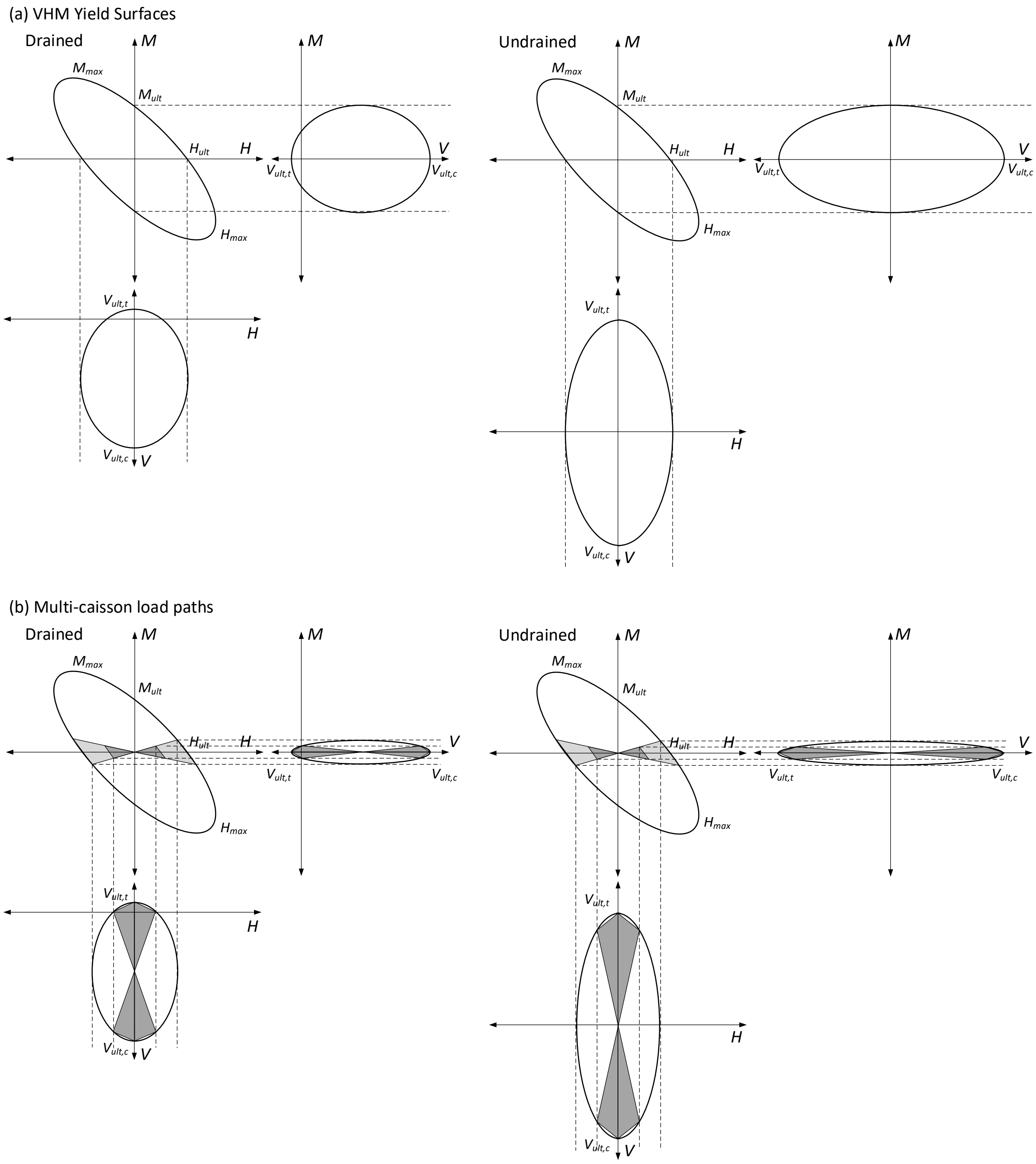
There is a long and extensive literature behind these methods, which involve both drained and undrained loading, surface foundations and embedded foundations (e.g. SICF, spud-cans for jack-up vessels). Such concepts applied to skirted or caisson foundations for **undrained loading** can be found in Bransby and Randolph (1998), Taiebat and Carter (2000), Taiebat and Carter (2005), Gourvenec (2008), Palix et al. (2011), Kay and Palix (2011), Randolph and Gourvenec (2011), Gourvenec and Barnett (2011), Skau et al. (2018) and others. The more recent formulations by Vulpe (2015) (L/D 0.1 to 0.5) and Karapiperis and Gerolymos (2014) (L/D 1 to 3) are probably most relevant.

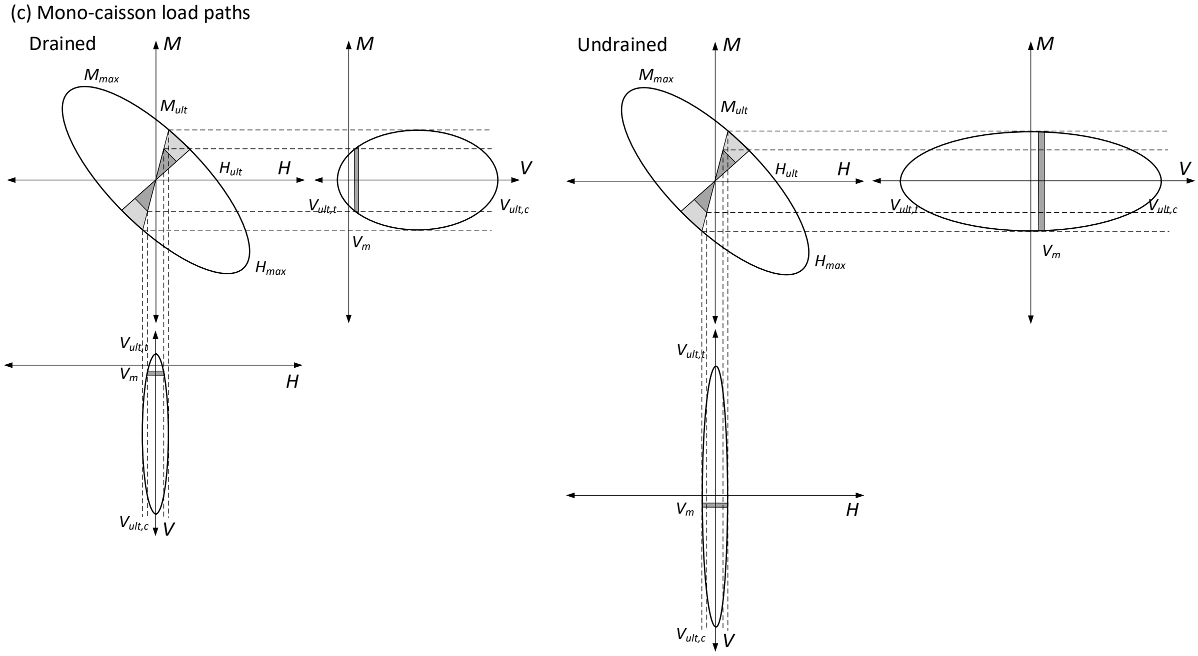
For **drained loading** there have been fewer studies, possibly as it is not easily suited to numerical studies. The work that has been undertaken focuses on experimental work, such as that completed by Byrne (2000)or Villalobos et al. (2009), as well as Kelly et al. (2006). These studies have shown that, for suction caisson foundations under drained loading, the VHM yield surface has a positive intercept at zero vertical load, due to the embedment of the caisson in the ground.

Figure 8-12 (b) and Figure 8-12 (c) show illustrative load paths for the multi-caisson and the mono-caisson structures, respectively, indicating that only a very small part of the VHM load space is actually relevant for design. The dimensions of the failure envelopes are often defined by the uniaxial vertical (compression and tension), horizontal and moment capacities ($V_{\mathrm{ult,c}}$, $V_{\mathrm{ult,t}}$, H_{ult}, M_{ult}, respectively). For the multi-caisson design the vertical capacities are most applicable, whilst for relevant L/D ratios the yield surface for mono-caisson design can be approximated by a plane. It should be noted that the vertical tension capacity ($V_{\mathrm{ult,t}}$) may be limited by cavitation, particularly in shallow water depths, for stiff clay and for dense sand. Many numerical studies for undrained loading have not considered how this limit may impact on the defined VHM yield surfaces, and so caution needs to be exercised in applying the results of numerical studies from the literature. In addition, whilst there has been extensive experimental verification of these yield surfaces for vertical compressive loading there are very few experimental studies that explore combined loading in tension.

