Seabed instability and 3D FE jack-up soil-structure interaction analysis

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1. Introduction

Seabed instability is an important aspect in the design of different offshore structures. Particularly for a jack-up drilling rig, which is supported by three independents legs, this becomes a crucial issue.

A geotechnical engineering analysis for the installation (preloading) and storm loading of the world's largest jack-up rig, temporarily installed next to a quay of a Norwegian yard, to be upgraded for production work on the North Sea, is given in this article.

From a preliminary site survey the seabed in the considered area was expected to be rock outcrop, undulating across the site. Considering that rig's footings have outer / inner skirts, which could not penetrate the rocky seabed, modification in the seabed conditions, creating flat areas at the footing's locations, through construction of shallow gravel banks, was initially proposed.

A detailed geotechnical investigation was carried out to verify the soil conditions. From the investigation a sediment layer of varying thickness overlying the undulating bedrock was identified.

Several possible rig locations were investigated and discussed to a final one, which was thoroughly assessed. The sediment layer consisted of a very soft to firm silt (mix) layer overlain by a thin layer of seabed sand. Therefore, preliminary engineering analyses, conventional and numerical, with originally low or increased elevations of the gravel banks, indicated instability of the free skirted spudcans under preloading conditions.

The two-dimensional (2D) and three-dimensional (3D) finite element (FE) analyses of the free skirted spudcans, which are usually applied for non-uniform soil conditions, were currently considered conservative. For a more realistic evaluation, jack-up structure - skirted spudcan - gravel bank - soil interaction effects were included in the analyses.

The full 3D structure-foundation model was applied for varying heights of the gravel banks. showing non-uniform skirted spudcan penetrations, rotations and sliding. The FE results from the final location and final heights of the gravel banks, showing that the structure forces are within the expected limits, are presented in the following.

2. Structure - Foundation System

The current jack-up drilling rig, the world's largest, is type independent leg cantilever. It operates in water depths up to 150 m and it has leg lengths of about 205 m.

2.1 Structure elements and stiffness

The considered jack-up rig is a complicated structure to be modelled in details. Therefore, in the 3D FE model calculations only the main structure elements were considered taking into account the interaction with the foundations.

Only the three legs and the hull were included in the FE model. The legs were simplified to 3D beam elements, and the hull to plate or floor elements with the equivalent thickness / area. The rig designer provided the geometry data for the legs and the hull.

2.2 Footing geometry

The considered jack-up footings have a diameter D = 22 m and are fitted with outer and internal skirts, which divide the spudcan into 6 compartments. Figure 1 show a photo view of the spudcan.

The vertical geometry of the spudcan structure is mainly given by: Distance from spudcan base to tip of outer skirts 2.3 m; Distance from spudcan base to tip of internal skirts 1.1 m;

Figure 1: Skirted spudcan view

The spudcan itself is almost a flat rigid plate. The transverse stiffnesses of the skirts are derived from the structural FE model of the spudcan. These thicknesses are applied in the 2D and 3D FE analyses employing beam and wall structural elements, respectively.

2.3 Soil conditions

To identify the seabed / soil conditions at the considered locations a new site survey, seismic, (sparker and pinger) and bathymetry was carried out.

From the survey, generally sediments of varying thickness overlaying hard ground / bedrock were found. The largest sediment thicknesses were seen at the largest water depth.

Gravity vibrocore samples taken from seabed could not reach the bedrock and showed mostly sediments of clayey, gravely sand. At the shallow water depths the bedrock outcrops the seabed.

After the interpretation of the seismic survey (sparker) geotechnical investigation including 5 piezo-cone penetration tests (PCPTs) and one vibrocore for each spudcan location were carried out. Good definition of the seabed level and the bedrock was found. However, discrepancies were recognized at some PCPT locations. The inconsistency was explained by the fact that the PCPTs were not carried out on the seismic lines.

