IMPACT OF CYCLIC LOADING ON SOIL BEHAVIOR / JACK-UP FOUNDATIONS CAPACITIES & NEED FOR DESIGN / CONSTRUCTION OF SEABED INTERVENTIONS

L. Kellezi*
Geo, Copenhagen, Denmark,
P. E. Gjerde
AkerBP, Stavanger, Norway

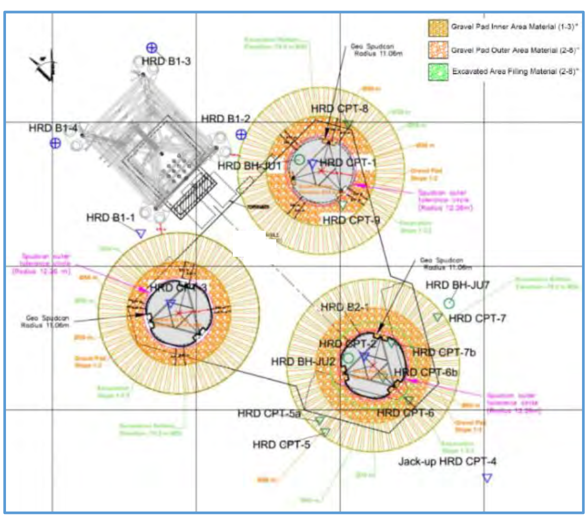
**corresponding author: Lindita Kellezi: LKE@geo.dk*

ABSTRACT
Due to interpreted critical soil conditions with regard to cyclic loading and cyclic soil stiffness & strength degradation based on cyclic laboratory testing of sand and clay soil samples, following DNVGL-RP-C212 guidelines, design and construction of seabed interventions (SIs) was found necessary at a Jack-up drilling location in the Norwegian Sector, North Sea. The SIs were designed to act, together with skirted spudcans, as foundations, to support cyclic loading for all year metocean conditions (100-year wave and wind with 10-year current, all-year directional environmental extremes), applicable to installation and operation of a CJ70 type Jack-up drilling rig. The water depth at the site is about 73m MSL. The work is based on the references listed at the end of the paper, and other important inputs and contributions from AkerBP & Sub-Contractors, and Geo.

KEY WORDS: Skirted spudcan; Cyclic loading; Drained anisotropy ratios; Undrained anisotropy ratios; Foundation cyclic capacity; Finite element analyses; Seabed Interventions (SI); Gravel bank (GB): Dredging:

INTRODUCTION & FINAL DESIGN / CONSTRUCTION

Due to soil variation, the finally designed and constructed seabed interventions (Sis) are of variable size at the three legs and consist of excavations, rock filling, and further dumping / constructing gravel pads (GPs). All three excavations are designed with slopes 1:2.5, where the depth of the excavation is 4m below seabed (bsb) for the Aft Legs, and 6m bsb for the FWD/BOW Leg (where cyclic testing of Sand units were carried out). The bottom diameters of the excavations are 36m, 38m and 30m for the STB, PS and FWD Legs, respectively. All three GPs are designed with slopes 1:2 and 5m height. The top diameters of the GPs are 36m, 38m and 40m for STB, PS and FWD Legs, respectively. The resulting total heights of the SIs (excavation + GP) are thus 9m for Aft Legs, and 11m for FWD Leg. More details in the design and construction might follow in another paper. Fig 1 shows location plan for the design conditions (a) and bathymetry survey after construction of SIs (b).



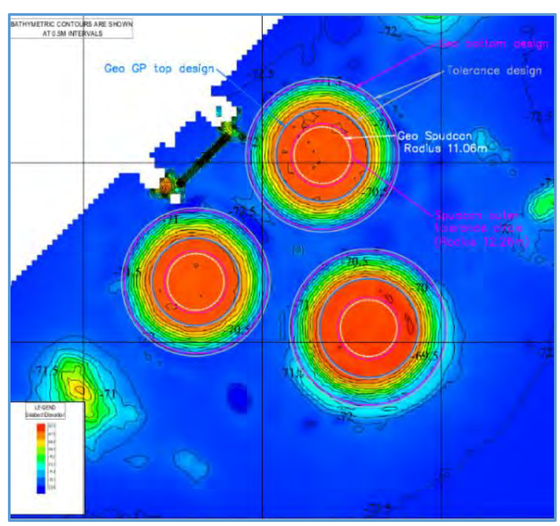


Figure 1 a) Location Plan b) Bathymetry Survey after SIs Construction

While no critical conditions are predicted for the installation phase, the purpose of the SIs is to remove part of the upper soils, (considered weak and sensible to cyclic loading), and help the bearing capacity of the remaining soil (through load spreading). This way, any non-uniform jack-up / leg settlements, due to local soil strength and stiffness degradation under rotation moment (M) cyclic loading (assuming spudcan fixities), while the rig is in operation mode with more than 40 m airgap, are avoided.

The SIs are constructed with (2-8)''(inch) (about (50-200)mm) rock material for the fill and the outer areas of the GPs, and (1-3)''(inch) (about (25-75)mm) gravel material for the inner areas of the GPs. The less coarse material is used to facilitate the penetration of the spudcan skirts to full base contact. Fig. 2 below shows the cross sections and dimensions of the GPs for each leg, respectively.

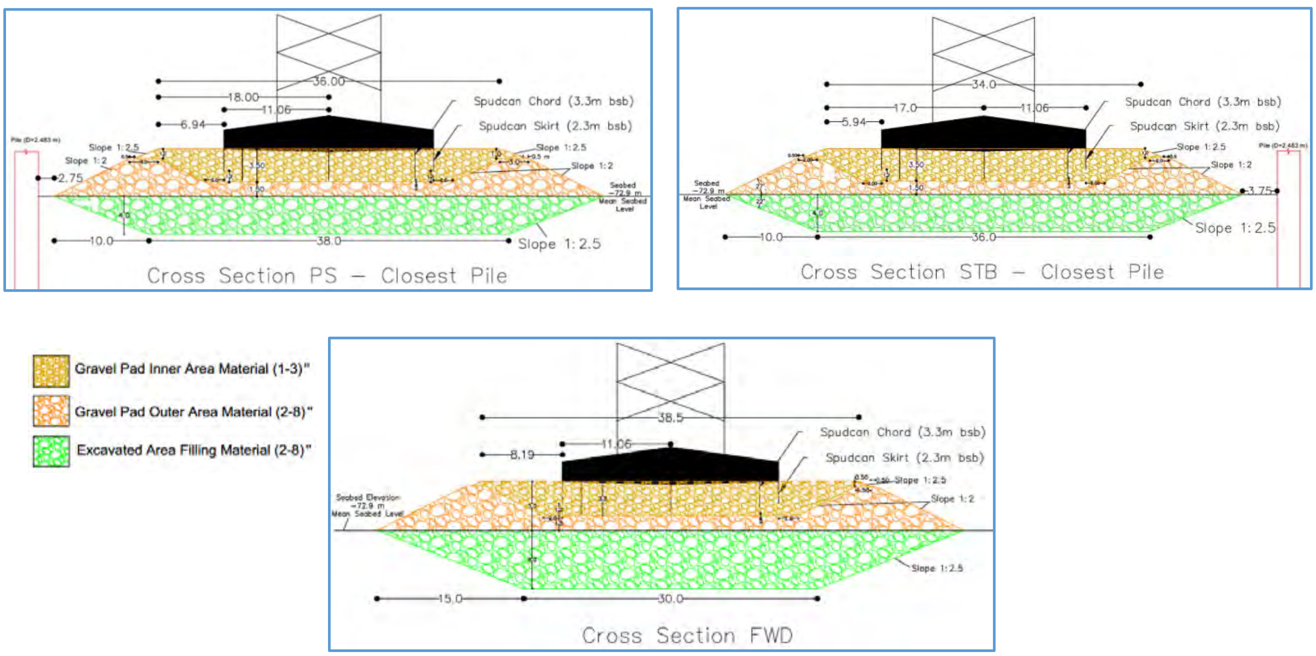


Figure 2 SI Cross Sections – Dimensions & Materials

SKIRTED SPUDCANS & DESIGN CYCLIC LOADS

The Jack-up rig skirted spudcans have an equivalent diameter of 22m and full contact area of about 384m^2 with outer skirts to 2.3m and three chords to 3.3m below spudcan largest contact area. The maximum preload capacity is about 215.7MN/leg (22.000tons/leg) and Still Water Reactions (SWR) vary for drilling and non-drilling / storm survival conditions from about (112-156)MN/leg. Distance between centres of skirted spudcans is 70m.