These VHM formulations may be used in isolation as a failure envelope to allow ULS design of the foundation for the extreme loads. However, they also provide a consistent framework for integrated structure-foundation load-displacement response using an elastic or non-linear elastic stiffness matrix inside the VHM yield surface as well as collapse analysis as discussed in Section 8.9.

Figure 8-12: Indicative VHM failure envelopes and load paths





8.5 Simplifying design assumptions

Several simplifying assumptions may be needed during design which could affect the calculation methods. These include:

- Soil plug stability: Can the soil inside the caisson be treated as a solid, rigid plug, so that the caisson can be treated as a solid embedded cylindrical foundation? This assumption clearly depends on the effective filling of any voids between the top of the soil plug and the caisson lid. Care needs to be exercised for moment loading calculations in soils of strength increasing with depth, in which a possible failure mechanism can be partially within the caisson skirt (e.g. see (Mana et al., 2013)).
- Installation effects: How will soil disturbance and potential loosening during installation be considered? Further discussion is included in Section 4.7.
- Cyclic effects: How will cyclic induced pore water pressures (if any) or strength degradation (if any) be incorporated into the design calculations? This is discussed in more detail in Section 8.6.1.7.
- Drainage paths in cohesive soils: The potential for preferential drainage paths due to fissured clays, soil-skirt gapping, or unfilled boreholes or CPT holes should be carefully considered and accounted for as necessary.
- Other considerations of lesser importance but still requiring consideration:
    - Caisson top valve(s): Will they be open or closed (vented or unvented) during the structure lifetime? Open vents may enable more rapid drainage and consolidation strength increases following cyclic loading but may also affect reverse end bearing capacity.
    - Effect of torsion: is the SICF likely to experience any significant torsional loads which need to be considered in design? Generic studies such as Taiebat and Carter (2005) allow the effect of torsional loading on the other components of soil resistance to be

assessed quickly. A decision can then be based on the potential impact of the torsional component.

These issues need to be evaluated by the geotechnical specialists in collaboration with the design team to ensure the geotechnical design assumptions are understood by all parties.

8.6 Ultimate resistance

Ultimate resistance calculations are appropriate for assessment of the Ultimate Limit State (ULS). This section outlines the principles of calculation for simplified soil profiles only. Methods for assessing the ultimate resistances are presented at the Load Reference Point (Section 5.2.2).

If the soil resistance is calculated at skirt tip level, the resistance components from side resistance, side friction, and the weight of the soil plug must be accounted for. In Appendix C a method for accounting for all the relevant terms is described.

8.6.1 Multi-caisson structures

8.6.1.1 Soil-structure interaction

Multi-caisson foundation systems are statically indeterminate, so the distribution of loads between caissons must be estimated either by making simplifying assumptions or by more detailed numerical analysis of the structure/foundation system.

Where simplified analyses are employed it is common to assume that only very small moment loads are transmitted to individual footings. Subject to this assumption, the vertical load on each individual footing can be determined for the cases of:

a) a three-legged platform subjected to loading in the direction of any of its axes of symmetry, or

b) a four-legged platform subjected to loading parallel to one of its sides.

For four-legged jacket platforms the diagonal loading is generally the governing case and additional assumptions are necessary (see Section 8.6.1.6 below).

8.6.1.2 Combined loading – bearing capacity approach (compression)

At all design stages, it will usually be necessary to consider the capacity for combined (V, H and M) loading. Appropriate calculation methods to illustrate the principles for homogeneous soils (single layer) and suitable for conceptual design are given in Appendix C.

For multi-caisson structures, where the structure only permits very small rotations of an individual footing, the moments developed on them may not be significant as far as ultimate capacity is concerned. The ultimate moment capacity of the entire foundation assembly can be deduced from simple statics and the ultimate vertical capacities (tension and compression including any horizontal load effects) of the individual footings, if the moment arises from “push-pull” action between downwind and upwind footings.

8.6.1.3 Combined loading – bearing capacity approach (tension)

In undrained vertical tension loading the ultimate tensile capacity may be estimated by applying a “reverse bearing capacity” calculation. Appropriate calculation methods are detailed in Appendix C. However, as discussed in Section 8.5, the potential for preferential flow paths which could result in drained behaviour should be considered. Mobilisation of reverse bearing capacity requires much larger deformations than for mobilisation of skirt friction (Bienen et al., 2018b).

To assess the appropriate tensile resistance to be considered for design a number of factors much be considered. These include the average load (compressive or tensile), the peak cyclic load, the number and duration of the tensile cycles, and the drainage rates applicable in the soil (see (Bienen et al., 2018c)). A site-specific understanding of the characteristics of both the loading and the resistance is essential to any design which involves uplift loads.

The potential for significant uplift displacement and reduction in stiffness should be carefully considered if tensile loads exceed the skirt frictional capacity. Several experimental studies at model and reduced field scale have indicated that, whilst ultimate tensile capacities of SICFs tends to be high, substantial displacements are required to mobilise these capacities. If displacements are to be kept small (as may be dictated by serviceability requirements), it has been observed that tensile loads should either not be permitted at all, or at least limited to the drained pull-out capacity of the caisson, i.e. the friction developed on the inside and outside of the caisson skirt.

8.6.1.4 Horizontal loading (sliding)

A simplified estimate of the ultimate horizontal capacity may be calculated by summing two components: the difference between active and passive earth pressures on the vertical faces of the caisson (accounting if necessary for the possibility of gap formation), multiplied by the projected horizontal area and the shear capacity of the base of the caisson. Appropriate calculation methods are given in 0. However, it should be noted that these methods do not consider 3D effects such as side shear of the SICF. In clay soils, alternatively the horizontal capacity can be defined by a lateral bearing factor (Randolph and Houlsby, 1984), (Kay and Palix, 2011)).

The potential for preferential sliding failure within softer/weaker soils below the skirt tip should be checked in layered soils. However, it should be noted that horizontal sliding failure is generally not a significant issue for offshore wind turbine foundations.

8.6.1.5 Combined loading – simplified VHM approach

An alternative to using the bearing capacity theories is to specify the combined loading VHM surface directly. For example Byrne (2000) explores VHM envelopes for flat footings and caissons in dense sand. Given that, for a multi-caisson structure the vertical loading conditions are dominant, the forces developed on the sides of the caisson may only make a minor contribution to the ultimate moment resistance. The results by Byrne (2000) suggest the yield surface for sand, without any allowance for tensile capacity, could be written as:

$$\sqrt{\left(\frac{H}{h_0 V_0}\right)^2 + \left(\frac{M}{m_0 D V_0}\right)^2 - \frac{2aHM}{h_0 m_0 D V_0^2}} - 4\frac{V}{V_0}\left(1 - \frac{V}{V_0}\right) = 0 \qquad \text{Equation 8-10}$$

Where:

$h_0 = 0.11$

$m_0 = 0.08$

$a =$ parameter defining eccentricity of yield surface. Byrne (2000) reports values in the range -0.3 to -0.8.

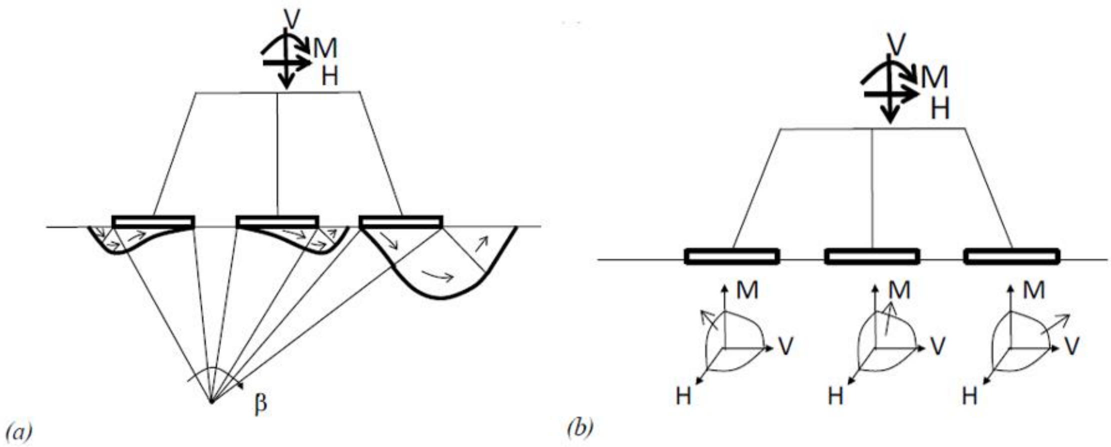
$V_0 = V_{ult,c}$ is the ultimate capacity under pure vertical loading as calculated in Appendix C.

