Independent verification of a CJ70 rig at Sleipner B – accounting for existing footprints and cyclic soil behaviour

K. Heurlin and R. Bendzovski
DNV GL

O. Aarsland
STATOIL ASA

ABSTRACT
Independent analyses have been carried out to investigate whether a CJ70 rig can be used for future drilling at the Sleipner B location. This was done as a supplement to and verification of the conclusions from a traditional site-specific assessment that had been carried out according to the methodology described in ISO 19905-1 (Site-specific assessment of mobile offshore units – Part 1: Jack-ups), however without accounting for effects of existing footprints or effects of cyclic loading. The existing footprints originated from a previous installation of a CJ62 type jack-up, equipped with thick box-type circumferential skirts, different from the more common thin walled skirts as used on the CJ70 spudcans. In addition, as the CJ62 has a shorter transverse leg distance than the CJ70, the spudcans will be placed partly offset from the existing footprints. The aim of the analysis was to evaluate potential effects of possible weak zones from existing footprints resulting from previous jack-up installations using the FE computer program PLAXIS 3D, including the effect of cyclic soil behaviour. Site-specific cyclic parameters were simulated in PLAXIS 3D to account for the non-linear stiffness behaviour. The independent analyses showed that the effect of having local weaker zones beneath the foundation due to footprints was for this case found to be negligible in terms of soil-structure interaction. Despite large discrepancies in terms of moment capacity and required rotations for mobilization of the moment resistance, the resulting rotational stiffness from the independent analysis fitted reasonable well with the resulting stiffness documented in the SSA for the governing load condition.

KEY WORDS: *CJ70, Jack-up, ISO 19905-1, PLAXIS, Footprints, Cyclic soil behaviour*

INTRODUCTION

A site-specific assessment (SSA) is normally carried out to demonstrate adequacy of the rig at a certain location and predefined limitations. The methodology described in ISO 19905-1 [1] forms in general a suitable basis for such assessments. There are however certain limitations with the ISO standard related to e.g. existing footprints from previous installations, cyclic behaviour of the soil and complex layering. The ISO code addresses such topics without giving explicit recommendations. Therefore, on behalf of Statoil ASA, DNV GL has further evaluated such effects as a supplement to and verification of an existing site-specific assessment for a CJ70 rig at the Sleipner B location (North Sea outside the coast of Norway).

This paper focuses on the soil-structure interaction, based on the outcome from the FE analyses including the effects of footprints and cyclic soil data. Furthermore, comparisons are made between soil-structure interaction evaluated by means of FE analyses and in accordance to guidance given in ISO 19905-1 [1].

JACK-UP AND SPUDCAN CONFIGURATION

The structure is a Gusto MSC jack-up of type CJ70-X150-MC drilling rig. The rig consists of an almost triangular pontoon shaped hull with three 'X'-braced truss legs. Leg and spudcan dimensions are summarized in Table 1.

Table 1 Leg and spudcan dimensions

Number of legs	3
Longitudinal leg distance	60.6m
Transverse leg distance	70m
Spudcan diameter (effective)	22.2m*
Spudcan skirt length	2.0m
Distance from leg chord leg to full bearing area	3.3m

*D=22m used in FE analyses as an equivalent diameter

The analyses are carried out for the standard CJ70 spudcan design illustrated in Figure 1. The spudcan has in the finite element model been modelled as an equivalent circle with a flat base. Circumferential skirts are modelled extending 2m below the base. The radial skirts have not been modelled, as they are expected to have limited effect on the soil-structure interaction when the base of the spudcan has full contact with the seabed, which is a basic assumption for the analyses performed.

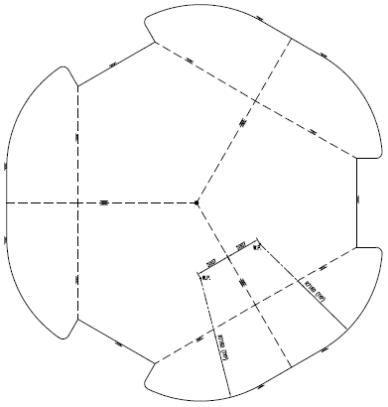

Figure 1 CJ70 spudcan overview from top

FOOTPRINTS

The jack-up West Epsilon (CJ62 rig design) has at two earlier occasions been operating at the Sleipner B location. The skirt arrangement used that time on the West Epsilon spudcans, differs significantly from the current CJ70 design. Instead of the more common thin walled skirts as used on the CJ70 spudcans, West Epsilon was equipped with thick box-type closed end circumferential skirts. For assessment of the effect that footprints have on the soil-structure interaction the spudcan geometry shown in Figure 2 has been considered. The spudcan diameter is approximately 18m and 21m excluding and including the circumferential skirts respectively.

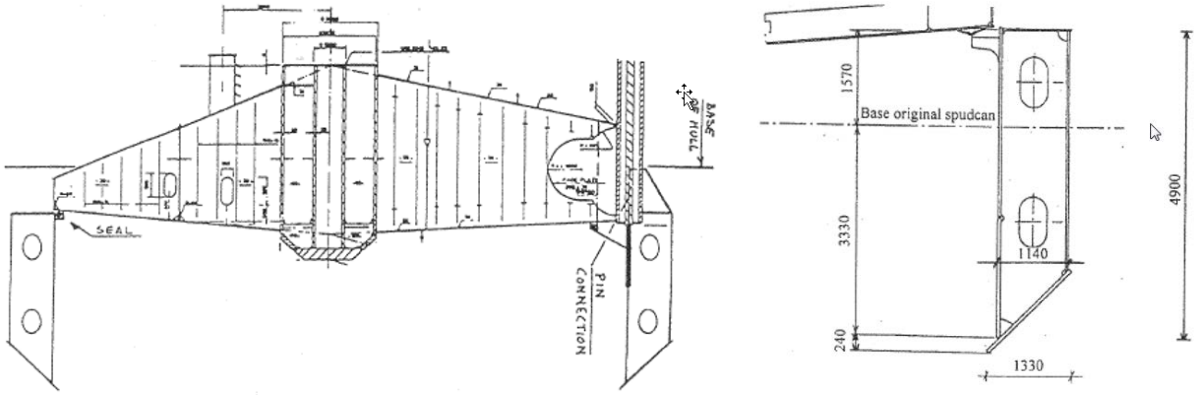

Figure 2 Spudcan and skirt arrangement for the previously used West Epsilon rig at the Sleipner B location

SOIL CONDITIONS

The soil data used as basis for all assessments and analyses in this report are based on the information given in the soil investigation report ‘Sleipner Soil Investigation 1991’ [2].

