ADVANCED SITE-SPECIFIC ANALYSIS OF SKIRTED SPUDCANS IN THE VIEW OF NORTH SEA EXPERIENCES

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ABSTRACT
Depending on the soil profiles and service conditions, site-specific assessment of jack-up platforms may require detailed and advanced geotechnical foundation assessments. In these assessments, foundation fixity or global soil spring stiffnesses play a significant role on dynamic behaviour and structural utilization of jack-up platforms, especially at large water depths. In order to establish reliable values for this fixity, it is vital to consider the complex non-linear behaviour of the different soil layers involved under combined average and cyclic loading. This paper summarizes the challenges and possible solutions of site-specific foundation assessment of jack-up structures with skirted spudcans, focusing on the estimation of monotonic and cyclic soil parameters, foundation penetration, capacity, dynamic and quasi-static stiffnesses, damping, earthquake analyses, large deformation effects, rate effects in the view of North Sea experiences.

KEY WORDS: Skirted spudcan, site-specific assessment, capacity, fixity, damping, advanced finite element analyses.

INTRODUCTION

Jack-up rigs are commonly deployed as mobile units for a wide spectrum of offshore activities, ranging from exploration and production of hydrocarbons to installation of wind turbines and their foundations. Site-specific assessment of the jack-ups is required as recommended in ISO [1]. ISO [1] specifies requirements and guidance for the site-specific assessment of independent leg jack-up units, which should include both extreme events and operational conditions. Proof of the robustness of the jack-up during the design storms makes it often necessary to take moment fixity of the footings into account in order to reduce the critical moment in the lower leg guides, reducing the inertia forces (i.e. reducing the natural period of the platform) and thus reducing the second-order moment in the leg (P-Δ effect). Footings with skirts that penetrate into the soil generally improve the moment fixity and bearing capacity compared to standard spudcans. However, from the assessment point of view, [2] has shown that the design soil parameters and related capacity and stiffness of skirted spudcans should be carefully considered as the simplified methodology described in ISO [1] is not always conservative, for instance for highly overconsolidated clays.

This paper presents the challenges and possible solutions for the advanced site-specific foundation assessment of skirted spudcans of jack-ups in the view of North Sea experiences in particular focusing on soil parameter assessment and advanced finite element analyses.

ASSESSMENT OF SOIL PARAMETERS

Assessment of soil parameters is key to a reliable geotechnical assessment of jack-up foundations. For reducing the uncertainties in the estimated foundation capacities and fixities, location-specific soil investigations are vital. Indeed, certifying authorities require a site-specific assessment of the jack-ups to reduce uncertainties and assure robustness of the jack-up structures [3]. In special cases, e.g. where the legs are situated in different soil units, it may be beneficial to perform leg specific analyses. It may be relevant to consider a predominant loading direction in addition.

This section summarizes different aspects of soil parameter assessment from monotonic to cyclic, highlighting challenges and possible solutions. In addition, some background on the cyclic accumulation and cyclic contour diagram concepts [4] is presented.

In-situ and lab measurements
Soil investigations are the key to geotechnical assessments. It is important to have a good coverage of the soil volume affecting the foundation response with high quality and relevant field and laboratory measurements, especially for advanced analyses. For sites with significant spatial variations in layering and soil properties, the uncertainty levels may become significant and need to be carefully considered in the assessment. Laboratory tests on undisturbed samples should be used to establish a reliable CPT interpretation framework, especially for low permeable dilative soils, e.g. OC clays and (dense) silts.

In the presence of OC clays and (dense) silts, it should be considered that the stress-strain behaviour of these soils is characterized by dilatancy. Further, these soils generally show a fairly stiff response during monotonic loading, while the behaviour becomes softer during cyclic loading. The behaviour is strongly affected by the content of fines, water content and stress history [4].

Cyclic laboratory tests can be costly but are quite valuable since they may reduce the uncertainties, with the condition that they are performed on high-quality samples and following anticipated stress paths (as illustrated in *Figure 1*). The location and number of samples should be carefully selected. The missing info can be estimated by employing published databases [4].

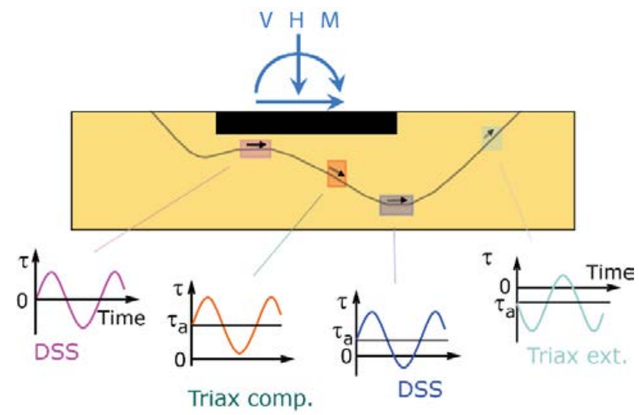

Figure 1 Stress paths beneath a foundation under combined HVM-loading [4]

Monotonic parameters
Finite element analyses have shown that the behaviour may be strongly dominated by the weakest zone near skirt tip level, as opposed to the concept assuming an average strength in a zone down to the spudcan diameter. As a result, lack of data at these shallow depths, especially for OC clays and silts, brings the challenge of extrapolating design shear strength profiles to skirt tip level and seabed. For example, the cone factor N_{kt} used for assessing the undrained shear strength of the soil can be quite high for high OCR clays, which is generally difficult to estimate. In dilative soils, the laboratory strength is generally selected at a threshold strain level (i.e., 10% axial strain in triaxial or 15% shear strain in direct simple shear tests), if the peak shear strength has not been reached before. However, this threshold strain level may be far lower than the strain level at which the maximum cone resistance is reached during CPT testing. The mechanism is schematized in *Figure 2.* Therefore, the selection of N_{kt} should take this aspect into account. For sites where the OCR varies significantly with depth, the SHANSEP approach [5] can be employed to combine the OCR data and shear strength data for a more reliable N_{kt} assessment and hence, more reliable shear strength profiles, as suggested by [6]. This will result in a variable N_{kt} with depth, as shown

in *Figure 3,* compared to the more classical constant-N_{kt} (with $N_{kt} = 15 - 20$) approach which does not take the stress-history of the deposit into account. Assuming a constant N_{kt} value may overestimate the design shear strength at shallow depths where OCR may be high.

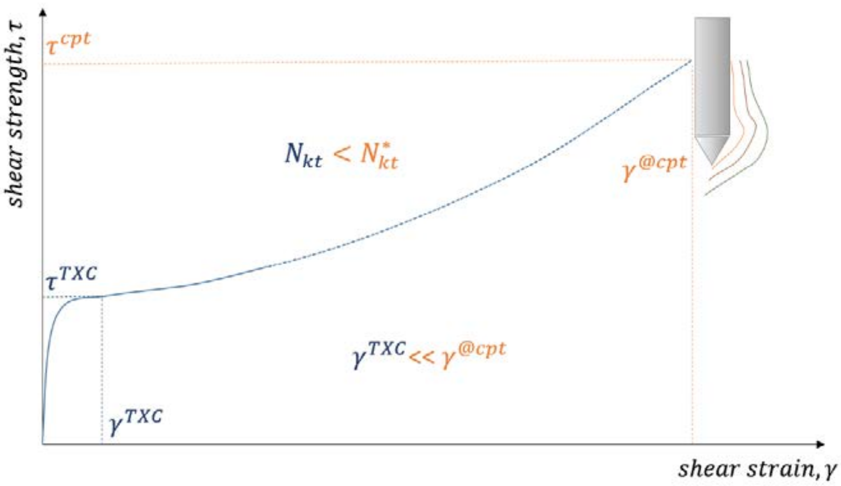

Figure 2 Large shear strains around a CPT as an explanation for the high N_{kt} factors for dilative soils.

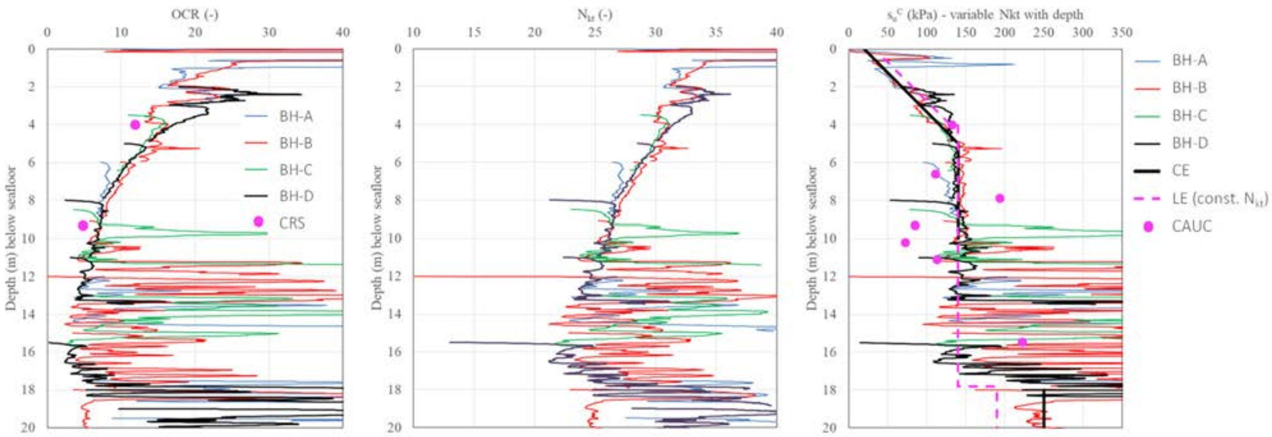

Figure 3 Example characteristic static triaxial shear strength assessment.

Similarly, standard assessment of OC sands based on CPT generally results in overestimated relative densities. Hence, one should consider these issues in the assessment, as the relative density is linked to the friction angle of the sand and hence to the shear strength [4].

Establishing characteristic design profiles can be challenging considering statistics since quite limited laboratory data is available in the majority of the site investigations. Generally, engineering judgement is required for establishing the design profiles.

