Bundle Towhead Foundation Design on Rock Berm Installed on Soft Clay

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ABSTRACT
As part of the BG Knarr project, a flowline bundle was installed during September 2014 including a template towhead (leading) connected by a 4.5km rigid bundle carrier pipeline to the FPSO towhead (trailing). The field is located in the norther part of the North Sea in a water depth of approximately 410 m. The two bundle towheads were landed on pre-installed rock berms. The rock berms were installed May 2013 and have an approximate size of 100 m x 50 m and a height of 2.5 m above seabed. The leading bundle towhead submerged weight is 270 t with a length of 33.5 m and width of 16 m while the tailing towhead submerged weight is 170 t with a length of 32.5 m and width of 6 m.

During production of the Knarr field, the bundle will expand longitudinally causing the towheads to displace up to 1 m on each rock berm. The geotechnical design comprised a series of bearing capacity calculations, using PLAXIS 2-D and PLAXIS 3-D. The main objective has been to validate that the towheads slide as near to the horizontal as possible on the rock berms, while maintaining an adequate level of safety against bearing failure of the rock berm and under laying very soft clay. It is believed that these towheads are the world's largest subsea sliding foundations.

Keywords: Bundle towhead foundation, rock berm on soft clay.

1 INTRODUCTION

A 4.5 km flowline bundle containing two production, one water injection and one service line and controls was installed during 2014, between the production template and the Knarr floating production storage and offloading (FPSO) vessel. The bundle has a towhead at each end which is used during installation to tow the bundle to the field and during production to connect flowlines and controls. The template towhead, during launch is shown in Figure 1-1. Reference is made to Goodlad (2013) for further information on bundle technology.

Figure 1-1: Launch of the template towhead. The bundle is seen on the shore side connected to the towhead.

During start-up of the system and also during production, temperature and pressure differences will lead to longitudinal expansion loads in the bundle, which exceeds the horizontal sliding capacity of the towhead foundations. The towheads are therefore required to slide on the seabed several times during the 20 years lifetime of the system.

In order to ensure the foundations can slide and prevent the towheads from rotating horizontal at the Knarr location, a rock berm was installed onto the very soft clay seabed 16 months prior to bundle installation. Rock berms were surveyed in November 2013 in order to identify settlements and document the consolidation and verify the strength increases in the very soft clay. Reference is made to Kahlström et al. (2015) for further details on the settlement survey.

The towhead foundation design methodology is a performance-based design where the bearing capacity is proven with an appropriate factor of safety while at the same time the towheads are allowed to slide on the rock berm. This paper outlines the experience with towhead foundation design on very soft clay.

2 BACKGROUND

2.1 Knarr Field Development

The Knarr oil and gas field is located approximately 112 km west of Florø, Norway, in the Norwegian Sector of the Northern North Sea at an average water depth of 410 m.

A subsea production template is tied back to the Knarr FPSO vessel using a pipeline bundle and flexible risers. Production oil and condensate will be exported by shuttle tanker from the Knarr FPSO whereas the gas is exported by pipeline to the St Fergus gas terminal in the UK, via the Far North Liquids and Associated Gas System (FLAGS).

The bundle carrier pipeline has a diameter of 1.01 m and is connected to a towhead structure at each end; the Template Towhead (Figure 1-1) and the FPSO towhead (Figure 2-1).

Figure 2-1: Picture of Knarr FPSO towhead during launch. The bundle is seen entering the water.

2.2 Rock Berm Structures

Due to the soft clay seabed, the towhead structures were installed on rock berm foundations. The rock berm foundations were constructed with a ramp of rock with an inclination of 1° to ensure a straight connection between the bundle and the towhead. The FPSO towhead rock berm is shown in Figure 2-2. Notably, the rock berm has a wing on each side, which is designed to support the flexible risers connected to the towhead. Reference is made to Kahlström et al. (2015) for more information on the rock berms and installation method of rock berms.

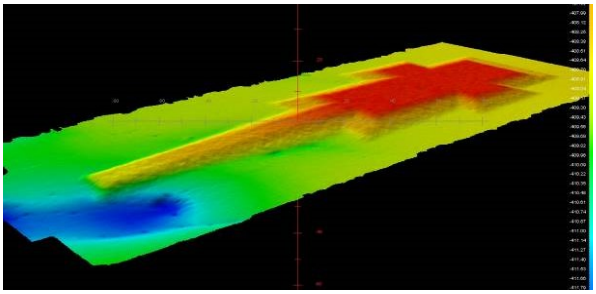

Figure 2-2: FPSO towhead rock berm with a ramp and two wings to support risers exiting the towhead.