More extensive formulations, and parameter values, for drained and undrained loading as discussed in Section 8.4. This includes VHM envelopes allowing for tensile capacity e.g. Villalobos et al (2009). The more recent formulations by Vulpe (2015) and Karapiperis and Gerolymos (2014) are probably most relevant for undrained VHM envelopes.

8.6.1.6 Foundation group effects

For ULS conditions, all foundations must be in a collapse state at the same time although the mechanisms will be different. Murff (2012) discusses this condition which is illustrated in Figure 8-13 and further discussion can be found in Bransby and Martin (1999) and Kim et al. (2014). However, for a tripod structure loaded on one of its axes of symmetry, or for a tetrapod structure loaded parallel to one side, ULS capacity will normally be governed by the capacity of any one caisson, loaded in either compression of tension: in other words, overall failure is governed by first failure, with only minor effects of load redistribution.

Figure 8-13: Kinematic constraints on failure mechanisms for multi-footing structures



Ref: Murff (2012) *McClelland lecture*

Multi-caisson foundations would rarely involve caissons at a sufficiently close spacing that group effects would result in lower capacity than that of the individual footings. However, the potential for group effects should always be checked. For very closely spaced footings detailed assessments should be undertaken to determine the grouping effects, especially for ultimate horizontal capacity.

The group effect can be evaluated initially by considering the extent of the failure zone for an individual SICF and determining if overlapping with another SICF failure zone is likely. If there is some doubt, finite element analysis is required to determine the reduction in ultimate capacity due to SICF interaction.

8.6.1.7 Multi-caisson cyclic loading effects

The effects of repeated loading (cycling) are usually considered by applying reduction factors on the strength values employed in ultimate limit state calculations. There is extensive discussion regarding cyclic loading in DNVGL-RP-C212, (2017).

Alternatively, the effects of cyclic loading may be treated variously according to the level of detail required:

- Macro modelling in which the nearness of the load vector to the yield surface defines the amount of degradation (pore pressure increases or permanent plastic strain). This approach is analogous to the approach used for axial pile design for cyclic loading (e.g. Jardine et al. (2005)).
- Finite element analysis with stress zoning – FE analyses are used to define shear stress ratios within the model thus allowing for the effect of various load levels to be assessed. Zones are defined based on the average and cyclic shear stress ratios and soil properties such as stiffness and strength are degraded accordingly as discussed in DNVGL-RP-C212 (2017) and Andersen (2015).
- Finite element analysis with cyclic accumulation modelling in which packets of cyclic loads are applied with accumulation of pore water pressure or strain are discussed in Versteele et al. (2013) and Skau and Jostad (2014).

Numerical methods used to analyse cyclic loading effects should be validated and benchmarked against relevant small-scale and field test results, particularly for sands and/or involving tensile loading. They should be used with an awareness of the simplifications inherent is such analyses and sensitivity studies are recommended.

Finite element calculations using soil material models that are based on strain and/or pore pressure accumulation have been used for design and have, to some extent, been validated through tests for Gravity Base Structures (GBS) in soft clay. For project specific applications the main uncertainty relates to reliable relevant cyclic laboratory tests and tests related to consolidation (permeability and coefficient of consolidation). Uncertainties have to be accounted for by conservative selection of parameters as discussed in Section 4.

A key factor in the application of any of these methods is consideration of the drainage which may occur during a storm. Failure to account for drainage and (possible) densification may result in excessive conservatism in the cyclic design analysis.

A number of experimental programs have explored the response of SICFs to vertical cyclic loading, particularly in sand soils, including under tensile loading and also under compressive loading (for example Byrne (2000), Kelly et al. (2006) & Bienen et al. (2018b). The key observations from these studies indicate the following:

- Purely compressive loading sequences lead to settlement, as might be expected.
- The stiffness of the caisson response increases with the average applied load.
- Net settlement occurs even when cyclic loading into tension is applied, if the average stress is sufficiently compressive.
- A change in stiffness of response occurs as the caisson moves from compression to tension during cycling with significant flexibility of response in tension. Selection of a representative stiffness for a caisson cycling into tension appears to be complex.
- Zero or tensile average stresses are not tolerable resulting in significant and ongoing upward displacement of the caisson, even when the loading is within the frictional capacity of the caisson. This is not likely to be relevant to fixed bottom structures but may be of concern to moorings for floating structures

Therefore, as discussed in Appendix C, although significant capacities are achievable for one-off tensile loads, significant caution should be exercised in designing a SICF for cycling out of compression and into tension for anything other than the ULS condition. As field experience of installed caissons is developed then it may be possible to be less cautious.

When evaluating the effects of cyclic loading, scenarios induced only by the application of load and resistance factors (for example apparent cycling into tension when using factored ULS loads and resistances) should be reviewed for applicability.

8.6.2 Mono-caissons

8.6.2.1 Vertical loading: compression

Mono-caissons will invariably be subjected to vertical compression, principally from the weight of the structure. The ultimate loading in compression may be calculated using the same procedures as in Section 8.6.1.2 and Appendix C. It is not necessary to address vertical tension.

8.6.2.2 Horizontal loading

The ultimate horizontal loading for the mono caisson may be calculated using the same procedures as in Section 8.6.1.4 and Appendix C.

8.6.2.3 Moment loading

Moment capacity may be assessed using bearing capacity methods combined with the effective area concept, see Appendix C.

Alternatively, for drained conditions (dense sand), and assuming that the forces developed on the sides of the caisson only make a minor contribution to the ultimate moment resistance, the approach in Section 8.6.1.5 can be adopted.

For undrained conditions, with uniform shear strength and full attachment, Gourvenec, (2007) suggests:

$$M_{ult} = N_{cm} A\, D\, s_u \qquad \text{Equation 8-11}$$

Where $N_{cm} = 0.67$ and

A is the area of the mono-caisson

The uniaxial loading cases cited above (i.e. with V = H = 0) are not of practical use unless the VHM envelope approach is to be used.

8.6.2.4 Combined loading

Combined loading may be treated using the methods described in Section 8.6.1.5. Given the typical form of the yield surface in the positive loading quadrant it is possible to approximate the surface by a plane, rather than a full three-dimensional surface, and which may be simpler to calibrate either via numerical modelling or experimental data. For example for drained capacity in loose sand Byrne and Houlsby (2003) suggest the following simplified calculation, based on model scale tests with $L/D = 0.5$:

$$M_{ult} = \frac{D}{f_1 + \frac{f_2}{k}}(V + f_3 W) \qquad \text{Equation 8-12}$$

Where:

$$k = \frac{M}{DH}$$

$$W = \gamma' \frac{\pi D^2 L}{4}$$

$$f_1 = 3\,;\;\; f_2 = 1 \; and \; f_3 = 0.65$$

Note that this calculation implies that the weight of the soil plug is only partially mobilised under the action of the loading.

A similar approach can be adopted for a clay soil, with numerical modelling used to calibrate the surface; a minimum of three calculations defining VHM loads at failure, are needed to identify the plane (assuming the points are not collinear).

8.6.2.5 Mono-caisson cyclic loading effects

The effects of cyclic / repeated loading may be treated as described in Section 8.6.1.7 and as also discussed in Section 8.8.

8.7 Stiffness

8.7.1 Small-strain shear modulus

Stiffness calculations adopting a shear modulus representative of relatively small strains are appropriate for assessment of the Fatigue Limit State (FLS) and Serviceability Limit State (SLS).

At sufficiently small deformations soil may be characterized by a small-strain shear modulus G_0 and Poisson's ratio (ν). The small strain Young's modulus (E_0) and bulk modulus (K_0) are related to these by:

$$E_0 = 2(1+\nu)G_0 \qquad \text{Equation 8-13}$$

$$K_0 = \frac{2(1+\nu)}{3(1-2\nu)}G_0 \qquad \text{Equation 8-14}$$

For undrained clays ν = 0.5 (usually considered as 0.49 for the purposes of numerical analysis) and for drained behaviour a typical value of Poisson's ratio is ν = 0.2.