The soil conditions at the Sleipner B location consists of a top sand layer (Unit 1A) with a thickness of approximately 1.1m. This is followed by an interstratified layer consisting of a stiff clay with seems/layers of sand with varying thickness to a depth of approximately 4.0m (Unit 1B). From 4.0m to 30.0m the soil consists of a stiff sandy silty clay which can be divided into three main sublayers (Units 2A, 2B and 2C), where 2B has a reduced strength, which could potentially be of concern regarding risk of punch through or rapid leg penetration.

According to ref. [2], the clay is over consolidated with an OCR in excess of 5 close to the seabed, reducing to between 2 and 3 at depths beyond 10m. The plasticity of the clay layers generally varies between 16% and 22% and the sensitivity used to evaluate the remoulded strength varies between 1.3 and 1.5. The submerged unit weight for the various soil units vary between 10.1kN/m^3 and 11.2kN/m^3, all calculations have conservatively assumed a submerged weight of 10kN/m^3 in all soil units.

The soil investigation performed at and in the vicinity of the jack-up location is extensive, consisting of 28 CPT tests and 7 sample boreholes. Due to a fairly large lateral variability across the soil investigation site, the undrained shear strength design profile was mainly evaluated based on sample and CPT boreholes within or close to the seabed zones anticipated to be influenced by the pre-loaded spudcans. To limit the effect of data scatter the design profile was assessed mainly based on the more reliable triaxial test results, rather than the simplified index testing such as pocket penetrometer and fall cone testing.

Some CPT locations suggest a very limited thickness of the top sand layer (Unit 1A), therefore this layer has conservatively been disregarded in the analysis and instead treated as an extension of the underlying clay layer (Unit 1B). Considering that the spudcans are skirted, the effect of Unit 1A will be limited. All analyses performed have conservatively been based on a lower estimate evaluation of the undrained shear strength.

Figure 3 present the undrained shear strength data and derived s_u^C design profiles. Figure 4 highlights the CPT and sample boreholes considered when evaluating the design profiles.

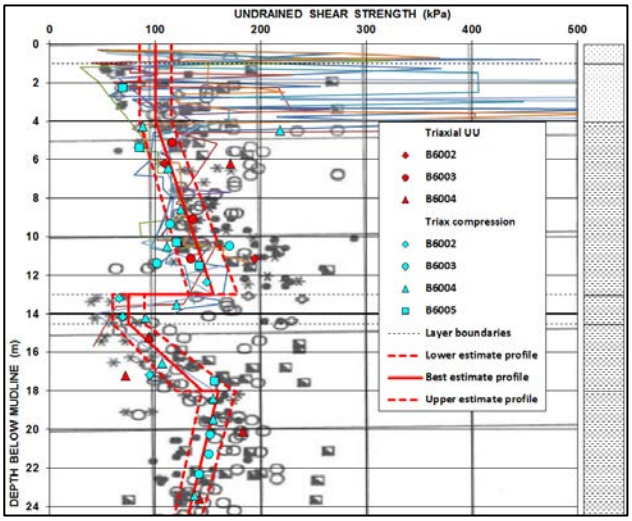

Figure 3 s_u^C from laboratory tests and derived from CPT data together with DNV GL design profiles.

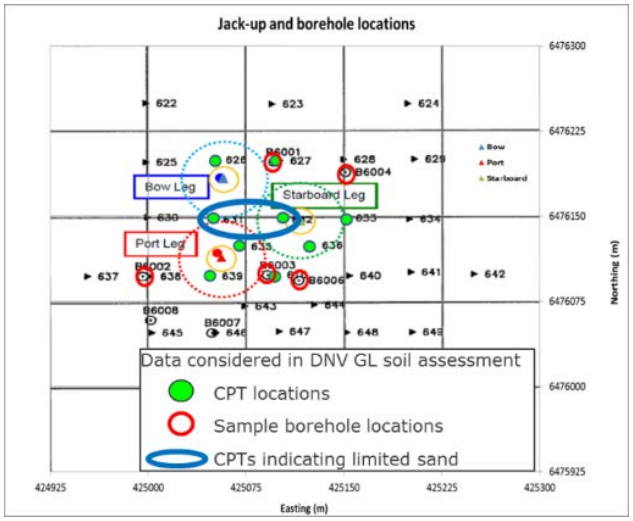

Figure 4 CPT and sample borehole locations considered in the independent soil strength design profile assessment. Yellow circles represent the spudcans and dotted circles a zone of influence reaching one spudcan diameter away from the outer boundary.

The undrained shear strength input applied in the finite element analyses have been based on the direct shear strength (s_u^{DSS}) as this is believed to be most representative for an average undrained shear strength considering the anisotropic behaviour of the soil. According to [2] the $s_u^{DSS} = 0.9*s_u^C$ for all layers, except for layer 1A/1B where no anisotropy has been recommended. In the independent analyses $s_u^{DSS} = 0.9*s_u^C$ has also been applied for the soil in layer 1A/1B. The capacity calculations performed using the limit equilibrium software Bearing1 [3] are based on input considering all the anisotropic undrained shear strengths (s_u^C, s_u^{DSS} and s_u^E). For this purpose, the shear strength in extension is according to [2] $s_u^E = 0.8*s_u^C$ for layers 2A and 2B and $0.65*s_u^C$ in layer 2C. For layer 1A/1B $s_u^E = 0.8*s_u^C$ has been assumed.

The static undrained shear strength parameters used for bearing capacity calculations for the preload condition and to evaluate the foundation stability in operation are summarized in Table 2.

Table 2 Static shear strengths – low estimate

Depth interval [m]	Layer No. [-]	s_u^C [kPa]	s_u^{DSS} [kPa]	s_u^E [kPa]
0 – 4	1A/1B	100	90	80
4 – 13	2A	90 - 140	81 -125	72 – 115
13 – 14.5	2B	63 - 69	58 - 62	50 – 54
14.5 – 18	2B	69 - 104	62 - 90	54 – 80
18 – 27	2C	160 - 120	144 - 108	100 – 78
27 - 30	2C	120	108	78

For calculation of the rotational foundation stiffness, cyclic shear strengths have been used. These have been established by determining a cyclic factor based on cyclic contour diagrams. Static and cyclic contour diagrams for both CAUC/CAUE and DSS tests have been developed by NGI for a different number of equivalent numbers of maximum shear stress cycles (N_{eq}). For the purpose of evaluating the cyclic shear strength the cyclic diagrams related to DSS tests have been used. Since no laboratory tests have been done in layer 1A and 1B, the stress-strain relationship for layer 2A has been applied for layer 1A and 1B in order to account for a non-linear behaviour in those layers. Examples of the static/cyclic contour diagrams used as basis for the analyses are presented in Figure 5a and Figure 6a . Based on experience from previous projects , cyclic diagrams valid for N_{eq}=10 has been applied which is expected to be in the conservative side for this type of structure and location.