Considering the limitations of the sparker survey, a pinger survey was carried out. With a less penetrating, but a smaller opening angle seismic source, the pinger survey was applied to better identify the slope of the bedrock and supplement the previous investigations in the area. Based on the pinger data combined with the existing soil information, a re-interpreted model of the sediment and bedrock surface was produced.

As a result, the original proposed locations were reduced to a final one. At each leg position four vertical cross sections showing the seabed and top bedrock profiles from centre of the spudcans out to a distance of 50 m, are presented in Figure 2.

To identify the soil conditions and the soil parameters applicable to the design of the gravel banks at the final location a new geotechnical investigation consisting of 5 boreholes, about 70 PCPTs and laboratory tests were carried out.
On the basis of all the geotechnical data it was evaluated that the soil conditions consist of overall quaternary marine sediments, mainly deposits consisting of a seabed layer of sand overlying clayey, sandy silt with variable thickness (0 - 9) m overlying crystalline bedrock.

2.4 Water depth

The water depth or the seabed elevations at the centre of the three spudcan locations are as seen from Figure 2, approximately –23 m at spudcan S1, -19 m at S2 and –26.5 m at S3.

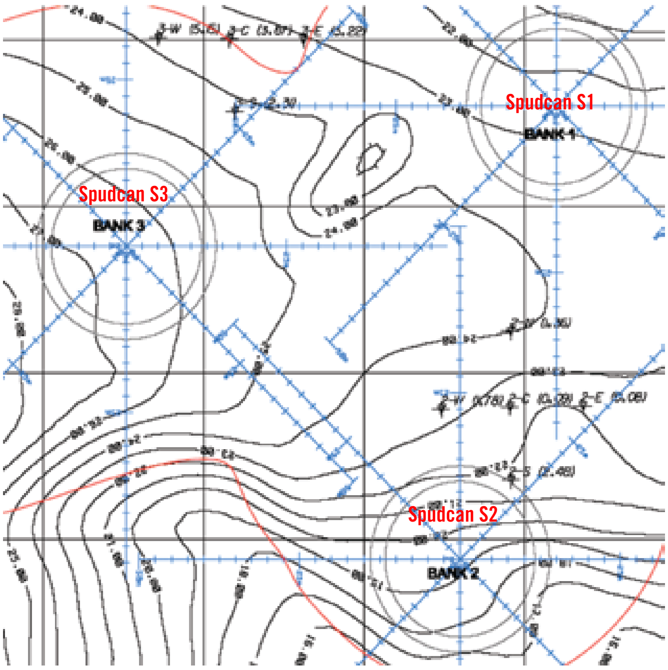

Figure 2: Seabed bedrock profile, final rig location

2.5 Design soil profiles and parameters

On the basis of the seismic surveys, PCPTs / boreholes and laboratory test results (classification and triaxial, unconsolidated undrained (UU) and consolidated isotropic drained (CID)) performed for the final location, the soil profiles and soil parameters applicable to the engineering assessment are derived. The soil parameters for the bedrock are evaluated based on the engineering experience.
There seems to be a good correlation with the PCPT data for the depths where samples were taken and laboratory tests performed. From the UU triaxial tests undrained shear strengths of value minimum c_u = 33 kPa are measured for the extracted samples. However, at the depths where lower cone strength as shown in Figure 3, is recorded from the PCPT, no soil sample could be extracted and no correlation is available. Under these circumstances the correlation $N = q_{net} / c_u = (15 – 20)$ is found applicable.
When applying such a correlation on the PCPT data undrained shear strength for the silt c_u = (15 – 30) kPa is assessed. Based on the test results and the engineering judgement initially c_u = 25 kPa for the silt layer and a friction angle $\varphi = 35°$ for the seabed sand layer were assessed as lower bound values. For the bedrock an undrained shear strength c_u = (1000 – 1500) kPa was assigned to represent the strong subsurface. A summary of the soil parameters applied in the analyses is given in Table 3.
The gravel bank material is modelled applying a unit weight $\gamma' = 11$ kN/m^3 and a friction angle $\varphi = 40°$. The deformation parameter E = 100000 kPa.