The Template Towhead and template including spool which is covered by Glass Reinforced Plastic (GRP) covers are illustrated in Figure 2-3.

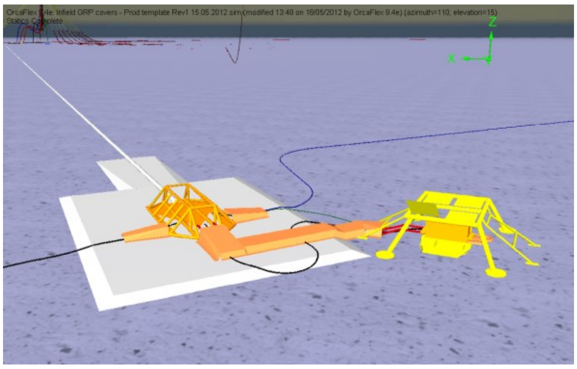

Figure 2-3: Illustration of template towhead on Rock berm in close proximity to template.

2.3 Towheads

The towheads are used to tow the bundle to the field suspended between two tugs using the controlled depth tow method. The towheads are made neutrally buoyant with use of buoyancy tanks during the tow. A survey/patrol vessel accompanies the tow, see Figure 2-4 for typical tow arrangement. The towheads also have a secondary purpose during production as the method of connection of flowlines and controls.

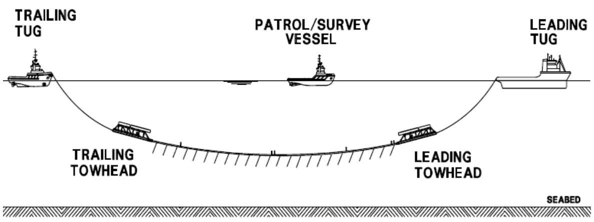

Figure 2-4: Typical bundle tow fleet arrangement.

The dimensions of the towheads and the steel mudmats are given in Table 2-1. For the template towhead, the mudmat consists of two parts with a 5.5 m gap in between. The two parts are connected at both sides of the towhead. Hence, the two mudmat parts will move as one. It should be noted that the length of the mudmats are smaller than the length of the towheads.

Table 2-1. Details of towheads and mudmats.

Structure	Submerged Weight [kN]	Length [m]	Mudmat [m x m]
Template towhead	2700	33.5	2 of 27 x 5.25, 5.5 m apart
FPSO towhead	1700	32.5	30 x 6

The Knarr towheads are designed with a free drainage cooling spools. To ensure self-drainage of the cooling spools during field life, the maximum allowable rotation in any direction of the towhead is 1.5°. This enforces strict requirements on the differential settlements.

2.4 Soil Conditions

Geotechnical core sampling was performed at the site to a depth of 30 m below the seafloor, revealing a 5-layered soil profile as described in Table 2-2. Layer I, in between 0.0 – 13.5 m, belongs to the Kleppe Senior Formation and consists of very soft to soft slightly sandy Glaciomarine clay, deposited after the last glacial peak. Layer I has a high plasticity and contain traces of organic material. Geophysical surveying revealed a flat boundary in between soil layer I and layer II. Due to the homogeneity of the soil, differential settlements are not expected.

Table 2-2. Description of soil profile.

Soil Layer	Average depth below seafloor, m	Soil description
I	0.0 - 13.5	Very soft to soft slightly sandy clay
II	13.5 - 14.8	Soft to firm slightly sandy clay
IIIa	14.8 - 17.6	Firm to stiff slightly sandy clay
IIIb	17.6 - 22.1	Loose to medium dense clayey sand
IIIc	22.1 – 30.0	Firm to stiff slightly sandy clay

Table 2-3 provides soil properties for each soil layer. In total, 6 Cone Penetration Test samples, 2 piston samples and 11 Wireline Push samples from a borehole drilled at the location of the trailing towhead rockberm were analyzed to obtain the soil data.

Table 2-3: Soil properties for each layer in the soil profile.

Soil Layer	Undrained shear strength, S_u^C, kPa	Water content, w, %	Submerged unit weight, γ', kN/m^3
I	5 - 24	61 - 87	6.0
II	24 - 53	23	10.3
IIIa	75	21	10.5
IIIb	-	17	10.5
IIIc	67 - 87	23	10.3

The soil conditions are further described in Kahlström et al. (2015).