In advanced foundation analyses, it is important to assess the effect of stress-path dependency, which is generally referred to as strength anisotropy. However, soil investigations generally lack sufficient test data at representative locations and depths. It is then common to perform different tests (e.g. DSS, CAUC and CAUE) on samples from different boreholes and depths and combine them to characterize a given soil unit.

Since jack-ups are normally preloaded during installation, the effect of preloading on the soil strength and stiffness parameters should be considered. In general, it is often assumed that preloading has a positive effect since after unloading to the operational load, the material is in an unloading/reloading state. This should imply a stiffer material response compared to a virgin loading state. However, this is not true for highly overconsolidated soil as frequently found in North Sea, since the material is already in an unloading/reloading state. In addition, in order to account for any positive effect of preloading, the soil needs to be drained under the increased vertical stresses.

Permeability

Hydraulic conductivity, generally referred to as permeability, is another crucial parameter for the assessment of silt and sand behaviour, i.e. drained, partially drained or undrained response at different operational conditions. Permeability is generally evaluated from oedometer tests. However, when limited or no data is available, the assessment should be based on basic soil parameters (i.e., water content, void ratio, grain size) and databases [7], as illustrated in *Figure 4*. The drawback of such a procedure is that the determination becomes empirical and may inherit a high level of uncertainty. The foundation response can be quite sensitive to the estimated permeability values.

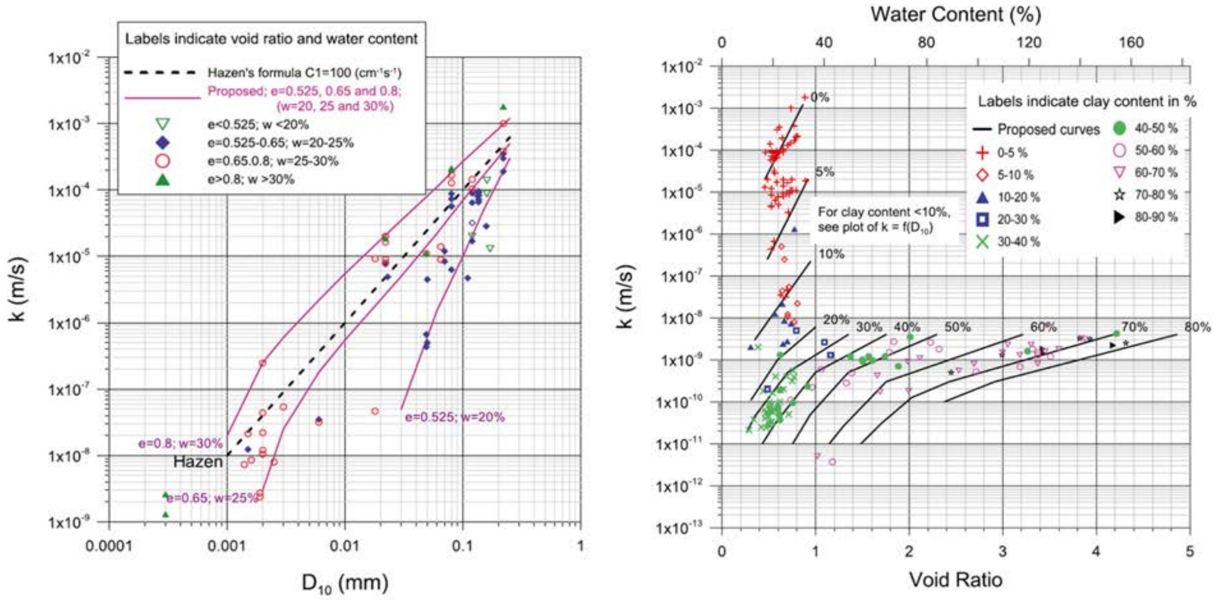

Figure 4 Estimation of permeability [7]

Similarly, stiffness parameters (i.e., constrained modulus), which can be estimated from oedometer tests as well as empirical correlations, are important for the assessment of drainage conditions together with permeability.

Foundation loads

The foundation loads (*Figure 5*) are obtained from dynamic and quasi-static structural analyses with input of foundation stiffnesses. Therefore, the foundation loads and stiffnesses need to be calculated in an iterative manner.

One of the main challenges is to define the average and cyclic load components (and corresponding cyclic/average ratios for the assessment of cyclic soil properties). One can define the average loads as the average of the peak-through values.

The assessment of cyclic stress-strain curves, as for instance shown in [2], requires the cyclic/average load ratio as input. This ratio is not unique for a given storm load history. It is load level as well as load component (V, H, and M) dependent. One may therefore select the ratio from the global overturning moment at seabed which dictates the foundation stiffness.

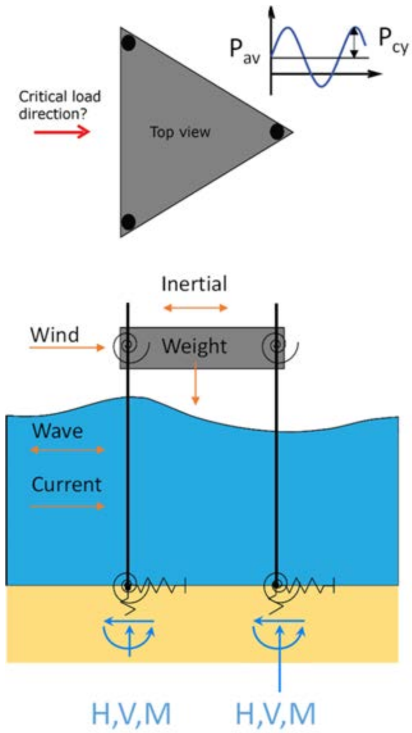

Figure 5 Sketch of a simplified structural model of jack-up with different global loads and foundation reactions.

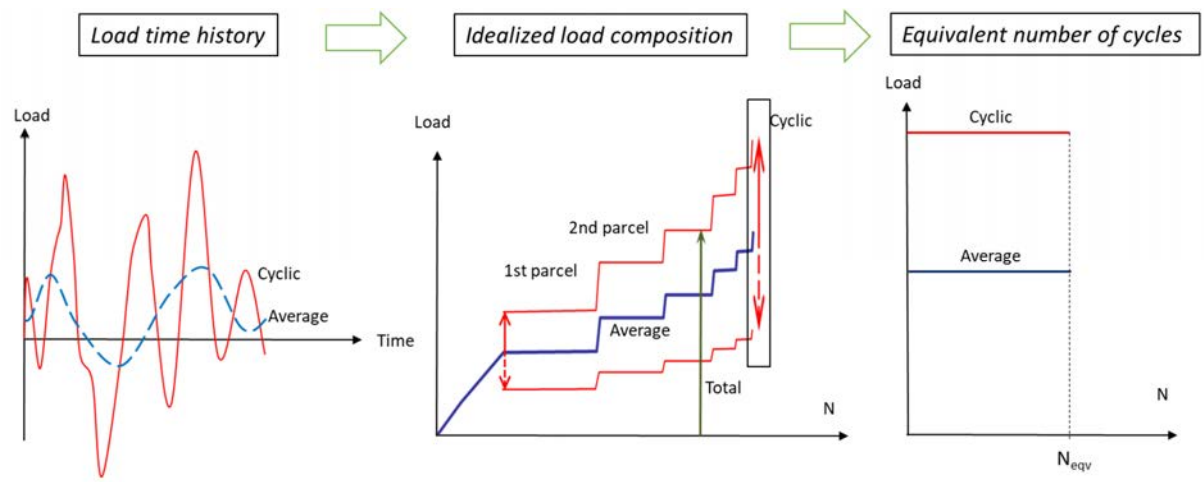

Figure 6 Total foundation load history decomposed into average and cyclic components, idealization of the load history and calculated equivalent load cycles.

Strain or pore-pressure accumulation procedures should be employed to estimate an equivalent number of constant load cycles (N_{eq}) giving the same amount of cyclic degradation as the irregular storm load history. For this purpose, the load histories need to be simplified into load parcels, which in many cases are sorted from the smallest to the largest amplitude with the aim to obtain the largest cyclic degradation for the maximum loads (*Figure 6*). How to determine the idealized load history is discussed in [8].

The basis for obtaining the stress-strain curves are the cyclic contour diagrams, which contain the average and cyclic shear strains for a given number of cycles of constant cyclic shear stress amplitudes and average shear stresses. The effect of preloading can be taken into account when performing the cyclic tests as a basis for the contour diagrams.

Determination of cyclic parameters

As mentioned briefly in the previous section, strain or pore-pressure accumulation procedures should be employed to estimate an equivalent number of load cycles (N_{eq}) of the maximum loads that gives the same cyclic effects (i.e. reduced shear strength or accumulated pore pressure) as the entire storm load history. The method is applied to each layer to estimate layer-specific cyclic degradation and eventually utilize the most appropriate cyclic contour diagram [4]. *Figure 7* illustrates the effective stress paths in undrained test of a contractive soil and definition of parameters as input to cyclic contours diagrams. *Figure 8* presents an example of 3D representation of cyclic and average shear strains as functions of average and cyclic shear stresses and the number of cycles [4].

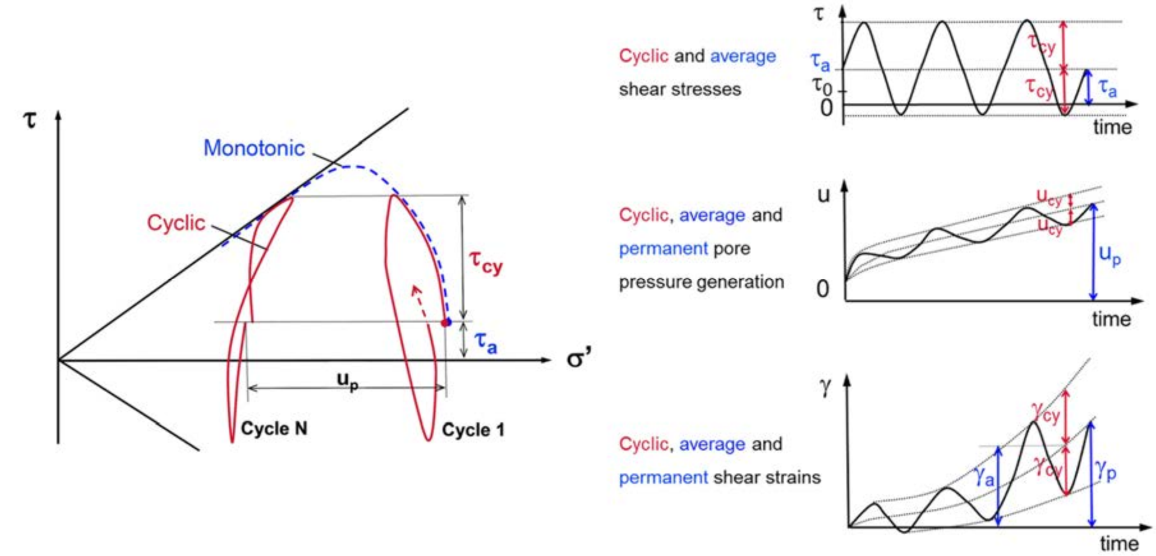

Figure 7 Effective stress paths in undrained test of a contractive soil and definition of parameters as input to cyclic contours diagrams [4].

