Guidelines for the Site-Specific Geotechnical Analyses of Jack-ups

230840-R-001 02 | 19 December 2023

Complete

Document Control

Document Information

Project Title	Guidelines Document
Document Title	Guidelines for the Site-Specific Geotechnical Analyses of Jack-ups
Fugro Project No.	230840
Fugro Document No.	230840-R-001
Project Duration	02 October 2023 - 19 December 2023
Issue Number	02
Issue Status	Complete

Client Information

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Revision History

Issue	Date	Status	Comments on Content	Prepared By	Checked By	Approved By
02	19 December 2023	Complete	Addressed client comments	TKG	NK/CE/JP/ WW/JC	TC
01	29 November 2023	Draft	Awaiting client comments	TKG/CE	TC/JP/ WW/JC	NK

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Abbreviations

CAU	Anisotropically consolidated undrained triaxial test
CAUC	Anisotropically consolidated undrained triaxial compression test
CAUE	Anisotropically consolidated undrained triaxial extension test
CD	Chart datum
CID	Isotropically consolidated drained triaxial test
CIU	Isotropically consolidated undrained triaxial tests
CLR	Cyclic load ratio
CPTu	Piezocone penetrometer
DS	Direct shear (Shear box) test
DSS	Direct simple shear test
LAT	Lowest astronomical tide
MinV	Miniature vane
MV	Motor vane
PP	Pocket penetrometer
PSA	Norwegian Petroleum Safety Authority
ROV	Remotely operated underwater vehicle
SI	Site investigation
SS	Simple shear
TV	Torvane
TXC	Triaxial compression
TXE	Triaxial extension
ULS	Ultimate limit state
UU	Unconsolidated undrained triaxial test
UXO	Unexploded ordnance

List of Symbols

Latin Scripts	
B_q	Pore pressure coefficient
CI	Consolidation index
c_h	Coefficient of consolidation for horizontal drainage
c_v	Coefficient of consolidation for vertical drainage
D	Widest diameter of spudcan
d	Diameter of the penetrometer/vane blade
D_r	Relative density
F_r	Normalised sleeve friction
f	Frequency of cyclic loading
G_{max}	Small strain or maximum shear modulus
h	Vane blade height
I_{RD}	Relative dilatancy
K_0	Earth pressure coefficient
LL	Liquid limit
N	Number of loading cycles
N_{ball}	Ball factor
N_{eq}	Equivalent number of loading cycles
N_f	Number of loading cycles to failure
N_{kt}	Cone factor
$N_{T\text{-}bar}$	T-bar factor
OCR	Over consolidation ratio
p'	Mean effective stress
PL	Plastic limit
I_p	Plasticity index
P_{ref}	Reference/atmospheric pressure (100 kPa)
Q	Normalised cone resistance
$Q_{crushing}$	Particle crushing strength on a natural log scale
q_c	Cone tip resistance
q_{net}	Net cone resistance
$q_{T\text{-}bar}$	T-bar penetration resistance
q_t	q_c corrected for pore pressure effects
$q_{drained}$	Spudcan capacity under drained conditions
$q_{undrained}$	Spudcan capacity under undrained conditions
$q_{part\_drained}$	Spudcan capacity under partially drained conditions
S_t	Sensitivity
s_u	Undrained shear strength

$s_{u,rem}$	Remoulded undrained shear strength
s_u^{DSS}	Undrained shear strength obtained from direct simple shear testing
s_u^C	Undrained shear strength obtained from triaxial testing in compression
s_u^E	Undrained shear strength obtained from triaxial testing in extension
T	Period of cyclic loading
T_{max}	Maximum torsional moment
T_{rem}	Remoulded torsional moment
u	Pore pressure
u_2	Pore pressure measured at the cone shoulder
u_{cy}	Cyclic pore pressure
u_p	Permanent pore pressure
u_a	Average or mean pore pressure
v	Penetration rate
v_n	Normalised velocity
Greek Scripts	
γ'	Submerged unit weight of soil
γ	Shear strain
γ_a	Average of mean shear strain
γ_{cy}	Cyclic shear strain
σ_{vc}'	Effective vertical stress
σ_{vc}	Total in-situ vertical stress
σ_1, σ_2, σ_3	Principal stresses
τ	Shear stress
τ_a	Average or mean shear stress
τ_{cy}	Cyclic shear stress
τ_{min}	Minimum shear stress
τ_{max}	Maximum shear stress
$\tau_{f, cy}$	Cyclic undrained shear strength
ϕ'	Effective friction angle
ϕ'_{cv}	Friction angle at constant volume or Critical state friction angle
ϕ'_{peak}	Peak friction angle
Δu	Additional pore pressure
$\Delta\gamma_a$	Additional average shear strain
$\Delta\tau_a$	Additional average shear stress

1. Introduction

1.1 Purpose of document

The Norwegian Petroleum Safety Authority (PSA), Norway's governmental supervisory agency with regulatory control over safety, work environment, emergency preparedness, and security, ensures that entities in the petroleum industry maintain high standards of health, safety, environmental protection, and emergency readiness.

Fugro has been commissioned by PSA to create guidelines for the site-specific geotechnical analyses of jack-ups. The purpose of this document is to address critical geotechnical aspects, such as cyclic loadings, drainage behaviour of soils (undrained/drained/partially drained behaviour) under various loading conditions, and the significance of soil anisotropic properties in geotechnical assessments, which have not been widely covered in the existing standards and regulatory provisions (e.g., ISO 19905-1:2023).

1.2 Scope of document

This document covers geotechnical guidelines that will offer essential recommendations for ground investigations, laboratory testing, and the establishment of geotechnical parameters to improve the geotechnical analysis for site-specific assessment of jack-ups. This document specifically aims to provide some guidance on cyclic loading, soil anisotropy and drainage behaviour of soils under various loading conditions for the jack-up spudcan geotechnical analysis. The guidelines are prepared exclusively for offshore activities on the Norwegian Continental Shelf, excluding fjords and Arctic conditions. Specifically, these guidelines are drafted for three-legged jack-ups with spudcans, widely utilized in the fossil fuel industry.

This guidelines document was prepared by reviewing various standards, independent reports produced by various organisations, and published articles. Although PSA requested to provide guidelines on three main topics (i.e., cyclic loading, drainage effects, and soil anisotropy), recommendations were also included on ground investigations, laboratory testing, and the establishment of geotechnical parameters to ensure continuity and to provide all necessary information in a single document.

Section 2 of this document covers the data required for jack-up site specific assessments, prepared by reviewing data from ISO 19905-1 (2023) and SNAME (2008). Section 3 deals with the required geotechnical site investigations and the derivation of geotechnical parameters for monotonic loading conditions. Section 3 has been developed primarily in accordance with the guidelines from ISO 19905-1 (2023) and RPS Energy (2011). Furthermore, additional insights from recent research and supplemental details that were not covered in existing standards have been incorporated into Section 3. Section 4 covers the derivation of geotechnical parameters for cyclic loads, mainly derived from the guidelines established by DNV (2021) and CFMS (2019). Section 4 also provides guidance on soil anisotropy and ways to account for soil anisotropy in geotechnical engineering analysis. Section 5 focuses on spudcan bearing capacity calculations under partial drainage and cyclic loading conditions. The documents reviewed have been referred to at appropriate places and listed in the 'References' section of the document.

1.3 Brief guidance on partial drainage, cyclic loading effects, and soil anisotropy in jack-up analysis

ISO 19905-1 (2023) provides methodology and references for performing site-specific jack-up analysis. Within the ISO framework, the key geotechnical parameters required for jack-up spudcan geotechnical analysis are :

- submerged unit weight of soil
- friction angle for sand
- undrained shear strength for clay
- shear modulus of soil

The friction angle of sand (determined from drained tests) and undrained shear strength of clay (determined from undrained tests) are not constant values for a particular soil, but rather depend on several parameters such as in-situ conditions (e.g., vertical stress, preconsolidation pressure, void ratio of clay, and relative density of sand), type of loading, loading rate, and loading direction, among others. For example, the friction angle of sand determined from drained laboratory tests may not be the same as the friction angle mobilised in the field under partially drained conditions. Similarly, the shear strength of soil determined assuming undrained cone penetration may not be a true representation of field conditions if there is partial drainage during cone penetration. Soil that behaved as partially drained or drained during cone penetration tests may behave as undrained or partially drained during the penetration of large-diameter foundations (such as jack-up spudcans). Further, in addition to soil inherent anisotropy, stress-path-induced soil anisotropy can lead to mobilising different shear strengths along the failure plane. Importantly, under cyclic loading, shear strength parameters determined from monotonic tests can significantly degrade in certain soils and under certain loading conditions. The following sections provide brief guidance on the effect of soil anisotropy, partial drainage, and cyclic loads on jack-up analysis.

1.3.1 Soil anisotropy

Soil anisotropy refers to the load direction dependence of soil strength and stiffness properties. The importance of soil anisotropy is particularly pronounced in undrained stability and bearing capacity scenarios, where load-path induced anisotropy impacts the distribution of undrained shear strength (s_u) along the slip surface, determined by the direction of the major principal stress.

Measuring undrained shear strength anisotropy in the laboratory generally involves conducting triaxial compression (TXC), triaxial extension (TXE), and direct simple shear (DSS) tests, by consolidating specimens anisotropically to appropriate in situ or consolidated stress levels before shearing. While DSS strength data can generally be considered a reasonable average of TXC, DSS, and TXE strengths in many cases (particularly in clay), this assumption may not always hold, especially for dense silt and sand. In simplified approaches, contour diagrams are established for DSS loading, and empirical anisotropy factors are employed to consider triaxial stress paths. Section 4.5 provides typical anisotropy factors available in the literature.

In most practical cases, practitioners tend to ignore soil anisotropy effects in spudcan analysis and may use the DSS strength as a reasonable estimate of the average strength. However, as noted above this simplifying assumption may not always be valid, albeit it will generally be conservative. The importance of considering soil anisotropy in spudcan analysis will depend on the anisotropy ratio determined for a specific soil type as ideally obtained from laboratory tests on site specific samples collected in the field. If a simple average of TXC, DSS, and TXE strengths does not seem to provide an appropriately representative strength (e.g. if the TXC strength is much higher than the DSS or TXE strength) then more advanced numerical analyses that can fully incorporate anisotropic behaviour may be considered (e.g. Jostad et al., 2015).

1.3.2 Effect of partial drainage

The basic presumption in ISO 19905-1 (2023) is that spudcans in clay will always respond undrained while those in sand will always respond drained. However, in practice while this is invariably true for clay it is not always the case in sand. General guidance highlighting the potential influence of partial drainage/consolidation during typical CPT testing and during spudcan penetration or storm loading is provided in Section 5.1 of the current report. Soils with coefficient of consolidation (c_v) ranging between 10,000 m^2/yr to 1,000,000 m^2/yr (i.e. all intermediate soils and some fine sands) may potentially result in partially drained spudcan penetration (for a nominal spudcan diameter of 15 m, with penetration rates in the range of 0.4 to 4 m/hour). Under storm loading conditions, the soil supporting the spudcans is likely to exhibit fully undrained behaviour for c_v less than 150,000 m^2/yr to 250,000 m^2/yr (i.e. any intermediate soil type), and partially drained conditions for c_v to at least 5,000,000 m^2/yr (i.e. well into the clean sand range). Guidance on the determination of spudcan bearing capacity during partially drained conditions is provided in Section 5.1.

1.3.3 Effect of cyclic loads

In many offshore environments, jack-ups may be subjected to significant wave loads in accordance with the specified design storm. These wave loads impose cyclic shear stresses into the soil and the effects of this should be considered for the in-place storm stability analysis.

Clayey soils are normally considered to respond undrained for all load effects during a design storm and may accumulate "damage" (reduction of soil strength/ stiffness) over the full duration of the storm. However, even in sand, the load duration of a single wave cycle may be so short that the soil will behave undrained during the time the design wave applies, although the soil is likely to be fully consolidated under the applied static deadload. For intermediate soils (i.e. clayey or silty sands/ silts), varying levels of drainage may apply depending on the soil consolidation properties that apply.

In all soil types, cyclic shear strengths may be determined based on total stress analysis, but in more sandy soil types consideration of excess pore pressure accumulation may also be useful as this will allow the effects of partial consolidation from one load cycle to the next to be determined. Thus, the behaviour of soil under cyclic loading is evaluated by considering the potential loss of undrained shear strength, the development and dissipation of excess pore water pressures, cyclic stiffness characteristics, and the accumulation of permanent strains within the soil.

Most jack-up units used in oil and gas operations impose large vertical stresses, via the spudcans, onto the underlying soil. As a result, the typical ratio of cyclic shear stress to average shear stress (τ_{cy}/τ_a, referred to as cyclic load ratio, CLR), induced during a storm may be relatively small (e.g. 0.1 to 0.2) as compared to many other types of offshore foundation system. Cyclic degradation under such conditions may therefore be more benign than in many other offshore foundation systems but nevertheless should not be ignored.

An assessment of the appropriate cyclic shear strength can be made from suitable laboratory tests (DSS and triaxial tests) on ideally intact samples, at insitu (and if appropriate) consolidated stress conditions. For soils where cyclic degradation may be a significant issue, a yield envelope approach is recommended to compute the cyclic bearing capacity of the spudcans, as discussed in Section 5.2.

In general, simple approaches with standard soil investigation and existing cyclic correlations for soils similar to those under investigation at identical in-situ conditions may be used for uniform soils with well-established properties and significant local experience. This is applicable, for instance, in reasonably less problematic soils like uniform dense to very dense sands and highly overconsolidated soils. However, advanced laboratory testing, including cyclic laboratory tests, are recommended in loose to medium dense sands, soft and sensitive cohesive soils, and in intermediate soils (silts, sandy silts, silty sands, etc), as well as in layered soils. Additionally, with the possibility of lower penetration in uniform dense to very dense sands, one must evaluate the potential scour effects on jack-up performance. The evaluation of scour and its influence on jack-up performance is beyond the scope of this document, but one can refer to ISO 19905-1 (2023) and DNV (2021) for more details on scour.

2. Data to Assemble for Jack-up Site-Specific Assessments

2.1 General guidance

ISO 19905-1 (2023) discusses the essential data required for jack-up site specific assessments. This section provides a concise summary of key points from ISO 19905-1 (2023).

2.2 Jack-up data

The data essential for conducting a site-specific assessment for a jack-up comprises the following (ISO 19905-1, 2023):

- Type of the jack-up
- Relevant technical drawings, specifications, and operations manual
- Comprehensive data related to the strength, stiffness, and operational aspects of the leg-to-hull connection
- Proposed lightship, variable load, and centres of gravity for each configuration, considering any alterations not reflected in the most recent operations manual
- Weight specifications
- Preloading capacity details
- Material composition specifications
- Specifics concerning the maximum spudcan capacity, such as reactions and distribution of bearing pressure, utilized in the design scenarios.
- Design parameters, along with any proposed deviations intended for the operation
- Comprehensive information regarding any modifications relevant to the assessment

2.3 Site setting

The geological information for the site evaluation must encompass precise details such as geographical coordinates, seafloor topography, and water depth, all referenced to a specific and clearly defined datum, e.g., Lowest Astronomical Tide (LAT) or Chart Datum (CD). It is crucial to note that charts primarily designed for use by shallow draft vessels might lack the requisite precision for siting jack-ups.

2.4 Metocean data

The jack-up should be assessed for the extreme storm event under the ultimate limit state (ULS) assessment. For jack-ups operating in Norwegian waters, ISO 19905-1 (2023) recommends considering 100 year joint probability metocean data for extreme storm event assessments (ULS assessment). In the absence of reliable 100 year site-specific joint probability data, a combination of 100 year waves, 100 year wind, and 10 year current can be applied (ISO 19905-1, 2023). Further information and specifics are provided in ISO 19905-1 (2023) and DNV-OS-C104 (2022). The following sections cover the essential metocean data required in the jack-up analysis:

2.4.1 Water depth

The necessary information includes the water depth at each jack-up site, referenced to the LAT.

2.4.2 Tidal range and storm surge

The site-specific assessment of crewed jack-ups necessitates the calculation of critical tidal factors, particularly the maximum tidal range and the storm surge for the assessment return period, for the designated jack-up location or operational area.

2.4.3 Wave data

The wave actions should be determined using an appropriate wave kinematics model in accordance with ISO 19905-1 (2023). For the evaluation of survival conditions, the extreme wave height environment should be calculated, considering a storm duration (three hours minimum) with intensity specified by the significant wave height for the assessment return period. If necessary, the seasonally adjusted wave height can be utilised for the specific operation. Additional comprehensive information is available in ISO 19905-1 (2023), providing guidance on this procedure.

2.4.4 Current velocity and profile

The extreme surface current velocity caused by wind shall align with the wind intensity for the assessment return period, with seasonal adjustments considered if necessary. In cases where directional data for other current components is accessible, the maximum surface flow from the mean spring tidal current, along with the surge current for the assessment return period (adjusted seasonally if needed), should be vectorially combined in the down-wind direction and incorporated with the wind-induced surface current.

The current profile can be characterised through a series of velocities at various elevations from the sea floor to the water surface. Unless site-specific data suggests otherwise and in the absence of additional residual currents (like circulation, eddy currents, slope currents, internal waves, inertial currents, etc.), an appropriate method recommended by ISO 19905-1 (2023) can be applied to calculate the current profile. For more detailed guidance, refer to ISO 19905-1 (2023).

2.4.5 Wind speed and profile

The wind speed to be considered for assessment shall correspond to the 1-minute sustained wind for the assessment return period, referenced at a height of 10.0 m above mean sea level. For a thorough understanding of this procedure, please refer ISO 19905-1 (2023).

In the absence of specific contrary data, it is assumed that wind, wave, and current loads are caused by individual return period extremes acting concurrently in the same direction as the extreme water level. Adjustments for seasonal variations in values can be made as per the duration of the operation.

2.5 Ground model

A ground model can be considered as an up-to-date state of knowledge of the seafloor and sub-seafloor conditions and processes that are relevant to the planning, developing and monitoring of a site. The offshore ground model serves as a predictive tool, offering insights into the geotechnical conditions beneath the seabed. It involves the systematic analysis and interpretation of geological, geophysical, and geotechnical data to create a representative model of the subsurface conditions. Figure 2.1 summarises the process involved in generating a ground model, and the subsequent sections provide further details on the data required for this process.