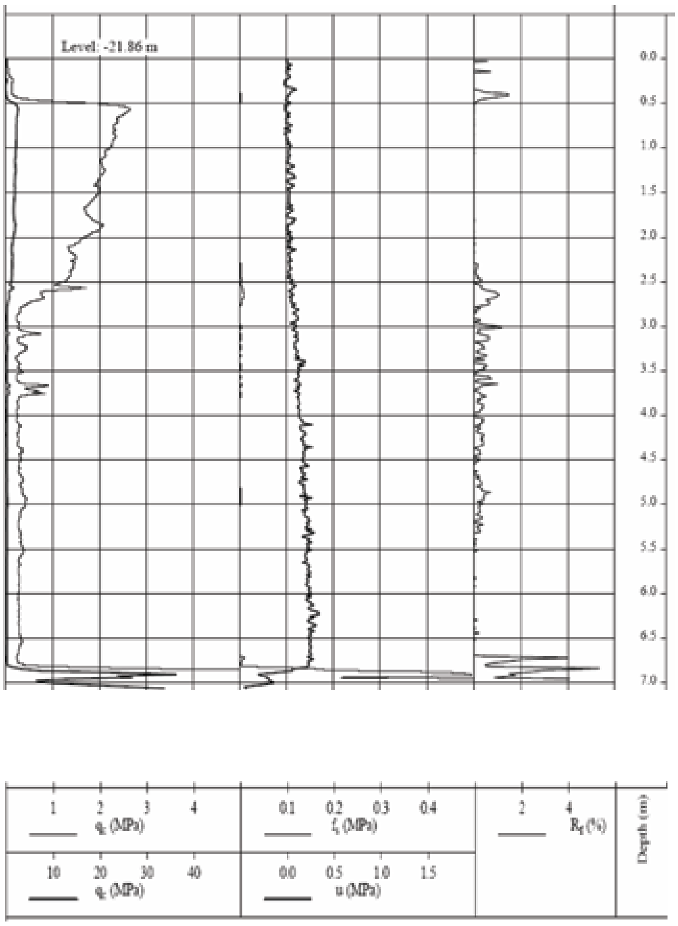

Figure 3: PCPT profile at S1 location

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Soil Type	h (m)	γ' (kN/m³)	φ (°)	cu (kN/m²)	E (kN/m²)
Sand, loose to medium dense	Varying	10.0	35	-	35000
Silt, very soft to firm	Varying	9.0	-	15/25/30	100*cu
Bedrock	Varying	12.0	-	1000/1500	200*cu

Table 1: Soil profile applied in the FE analyses

3. Structure - Foundation Analyses

Different analyses consisting of conventional and 2D / 3D FE modelling are carried out.

3.1 Preliminary 2D and 3D FE modelling, low gravel banks

The 2D FE free skirted spudcan - low gravel bank - soil interaction analyses with Plaxis 2D Version 8 (2002) showed instability of the S1 and S3. However, the 2D analyses were considered very conservative due to 3D soil conditions.

Under these circumstances 3D FE modelling of the free skirted spudcan - gravel bank - soil interaction was performed, The model was built with Plaxis 3DFoundation (2006) assigning boreholes at the location where soil profile changes. An implicit interpolation between the boreholes is carried out during the calculation. By this method the soil conditions at the spudcan area are modelled to the extent the seismic survey and the geotechnical investigation allow.

Mohr Coulomb constitutive soil model for the soil layers in the drained (seabed sand and gravel bank) and undrained (bedrock and clay / silt / mix) conditions are applied. The preliminary analyses applying c_u = 25 kPa for the silt layer showed large rotations and horizontal movement for the free S1.

To take into account the structure foundation interaction it was discussed to apply some stabilizing loads on the spudcan while preloading. After many calculation attempts it was found difficult to assess the limited reaction forces needed to stabilize the spudcan and the procedure was cancelled. The issue of skirted spudcan – structure – skirted spudcan - soil interaction was raised at this time.