Regarding VHM design cyclic loads, four load scenarios (LSs) are derived from structural site specific assessment (SSA) contractor, one for non-drilling / storm survival most critical for FWD Leg (LS1) and three for drilling, most critical for each leg respectively (LS2/FWD, LS3/PS, LS4 / STB).

SOIL CONDITIONS & CYCLIC TESTING

Geotechnical soil investigations carried out at the site consist of BH/CPTs plotted in the location plan in Fig.1a.The identified soil units to the depth of interest include sands of the Fourth Formation (Unit II), and clays & sands of the Fisher Formation (Unit III), both susceptible to cyclic loading. The top Holocene sands / silts are considered non-bearing. The soil conditions are quite variable between the three leg locations, and also within each SI footprint area as shown in the CPT cross sections given in Fig.3.

The standard onshore laboratory testing has been extended with a series of advanced tests. The available information for the assessment of the cyclic capacities of soil units included tests like, DSS, CV-DSS, DSScy, CADc, CADe, CAUc, CAUe, CAUcy and based on them, the site specific cyclic contour diagrams were developed by the lab contractor. Cyclic tests are carried out with period of vibration T=10s (similar to assumed metocean loading) and in almost all the cyclic tests on BH-JU2 (FWD SI), the specimens have been consolidated to a common preconsolidation pressure of 200kPa. For the clay the data fits reasonably well with contour diagrams for Drammen Clay OCR 2.

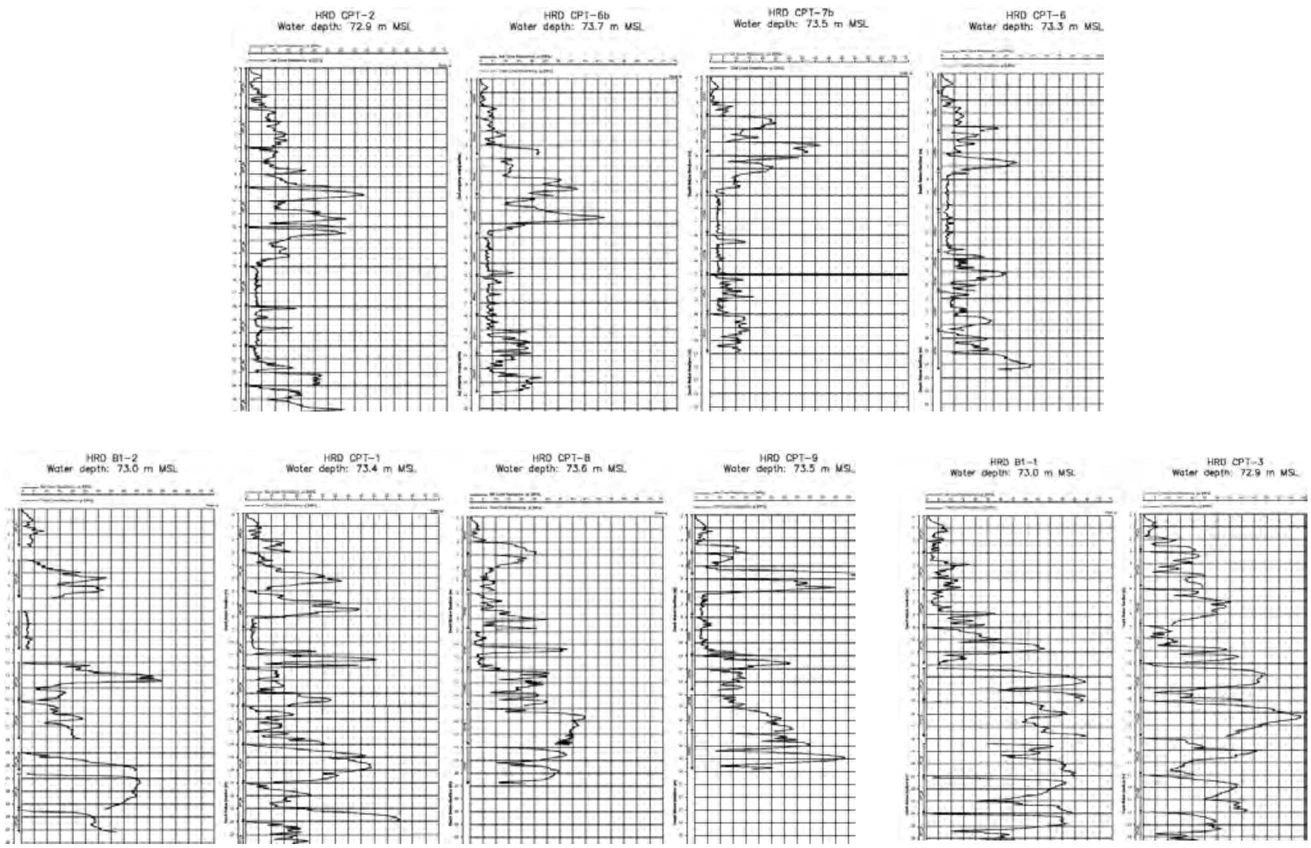


Figure 3 CPT Cross Sections: FWD Leg (up), PS Leg (left) STB Leg (right)

Based on the available soil data and mainly on the CPT cross sections in Fig.3, generalized Best Estimate (BE) characteristic soil profiles (SPs) are assessed for each leg, applied first for static load conditions.

- The BE SP at FWD Leg location is based primarily on the CPT-2 and BH JU2. However, due to higher position of Unit III clay in CPT-6b and CPT-7b, positioned at the edge of the spudcan footprint, the profile is generalized including this clay at a representative level, higher than encountered in CPT-2.
- The BE SP at PS Leg location is primarily based on CPT-1, while CPT-8 indicates a lower capacity profile with weaker sandy/clayey deposit at a higher level, and the level and thickness of clay at B1-2, near the jacket pile, and at CPT-9, is higher than at CPT-1 and similar to the adverse condition interpreted from B1-2. These are incorporated as a thicker clay layer in the BE profile with the top similar to the top of clay indicated in CPT-9.
- The BE SP at STB Leg location is primarily based on CPT-3. There are currently no other CPT/BHs within the footprint of the spudcan and planned SI. However, the neighbouring B1-1 at the position of the jacket pile shows adverse conditions in comparison to CPT-3.

DESIGN PRINCIPLES & CONSIDERATIONS

The assessments of the reference and cyclic capacities for all three legs, respectively, are based on FE modelling of skirted spudcan-SI-soil interaction carried out in Plaxis FE program, applying 2D FE axisymmetric and 3D FE models. Axisymmetric models are made for the evaluation and calibration of the SI geometries and material parameters, as an input to the 3D FE models carried out for the evaluation of fixity in the considered LSs. FE analyses are performed using the BE (characteristic) SPs for each leg, respectively, and based on the following assumptions:

- Axisymmetric FE mesh has been generated using 15-noded triangular Fes; The models are built with refinement of (4000-4500) FEs; Modelling of skirted spudcan-SI-soil interaction is carried out assuming equivalent vertical loads including the contributions of vertical loads (V) and moments (M) in order to define the extent of SI; The initial geostatic conditions are calculated first; Plaxis Plastic analyses (small deformation theory) are carried out; The spudcan is modelled as a weightless elastic (drained) body with full base contact on top of GPs, hence penetrations 3.3m referring to chord tip; The skirts are modelled employing plate elements; As the baseline for the cyclic capacity assessments, the models for SWR state are based on:
    - Sand modelled in drained conditions using Mohr-Coulomb plastic constitutive model.
    - Clay modelled in undrained conditions using Linear elastic / Tresca plastic constitutive model.

  Hardening soil models showed similar results. The elastic deformation parameters (E moduli) for sands are calculated based on the CPT data and for clay based on undrained shear strength (c_u), assuming $E = 300*c_u$; The capacity analyses are displacement-driven. Vertical reference cyclic capacities are evaluated for 1.0m skirted spudcan penetration.

- 3D FE models are generated as half space models in which, the horizontal force (H) direction lays on the plane of symmetry and the M axis is orthogonal to the plane of symmetry; 3D mesh has been generated using 10-noded tetrahedral FEs; The models are built with refinement of about 100000 FEs; 3D modelling utilized scaling factor for the loads to compensate for the effect of different FE type and mesh refinement between axisymmetric and 3D analyses. The factor is obtained as a ratio between the vertical cyclic capacities in 3D and axisymmetric analyses.