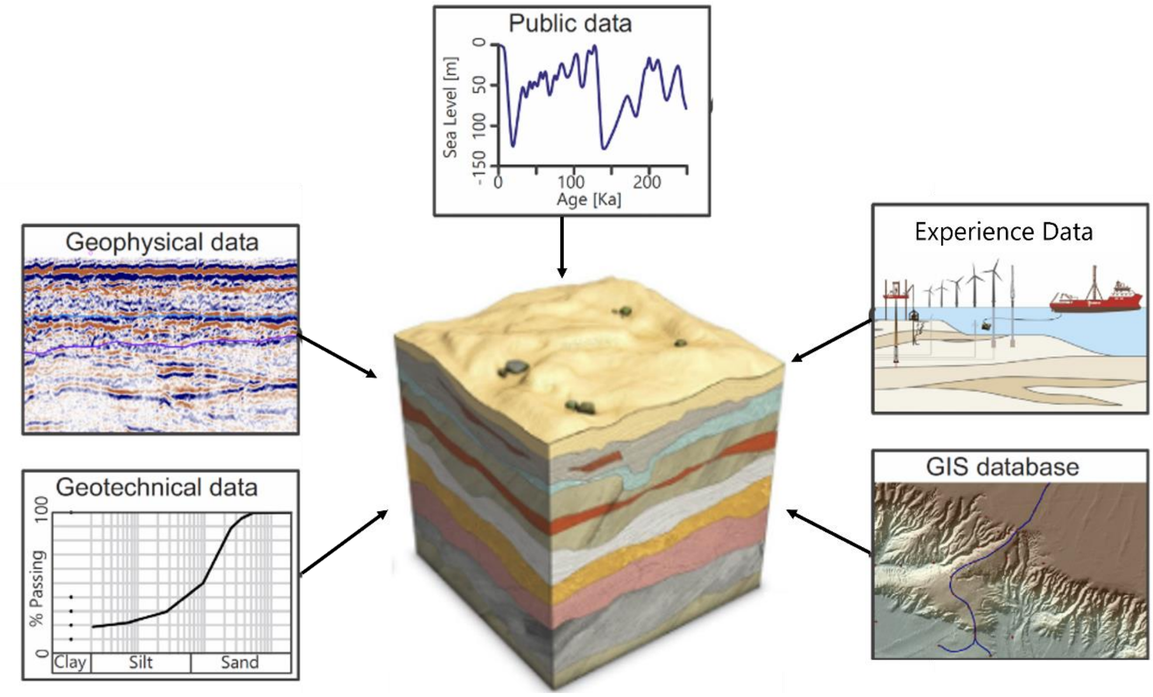

Figure 2.1: Data required for the generation of a ground model

2.5.1 Geophysical data

The designated site for the jack-up must undergo a thorough assessment for potential geohazards. The data obtained from the surveys and investigations outlined in the subsequent sections is vital, especially in areas lacking prior operational data. However, in regions where relevant information is already accessible, it may be possible to streamline the criteria mentioned below by utilising data from prior surveys or activities within the vicinity. For additional guidance, please refer ISO 19901-10 (2021) and ISO 19905-1 (2023).

2.5.1.1 Bathymetric survey

An appropriate bathymetric survey should be supplied for an area approximately 1km^2, centred on the proposed site. Line spacing of the survey should typically be not greater than 100 m x 100 m over the survey area. Interlining should be performed within an area 500 m x 500 m, centred on the proposed site. Interlining should have spacing less than 25 m x 50 m. Further interlining should be performed if any irregularities are detected, or if a known stand-off position sits outside the defined (500 m x 500 m) interlined area. Such surveys are normally carried out using acoustic reflection systems.

2.5.1.2 Seafloor survey

A survey of the seafloor is imperative to identify various features, encompassing natural elements like sand waves, rock outcrops, and boulders, as well as human-made installations such as offshore structures, subsea pipelines, cables, rock deposits, scour pits, jack-up imprints, wrecks, and any debris on the seafloor. This survey should utilise side-scan sonar or high-resolution multibeam echosounder techniques to ensure high-quality data, enabling the identification of obstacles and seabed characteristics within the immediate area, typically a 1 km^2 around the intended location. The chosen slant range should ensure a minimum of 100% overlap between adjacent survey lines. In instances where buried pipelines, cables, or metallic debris close to the sea floor are suspected, a magnetometer survey may also be necessary. Additional inspections, potentially involving an remotely operated under-water vehicle (ROV) or diver, may be necessary in conjunction with sidescan sonar and magnetometer surveys to safely position the jack-up.

Specialized surveys are crucial for detecting submerged pipelines or cables, sunken anchors and chains, wrecks, unexploded ordnance (UXO), or other metallic debris lying on or beneath the seafloor. Although these surveys might be waived, it should be justified through a thorough site-specific assessment, including an in-depth analysis of available evidence.

It is important to note that seafloor surveys can become outdated, particularly in areas of active construction or drilling operations, or regions with shifting sediments. Prudent judgment should be exercised regarding the relevance and validity of all surveys, especially with respect to changing conditions. For open locations, the maximum validity period for seabed surveys relating to debris and mobile sediment conditions should be determined, considering local circumstances. However, in locations close to existing installations, seabed surveys for debris and sediment conditions should, where practical, be conducted immediately before the jack-up arrives at the location.

2.5.1.3 Shallow seismic survey

The shallow seismic survey has primary goals including determining near-surface soil stratigraphy and detecting shallow gas concentrations. However, due to the qualitative nature of seismic surveys, a comprehensive foundation appraisal requires correlation with soil boring data in the vicinity based on similar stratigraphy. The survey should cover an area of approximately 1 km^2, centred on the proposed site, with line spacing not exceeding 100 m x 250 m. The equipment used should identify reflectors of 0.5 m and thicker to a depth of 30 m or the anticipated footing penetration plus 1.5 times the footing diameter if greater. The survey report should encompass two perpendicular vertical cross-sections passing through the proposed site, displaying all relevant reflectors and associated geological information. Additionally, the seismic survey data should be carefully reviewed to identify any lateral variability in soil layering or sub-surface hazards like steep-sided channels filled with soft soils.

2.5.2 Geotechnical data

The insights derived from the geophysical site survey lay the foundation for planning the subsequent geotechnical site investigation (SI). Tailoring the scope of work is contingent on the vertical and lateral variability of the soil and the presence of any potential geohazards.

Intrusive geotechnical SIs play a crucial role in validating the geophysical data and acquiring vital geotechnical index, strength and stiffness measurements. This ensures confirmation or further fine-tuning of the interpreted ground model. Thorough data collection is indispensable, enabling detailed engineering characterisation of each soil unit and providing a grasp of the spatial variation of the geotechnical properties.

This comprehensive dataset aids in establishing soil design profiles, encompassing engineering strength parameters crucial for site-specific predictive bearing capacity analyses concerning jack-up foundations.

2.5.2.1 Soil data required for jack-up foundation site specific assessment

The soil drainage behaviour governs the soil parameters required for the jack-up foundation analysis. Table 2.1 presents the fundamental soil properties essential for jack-up foundation assessment, considering homogeneous clay and sand. In cases of silty material, the properties defined for clay may apply, but partial drainage characteristics should be taken into account (refer to Section 5.1). In highly heterogeneous and intricate ground conditions where advanced foundation performance modelling is pursued, a more extensive set of soil properties will be required for accurate assessment. CFMS (2019) presents additional parameters that are required to characterise some non-standard soils (e.g., carbonate sands with or without cementation, soils of volcanic origin, chalk, and organic soils), though these are unlikely to be encountered offshore on the Norwegian Continental Shelf. Sections 3 and 4 provide details on determining the required geotechnical parameters for jack-up analysis.

Table 2.1: Soil properties required for site specific assessment of jack-up foundations

<table>
<tr><th>Soil type</th><th>Monotonic strength properties</th><th>Index properties and additional parameters</th><th>Cyclic properties</th></tr>
<tr><td>Clay (undrained)</td><td>Undrained shear strength,
Soil sensitivity</td><td>Water content,
Plastic Limit,
Liquid limit,
Submerged unit weight,
Coefficient of consolidation,
Over consolidation ratio,
Carbonate content</td><td rowspan="2">Cyclic shear strength under various combinations of average and cyclic shear stresses for triaxial and/or simple shear stress paths,
Cyclic shear strain versus cyclic shear stress for triaxial and/or simple shear stress paths,
Cyclic shear strain and pore pressure contour diagrams,
Shear modulus at very small strain (G_o or G_{max}) and shear modulus degradation curves,
Damping</td></tr>
<tr><td>Sand (drained)</td><td>Friction angle</td><td>Particle size distribution,
Relative density,
Submerged unit weight,
Carbonate content</td></tr>
</table>

3. Marine Soil Investigations and Derivation of Geotechnical Parameters for Jack-up Site-Specific Assessments

3.1 Marine soil investigation planning

The extent and detail of in-situ tests, soil sampling, and laboratory analyses required for analytical purposes are contingent upon the specific ground conditions. In cases where the ground consists of a continuous and homogeneous soil layer, minimal data acquisition is needed, albeit sufficient to validate its homogeneity. Conversely, when dealing with highly variable and complex ground conditions where advanced foundation performance modelling is envisaged, a significantly larger volume of information becomes necessary.

Ideally, the geotechnical SI should be thoroughly planned and conducted well in advance of deploying the jack-up to the field. This proactive approach allows ample time for data collection, thorough interpretation, and a comprehensive site-specific assessment.

The scope of the geotechnical SI should be designed with careful consideration of anticipated ground conditions which can be developed from a good desk study and, if available, previous site experience. Variations in ground conditions can arise from geological processes or human intervention, potentially impacting the foundation's performance. A geophysical site survey helps identify potential variations in ground conditions, aiding in initial planning. However, it is essential to remain flexible in the SI work scope to accommodate unexpected ground conditions encountered during the investigation and allow the site survey to be amended to encompass the findings, ensuring a thorough understanding of the geotechnical characteristics of the site. This adaptability is crucial for accurate foundation assessment and subsequent jack-up deployment.

3.2 Site investigation requirements

A comprehensive geotechnical investigation is essential for jack-up site-specific assessment. The level of complexity and requirements for any new geotechnical investigation at a site are related to the volume of data available at the site and the complexity of the geotechnical conditions. The minimum requirements are outlined in various existing standards, primarily ISO 19905-1 (2023) and DNV (2021), and these should be followed. The investigation should cover all layers and transition zones, and adequate sampling rates should be ensured. According to ISO 19905-1 (2023) and DNV (2021), the borehole depth for soil borings to obtain reliable soil parameters is recommended to be a minimum of 1.5 spudcan diameters below the predicted depth of spudcan penetration or 30 m, whichever is the greater. If needed, the intended depth of SI can be extended further to accommodate future jack-up installation operations. This might be particularly relevant if a different rig with a higher pre-load capacity is to be utilized or if there are different structural needs, such as the depth for the conductor setting and the early data required for platform design.

A meticulous investigation should leave no gaps in the thorough assessment of the site, and if needed, a combination of high quality undisturbed soil sampling, in situ testing, and laboratory testing should be conducted. Recognized in situ soil testing tools include piezocone penetrometer (CPTu), vane shear, T-bar and ball penetrometer tests (see Section 3.3 for more details). Additional testing for shear moduli and cyclic/dynamic behaviour, may be necessary for a thorough analysis, especially where cyclic loading can affect soil strength (see Section 4 for more details).

DNV standards are generally adhered to for offshore operations within the Norwegian Continental Shelf. If there is a lack of prior knowledge regarding soil conditions, DNV (2021) suggests performing, along with a geophysical survey, at least one boring with soil sampling and CPTu testing at each leg location. The number and location of boreholes should consider soil variability, regional experience, and geophysical findings. The determination of whether additional soil borings are necessary should consider the following factors:

- The degree to which the soil profiles obtained from the borings align with each other
- The potential impact of variations in shear strength, derived from the measured values, on the risk of problematic penetrations during preload and operation
- The absence of anomalies, such as buried channels, as revealed by the geophysical survey.

The necessity of performing boreholes at every leg location may be unnecessary in areas with no lateral variability. The findings from the initial borehole(s) carried out at the site may influence the extent and scope of the geotechnical site investigation. Annex-D of ISO 19905-1 (2023), as recommended by DNV (2021), addresses various site conditions (or geological settings) and provides recommendations for conducting offshore geotechnical site investigation works necessary for a jack-up foundation site-specific assessment.

In cases where the combination of information from geological deposition, stratification from the geophysical survey, and soil parameters obtained from one boring to the minimum required depth eliminates the possibility of adverse or critical performance of the jack-up, one boring may be sufficient. However, drawing such a conclusion should not solely rely on specific shear strength profiles from one boring without considering potential significant deviations based on the known geological deposition history.

Exercise particular caution when there are possibilities of punch-through failures, especially when penetrating a strong top layer into a weaker layer below, either during preloading or operation (DNV, 2021). Consulting a qualified geotechnical engineer experienced in jack-up foundation assessments is advisable to ensure the site investigation's adequacy.

3.3 Field soil investigation activities

3.3.1 In-situ penetration tests

Penetrometer tests utilising a deep push seabed system offer continuous soil strength profiles with minimal disruption, making them highly recommended if sufficient penetration depth can be attained. On the other hand, tests using a downhole system yield profiles typically with data gaps, which can be minimised by overlapping the penetrometer pushes. Additionally, the drilling operation during these tests causes some level of soil disturbance, particularly noticeable at the beginning of each cone stroke.

A continuous penetrometer profile is of particular importance at sites where:

- Precise delineation of lateral or vertical soil layers is vital for accurate ground model interpretation;
- Even minor variations or uncertainties (sensitivity) in the interpreted soil strength profile within a small depth can significantly impact the spudcan load versus penetration profile; and
- There is a potential risk of dissolved gas within the soil matrix when utilizing down-hole site investigation methods (Kortekaas and Puechen, 2008).

Piezocone penetration tests (CPTu) are widely utilised in the field due to their ability to provide a continuous soil profile. These tests yield data on tip penetration resistance, sleeve friction, and pore pressure. They are carried out either continuously in seabed mode, or with strokes of up to 4.5 m in downhole mode. Unlike discrete soil sampling and testing, the continuous plots from cone penetration tests allow for the delineation of stratigraphic boundaries and trends in soil strength. Typically, this data is compared with laboratory strength test results. Furthermore, CPTu tests can be extended to encompass dissipation tests, enabling an estimation of soil consolidation coefficients (see Section 3.5.4).

In comparison, full-flow penetrometers, such as cylindrical T-bar and spherical ball penetrometer offer certain advantages over the traditional CPTu. However, it is worth noting that these tests are better suited for use in soils with lower strength, potentially weaker than those capable of supporting spudcans. As a result, these penetrometers are not extensively used in geotechnical site investigations for jack-up assessments and the understanding of the penetration behaviours of these penetrometers is restricted to clayey soil conditions.

In-situ vane tests provide discrete measurements of the intact and residual undrained shear strength of clays, typically for clays with undrained shear strength less than 200 kPa. However, the results may be sensitive to drainage effects, material heterogeneity, and in-situ stress anisotropy. Adopting different aspect ratio vanes can assist in evaluating soil anisotropy. On the other hand, penetrometer systems are more versatile and quicker than vane testing systems, encouraging their use.

3.3.2 Soil sampling

Soil sampling is a critical step providing material for visual examination and laboratory testing to determine the geological provenance, characteristics, and geotechnical engineering design parameters of the soil. The geotechnical engineering design parameters can also be assessed through in-situ field testing. Continuous in-situ profiling is valuable for accurately determining layer boundaries and material variation.

The quality of the geotechnical SI significantly impacts spudcan penetration prediction. Hence, sampling and field testing need meticulous planning, often requiring a dedicated offshore geotechnical vessel. Dynamically positioned geotechnical vessels equipped with a hard tie heave compensation system can assist in obtaining soil samples with minimal disturbance, even in soft clays.

A variety of techniques and tools are employed for offshore soil sampling, tailored to the specific conditions and soil types encountered. During soil sampling, it is crucial to minimise material disturbance, as excessive soil disturbance during sampling will make laboratory test results unrepresentative of in-situ conditions.

Table 3.1 provides a comparison of sample quality and recovery for seabed samplers and down-hole sampling equipment, based on data from ISSMGE (2005). For additional information on the equipment/systems, operational water depths, and maximum penetration depths, please refer to ISSMGE (2005).

Table 3.1: Sample quality and recovery from various sampling equipment (ISSMGE, 2005)

Type of equipment	Sample quality		Recovery (Relative to length of sample tube)	
	Sand	Clay	Sand	Clay
Seabed sampling equipment:				
Gravity / Piston corer	2	3	1	3 - 4
Vibrocorer	2 - 3	2 - 3	3 - 4	2 - 3
Grab sampler	1 - 2	1	1 - 2	2
Box corer	1 - 2	5	1	5
Rotary corer	1	2	1	3
Downhole sampling equipment:				
Hydraulic piston sampler	3 - 4	5	3	5
Hydraulic push sampler	3 - 4	4 - 5	3	5
Hammer sampler	2 - 3	2 - 3	3 - 4	3 - 4
Rotary coring	1	2	1	3

Notes:
1: Poor or inappropriate
2: Acceptable for non-critical analyses
3: Moderately good
4: Good
5: Very good

Downhole sampling using piston or push samplers is favoured for its ability to recover relatively undisturbed soil samples from significant depths. The "Shelby tube" is one widely utilized tool in the specialist geotechnical drilling vessels. The Shelby tube, a thin-walled steel tube with a cutting edge, provides high-quality undisturbed samples in low to high strength cohesive soils, including sensitive clays, provided that the soil has not been disturbed by the drilling process. In most cases, Shelby tubes may struggle to retain clean granular soil samples. To address this limitation, certain contractors opt for modified tubes equipped with core catcher systems. Although these modified tubes can cause more disruption to the soil, they enhance the likelihood of successfully acquiring soil samples. In harder ground, alternative techniques like percussive sampling or rotary (e.g., piggy-back) coring may be employed, although they often produce more disturbed samples. Additionally, specialised equipment like hammer sample tubes and liner barrels are employed for granular and extremely low strength cohesive samples, respectively.

The spudcan penetration performance of jack-ups can be notably influenced by minor variations in soil properties. Continuous vertical soil profiles are highly recommended for precise evaluation of these properties. In cases where continuous profiling is not feasible, it is essential to record minimal gaps between data points. The target data gaps should not exceed 0.2 m, although it is acknowledged that achieving this may be impractical for certain geotechnical systems. In such cases, the maximum allowable gap may need to be extended to 0.5 m. Due consideration should be given to the consequence of data gaps in terms of their location and size and how this may influence interpretation of the ground model and the necessity to acquire additional data.

The soil samples are extruded from the sample tubes on the geotechnical SI vessel in the same direction as they were collected to minimise disruption due to stress reversal. Once removed, these samples undergo careful separation from drill cuttings and any significantly disturbed material. They are then meticulously described, photographed, and catalogued in accordance with recommended industry practice (e.g., ISO 19901-8 (2023), ISO 14688-1 (2017), ISO 14688-2 (2017), and ISO 14689 (2017)). Specifically chosen undisturbed samples are securely stored in sealed containers, often filled with wax, to maintain their moisture content and prevent additional disturbance.

Refer to RPS Energy (2011) for comprehensive guidelines and precautions during soil sampling, including considerations regarding data gaps and their impact on interpreting the ground model and the necessity for additional data acquisition.

3.4 Soil laboratory testing

On board the specialist geotechnical SI vessels, a range of different soil tests are available, including routine offshore laboratory index tests. Advanced stress path tests are conducted in specialist onshore soil testing laboratories. Common offshore tests encompass moisture content and density determination, qualitative carbonate content analysis, along with elementary strength tests for fine-grained soils, such as pocket penetrometer, torvane, laboratory vane, fall-cone, and unconsolidated undrained triaxial tests. These index tests aid in classifying the soil type and furnish initial parameters for the preliminary assessment of jack-up foundation.

