Appendix to report:

SBJ-33-C5-OON-22-RE-022

MARINE GEOTECHNICAL DESIGN

Appendix title:

APPENDIX A: DESIGN BRIEF

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CONCEPT DEVELOPMENT FLOATING BRIDGE E39 BJØRNAFJORDEN

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1 INTRODUCTION

1.1 General

The purpose of this document is to summarize the requirements and principles for how we plan to design the foundation solutions for this stage of the project. is in order to explain how we interpret and apply the design requirements given by the client. It is here assumed that the design basis used in this phase is also valid for the next phases. In this document we have also explained how we use the results produced in previous phases of the project, performed by Multiconsult with cooperating companies. It is agreed with the client that we may use the results when found adequate, relevant and correct for our concepts.

Included are also descriptions of how we will perform analysis for our design and assumptions made regarding soil conditions.

Note that this document is applicable for the selected concept K12. For the screening phase where the four concepts K11-K144 are evaluated, more rough and superior evaluations are made.

1.2 Abbreviations and definitions

- SLS Serviceability Limit State
- ULS Ultimate Limit State
- ALS Accidental Limit State
- FAT Fatigue limit state
- CC Consequence class
- RC Reliability class
- FoS Factor of safety
- MBL Minimum breaking load
- OON Olav Olsen and Norconsult AS joint work collaboration
- PGA Peak Ground Acceleration
- γ_m Soil material factor

2 BASIS FOR DESIGN

2.1 Introduction

Design requirements are given in the Basis documents issued by the Client, where rules and regulations valid for the project are specified. We have included our interpretation of the requirements in this document. Where we have identified disagreement between the rules given, our interpretation of the rules is given.

In the following chapters, only principal documents, most important for the design are included.

2.2 Governing documents

- **Design basis documents**

Main Design basis documents are:

- SBJ-02 C4-SVV-02-RE-004_0 Design Basis – Geotechnical design
- SBJ-32 C4-SVV-26-BA-001_3 Design Basis – Mooring and anchor
- SBJ-32 C4-SVV-90-BA-001_0 Design Basis – Bjørnafjorden floating brigdes

- **Rules and regulations**

Most relevant rules and regulations listed as prioritized by the client are:

- Handbook N200: Vegbygging (General rules for road construction, 2018)
- Handbook N400: Bruprosjektering (Rules for bridges design, 2015)
- Handbook V220: Geoteknikk i vegbygging (Guidelines for geotechnical design, 2018)
- Handbook V221: Grunnforsterkning, fyllinger og skråninger (Guidelines for Ground improvement, fillings and slopes). design, 2014
- NS-EN 1997-1:2004+A1:2013+NA:2016: Eurocode 7: Geotechnical design – Part 1: General rules
- NS-EN 1998-1:2004+A1:2003+NA:2014: Eurocode 8 Design of structures for earthquake resistance – Part 1: General rules seismic actions and rules for buildings
- NS-EN 1998-2:2005+A1:2009+A2:2011+NA:2014: Eurocode 8 Design of structures for earthquake resistance – Part 2: Bridges
- NS-EN 1998-5:2004+NA:2014: Eurocode 8 Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects
- Forskrift om posisjonerings – og ankringssystemer på flyttbare innretninger (Ankringsforskriften 09). FOR-2009-07-10-998

Additionally, the following off-shore standards and recommended practice are followed for anchor design:

- DNVGL-OS-C101 Design of offshore steel structures, general LRFD method, 2016
- DNVGL-OS-E301 Position mooring, 2015
- DNVGL-RP-E303 Geotechnical design and installation of suction anchors, 2017
- DNVGL-RP-E302 Design and installation of plate anchors in clay, 2017
- NS-EN ISO 19901-7 Dynamisk posisjonering og forankring av flytende innretninger og flyttbare innretninger til havs, 2013

2.3 Consequence class

According to Design Basis - Bjørnafjorden floating bridges ref./3/, the bridge is categorized as CC3 and RC3 according to Eurocode. The Design Basis allows for particular members of the structure to be categorized as CC2 and RC2. Furthermore, in Design Basis – Geotechnical design ref./1/ it is stated that the general consequence class for the project is CC3 and for other components which are not critical for the global stability of the bridge a lower consequence class can be assessed.