Calculated Target Vertical Cyclic Capacities (VCC)

In order to preliminary dimension the SIs and calibrate the material models for the 3D FE analyses, as mentioned above, axisymmetric FE models are made for the three leg locations with two SWRs per leg (6 axisymmetric FE models in total). As Plaxis axisymmetric models cannot include H and M loads, in order to account for the high M loads, the target vertical cyclic capacities (VCC) were derived based on the reduction of spudcan area due to the eccentricity caused by the M.

The design target VCCs are calculated taking a safety factor (SF) of 1.25. The initial analyses are based on the loads derived from the SSA for the fixity conditions, including the four LSs, one for non-drilling (LS1) and three for drilling (LS2, LS3, LS4), mentioned above. This showed however to be conservative in comparison to the 3D FE results. Therefore, a new set of LSs based on pinned conditions on one leg and fixity on the other two legs was derived from the SSA for the special purpose of application in axisymmetric analyses.

Upon establishing the geometry and material models in axisymmetric analyses for each leg, 3D analyses for evaluating foundation fixities are based directly on the four fixity LSs (Six 3D FE models and twelve LSs in total). Foundation bearing capacity checks and stiffness curves for VHM are derived for each LS to account for the effect of cyclic degradation. This is an iterative procedure carried out until structural and foundation checks are satisfied.

Procedure for Evaluation of Cyclic Capacity

The procedure, proposed by certifying contractor, consisted of:

- Establish consolidation conditions during preloading presuming that consolidated loads correspond to SWRs, so, no detailed consolidation analysis is carried out;
- Carry out static analyses with BE SPs for SWR loads as described above, with sand layers considered fully consolidated for the SWRs and the excavation infill / rock dump materials modelled as drained Mohr-Coulomb materials too, and calculate vertical effective stresses at different zones;
- Calculate static reference capacity and initial mobilization. In this case the natural soil materials in the zone of interest are modelled in Plaxis using drained NGI-ADP soil model with parameters derived based on site specific contour diagrams available (Unit II and Unit III at BH-JU2/FWD Leg);
- Calculate equivalent number of cycles. Detailed analyses of N_{eq}, based on strain or pore pressure accumulation has not been carried out, but it is conservatively agreed that $N_{eq}=10$ is sufficiently representative;
- Calculate load paths. It is assumed that strains at failure for DSS, triaxial compression (TXC) and extension (TXE) states are compatible and the cyclic soil parameters are obtained considering relevant load paths;
- Determine cyclic strength from cyclic contour strength diagrams. The initial mobilization is presumed to be globally uniform and evaluated as the ratio of SWR in relevant operation mode (drilling or storm survival / non-drilling), to the vertical capacity (VC) for drained conditions;
- Calculate cyclic anisotropy factors at failure from the triaxial cyclic diagrams derived from the site specific contour diagrams where available. Otherwise, the anisotropy factors are conservatively taken in agreement with the available contour diagrams for Unit II as suggested by the lab contractor;
- Determine nonlinear soil model accounting for cyclic effects modelled using drained NGI-ADP model with the parameters scaled using the previously derived anisotropy factors:
- Calculate the undrained cyclic capacity. The natural soil materials in the zone of interest are modelled using undrained NGI-ADP model. In reference to DNV-RP-C212, SF=1.25 is applied to obtain design values based on the characteristic BE capacities.

Anisotropy Ratios-Site Specific Cyclic Contour Diagrams for Unit II (Sand)

Table 1 presents the main results of the DSS test for the evaluation of the anisotropy ratios in drained Unit II.

TABLE 1 Overview of the Direct Simple Shear (DSS) Tests on Sand/Unit II

Boring No.	Tube part test	Type of test 1)	STATIC TESTING			
			τ_f kPa	U_f kPa	γ_f %	τ_f/σ'_{vc} -
HRD B1-2	BATCH-2-A	CV-DSS	79.5	114.9	1.0	0.22
HRD B1-2	BATCH-2-cyc01	DSScy				
HRD B1-2	BATCH-2-cyc02-1	DSScy				
HRD B1-2	BATCH-2-cyc02A	DSScy				
HRD B1-2	BATCH-2-cyc03	DSScy				
HRD-BH-JU2	BATCH-1-A	CV-DSS	31.94	71.9	1.0	0.16
HRD-BH-JU2	BATCH-1-B	DSS	78.8	2.1	15.0	0.39
HRD-BH-JU2	BATCH-1-C	CV-DSS	33.8	37.2	1.1	0.27
HRD-BH-JU2	BATCH-1-A	CV-DSS	53.8	24.3	0.8	0.27

Table 2 presents main results of triaxial tests used to evaluate anisotropy ratios in drained conditions on sand.

Table 3 presents the main results of the tests for evaluation of drained anisotropy ratios in Unit II. The ratios τ_f/σ'_{ref} of failure strength τ_f to reference vertical stress σ'_{ref} presented in the table are obtained for the reference strengths calculated with the formula as shown in Table 3 where p_a is the atmospheric pressure (100kPa) and σ'_c is the vertical consolidation stress.

TABLE 2 Overview of the Drained Triaxial (CAD/CID) Tests on Sand/Unit II

Boring No.	Tube part test	Type of test 1)	Perm k	Precycling τ_{cy}	Precycling N_{cy}	STATIC TESTING τ_f	ε_f	ϕ'	τ_f/σ'_{ac}	COMMENTS
			m/s * 10⁻⁷	kPa	-	kPa	%	deg	-	
HRD B1-2	4WAXA-1	CADC	-			233.8	8.0	37.0	1.29	Described as CLAY
HRD B1-2	BATCH2-A	CADC	-			277.9	24.8	31.5	0.76	Reconstituted
HRD B1-3	2BAGB-A	CADC	484.5			15.4	0.9	35.4	1.18	Reconstituted
HRD B1-3	BATCH5-A	CADC	-			165.9	10.0	31.1	0.90	Reconstituted
HRD B1-4	BATCH1-A2	CADC	80.2			93.5	1.9	40.3	1.56	Reconstituted
HRD B1-4	BATCH3-A	CADC	-			250.3	2.2	34.1	1.09	Reconstituted
HRD B1-4	BATCH4-A	CADC	-			269.5	1.6	35.3	0.82	Reconstituted
HRD BH-JU2	BATCH1-CAD	CADC	1.2	8	400	91.4	10.0	30.9	0.46	Reconsituted. Bender element Gmax = 55.5 MPa
HRD BH-JU2	BATCH1-3	CADE	-	8	400	-30.3	-7.9	30.5	-0.15	Tested by unloading vertical stress

$(su,E / su,C)_{min} = 30.3 / 91.4 = 0.33$

TABLE 3 Anisotropy Ratios in Sand/Unit II Applicable for Drained Conditions

$$s_{u,D} = \left(\frac{\tau_f}{\sigma'_{ref}}\right)_{drained,DSS} \cdot \sigma'_{ref}$$

$$s_{u,C} = \left(\frac{\tau_f}{\sigma'_{ref}}\right)_{drained,CAUc} \cdot \sigma'_{ref}$$

$$s_{u,E} = \left(\frac{\tau_f}{\sigma'_{ref}}\right)_{drained,CAUe} \cdot \sigma'_{ref}$$

DRAINED			
for σ'ac=	200	kPa	
σ'ref	186.6	kPa	$\sigma'_{ref} = p_a * (\sigma'_c/p_a)$^0.9
Anisotropy factors			
su,D=σ'ref*	=78.8/186.6	=0.42	
su,A=σ'ref*	=91.4/186.6	=0.49	
su,P=σ'ref*	=30.3/186.7	=0.16; up to 0.49 for su,P=su,A	

The ratios τ_f/σ'_{ref} applicable to the evaluation of the total capacity in cyclic conditions along a DSS path are obtained as sums of the drained mobilization τ_0/σ'_{ref} and corresponding readouts τ_{cy}/σ'_{ref} from the contour plots for applicable failure states (DSS, TXC, TXE). Readout points are found on the crossing of the DSS path (vertical) and the relevant contour (N=10). For the inclined load paths, the ratios τ_f/σ'_{ref} are obtained as sums of projections of the readouts, where the readout points are found on the crossing of the inclined path and the relevant contour.

Fig.4 and Fig.5 illustrate the evaluation of the anisotropy factors for the cyclic conditions, based on contour plots for DSS and Triaxial conditions, respectively, for Unit II, N=10, τ_0/σ'_{ref}=0.5 (mobilization for example SWR/VC=0.5 calculated for the BE SP). The calculated undrained anisotropy factors are presented in Table 4. The presented ratios are indicating failure strains of γ_a~10% and γ_{cy}<0.05% in all the failure modes and are thus considered compatible.