The small-strain shear modulus (G_0) may be measured or estimated. The modulus can be measured directly by, for instance, resonant column testing, or through measurements of the shear wave velocity in-situ.

The small-strain modulus may be estimated as follows:

- For clays the small strain shear modulus (G_0) is usually estimated from the undrained shear strength by Equation 8-15 with values of the rigidity index I_r typically in the range 500-2000 or potentially lower in the case of highly plastic clays (Andersen, 2015).

$$\frac{G_0}{s_u} = I_r \qquad \text{Equation 8-15}$$

- For sands the modulus (G_0) may be estimated by a formula such as:

$$\frac{G_0}{p_a} = I_s \left(\frac{p'}{p_a}\right)^n \qquad \text{Equation 8-16}$$

Where:

p_a	=	is a reference pressure (usually atmospheric pressure, (p_a=100kPa);
p'	=	is a representative mean effective stress beneath the foundation;
n	=	is an exponent approximately equal to 0.5;
I_s	=	is a stiffness factor typically in the range 300-1500.

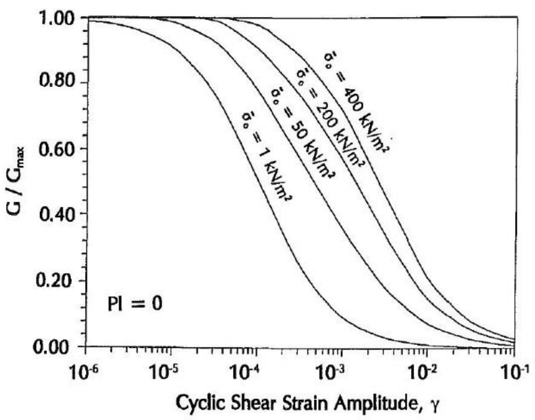
Further discussion can be found in, for instance, Benz (2007).

8.7.2 Modulus reduction with strain

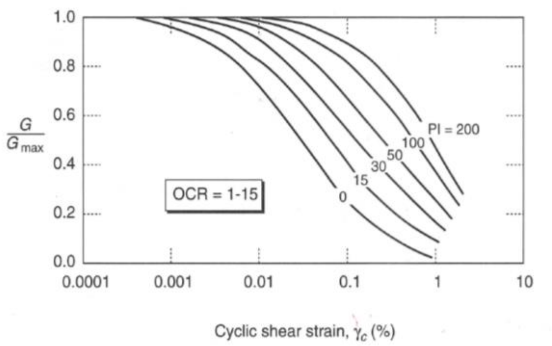
As discussed in Section 8.1, the response of the SICF must normally be linearized at a specific load level due to the non-linear stress-strain response of the soil, to obtain the correct stiffness at the required load level. This requires selection of appropriate modulus reduction factors according to the type of analysis being performed i.e. for ULS/ALS or SLS analyses.

The non-linear response of a soil element is obtained by static, dynamic and cyclic tests on soil samples. Figure 8-14 shows examples of modulus reduction curves (G/G_{max} v shear strain). Curves such as these should be carefully selected from the database of available literature (for instance Ishihara (1995)) and site-specific laboratory tests.

Figure 8-14: Example shear modulus reduction with shear strain



a) Cohesionless soils (Ishibashi, 1992)



b) Cohesive soils (Kramer, 1996)

Soil element non-linear response may be introduced into simplified analyses by considering average operation levels of shear strain as a function of the distance of the load to the failure envelope.

One simplified industry approach which has been accepted considers the modulus applicable to a strain corresponding to 50% of the peak stress (ε_{50}) for ULS analyses.

Modulus reduction relationships are often available in standard FE packages.

8.7.3 Vertical, horizontal and rotational stiffness

The elastic response of a caisson, treated as a rigid cylindrical foundation, may be represented by the matrix equation (considering a 2D formulation):

$$\begin{bmatrix} V/G_0D^2 \\ H/G_0D^2 \\ M/G_0D^3 \end{bmatrix} = \begin{bmatrix} K_V & 0 & 0 \\ 0 & K_H & K_C \\ 0 & K_C & K_M \end{bmatrix} \begin{bmatrix} w/D \\ u/D \\ \theta \end{bmatrix}$$ Equation 8-17

Where:

V, H, M = vertical, horizontal and moment loading

G_0 = small-strain shear modulus

D = caisson outer diameter

K_V, K_H, K_M, K_C = vertical, horizontal, moment and coupling stiffness, respectively

w, u, θ = vertical, horizontal and rotational deformation, respectively

The load and displacement terms and sign convention are defined in Section 2.2, and K_V, K_H, K_M, K_C are dimensionless factors. In the definition of these factors the position of the reference point for loads and displacements should be carefully considered and accounted for.

Values of the dimensionless stiffness factors for rigid caissons are found in Bell (1991), Ngo-Tran (1996), and Doherty and Deeks (2003). Other solutions for general shaped embedded foundations are provided by Gazetas (1991). Extension to a 3D form as required for structural analysis (see Section 5.3) is straightforward and only requires a torsional stiffness term to be added (see DNVGL-RP-C212 (2017); Suryasentana et al. (2017)).

In the above the caisson itself is assumed to act as a rigid unit. A method for accounting for the flexibility of the skirt wall is described by Doherty et al. (2005).

These solutions assume full top plate contact with the soil and do not allow for the top plate stiffness, which is likely to be important as discussed in Shonberg et al. (2017).

These solutions do not account for the potential reduction in stiffness if the net uplift load is tensile or if partial drainage occurs (Bienen et al., 2018a).

Foundation group effects for multi-caisson structures should also be evaluated. Elastic theory is likely to provide a conservative estimate of the interaction between two SICFs.

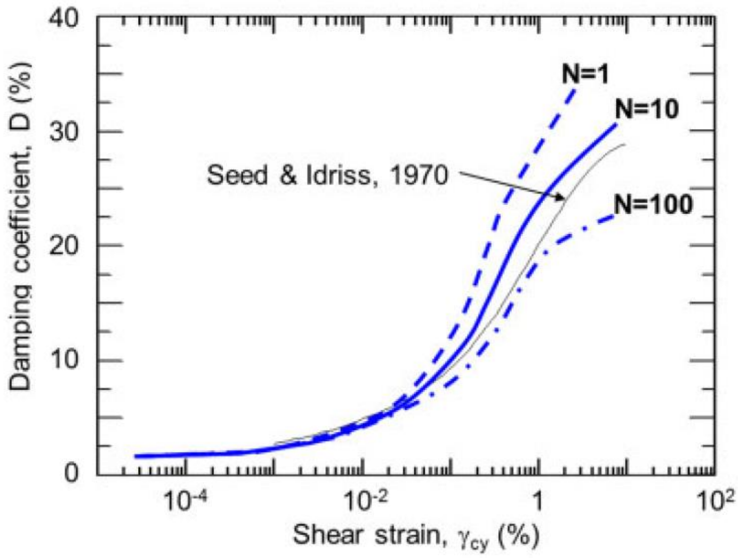
Foundation stiffness is not necessarily constant with time due to the effect of cyclic loading on soil properties. Experimental data available to date suggests that an increase in stiffness in sand may be anticipated and was found to be independent of relative density for monopiles (LeBlanc et al., 2010). However, Zhu et al. (2013) showed no significant changes in the moment stiffness of a caisson in sand.

8.7.4 Damping

Soils dissipate energy when subject to cyclic loading due to inter-particle friction, structure rearrangement, and pore fluid viscosity. The magnitude of the energy loss is known as the internal or material damping, and in soils it is dependent on many factors including the strain amplitude and the frequency or velocity. While friction and soil structure related damping are strain-dependent, the effect of pore fluid viscosity is frequency dependent as it is controlled by the permeability of the soil. Material damping is discussed in detail in (Kramer, 1996).

An example of the dependency of material damping on the cyclic strain intensity is shown in Figure 8-15 for clays. For sands, the damping ratio is also dependent on the void ratio and the confining pressure.

Figure 8-15: Example of dependency of damping ratio on cyclic shear strain



Ref: Andersen (2015) - clays

Correlations between damping ratio and G/G_{max} have been suggested by Ishibashi and Zhang, (1993), Ishihara (1995)and Zhang et al. (2005).

For SICFs, there will be large variations in material damping around the caisson due to the variations in shear strain, and localized pore water pressure variations and associated viscous damping.

Specific analyses would be required to develop the global damping ratio to be considered associated with a given load intensity.