3 LOADS

3.1 Trawl (fishing) loads

The FPSO towhead is located within a 'no fishing zone'. The template towhead is designed as an overtrawlable structure. The trawl load can either consist of trawl net friction or of trawl board over pull, given in NORSOK U-001 (2002):

- The trawl net friction consists of 2x 200 kN applied at 0-20° relative to horizontal acting in opposite corners of the structure and acting in the same direction.
- The trawl board over pull load of 300 kN applied horizontally at 0-20° relative to horizontal at the most critical point.

3.2 Hydrodynamic current loads

The towheads are exposed to hydrodynamic loading from currents. The characteristic hydrodynamic horizontal load is 43 kN for the template towhead and 20 kN for the FPSO towhead. Given that the trawl load on the template towhead exceeds the hydrodynamic load, only the FPSO towhead is designed for hydrodynamic load.

3.3 Expansion loads

During the lifetime of the BG Knarr field, a number of bundle start-up and shut-down cycles are expected to occur, in which the temperature and pressure in the bundle changes. The temperature and pressure change will lead to expansion and contraction of the bundle.

For a gravity based foundation of the towheads, it is not possible to statically resist the bundle expansion load due to the ratio between the horizontal load and the submerged weight of the towheads. Therefore, the towheads are designed to slide on top of the rock berms. The template and the FPSO towheads are expected to experience a maximum longitudinal horizontal displacement of 1 m. The force applied to the towheads will correspond to the sliding capacity of the interface between the towheads and the rock berms. The sliding capacity can theoretically be determined as the submerged weight of the towheads multiplied with $\tan(\varphi_{rock,inter})$, in which $\varphi_{rock,inter}$ denotes the rock / steel interface friction angle. The bundle expansion load acts over the lower 1.05 m of the towhead.

4 DESIGN

4.1 Introduction

As stated, the contraction and expansion forces induced through the bundle during flow shut-down and start-up cycles lead to loads on the towheads of such a magnitude that the sliding capacity of the underlying rock berm will be exceeded and cause the structure to shift. Alternatives to the rock berms were considered early in the design process. One alternative was a sliding mechanism in the towheads; however, this turned out to increase in deadweight of the towheads coursing installation problems. The second alternative was fixing the towheads with use of suction buckets. The suction bucket was found infeasible due to the expansion from the bundle.

The rock berm dimensions are determined such that an adequate safety is obtained against a failure mode occurring within the clay or rock berm.

The critical failure mode during flow shut-down and start-up cycles is sliding between the rock berms and the clay, which means that the upper 5 m of the soil profile is of principal importance to the design.

In the following sections, the performance-based design methodology and settlement assessment is described in further detail.

4.2 Performance-based design

The rock berm dimensions are designed such that the rock berms do not become unstable during bundle expansions and such that the towhead foundations can withstand trawl loading, hydrodynamic loading and/or self-weight of the rock berms and towheads. The dimensions of the foundation are designed based on an iterative procedure ensuring an adequate level of safety in line with DNV partial factors. The design approach is similar to the combined failure approach for sand described by Cathie et al. (2008).

It is found that the load from bundle expansion is governing for the length of the rock berms, i.e. the size of the rock berms in the direction parallel to the bundle. Furthermore, the width of the rock berms, i.e. the size of the rock berms in the direction transverse to the bundle direction, also has an influence of the stability of the rock berms during bundle expansion loading due to three-dimensional distribution of stresses. The stability of the rock berms when exposed to bundle expansion loading has been assessed by means of numerical calculations in PLAXIS 2D Version 2011-2 and PLAXIS 3D 2012. The purpose of the numerical models were to investigate the safety against failure. Hence, the Mohr-Coulomb material model was considered to be sufficiently advanced.

Analytical analyses of the rock berm stability during bundle expansion loads have been conducted in order to validate the numerical models. In the analytical modelling, the capacity of three kinematic failure modes were assessed. These included; a failure mode consisting of mudmats sliding on top of rock berm (this failure mechanism is the desired failure and hence considered as controlled sliding); a failure mode consisting of the entire rock berm sliding on top of the clay, see Figure 4-1; and a failure mechanism consisting of sliding between the rock berm and the subsoil directly beneath the towhead and a passive wedge forming in the rock berm in front of the mudmats, see Figure 4-2.

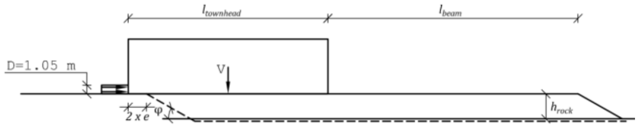

Figure 4-1: Berm failure on top of clay.