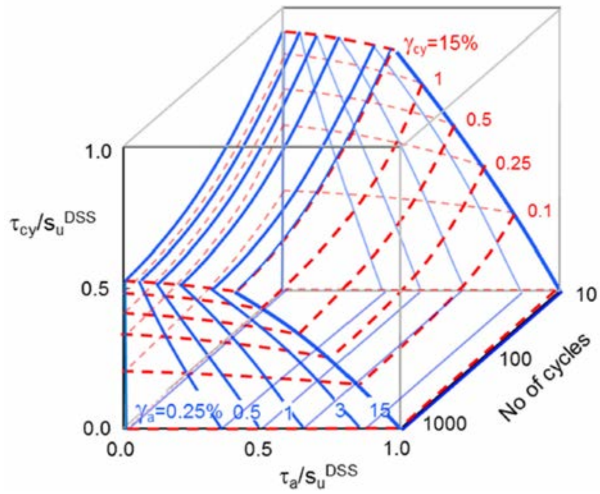

Figure 8 Example 3D representation of cyclic and average shear strains as functions of average and cyclic shear stresses and the number of cycles – Normally consolidated Drammen Clay [4].

The accumulation procedures can further account for partial drainage. However, the sorting of load parcels, i.e. from smallest to the highest amplitude which dictates the accumulation, is generally conservative. In reality, the largest loads do not come immediately after each other, allowing for partial drainage between different large load cycles, especially in sandy and silty soils.

The accumulation procedures should account for both horizontal and vertical drainage when relevant. Depending on the project demand and complexity, the Partially Drained Cyclic Accumulation Model (PDCAM) can be utilized for accounting for partial drainage throughout the whole soil domain during cyclic loading.

Ideally, one should construct contour diagrams for each soil layer for the specific location of the spudcans. However, construction of the contour diagrams requires a significant number of cyclic tests. Therefore, databases for different soil types are often utilized with adjustments for the site/location-specific properties for establishing contour diagrams. If available and reliable, it is recommended to use the site/location-specific cyclic tests to adjust the contour diagrams originating from databases.

The cyclic tests incorporate the rate effects, which increase the cyclic shear strength and stiffness for a low number of cycles compared to standard monotonic tests. Hence, the negative effect of cyclic degradation might be compensated and even overruled by rate effects for low N_{eq}.

As for the monotonic behaviour, an important aspect of cyclic soil behaviour is the cyclic strength anisotropy, i.e. stress path dependent cyclic shear strength as illustrated in *Figure 1*.

Establishing input for capacity and stiffness (FE) analyses
Finite element (FE) analyses of the spudcan foundations require representative soil behaviour, which can be described by different constitutive models. The methodology described in [2] and in this paper utilizes two sets of input: cyclic and total (average+cyclic) shear stress-strain curves for calculation of foundation stiffnesses as input to dynamic and quasi-static structural analyses.

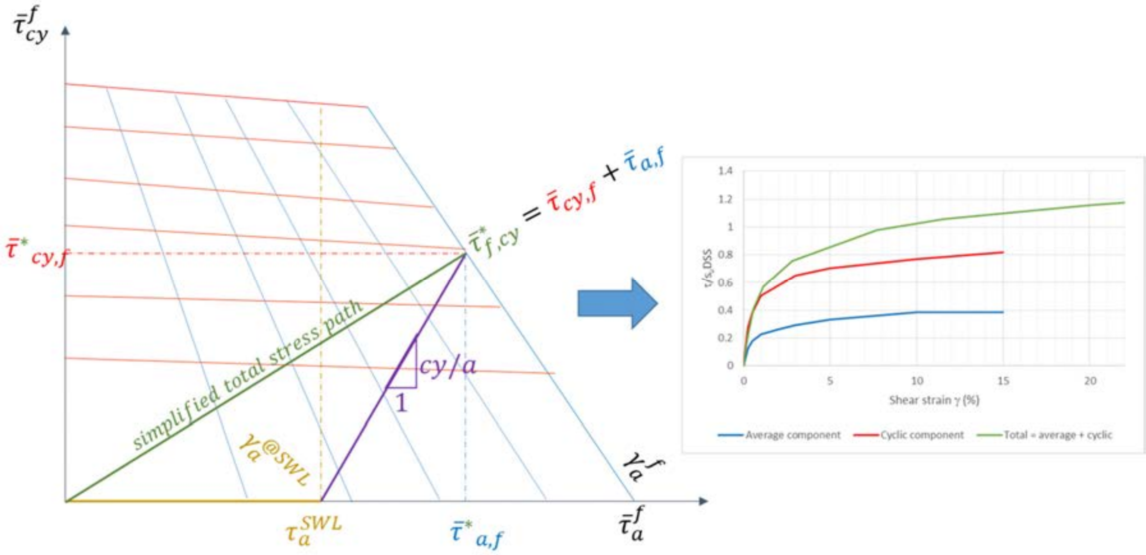

Figure 9 Sketch of the utilization of contour diagram for the extraction of stress-strain curves for jack-ups with a given weight/still water load (SWL)

Figure 9 illustrates the use of cyclic contour diagrams, for the assessment of shear strength versus strain curves as input to the calculation of foundation capacity and stiffnesses. The procedure combines the effect of cyclic soil response with foundation loads as well as cyclic to average load ratios. The still water load determines the initial mobilization. Hence the cyclic stress-strain curves can be sensitive to SWL load level.

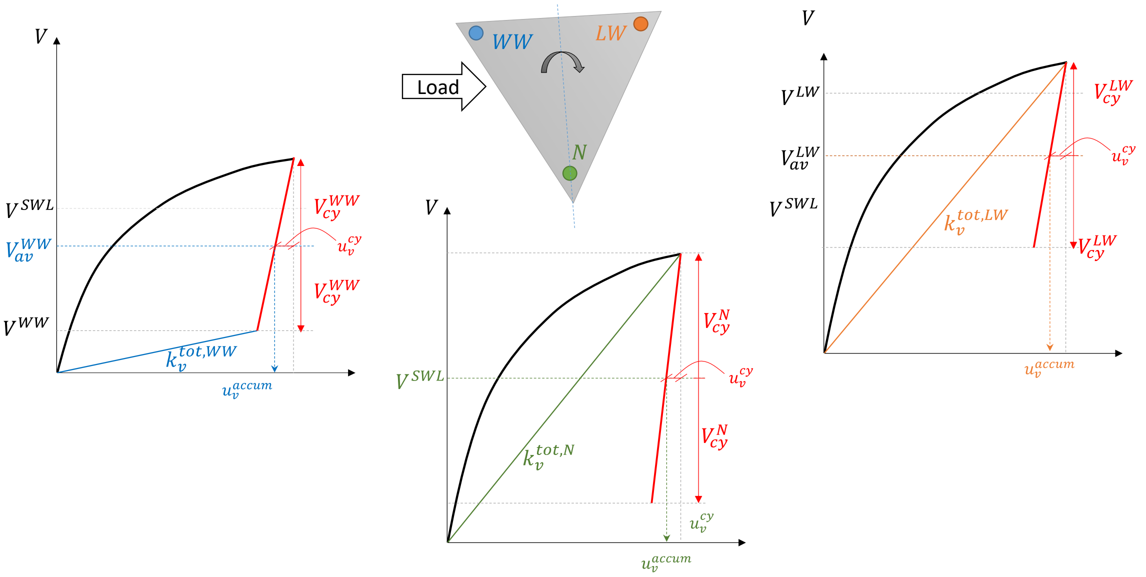

Figure 10 Sketch of vertical load-displacement response, accumulated and cyclic displacements, and determination of secant stiffnesses.

One important challenge is the summation of cyclic and average responses. The example in *Figure 10* is for the windward leg: one should be aware of the accumulated average displacements when calculating total stiffnesses. In other words, the cyclic shear stresses (which in this content always are positive values) from the cyclic loads should not reduce the (average) shear stresses from the average loads. In order to account for this effect, the cyclic shear strains (amplitude) always should be added to the average shear strain when calculating the total shear strain (sum of average and cyclic shear strain) and thus be consistent with the data in the cyclic contour diagrams. Therefore, the absolute value of the cyclic vertical load should be added to the average vertical leg load when calculating the rotational stiffness of the windward leg (see *Figure 10*). Alternatively, the analyses could be divided into calculations of the cyclic and average response, and adding these together.

PENETRATION ANALYSES

Simplified bearing capacity

Industry practice utilizes mostly the simplified bearing capacity methods for the calculation of possible spudcan leg penetration assessments. The methods become as advanced as accounting for different layer interactions, backflow, etc., e.g. [9]. Nevertheless, the methods have quite many limitations due to their empirical nature. For example, these models cannot account for the effects of geometrical changes due to large deformations.

Lagrangian (small deformation) FEAs

One can consider the effect of geometrical changes, by employing simplified FEAs, e.g. [10] and [11]. These methods have their limitations and not very well suited for deep penetrations as they cannot capture complex mechanisms (e.g. backflow).