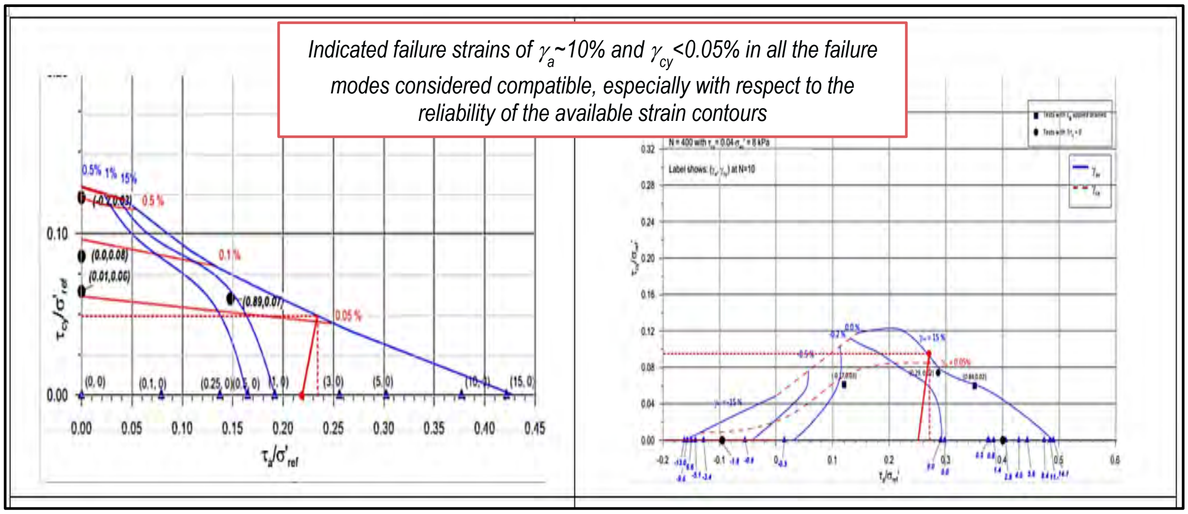


Figure 4 DSS Contour Readout for a chosen Load Path   *Figure 5 Triaxial Contour Readout for a chosen Load Path*

TABLE 4 Example of Anisotropy Ratios in Sand / Unit II Applicable for Cyclic Conditions

	DSS	C	E
Drained anisotropy factor	0.425	0.49	0.16
Mobilized average, τ0/σ'ref	0.212	0.244	0.080
Cyclic, τcy/σ'ref	0.065	0.115	0.025
total, τf/sv'	**0.277**	**0.359**	**0.105**
DSS/A	0.771		
P/A	0.292		

Anisotropy Ratios from Reference Diagrams for Unit II Stronger Sand

Lab contractor suggested contours presented in Fig. 6. In the final analyses, the parameters are evaluated based on those contours for N=10 DSS contour, and N=100 Triaxial contour plots, following the strain compatible load paths.

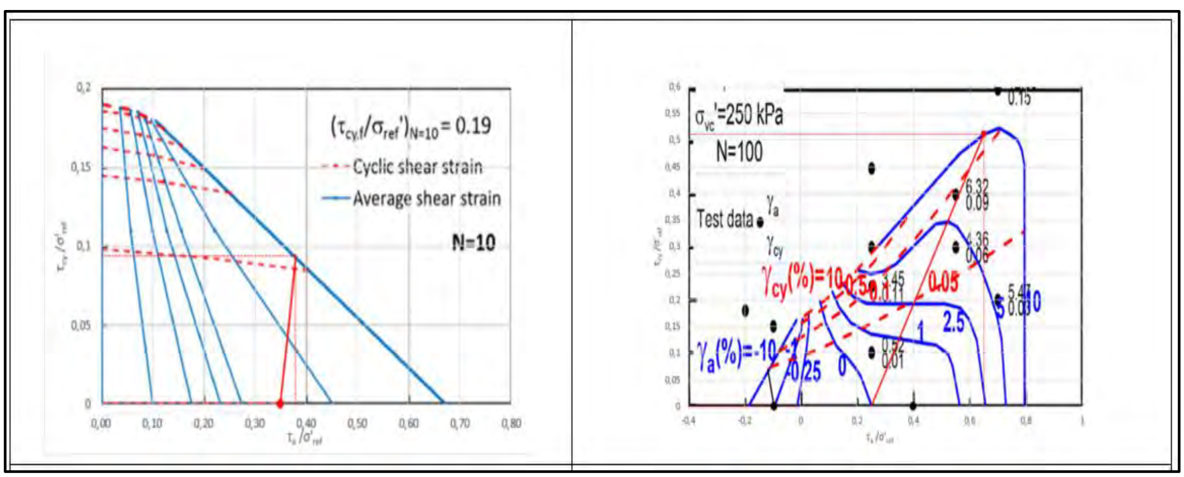


Figure 6 Example of Considered DSS & Triaxial Contours for the More Competent Sand of Unit II

Anisotropy Ratios Unit III, Clay & Different Sands of Unit II

In order to establish the reference capacity, (VC), the drained anisotropy ratios are based on su,D/su,C=0.75. Based on a mobilization ratio SWR/VC=0.5, the cyclic anisotropy ratios for Unit III clay are evaluated from the contour plots for Drammen clay OCR 2. Example of readouts from the DSS and Triaxial cyclic contour diagrams, Unit III, N=10, τ_0/σ'_{ref} =0.5 are shown in Fig. 7. This exercise showed that in the zone of the failure propagation through the clay of Unit III, the DSS shear strength in cyclic conditions remains similar to,
or exceeds, the DSS shear strength in static conditions. In line with that, the subsequent analyses are carried out utilizing the BE static shear strength in clay as a safe simplification of the model.

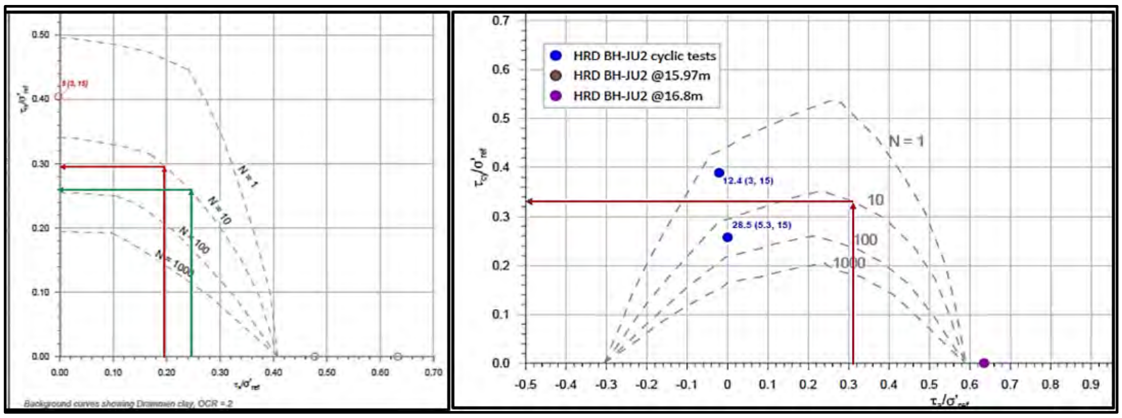


Figure 7 Example of Readouts from the DSS and Triaxial Cyclic Contour Diagrams, Clay / Unit III, N=10, τ_0/σ'_{ref} =0.5.