The first full 3D model consisted of low gravel banks at elevations –20.0 m, (about 3 m height), -13.5 m, (about 5.5 m height), -24.0 m, (about 2.5 m height), for S1, S2 and S3, respectively. Large penetrations and horizontal movements, particularly for S3 were calculated. The reaction forces in the structure were far beyond the limits. Under these circumstances the effect of the higher gravel banks at S1 and S3 locations were investigated.

3.2 Conventional skirted spudcan differential penetration

Based on SNAME (2002) and Hansen (1970) conventional skirted spudcan penetration analyses were carried out at each spudcan location to get an idea on the effect of the height of the gravel banks on the spudcan differential penetration. These were also compared with some FE axisymmetric analyses of the spudcan penetration.

Such analyses are previously carried out by Kellezi & Stromann (2003), Kellezi et al. (2005a,b),

In the analyses c_u = 25 kPa for the silt was applied. The results for location S1 and gravel bank at elevation –19 m are given for illustration in Figure 4. Two extreme soil profiles within the spudcan area are chosen, which are expected to give max and min penetrations. The differences in penetrations give the expected differential penetration of the free spudcan. The elevation of the gravel bank is moved from -21 m to -19 m to -14 m.. No punch through risk is expected for any of the scenarios.

For S2 the height of the gravel bank is determined from the length of the spudcan skirt/chord, plus some tolerance. The top of the bank will be at -13.5 m and small differential penetrations are expected.

For S3 two extreme soil profiles are chosen as well expected to give max and min penetrations. The elevation of the gravel bank is moved from -25 m to -23 m to -21 m to -18 m to -16 m.. No punch through risk is expected for any of the scenarios.

3.3 Preliminary 3D FE modelling, high gravel banks

To make the location applicable for the rig installation based on the conventional and FE axisymmetric results, higher gravel banks were proposed. The soil mechanic principle of load spreading is used. Higher banks will increase the bearing capacity of the silt layer as a result of increasing fictive bearing area.

Except for the preloading phase this model was also calculated for the storm load, wind speed 33 m/s. The storm load may come from any direction so different analyses are needed to define the critical one. The 3D model calculation procedure consists of 3 load stages, which are:

Preloading to max vertical load V =145 MN; Unload to vertical V = 112 MN, V = 100 MN, V = 115 MN for S1, S2, S3, respectively; Apply storm loads, horizontal H = 6.4 MN, moment M = 345.6 MNm at the most critical plane;

The horizontal force is applied at the hull plate pointing towards the critical leg. The moment is implemented as a set of two vertical loads, applied downward at the critical leg-hull connection and upwards at a point in the hull between the other two legs as shown in Figure 6. Except the 3 load phases, an initial phase is calculated consisting of the construction of the gravel banks.

The limited combined loads at the structure, one single leg, are calculated as:

Horizontal shear force Q = 18 MN; Vertical force (at hull) V = 145 MN; Bending moment at hull M = 325 MNm;

Taking into account the limits for the structure reaction forces and the result from the 3D FE structure – foundation models with increased height of the gravel banks at S1 and S3, a reassessment was found necessary.

The soil strength for the silt layer c_u = 25 kPa, as mentioned previously, was evaluated based on the engineering judgement. This is however, not a lower bound assessment based on the PCPT data and usual North Sea (q_{net} - c_u) correlation.

After reviewing the available soil data, to increase to some level safety concerning the soil parameters, DNV (1992) it was decided to reduce the shear strength for the silt layer from c_u = 25 kPa to c_u = 15 kPa. This strength is considered a lower bound design value, when taking into account the consolidation during construction of the banks and two weeks rig location with lightweight.

Investigating different 3D FE models with slightly different heights of the gravel banks, which could indicate less spudcan rotation / sliding, a final model, was constructed and given in detail in the next section.

3.4 Final 3D FE modelling, high gravel banks

The model scenario with gravel bank elevations at –14.5 m, (height about 8.5 m), -13.5 m, (height about 5.5 m), -15.8 m, (height about 10.7 m) for S1, S2 and S3 locations, respectively was chosen as final as the reaction forces and the amount of sliding were within the structure limits. Despite, this is the largest model with respect to mesh size, which could be run from the workstation.