For the concepts which relies on a mooring system, the individual components in the mooring system including anchors is thereby regarded as CC3 and RC3 according to Eurocode. Furthermore, it is defined in the Design Basis for mooring and anchoring ref. /2/ that the mooring system shall be designed with safety factors in NS-EN-ISO 19901-7 CC3. Hence, the consequence class according to DNVGL standards is set to 2, where failure may lead to unacceptable consequences which is the strictest consequence class in the DNV-regulation.

2.4 Redundancy

The redundancy requirements are given in the Design Basis for Mooring and Anchoring, ref./2/. Two different requirements are given:

<u>According to chapter 3.1 – Operation condition:</u>
The bridge shall be designed to operate with two lines damaged or out of service for 2 years for every 25 years of life.

<u>According to chapter 4.7.4 In service and replacement:</u>
The bridge shall be designed to operate with two lines damaged or out of service for two years. Traffic loads and environmental loads shall be considered for this condition. ULS and FAT conditions shall be concluded in the calculation.

These requirements are interpreted as:

1. Two random lines (not neighbor lines) may be out of service, and the remaining anchors and mooring lines must have sufficient structural and holding capacity to withstand the loads in this condition. As the period is 2 years, this is assumed to be an ULS condition. Design loads will be taken from the global analysis and corrected based on local analysis of the mooring system.
2. When two neighbor lines in one anchor group are out of service it assumed to be an ALS condition and the safety factors are thus reduced.

Note that neighbor lines are here defined as two lines within same anchor group on the same side of the bridge girder.

For Geo-hazard and slope stability the requirements are given in chapter 2.5 and described in chapter 5.

2.5 Summary

The table below gives a brief version of the design requirements applied for this phase.

> *Table 2-1 Summary of design requirements*

<table>
<tr><th>Condition</th><th>Design requirement</th><th>Comment</th></tr>
<tr><td>Local slope stability
ULS-condition</td><td>Effective: $\gamma_m \geq 1.6$
Total: $\gamma_m \geq 1.6$</td><td></td></tr>
<tr><td>Global slope stability
ULS-condition</td><td>Effective: $\gamma_m \geq 1.4$
Total: $\gamma_m \geq 1.4$
For special cases:
Effective: $\gamma_m \geq 1.25$
Total: $\gamma_m \geq 1.3$</td><td>Lower factor of safety may be used where no potential factors are identified to reduce the stability of the slope. This option will not be applied for this phase</td></tr>
<tr><td>Earthquake (2750 years event)
Seismic ALS-condition</td><td><u>Pseudo-static analysis:</u>
Fill materials: $\gamma_m \geq 1.2$
Clay and other materials: $\gamma_m \geq 1.1$

<u>Dynamic analysis:</u>
Transient shear strain $\gamma_p \leq 3\%$</td><td>A dynamic analysis will be performed if the pseudo-static criteria are not satisfied, ref. /1/

Rate effects and cyclic degradation are assumed to have no negative impact on strength parameters. The effect is assumed to be 0</td></tr>
<tr><td>Holding capacity of anchors
ULS- & ALS-condition</td><td>Soil material factor γ_m
<table>
<tr><th>Anchor type</th><th>ULS</th><th>ALS</th></tr>
<tr><td>Gravity</td><td>1.3</td><td>1,0</td></tr>
<tr><td>Combined</td><td>1.3</td><td>1.0</td></tr>
<tr><td>Suction</td><td>1.2</td><td>1.2</td></tr>
<tr><td>Mixed</td><td>1.2</td><td>1.2</td></tr>
<tr><td>Plate</td><td>1.4</td><td>1.3</td></tr>
</table></td><td>The different anchor types are defined in chapter 6.1.

Material factors for suction and plate anchors are calibrated for undrained failure modes, ref. /26/ and /27/. It's here deemed satisfactory for anchor design given that the net vertical load is in the gravitational direction during operational loading.