Although undrained soil strength tests can be promptly and conveniently conducted in offshore laboratories, they evaluate shear strength through varied failure mechanisms. This can lead to notable scatter in strength data and may not precisely reflect sample quality. Certain offshore soil laboratory assessments, such as the fall cone and motor vane tests, can offer insights into soil sensitivity.

Conducting comprehensive soil tests in specialized onshore laboratories is highly recommended. For fine-grained soils, tests such as anisotropically consolidated undrained triaxial compression (CAUC), direct simple shear (DSS), and anisotropically consolidated undrained triaxial extension (CAUE), offer essential data for calibrating in-situ penetrometer factors (refer to Section 3.5.2.1). Additionally, oedometer tests furnish valuable insights into soil compressibility (and in turn stiffness) and drainage behaviour.

Obtaining undisturbed samples of cohesionless soils is rarely feasible. Therefore, it is preferable to ascertain sand density and interpreted strength through in-situ testing, such as cone penetration tests (see Section 3.5.5.1), rather than relying solely on laboratory testing. Where laboratory testing is carried out, the samples should be reconstituted to their in-situ density, and the tests should commence with the samples consolidated to their in-situ stress, bearing in mind their interpreted stress history. This is achievable in consolidated drained triaxial or direct shear tests which are recommended for this purpose. Further, some types of soils (e.g., soft clays, sensitive clays, loose to medium dense sands, silts, and carbonate soils) may undergo a significant degradation of their mechanical properties due to cyclic loading. Therefore, to assess the behavior of soil under cyclic loads, it may be necessary to conduct cyclic simple shear and cyclic triaxial tests (refer to Section 4).

The testing plan for onshore laboratories will be tailored based on the broader SI goals, ground conditions, and the quantity and quality of undisturbed soil samples. The onshore laboratory testing program should be planned and specified by a proficient geotechnical engineer, well-versed in project requirements and objectives. ISO 19901-8 (2023) provides a comprehensive guidelines on the marine soil laboratory testing. It is recommended that all the soil tests conducted for a specific project are conducted in accordance with a single standard to maintain consistency, mostly by following the local/regional standards if available.

3.5 Derivation of monotonic shear strength parameters

The following sections primarily offer guidance on establishing shear strength parameters for clay, sand, and transitional soils displaying intermediate drainage properties. It is important to emphasise that there is no singular "true" soil strength, as the mobilised strength is influenced by the stress path and consequently, the testing method employed (see Section 4.6 for more details). The resulting strength reported can also be affected by variations in sample quality or deviations from the specified test procedure. When utilising multiple test methods, the suitability of soil strength values derived from each method must be evaluated, considering the quality and reliability of the test in addition to the quality and representativeness of the sample used (see Section 3.6 for characteristic values).

The interpretation of shear strength test data should align with the strength relevant for shearing around a spudcan and the specific bearing capacity formulation in use. Moreover, it is essential to account for the differing scales of the spudcan and the testing device.

Figure 3.1 provides a concise overview of various methods to determine the monotonic shear strength of soil. The interpretation of other properties of interest, such as soil sensitivity and consolidation properties, is also discussed in the following sections. These discussions may implicitly address factors like shearing rate, sensitivity, and partially drained response.

3.5.1 Determination of soil type and stratification

To determine the soil stratification, the desk study, geophysical site survey report, and geotechnical testing results should be examined comprehensively and in an integrated manner. As part of this process, the variation of subsoil conditions across the planned jack-up spudcan locations should be assessed. For details on planning the site investigation, please refer to Section 3.2.

Data from CPTu tests, when combined with available soil classification charts, provides a reliable soil profile. Pore pressure has an impact on the measurement of cone resistance as it exerts influence on the outer annulus of the back of the cone, resulting in a reduction in the resistance recorded by the load cell. It is crucial to adjust the cone tip resistance, q_c, for pore pressure effects to ensure precise soil characterisation. Site investigation reports should present the corrected (total) cone resistance, q_t. The net cone resistance, q_{net}, is subsequently determined by subtracting the overburden stress (see Equation 3.1).

Robertson (1990, 2016) charts based on normalised cone resistance, Q $(= \frac{q_{net}}{\sigma'_{vc}})$, and pore pressure coefficient, B_q $(= \frac{\Delta u}{q_{net}})$, or Q and normalised sleeve friction, F_r, can classify sands and clays (where, σ'_{vc} is the effective vertical stress, and Δu is the excess pore pressure). Schneider et al. (2008) enhance classification for transitional soils by considering drainage behavior. Nevertheless, empirical charts should be validated with site-specific geotechnical data (i.e., borehole logs and laboratory test data).

The advancing piezocone test anticipates the effect of a weaker or stronger layer below before the cone tip penetrates the layer. It requires substantial penetration (5 to 10 diameters) into a new layer to achieve a steady state resistance. In contrast, the cone friction sleeve measures the average friction of the material as it traverses through it. This measurement can be useful in delineating layer boundaries. However, pore pressure data is usually considered as more reliable than sleeve friction data for offshore soil classification.

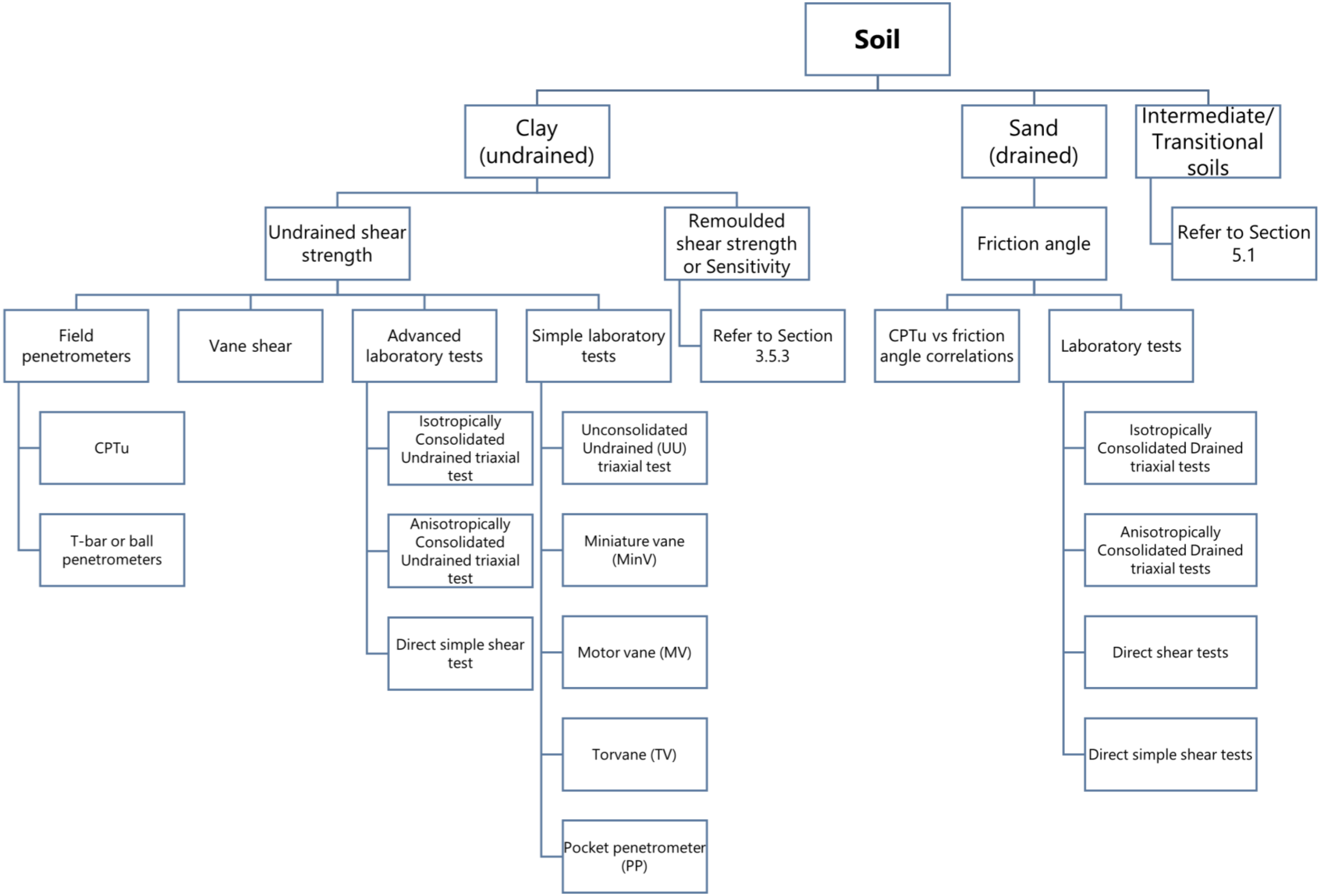

Figure 3.1: A concise overview of various methods to determine the monotonic shear strength of soil

Furthermore, it is feasible to assess drainage conditions during penetration by utilising pore pressure measurements (e.g., dissipiation tests, refer to Section 3.5.4). This includes the identification of partial drainage in intermediate soils using non-dimensional normalised velocity (refer to Section 5.1). Such an approach facilitates the accurate evaluation of representative soil parameters. Intermediate soils typically exhibit an increase in penetration resistance (attributed to partial consolidation) but a decrease in excess pore pressure ratio, B_q. Without detecting partial drainage, there is a risk of significantly overestimating the undrained shear strength of the soil, leading to potential consequences for bearing capacity (load-penetration) predictions (Erbrich, 2005).

Continuous penetrometer profiling is recommended for comprehensive site characterisation, minimising data gaps and improving layer identification. Intermittent profiling with soil sampling can introduce data quality issues and interpretation uncertainties due to soil disturbance. Careful planning and minimising soil disturbance during operations are important in intermittent profiling.

It is also recommended that penetrometer tests:

- be conducted near continuous sampling boreholes, maintaining a separation of approximately 5 m to prevent interference; and
- incorporate pore pressure measurements, as the additional information aids the identification of soil type and layering.

These guidelines are applicable unless a comprehensive ground model has been confidently established using existing data, potentially including prior geotechnical information. Such data should indicate consistent ground conditions without anticipated foundation issues.

The following sections consider monotonic shear strength measurement in:

- Clay: assuming undrained conditions for both testing and spudcan penetration;
- Sand: assuming drained conditions for both testing and spudcan penetration; and
- Silts: encompassing soils with intermediate drainage properties, where both field and laboratory testing (and potentially spudcan penetration) might occur under conditions of partial consolidation.

3.5.2 Undrained shear strength of clay

For the site-specific jack-up analysis in clayey (undrained) soils, essential shear strength parameters include undrained shear strength, s_u, and remoulded shear strength, $s_{u,rem}$ (or soil sensitivity, S_t). The guidelines in this section aim to provide advice on interpretation of these parameters derived from a number of commonly conducted strength tests. The accuracy of these interpreted strength parameters significantly hinges on the quality of both the testing process and the samples. Additional parameters, such as remoulded shear strength and consolidation characteristics, are covered in Sections 3.5.3 and 3.5.4, respectively.

3.5.2.1 Derivation of s_u from field penetrometers

As discussed in Section 3.3.1, CPTu's are widely conducted in the field due to their ability to provide a continuous soil profile. These tests, typically performed with standard cone diameters (35.7 mm or larger) at a standard penetration rate of 20 mm/s, effectively capture clay strength under undrained conditions. The same applies to T-bar and spherical (ball) penetrometers of similar scales.

When interpreting penetrometer data, a vital verification is to ensure undrained conditions, which can be validated by examining the normalised penetrometer velocity $v_n = v \cdot d/c_v$, ensuring it surpasses approximately 10. Here, v is the penetration rate, d is the penetrometer diameter, and c_v is the coefficient of consolidation under vertical conditions, determinable from oedometer tests (as discussed in Section 3.5.4).

For soils of intermediate drainage characteristics, as elaborated in Section 5.1, it is crucial to consider the effect of soil drainage condition on soil strength measurement.

Values of s_u derived from the net cone resistance (q_{net}) through a cone factor, N_{kt}, are given as:

$$s_u = \frac{q_{net}}{N_{kt}} = \frac{q_t - \sigma_{vc}}{N_{kt}}$$

Equation 3.1

Where:

q_t = q_c corrected for pore pressure effects

q_c = Measured cone resistance

σ_{vc} = Total in-situ vertical stress

The N_{kt} factor is not a fixed value but is influenced by stress and strength anisotropy, rigidity index, strain softening, and rate effects. Typically, reported N_{kt} values range between 9 and 20. These values are derived empirically on a site-specific basis and often reflect variations in shear strength measures used for calibration. This variability can be systematic, differing between laboratory or field vane tests, and may be influenced by sample disturbance.

While it is common practice to adopt a range for N_{kt} to accommodate ground uncertainty and heterogeneity, a more precise N_{kt} evaluation can offer a best estimate, particularly for relatively uniform clay sediments. A close estimation of N_{kt} can be achieved by calibrating cone test results against the average strengths obtained from high-quality laboratory tests (e.g., Low et al., 2010) like CAUC, DSS, and CAUE (see Section 3.4). If triaxial testing data is unavailable, calibration can rely on DSS results. This calibration process can be performed at a regional level when consistent ground conditions prevail across the region, eliminating the need for site-specific calibrations. As high quality laboraty data is usually limited and only available at discrete locations, determining the calibrated N_{kt} from high quality laboratory tests will enable to determine the reliable and continuous s_u profile (or s_u profile variation with depth) from continuous CPTu data.

In situations where detailed soil test data is lacking, the precise N_{kt} value remains uncertain. However, RPS Energy (2011) has proposed a recommended N_{kt} value of 13.5±15% (based on Low et al., 2010), where the 15% represents a standard deviation from the mean N_{kt}. This value can be utilized to correlate cone test results with average shear strength (CAUC, DSS, and CAUE) in soft clays, particularly those less than 30 metres in depth or approximately < 50 kPa. It is important to note that this value was derived from a comprehensive global database of high-quality laboratory test results (Low et al., 2010). In cases of stronger or deeper, older sediments with a higher rigidity index (and where high-quality evidence to that effect is available), a higher N_{kt} value might be more suitable (RPS Energy, 2011).

RPS Energy (2011) suggested regional N_{kt} values of 15 to 20 for the North Sea, 15 to 20 for the Gulf of Mexico, and ranges of 12 to 18 or even as high as 15 to 25 for other parts of the world (and dependent on regional geology). Refer to RPS Energy (2011) for further discussion in this regard.

From T-bar or ball penetrometers test results, the s_u values are derived as:

$$s_u = \frac{q_{net}}{N_{T-bar}}$$

Equation 3.2

$$s_u = \frac{q_{net}}{N_{ball}}$$

Equation 3.3

In accordance with RPS Energy (2011) guidelines, suitable penetrometer factors for interpreting a T-bar or ball penetrometer in soft clay have been suggested as N_{T-bar} = 12.0 and N_{ball} = 12.0, respectively (Low et al. 2010). These values were established in relation to the average shear strength (CAUC, DSS, and CAUE) and represent the mean from a comprehensive global database. It is important to note that values ranging from 10 to 14 were identified for the T-bar, indicating regional variations, especially considering strain rate dependency in certain soils. Alternatively, appropriate factors obtained at a regional level, where conditions are consistently uniform across the region, can also be applied. The provided penetrometer factors are relevant only once a consistent penetration state is achieved in layered soils, although this occurs over a relatively short distance considering typical penetrometer sizes (see Section 3.5.1).

3.5.2.2 Derivation of s_u from vane shear

Field vane tests provide discrete measurement of clay strength properties, with intact s_u derived as:

$$s_u = \frac{T_{max}}{\pi d^2 \left(\frac{h}{2} + \frac{d}{6}\right)}$$

Equation 3.4

Where:

T_{max} = Maximum torsional moment

h = Vane blade height

d = Vane blade diameter

Vane shear tests also provide measurement of remoulded shear strength. In onshore practice or laboratory tests, a remoulded shear strength is determined by rotating the vane rapidly for ten revolutions, and then reverting to the original test speed in order to obtain a remoulded strength (by substituting T_{max} in Equation 3.4 by a remoulded torsional moment, T_{rem}). Consequently, this facilitates the assessment of soil sensitivity. In offshore vane tests, it is important to rotate the tool adequately to achieve fully remoulded conditions, where the undrained shear strength stabilizes and does not significantly decrease with continued rotation. This enables the assessment of soil strength sensitivity. For more detailed information on remoulded undrained shear strength, please refer to Section 3.5.3.

It is essential to recognise that the outcomes of vane tests are highly sensitive to the testing procedures, particularly the waiting period before rotation and the rate of rotation. Consequently, these results are heavily operator dependent.

3.5.2.3 Interpretation of s_u based on high quality laboratory testing results

Enhancing the reliability of site assessment involves leveraging high-quality laboratory strength data, which necessitates appropriate testing methodologies such as CAUC, CAUE, and DSS tests. These tests involve reconsolidating soil specimens to effective stresses equivalent to in-situ conditions, and if possible replicating the soil stress history (as stress path influences the soil response), before subjecting them to shearing.

To ensure the accuracy of the results obtained from these tests, the quality of the samples is crucial. It is imperative to evaluate the degree of sample disturbance and identify any inclusions or fabric that could potentially affect the tests. This evaluation can be facilitated using radiographic techniques.

As discussed in Section 3.5.2.1, the combination of continuous penetrometer and high quality laboratory data provides a basis for deriving the cone N_{kt} factor. A meticulous examination of laboratory data and soil characteristics is essential to anticipate potential issues arising from factors like: sample disturbance due to stress relief; the presence of gassy sediments; dubious penetrometer data; or silty material where partial drainage/consolidation might have altered penetration resistance compared to undrained conditions (refer to Section 5.1). If a significant volume of high-quality laboratory tests has been conducted under geologically consistent conditions, these data should be taken into account for determining N_{kt} cone factors.

Moreover, the data obtained from advanced soil testing offers a structured framework to delve into problematic soil conditions, enabling a thorough exploration of secondary characteristics such as rate dependency of shear strength, consolidation parameters, and sensitivity. This comprehensive analysis significantly contributes to refining design predictions.

3.5.2.4 Interpretation of s_u based on standard laboratory testing results

The following are some quick and simple standard laboratory tests typically conducted immediately after the samples are extruded in the offshore laboratory to determine the s_u of clayey soils:

- Unconsolidated undrained triaxial test (UU),
- Miniature vane (MinV),
- Motor vane (MV),
- Torvane (TV),
- Pocket penetrometer (PP)

It is noted that the results from these types of tests often exhibit significant variability both within a specific type of test and across different tests, and their outcomes depend on the operator skill in addition to relying on a large number of results to be able to assess a statistically representative value of undrained shear strength. Such tests might suffice when there is a reasonable understanding of the ground conditions and knowledge of successful jack-up operations in the area (refer to recommendations in Section 3.2). Additionally, this data could be utilized to extend or interpolate results obtained from high-quality laboratory tests. Since the samples are not reconsolidated before testing, the reliability of these tests is affected by the unquantifiable sample quality, the issue of the scale of the sample size relative to the geological feature and the lack of confining stress. Further insights regarding the application of simple laboratory tests are discussed in Section 3.4. For a further discussion on determining s_u using standard laboratory tests, one can refer to RPS Energy (2011).