The 2D build-up model geometry is given in Figure 5. The skirted spudcans are simplified by octahedrons. The spudcan is flat and in full contact with the gravel bank soil from the start of the preloading. The 3 chords and the inner skirts are not included. The tip of the outer skirts is from the start of the analyses at elevation calculated from bank elevation minus 2.3 m (the skirt length).

The jack-up structure is modelled in a simple way using vertical 3D beam elements for the 3 legs and plate / floor elements for the hull, as shown in the Figure 6. The leg elements are based on the Mindlin's beam theory. In addition, the elements can change length due to applied axial force. The leg beams and the spudcan plates at the connection points can simulate the 6 degrees of freedoms.

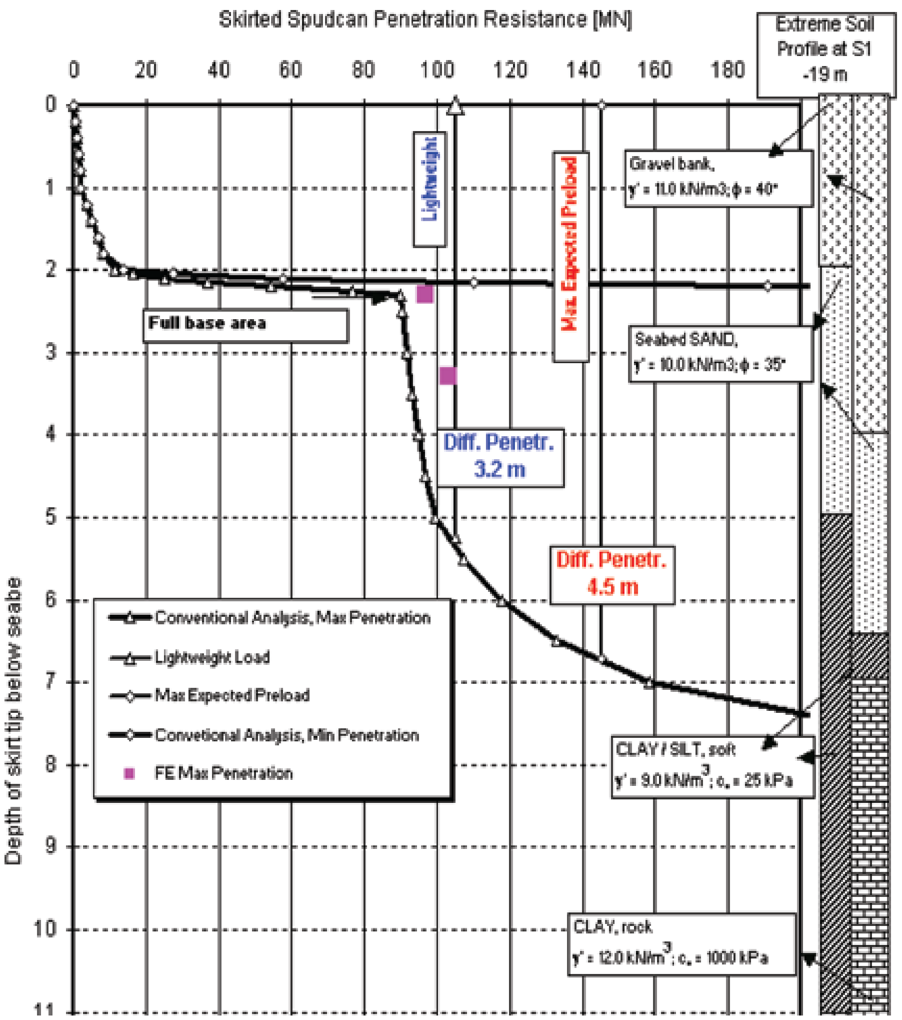

Figure 4: Conventional skirted spudcan differential penetration analysis, S1 extreme soil conditions, and gravel bank at –19 m

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The soil conditions, (soil profiles derived from the seismic, PCPT / borehole data at different cross sections), are modelled by implementing boreholes, as seen from the horizontal planes in Figure 5. Some of the soil profiles / sections with final gravel banks designed based on the 3D FE structure - foundation model are given in Figure 7, 8, 9.