Based on maximum values of environmental loading taken from the site specific assessment [4], considering wind as static loading, and wave/current and inertia as dynamic loading, it is found that the ratio between the average and the cyclic environmental load is approximately 1. It is then assumed that the same ratio applies to the foundation restraint moment and to the soil shear stresses, i.e. M_{cy}/M_{av}=1. The average stress factor of the stress path for zero cyclic stress (which represents the stress utilization for the permanent vertical load) is calculated to be around 0.5 since the static weight of the platform is close to half the bearing capacity. Examples of the stress path is shown with a red line in Figure 5a and Figure 6a. The cyclic shear strength, $s_u^{DSS_cy}$ at various strain levels is calculated as the sum of static and cyclic shear stress at failure. The resulting normalized stress-strain relationship based on the given contour diagrams and stress path has been presented in Figure 5b and Figure 6b. A summary of the characteristic soil parameters used in the soil-structure interaction analyses are presented in Table 3. The soil strength within the footprint zones have been assumed to be fully remoulded, assuming a sensitivity of 1.3 in accordance to data given in [2] for Unit 1B. The soil stiffness for the remoulded soil has been reduced by a factor corresponding to the sensitivity. Assuming a fully remoulded soil within the footprints is believed to be conservative, as it is highly likely that the soil would regains some of its original strength since the previous jack-up West Epsilon was at the site in 2011.

The PLAXIS 3D finite element program [5] has an incorporated tool which allows to simulate lab tests, such as triaxial or DSS laboratory tests. This tool has been used to calibrate necessary strength and stiffness parameters as input for the PLAXIS 3D analyses. Since the stiffness is a function of the shear strength, the calibration was carried out using average shear strength within each of the defined sub-layers. An example of the curve fitting performed for layer 2A is presented in Figure 7. It should be noted that for the loading conditions assessed, the cyclic shear strength is approximately 15% -20% higher than the static strength, i.e. being beneficial for design.

Table 3 Summary soil parameters for PLAXIS 3D analyses, further description presented in ref. [5]

Depth interval [m]	Layer No. [-]	s_u^{DSS} [kPa]	triaxial loading stiffness at 50% shear mobilization, E_{50}^{ref} [kPa]	oedometer loading stiffness, E_{oed}^{ref}[kPa]	triaxial unloading stiffness, E_{ur}^{ref} [kPa]	R_f [-]
0 – 4	1A/1B	101.3*	6790	6790	20370	0.90
4 – 7	2A	91.1 – 107.6	10000	10000	30000	0.91
7 – 10	2A	107.6 – 124.2	11660	11660	34980	0.91
10 – 13	2A	124.2 – 140.7	13320	13320	39960	0.91
13 – 14.5	2B	67.3 – 71.9	4000	4000	12000	0.85
14.5 – 18	2B	71.9 – 104.4	5000	5000	15000	0.84
18 - 27	2C	167 – 125.3	14000	14000	42000	0.90
27 – 40	2C	125.3	12500	12500	37500	0.90