The degradation of cyclic strength is assessed, and assumed to be neglectable</td></tr>
<tr><td>Landslide impact
ALS-condition</td><td>Anchors will be evaluated for landslide impact.</td><td>Detailed calculations will not be performed.</td></tr>
<tr><td>Settlements
SLS-condition</td><td>Settlements will be checked in relation to allowable deformations in anchoring system. Lateral consolidation and creep deformation due to operational load, i.e. permanent horizontal pre-tension, shall be studied.</td><td>Allowable deformations will be decided based on global and mooring analysis.</td></tr>
<tr><td>Effect of sedimentation on long-term stability of slopes</td><td>The effect of 30 cm sedimentation shall be studied with respect to slope stability.</td><td>30 cm is within the accuracy of the bathymetry information. The effect is assumed not be critical and will not be performed</td></tr>
<tr><td>Probabilistic analysis</td><td>Probabilistic analysis can be considered if it seems necessary.</td><td>Such analysis is not assumed to be expedient in this phase of the project.</td></tr>
</table>

3 DESIGN DOCUMENTS FROM PREVIOUS PHASES

3.1 Relevant documents

Results attained in the previous phases will be used in our design, where considered relevant. Interpretation of soil parameters done by NGI ref. /17/ will be used for design calculations. Further soil parameters are taken from the design of anchorages ref. /19/, where deemed relevant. This is further described in chapter 4 and 7.

The maps, showing bathymetry, slope angle, azimuth, isopach, watershed, static factor of safety (FoS) and permanent transient shear strain for 2750 years return, given in ref. /13/, will be used for anchor site and geohazard evaluation. Where relevant, slope stability profiles calculated in Plaxis 2D by NGI in /13/ are used, as shown in chapter 5.

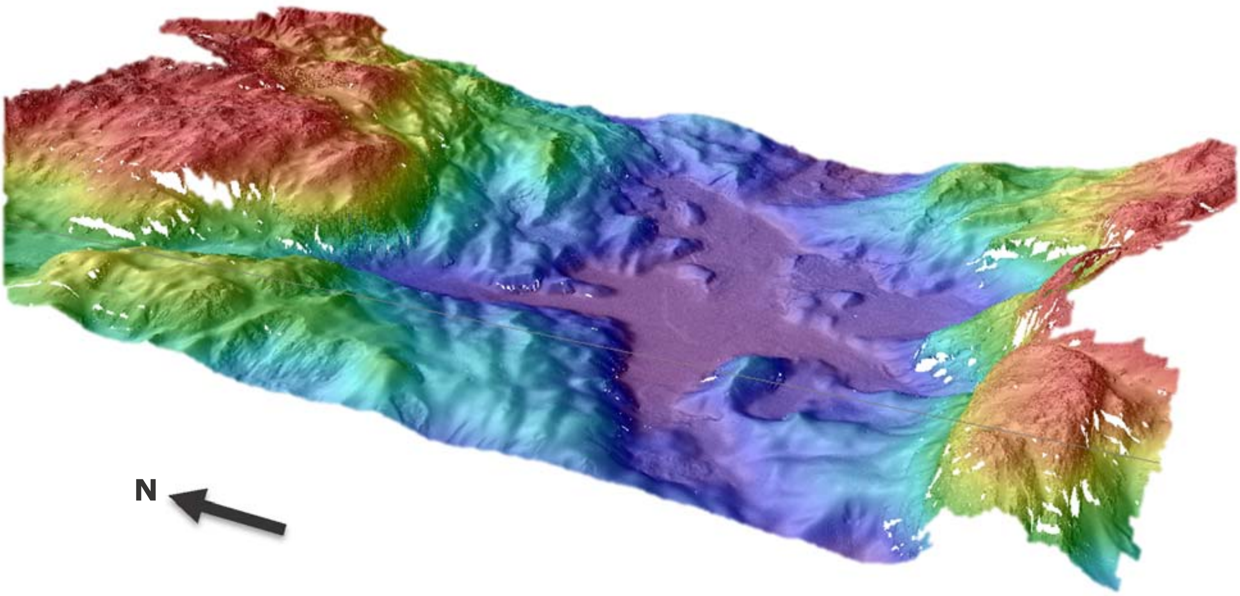
For evaluation of liquefaction the results given in ref. /14/ will be used.