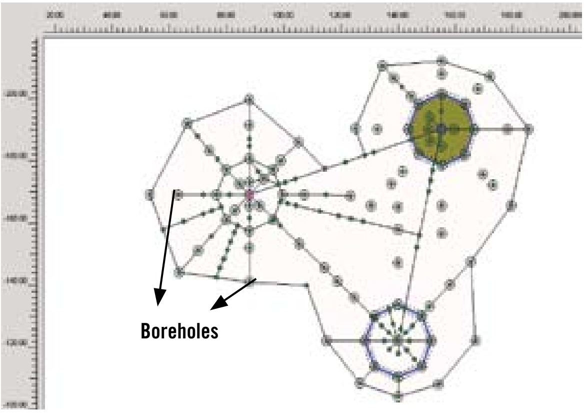

Figure 5: 3D FE structure-foundation model, 2D build-up, horizontal plane at S1 level, -14.5 m

The gravel bank sand material was specified to correspond to the soil strength applied in the analyses. The construction of the banks was performed following a procedure, which gives the possibility for some consolidation or drainage for the silt layer to occur. The total volume of the sand material used was about 60000 m^3.

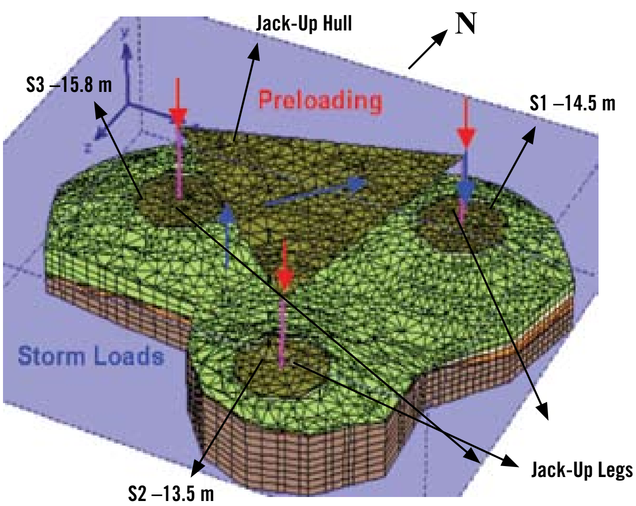

Figure 6: 3D FE structure-foundation model, final gravel banks

The results for the initial phase, including the construction of the gravel banks, are given in Figure 10. Vertical non-uniform settlements of about (20 – 40) cm are expected taken into account in the calculation of the total gravel volume.

The results for the preloading phase as total structure displacements are given in Figure 11. At the end of this phase the maximum calculated reaction forces are M = 211.04 MNm, shear force Q_{max} = 15.13 MN, differential penetration at S1, about 20 cm, sliding of S1, about 12 cm. For these values the structure integrity is found satisfactory.

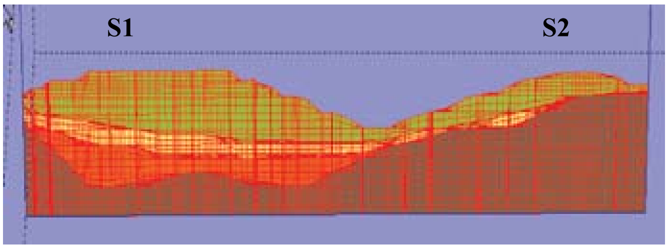

Figure 7: Cross section profile North – South at S1 across the 3D FE model (not to scale)