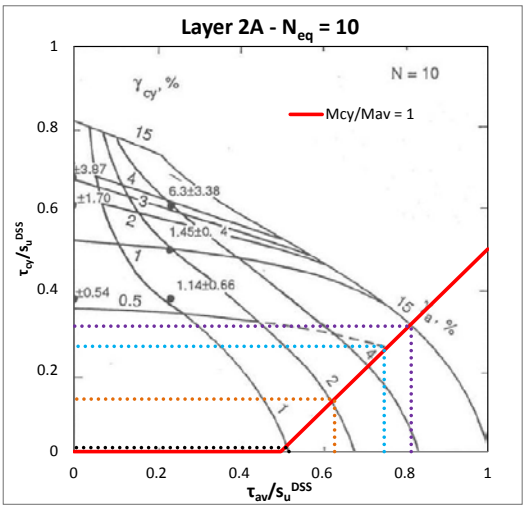

Figure 5a Cyclic contour diagrams

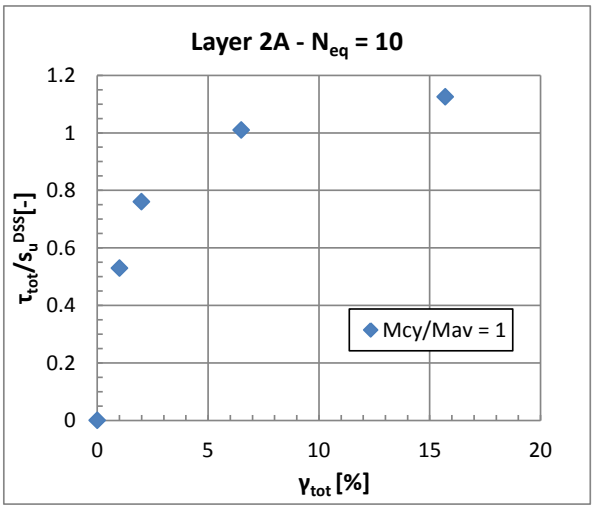

Figure 5b Normalized stress-strain relationship

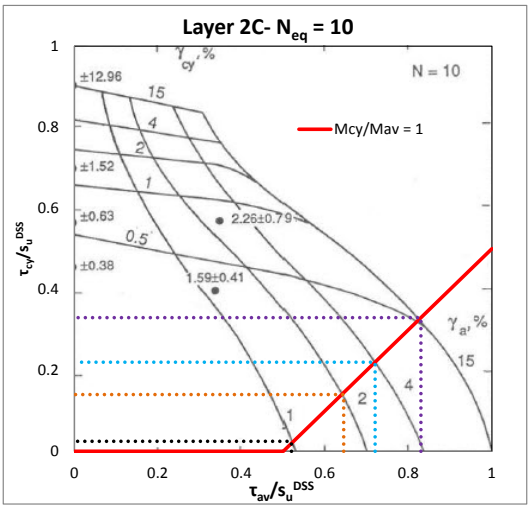

Figure 6a Cyclic contour diagrams

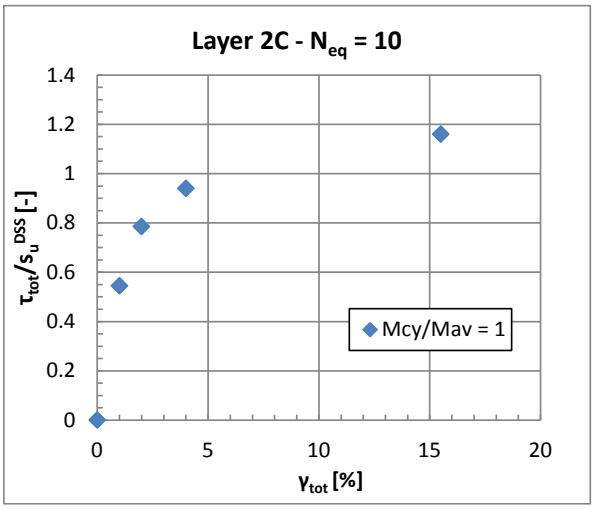

Figure 6b Normalized stress-strain relationship

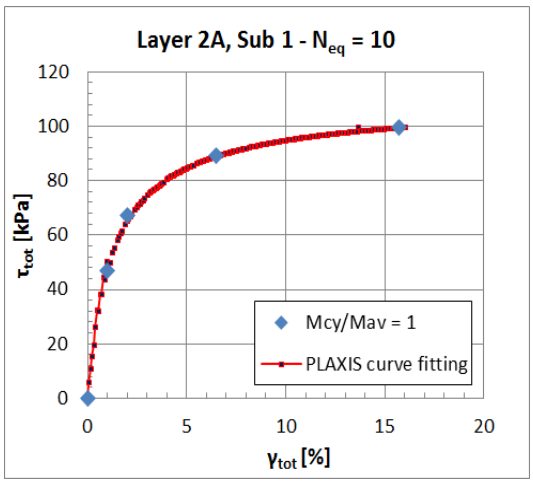

Figure 7 Example - calibrated stress-strain relationship with combined static and cyclic DSS data

ASSESSMENT OF FOOTPRINTS FROM PREVIOUS INSTALLATIONS

Two previous jack-up installations have taken place at the location. Based on the information from the seabed survey, a qualitative assessment has been carried out to define a representative case with local disturbed footprint zones. The potential effect of the local weak zone is then further analysed in PLAXIS 3D for comparison to the analysis assuming no footprints, allowing to make an overall evaluation of the effect of the local weak zones.

Considering that the footprints are formed from skirted spudcans with shallow embedment of the spudcan base, the case of a spudcan potentially sliding into a nearby footprint is not relevant and has not been assessed in this study.

The previous operations were made with a CJ-62 rig design, which has a shorter transverse leg distance than the CJ-70 rig design analysed in this report. Bearing this in mind, the CJ-70 spudcans will be placed partly over the previous footprints. Based on the seabed survey data together with the planned CJ-70 location it has been possible to evaluate the spudcan positions relative to the existing footprints. An overview of the CJ-70 planned spudcan positions relative to the existing footprints is shown in Figure 8. The CJ-62 rig has been operating at two different time periods: 1995-2000 and 2009-2011. As can be seen from the bathymetry images, the positions during the two periods of operation are slightly offset.

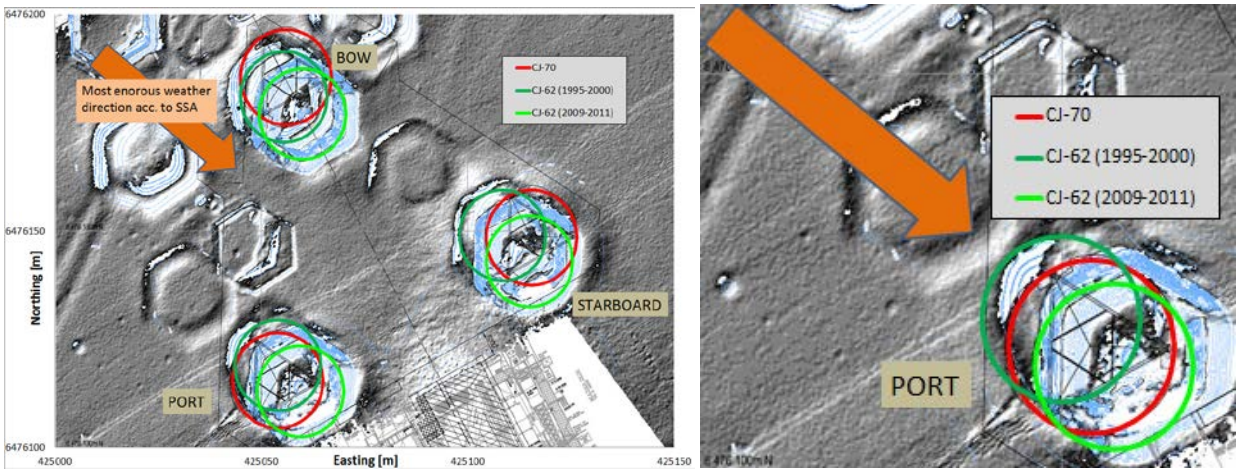

Figure 8 Overview footprints from seabed survey together with planned CJ-70 location

From the Site Specific Assessment report [4] it has been identified that environmental loading from 330º degrees relative to platform north is the governing load direction. This wave load direction is relatively parallel with the offset direction of the footprints. It is not straight forward to predict for which of the three spudcans will have the largest effect and is the most relevant to model in PLAXIS. PORT and STARBOARD legs differs from the BOW leg by having existing footprints both on the rear and front side and for the governing weather direction(s) PORT and STARBOARD legs will also be exposed to highest vertical and shear forces. In addition, based on the initial analyses assuming no footprints, it is seen that the highest mobilisation of the soil strength is around and below the skirts. It is therefore believed that the highest effects of footprints will be where most of the skirt is directly in contact with a local weak zone. Therefore, as a simplified but representative case it has been assumed that the centre of the two footprints and of the CJ-70 spudcan are aligned along a line parallel in the wave loading direction. This is believed to be representative for both PORT and STARBOARD legs.

The width and depth of the locally disturbed zones have been taken as 3.3m and 6m respectively. This is based on a conservative assumption of having fully remoulded soil when extracting the 1.33m thick and 4.9m deep add-on skirts (Figure 2) creating a reversed end bearing failure zone. This fits rather well with the results from an analysis carried out by GEOVEST in 1999 [7] for extraction of the CJ-62 spudcans, see Figure 9.