4 BATHYMETRI AND SOIL CONDITONS

4.1 Bathymetry and derived maps

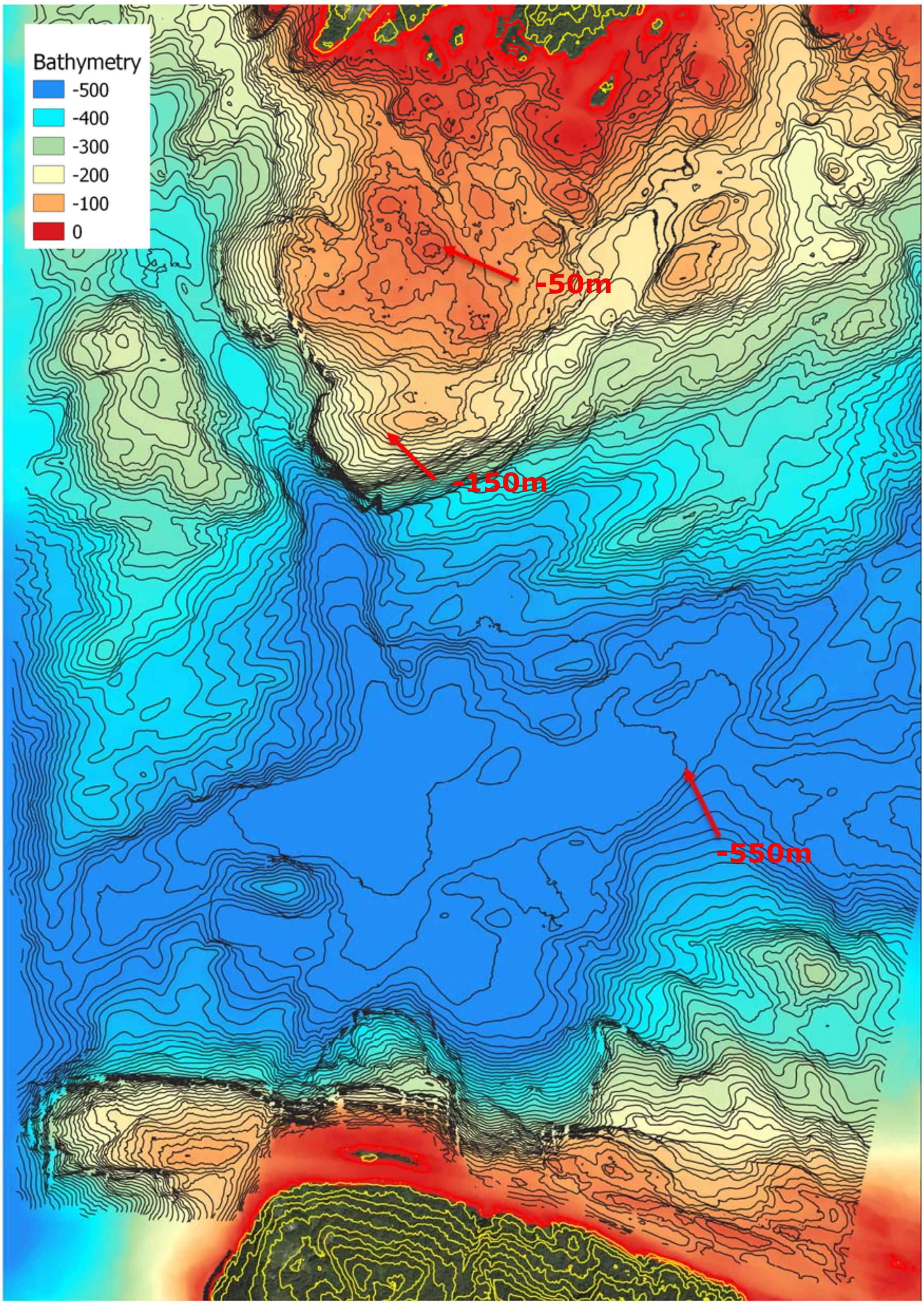
As stated in the Design Basis ref. /1/, the bathymetry data illustrates variable seabed conditions. The fjord is asymmetrical with undulating seabed as shown in Figure 4-1 3D representation of the bathymetry. The coloring indicates the height where red is the highest elevation and purple being the lowest elevation.. On the southern side there is a steep inclination down to the basin. The basin itself stretches out almost two thirds of the crossing distance and has depth of about -550 m. The last part in the north, which is shallower from about -150 m to -50 m depth, consists mainly of exposed bedrock as shown in Figure 4-2.

The post-processed results done by NGI in the last phase ref. /13/ has been used for concept screening. This was done due to time constraints and since it contained additional post-processed data. For the chosen concept the high-quality bathymetry data and the survey done in 2018 will be post-processed and evaluated.