3.5.3 Determination of remoulded shear strength or sensitivity of clay

Remoulded shear strength ($s_{u,rem}$) is the magnitude of the shear stress that a disturbed soil can sustain in an undrained condition. Understanding the $s_{u,rem}$ and hence sensitivity, S_t $(=s_u/s_{u,rem})$, is essential for evaluating strain softening effects, especially in highly sensitive clays. Strain softening can decrease the average operational strength during spudcan penetration, consequently affecting penetration depth.

Various tests, such as in-situ full-flow penetrometer tests, vane tests (both in the field and laboratory), UU tests, and fall cone tests on remoulded soil, offer methods to measure remoulded shear strength. These tests often yield varying results, influenced by rate effects, spanning from approximately 2 x 10^{-4} s^{-1} for a UU test, 10^{-2} s^{-1} for vane tests with rotation rate of 0.1 °/s, 2 x 10^{-1} s^{-1} for full-flow penetrometer tests, and 2.5 x 10^{1} s^{-1} for fall cone tests. By comparison, typical spudcan installation ratios of v/D are in the range 10^{-5} to 10^{-3} s^{-1} (RPS Energy, 2011).

For instance, in-situ cyclic full-flow penetrometer tests (using T-bar or Ball), involving 10 cycles of penetration and extraction, generally provide a well-defined remoulded penetration resistance. However, the reduction in penetration resistance between initial and post-cyclic states is usually less than the true sensitivity at the elemental test level due to partial remoulding during initial penetration. Nonetheless, these tests offer a relevant measure of $s_{u,rem}$, directly applicable to spudcan performance.

Additionally, understanding the discrepancies in remoulded strength recorded across different tests is essential. Lunne and Andersen (2007) attribute these differences to rate effects. The typical strain rates associated with each test, such as UU tests, vane tests, full-flow penetrometer tests, and fall cone tests, play a crucial role in determining remoulded shear strength. Furthermore, considering the specific conditions of the soil near the vane during testing can provide valuable insights.

For a more comprehensive understanding of determining $s_{u,rem}$, and to explore additional factors influencing these measurements, please refer to RPS Energy (2011).

An indication of relative reliability of clay undrained shear strength and remoulded shear strength measurements, as assessed by RPS Energy (2011), is presented in Table 3.2. Whilst different practitioners may disagree with the relative reliability assessments given in Table 3.2, it serves as a good indicative guide.

Table 3.2: Reliability of tests in measuring strength parameters of clays (RPS Energy, 2011)

Test type	Soil profiling*	Intact s_u (kPa)*				Remoulded s_u*
		< 20 kPa	21 - 40 kPa	41 - 80 kPa	> 80 kPa	
Piezocone	1	2	2	2	2	4-5
T-bar and Ball penetrometers	1 (with pore pressure measurement)	1-2	1-2	1-2	1-2	1-2
In-situ Vane^	-	1-2	1-2	1-2	1-2	3
Unconsolidated Undrained test#	-	4-5	4-5	3-5	3-5	2-3
Motor Vane#	-	3-5	3-5	3-5	4-5	2-3
Torvane#	-	3-5	3-5	3-5	4-5	-
Pocket penetrometers#	-	4-5	4-5	4-5	4-5	-
CIU/CAU/DSS#	-	2	1-2	1-2	1-2	2

Notes:
* Rating: 1 - High reliability; 2 - High to moderate reliability; 3 - Moderate reliability;
4 - Moderate to low reliability; 5 - Low reliability.
^ Based on assumption that the tests are conducted according to standard procedures
# The test result reliability is dependent on the sample quality (or degree of sample disturbance) and soil homogeneity

3.5.4 Determination of consolidation parameters for clay

Oedometer test results offer valuable insights into soil consolidation properties and soil stress history, particularly the vertical consolidation coefficient (c_v) and the yield stress ratio (YSR) as an indication of the over consolidation ratio (OCR). c_v is crucial for evaluating drainage characteristics during spudcan penetration (as discussed in Section 5.1) and approximating in-situ stress conditions in advanced soil testing. Furthermore, understanding consolidation characteristics is vital for analysing spudcan extraction, requiring an estimation of how soil strength changes over time. YSR provides an estimate of the maximum overburden stress that the sample has been subjected to and helps in reconstructing the stress history of the soil and in understanding the mechanical behaviour of the soil.

Dissipation tests, performed using in-situ penetrometers equipped with pore pressure measurement capabilities, provide an estimate of the soil's consolidation coefficient (c_h) in cases where drainage occurs mainly horizontally. These tests typically occur at the conclusion of individual cone strokes within the specific soil layer of interest. Pore pressure changes are monitored until at least 50% of consolidation is achieved. Typically, the values of c_h range from 3 to 5 times the c_v obtained from an oedometer test conducted at the same void ratio (RPS Energy, 2011).

3.5.5 Shear strength parameters for sand

In sands, bearing capacity of a spudcan strongly depends on the adopted soil friction angle, ϕ'.

According to ISO 19905-1 (2023), the apparent friction angle mobilised during spudcan penetration in sand is influenced by:

i. Soil relative density and, consequently, dilatancy: The peak friction angle increases with relative density;
ii. Spudcan size and, consequently, stress level within the failing soil: The peak friction angle decreases as the stress level increases;
iii. Progressive failure: Soil elements at different locations within the failure mechanism experience varying levels of shear strain;
iv. Progressive failure due to pre-shearing of the soil by the conical spudcan tip, leading to a reduction in the mobilised peak strength;
v. Compression of the foundation soil, resulting in additional settlement;
vi. Level of drainage (excess pore pressure development), influencing effective stress and, consequently, soil strength.

The determination of soil friction angle can be conducted through laboratory tests, such as triaxial compression tests. It is essential to perform these tests on samples with the appropriate relative density and stress level, considering the effects mentioned in points (i) and (ii) above. Various methods have been suggested for choosing a representative stress level between the in-situ stress and the (average) foundation bearing pressure. Typically, a stress level around 10% of the bearing pressure is deemed suitable, as outlined in ISO 19905-1 (2023). Alternatively, existing correlations with CPT parameters to infer the soil relative density, from which the peak friction angle can be estimated (ISO 19905-1, 2023).

However, the apparent friction angle mobilised during spudcan penetration is lower than the peak value measured in the laboratory (or inferred using CPT correlations), due to mechanisms (iii) to (v) discussed above. ISO 19905-1 (2023) mentioned that this apparent friction angle is similar to the critical state friction angle, increasing by up to 5° with increasing relative density.

If preloading is performed too quickly for drained conditions to prevail, it can result in the generation of positive excess pore pressures beneath the spudcan, causing a decrease in bearing capacity (mechanism (vi) discussed above). This scenario is of particular significance for skirted spudcans (ISO 19905-1, 2023).

Further, although carbonate sands are not common along the Norwegian Continental Shelf, ISO 19901-8 (2023) recommends considering a reduction of the friction angles in the range of 3° to 7° for both cemented and uncemented carbonate sands when conducting spudcan penetration analysis by applying the conventional general shear failure model.

The following sections provide additional details for the determination of ϕ'.

3.5.5.1 Based on in-situ testing results

Kulhawy and Mayne (1990) proposed the following correlation between peak friction angle and CPTu tip resistance q_c:

$$\phi'_{peak} = 17.6 + 11\, log \left[\frac{q_c \,/\, p_{ref}}{\left(\sigma'_{vc} \,/\, p_{ref}\right)^{0.5}} \right]$$

Equation 3.5

where p_{ref} is the atmospheric pressure (100 kPa).

In accordance with RPS Energy (2011), the application of Equation 3.5 typically results in peak friction angles exceeding 30º for clean sands. However, in silty sands where the normalized cone resistance is lower, the friction angles may fall within the range of 20º to 30º. These lower values are indicative of the soil's high compressibility and/or ground variability, rather than representing a "true" friction angle obtainable through a direct shear test. The recommended approach in such scenarios involves utilizing the "true" friction angles measured in appropriate laboratory tests and adjusting the calculated bearing capacity to accommodate the impact of soil compressibility and/or ground variability.

In order to consider the influence of stress levels on ϕ', it is advisable to estimate the design ϕ' value by correlating it with the relative density (D_r) and the critical state friction angle (ϕ_{cv}). This correlation should be based on a suitable strength-dilatancy relationship that factors in the mean effective stress (p') during bearing failure (RPS Energy, 2011).

Given that the ϕ_{cv}, typically has a narrow range, particularly for silica sand, it is feasible to directly estimate the in-situ D_r from the CPTu q_c (specifically q_{net}, though corrections for pore pressure and overburden stress are generally negligible for sands). Lunne et al. (1997) provide several commonly utilized empirical expressions for inferring D_r from q_c. A widely employed expression, derived from the one introduced by Jamiolkowski et al. (2003) and extensively used in sandy North Sea sites, is as follows:

$$D_r(dry) = \frac{1}{0.0296} \ln \left[q_c \Big/ 2.494 \left[\frac{\sigma'_{vc} \left[\frac{1 + 2.K_0}{3} \right]}{100} \right]^{0.46} \right]$$

Equation 3.6

$$D_r(saturated) = \left[\frac{-1.87 + 2.32 \ln \frac{1000.q_c}{100.\sigma'_{vc}}}{100} + 1 \right] .D_r(dry)$$

Equation 3.7

where, K_0 is the earth pressure coefficient. Defining a reliable estimate of K_0 in sand is inherently difficult. For the determination of saturated D_r from CPTs, a range in K_0 values of 0.5 and 2.0 are normally considered.

Bolton (1986) general strength dilatancy framework can be employed to determine the design value of ϕ' from computed D_r. This framework allows for different sand types and loading conditions. It is expressed as:

$$\phi' = \phi_{cv} + m \cdot I_{RD}$$

Equation 3.8

$$I_{RD} = D_r \left[Q_{crushing} - \ln (p') \right] - 1$$

Equation 3.9

Where:

m = Constant, taken as 3 for failure under triaxial or general loading conditions and 5 under plane-strain conditions

I_{RD} = Relative dilatancy ($0 \leq I_{RD} \leq 4$)

$Q_{crushing}$ = Particle crushing strength on a natural log scale.

The value of ϕ_{cv} can be determined from direct shear tests performed on sand, from the "steady state" friction angle in the later stages of the test (typically at large (>20%) strains). RPS Energy (2011) provides some reported values for ϕ_{cv} and $Q_{crushing}$ and briefly discusses about the effect of p' on ϕ'.

3.5.5.2 Based on high quality laboratory testing results

The primary laboratory tests used onshore for determining effective strength parameters in sand are isotropically consolidated drained (CID) triaxial compression tests and direct shear (DS) tests. Alternatively, direct simple shear (DSS) tests can be considered, especially DSS when advanced strength parameters are required.

Sampling cohesionless materials from the seabed unavoidably leads to sample disturbance. The samples are reconstituted in the laboratory to approximate their in-situ state, with the relative density typically estimated from cone resistance measurements (see Equations 3.6 and

3.7). Subsequently, appropriate effective stresses are applied before the shearing stage. It is crucial that during shearing stage, to ensure a sufficiently slow shearing rate to prevent the development of excess pore pressure, maintaining drained conditions.

The obtained ϕ' from laboratory tests represent a peak frictional strength mobilised under specific loading condition i.e., triaxial condition simulated in CID tests and plane strain condition in DS tests. In theory, under the same stress level, the ϕ' obtained from DS tests, ϕ'_{DS}, is expected to be higher than the ϕ' from CID tests, ϕ'_{CID}.

When selecting the design value of ϕ', it is important to consider the various loading paths within the soil mass during spudcan penetration (see Section 4.6).

3.5.5.3 Effects of compressibility and progressive mobilisation

The aforementioned procedures offer an estimation of the peak ϕ', encompassing the influence of the average stress level within the soil. The published literature clearly suggests that the average shear strength along a slip line decreases with increasing footing size. This indicates an increase in the relative compressibility of the foundation material with increasing footing size. Further, it is important to emphasise that as a spudcan steadily penetrates the soil, the peak strength is not mobilised simultaneously throughout the deforming soil. Consequently, computations of spudcan resistance relying solely on the peak strength of a rigid-plastic soil lead to an overestimation of resistance. SNAME (2008) addresses this concern by employing reduced friction angles. RPS Energy (2011) proposed an alternative approach in which a mobilisation factor is applied to the calculated resistance, refer to RPS Energy (2011) for more details.

3.5.6 Failure criteria

In the absence of peak or residual shear strength in the laboratory tests, the shear stress corresponding to 10% axial strain in triaxial tests or 15% shear strain in direct simple shear tests is considered as the shear strength of the soil. Axial strain is typically defined as the ratio of the change in specimen height during shearing to the initial specimen height, whereas shear strain is defined as the ratio of lateral displacement to the specimen height before shearing.

3.6 Characteristic values

Considering the shear strength parameters are determined from various methods (derived from field tests, laboratory tests and existing correlations), it is ideal to represent shear strength parameters in terms of characteristic values. The characteristic values, developed from statistical analysis, can provide a rational means for deriving shear strength profiles required for predictive jack-up bearing capacity analyses, with the soil layer depths and any lateral or vertical variability within the soil layers identified and incorporated within the analysis. For accurate incorporation of local soil characteristics into the soil strength derivation process, quantitative data, like test reliability (see example Table 3.2) or relative strength variance measurements, can be utilized (e.g., by adopting weighting factors to modify discrete data values). For example, according to DNV-RP-C207 (2021), to convert UU test results to obtain benchmark CIU test results that can be pooled with results from actual CIU tests, it is common to multiply the strength data from the UU tests by a factor. This factor is often set equal to 1.2 (DNV-RP-C207, 2021). However, the manipulation of raw data must be justifiable and approached with caution.

Further, in accordance with DNV (2021), characteristic values of soil properties are used in conjunction with partial safety factors in foundation analysis. The combination of a characteristic value and a partial safety factor forms a pair, and careful consideration is needed when using these pairs for different definitions of characteristic values.

Soil reports often provide lower and upper bound values as characteristic values for design. Lower bounds are primarily intended for ultimate limit state design, where low strengths are unfavorable. In the design of structures subjected to cyclic loading or dynamic behavior, sensitivity studies may be required for both lower and upper bound values of relevant soil properties in supporting foundation soils.

When estimating the distribution of soil strength based on limited test data, the statistical uncertainty of distribution parameters and characteristic values must be considered. Confidence bands on the derived soil strength profile become useful when introducing reliability analysis to spudcan penetration assessment, allowing for possible deviations between field measurement and prediction.

The adoption of a conservative estimate for the shear strength profile, common in other bearing capacity assessments, is not suitable for jack-up installation calculations requiring an accurate estimate of actual penetration. Instead, using confidence bands on soil strength profiles, considering uncertainties in soil strength measurement, proves beneficial for assessment purposes.

Confidence bands should align with the consequences of the final bearing capacity analysis. For instance, in soft clay situations where the final penetration depth is critical (so the jack-up does not run out of leg length), lower than the best estimate profiles of strength are essential. In cases with punch-through potential, worst-case scenarios may involve upper strengths in one layer and lower strengths in the next. Indeed, it is possible that various profiles may need analysis, with each reflecting bounds on the specific problem and its consequences. Additional guidance on the variability of shear strength parameters and their influence on spudcan bearing capacity or penetration analysis calculations can be found in RPS Energy (2011).

3.7 Small strain or maximum shear modulus

The small strain or maximum shear modulus of soil (G_{max}) is required for the foundation stiffness calculations. Small strain or maximum shear modulus of soil can be determined from laboratory tests (e.g., bender element tests), field seismic cone penetrometer tests, geophysical PS logging, or using established correlations between G_{max} and various soil parameters (e.g., plasticity index (I_p), over consolidation ratio (OCR), void ratio, CPT q_c, etc).

For clays, Mayne and Rix (1993) proposed a relationship between G_{max} and q_c, as shown in Equation 3.10.

$$G_{max} = 2.78 \cdot q_c^{1.335}$$

Equation 3.10

Andersen (2015) proposed the following correlations for clays:

$$G_{max} \Big/ s_u^{DSS} = \left(30 + \frac{300}{\frac{I_p}{100} + 0.03} \right) \cdot OCR^{-0.25}$$

Equation 3.11

$$G_{max} \Big/ \sigma'_{ref} = \left(30 + \frac{75}{\frac{I_p}{100} + 0.03} \right) \cdot OCR^{0.5}$$

Equation 3.12

Rix and Stokoe (1991) proposed a relationship between G_{max} and q_c for sands, as shown in Equation 3.13.

$$G_{max} = 1634 \cdot (q_c)^{0.25} \cdot (\sigma'_{vc})^{0.375}$$

Equation 3.13

ISO 19905-1 (2023) provides additional methods/equations for determining G_{max} for both sands and clays, and appropriate stiffness degradation factors under non-linear conditions.

3.8 Cyclic shear strength

The influence of cyclic loads should be considered when analysing their impact on geotechnical parameters. These effects encompass various aspects, primarily involving:

- Changes in shear strength and shear moduli: these changes can occur due to the accumulation of loading cycles.
- Impact of loading rate: the strength and moduli of the soil may be modified in relation to the loading rate.

These modifications are closely related to variations in pore pressures. These combined effects can have a significant impact on the long-term response of spudcans, including cyclic movements, settlements, and horizontal displacements. Additionally, the stiffness of the soil-foundation system plays a role in influencing the natural period and resistance to fatigue of the structure. To fully understand these effects, specific laboratory tests are necessary to assess the cyclic behavior of soils (CFMS, 2019).

In all soil types, cyclic shear strengths may be determined based on total stress analysis for jack-ups under storm conditions. Therefore, the behavior of soil under cyclic loading is evaluated by considering potential loss of undrained shear strength, development and dissipation of excess pore water pressures, cyclic stiffness characteristics, and accumulation of permanent strains within the soil. Section 4 covers the derivation of cyclic geotechnical parameters more in detail.

4. Derivation of Geotechnical Parameters for Cyclic Loads

4.1 Introduction

Cyclic loading generally arises from the effects of wave and wind loads (even from seismic loading, but this is not normally a governing design criterion for Norway), as well as the responses to these environmental forces. For jack-ups, storm induced cyclic loading is usually considered in the engineering analysis. Cyclic loading results in the generation of excess pore pressures, which decrease the effective stresses within the seabed. This leads to the development of both average and cyclic shear strains over successive loading cycles, ultimately resulting in a reduction in the shear strength or stiffness of the seabed sediments. These consequences must be considered when assessing the characteristic soil shear strength for design purposes and when evaluating cyclic and permanent displacements and rotations of foundations. This evaluation is essential when analysing the structural response of jack-up legs and fixation systems (DNV, 2021).