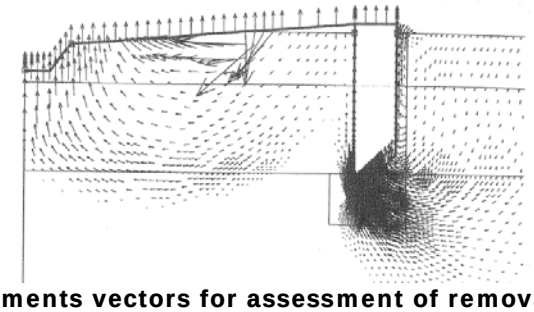

Figure 9 Soil displacements vectors for assessment of removal of CJ-62 spudcans [7]

DESCRIPTION OF THE PLAXIS MODELS

The finite element program PLAXIS has been used for the geotechnical analyses [5 & 6]. The following analyses have been performed:

- PLAXIS 2D: axisymmetric analyses for calculation of the pure vertical bearing capacity and as result from that assess whether punch-through type of failure could occur.
- PLAXIS 3D: analyses to establish non-linear moment – rotation relationship for characteristic soil properties, with and without footprints.

The finite element PLAXIS 3D models, with and without footprints, are presented in Figure 10. Since the horizontal load and moment on the spudcan foundation is acting in the same direction, it is possible to model only half of the model and still account for the 3D effects. This allows to reduce the calculation time, and still maintain a sufficiently high mesh resolution. The diameter of the spudcan is 22m and the depth of the circumferential skirt is 2m. The consolidation from previous jack-up installations has been disregarded in the analyses since the in-situ pre-consolidation stress is slightly higher than the average vertical stress from still water load beneath the spudcan for the operational conditions of the West Epsilon rig. The footprints are modelled as cylindrical volumes with a reduced soil strength compared to the in-situ soil. The reduced soil strength is conservatively assumed to be taken as the fully remoulded strength compared to in-situ soil, i.e. no regain of soil strength has been accounted for.

In PLAXIS 3D the forces (vertical and shear) and moment have been applied as distributed pressure on the base plate. The vertical and horizontal forces are applied initially prior to applying moment. The moment is then increased, keeping the vertical and horizontal forces constant. By this, one usually assures that when iterating between structural and foundation performance one has the intended horizontal and vertical force at the point of convergence.

For calculation of the moment-rotation stiffness, the hardening soil model has been used to account for the non-linear behaviour of the hyperbolic stress-strain curve. All soil layers in the analyses have been modelled as undrained by using the ‘Hardening soil model - Undrained B’-option in PLAXIS.

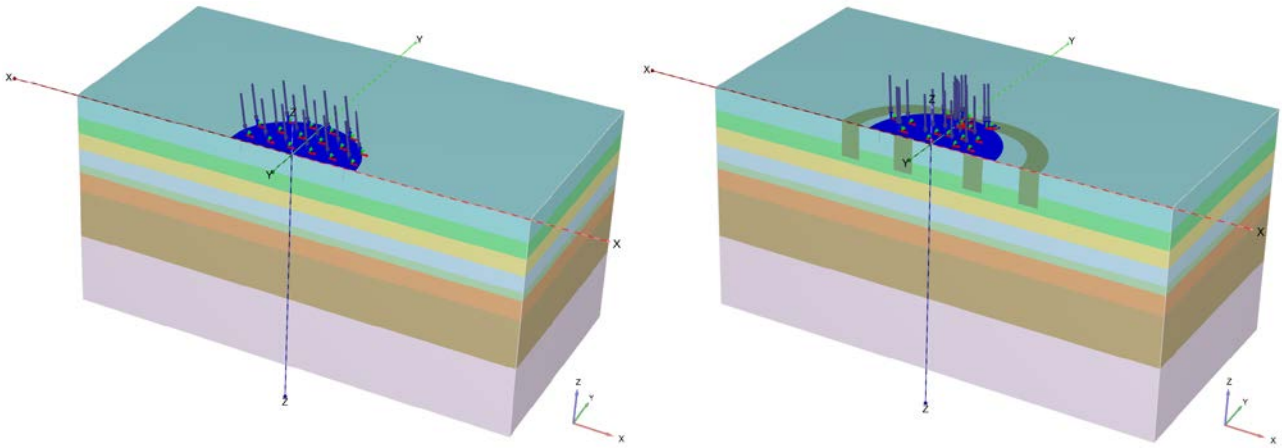

Figure 10 PLAXIS 3D model for calculation of non-linear moment – rotation behaviour – with and without footprints

PLAXIS 2D has been used as basis for calculation of the vertical bearing capacity. Since it is an axisymmetrical model, only half of the model is shown, see Figure 11. The radius of the spudcan is 11m and the depth of the circumferential skirt is 2m. No inner skirts are modelled in this analysis as it has no effects on the vertical bearing capacity.

The traditional elastic-plastic Mohr-Coulomb soil model has been used for the PLAXIS 2D model, as the displacement development is not essential for these analyses.

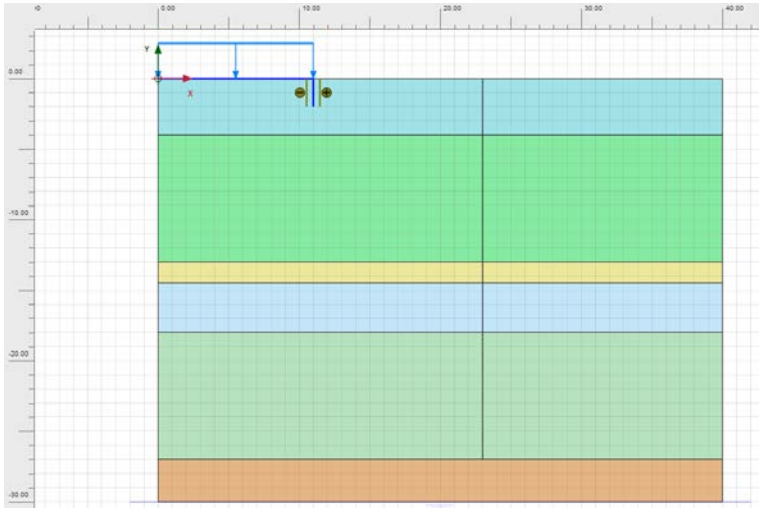

Figure 11 PLAXIS 2D axisymmetric model for check of pure vertical bearing capacity

A shear strength anisotropy is not accounted for in the PLAXIS 2D and 3D calculations, however the DSS strength is considered as representative as an average shear strength accounting for the soil anisotropy. A constant isotropic shear strength, selected as the direct shear strength s_u^{DSS}, is therefore defined for each layer. The roughness factor applied to the interface between a steel member and soil is taken as 0.7 for all layers.