Byrne (2000) and Skau et al. (2018) identifies that caisson response under small numbers of moment (and horizontal) loading cycles exhibited “Masing” type behavior, so that the response at macro level could be represented by a kinematic hardening model.

8.7.5 Layered profiles

Layered soil profiles will normally require FE analyses to obtain an accurate assessment of the components of the SICF stiffness matrix (Equation 8-17).

8.8 Permanent deformations

8.8.1 General

Permanent deformation of SICFs may arise from a combination of the following components which all need to be considered:

- Elastic settlement
- Consolidation settlement
- Creep settlement
- Shakedown (short-term densification and load redistribution)
- Settlements due to repeated loading (long-term densification)

Serviceability (SLS) analysis includes demonstrating that consolidation and cyclic loading does not cause excessive settlement or rotation of the structure. Evaluation of future permanent displacements of a structure is therefore an essential element of SLS assessment, but methods are currently not fully developed and may be unreliable. However, from the limited experience of structures which have been installed and monitored, the performance does not give cause for concern. Some of the publicly available field data is summarised in Appendix B. Some preliminary guidelines are proposed in this document.

Permanent deformations of a SICF will depend both on the permanent load (for settlements), and on the characteristics of the environmental loading (potentially causing differential settlements or tilt).

8.8.2 Permanent vertical deformation (settlement)

Permanent vertical settlement could occur for a variety of reasons as discussed above. Lateral variations in soil properties across a multi-caisson structure should be considered as they could give rise to differential settlements and permanent tilt.

Elastic, consolidation and creep settlement may be assessed by conventional methods applicable to shallow foundations (ISO 19901-4 (2016), DNVGL-RP-C212 (2017)) and foundation engineering text books. Shakedown and other settlements caused by repeated loading on SICFs are discussed in this section.

In the context of SICFs, shakedown is the term applied to the initial “bedding in” of the caisson following installation when subject to the initial significant environmental loads. Suction installation causes some disturbance of the soil and, immediately after installation, the long-term distribution of load on the skirts and top plate is not yet established. The first storm loading causes local stress redistribution and local densification which could result in “shakedown” settlement. In the absence of documented case histories, it is proposed the shakedown settlement be assessed by considering the effect of load redistribution and local densification of the soil inside the skirts. Shakedown may be mitigated by underbase filling.

Settlement due to repeated loading may be assessed in various ways:

- In sands, cyclic induced settlement can be assessed by considering the intensity of cyclic loading in each supporting layer (inside and below the caisson) and the resulting volume decrease with numbers of cycles assuming drained conditions. Empirical methods based on limited laboratory data may also be used to inform the settlement estimates (e.g. Zhu et al., 2013) but the limitations of such methods must also be considered.

- In clays, by assessing the excess pore pressures induced by cyclic loading and then considering the additional settlement when drainage occurs (using the reloading modulus).
- By more complex numerical methods which model the effects of cyclic loading element by element (Versteele et al., (2013); Lupea, (2013), Jostad et al., 2014; Jostad and Andresen, 2009; Skau and Jostad, 2014). However, the reliability of these methods is not yet generally demonstrated, and they require validation and benchmarking.

8.8.3 Permanent rotation (tilt)

Permanent rotation of multi-suction caisson structures (global rotation) should be assessed by considering differential settlements as discussed in Section 8.8.2 including the effects of interaction between the individual SICF s if relevant.

For mono-caisson structures, permanent tilt due to accumulated rotation of the SICF during cyclic loading is a design case which should be investigated. The work by Zhu et al (2013) and Zhu et al (2017) demonstrates that the accumulated angular rotation follows a power law relationship with the number of cycles.

Studies at laboratory scale both at 1g and in a centrifuge, indicate that permanent rotation does accumulate with cyclic amplitude and the greatest accumulation occurs for a cyclic regime between one-way and two-way loading (Zhu et al. (2013); (Cox and Bhattacharya, 2016). In drained sand conditions, there is an initial rapid accumulation of rotation which slows with additional loading as found for monopiles (LeBlanc et al. (2010); Foglia et al, 2014).

There is some evidence from cyclic element testing that there is a threshold strain level below which accumulated deformation does not occur (Vucetic (1994); Mortezaie and Vucetic, (2016)). Nielsen et al., (2017)) report 1g laboratory results for cyclic loading of SICFs where rotation was not measurable at low cyclic load levels. Kim et al (2014) also provides relevant test data. Further investigation is required before firm guidelines can be developed for defining a threshold load level.

No change in stiffness with cycle number is observed. This is further reinforced in work by Cox and Bhattacharya (2016).

8.9 Advanced modelling approaches

8.9.1 Finite element analysis

Finite Element Analysis (FEA) is widely used for detailed design of SICFs. Compared to the bearing capacity analysis discussed above, best-practice FEA can provide a more accurate model for foundation capacity in complex soil profiles, and 3D packages automatically account for geometrical effects. FEA can also be used to account for consolidation effects and other time or rate-dependent behavior. Advanced FEA is likely to take more time and require special expertise and is therefore more suitable for detailed design stages and verification of other methods. Benchmarking and validation of advanced models is generally required.

There are a range of FEA software packages and soil constitutive models that are available. Some features that may be needed include:

- Strain and pressure dependent soil stiffness
- Soil strength anisotropy and cyclic degradation effects
- Interface elements to model interface friction and set-up
- Limiting tension criteria to simulate cracking or cavitation

The most appropriate package, soil model types, soil layering, engineering parameters and modelling should be selected by a senior geotechnical engineer experienced with FEA. Advanced numerical analysis should always be performed by a geotechnical engineer experienced with numerical analysis and aware of the behavior of the soil models being used and the limitations of the analysis method.

8.9.2 VHM envelopes and the use of VHM approach

Generally, it will be necessary to adopt more advanced techniques to define the yield surfaces, particularly for mono-caisson designs. Most sites consist of layered and variable soil conditions for which the standard calculations for combined loading may be difficult to apply without undue conservatism.

The basic approach for defining VHM envelopes would follow this process:

1) Define the maximum resistances (V_{max}, H_{max}, M_{max}) or the uniaxial ultimate resistances (V_{ult}, H_{ult}, M_{ult}). Simplified bearing capacity, limit equilibrium, plasticity, or finite element analysis may be used.
2) Define the shape of the envelope covering the load range applicable to the SICF. Existing generic studies defining the shape of the failure envelope may be used where the applicability is confirmed. Alternatively, specific analyses using limit equilibrium, plasticity, or finite element analysis may be used.

The adoption of the VHM approach includes the following benefits:

1) All design stages (from conceptual to final design) may be accommodated by incremental improvement of the VHM envelope as further data or more sophisticated analyse become available (improving reliability and removing conservatisms)
2) All SICF design load cases can be assessed and presented using a single VHM envelope and the change in an individual action component on the proximity to the yield surface is indicated;
3) Finite element analysis may be adopted at any stage to improve the VHM envelope definition;
4) For multi-caisson structures, the VHM surface may be considered as part of an elasto-plastic macro-element model and combined with a flow rule to facilitate SICF displacement response as a function of load level for push-over analysis (see for example Bransby and Martin (1999));
5) Using a VHM framework in the form of a macro-element for both non-linear stiffness (as a function of proximity to the yield surface) and failure naturally allows integrated foundation and structural design analysis, as discussed in Section 5.5.

8.9.3 Macro-element modelling

An approach which is still mostly in the research stage consists in modelling the SICF response using a macro-element or force-resultant model. However, because it holds promise for use with integrated structural and foundation design and is likely to become more widely used in the future, it is introduced in this document.

A full elasto-plastic macro-element model requires definition of:

a) the yield surface (VHMT envelope),

b) elastic or non-linear response within the yield surface (see Section 8.7),

c) the hardening behaviour (often linked with the vertical penetration response in applications to jack-up spud cans), and,

d) a flow rule to determine the incremental plastic displacements at failure.

Full implementations of such models (for application to jack-ups) can be found in Martin (1994) or Cassidy (1999) and extensions to full 6 degree of freedom loading in Bienen et al. (2007), albeit for surface foundations. The implementation for a caisson foundation in soft clay can be found in Cassidy et al. (2006). Note that extension of yield envelopes into a full elasto-plastic macro-element may require some simplification of the yield function expressions to ensure numerical robustness (e.g. to avoid singularities, to ensure convexity, etc).