Large deformation FEAs

Spudcan penetration often involves large deformation in surrounding soil as well as the possible backflow soil. Although having advantages of both computational simplicity and efficiency, the conventional small-strain Lagrangian approach and the Updated Lagrangian (i.e. updated mesh) approach used in the finite element method may suffer from serious numerical difficulties when extreme mesh distortion occurs. Several advanced modelling techniques including Material Point Method (MPM), Smooth Particle Hydrodynamics (SPH), Particle Finite Element Method (PFEM), Remeshing and Interpolating Technique with Small Strain (RITSS), Arbitrary Lagrangian-Eulerian (ALE), etc. have been developed to overcome the numerical issues. Among them, the Coupled Eulerian-Lagrangian (CEL) technique has been widely used at NGI and been proven as a suitable and efficient modelling approach for capturing complex mechanisms and geometric nonlinearity effect during spudcan penetrating through layered soils. For example, [12], [13], [14] successfully applied the CEL and the ALE methods to evaluate the installation effects and compute the bearing resistance of offshore anchors and foundations. Through a series of back-analyses of different centrifuge tests of spudcan penetration, [9] and [15] found out and discussed main advantages of using the CEL method compared to some semi-empirical methods recommended in industry guidelines and the small strain finite element method with the Press-Replace (PR) technique [11]. Several large deformation FE analyses using the CEL were also performed by [15] to back-calculate results of centrifuge tests of unstiffened/stiffened suction anchors as well as to evaluate potential interaction between spudcan and neighbouring subsea template during jack-up installation. Very good agreement between the numerical predictions and the measurements was observed.

Recently, [16] applied the CEL technique in Abaqus to investigate the interaction of spudcan and soil in different sloping angles of the seabed and compared with the existing centrifuge tests. Reasonably good agreement was found by the authors.

A good example of challenging soil conditions with silty layer can be seen in *Figure 11*. Considering dilation indicates significantly lower penetration depth with high potential of punch-through.

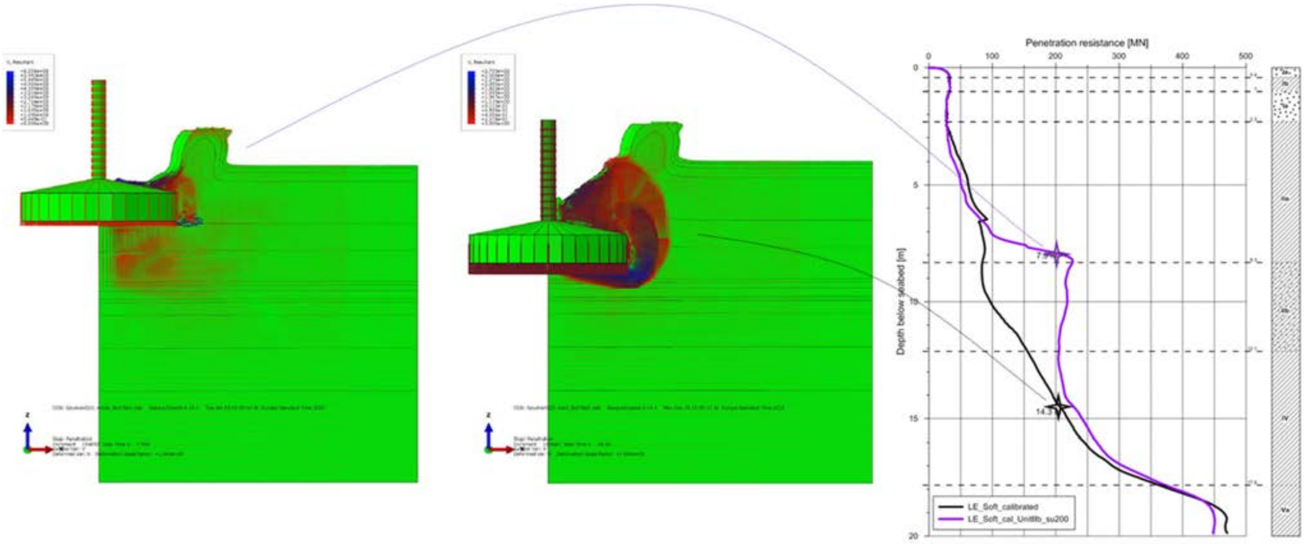

Figure 11 Summary of large deformation Abaqus CEL analyses of spudcan penetration.

Jack-up analyses require foundation capacity checks with load and material factors. The standard approach is employing ISO [1]. At NGI, axisymmetric analyses (with higher-order elements) are used as a reference for calibrating 3D FEAs, which inherit overshoot. A robust and cost-effective 2D model with side shear (HVMCap / BIFURC2D) can be calibrated for vertical capacity, e.g. using FEAs employing higher-order elements, i.e. Plaxis axisymmetric analyses using 15-noded triangular elements. The assumption is that the side shear factors are valid for combined loading (i.e. VHM). Cyclic soil shear strength values are used for obtaining capacity envelopes.

Several analyses at different sites and locations indicate that the failure modes can be complex (see *Figure 12*) and generally quite sensitive to shear strength profile just below the skirt tip level. In this cases the calculated capacity is found to be significatly lower than obtained from cassical bearing capacity equations using an average strength over a depth equal to the diameter of the footing.

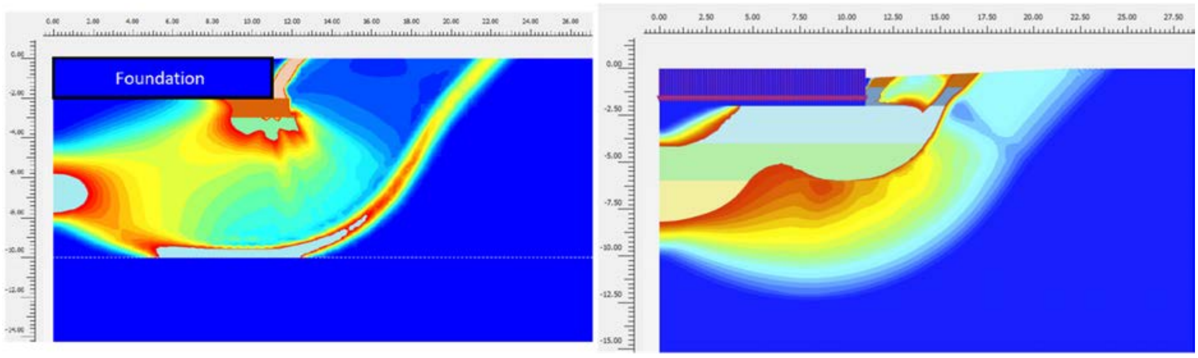

Figure 12 Example failure mechanisms (contours of incremental shear strains) for two different soil profiles

FOUNDATION STIFFNESS

Foundation stiffnesses (fixity) or global soil spring stiffnesses play an important role on dynamic behaviour and structural utilization of jack-up platforms, especially at large water depths. In order to establish accurate values for this fixity, one needs to account for the complex non-linear behaviour of the different soil layers involved under combined average and cyclic loading. The procedure for establishing non-linear stress-strain relationships to be used as input to finite element analyses of jack-up footings together with challenges associated is presented in the previous section. The calculated non-linear load-displacement relationships (foundation stiffnesses) of the individual footings are divided into cyclic and total (average plus cyclic) components to be used as input to the dynamic and quasi-static structural analyses of the jack-up, respectively.

Since the stress-strain curves used in the analyses are based on results from cyclic tests, and therefore only are valid at the actual stress state after an equivalent number of cycles, it cannot be used to follow the actual load history as shown in *Figure 13*. Instead, the loads are applied monotonically from zero to the actual loads and only the results at the end of the analysis are used.

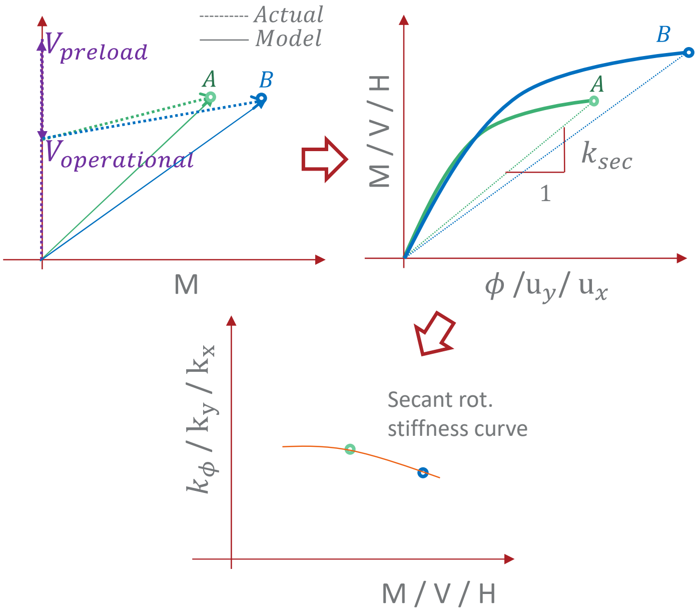

Figure 13 Procedure for calculation of foundation secant stiffness curves

The aforementioned secant stiffness curves are sensitive to different load components, i.e. there is an effect of coupling between for instance the horizontal load and the overturning moment (see *Figure 15*). The calculated stiffnesses can be best represented as matrices with several off-diagonal coupling terms. However, as sketched in *Figure 14*, at a load reference point (LRP) where negligible horizontal displacements evolve with the applied moment, stiffnesses can be represented as vectors. Although it gives a better understanding of the foundation response, it is less practical to use the LRP due to its load level dependency. Hence, in practice, one can ignore the coupling terms as long as the load levels and ratios, and the attachment point are kept same, and then perform several stiffness analyses to cover anticipated range of loads to provide good coverage of stiffness for more efficient iteration with the structural analyses.