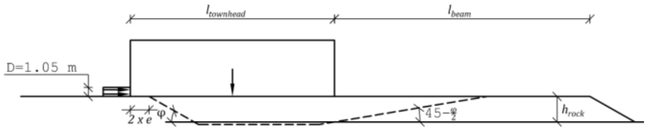

Figure 4-2: Passive failure mechanism in berm.

The undrained shear strength in the analysis was calibrated to consider the consolidation effects of the pre-installed rock berm. A conservative consolidation period of six months was chosen rather than the actual planned duration between installation of the rock berm and towhead installation of 15 months. Further, as the main direction of the slip surface is horizontal the undrained shear strength was therefore analysed as direct simple shear. The use of the direct simple shear strength accounts for strength anisotropy was in accordance with DNV (1992).

The width of the rock berms were primarily governed by trawl load, hydrodynamic load and/or self-weight of the rock berms and towheads. However, it should be noted that the bundle expansion load also affected the necessary width of the rock berms due to three-dimensional distribution of stresses. The stability of the rock berms against trawl load, hydrodynamic load and/or self-weight of the rock berms and towheads was assessed by means of PLAXIS 2D. For these design cases, three-dimensional effects were considered to be insignificant. Further, two-dimensional analyses of the rock berm stability against these design cases were considered to be conservative. In the numerical modelling, infinite failure modes were assessed. Hence, the failure modes in the numerical modelling accounted for bearing capacity failure, slope failure, and any combinations of these. An example of the two-dimensional failure mode of a towhead exposed to trawl loading acting in the direction perpendicular to the bundle axis is given in Figure 4-3.

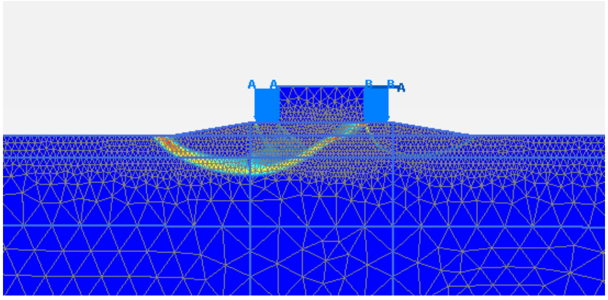

Figure 4-3. Deviatoric strains at failure. Dark blue colour indicates no or little deviatoric strains. Light blue/yellow/red colours indicate significant deviatoric strains.

4.2.1 Design methodology

The design methodology adopted was to prove that sliding was the governing failure mechanism with an acceptable margin of safety. Two methods have been considered:

1. Undrained shear strength was reduced until the failure mechanism changed from a sliding mechanism to a failure mechanism in the clay.
2. Friction in rock - structure interface friction was increased until a non-sliding failure mechanism was encountered. By increasing the rock - structure interface friction stresses were increased in the subsoil.

These two methods can be illustrated graphically in the vertical and horizontal forces (VH) stability envelope, see DNV (1992) for further details on the stability envelope. The stability envelope is a combination of the undrained stability envelope overlaid the drained stability envelope. This was also proven for the Knarr towheads with use of finite element analysis.

The undrained shear strength reduction method "shrank" the size of the undrained envelope within the VH stability envelope. This is illustrated in Figure 4-4 with the blue arrows. It was shown that the failure mechanism changed from a sliding between the towhead and the rock berm to a sliding failure in the clay with a partial factor of 2.0. The expansion load is illustrated as a yellow arrow exceeding the stability envelope.

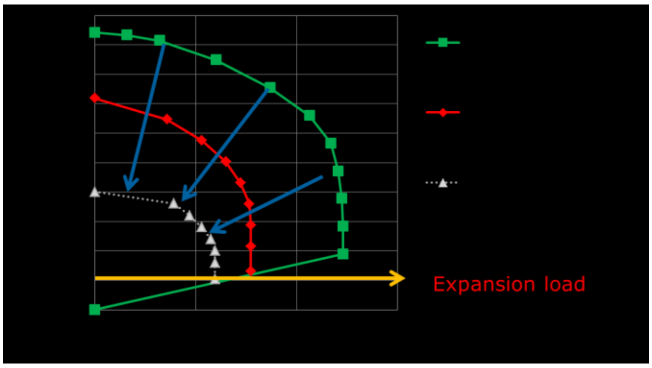

Figure 4-4: Capacity assessment, undrained shear strength reduced.