DESCRIPTION OF THE BEARING1 MODEL FOR STABILITY CALCULATIONS

The software BEARING1 [3] has been used to document the foundation stability for undrained loading. It provides limit equilibrium type of solutions derived for different types of failure modes (both force and moment equilibrium). The program solutions are in principal based on plain strain failure modes, however accounting for 3D effects by 'side shear' on two vertical planes that enclose the failure volume defined by the 'plane strain surface' and the two side planes. The program accounts for the anisotropic shear strengths which mean that different undrained shear strengths can be defined for the active, direct and passive shear zones.

The BEARING1 results are considered being on the conservative side, as the program may underestimate the embedment effect, although less when there is a horizontal force involved as for the ULS analysis.

RESULTS FROM PLAXIS ANALYSES AND BEARING1 CALCULATIONS

PLAXIS 2D and BEARING1- Vertical capacity/penetration curve

The finite element program PLAXIS 2D has been used to estimate the penetration depth during preloading. During penetration, the soil will in some zones experience high strains – locally extremely high strains. Beyond certain strain levels the soil is likely to soften. These effects cannot explicitly be accounted for in PLAXIS and are therefore ignored. This can however be justified both due to the soil sensitivity being low, i.e. limited softening will occur, and that as the spudcan becomes further embedded it will encounter new "virgin" soil which has not been subjected to large strain levels.

A reference capacity has been calculated using a PLAXIS 2D small strain static assessment, assuming full base contact and disregarding any effects of heave or change of the soil stratigraphy boundaries. This has been referred to as the 'PLAXIS whished in-place'-analysis. However, during spudcan penetration, the soil around the foundation will heave and the soil stratigraphy boundaries will be effected. This is explicitly accounted for in PLAXIS when using the updated mesh option. To illustrate a typical penetration curve beyond the 'whished in-place' capacity, the analysis was continued past this capacity using the updated mesh option.

The penetration curve is presented in Figure 12. It should be noted that the reference penetration depth has been taken at the chord tip depth of 3m. For comparison, the vertical capacity from the BEARING1 – Mode 4 analysis representing 3m chord tip penetration has been shown. Furthermore, a spudcan load-penetration plot based on traditional bearing capacity formulations assuming a "wished in place" spudcan at the various depths has been included.

Based on the PLAXIS 2D penetration curve one can conclude that the static bearing capacity is higher than the preload reaction of 22700 tonnes. Considering the PLAXIS results, there are no signs for any punch - through failure mechanisms for the load levels investigated.

The results based on traditional wished in-place closed form bearing capacity solutions [1], should be treated as an example, as results may vary depending on which methods are used and what assumptions are made. The accuracy of such methods often become reduced for complex soil layering, as these are mainly suited for simple soil stratigraphy. For the case presented in Figure 12, the results for the wished in place bearing capacity solutions, show that it is likely that a punch-through or rapid penetration scenario may occur. However, based on the finite element results, such scenario could be discarded.

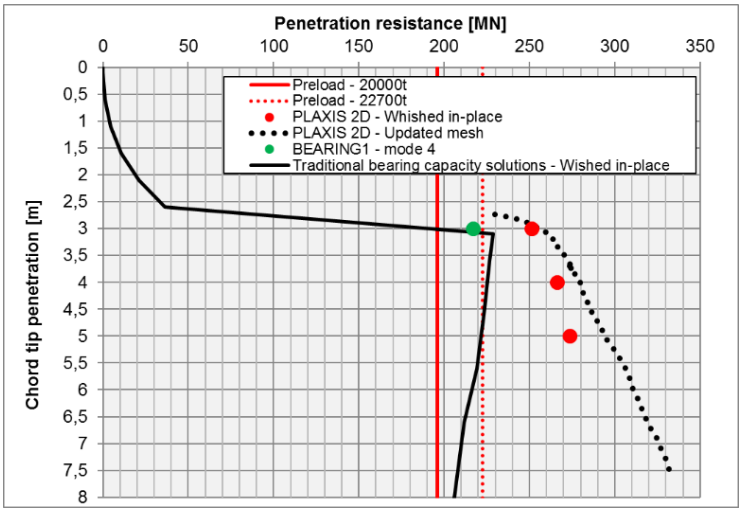

Figure 12 Penetration curve and bearing capacity based on PLAXIS 2D, Bearing1 and traditional wished in place bearing capacity solutions

PLAXIS 3D – moment – rotation behaviour

The loads referred to are based on the most critical wave direction (330 degrees heading relative to the platform north), [4]. The vertical and horizontal forces are applied initially without any moment. The moment is then incrementally increased, keeping the vertical and horizontal reactions constant. The following forces and moments have been used as input for the PLAXIS 3D analyses:

Vertical force: Bow (Leg 1) = 6422 tonnes
Port (Leg 2) = 13968 tonnes
Starboard (Leg 3) = 17979 tonnes
Shear force: Bow (Leg 1) = 910 tonnes
Port (Leg 2) = 897 tonnes
Starboard (Leg 3) = 746 tonnes

The structural stiffness (jack-up fixity) shown in Figure 13 through Figure 15 as an assumed line connecting the spudcan moment occurring when the spudcan is fully restrained with the spudcan rotation occurring when the spudcan is free to rotate (pinned). This line represents the approximate position of where responses being compatible with soil and structural stiffness should end up for the load condition considered. This makes it possible to compare the moment – rotation behaviour according to ISO 19905-1 with the results from the PLAXIS 3D analyses.

The results obtained from the PLAXIS 3D analyses when using the forces presented above are presented in Figure 13. The solid lines represent the PLAXIS curves with no footprints, while the dashed lines represent the results including the effect from footprints. Only BOW and PORT legs have been assessed with footprints. It is however expected that the same trends as seen for BOW and PORT legs can be extrapolated for the STARBOARD leg. Hence, based on this comparison it can be concluded that the footprints have minimal effect on the rotational stiffness and could therefore be disregarded. The moderate effect is due to the low sensitivity and strength reduction of the clay in the disturbed zones and that the disturbed zones are local with limited extension in relation to the size of the CJ70 spudcans.