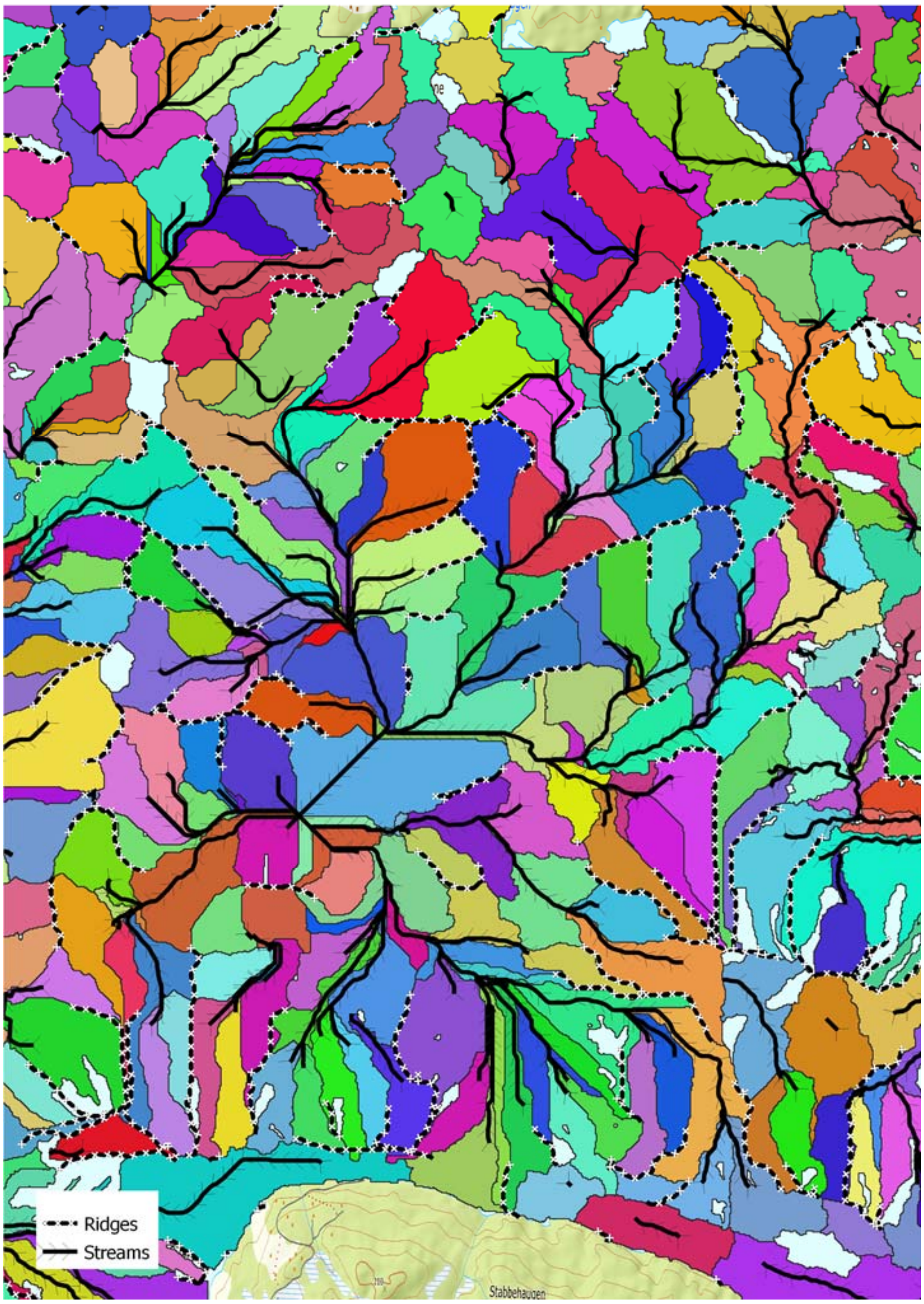


> *Figure 4-1* 3D representation of the bathymetry. The coloring indicates the height where red is the highest elevation and purple being the lowest elevation.

Several other maps are derived from the bathymetry data. Among other a map of the slope angle, the azimuth and the watershed has been interpreted with the software GlobalMapper by NGI in the last phase ref. /13/. The slope angle map gives the steepness at each pixel, while the azimuth map indicates which direction the seabed inclines, measured relative to the North. The watershed map is a combination of the latter two and on land is normally used to estimate the catchment area, also known as flood runoff areas. In this case the watershed map is used to check potential surface avalanches which may impact the anchor locations.