A PLAXIS 2-D illustration of the failure mechanism when it changed from sliding on the rock berm to a failure in the clay underneath is given in Figure 4-5. The failure mechanism is a result of the undrained shear strength being reduced with a partial safety factor.

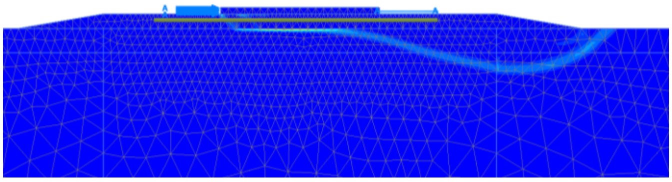

Figure 4-5: Horizontal failure mechanism developed in the clay.

It is worth noting the similarity between the failure mechanism identified by FEM in Figure 4-5 and the analytical failure mechanism defined in Figure 4-1. This supports the use of direct simple shear strength for the very soft clay model.

The increase in rock – structure friction can also be illustrated graphically in the VH stability envelope. In Figure 4-6, the change to the VH envelope caused by the increase in rock - structure friction is illustrated by the blue arrow. The expansion load is illustrated as a yellow arrow exceeding the stability envelope.

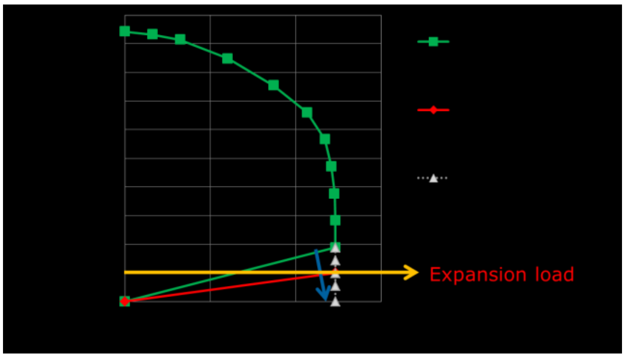

Figure 4-6: capacity assessment, friction angle increased.

From Figure 4-6, it was shown that at a certain increase in friction angle, the failure mechanism changed from sliding failure along the rock structure sliding limit to a sliding failure within the clay. The sliding limit in the clay was defined as the vertical part of the clay stability envelope, see DNV (1992).

The factor of safety was increased by increasing the towhead bearing area (the length of the rock berms) or decreasing the towhead submerged weight. As expected, both of these methods were shown to distribute less pressure onto the clay and thereby increase the factor of safety. Also note that the rock berm height is governing for the safety against failures of the type illustrated in Figure 4-2.

4.3 Settlements

The purpose of the settlement assessment was to determine the magnitude of the total and differential settlements of the Template Towhead and FPSO Towhead. The differential settlements were restricted to 1° to ensure free draining of the cooling spools. For further details on the cooling spools, see Goodlad (2013).

The settlement of the pre-installed rock berm was measured twice between rock berm installation and towhead installation, 31 months & 200 days post rock installation. The results are described in Kahlström et al. (2015).

The template towhead is connected to a series of umbilicals and flowlines, which are protected by rock dumped GRP covers. This introduced an uneven distributed load over the rock berm, see Figure 2-3 and Figure 4-7.

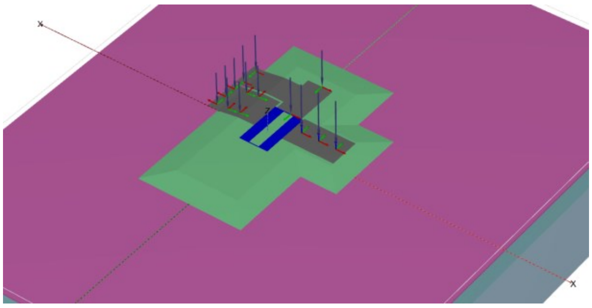

Figure 4-7: Geometry in PLAXIS 3D of the Template Towhead area.

In order to determine the differential settlements over a 20-year lifespan for the Towhead structures, the problem was considered as 3-dimensional. Analysis in PLAXIS 3D was selected to compute the differential settlements of the Towhead structures.

As described in Kahlström et al. (2015), the most accurate soil model adopted to predict settlements in PLAXIS 3D is the soft soil creep material model (SSCM). The input parameters have been validated by the on-site settlement survey.

Figure 4-7 shows the geometry of the problem as modelled in PLAXIS. In total, the geometry is 230 x 300 m^2 large and 60 m deep. The rock dumped GRP structures were modelled as a vertical surface load. The soil layers were created with reference to Table 2-2. The towheads were modelled as stiff plates with sizing equal to the mudmat area, having a distributed vertical surface load with magnitudes equal to the submerged weight of the structure.