TABLE 5 Example of Calculated and Assumed NGI-ADP Drained Anisotropy Ratios for Sands/UNIT II & Clay/Unit III

Drained Anisotropy ratio	Unit II Less competent sand	Unit II More competent sand	Unit III Clay
su,A/σ'ref	0.49	0.80	0.58
su,D/σ'ref	0.42	0.67	0.46
su,P/σ'ref	0.42	0.67	0.46
su,D/su,A	0.86	0.84	0.75
su,P/su,A	0.86	0.84	0.75

TABLE 6 Example of Calculated and Assumed NGI-ADP Undrained Anisotropy Ratios for Sands/UNIT II & Clay/Unit III

Undrained Anisotropy ratio	Unit II Less competent sand	Unit II More competent sand	Unit III Clay
su,A/σ'ref	0.36	1.09	0.62
su,D/σ'ref	0.26	0.435	0.49
su,P/σ'ref	0.26	0.435	0.49
su,D/su,A	0.73	0.4	0.8
su,P/su,A	0.73	0.4	0.8

Cyclic Load Paths
Initial analyses are based on the readouts from the contour diagrams following the DSS load path. This path sets on the horizontal axis from a point defined by the mobilization of static capacity (VC) and is vertical until it crosses the N=10 contour, both in DSS and triaxial contour diagrams. Final analyses are based on the readouts from the contour diagrams following the strain compatible load paths defined by the cyclic M. The influence of V and H forces on the load path defined based solely on the variation of M is negligible. For the strain compatible M load paths, firstly the readouts are obtained from the DSS contour along the M load path. Based on this, the readout for the M load path from the triaxial diagram is adjusted to achieve strain compatibility between the DSS and triaxial failure states.

Elastic Soil Deformation / Stiffness Parameters
For the application with NGI-ADP models, a BE of the shear modulus, G_{max}/G_0, is evaluated from the CPT and available lab data and checked with CPT interpretation software output. In order to simulate the constant shear modulus throughout the respective zones of each layer, for both drained and undrained conditions, a variation of G_{ur}/s_uA is applied based on the variation of the s_uA through the zones and conditions within a layer. While strength decreases from active to passive areas, stiffness ratio G_{ur}/s_uA increases.

DESIGN OF SI AT FWD (FORWARD) LEG LOCATION
Due to differences in SWRs the FE models are respectively built-up for:
- Storm survival / non-drilling mode LS1, 2D FE axisymmetric and corresponding 3D FE.
- Drilling mode LS2, 2D FE axisymmetric and corresponding 3D FE, which served also for LS3 and LS4.

Based on the generalized characteristic BE SP, seven zones are defined for the more competent sand layers above 7.6m bsb, and additional six for the less competent sand layers between 7.6m bsb and the top of the clay of Unit III. The zones based on the SWRs for two respective loading modes (drilling & non-drilling) are shown in Fig.8.

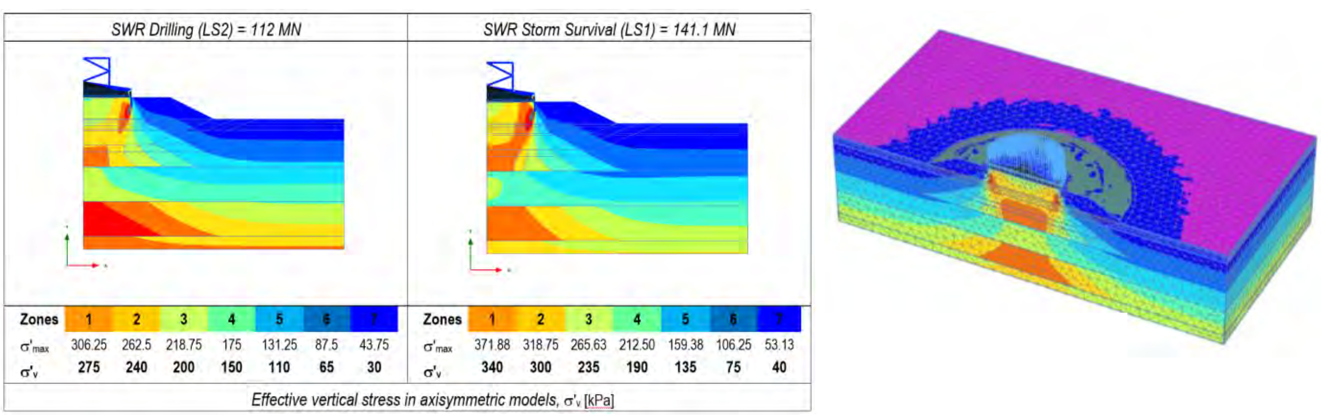


Figure 8 NGI-ADP Zones in Axisymmetric Models for FWD Leg Location

Based on soil data G_{max} is set to 75MPa for the weaker Sand and 105MPa for the denser most competent sand between (7.6-11.0)m bsb. For the clay layer, G_{max}= 85MPa is set. The parameters of NGI-ADP model applicable to the cyclic capacity assessments are derived based on contour plots presented above according to the calculated load paths.

Vertical reference capacity (VC) evaluated for the penetration of 1.0m based on the axisymmetric Plaxis model using drained NGI-ADP material models are:
- VC = 264 MN for drilling conditions.
- VC = 275 MN for non-drilling conditions i.e. storm survival mode.

Based on mobilization and the derived undrained cyclic parameters vertical cyclic capacities (VCC) are calculated:

- VCC=238 MN for drilling conditions
- VCC=269 MN for non-drilling conditions i.e. storm survival mode.

A general overview of the 2D & 3D FE models for LS1 and final 3D result of failure pattern from safety analysis is given in Fig. 9

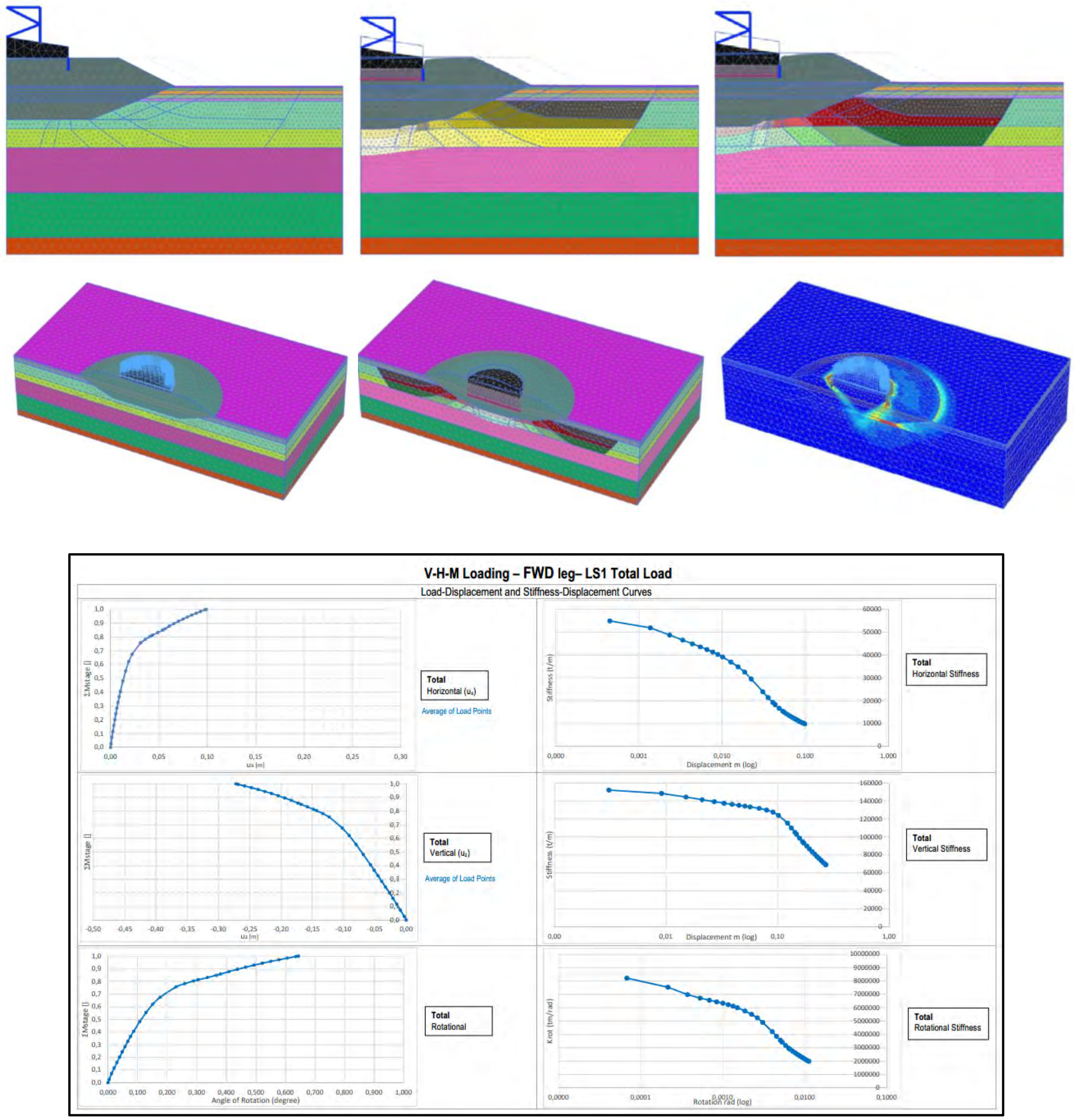


Figure 9 2D & 3D FE Models for FWD Leg, (Non-Drilling) LS1

Based on the calculation of the vertical cyclic capacity in 3D models, the following correction factors (CF) between 3D and axisymmetric models are obtained:

- for the storm survival / non-drilling mode: CF=1.15
- for the drilling modes: CF=1.21

The correlation factors are applied on the VHM loads for each LS. Safety calculations are performed afterwards. Results of the foundation capacity / safety envelopes for each of the LS are presented in Table (7 & 8). In addition, stiffness are calculated for each LS and results for VHM loads are also presented in Fig 9 for the most critical LS1.