Other relevant references providing further information include: Nguyen-Sy and Houlsby (2005), Heron et al. (2015) and Skau et al. (2018).

8.10 Other considerations relating to in-service performance

8.10.1 Scour and scour protection

In-service performance will be affected by local and global scour if it occurs. Scour evaluations and scour protection requirements are essential elements of SICF design requirements.

For SICFs in which scour protection is part of the foundation design, the integrity of the protection measures should be confirmed at intervals to be determined according to the project requirements.

If scour protection is not part of the design, the design must take account of scour which could develop over the lifetime of the structure. Surveys should be undertaken to confirm that any scour that does occur remains within the design limits, and to allow remedial works to be undertaken in due time if required.

There are some general recommendations regarding scour assessment in DNVGL-ST-0126 (2016) with more detailed discussion in Whitehouse (2004). The risks of self-induced scour or piping around skirts due to high hydraulic gradients induced by cyclic loading should also be assessed.

8.10.2 Interaction with jack-up spudcans

Jack up spudcans can have a large lateral area and can penetrate a significant depth below seabed level, resulting in significant seabed disturbance in surrounding soils. Current jack up guidance (ISO 19905-1, 2016) (SNAME, 2008) recommends that interaction effects should be considered when structures are within 1 equivalent diameter. For SICFs the offset between the edge of the spud can and edge of the SIFCs could be significantly less than this.

Jack-up operations near to SICFs may affect the properties of the soils supporting the SICF and therefore the available SICF capacity/stiffness response could be affected or spudcans could potentially inducing deformations directly. These effects should be assessed from an early project stage to ensure that SICFs are viable and constructible with available vessels.

8.11 Observed in-service performance

Only a few field-scale observations on suction caisson foundations are currently available and some results are summarized in Appendix B. Guidance provided in this section is consistent with the current understanding of observed performance.

9. DECOMMISSIONING

The decommissioning of offshore structures is an important consideration, and offshore regulations in the European Union require that it should be considered during the design process, and that the seabed should be returned to its original state after decommissioning.

The extraction of SICFs from the seabed can be achieved by the reverse of the installation procedure detailed in Section 6.1, where water is pumped into the caisson to create an 'overpressure' which pushes the SICF out of the seabed, possibly in parallel with a crane pull.

9.1 Soil properties

When selecting soil parameters for decommissioning, potential ageing effects including consolidation, thixotropy and the long-term densification caused by cyclic loading should be considered:

- Consolidation, thixotropy and ageing could result in a significant increase in the soil strength, potentially to levels higher than the original undisturbed strength in some instances (highly fissured clays for example);
- Cyclic loading, especially small amplitude loading over the operational lifetime could result in an increased adhesion/friction at the skirt wall.

The upper bound estimates for the extraction calculations should be based on the appropriate original in-situ soil and strength profiles (characteristic and/or upper bound) and include an appropriate conservative allowance for ageing.

Since these time-related effects are not well characterised, a wide range of conditions should be considered for decommissioning engineering and consideration should be given to ensure a suitably high soil resistance is adopted.

Structural monitoring and changes to the structure response during the operational lifetime of the structure may also provide input to the assessment of soil improvement with time.

9.2 Extraction pressures

The required extraction overpressures are calculated in the same way as the installation pressures detailed in Section 6.2, where the skirt friction is determined based upon the skirt geometry and soil parameters including the aging effects discussed above.

When an overpressure is applied, the downward hydraulic gradient increases the internal friction on the skirts and this should be taken into account.

As for the installation calculations (Section 6.2), the extraction pressure which can be applied will be limited by:

- Pump capacity (pressure and flow);
- Structural stability of the SICF (shell buckling is generally not an issue for extraction since an overpressure is applied);
- Liquefaction of the soil at the tip of the caisson before the required extraction pressure has been achieved.

9.3 Review of as-built data

When assessing decommissioning after the structure has been built, the as-built data should be reviewed to confirm the original design assumptions and the in-situ soil conditions. Data to be collected and reviewed include the self-weight penetration depth and installation pressures which can be used to inform selection of the most appropriate soil profile and soil parameters, in conjunction with set-up/ageing factors.

Other data such as as-built geometry, in-place weight at time of decommissioning, ability of skirts and base to support overpressure, and grout volumes and properties should also be gathered and reviewed for efficient decommissioning design.

9.4 Practical considerations

Decommissioning should be considered during the original design process to minimise retrospective engineering. Examples where decommissioning requirements could affect the design include:

- Lifting points – if the sub-structure is to be partially dismantled offshore then the SICFs may require lifting points to assist with the extraction and recovery to deck.
- Structural integrity of the caisson and hatches – all vents must be closable and other sources of leakage avoided to develop sufficient overpressure to extract the SICF.
- Underbase grout – the re-use of vents and piping work used for installation should be considered during underbase grouting design to ensure the pressures and flows needed during extraction can be achieved.

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APPENDIX A – GEOTECHNICAL PARAMETERS AND INDICATIVE TESTS

Table A-1: Geotechnical parameters and indicative test types

Geotechnical Parameter	Potential testing methods	
	Cohesive	**Cohesionless**
Soil classification	Visual description Grain size (sieve/hydrometer) Unit weight Water content Atterberg limits PCPT sleeve friction, pore pressure	Visual description Grain size (sieve) Unit weight Water content Carbonate content PCPT sleeve friction, pore pressure
Permeability	PCPT dissipation test BAT probe Permeability test	PCPT dissipation test BAT probe Permeability test
Undrained shear strength (intact and remoulded)	CPT T-bar In-situ vane Triaxial tests (UU & CAU) DSS Fallcone / Labvane / Pocket pen Anisotropy (CAUc, CAUe, DSS) Sensitivity Thixotropy (t_{thix})	Triaxial tests (CAU) DSS
Friction Angle / Relative density	Triaxial (CAU, CD)	CPT Direct shear Triaxial (CID) Ring shear

Geotechnical Parameter	Potential testing methods	
	Cohesive	Cohesionless
Shear modulus & damping (including small strain stiffness and unload/reload stiffness)	Pressuremeter / HPD PS Logging Seismic cone Resonant column Seismic refraction, surface wave	Pressuremeter / HPD PS Logging Seismic cone Resonant column Seismic refraction, surface wave
Stiffness modulus	Triaxial Oedometer	Triaxial Oedometer
Consolidation	PCPT Oedometer	PCPT
Rate effects / cyclic behaviour	T-bar / ball penetrometer Cyclic simple shear Cyclic triaxial testing	Cyclic simple shear Cyclic triaxial testing
Gas content (if required)	BAT probe Geochemical	BAT probe Geochemical

APPENDIX B - OBSERVED IN-SERVICE PERFORMANCE

Only a few field-scale observations on suction caisson foundations are currently publicly available and the key results are summarized in this section. They provide some further insights into the analysis and design methods which are largely based on laboratory scale testing.

Extensive monitoring was performed on the Draupner E jacket, the first platform founded on SICFs (Tjelta, 2014). During the first winter season the jacket encountered a 26 m high "monster wave", which resulted in a 15 MN tension load on the upwind SICF. As a result, an 80 kPa underpressure developed inside the SICF. Hence, an undrained response in tension was observed for a peak load, although the SICFs are installed in sand. At the same time it was found that negligible pore pressure accumulated during the design storm, indicating that drainage in sand can be considered for assessment of cyclic loading.

Field tests on 1.5m and 3.0m diameter SICFs in sand and clay are reported by Houlsby et al. (2005), and Houlsby et al. (2006). An initially stiff moment-rotation response was followed by reducing stiffness and increasing hysteresis with load amplitude. In the sand tests, a slight shakedown was apparent in the initial stages at low amplitude cyclic moment/rotation loading with slight stiffening over several cycles of the same moment magnitude. The moment-rotation response reached a steady state by the end of the cyclic loading packet. Since the rotation of the caisson is approximately proportional to the shear strain amplitude in the soil, the modulus degradation with shear strain found in element tests was also observed in these field tests.

Under vertical cyclic loading in the sand test, the stiffness decreases as the amplitude increases. However, a substantial reduction in stiffness was noted when the caisson was cycled into tension. If the mean vertical load remained compressive, the caisson ratcheted into the sand even if tensile loads were applied.

Liingaard, (2006) reports the dynamic response of a 12m diameter x 6m deep test caisson in sand at Frederikshavn. The natural rocking frequency of the foundation corresponded to the in-situ shear modulus (40-80MPa) estimated from CPT data.