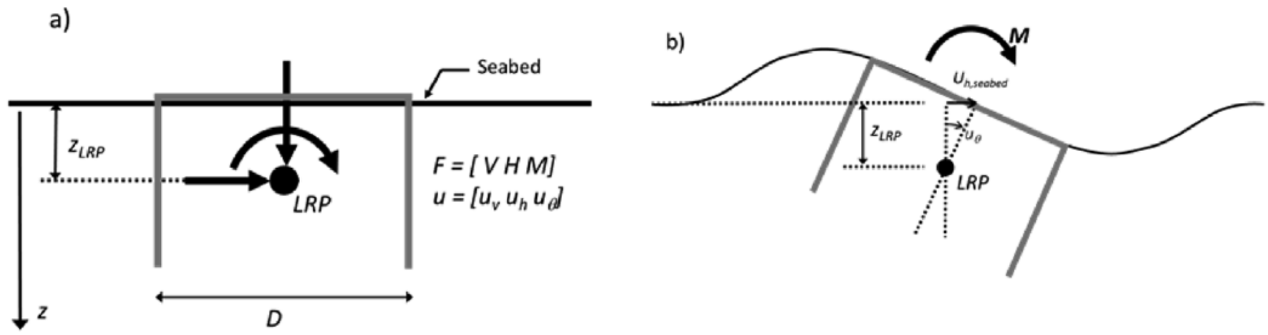

Figure 14 a) Definition of load and displacement vectors, b) Physical interpretation of the load reference point LRP. [17]

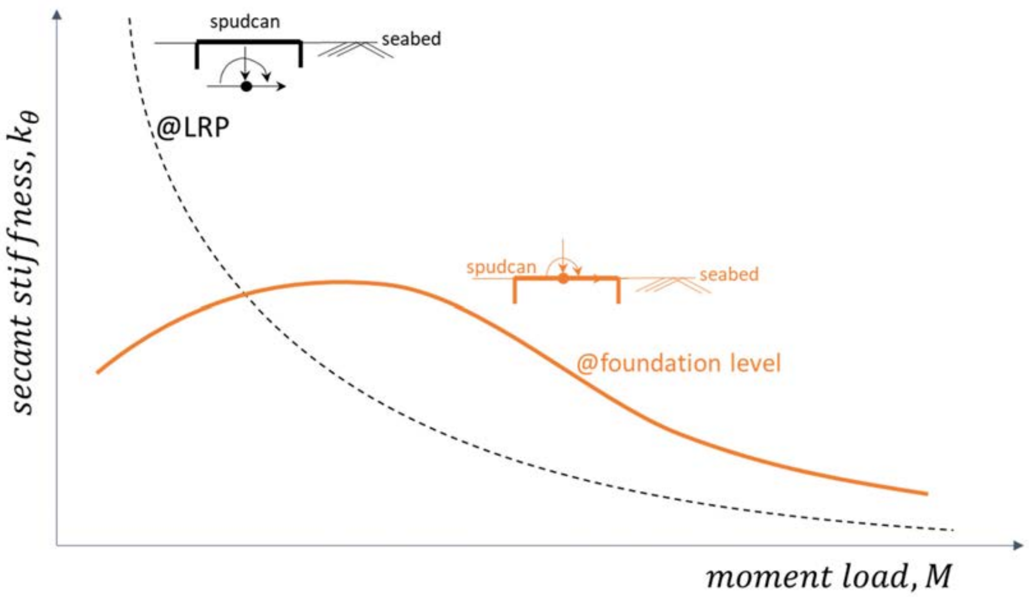

Figure 15 Effect of horizontal load on secant stiffness response

Figure 16 presents the cross-section of an example three-dimensional (3D) finite element (FE) model of the NGI in-house FE program INFIDEL. The INFIDEL program uses near-field and far-field elements to model the soil volume and a cylindrical boundary separate them. Near-field elements are ring elements and their upper and lower faces are horizontal, while their inner and outer faces vertical circular cylinders. The elements are interconnected along the four corner node circles. A column of thin elements was used to model the interface behaviour along the outer skirt. Due to inner skirts that divide the soil into several compartments, a zone of very stiff linear elastic material up to inner skirt tip level (1 m) can be assumed (assuming complete skirt penetration with a full contact with base).

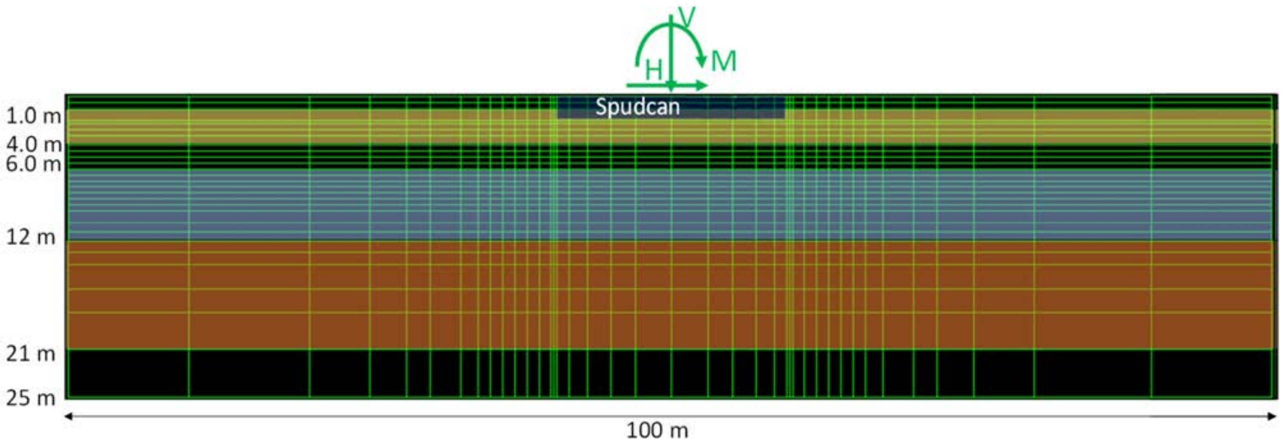

Figure 16 Example finite element mesh of INFIDEL used for stiffness calculations

Figure 17 presents the finite element calculation results (shear stress mobilization ratios) of Plaxis 3D for two different layering and load cases. These type of plots are very useful for the assessment of the contribution of the soil reaction and deformations of different layers.

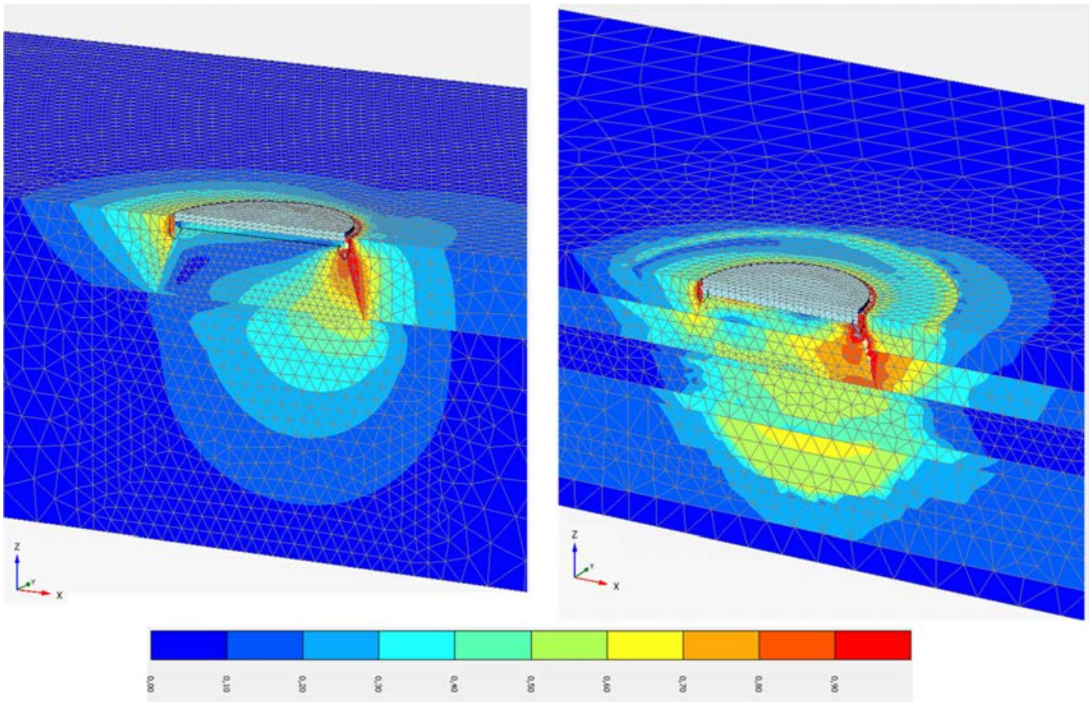

Figure 17 Example finite element calculation results (shear mobilization ratios) of Plaxis 3D for two different layering and load cases.

SOIL AND FOUNDATION DAMPING

Soil material damping reduces the resonant response of jack-up platforms and the corresponding demands in the leg's truss members and the foundation loads. To optimize the design, hysteretic foundation damping, for use in the structural analysis, can be computed with different FE-tools (e.g. INFIDEL, or Comsol/Plaxis). Due to the low vibration frequency of typical wave loads acting on jack-up foundations, soil radiation damping does not contribute to foundation damping, while it may contribute to some extent to foundation damping for earthquake loading.

To compute load-dependent hysteretic foundation damping, hysteretic soil damping versus strain curves are needed for each soil layer (see *Figure 18a*) in addition to stress-strain curves, where the global hysteretic foundation damping is computed as,

$$D_{found} = \frac{E_h}{4\pi E_s}$$

the ratio of the integration of hysteretic, E_h, and elastic energy, E_s, in all soil elements.

Since the soil strain diminishes with distance from the foundation, the largest contribution to the foundation damping comes from the soil in the vicinity of the foundation. As an example, *Figure 18b* shows the distribution of soil material damping around a skirted spudcan foundation during storm loading. The soil slightly larger than a spherical zone following spudcan diameter contributes the majority of the foundation damping, however the size of this zone will vary with the type of foundation and loading (moment, horizontal, vertical), and soil properties.