DESIGN OF SI AT PS (PORT) LEG LOCATION

Due to differences in SWRs the FE models are respectively built-up for:

- Drilling mode LS3, 2D FE axisymmetric and corresponding 3D FE, which served also for LS2 and LS4.
- Storm survival / non-drilling mode LS1, 2D FE axisymmetric and corresponding 3D FE.

Based on the generalized characteristic BE SP, seven zones are defined for the less competent sand layers from (3-4)m (on the side of excavation) and (5.5-6.5)m, and seven zones for the remaining most competent sand layers. The zones based on the SWRs for the two respective loading modes (drilling and non-drilling) are shown in Fig.10.

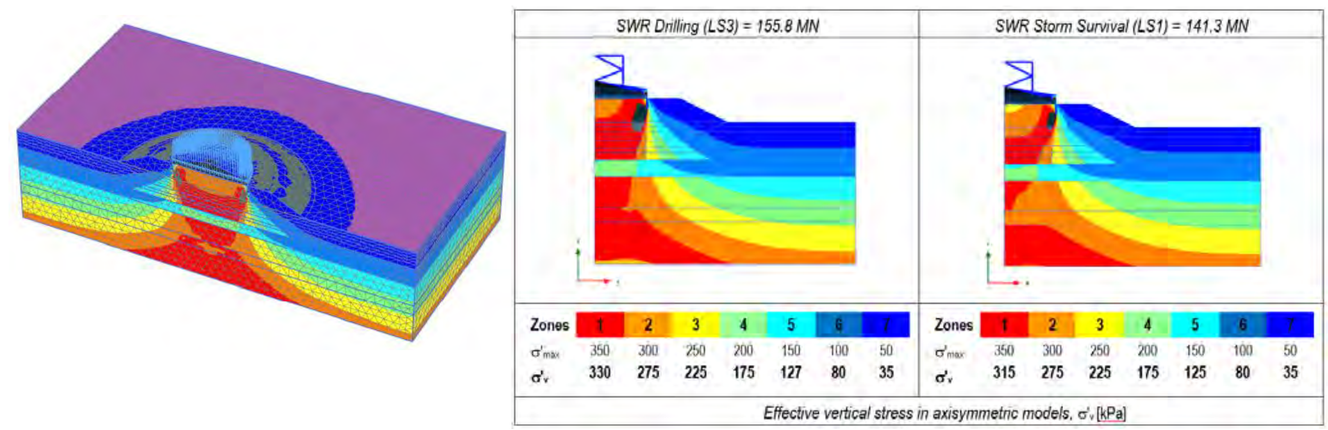


Figure 10 NGI ADP Zones in Axisymmetric Models for PS Leg Location

Based on soil data G_{max} is set to (50-77)MPa for all sand layers between 2.5m bsb and clay layer, and 68MPa for the clay layer. The parameters of NGI-ADP model applicable to the cyclic capacity assessments are derived based on contour plots presented above according to the calculated load paths.

Vertical reference capacity (VC) evaluated for the penetration of 1 m based on the axisymmetric Plaxis model using drained NGI-ADP material models are:

- VC = 245 MN for drilling mode.
- VC = 234 MN for non-drilling conditions i.e. storm survival mode.

Based on mobilization and the derived undrained cyclic parameters, vertical cyclic capacities (VCC) are calculated:

- VCC=273 MN for drilling conditions
- VCC=246 MN for non-drilling conditions i.e. storm survival mode.

A general overview of the 2D & 3D FE models for LS4 and final 3D result of failure pattern from safety analysis is given in Fig. 11

Based on the calculation of the vertical cyclic capacity in 3D models, the following correction factors (CF) between 3D and axisymmetric models are obtained:

- for storm survival / non-drilling mode: CF=1.21
- for drilling modes: CF=1.20

The correlation factors are applied on the VHM loads for each LS. Safety calculations are performed afterwards. Results of the foundation capacity / safety envelopes for each of the LS are presented in Table 9. In addition, stiffness are calculated for each LS and results for VHM loads are also presented in Fig 11 for the most critical load scenario LS3.

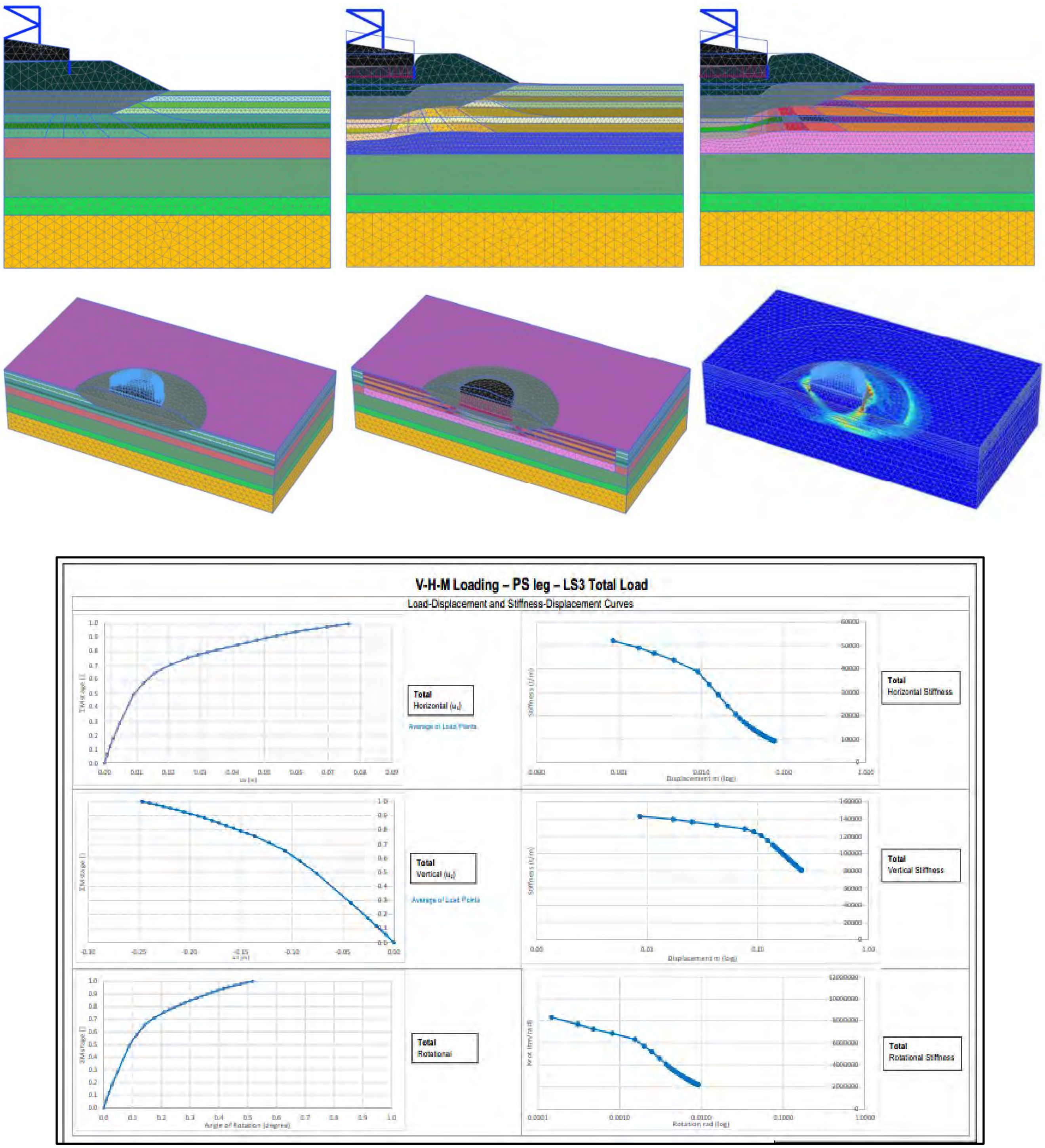


Figure 11 2D & 3D FE Models for PS Leg, (Drilling) LS3

DESIGN OF SI AT STB (STARBOARD) LEG LOCATION

Due to differences in SWRs the FE models are respectively built-up for:

- Drilling mode LS4, 2D FE axisymmetric and corresponding 3D FE, which served also for LS2 and LS3.
- Storm survival / non-drilling mode LS1, 2D FE axisymmetric and corresponding 3D FE.