> *Figure 4-2* – Bathymetry of Bjørnafjorden with 10 m equidistance

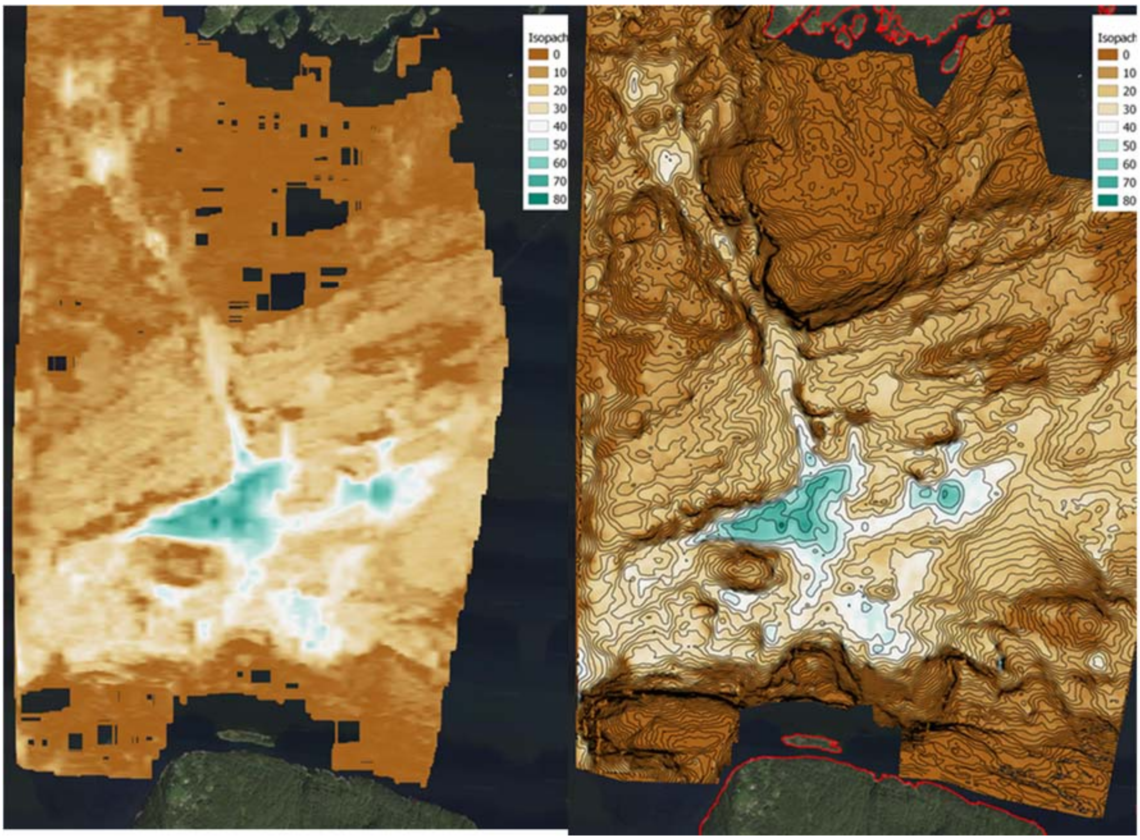


> *Figure 4-3* – Watershed of Bjørnafjorden with 50 000 m^2 drainage area.

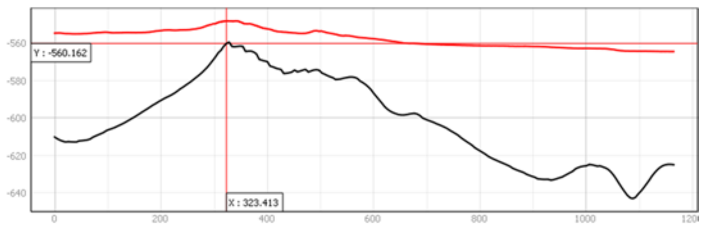
The watershed analysis map is shown above in Figure 4-3 with ridges and streams. A ridge is the highest local elevation and creates a natural boundary between two or more basins. Streams is the lowest local elevation and is similar to a valley. On land this would typically be rivers during flooding. This implies that anchors located in streams are more susceptible to landslides than anchors located near ridges. One should also note that the watershed map has no information of depth. Thus, this may give a wrong impression of the Geohazard since the inclination at seabed is not necessarily the same at bedrock.