4.2 Characterisation of cyclic loadings

4.2.1 Definitions

For the ideal case of cyclic loadings with a constant amplitude and a constant period (see Figure 4.1), referred as regular loading, it is easy to characterize the loading by means of the following terms:

- τ_a: Average (or mean) shear stress component of the cyclic load
- τ_{cy}: Cyclic shear stress component of the cyclic load
- τ_{min}: Minimum shear stress (= $\tau_a - \tau_{cy}$)
- τ_{max}: Maximum shear stress (= $\tau_a + \tau_{cy}$)
- T: Period of cycles (T = $1/f$ with f = frequency of cycles)
- N: Number of cycles

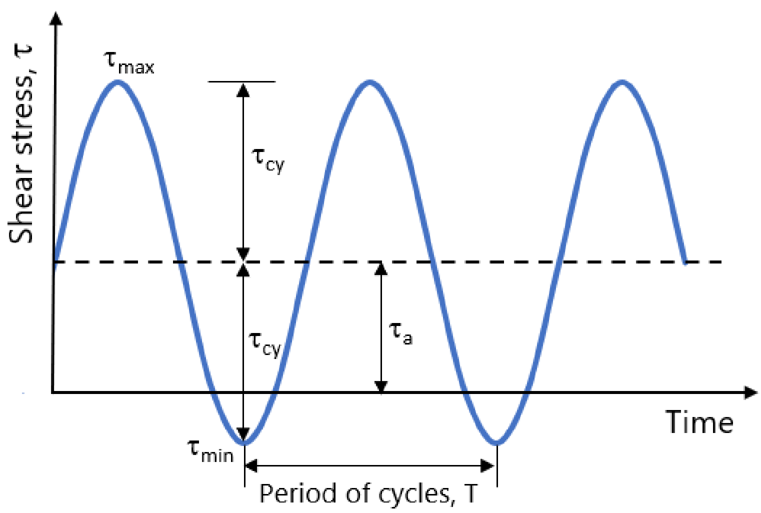

Figure 4.1: Cyclic loading terminology from a constant amplitude and frequency cyclic load

Based on Figure 4.1, one can distinguish:

- one-way loadings, for which $\tau_{cy} < \tau_a$
- two-way loadings, for which $\tau_{cy} > \tau_a$

Moreover, cyclic loading can be classified as either 'symmetric' or 'asymmetric.' 'Symmetric' cyclic loading is a specific type of two-way loading that involves zero mean stress (τ_a) and is often referred to as zero mean stress cyclic loading. On the other hand, 'asymmetric' cyclic loading pertains to cycling with non-zero mean stress and is also known as non-zero mean stress cyclic loading.

4.2.2 Equivalent cyclic loading from actual cyclic loading

For jack-ups that are subjected to significant wave loads, usually the stress history from a single storm, which is then usually the most severe storm that jack-up will be subjected to, or a specified design storm should be considered in the analysis. In the absence of storm data for a particular site, DNV (2021) suggests a storm profile that can be considered in the analysis.

The time-histories of cyclic loads in offshore conditions often exhibit irregular amplitudes and random distributions over time. This is in contrast to controlled laboratory tests that simulate soil behavior under cyclic loading using consistent cycles with fixed amplitudes and frequencies.

During the design phase, a critical task involves converting the actual random loads into regular, standardised ones. This is achieved using cycle counting methods, typically based on "rainflow" analyses, to transform real load histograms into sequences of idealised cycles featuring uniform amplitudes and frequencies.

Subsequently, Miner's Rule cumulative damage concept, established by Downing and Socie (1982), is applied to derive the equivalent cyclic loads from fatigue curves (like Wölher or S-N curves). These curves indicate the number of cycles to failure under constant amplitude stress cycles. Detailed information on this topic is available in SOLCYP (2017), which also discusses the validity of the Miner's Rule hypothesis for soils (independency of the order of application of cycles series and of frequency). Figure 4.2 summarizes the determination of equivalent loading cycles from the actual cyclic loading (CFMS, 2019).

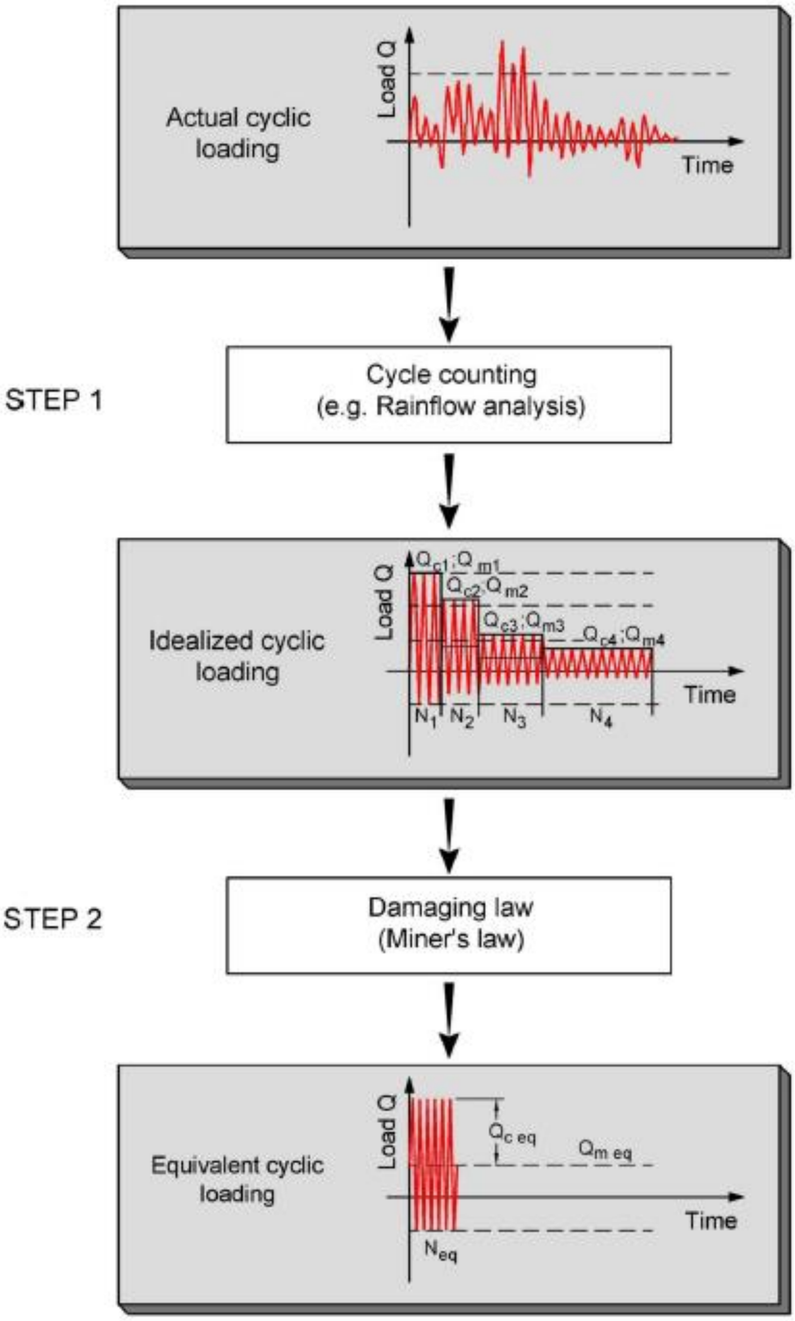

Figure 4.2: Actual cyclic loading to equivalent cyclic loading (SOLCYP, 2017; CFMS, 2019)

4.2.3 Laboratory tests to evaluate the cyclic parameters

The cyclic shear strength typically differs from the monotonic strength of the soil, and its magnitude is influenced by the applied load history, strength degradation resulting from the associated pore pressure buildup, and potential rate effects associated with the frequency of cyclic loading.

Clayey soils are normally considered to respond undrained for all load effects during a design storm and may accumulate "damage" (reduction of soil strength/ stiffness) over the full duration of the storm. However, even in sand, the load duration of a single wave cycle may be so short that the soil will behave undrained during the time the design wave applies, although the soil is likely to be fully consolidated under the applied static deadload. For intermediate soils (i.e. clayey or silty sands/ silts), varying levels of drainage may apply depending on the soil consolidation properties that apply.

In all soil types, cyclic shear strengths may be determined based on total stress analysis, but in more sandy soil types consideration of excess pore pressure accumulation may also be useful as this will allow the effects of partial consolidation from one load cycle to the next to be determined. Thus, the behaviour of soil under cyclic loading is evaluated by considering the potential loss of undrained shear strength, the development and dissipation of excess pore water pressures, cyclic stiffness characteristics, and the accumulation of permanent strains within the soil.

Although storm loading exhibits irregular amplitudes and frequencies, cyclic loading tests with constant stress amplitudes and frequencies are the prevalent method for investigating soil cyclic behavior (see Section 4.3.3 for the determination of equivalent number of loading cycles). These tests involve defining cyclic stress (τ_{cy}) as the stress amplitude and average stress (τ_a) as the mean applied stress around which cyclic loading occurs.

By combining laboratory data from cyclic and monotonic strength tests, it is generally possible to interpolate suitable cyclic strengths for various degrees of cyclic loading bias. Cyclic loading tests are typically conducted at frequencies ranging from 0.05 to 0.1 Hz, reflecting typical wave loading frequencies. These tests are commonly performed using either triaxial or DSS tests (Randolph and Gourvenec, 2011).

The following section provide details on developing the contour diagrams to determine the mobilised cyclic shear stresses, and accumulated shear strains and pore pressures during a design storm.

4.3 Determination of soil cyclic resistance through contour diagrams

Results of cyclic loading tests (either triaxial or simple shear tests) can be used to construct curves depicting cyclic resistance through strain and pore pressure contours.

4.3.1 Strain contour diagram

For soils exhibiting undrained behavior under cyclic loading, a strain contour diagram illustrates the relationship between the number of shear stress cycles, *N*, with a constant shear stress amplitude, τ, required to achieve a cyclic shear strain amplitude, γ. It is a common practice to normalise the shear stress axis of the strain-contour diagram with respect to the s_u.

For instance, Figure 4.3 shows the generation of a strain contour diagram constructed with results from both monotonic and four different undrained symmetric cyclic simple shear tests, conducted at various τ_{cy}/s_u ratios. The number of cycles needed to attain shear strain levels of 0.2%, 0.5%, 1%, 2%, 5%, and 15% is determined from the test data for each cyclic test and presented in Figure 4.3a. These points, representing different shear strain levels, are then connected to form contours of equal shear strain, as depicted in Figure 4.3b. This allows the identification of the number of cycles required to induce a specific shear strain for any value of τ_{cy}/s_u from Figure 4.3b.

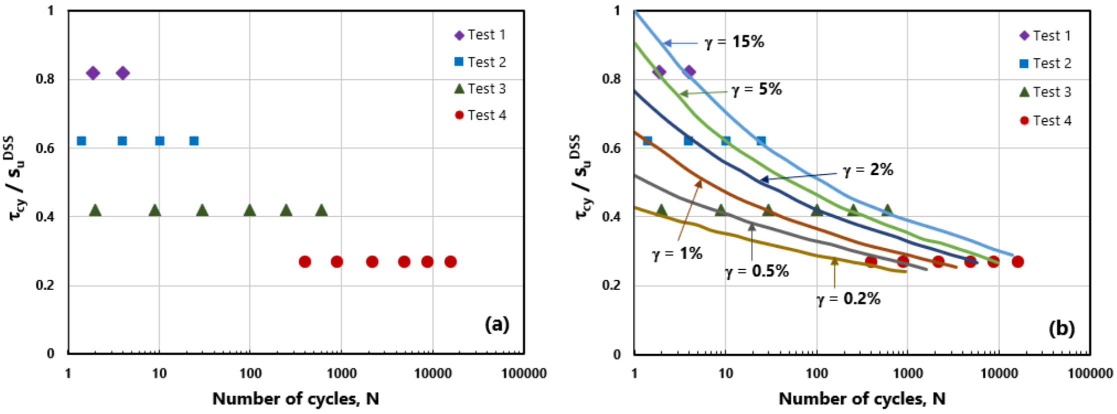

Figure 4.3: Generation of strain contour diagram for τ_a = 0 (modified after CFMS, 2019)

4.3.2 Pore-pressure contour diagram

Pore-pressure accumulation resulting from cyclic loading can be anticipated through pore-pressure contour diagrams. For soils exhibiting undrained behaviour under cyclic loading, a pore-pressure contour diagram indicates the number of loading cycles *N*, needed at a given shear stress amplitude τ, to achieve a predetermined excess pore pressure level, Δu. It is common practice to normalize the shear stress axis of the pore-pressure contour diagram and generated excess pore water pressures with respect to the initial effective vertical stress, σ'_{vc}.

For instance, Figure 4.4 shows the generation of a pore-pressure contour diagram constructed with results from four different undrained symmetric cyclic simple shear tests, conducted at various τ_{cy}/σ'_{vc} ratios. The number of cycles needed to attain normalized excess pore-water pressure ($\Delta u/\sigma'_{vc}$) ratios of 0.99, 0.9, 0.7, 0.5, 0.3, 0.1 and 0.05 is determined from the test data for each cyclic test and presented in Figure 4.4a. These points, representing different $\Delta u/\sigma'_{vc}$ ratios, are then connected to form contours of equal $\Delta u/\sigma'_{vc}$ ratio, as depicted in Figure 4.4b. This allows the identification of the number of cycles required to generate a specific $\Delta u/\sigma'_{vc}$ for any value of τ_{cy}/σ'_{vc} from Figure 4.4b.

To develop a strain or pore pressure contour diagram tailored to a specific sand type, it is crucial to conduct laboratory tests on the sand at a relative density which closely resembles the in-situ conditions. The relative density of in-situ sand can be derived from field cone penetration tests (see Section 3.5.5.1).

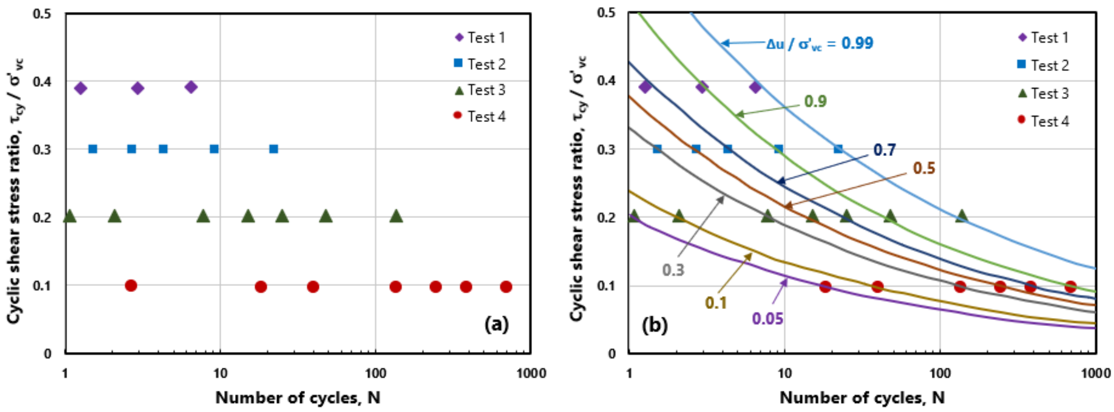

Figure 4.4: Generation of pore pressure contour diagram for τ_a = 0 (modified after CFMS, 2019)

Figures 4.3 and 4.4 consider τ_a as zero. In design scenarios where soil behaviour is influenced by a combination of average and cyclic loading (as in jack-up analysis with cyclic load ratio around 0.1 to 0.2 during storm conditions, see Section 5.2), the cyclic loading may lead to an increase in both cyclic and average strains. In such cases, laboratory tests should be conducted to create strain-contour diagrams that represent different average shear stress conditions. DNV (2021), SOLCYP (2017), and CFMS (2019) offer additional recommendations on the development of contour diagrams.

4.3.3 Procedure for calculation of equivalent number of loading cycles, cumulative strains and pore pressures

The accumulated cyclic pore pressure caused by a particular stress history, for example in a storm, may be determined by application of the pore pressure-contour diagram in conjunction with the pore pressure accumulation method.

Further, as discussed in Section 4.2.2, an "idealised" loading refers to series of cycles with constant amplitudes derived from real loads using counting methods, and "equivalent" loading denotes loads that cause the same damage on the material as the actual loads, as indicated in Figure 4.2. This equivalent number of loading cycles that can cause a similar damage as a particular storm load can also be determined from the pore pressure contour diagram.

The methodology to determine the accumulated cyclic pore pressure and the corresponding equivalent loading cycles for a particular storm loading involves:

- Commencing with the smallest (most frequent) loading level, the excess pore pressure ratio that would generate under that cyclic shear stress is estimated by plotting the data point at the corresponding values of τ_{cy}/σ'_{vc} and the number of cycles;
- The hypothetical contour for that specific excess pore pressure ratio is then retraced (in parallel with the nearest actual contour) to reach the subsequent higher cyclic shear stress level in the storm sequence. This point signifies an equivalent number of cycles at this cyclic shear stress level to produce the same level of damage as the larger number of cycles at the lower cyclic shear stress level;

- The procedure is iterated, with the endpoint from each stage plotted by adding the number of cycles at the new cyclic shear stress level to the (deduced) equivalent number of cycles obtained thus far (from the preceding loading levels);
- The iteration concludes at the peak design load level, and the final point signifies the equivalent number of cycles for that design load level and the specific storm sequence.

The example depicted in Figure 4.5 indicates an equivalent number of cycles, N_{eq} ~ 22, implying that it would require approximately 22 cycles of the maximum wave to generate the same excess pore pressure ratio as that induced by the entire storm comprising varying waves (Randolph and Gourvenec, 2011; CFMS, 2019).

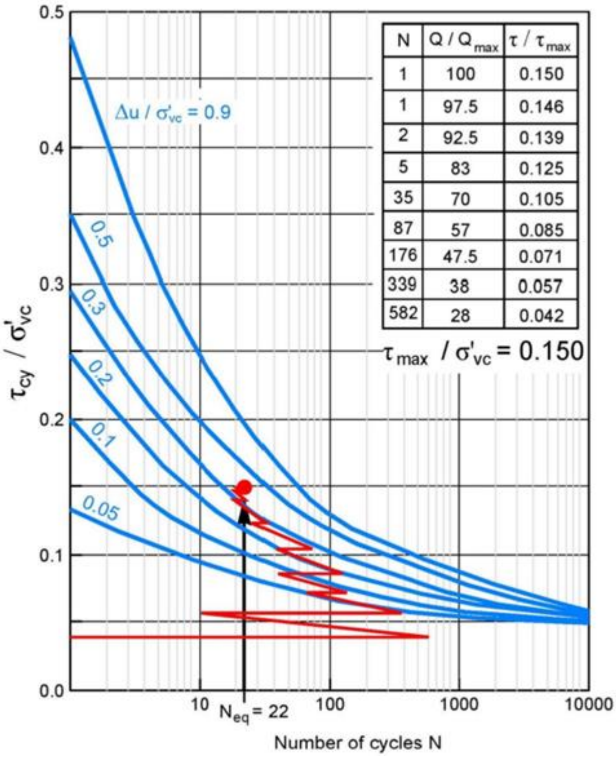

Figure 4.5: Determination of equivalent number of loading cycles and accumulation of pore pressures under cyclic loading (CFMS, 2019)

Similarly, the prediction of accumulated strain in clay resulting from a history of applied shear stress amplitudes, as seen in a severe storm, in essence, can be anticipated using a strain accumulation method. This method is analogous to the pore-pressure accumulation method employed to predict accumulated pore pressures in sand.