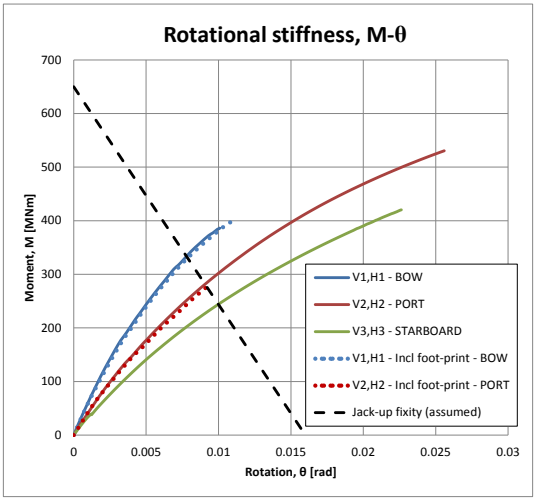

Figure 13 Moment – rotation curves from PLAXIS 3D – with and without footprints

Comparison moment – rotation between PLAXIS 3D and ISO 19905-1

This section documents the comparison between the results obtained from PLAXIS 3D and the moment – rotation curves when using the ISO 19905-1 [1]. To facilitate the comparison of the rotational stiffness documented in SSA [4] with the independent PLAXIS 3D analyses, input data (vertical and horizontal force and initial rotational stiffness) has been taken from the SSA and applied into the formulations for yield surface (limiting capacity for combinations of vertical force, horizontal force and moment) and for stiffness degradation as function of the utilisation of the yield capacity, all of which is described in ISO 19905-1. It is then possible to derive the moment – rotation behaviour for each of the legs based on the acting vertical and horizontal loads taken from the SSA analyses and to show as a check that the moment and rotation from the SSA analyses plots on the derived moment-rotation curve.

The comparisons for each leg are presented Figure 14 and Figure 15. Based on the comparison, following observations have been done:

- PLAXIS 3D demonstrates a significantly higher ultimate moment capacity, while the degradation formula in ISO 19905-1 makes the moment yield faster and at smaller rotations.
- The moment - rotation according to ISO 19905-1 mobilizes significantly more moment for smaller rotations compared to PLAXIS 3D, resulting in the maximum moment resistance becoming mobilised at a significantly lower rotation.
- Despite these differences there is a reasonably good match between PLAXIS 3D and ISO 19905-1 at the point where the ‘structural stiffness line’ intersects with the moment - rotation curves.

Based on the results it can be concluded that the rotational stiffness resulting from the SSA analyses is representative for this rig and governing load condition.

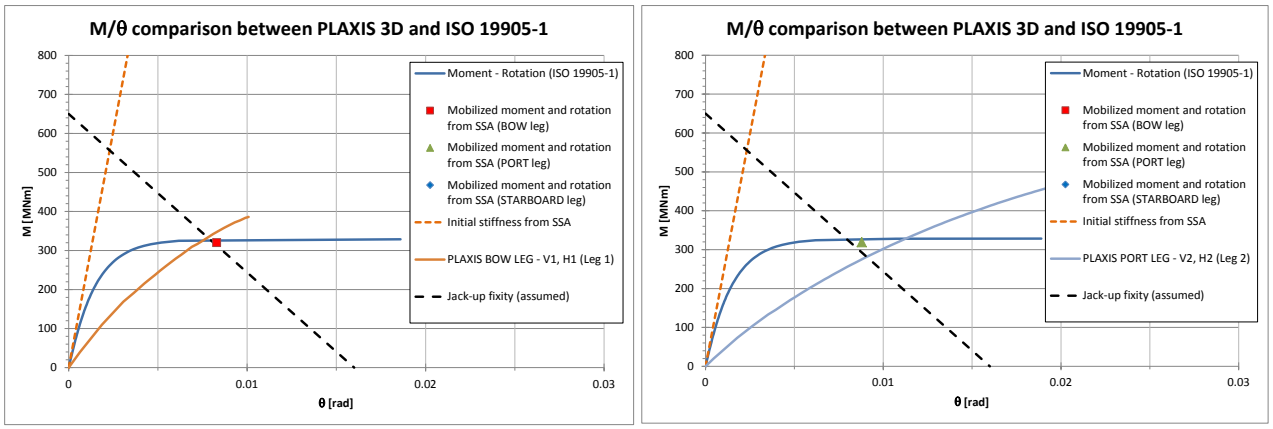

Figure 14 Moment – rotation, PLAXIS 3D vs. ISO 19905-1, Heading 30 degrees. Left: Leg 1. Right: Leg 2

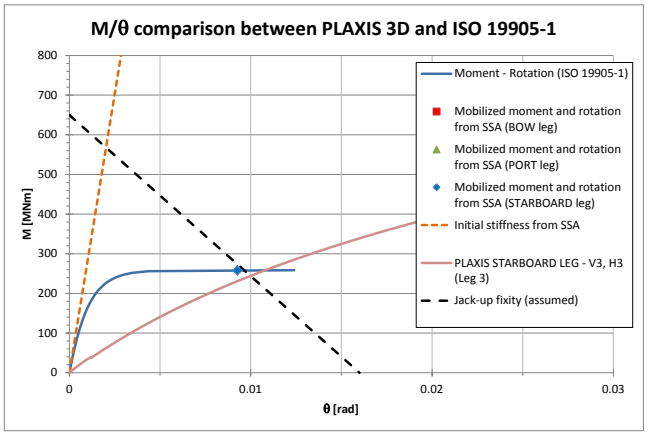

Figure 15 Moment – rotation, PLAXIS 3D vs. ISO 19905-1, Heading 30 degrees, Leg 3

Discussion of possible reasons for difference between PLAXIS 3D and ISO 19905-1 in terms of rotational stiffness

The good match between soil and structural compatible responses from our analyses as compared with the SSA results may be an occasional coincidence. The reason for the higher moment capacity achieved from the analyses is likely due to the following:

- In the SSA analyses the maximum moment capacity is related to the preloading (as if the preload is the limiting capacity), whereas the PLAXIS analyses is based on analysis with characteristic soil parameters and there are still some small margins between the preload and calculated vertical capacity.
- The relation between the maximum moment capacity and vertical capacity in the ISO formulations in combination with the yield surface formulations, may not fully account for the possibility of mobilising suction and may therefore underestimate the available capacity for skirted foundations.

The rotation by which the available moment capacity is mobilised using the ISO formulation as seen from the moment rotation curves is in our opinion considerably too low considering a reasonable stress-strain characteristic of the soil. We do not know the full background behind the development of the ISO formulations. However, we observe that in the last revision of [1] the recommendations for selecting the maximum shear modulus (G_{max}) has been changed to recommend higher values for clays – typically by a factor of 3. Thus, the initial stiffness input used in the stiffness degradation formula is increased by an equivalent factor. At the same time, the degradation formula has not been changed, with the result that the rotation corresponding to any moment has become 3 times lower.

The formulations presented in [1], are based on a combination of empirical correlations and solutions for idealised homogeneous soil conditions of either clay or sand, the latter assuming drained conditions. Accounting for site specific layered soil may result in different capacity relations. Also, accounting for that sands may behave almost undrained and accounting for cyclic effects could also alter the capacity relations.