4.2 Isopach

Acoustic measurements were done in 2016 and 2018 by DOF SubSea. The isopach post-processed by NGI is shown below in Figure 4-4 a, while Figure 4-4 b is the post-processed data done by OON which includes data from 2018. It is stated in the Geohazard rapport by Multiconsult/NGI ref. /13/ that the data quality from 2016 is variable because of the restriction regarding equipment and configuration given by the Fishery Directorate and the challenging topography. It is also stated that most of the data were collected from ROV systems and thus the vertical datum had to be corrected. The recorded time measurement was afterwards processed to an isopach map, which gives an indication of the sediment thickness in the fjord. Measurement of the track plot gives roughly a gridding distance of 50 m between each line. Although deviation in depth to bedrock is expected, in this phase the isopach is assumed to be exact. The bedrock can thereby be calculated by subtracting the bathymetry with the isopach map such that profiles as shown in Figure 4-5 can easily be viewed in QGIS.



> *Figure 4-4 a) Isopach provided by Multiconsult/NGI 2017 b) Interpolated isopach from data provided by DOF 2018 with 10 m equidistance.*



> *Figure 4-5* Profile view with seabed and calculated bedrock

4.3 Soil conditions

4.3.1 Material parameters

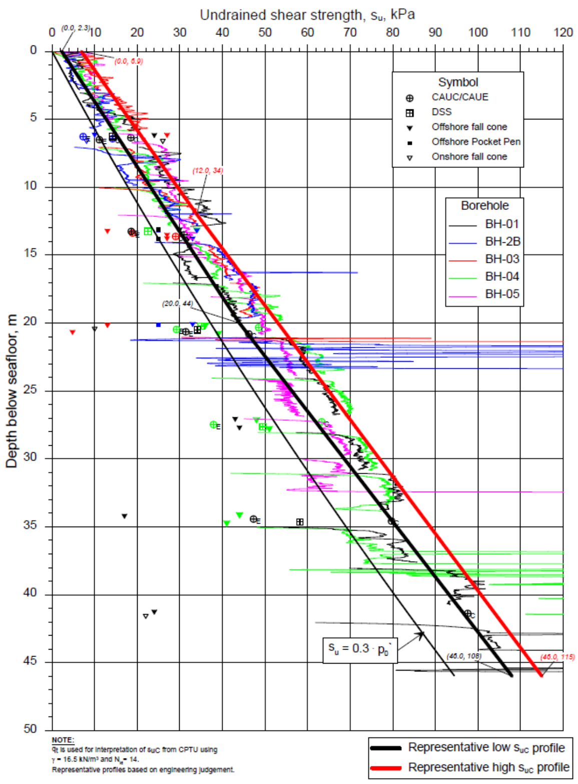
In-situ geotechnical data and soil samples are only collected at 5 locations, all of them taken in the central flat seabed basin. The samples have been tested and representative soil parameters have been proposed by NGI ref. /17/. The geotechnical data shows homogeneous conditions with low-sensitive NC-clay and will be used for geohazard evaluations and anchor design in this phase.

> *Table 4-1* Summary of representative soil parameters ref. /17/

z (m)	w (%)	γ (kN/m³)	γ_s (kN/m³)	I_p (%)	FC (%)	CC (%)	OCR (-)	S_t (-)	S_{uD}/S_{uC} (-)	S_{uE}/S_{uC} (-)
0 - 46	70 - 41	15.7 - 18.0	27.3	35	96	50	3.6 - 1.2	4	0.75	0.60

Note that the soil density increases linearly with depth. As a simplification a constant value will be used in calculations. In most cases 16 kN/m³ will be used. For special with deep failure zones the value may be increased and vice versa for shallow failure zones.

Additionally, CPTU results have been used to estimate undrained shear strength profiles with depth. In the design basis for mooring and anchor, ref. /2/, it is stated that the characteristic undrained shear strength shall be taken as the mean value, accounting for soil variability. The mean characteristic shear strength is here taken as the average of the representative low and high estimates and reduced with 10% to account for variabilities. The mean characteristic shear strength will be used both for holding and penetration calculations, and for stability calculations.