In the absence of a relationship between strain, stress and pore pressure (as described in Sections 4.3.1 and 4.3.2), please refer to DNV (2021) for additional guidance.

4.4 Cyclic shear strength, deformations, and failure criterion under cyclic loading

An illustration of a soil response under cyclic loading is shown in Figure 4.6, featuring the common scenario of an asymmetrical undrained cyclic simple shear test.

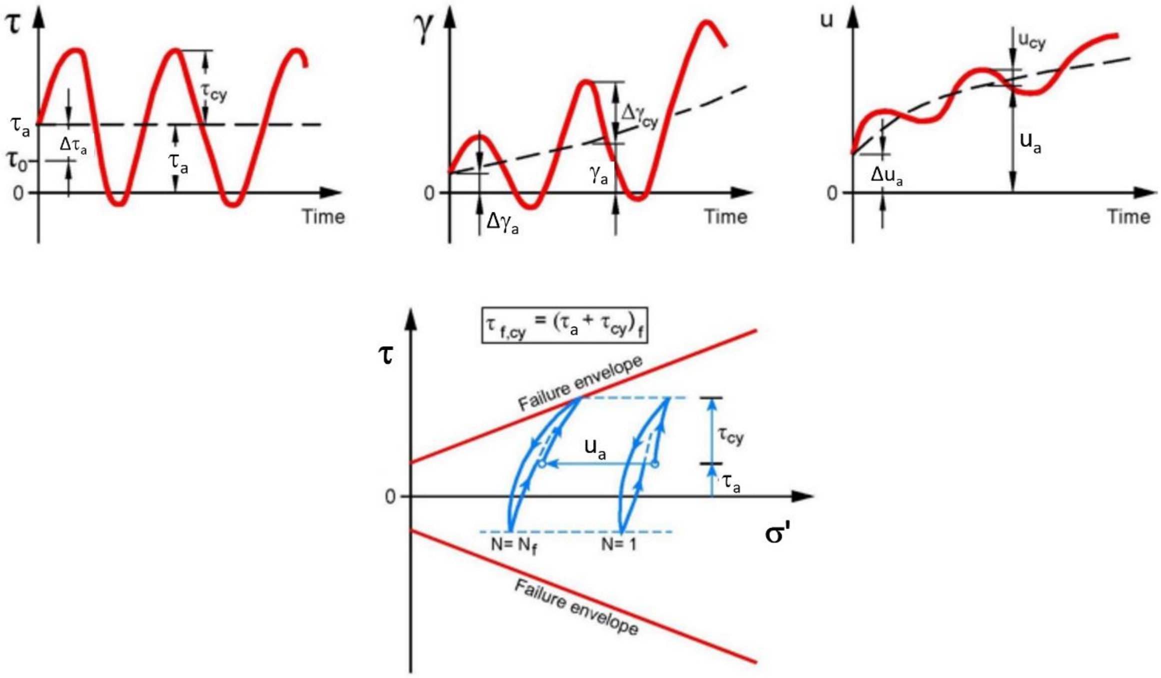

Figure 4.6: Evolution of strains and excess pore pressures with time in a non-symmetrical cyclic simple shear test (CFMS, 2019)

The excess pore water pressure resulting from cyclic loading shifts the effective stress path towards the failure envelope. Once a specific number of cycles ($N = N_f$) is reached, the cyclic failure envelope may be attained, resulting in significant strains. The cyclic shear strength ($\tau_{f,cy}$) is the summation of τ_a and τ_{cy} that have led to failure under a number of cycles N_f :

$$\tau_{f,cy} = (\tau_a + \tau_{cy})_f$$

Equation 4.1

Cyclic shear strength of soil is not constant, and is dependant on:

- The amplitude of mean shear stress, τ_a, and the cyclic shear stress, τ_{cy}, each influencing the development of permanent and cyclic strains differently;
- The loading mode (simple shear, compression, extension);
- The drainage conditions applied to the sample, which can be either fully drained or fully undrained;
- The loading frequency f (or the period T);
- The loading rate, which directly impacts the undrained shear strength of clays;
- The number of cycles N, representing the quantity of cycles characterizing a cyclic event, which can range from just a few cycles to several thousand or even millions of cycles.

4.4.1 Cyclic shear strength of soil

Under cyclic loading, failure can manifest as either signficant cyclic shear strains (γ_{cy}), substantial average shear strains (γ_a) or a combination of both, depending on the combination of cyclic (τ_{cy}) and average shear stress (τ_a).

Strength contour diagrams facilitate identifying the number of cycles leading to failure, and the failure mode. When average shear stresses (or cyclic stresses) begin to surpass cyclic shear stress (or average shear stress), ground failure predominantly occurs due to increase in average shear strain (γ_a) (or an increase in cyclic strain, γ_{cy}).

An example contour diagram of the Drammen clay (OCR=1) obtained from the cyclic simple shear test data and triaxial test data (Anderson, 2015) is illustrated in Figures 4.7 and 4.8, respectively. The normalisation of axes is carried out by considering the s_u obtained from corresponding monotonic DSS and triaxial tests.

From Figures 4.7 and 4.8, the cyclic shear strength ($\tau_{f,cy}$) is the summation of τ_a and τ_{cy} that have led to failure under a number of cycles, N_f, as shown in Figures 4.9 and 4.10 for DSS and triaxial conditions, respectively.

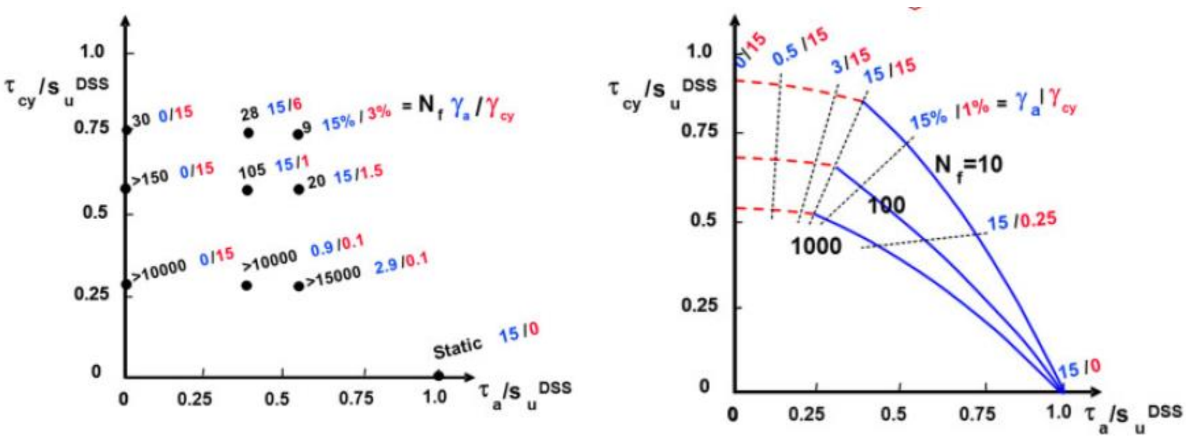

Figure 4.7: Construction of contour diagram with number of cycles to failure as a function of average and cyclic shear stresses by performing cyclic simple shear tests (Andersen, 2015)

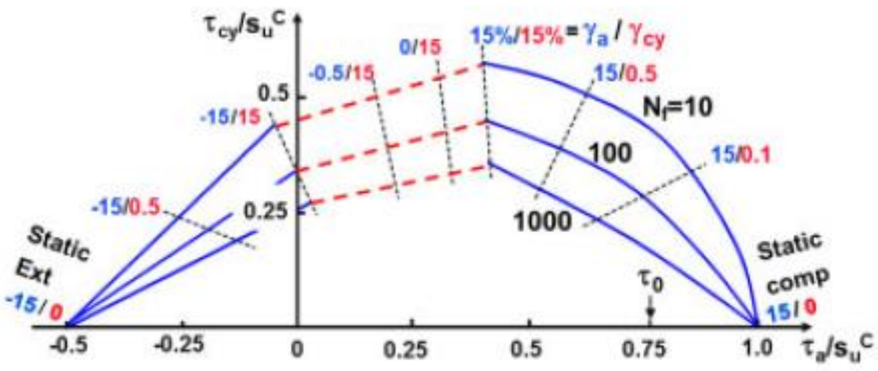

Figure 4.8: Construction of contour diagram with number of cycles to failure as a function of average and cyclic shear stresses by performing triaxial tests (Andersen, 2015)

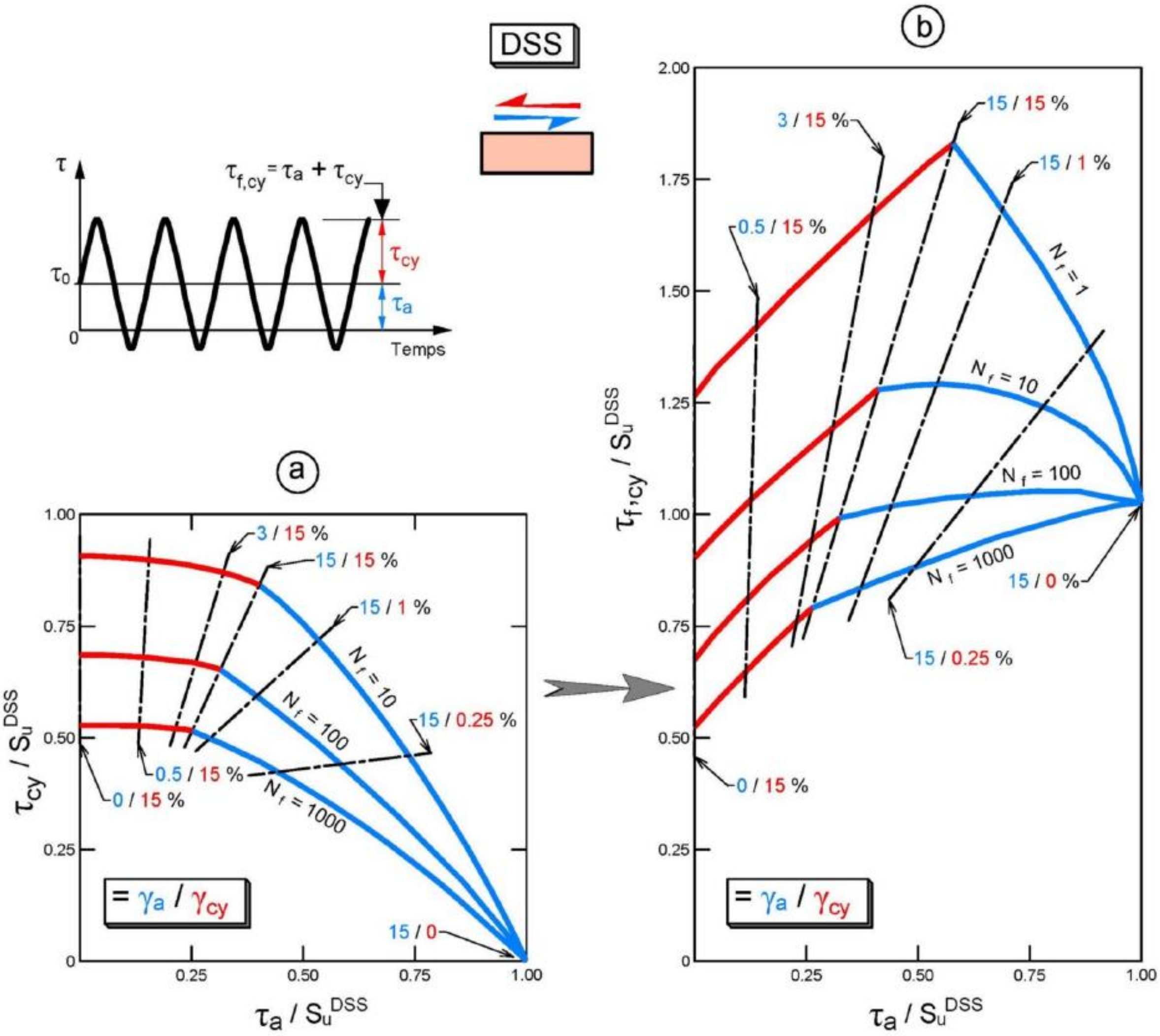

Figure 4.9: Cyclic shear strength from simple shear tests for normally consolidated Drammen clay (Andersen, 2015)

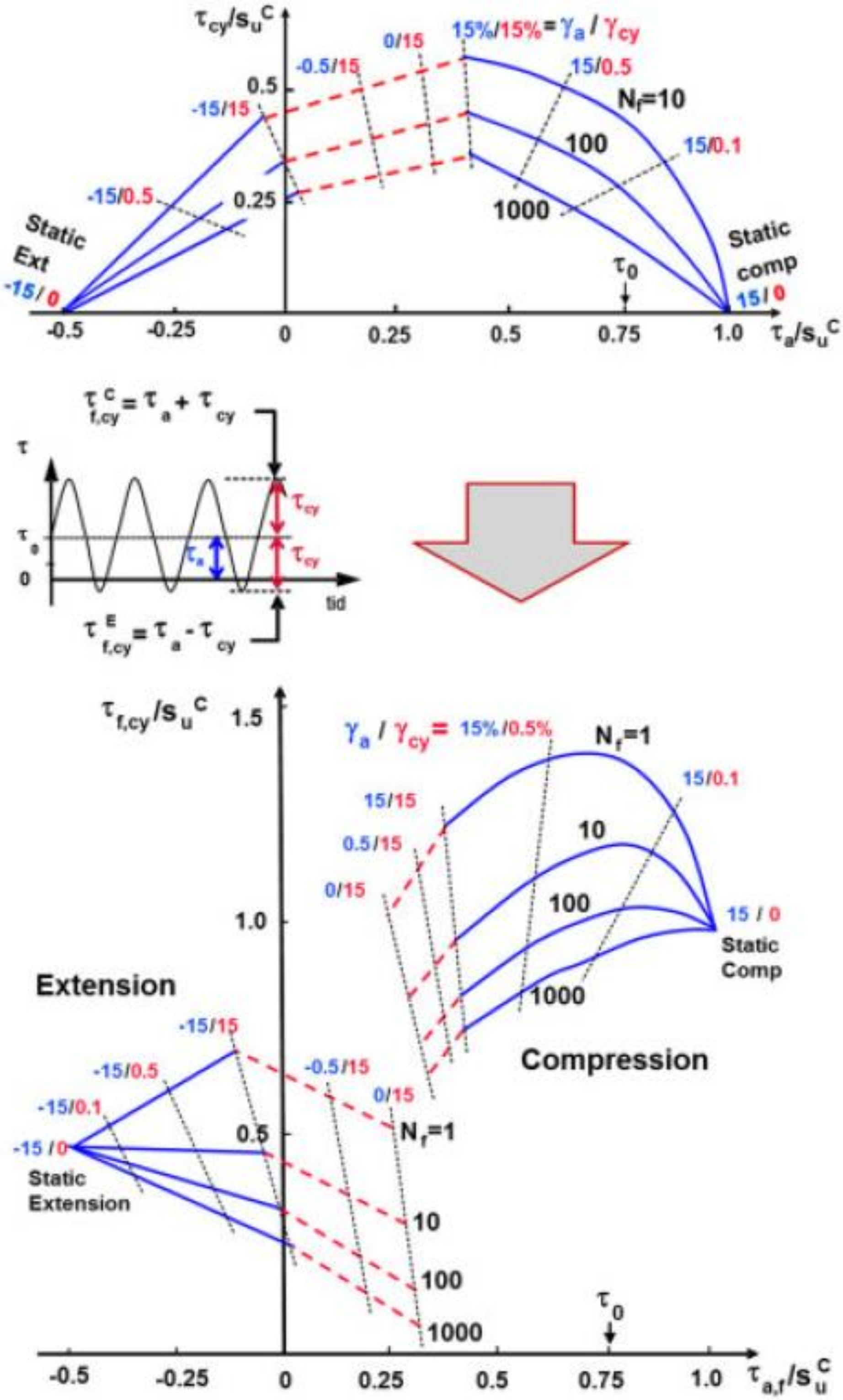

Figure 4.10: Cyclic triaxial compression and extension shear strengths for normally consolidated Drammen clay (Andersen, 2015)

Similar strength contour diagrams can be developed by performing undrained tests on sands. When determining the strength contour diagrams from laboratory cyclic tests, the rate effect is accounted for through the use of a realistic load cycle period in the tests.

4.4.2 Deformations under cyclic loading

Under cyclic loading, the accumulation of permanent shear strains and the dissipation of cyclically induced pore pressures will lead to deformations. Cyclic loading induced deformations can be determined from shear stress-strain and stress-pore pressure relationships determined from the laboratory tests. Figure 4.11 shows example strain contour diagrams (Andersen, 2015) that can be generated from the experimental data to assess deformations under cyclic loading.

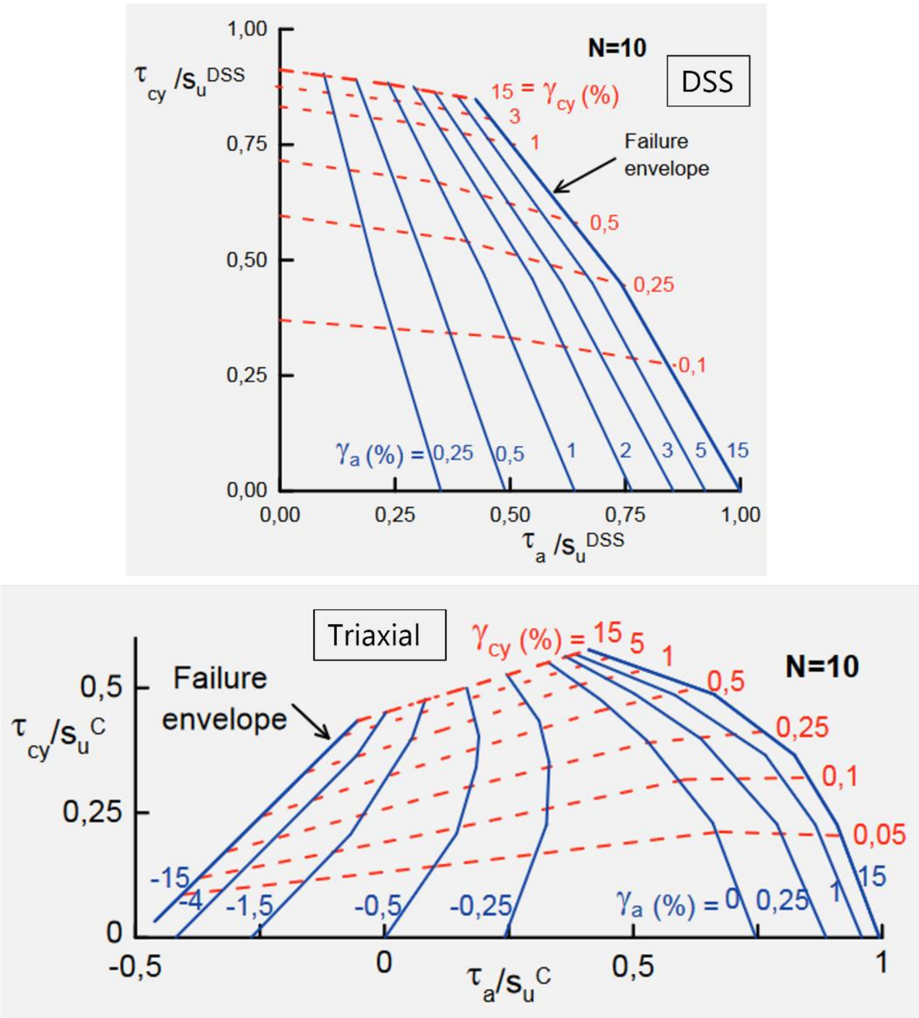

Figure 4.11: Strain contour diagrams to determine the deformations under cyclic loading (Andersen, 2015)

4.4.3 Failure criterion for clays and sands

Under cyclic loading, failure is defined as being reached when the total shear strain amplitude accumulates to a point where it exceeds the designated failure strain. In the context of one-way cyclic loading, failure occurs when the sum of accumulated permanent and cyclic shear strains surpasses the specified failure strain.