Some data on the response of a 15m diameter x 7.5m skirt length mono caisson located in the Dogger Bank area of the North Sea is presented by Nielsen et al. (2017). During initial shakedown over the first 1-2 months, a small tilt was observed which stabilized. Thereafter, even during a large storm, no further permanent displacement/rotation was observed.

Shonberg et al., (2017) describe extensive instrumentation and monitoring of a tripod structure with suction caisson foundations installed in medium dense to very dense sand. There was a clear but very minor change to the inclination of the structure immediately after installation (0.005°). Since that time the inclination has remained below 0.01° and the environmental loads have had little impact on the overall inclination. Shonberg also noted that modelling the top cap with a realistic flexibility (not rigid) was necessary to capture accurately the vertical stiffness response, and the dynamic pore pressure response carried a significant portion of the dynamic vertical wave loading. Vertical stiffness differences between individual caissons of the tripod was observed due to differences in mean vertical load which increased the mean effective stress and stiffness for the more heavily loaded caissons.

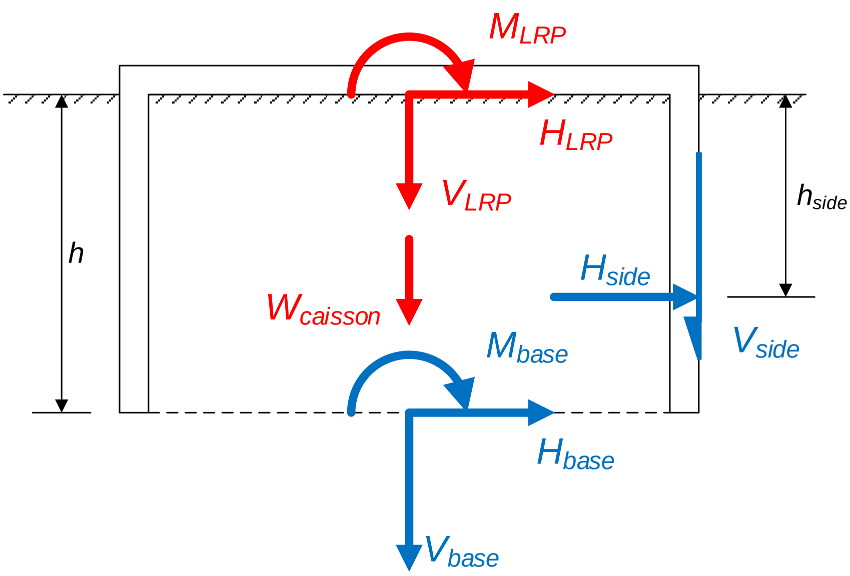
APPENDIX C – SIMPLIFIED CAPACITY METHODS

Introduction

An estimate of the ultimate capacity of a caisson may be determined using the methods described by either Brinch Hansen (1970) or Vesic (1975). Both methods apply shape and depth factors to modify the standard Terzaghi bearing capacity formula. In this document, the Brinch Hansen equations are adopted (see ISO 19901-4, 2016, DNVGL-RP-C212, 2017). The published solutions apply at skirt tip level (*h* below mudline – see Figure C-1).

This section addresses the modifications necessary to convert the solutions so that they refer to the Load Reference Point at mudline level, consistent with the main text of this document. This involves converting the loads (specifically the moment) and making allowance for the contribution of external friction and lateral pressure on the skirt.

Figure C-1 : Load – capacity conversion



*Note: Applied forces from structure at Load Reference Point (red), and resulting forces applied to soil from side and base of caisson (blue)

Figure C-1 shows schematically the forces applied at the Load Reference Point (LRP), the weight of the caisson and trapped soil and the consequent forces on the side and base of the caisson. These are related by the following expressions:

$$V_{LRP} + W_{caisson} = V_{base} + V_{side} \quad \text{Equation C.1}$$

$$H_{LRP} = H_{base} + H_{side} \quad \text{Equation C.2}$$

$$M_{LRP} = M_{base} - h_{side}H_{side} - hH_{base} \quad \text{Equation C.3}$$

The term $W_{caisson}$ should include the (buoyant) weight of the soil trapped in the caisson $\frac{\pi D_i^2 h\gamma'}{4}$, as well as the weight of any part of the caisson structure not already accounted for in V_{LRP}. It should be noted that because of the way the limit equilibrium methods below are formulated, finding a single solution for the allowable loads is rarely possible. Instead, a series of inequalities should be checked in which the available resistance should always be larger than the applied loads in each specific direction. The minimum SICF size that satisfies all inequalities will be the optimal solution.

The ultimate load in compression $V_{ult,c}$ is defined as the value of V_{LRP} for the case $H_{LRP} = M_{LRP} = 0$.

Soil resistance on side of caisson

Caisson Skirt - Vertical Resistance:

In clay (undrained) the vertical load capacity in compression (downward load) on the outside (external surface) of the caisson may be estimated as:

$$V_{side} = (\pi Dh)\alpha s_{u1} \quad \text{Equation C.4}$$

where α is an adhesion factor as used in pile capacity analysis.

In sand (drained) the vertical load capacity in compression (downward load) on the side of the caisson may be estimated as:

$$V_{side} = (\pi Dh)\frac{\gamma' h}{2} K \tan\delta \quad \text{Equation C.5}$$

where $K \tan\delta$ is a friction factor as used in pile capacity analysis.

The skirt resistance is usually taken as the external surface with an appropriate base capacity. However, in some situations the designer may want to consider internal and external resistance and should modify the equations appropriately.

For either the undrained or drained case, it is recommended that dedicated testing is undertaken to determine the α or $K \tan\delta$ value with confidence, to allow for an optimised SICF design.

For vertical load in tension the side and base contributions cannot be treated in separation, and this case is treated in Section C.6.

Caisson Skirt - Horizontal resistance:

For undrained analysis, in all but the softest clay it could be assumed that under ultimate lateral loading conditions a tension crack would open on the active side of the caisson, and the net horizontal capacity (Randolph and Houlsby, 1984) on the passive side of the caisson could be estimated as:

$$H_{side} = (Dh)\left(\frac{\gamma' h}{2} + 2s_{u1}\right) \quad \text{Equation C.6}$$

The position of the effective depth of action of the horizontal load depends on the variation of undrained strength with depth. For instance if the strength is given by $s_u = s_{u0} + \rho z$ then

$$s_{u1} = s_{u0} + \rho\frac{h}{2} \quad \text{Equation C.7}$$

$$H_{side} = (Dh)\left(\frac{\gamma' h}{2} + 2s_{u0} + 2\frac{\rho h}{2}\right) \quad \text{Equation C.8}$$

$$h_{side}H_{side} = (Dh)\left(\frac{\gamma' h^2}{3} + 2s_{u0}\frac{h}{2} + 2\frac{\rho h^2}{3}\right) \quad \text{Equation C.9}$$

In sand, active and passive pressures may be assumed on the sides of the caisson, and the horizontal capacity on the side of the caisson could be estimated as:

$H_{side} = \frac{\gamma' h^2 D}{2}(K_p - K_a)$ Equation C.10

The earth pressure coefficients may be estimated from the standard expressions $K_p = \frac{1}{K_a} = \frac{1+\sin\varphi'}{1-\sin\varphi'}$.

The position of the effective depth of action of the horizontal load may be estimated as

$h_{side} = \frac{2h}{3}$ Equation C.11

More sophisticated estimates of the lateral resistance could be made accounting for three dimensional effects.

Effective area

If applied moment on the base of the caisson is not under consideration this section is not relevant, and the effective area, breadth and length are given simply as:

$A_{eff} = A = \frac{\pi D^2}{4}, B_{eff} = L_{eff} = D.$

In a simplified foundation capacity method using the "effective area" concept, the combination of a vertical load V_{base} and an overturning moment M_{base} is treated as the statically equivalent pure vertical load V_{base} at an eccentricity $e = \frac{M_{base}}{V_{base}}$ from the centreline of the foundation (see DNVGL-RP-C212 for a discussion). The capacity is then calculated for a symmetric foundation with a reduced "effective area", with this vertical load applied at its centre. This procedure is always conservative, as it ignores part of the foundation capacity. The effective area, breadth and length the foundation need to be calculated, and for a circular foundation these are given by:

$A_{eff} = 2\left(\frac{D^2}{4} arccos\left(\frac{2e}{D}\right) - e\sqrt{\frac{D^2}{4} - e^2}\right)$ Equation C.12

Then making use of the intermediate variables $B_e = D - 2e$ and $L_e = \sqrt{D^2 - (D - B_e)^2}$:

$L_{eff} = \sqrt{A_{eff}\frac{L_e}{B_e}}$ Equation C.13

$B_{eff} = L_{eff}\frac{B_e}{L_e} = \sqrt{A_{eff}\frac{B_e}{L_e}}$ Equation C.14

It is noted that for multi-caisson structures, where the structure only permits very small rotations of an individual footing, the moments developed on them may not be significant as far as ultimate capacity is concerned. The ultimate moment capacity of the entire foundation assembly can be deduced from simple statics and the ultimate vertical capacities (tension and compression) of the individual footings, if the moment arises from "push-pull" action between downwind and upwind footings.