The empirical correlations behind the ISO correlations are basically based on experience from Gulf of Mexico, where normally consolidated plastic clays prevail and the jack-up legs are deeply penetrated, which is different from the typical North Sea conditions. The normalised cyclic shear strength is probably higher for the Gulf of Mexico clay. Generally, there are many uncertainties when calibrating foundation stiffness from measured response. This may relate to measuring wave heights and periods, estimating wave forces and to deriving foundation stiffness from measured response, whether this is related to measured displacements or oscillation frequency. Calibrations may likely not relate to extreme design sea state, thus stiffness close to full utilisation may be difficult to verify.

Foundation capacities

The foundation capacity cannot for this project be proved through stability calculations with factored design shear strengths (γ_m=1.25) for a condition where the spudcan is not penetrated beyond full base contact. Therefore, the foundation stability during operation must rely upon the preloading as a proof loading test. The combined loading from vertical and horizontal forces and moment shall not cause a higher soil utilisation than that from the vertical preload divided by a material factor of 1.1.

The foundation capacity is documented using the BEARING1 program. The calculated safety factor in BEARING1 has explicitly not been used for documentation of the foundation capacity in ULS conditions, but has instead been used to find the relative safety factor between the operating loads and the preload. The footprints have not been accounted for in the bearing capacity evaluations. However, based on the BEARING1 results it was seen that relatively weaker sublayer 2B is governing the capacity.

The yield envelopes presented in the SSA are based on the formulations given in [1], which relates the capacity to the specified preload reaction. The shape of the yield curve is function of different factors, e.g. soil type and foundation embedment but for idealised conditions of homogeneous clay or homogeneous sand. For the current soil profile and the skirted spudcan it is believed that the yield curves presented in the SSA are on the conservative side. To investigate what effect the embedment has on the yield envelope for load combinations where the vertical component is dominating, series of BEARING1 runs have been performed and compared with a yield envelope taken from [4]. The results are presented in Figure 16.

The yield envelope used in the SSA is steeper, which is on the conservative side. It should be noted that for other conditions, particularly on soil with significantly increasing soil shear strength with depth, or a relatively thin layer governing the horizontal resistance, the idealized ISO curve may be on the non-conservative side.

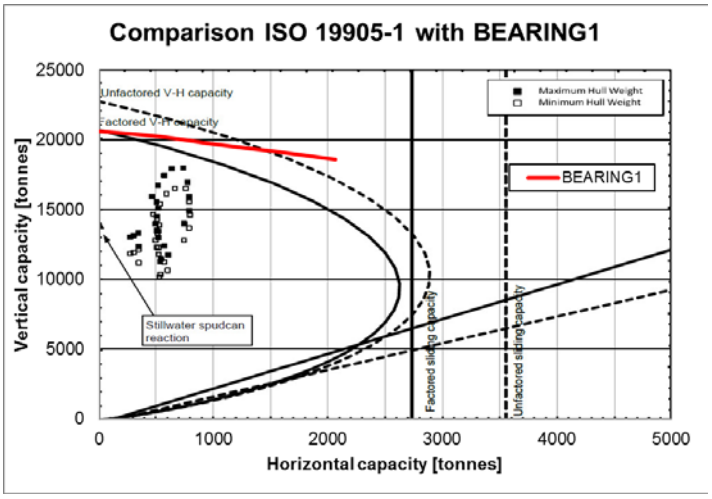

Figure 16 Comparison of the yield envelope between SSA and BEARING1

CONCLUSIONS

Based on the independent analyses carried out, following findings have been made:

- The penetration analyses based on closed form bearing capacity solutions show possible risk for punch-through or rapid penetration during preloading. However, based on the finite element results, such scenario could be discarded for the maximum prescribed preload reaction of 22700 tonnes. The main reason for the differences is believed to be that the finite element analysis better allows to account for the true effect of the soil layering. Furthermore, if the PLAXIS updated mesh option is applied, effects of heave or change of the soil stratigraphy boundaries will be accounted for. These are effects which are not be captured when using traditional "wished in place" closed form bearing capacity solutions.
- The foundation stability is found acceptable for the operating conditions using the traditional preloading concept. For this particular case, calculations performed using the software BEARING1, confirmed that the approach of determining the capacity envelope according to [1] was conservative.
- The moment - rotation according to [1] mobilizes significantly more moment for smaller rotations compared to PLAXIS 3D, i.e. having a significantly stiffer response. Despite large discrepancies in evaluation of maximum moment capacity and in required rotations for mobilization of moment resistance, the resulting rotational stiffness from the independent analyses in this case fits reasonable well with the resulting stiffness documented in the SSA for the governing load conditions.
- The effect of having local weaker zones beneath the foundation due to footprints from previously installed jack-ups is for this case found to be negligible in terms of soil-structure interaction. The moderate effect is believed to be partly due to the low sensitivity and strength reduction of the clay in the disturbed zones and mainly because the disturbed zones are local with limited extension in relation to the size of the CJ70 spudcans.

As results from this study show, the soil-structure interaction results may differ when comparing spudcan penetration, rotational stiffness and capacity results based on ISO 19905-1 [1] compared to results from a more detailed assessment by means of finite element. This is believed to be especially true for sites with complex soil conditions. Therefore, for situations where foundation or structural utilization is close to unity, the site has challenging soil conditions, cyclic response needs to be accounted for, there is possible interaction with adjacent structures and/or for long term rig placements, supplementary analysis by means finite element are justified and could prove to be very useful.

REFERENCES

[1] ISO 19905-1, Petroleum and natural gas industries – Site-Specific assessment of mobile offshore units – Part 1, Jack-ups, First edition, 2012.

[2] Sleipner Soil Investigation 1991 – Sleipner B, Soil parameters for design, NGI report 912535-1, Revision: 1, dated 29th January 1993

[3] BEARING1, A computer program for analysis of the bearing capacity of a foundation on clay subjected to undrained loading, Revision: 2, dated 15th May 1992

[4] Site specific assessment – Maersk integrator at Sleipner B, Noble Denton Marine Services, Report No. L31196 Revision: 0, dated 1st February 2016

[5] PLAXIS 3D 2016 version, Finite Element Code for Soil and Rock Analysis, PLAXIS b.v., the Netherlands

[6] PLAXIS 2D AE.02 version, Finite Element Code for Soil and Rock Analysis, PLAXIS b.v., the Netherlands

[7] Sleipner Vest West Epsilon – Removal and reinstallation, GEOVEST Report No 99.087-1, Revision: 0, dated 11th December 1998