The commonly chosen failure strain for soil is a shear strain of 15%. However, for certain soils, a strain less than 15% can be considered as the failure strain.

4.5 Guidelines to establish cyclic contour diagrams

Different practitioners and organisations may follow various methods to generate cyclic contour diagrams, depending on the available soil data, experience of working on the particular site or similar sites, or similar types of soils. An existing data base (e.g., Andersen 2023) can be used in feasibility studies or for generating site-specific cyclic contour diagrams. Andersen (2015) proposed the following approach to generate site-specific cyclic contour diagrams:

- find contour diagrams for a soil similar to the one to be investigated and that covers the relevant parameters for the actual conditions from a database or establish contour diagrams from correlations (e.g., see Andersen 2023);
- perform monotonic test(s) and 3 cyclic tests at various combinations of average and cyclic shear stresses and compare the results with contours in the existing data base;
- If the obtained results do not match acceptably with the existing reference contours of similar soil type, then conduct additional cyclic tests at a different average and cyclic shear stress combination if necessary;
- If the reference contour set needs to be significantly modified, a total of 5 triaxial and 5 DSS cyclic tests, as indicated in Figure 4.12, is probably a minimum. Additional tests may be required when establishing a new set of contours. It is advisable to include tests with both high and low cyclic shear stresses and different combinations of average and cyclic shear stresses (Andersen, 2015).

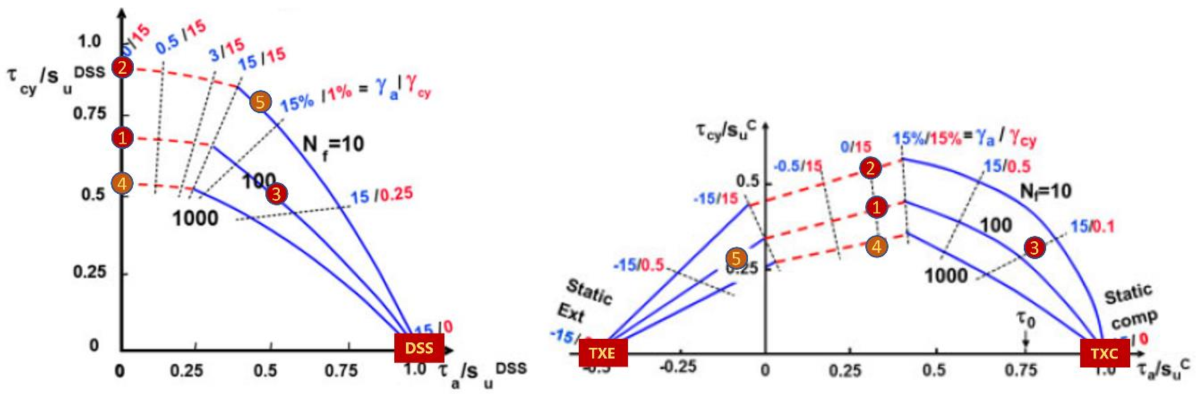

Figure 4.12: Basic guidelines to establish cyclic contour diagrams

Cyclic soil behaviour should be assessed for each different soil type present and should also cover the range of applicable consolidation stress. For each group of tests both monotonic and cyclic tests must be from the same soil unit and be as proximate to each other as possible to minimise scatter. For monotonic tests, it is recommended that for each test group, one test should be conducted at the standard monotonic shearing rate (around 3 - 5% per hour) and a second at a rate of shearing consistent to that applied in the cyclic tests. This helps in constructing contours of cyclic shear stress versus the number of loading cycles and provides insight as to how strain rate effects influence the monotonic shear strength.

For certain cases, the emphasis of contour diagram should be placed on the required combination of average and cyclic shear stresses. For example, for jack-ups with spudcan footings, the typical ratio of cyclic shear stress to average shear stress induced during a storm may be relatively small, typically around 0.1 to 0.2 (see Section 5.2). Therefore, the focus of the contour diagrams or cyclic shear strength should be around the possible field average and cyclic shear stresses.

In general, simple approaches with standard soil investigation and existing cyclic correlations for soils similar to those under investigation at identical in-situ conditions may be used for uniform soils with well-established properties and significant local experience. This is applicable, for instance, in reasonably less problematic soils like uniform dense to very dense sands and highly overconsolidated soils. However, advanced laboratory testing, including cyclic laboratory tests, are recommended in loose to medium dense sands, soft and sensitive cohesive soils, and in intermediate soils (silts, sandy silts, silty sands, etc), as well as in layered soils.

4.6 Soil anisotropy

Soil anisotropy refers to the load direction dependence of soil strength and stiffness properties. Importance of soil anisotropy is particularly pronounced in undrained stability and bearing capacity scenarios, where load-path induced anisotropy impacts the distribution of s_u along the slip surface, determined by the direction of the major principal stress.

In cyclic loading conditions, the variability in the relative magnitudes of average and cyclic shear stresses across the potential failure mechanism influences the available shear strength and the accumulation of shear strain at different points. The diverse stress paths experienced by soil beneath a foundation are depicted in Figure 4.13, illustrating the anisotropic nature of soil shear strengths approximating those measured in triaxial compression (TXC), simple shear (SS), or triaxial extension (TXE) tests.

Measuring undrained shear strength anisotropy in the laboratory generally involves conducting TXC, TXE, and DSS tests, by consolidating specimens anisotropically to appropriate in-situ or consolidated stress levels before shearing. The stress-strain response under undrained TXC, DSS, and TXE conditions is depicted in Figure 4.14, indicating higher small-strain stiffness and shear strength in TXC conditions, followed by DSS and TXE conditions.

While DSS strength data can generally be considered a reasonable average of TXC, DSS, and TXE data in many cases (particularly in clay), this assumption may not always hold, especially for dense silt and sand. The undrained shear strength determined from TXC conditions can be significantly higher than the DSS strength in dense sand and silt, and the strength from TXE conditions can also be higher than the DSS strength. This disparity can profoundly impact the failure mechanism, as well as the capacity and stiffness of a foundation. Designing structures on dense sands and silts solely based on DSS strength, without accounting for anisotropy, may lead to a significant underestimation of both capacity and stiffness (Andersen et al., 2023). The cyclic shear strength of soil under different loading (simple shear and triaxial) conditions is discussed in Section 4.4.1 (see Figures 4.9 and 4.10). Therefore, spudcan analysis should account for stress-path-induced strength anisotropy using different curves for DSS, TXC, and TXE, as shown in Figures 4.9, 4.10, and 4.14.

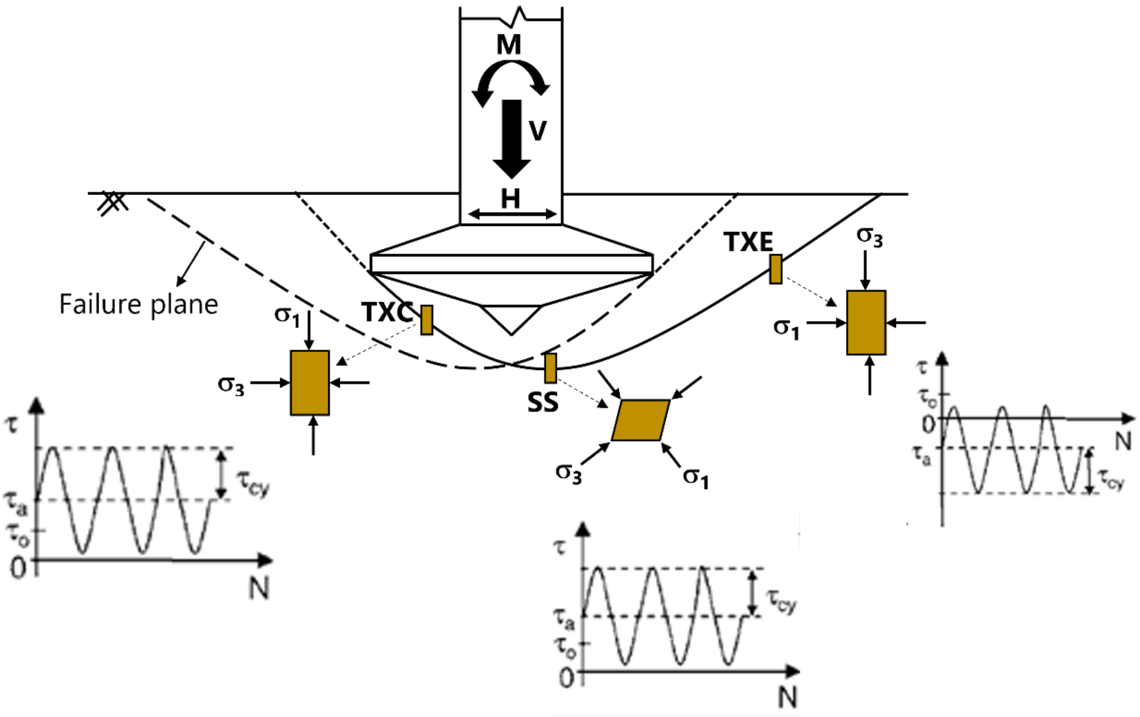

Figure 4.13: Simplified stress conditions along a potential failure surface beneath a shallow/spudcan foundation (TXC = triaxial compression, SS = simple shear, TXE = triaxial extension)

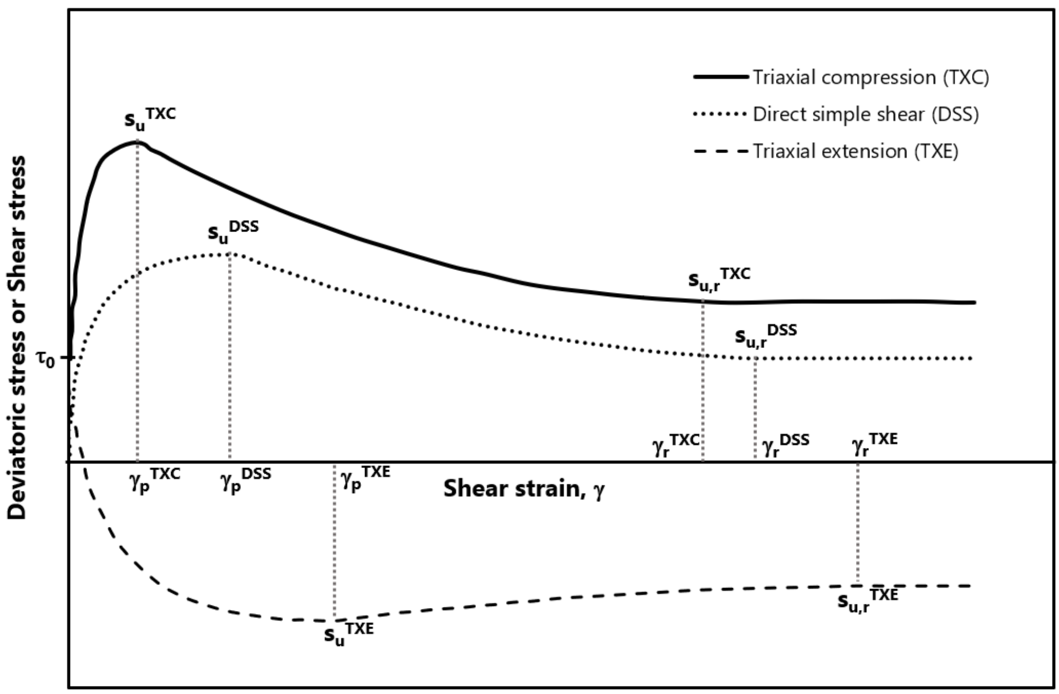

Figure 4.14: Typical stress strain curves of soil from undrained triaxial compression (TXC), direct simple shear (DSS), and triaxial extension (TXE) tests

In simplified approaches, contour diagrams are established for DSS loading, and empirical anisotropy ratios are employed to consider triaxial stress paths. Andersen (2015) and Andersen et al. (2023) provide an anisotropy ratio database for various soil types under static and cyclic loading conditions. These are presented in Tables 4.1 and 4.2 for clays and sands, respectively. These anisotropy ratios are determined for specific soils, therefore must be interpreted cautiously when applyied to different soil types. As Tables 4.1 and 4.2 indicate, the soil anisotropy ratio (in compression) can be high in dense sands.

Table 4.1: Approximate anisotropy ratios for undrained soils/clay (Andersen, 2023)

Loading	OCR	CAUC/DSS		CAUE/DSS	
		Total	Cyclic	Total	Cyclic
Static	-	1.25#	-	0.78#	-
	-	1.45^	-	0.61^	-
	1 - 40	1.45	-	0.78	-
Cyclic	1	1.25	1	0.5	0.65
	4	1.25	1	0.75	1
	40	1	1	0.75	1

Notes:
- Offshore samples
^ - High quality samples

Table 4.2: Approximate anisotropy ratios for sand (Andersen, 2023)

Loading	D_r (%)	Drainage	CAUC/DSS		CAUE/DSS	
			Total	Cyclic	Total	Cyclic
Static	≥ 80	U	4	-	1.1	-
	70 - 80	U	3	-	1	-
	60 - 70	U	2	-	0.7	-
	< 60	U	1.45	-	0.7	-
	All D_r	D+	1# - 2.25^	-	2.25	-
	All D_r	D-	0.45	-	0.2# - 0.45^	-
Cyclic	≥ 80	U	2$	-	1.35$	-
			(1.6 - 2.3)*		(0.6 - 2)*	
	≥ 80	D+	2.5$	2.7	1.5$	-
			(2 - 3.5)*		(1 - 1.8)*	
	≥ 80	D-	1.5$	-	0.6$	1.5
			(1.1 - 1.8)*		(0.4 - 0.75)*	
	80 - 60	U, D+, D-	γ	-	γ	-
	< 60	U	1.25	1	0.5	0.65

Notes:
U refers to $\Delta\tau_a$ applied undrained
D+ refers to $\Delta\tau_a$ applied drained by increasing the normal stress
D- refers to $\Delta\tau_a$ applied drained by decreasing the normal stress
- $K_0' = 0.5$
^ - $K_0' = 1.0$
$ - Best estimate
* - Range
γ - Scale linearly between 80% and 60%

Further, the s_u anisotropy is highest when the clay has a low plasticity, and the soil strength becomes more isotropic with increasing plasticity (Bjerrum, 1973).

In most practical cases, practitioners tend to ignore soil anisotropy effects in spudcan analysis and may use the DSS strength as a reasonable estimate of the average strength. However, as discussed in the above sections, this simplifying assumption may not always be valid, albeit it will generally be conservative. The importance of considering soil anisotropy in spudcan analysis will depend on the anisotropy ratio determined for a specific soil type as ideally obtained from laboratory tests on site specific samples collected in the field. If a simple average of TXC, DSS, and TXE strengths does not seem to provide an appropriately representative strength (e.g. if the TXC strength is much higher than the DSS/ TXE strength) then more advanced numerical analyses that can fully incorporate anisotropic behaviour may be considered (e.g. Jostad et al., 2015).

5. Spudcan Bearing Capacity under Partial Drainage and Cyclic Loading Conditions

During preloading, the soil beneath the spudcan fails as the jack-up leg is loaded until equilibrium is achieved at the end of the preloading operation. Figure 5.1 illustrates diverse failure mechanisms associated with different soil conditions. These mechanisms encompass conventional bearing capacity failure in uniform soils, potential punch-through in layered soils, squeezing, and various combinations of these phenomena.

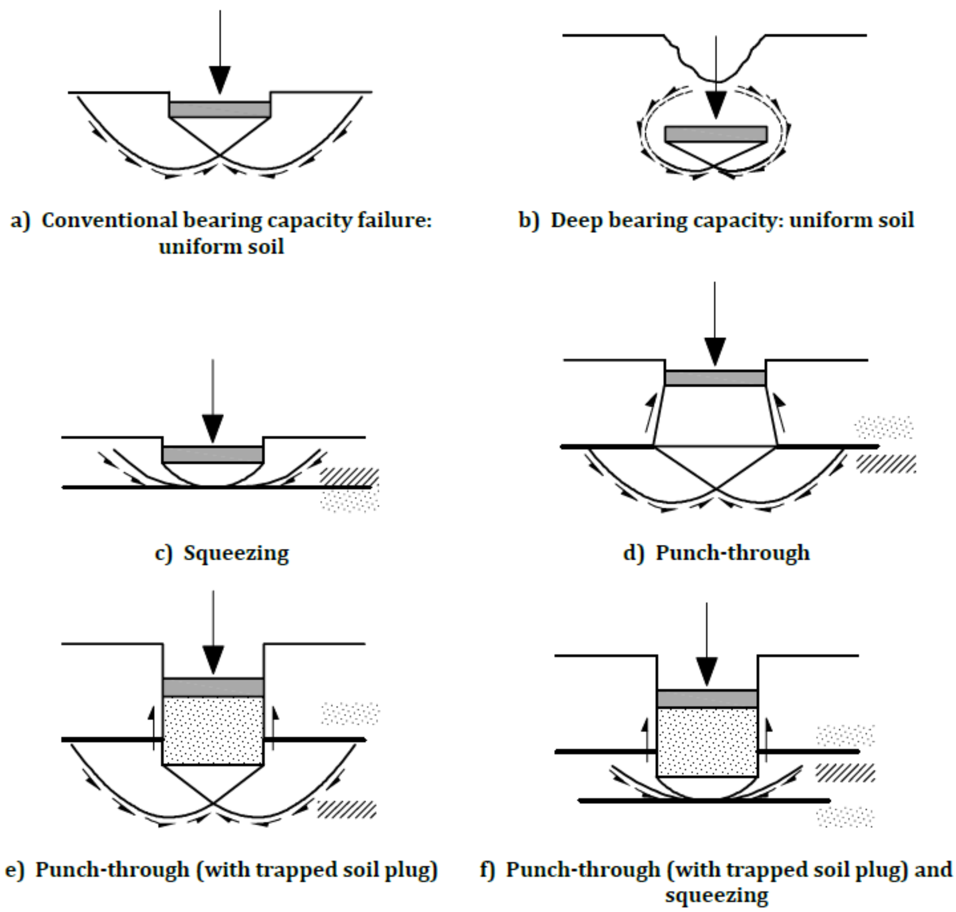

Figure 5.1: Spudcan bearing failure mechanisms (ISO 19905-1, 2023)

Jack-up leg penetration or spudcan bearing capacity analysis for the failure mechanisms shown in Figure 5.1 are covered in ISO 19905-1 (2023). The following sections cover the spudcan bearing capacity analysis in intermediate or transitional soils and under cyclic loading conditions.