Bearing capacity: undrained analysis

For undrained conditions with uniform strength with depth the capacity is given by:

$V_{base} = A_{eff}(N_c s_{u2}(1 + s_{ca} + d_{ca} - i_{ca}) + \gamma' h)$ Equation C.15

where:

N_c is the conventional bearing capacity factor $N_c = 2 + \pi \approx 5.14$

s_{u2} is the undrained shear strength at the foundation base level

s_{ca} is a shape factor defined as:

$$s_{ca} = 0.2(1 - 2 \cdot i_{ca})\frac{B_{eff}}{L_{eff}}$$

d_{ca} is a depth factor defined as:

$$d_{ca} = 0.3\, arctan\left(\frac{h}{B_{eff}}\right)$$

i_{ca} is an inclination factor defined as:

$$i_{ca} = 0.5 - 0.5\sqrt{1 - \frac{H_{base}}{A_{eff}s_{u2}}}$$

In this section if horizontal load is not under consideration the formulae may be used with $i_{ca} = 0$.

For undrained conditions with strength increasing linearly with depth, the method of Davis and Booker, (1973), as detailed in DNVGL-RP-C212 may be used. For purely vertical loading the method by (Houlsby and Martin, 2003) may be more appropriate.

Bearing capacity: drained analysis

For drained conditions (sand) at skirt tip level:

$$V_{base} = A_{eff}\left(\frac{1}{2}\gamma' B_{eff} N_\gamma s_\gamma d_\gamma i_\gamma + \gamma' h N_q s_q d_q i_q\right) \qquad \text{Equation C.16}$$

where:

N_q and N_γ are conventional bearing capacity factors given by:

$$N_q = tan^2\left(\frac{\pi}{4} + \frac{\phi'}{2}\right)e^{\pi \tan\phi'},$$

$N_\gamma = 1.5(N_q - 1)\, tan\,\phi'$, (this being an approximate formula only)

s_q and s_γ are shape factors given by:

$$s_q = 1 + i_q\left(\frac{B'}{L'}\right) sin\,\phi',$$

$$s_\gamma = 1 - 0.4 i_\gamma\left(\frac{B'}{L'}\right)$$

d_q is a depth factor given by:

$$d_q = 1 + 1.2\left(\frac{h}{B'}\right) tan\,\phi'\,(1 - sin\,\phi')^2$$

i_q and i_γ are load inclination factors given by:

$$i_q = 1 - 0.5\left(\frac{H_{base}}{V_{base}}\right)^5,$$

$$i_\gamma = 1 - 0.7\left(\frac{H_{base}}{V_{base}}\right)^5$$

In this section if horizontal load is not under consideration the formulae may be used with $i_q = i_\gamma = 1$.

Sliding check

The horizontal resistance of a SICF is dependent on the elevation of the load application point due to the coupling between horizontal loads and moments. A maximum value of the horizontal resistance$H_{ult} = H_{max}$) is found when the horizontal loading is applied at the elevation which results in horizontal translation (without rotation). Such a case is only relevant to multi-caisson foundations.

In addition to combined loading calculations it is usual also to carry out a sliding check against purely horizontal load. In that case the horizontal load on the skirt wall will be combined with a base resistance of $H_{base} = \frac{\pi D^2}{4} s_{u2}$ in clay and $H_{base} = V_{base} \tan \phi'$ in sand. Note (1) that it is not usual to reduce the area to the effective area for this calculation and (2) because the base of the caisson involves soil-on-soil contact the soil strength may be fully mobilised at the base.

The potential for preferential sliding failure within softer/weaker soils below the skirt tip should be checked in layered soils.

Vertical loading: tension

In undrained vertical tension loading the ultimate tensile capacity may be estimated by applying a "reverse bearing capacity" calculation. However, the potential for preferential flow paths which could result in drained behaviour should be considered. Employing the methods detailed above, and with careful attention to the differing role of the weight of the soil trapped within the caisson in compression and tension, the result is:

$$V_{ult,t} = W_{caisson} + N_c^* A s_{u2} + \alpha \pi D h s_{u1} \quad \text{Equation C.17}$$

where:

N_c^* is a reverse bearing capacity factory for a circular embedded foundation.

In all but soft clays the ultimate tensile capacity in clay may be limited by cavitation at the caisson base:

$$V_{ult,t} = \frac{\pi D^2}{4} \gamma h + \frac{\pi D^2}{4} (p_a + \gamma_w h_w - p_{void}) + \alpha \pi D h s_{u1} \quad \text{Equation C.18}$$

And with cavitation under the lid as:

$$V_{ult,t} = \frac{\pi D^2}{4} (p_a + \gamma_w h_w - p_{void}) + 2\alpha \pi D h s_{u1} \quad \text{Equation C.19}$$

In each case$p_{void} = 0$ if full vacuum is assumed in the void (*i.e.* no flow of water).

In sands two cases need to be considered for tensile loading: rapid (undrained) loading and slow (drained) loading.

For rapid loading the tensile capacity may be very high and invariably controlled by cavitation under the footing base or caisson lid, the ultimate load is calculated as the smaller of the following:

For cavitation at footing base:

$$V_{ult,t} = \frac{\pi D^2}{4} \gamma h + \frac{\pi D^2}{4} (p_a + \gamma_w h_w - p_{void}) + \pi D \frac{\gamma' h^2}{2} K \tan \delta \quad \text{Equation C.20}$$

For cavitation below caisson lid:

$$V_{ult,t} = \frac{\pi D^2}{4} (p_a + \gamma_w h_w - p_{void}) + 2\pi D \frac{\gamma' h^2}{2} K \tan \delta \quad \text{Equation C.21}$$

In each case $p_{void} = 0$ if full vacuum is assumed in the void (*i.e.* no flow of water).

If full communication of free water into the void is assumed the above expressions become:

For cavitation at footing base:

$$V_{ult,t} = \frac{\pi D^2}{4} \gamma h - \frac{\pi D^2}{4} \gamma_w h + \pi D \frac{\gamma' h^2}{2} K \tan \delta \quad \text{Equation C.22}$$

For cavitation below caisson lid:

$$V_{ult,t} = 2\pi D \frac{\gamma' h^2}{2} K \tan\delta$$ Equation C.23

For slow loading the tensile capacity arises only from friction on the sides of the caisson and is calculated as:

$$V_{ult,t} = 2\pi D \frac{\gamma' h^2}{2} K \tan\delta$$ Equation C.24

For more precise design calculations it may be important to assess the reduction in vertical tensile capacity due to “updrag” on the soil from the caisson, in which the vertical effective stress around the caisson skirt is reduced due to the upward friction from the skirt onto the soil. More details about this calculation, which involves similar principles to that required for caisson installation, can be found in Houlsby et al. (2005).

Experimental data in sand indicate that while large ultimate tensile capacities of caissons may exist, these can only be mobilised at substantial displacements, as much as 10% of the caisson diameter (for example Kelly et al. (2006) or (Bienen et al., 2018a)). Mana et al (2013) found for a SICF in clay that displacements were limited up to approximately 50% of the undrained capacity for both compression and tension, which is typically the maximum load level allowed for SLS conditions considering load and material factors applicable for ULS conditions as given in Section 3.3.

The potential for significant uplift displacement should be carefully considered if tensile loads are allowed to exceed the skirt frictional capacity. For sands this is calculated as for the slow loading above, and for clays it might be taken as:

$$V_{ult,t} = 2\pi D h \alpha s_{u1}$$ Equation C.25

Combined loading – tension

The effects of horizontal and moment loading in combination with tension loading are very poorly understood and should be treated with extreme caution.