Based on the generalized characteristic BE SP, seven zones are defined for the top competent sand layer from (1.8-8.5)m bsb, five zones for the bottom competent sand layer from (10.5-19.6) m bsb and six zones for the middle less competent sand layer from (8.5-10.5)m bsb. The zones based on the SWRs for the two respective loading modes (drilling and non-drilling) are shown in Fig.12.

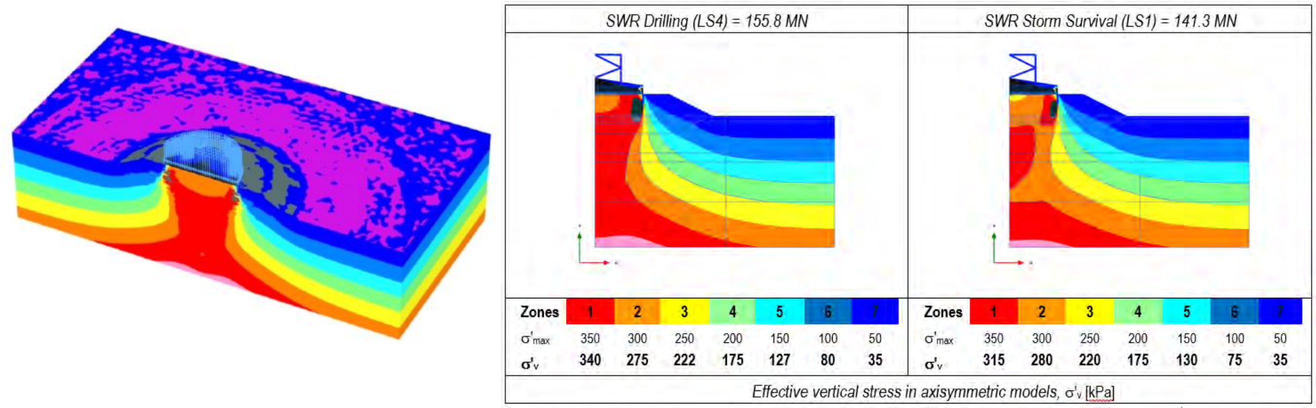


Figure 12 NGI ADP Zones in Axisymmetric Models for STB Leg Location

Based on soil data G_{max} is set to (50-75)MPa for the less competent sand layer and for the competent sand layers is set to (57.5 75.0)MPa. The parameters of NGI-ADP model applicable to the cyclic capacity assessments are derived based on contour plots presented above according to the calculated load paths.

Vertical reference capacity (VC) evaluated for the penetration of 1 m based on the axisymmetric Plaxis model using drained NGI-ADP material models are:

- VC = 298 MN for drilling mode.
- VC = 286 MN for non-drilling conditions i.e. storm survival mode.

Based on mobilization and the derived undrained cyclic parameters, vertical cyclic capacities (VCC) are calculated:

- VCC=263 MN for drilling conditions
- VCC=246 MN for non-drilling conditions i.e. storm survival mode.

A general overview of the 2D & 3D FE models for LS4 and final 3D result of failure pattern from safety analysis is given in Fig. 13

Based on the calculation of the vertical cyclic capacity in 3D models, the following correction factors (CF) between 3D and axisymmetric models are obtained:

- for storm survival / non-drilling mode: CF=1.21
- for drilling modes: CF=1.23

The correlation factors are applied on the VHM loads for each LS. Safety calculations are performed afterwards. Results of the foundation capacity / safety envelopes for each of the LS are presented in Table 10. In addition, stiffness are calculated for each LS and results for VHM loads are also presented in Fig 13 for the most critical load scenario LS4.

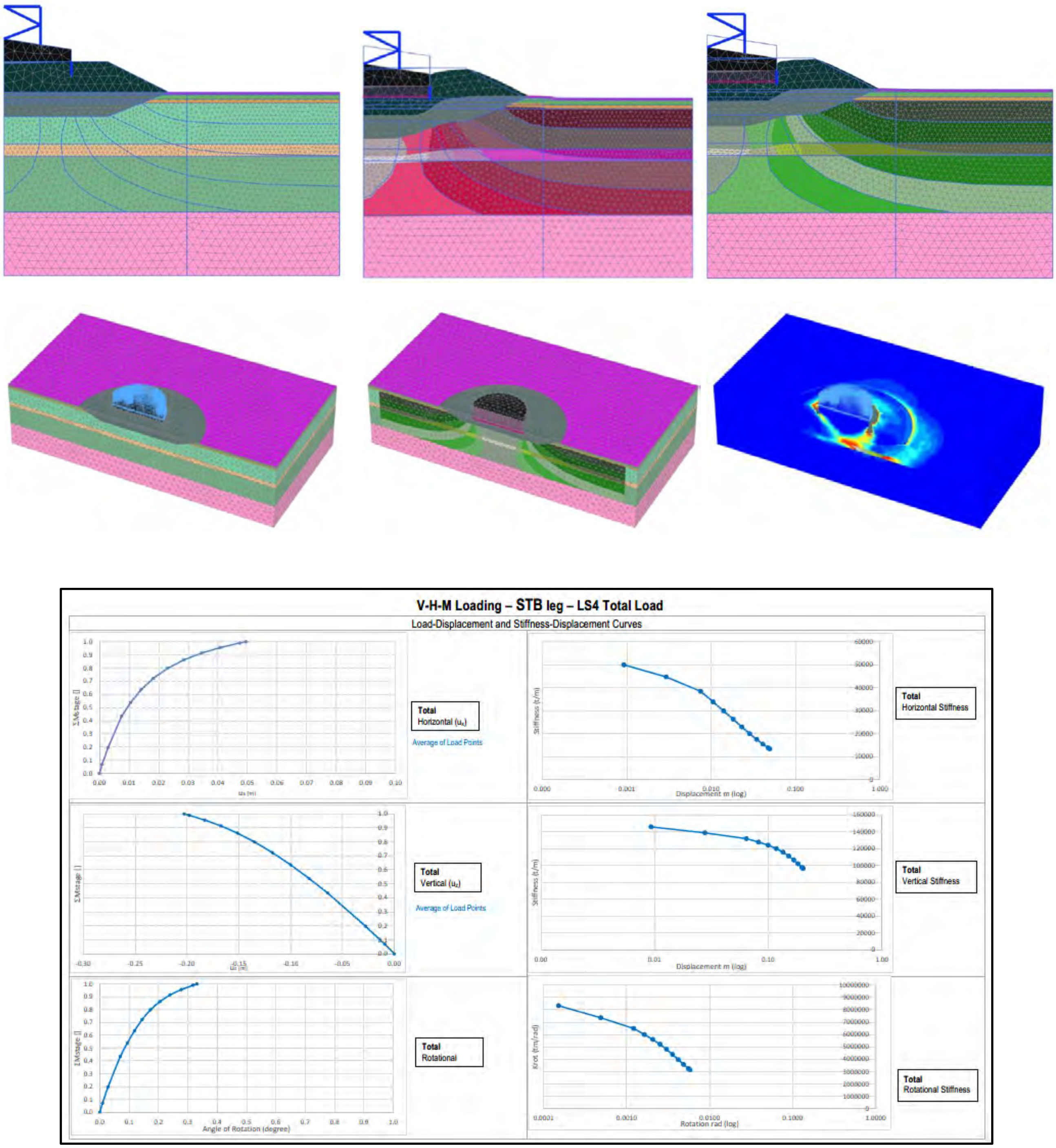


Figure 13 2D & 3D FE Models for STB Leg, (Drilling) LS4

FOUNDATION CAPACITIES & SAFETIES

LS1: For FWD (BOW) Leg, storm survival (storm heading 315° wrt North), corresponds to non-drilling conditions being the most critical. Stiffness curves for LS1 are shown in Fig. 9 while the safety factors (SFs) for the foundation checks for all legs are given in Table 7 below. As expected, FWD Leg is the most critical under the storm survival mode, having also the lowest SF.