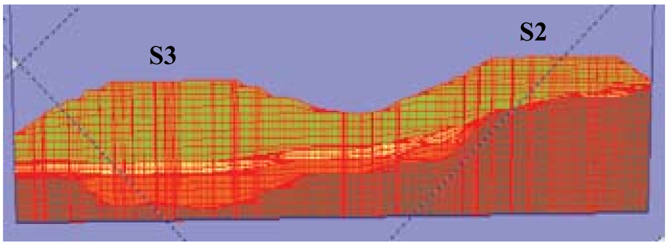

Figure 8: Cross section profile North West – South East at S2 across the 3D FE model (not to scale)

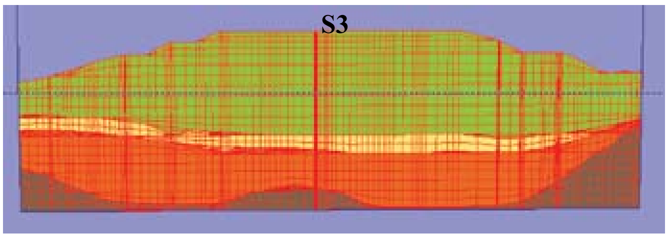

Figure 9: Cross section profile North - South at S3 across the 3D FE model (not to scale)

The results for the unloading phase show slight changes in the deformations and structure reaction forces. The results from the storm analyses show also slight changes in the deformations and structure reaction forces.

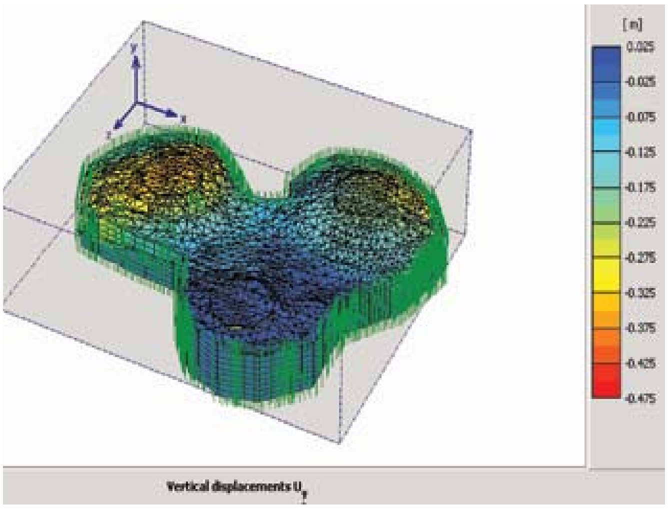

Figure 10: Initial phase, construction of the final gravel banks

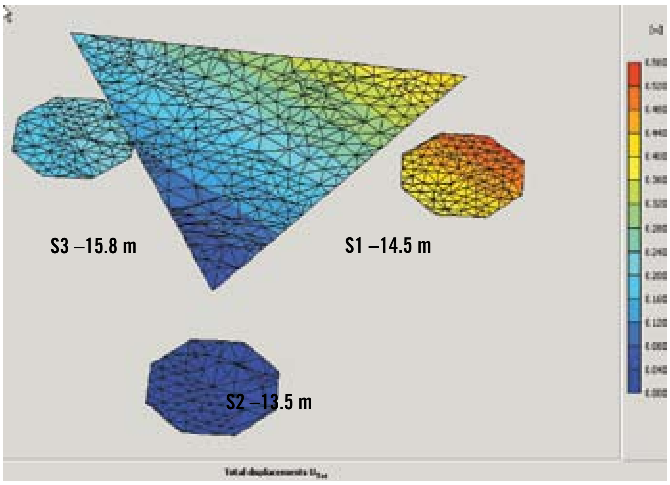

Figure 11: Preloading phase, structure total displacements (only plate elements shown)

Conclusions

3D FE structure - foundation interaction analyses are carried out for the installation of a jack-up rig, offshore, Norway, where seabed instability was a concern.

Gravel banks were designed at the skirted spudcan locations with different heights, ensuring that the structure reaction forces developed due to footings rotation / sliding, do not exceed the calculated limits.

The jack-up rig was successfully installed at the location and spudcan penetrations / displacements similar to the predicted values were recorded.

Acknowledgement

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