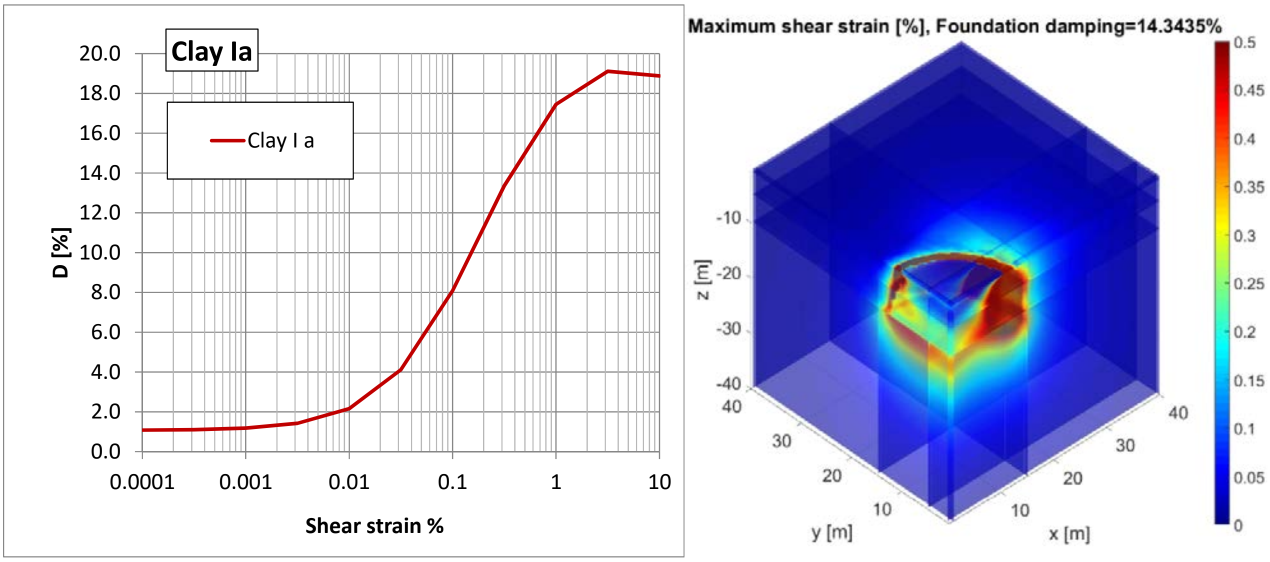

Figure 18 a) Example of soil damping curve for a clayey material. b) Example of calculated soil material damping around a spudcan for a ULS load case.

The soil damping and thus also foundation damping is dependent on the soil strain and will vary with the loads acting on the foundation. Site-specific strain-dependent soil damping can be evaluated based on laboratory test. To establish a complete damping curve over a wide range of soil strain a combination of standard laboratory tests such as Resonant Column test (RC) and DSS/TX is used. There are large amount of laboratory-based soil damping curves in the literature see e.g. [18],[19],[20],[21] and references therein in case of lack of site-specific soil samples or in early phases of projects. In addition to soil shear strain other parameters that have an important influence on the material damping is the plasticity index, the effective confining pressure, and over consolidation ratio.

Some researchers have observed a rate dependency of the damping [22] with an increase in damping value with a factor 2 per log cycle strain rate. The damping's rate dependency should be considered when performing and evaluating laboratory test and when computing foundation damping. Further research is needed to understand what mechanisms in the soil are causing the damping, under which conditions do they occur and are the mechanisms the same in all types of soils.

For offshore structures subjected to wave loading both average and cyclic loads are important for the foundation response as described earlier. To evaluate properly the soil damping in laboratory tests only the soil damping due to cyclic loading is of interest. To remove the apparent damping due to permanent strains observed in laboratory experiments different approaches have been proposed and evaluated by [23].
The foundation damping ratios predicted for spudcan foundations for jack-up for storm loading are often large ($D_{fnd} = 10 - 15\%$). An example of foundation damping versus footing moment load for a typical spudcan foundation is shown in *Figure 19* for the windward, leeward and neutral leg. The foundation damping increases quickly with increasing moment load and increasing vertical load.

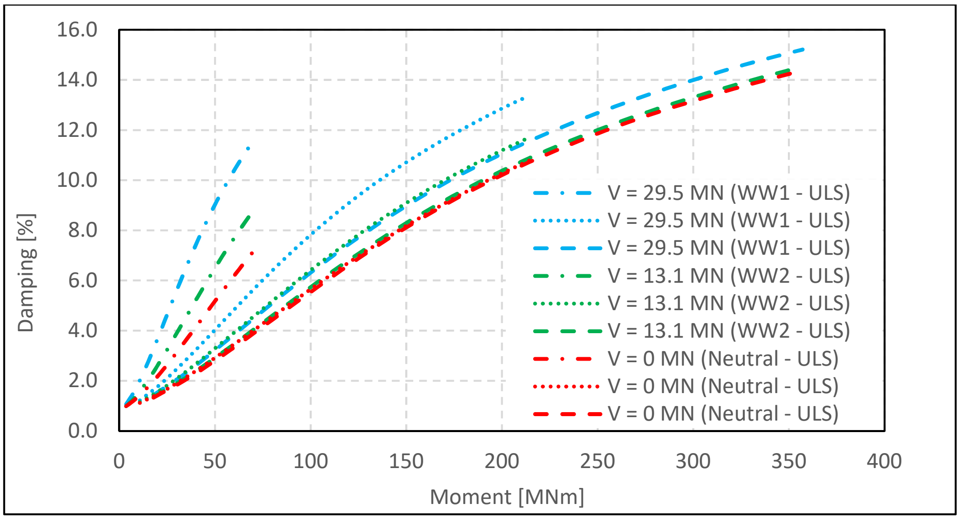

Figure 19 Example of damping versus moment curve for different vertical and horizontal loads for windward and leeward legs.

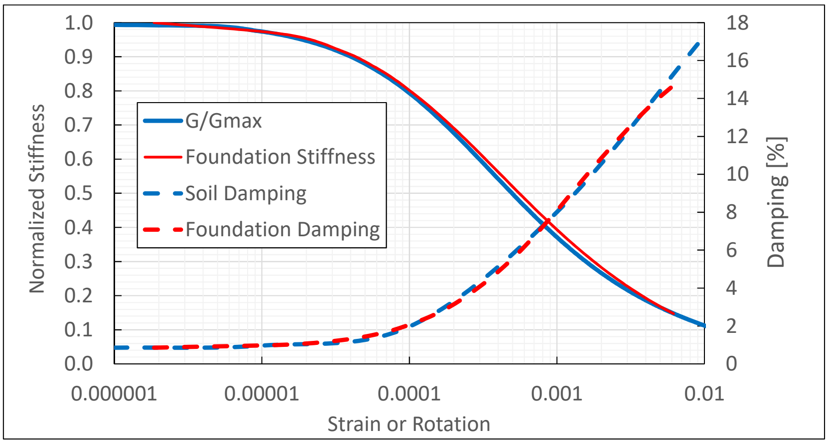

Figure 20 Normalized soil and foundation stiffness, and soil and foundation damping, versus shear strain and rotation for a constant soil profile.

The hysteretic foundation damping shown in *Figure 19* cannot be used directly in the structural time-domain analysis and the damping could be converted to dashpot placed at the mudline [24]. However, such dashpot will be frequency-dependent, and if calibrated for the frequency of the fundamental vibration mode it will likely overestimate the foundation damping for higher vibration modes. An improved alternative to dashpot is a kinematic hardening model with modified foundation moment-rotation response [25]. Further improved models for accounting for foundation damping are so-called macro-element models [26][27][28] and can be calibrated for different loading conditions and load-dependent damping.

Recent analyses with a constant soil profile indicate there may be strong similarities between rotational foundation stiffness and damping versus rotation and soil stiffness and damping versus shear strain curves as shown in *Figure 20*. Further investigation may allow for a simple procedure to establish foundation stiffness and damping which can be useful in early phases of projects when project-specific stress-strain curves are not available.

EARTHQUAKE ANALYSES

Earthquake analyses of jack-ups are often carried out using the mode superposition technique and response spectra of the design earthquake. For such analyses, one traditionally needs the foundation stiffnesses for the spudcans, sometimes referred to as soil springs. These springs are often computed using equivalent linear methods with soil moduli representing the level of soil nonlinearity during earthquake shaking.

A potentially problematic issue related to earthquake loading is soil liquefaction. Liquefaction is generally not a design issue in the North Sea. However, considering the importance of liquefaction in seismic regions and major impact on spudcan foundations, a brief account of it is included here.

Over the past few decades, laboratory tests (e.g. [29]) and CPT/SPT based liquefaction triggering procedures (e.g. [30]) have been conducted to predict the liquefaction potential and the possible consequences of it on structures. Despite extensive research performed on liquefaction, the empirical methods based on CPT and SPT are still the most commonly used because they are based on the calibration of these methods against observed field data. In the empirical methods, the normalized cyclic shear strength, often referred to as cyclic resistance ratio CRR, is determined for the measured value of CPT/SPT and the earthquake magnitude. The CRR is then compared with the earthquake-induced cyclic shear stress normalized by the effective vertical stress, denoted as the cyclic stress ratio CSR.

While this procedure is commonly used for assessment of liquefaction in the free field, it can be extended to the conditions under the structure by including the additional shear stresses from the structure's shaking in the CSR and increasing the CRR for the additional overburden pressure from the structure. In cases where the structural response to earthquake shaking is strong (like low to medium-rise buildings and GBSs), the increase in CSR outweighs the increase in CRR, which results in a more critical condition under the structure. The situation is opposite in structures with long natural periods (such as foundations of large bridges). Jack-ups often have long natural periods; therefore, the liquefaction resistance under the spudcans is higher than the resistance outside the footprint of the spudcans, and as a result, there is a possibility that in sites with liquefaction potential, the spudcans have sufficient bearing capacity. A more reliable approach to assess the performance of complex structures, such as jack-ups, in liquefiable soils is through advanced numerical FE simulations. A number of constitutive models exist for liquefaction (e.g. [31][32][33]) which have been implemented in FE codes and have been applied to various design cases (e.g. [34][35][36]). These models require diligent parameter calibration and extensive sensitivity analyses to capture the range of possible responses.