TABLE 7 LS1 SFs for VHM Cyclic Capacities

Loads	Applied VHM Loads (Total (crest))			VHM Cyclic Cap. acc. to Safety			Load Correction Factor	SF After VHM Loads
	Vertical	Horizontal	Moment	Vertical	Horizontal	Moment		
	[t/leg]	[t/leg]	[tm/leg]	[t/leg]	[t/leg]	[tm/leg]	2D-3D	Cyclic
Forward	18773.0	978.9	22382.8	**24217**	**1263**	**28874**	1.15	**1.29**
Starboard	13995.9	1172.3	22925.6	**19454**	**1629**	**31867**	1.21	**1.39**
Port (updated)	13995.9	1172.3	22925.6	**19874**	**1665**	**32554**	1.21	**1.42**

LS2: For FWD (BOW) Leg, drilling conditions, LS2 (storm heading of 315° wrt North) corresponds to the most critical case. The SFs for the foundation checks for all the legs can be found in Table 8 below. The results show the lowest SF for FWD Leg, having also the most onerous load combination for this LS.

TABLE 8 LS2 SFs for VHM Cyclic Capacities

Loads	Applied VHM Loads (Total (crest))			VHM Cyclic Cap. acc. to Safety			Load Correction Factor	SF After VHM Loads
	Vertical	Horizontal	Moment	Vertical	Horizontal	Moment		
	[t/leg]	[t/leg]	[tm/leg]	[t/leg]	[t/leg]	[tm/leg]	2D-3D	Cyclic
Forward	15759.4	999.0	21906.2	**20330**	**1289**	**28259**	1.21	**1.29**
Starboard	15504.6	1162.1	22762.5	**21396**	**1604**	**31412**	1.23	**1.38**
Port (updated)	15504.6	1162.1	22762.5	**21551**	**1615**	**31640**	1.21	**1.39**

LS3: For PS Leg, LS3 (storm heading of 195° wrt North) is considered the most critical. Stiffness curves for LS3 are shown in Fig. 11 while SFs for foundation checks for all the legs are given in Table 9 showing also the lowest SF at this leg. Using the stiffness derived from LS3, applying iterative calculations, two additional sets of footing loads, corresponding to the same storm heading 195° wrt North (called LS3 Rev.1) and a 255° wrt North, which governs the chord utilisation (called LS3 Rev.1 Additional, due to different heading) are derived from the SSA. Hence, additional analyses are carried out for LS3 Rev.1, and SF=1.25 is derived for PS Leg satisfying the bearing capacity checks. The SFs for this LS for all legs can be found in Table 9 below.

TABLE 9 LS3 Rev.1 SFs for VHM Cyclic Capacities

Loads	Applied VHM Loads (Total (crest))			VHM Cyclic Cap. acc. to Safety			Load Correction Factor	SF After VHM Loads
	Vertical	Horizontal	Moment	Vertical	Horizontal	Moment		
	[t/leg]	[t/leg]	[tm/leg]	[t/leg]	[t/leg]	[tm/leg]	2D-3D	Cyclic
Forward	9334,4	862,4	18733,9	**15775**	**1457**	**31660**	1,21	**1,69**
Starboard	13920,5	854,2	21932,7	**21020**	**1290**	**33118**	1,23	**1,51**
Port (updated)	19932,7	622,8	16754,3	**24916**	**779**	**20943**	1,21	**1,25**

LS4: For STB Leg, LS4 (storm heading of 75° wrt North) results in the most onerous load combination in drilling conditions. Stiffness curves for this load are given in Fig.13 while the SFs for all the legs can be found in Table 10 below. STB Leg has the lowest SF for this LS. Limited SF=1.25 is however interpreted also due to limited number of CPTs at this leg.

TABLE 10 LS4 SF for VHM Cyclic Capacities

Loads	Applied VHM Loads (Total (crest))			VHM Cyclic Cap. acc. to Safety			Load Correction Factor	SF After VHM Loads
	Vertical	Horizontal	Moment	Vertical	Horizontal	Moment		
	[t/leg]	[t/leg]	[tm/leg]	[t/leg]	[t/leg]	[tm/leg]	2D-3D	Cyclic
Forward	11060.1	876.7	18552.5	**17364**	**1376**	**29127**	1.21	**1.57**
Starboard	19541.3	662.6	18144.8	**24231**	**822**	**22499**	1.23	**1.25**
Port	15586.1	856.3	18817.5	**22351**	**1228**	**26984**	1,21	**1,43**

The design of SIs at the Jack-up drilling location, follows DNVGL-RP-C212 and is expected to insure sufficient foundation fixity during rig operation at the site. The SIs, together with rig's skirted spudcans, will act as foundations to support cyclic loading for all year metocean conditions ensuring structure integrity & safety. The jack-up rig is successfully installed over the seabed interventions measuring penetrations to full base contact as predicted and designed, and is currently operating at the site.

ACKNOWLEDGEMENT
To AkerBP, Stavanger team and Sub-Contractors (DNVGL, NGI, ND, MD, DO, Boskalis etc.).
To Geo, Copenhagen team: F. Gobuzi, M.R. Jensen, N. Katic, R. Keshvary, S.S. Sundararajan, T.C. Larsen, etc.

REFERENCES
/1.1/ DNV-GL, Offshore soil mechanics and geotechnical engineering, Recommended Practice, DNVGL-RP-C212, Edition, August 2017.
/1.2/ Eurocode 7: Geotechnical Design – Part 1; General Rules. EN 1997-1 2004
/1.3/ EN ISO 19905-1 Petroleum and Natural Gas Industries – Site Specific Assessment of Mobile Off shore Units – Part 1 Jack-ups (ISO 19905-1:2016)
/1.4/ SNAME T&R Bulletin 5-5A, Site Specific Assessment of Mobile Jack-Up Units, Society of Naval Architects and MarinRefe Engineers. 2008
/1.5/ Plaxis 2D & 3D, 2016.1.21797.5460 Version, Finite Element Code for Soil and Rock Analysis. Delft University of Technology and Plaxis bv. The Netherlands, 2008
/1.6/ Andersen, K.H., Cyclic parameters for offshore foundation design. The 3rd ISSMGE McClelland Lecture, 2015, ISFOG, Olso, Norway.
/1.7/ Hansen, J.B., A revised and extended formula for bearing capacity. Bulletin No. 28. The Danish Geotechnical Institute. 1970
/1.8/ Jostad, H.P., Torgersrud, Ø., Engin, H.K., Hofstede, H. (2015), 'A FE procedure for calculation of fixity of jack-up foundations with skirts using cyclic strain contour diagrams', International Conference: The Jack-Up Platform 2015
/1.9/ Kellezi, L., Stadsgaard, H. (2019), 'Design of 'High & Narrow' Gravel Banks Applicable to Skirted Spudcans and Jack-up Installation close to Jacket Structure', 17th Int. Conf.: The Jack-Up Platform Design, Construction & Operation, 12-13 Sept. 2019, City Univ. London, Proceed. Paper 6.
/1.10/ Kellezi, L., Xu, L. Molina, C. (2017) 'Site Investigation & Seabed Remediation to Avoid Sequential Jack-up Punch Through Failure' Offshore Site Investigation Geotechnics 8th Int. Conference Proceeding pp. 1011-1018(8), Publisher: Society for Underwater Technology
/1.11/ Kellezi, L., Koreta, O., Sundararajan S. (2017), 'Skirted Spudcan-Soil Interaction under Combined Loading-Gap Conditions After Preloading' 16th International Conference 'The Jack-up Platform Design, Construction & Operation, September, London, UK.
/1.12/ Kellezi, L, Stadsgaard, H.(2012), 'Design of Gravel Banks – a Way to Avoid Jack-Up Spudcan Punch Through Type of Failure', OTC 2012, Houston, USA, April-May 2012, Paper no. OTC 23184.
/1.13/ Kellezi, L., & Kudsk, G. (2009), 'Jack-up Foundation, FE Modelling of Punch Through for Sand over Clay' 12th International Conf. on Jack-up Platform. Sept. London UK, Proc. page 1-12.
/1.14/ Kellezi, L., Kudsk, G., Hofstede, H. (2008), 'Skirted Footings Capacity for Combined Loads and Layered Soil Conditions. 2nd BGA International Conf. on Foundations, ICOF 2008, June, Dundee, Scotland, Proc. Volume 1 page 923-935