5.1 Spudcan bearing capacity in intermediate soils

The behaviour of soil during spudcan penetration when the soil conditions are either fully drained (usually assumed for sand) or fully undrained (usually assumed for clay) is well-established (refer to ISO 19905-1, 2023). However in intermediate soils (silty or clayey sand/ sandy silt) spudcan penetration may occur under partially drained conditions, which complicates the prediction of spudcan penetration resistance in such soils. The degree to which the foundation soil may exhibit partial drainage depends on several factors including the foundation size, loading rate, and the permeability and deformation properties of the soil. Even in the case of sand, which is typically assumed to be fully drained during penetration, partially drained conditions may occur during the passage of large wave cycles associated with a design storm event, which are typically characterized by cyclic periods of around 10-20 seconds. Such waves subject the foundation to cyclic vertical, horizontal, and moment loads simultaneously.

General guidance highlighting the potential influence of partial consolidation during typical CPTu testing, as well as during spudcan penetration and under storm loading is provided in Table 5.1, quantified in terms of the normalised velocity (v_n),

$$v_n = \frac{vD}{c_v} \; or \; \frac{vd}{c_v}$$

Equation 5.1

Where:

v = Penetration rate

D = Diameter of the spudcan

d = Diameter of the penetrometer

c_v = Coefficient of consolidation of soil

Penetration of a spudcan or CPTu is expected to occur in an approximately fully undrained manner for v_n greater than 10, but becomes close to fully drained for v_n less than about 0.1. As shown in Table 5.1, sediments with c_v values ranging from around 2400 m^2/yr up to 250,000 m^2/yr are likely to be partially drained during cone penetration.

The spudcan installation process typically encompasses a variety of penetration rates and hence a potential range in the degree of soil consolidation. RPS Energy (2011) suggest typical penetration rates within the range 0.4 to 4 m/hour. It can be noted from Table 5.1 that this implies a lesser degree of drainage than during a CPTu test, and hence soil with c_v ranging between 10,000 m^2/yr to 1,000,000 m^2/yr (i.e. all intermediate soils and some fine sands) may potentially result in partially drained spudcan penetration (typical spudcan *D*= 10 m to 20 m, we have assumed a nominal *D* = 15 m for these calculations).

During passage of large wave cycles associated with storm conditions the spudcan should only exhibit relatively modest "elastic" movements in response to the application of the design loads, but as noted earlier these would be applied over short cycle periods of 10 to 20 secs. On this basis, the deformation (penetration) rate during application of a single wave load cycle might be expected to be in the order of 5 to 20 mm/s (18 to 72 m/hr). As indicated in Table 5.1, under such loading conditions the soil supporting the spudcans is likely to exhibit fully undrained behaviour for c_v less than 150,000 m^2/yr to 250,000 m^2/yr (i.e. any intermediate soil type), and partially drained conditions for c_v to at least 5,000,000 m^2/yr (i.e. well into the clean sand range). Notwithstanding, it should be appreciated that the above calculations only consider application of an individual 'large' wave load cycle. Whether undrained or partially drained cyclic loading effects also need consideration or not will depend on how high v_n actually is. For most sands, v_n will likely be in the partially drained range for a single cycle, and in such cases cyclic degradation (caused by the soil softening induced by the accumulation of excess pore pressures) is unlikely to be significant. However, in most intermediate (and finer) soils, v_n is likely to be well above the undrained limit and hence pore pressures may accumulate from load cycle to load cycle.

Table 5.1: Normalised velocity values for CPTu and Spudcans in different soil types

Soil type	c_v (m^2/yr)	$v_n = vD/c_v$ or vd/c_v		
		CPTu* d = 35.7 mm	Spudcan penetration# D = 15 m (nominal)	Spudcan under storm conditions^ D = 15 m (nominal)
Clay	< 100	> 10.0	> 10.0	> 10.0
Silty clay to silty sand	100	> 10.0	> 10.0	> 10.0
	1000	> 10.0	> 10.0	> 10.0
	2400	9.2	> 10.0	> 10.0
	5000	4.4	> 10.0	> 10.0
	10000	2.2	5.2 - > 10.0	> 10.0
	30000	0.7	1.7 - >10.0	> 10.0
	50000	0.4	1.0 - > 10.0	> 10.0
	150000	0.15	0.34 - 3.4	> 10.0
Sand	250000	< 0.1	0.21 -2.1	9.3 - > 10.0
	1000000	< 0.1	< 0.1 – 0.5	2.3 - 9.3
	5000000	< 0.1	< 0.1	0.5 - 1.9
	>2000000	< 0.1	< 0.1	< 0.1 – 0.5

Notes:
* - Calculated based on v = 20 mm/s;
# - Calculated based on v = 0.4 m/hr to 4 m/hr;
^ - Calculated based on v = 18 to 72 m/hr;

The aforementioned results imply that undrained bearing capacity theory is appropriate for predicting spudcan penetration during installation in many intermediate soils and for assessing the spudcan bearing response in all such soils (and many sands) during storm loading (with or without extra softening under cyclic loading).

Fully drained bearing capacity theory is strictly only appropriate for assessing the spudcan penetration response during installation for sands with c_v > 1,000,000 m^2/yr to 5,000,000 m^2/yr.

Where undrained or partially drained soil conditions are expected an assessment should be made of the appropriate undrained shear strength. It is recommended that this is achieved by taking samples of the soil and conducting laboratory tests under representative stress conditions of initial in situ stress state to determine the monotonic undrained shear strength.

The variation of spudcan bearing capacity with v_n is represented in simplified form in Figure 5.2. In most cases the drained bearing resistance will be significantly greater than the undrained resistance, with the partially drained condition falling inbetween (except in the cases of dense sand at low confining stresses). The drained ($q_{drained}$) and undrained ($q_{undrained}$) spudcan bearing capacities can be determined based on conventional theory (as per ISO 19905-1, 2023), using friction angles and the laboratory-determined undrained shear strength of the soil, respectively. It is then possible to interpolate the bearing capacity for the appropriate degree of partial drainage ($q_{par\_drained}$) based on the estimated applicable range of v_n.

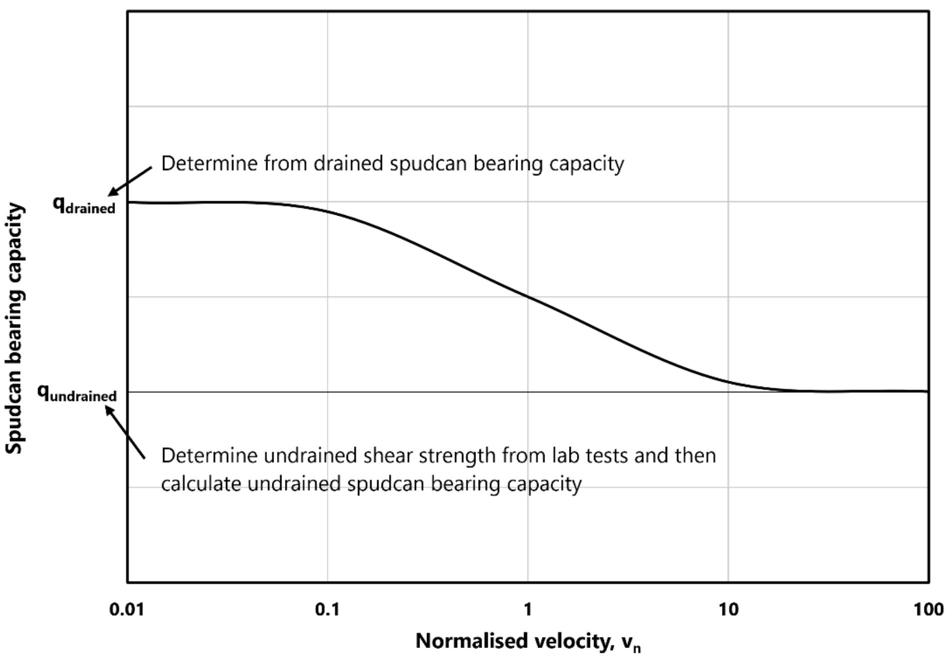

Figure 5.2: Variation of spudcan bearing capacity with normalised velocity

The extent of drainage during cone penetration can be quantified in terms of a consolidation index, denoted as *CI* and represented as:

$$CI = \frac{q_{par\_drained} - q_{undrained}}{q_{drained} - q_{undrained}}$$

Equation 5.2

The relationship between *CI* and the consolidation coefficient can be expressed in terms of v_n (see Equation 5.1). According to House et al. (2001), the variation of *CI* with v_n may be expressed as:

$$CI = \frac{1}{1 + (v_n/v_{50})^c}$$

Equation 5.3

where, v_{50} is normalized velocity for *CI* = 0.5 and *c* is an adjustable power that ranges from 1 to 1.5.

For v_{50} = 1 (which is recommended), the variation of *CI* with v_n is shown in Figure 5.3.

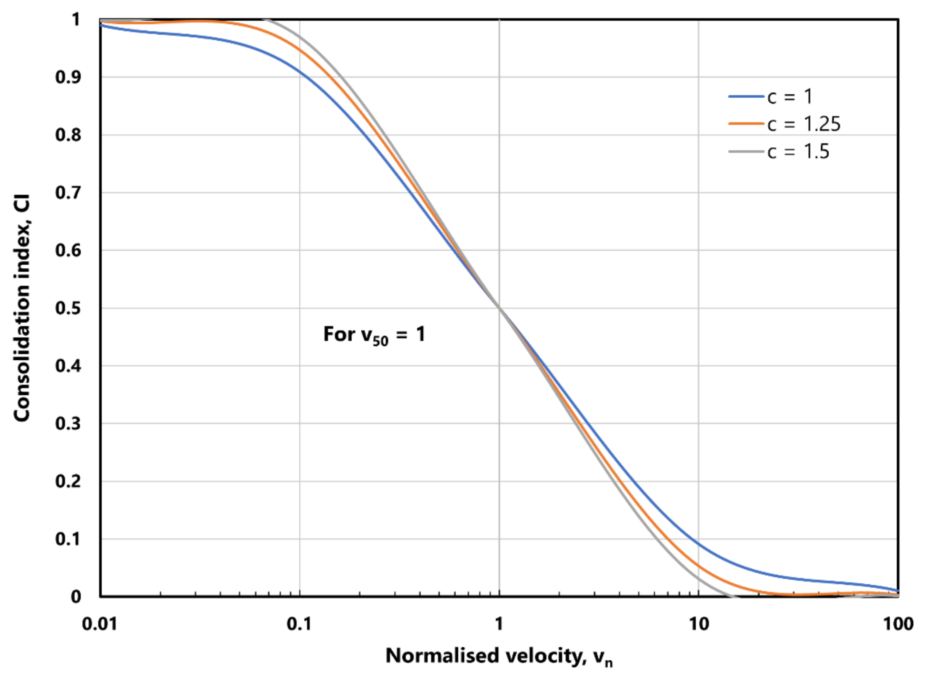

Figure 5.3: Variation of consolidation index with normalised velocity for v_{50} = 1

For known values of $q_{drained}$, $q_{undrained}$, v_n and assuming v_{50} =1, *CI* can be computed using Equation 5.3 and then the spudcan resistance determined using Equation 5.4. For this assessment it is suggested that a range of *c* be considered between 1.0 and 1.5.

$$q_{par\_drained} = [CI \cdot (q_{drained} - q_{undrained})] + q_{undrained}$$

Equation 5.4

5.2 Spudcan bearing capacity under cyclic loads

Most jack-up units used in oil and gas operations impose large vertical stresses, via the spudcans, onto the underlying soil. As a result, the typical ratio of cyclic shear stress to average shear stress (τ_{cy}/τ_a, referred to as cyclic load ratio, CLR), induced during a storm may be relatively small (e.g. 0.1 to 0.2) as compared to many other types of offshore foundation system.

Figure 5.4 shows the typical variation of cyclic shear stress with time for τ_a significantly greater than τ_{cy}.

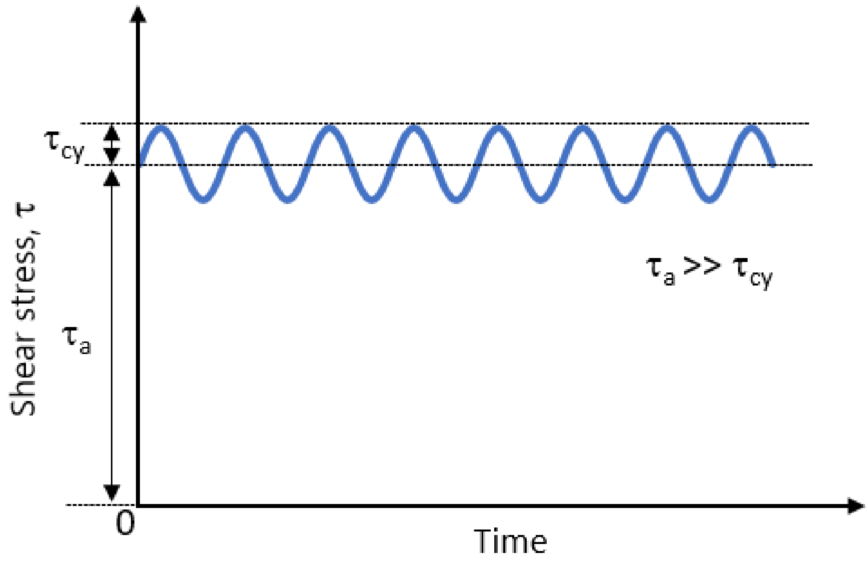

Figure 5.4: Typical asymmetric (equivalent) cyclic loading that the jack-ups will be subjected to during storms

During storm or significant cyclic loading conditions, preventing 'cyclic failure' is crucial to avoid catastrophic failure of jackup foundations, marked by uncontrollable and progressive settlement. Relying solely on static pre-load for determining the 'vertical bearing capacity' becomes unreliable in such scenarios. Under undrained cyclic loading, a lower vertical load might trigger cyclic failure.

For soils prone to cyclic degradation, Erbrich (2005) proposed an approach illustrated in Figure 5.5 to ensure adequate jack-up stability under storm conditions. Figure 5.5a denotes the depth at which in-situ monotonic soil strength (see Section 3.5) first supports the jackup preload, while the circular symbol indicates the available cyclic soil strength at that depth (see Section 4.4.1). In Figure 5.5b, based on the monotonic strength the 'yield envelope under monotonic loads' indicates the limit of the range of vertical/ horizontal load combinations within which no further penetration will occur. Under cyclic loading, Erbrich (2005) recommends maintaining the yield envelope but anchoring it with a calculated 'cyclic vertical bearing capacity' instead of the applied static preload (Figure 5.5b), termed as the 'equivalent preload'. The corresponding cyclic yield envelope is labeled the 'degraded cyclic yield envelope.' The 'safe zone,' determined by applying a material factor to the cyclic yield envelope, is delineated by a broken line in Figure 5.5b. Erbrich (2005) recommends adopting conventional material factors for shallow foundations, i.e., between 1.25 and 1.3, as there is effectively no-load testing of the spudcans under cyclic loading conditions.

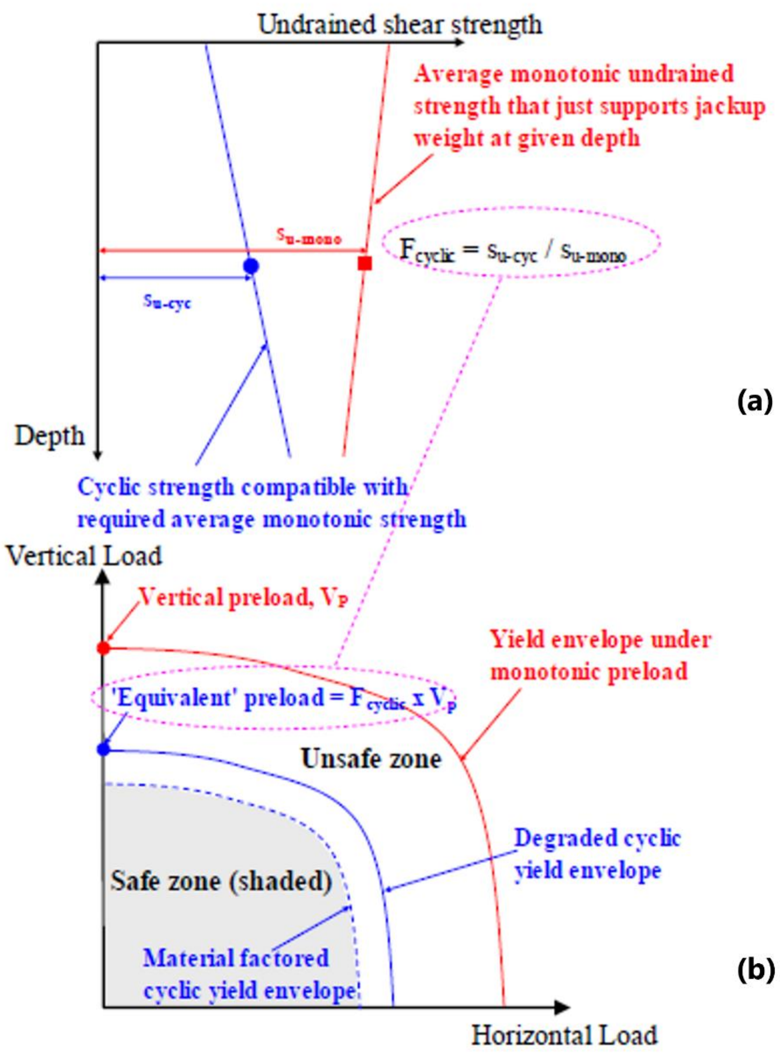

Figure 5.5: Modified yield envelope approach for cyclically de-gradable soils (Erbrich, 2005)

5.3 Other issues

This document focuses only on specific aspects as requested by PSA. Critical considerations such as scour assessment, spudcan eccentricity, ground preparation methods, geohazards, leg extraction issues, spudcan-footprint interaction, etc., play a crucial role in evaluating the capacity and stability of spudcan foundations. Therefore, this document should be regarded as guidance only for the covered topics. However, in practice, all issues related to spudcan capacity and stability must be addressed by following various existing standards, such as ISO 19905-1 (2023) and DNV (2021). Moreover, foundation or spudcan assessment is one among many issues that need to be considered in site specific jack-up assessments. DNV-OS-C104 (2022) and DNV-RP-C104 (2022) provide additional guidance (technical principles and requirements) for executing site-specific assessments for elevated conditions.

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