Figure 4-8 illustrates the 20 year vertical settlement of the template towhead including creep. The rock dumped GRP covers created a differential pressure on the soil beneath. The additional pressure resulted in larger local settlements. The increase in local settlements caused differential settlements of the Template Towhead. The maximum 20-year differential rotational settlement was predicted to be 0.25° in a longitudinal direction. The transverse rotation was found to be less than 0.25°. The predicted maximum settlement was predicted to be 85 cm.

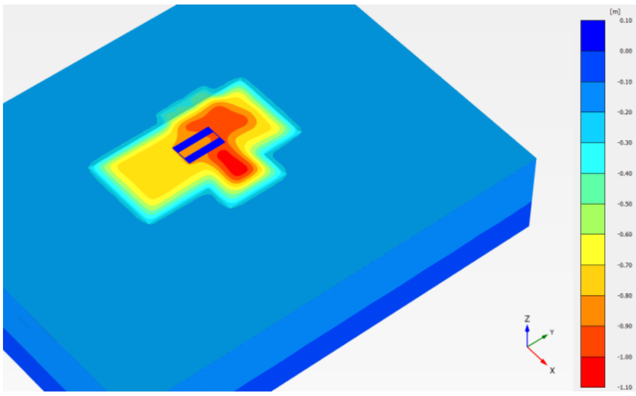

Figure 4-8: 20 year vertical settlement of Template Towhead area including creep.

In order to reduce the risk of that the rotational settlement exceeding the allowable rotation of 1°, it was decided that additional rock should be installed at the opposite end of the towhead to even out the differential pressure. Design proved that the additional rock berm would not increase the total settlements, but it would reduce the differential settlements.

5 OFFSHORE OBSERVATIONS

Following installation, a digiquartz survey was carried out to calculate the pitch and roll of the towheads approximately one week after installation. The results of the of the survey are given in Table 4-1. It is seen that the maximum recorded inclination was 0.04°.

Table 4-1: Installation inclination of the towheads.

Towhead:	**Pitch (°)**	**Roll (°)**
FPSO	-0.02	0.04
Template	0.04	-0.02

The FPSO towhead is shown in Figure 4-9 on top of the rock berm. The picture indicates a flat rock berm supporting the FPSO towhead.

Figure 4-9: Picture taken four days after installation of FPSO towhead on rock berm.

The position of the towheads have not been accurately measured after the pressurisation of the bundle. Therefore, at this time it has not been possible to quantify the actual displacement/rotation due to expansion of the bundle.

6 CONCLUSION

Rock berm foundations, situated on a subsoil consisting of very soft clay, have been designed and installed for the two towhead structures for the BG Knarr project. The subsoil consists of very soft clay. The towhead structures are connected with a bundle.

The governing load is an expansion load due to pressure and temperature changes in the bundle occurring during start-up and operation. The expansion load exceeds the weight of the towheads and therefore the rock berms have been designed to ensure that the towheads can slide on the rock berms and to prevent the towheads from rotating. The design methodology has been to prove that the sliding mechanism is the governing failure mechanism with an acceptable margin of safety.

The towhead behaviour during bundle expansions has been assessed by means of two- and three-dimensional finite element modelling using the Mohr-Coulomb model.

The settlements of the towheads have been assessed with use of three-dimensional numerical models using the soft soil creep material model. The material model was calibrated against settlement observations of the rock berms.

7 ACKNOWLEDGEMENT

The authors would like to acknowledge Subsea 7 and BG Norge for permission to publish the results presented in the paper. The authors are grateful to Dr. Daniel Karunakaran and Project Manager Oddbjørn K. Olsen for their comments and review to this paper. It is emphasized that the conclusions put forth reflect the views of the authors alone, and not necessarily those of Subsea 7.

8 REFERENCES

Cathie, D. N., Morgan, N. and Jaeck, C. (2008). Design of Sliding Foundations for Subsea Structures. ICOF2008.

DNV (1992), ”Foundations” classification note 30.4.

Goodlad, M. (2013), Bundle Technology for the Future, SPE Offshore Europe Oil and Gas Conference and Exhibition in Aberdeen.

Kahlström, M. et al. (2015). Predicted and observed settlements or face subsea rock installation - comparison between field measurements and FE simulation of North Sea clay. ISFOG, Oslo, Norway.

NORSOK U-001 (2002). Subsea production systems, revision 3 October 2002.