ADVANCED FINITE ELEMENT ANALYSES

Estimating cyclic accumulation effects with undrained layers/boundaries

Low permeable soils that behave almost undrained within several hours subjected to cyclic and average loadings will generally reduce the shear stiffness and the undrained shear strength of the soil. During the last 40 years, NGI has developed procedures for modelling behaviour of soil subjected to combined average and cyclic loading. Among them, the model called UDCAM (Undrained Cyclic Accumulation Model) is a finite element-based calculation procedure that accounts for the effect of cyclic loading of soils under undrained conditions [37] using the strain accumulation method [38]. Instead of analysing the cyclic load history in the time domain (implicit method), it considers the behaviour during application of the loads in so-called load parcels of constant average and cyclic load amplitudes (explicit method). The model then finds the equivalent number of cycles (N_{eq}), the reduced cyclic stiffnesses and accumulated permanent strains and cyclic shear strength.

Estimating cyclic accumulation effects with partially drained layers/boundaries

Simplified pore pressure accumulation analyses cannot handle the dissipation of pore pressure for varying permeability in different directions. In order to account for the partial drainage in all directions, through other layers, the pore pressure accumulation can be done using FEAs with advanced constitutive models, e.g. the Partially Drained Cyclic Accumulation Model (PDCAM) [39], which calculates the accumulated pore pressure and the corresponding equivalent number of undrained cycles (N_{eq}) under cyclic loading. PDCAM does not predict the behaviour within a cycle but predicts shear strain amplitudes and development of strains during a cyclic load history. PDCAM is implemented in the NGI in-house FE program BIFURC.

Figure 21 presents an example 2D FE model as an axisymmetric model of a 22 m diameter spudcan with 2 m skirt length. Drainage boundaries are specified at the seabed and the vertical outer boundary. No drainage through the spudcan is allowed. PDCAM requires average and cyclic shear strain contour diagrams and pore pressure contour diagrams.

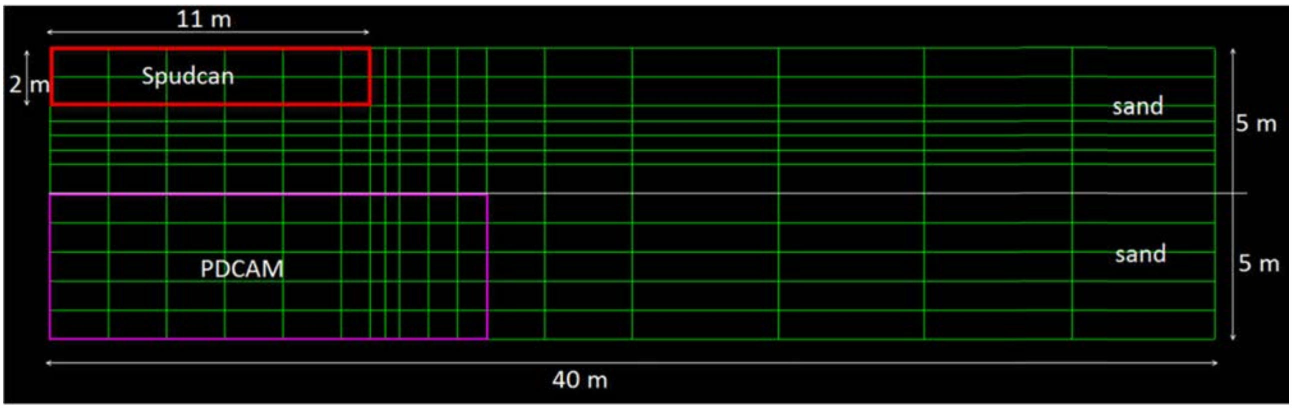

Figure 21 Finite element model for PDCAM analysis to obtain N_{eq}

A selected North Sea storm load history which includes 18-hours build-up and 6 hours peak period is applied as load parcels to obtain N_{eq} at failure. *Figure 22* presents the estimated equivalent number of cycles (N_{eq}) at failure in the soil in a fully coupled pore pressure accumulation and dissipation analysis. PDCAM predicts much lower degradation of soil ($N_{eq} \approx 10$) in contrast to a simplified procedure considering only radial pore pressure dissipation ($N_{eq} \approx 20$).

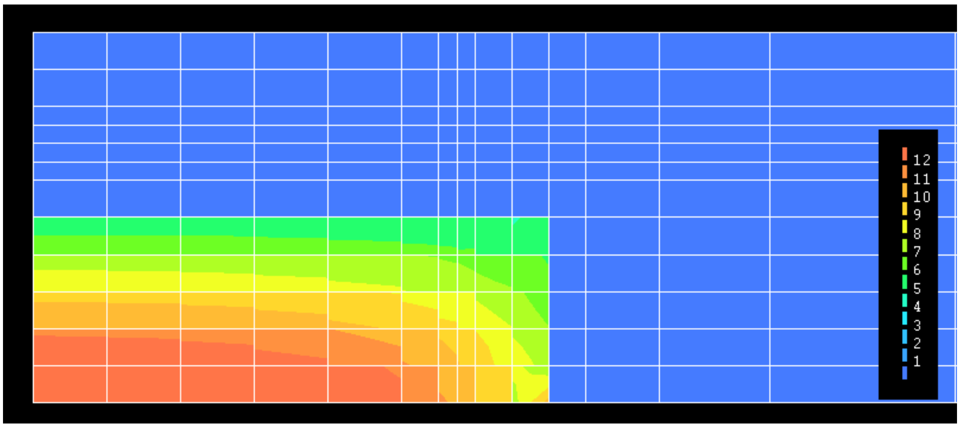

Figure 22 Distribution of the equivalent number of cycles, N_{eq}, at the end of applied storm history scaled to cause failure.

Incorporating large deformation (geometrical change) effects

Large deformation finite element analyses (LDFEAs) e.g. Plaxis Updated Mesh, Abaqus CEL, ALE, can be used to estimate displacements of the spudcan foundation during ULS/ALS loading by accounting for geometrical changes. *Figure 23* illustrates the example of expanding capacity envelope due to large vertical displacements (penetration) of the spudcan, which can be predicted with LDFEAs.

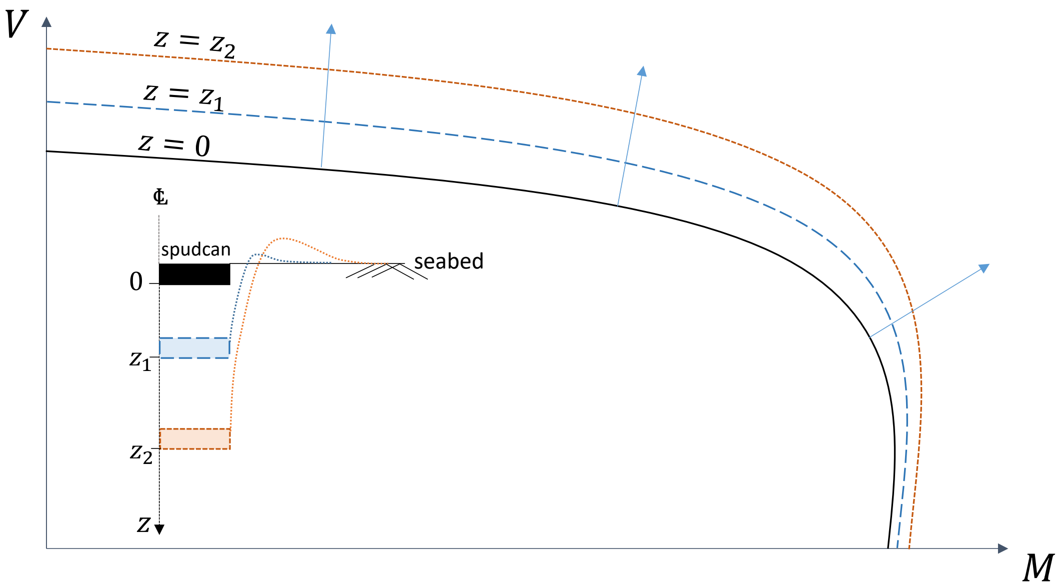

Figure 23 Illustration of expansion of capacity envelope due to large vertical displacements of spudcan

One can also consider the geometrical effects on foundation stiffnesses. *Figure 24* compares the extent of shear mobilization zones: noting that the lower the mobilization the higher the foundation stiffness. Depending on the soil profile, one may obtain higher foundation stiffness using LDFEA (e.g. compression of the weakest soil beneath the skirt tip level).

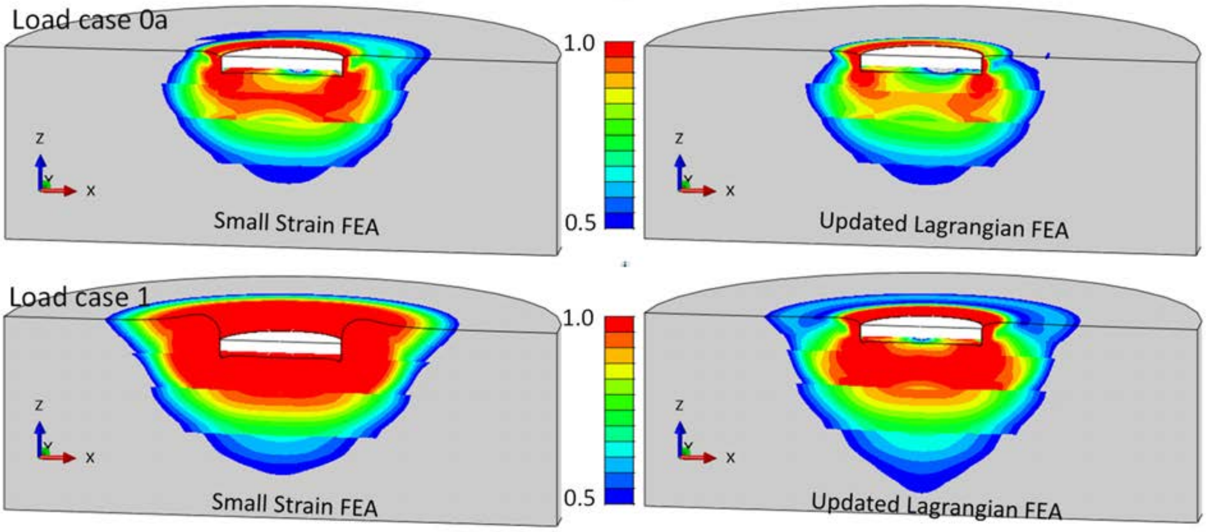

Figure 24 Contour plots of the degree of mobilisation of undrained shear strengths in soils for different load cases. Calculated results of *small strain* (SS) FEA (left) and *Updated Lagrangian* (UL) FEA (right).

RATE EFFECTS

The cyclic laboratory tests already incorporate the rate effects if the tests are performed with the same load frequency as the considered storm history. Storm loads can contain asymmetric peak loads, resulting in a sudden change in average load, which can be faster than the incorporated rate effects in the reference lab tests (*Figure 25*). Hence, one may utilize the additional rate effects if needed, considering (conservatively) the duration of equivalent load cycles and database of [40] given in *Figure 26*. *Figure 27* illustrates the simplified revision of a contour diagram for an estimated increase of average shear strength to account for additional rate effects.

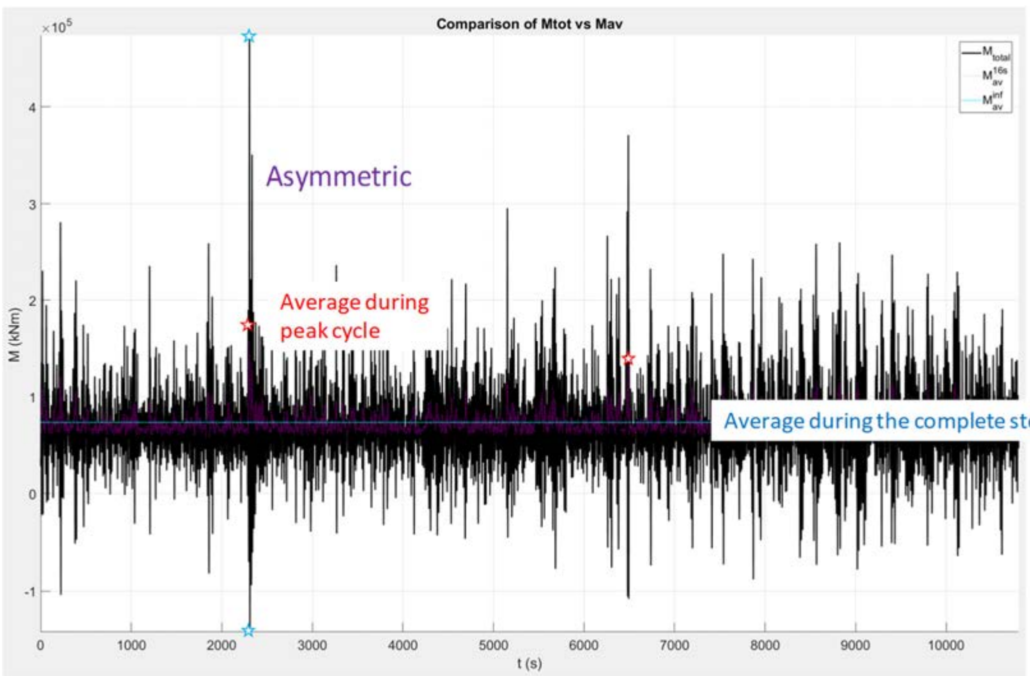

Figure 25 Example storm load history with asymmetric peaks.

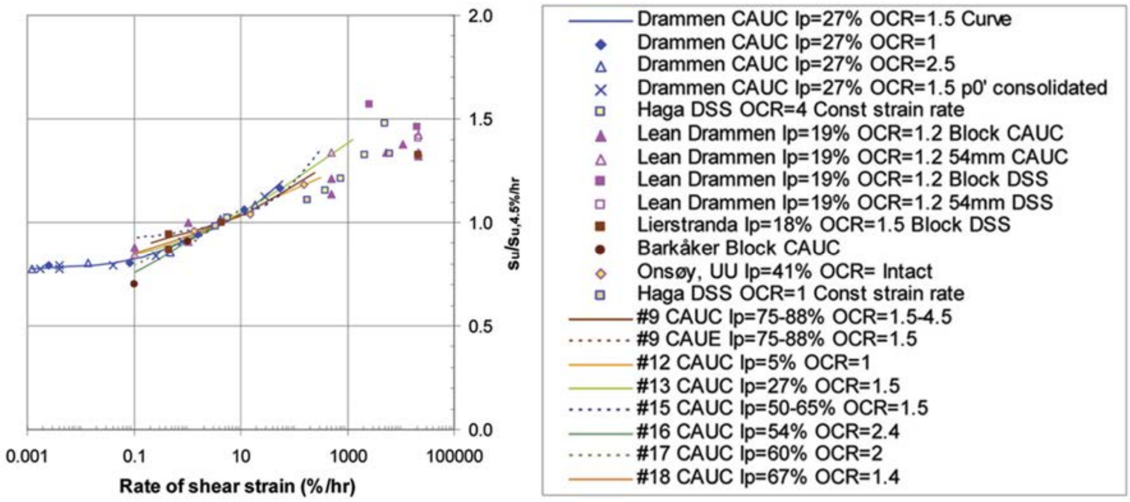

Figure 26 Rate effects and estimation from database [40]

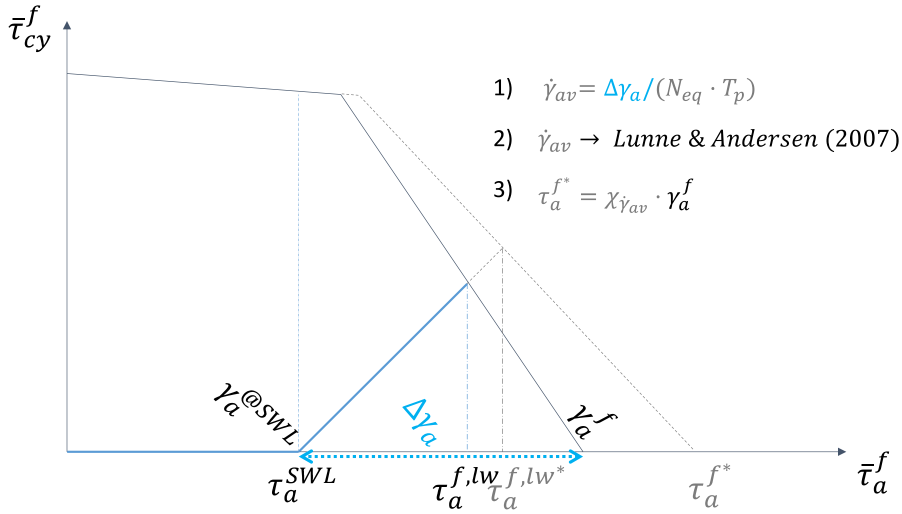

Figure 27 Illustration of modification of contour diagram due to (additional) rate effects caused by the change in average strains

DISCUSSIONS & CONCLUSIONS

Advanced foundation analyses of jack-ups provide valuable information in reducing the uncertainties of the foundation response taking into account stress history, cyclic behaviour, rate effects large deformation effects and complex soil layering. This paper highlights the important aspects and challenges of the advanced site-specific analysis of jack-ups with skirted spudcans in the view of North Sea experiences, with particular focus on the following aspects:

- Structural aspects:
  - The simplified methodology described in ISO [1] may result in underestimation of the structural utilisations.
- Soil parameters:
  - The design strength profiles should be established preferably from in-situ (i.e., CPT) and laboratory tests on adjacent boreholes (BHs). A proper CPT calibration allows extrapolating soil properties between discrete sampling depths. In overconsolidated clays and silts, the selection of the cone factor N_{kt} should account for the OCR and its variation with depth. The selection of a constant N_{kt} may result in unconservative design shear strength, especially at shallow depths that affect the spudcan capacity.

  - Soil stiffness and permeability are crucial to evaluate whether the foundation behaviour is governed by drained, partially drained or undrained conditions. Relevant parameters should be determined from laboratory oedometer tests or from databases when site-specific data is limited or unavailable.
  - Cyclic properties should be preferably determined from site/layer-specific laboratory tests. In the absence of sufficient site-specific data, cyclic contour diagrams can be selected from databases on similar soils.
  - Stress-path dependency of shear strength (i.e., anisotropy) should be investigated from triaxial and DSS tests to perform advanced foundation analyses.
  - Cyclic degradation of each soil layer, expressed in terms of the equivalent number of cycles (N_{eq}) resulting from a given storm load composition, can be determined according to a simplified strain/pore pressure accumulation procedure. This procedure may be conservative unless it accounts for the actual 3D drainage path. Advanced FE accumulation procedure using PDCAM or UDCAM approaches consider the interaction between the soil layers in the domain and is likely to result in lower Neq values compared to a simplified approach.
- Loads:
  - As loads and stiffness are dependent on each other, an iterative procedure is generally required.
  - Cyclic degradation due to storm load history should be considered by evaluation of dynamic storm loading from the jack-up onto the spudcan foundation by means of load parcels.
  - Design storm loads from the jack-up on the foundation needs to be subdivided into what can be considered as average and cyclic components. This is not a straightforward task considering the fact that the foundation rotational stiffnesses are strongly dependent on the loads.
- Foundation stiffness and capacities:
  - It is important to distinguish between cyclic foundation stiffnesses used in a dynamic structural analysis and the total foundation stiffnesses used to check the integrity of the jack-up in a quasi-static analysis.
  - The non-linear foundation stiffness depends on the soil type and loading condition.
  - The moment fixity under the peak loads may be highly affected by cyclic degradation and strain accumulation.
  - Cyclic foundation stiffness and damping curves for use in the structural dynamic analysis are calculated based on results from cyclic laboratory tests.
  - The design soil parameters and related capacity and stiffness should be carefully considered as there is no guarantee that the fixities calculated by standard ISO methods are always conservative for these skirted spudcan foundations, for instance for highly overconsolidated clays.
  - Large deformation effects can be employed to improve capacity and fixity in cases with increasing shear strength profile. This also means that the effect of large deformation may be important to account for in cases with decreasing shear strength with depth since the capacity and stiffness may be reduced.

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