> *Figure 4-6* Active shear strength profiles with depth /17/

$$S_{uC,low}(z) = \begin{cases} 2.30 + 2.085 \cdot z & \text{for} \quad z < 20m \\ -5.23 + 2.46 \cdot z & \text{for} \quad 20m \le z \le 46m \end{cases}$$

$$S_{uC,high}(z) = \begin{cases} 6.90 + 2.26 \cdot z & \text{for} \quad z < 12m \\ 5.52 + 2.38 \cdot z & \text{for} \quad 12m \le z \le 46m \end{cases}$$

$$S_{uC,avg}(z) = 0.9 \cdot \frac{S_{uC,high} + S_{uC,low}}{2} = \begin{cases} 4.14 + 1.96 \cdot z & \text{for} \quad z < 12m \\ 3.52 + 2.01 \cdot z & \text{for} \quad 12m \le z < 20m \\ 0.15 + 2.18 \cdot z & \text{for} \quad 20m \le z \le 46m \end{cases}$$

The mean average shear strength $S_{uC,avg}$, which is shown above, will in certain calculations be simplified to
$S_{uc} = 4 + 2z$. Note that this is the active shear strength and the shear strength anisotropy factors given in Table 4-1 will be used where appropriate. In calculation models with no anisotropy, the direct shear strength will be used.

For drained calculations the friction angle is set to 32° and the attraction to 2 kPa, which is the same as cohesion equal to 1.25 kPa.

4.3.2 Seismic elastic response spectra

The peak ground accelerations (PGA) developed by Multiconsult/NGI, chapter 5 ref. /19/, will be used in earthquake design calculations. Note that the acceleration time series used in the analysis to find the PGA's are different from the ones used 1D dynamic slope calculations described in chapter 5.1.

One should also note that the time series used in the elastic response analysis is different from the ones provided by NORSAR. Since the moment magnitude is similar for the time series and the PGA from ~~NORSOR~~ NORSAR ref. /28/ is 130 cm/s^2 while Eurocode 8 give PGA of 132.8 cm/s^2 ref. /19/, it's assumed that these results are slightly conservative and can thus be used in this phase.

> *Table 4-2* Maximum accelerations from elastic response analysis ref. /19/.

Profile depth	S_e
[m]	[m/s^2]
0 (Ground type A)	3.32
9	3.72
16	3.76
26	3.32
36	3.49

5 GEOHAZARD AND SLOPE STABILITY

5.1 Introduction and previous work

The maps described in chapter 3.1 will be used for preliminary anchor location and slope stability evaluation. Bathymetry and derived maps are described in chapter 4.1. The maps based on 1D-calculations are:

- Static FoS: Calculated static factor of safety for one-dimensional soil column with respect to slope angle, sediment thickness and soil strength data.
- Transient shear strain: Calculated transient permanent shear strain for one-dimensional soil column with different acceleration time series. In the analysis 8 earthquake time series was scaled to the corresponding 2750 years return period with respect to the peak ground acceleration, ref. /13/. Dynamic slope stability is calculated by a non-linear total stress analyses in the time domain using a lumped mass system.

Furthermore, NGI have also performed PLAXIS 2D slope stability analysis for 40 profiles and PLAXIS 2D dynamic slope stability analysis for 1 profile.

5.2 Geohazard

The major concern with anchor location evaluation is avalanches caused by earthquake, erosion and/or other changes in the in-situ conditions. Several areas in the Bjørnafjorden has poor slope stability and the risk of surface and deep plowing landslides is therefore high. We have used the definition given in the design basis ref. /1/ to differentiate between local and global slope stability. Our approach to verify sufficient local slope stability is covered in chapter 5.3.1, whereas global slope stability is covered in chapter 5.3.2.

Geohazard assessments with regards to earthquake generated slides will be carried out according to Eurocode 8, as specified in the design basis ref. /1/. Note that this implies the DNVGL codes shall not be used earthquake design. Our approach to verify sufficient slope stability with regards to earthquakes is covered in chapter 5.3.4.

For anchor location evaluations the preliminary criteria given in the design basis ref. /2/ have been used as a limit. The criteria are with respect to seabed inclination and soil thickness and is mainly tied to anchor holding capacity. We have chosen criteria which are somewhat stricter than the limit, and our criteria are shown in chapter 6.1

The Design Basis ref. /1/, states that probabilistic analysis can be considered if it seems necessary. Such analysis is not assumed to be expedient in this phase of the project and will not be performed.

5.3 Slope stability

5.3.1 Local slope stability

Preliminary local slope stability will be determined using the static FoS map which is described in chapter 5.1. Stability control with regards to dead weight will be calculated with simple bearing capacity formulas, GeoSuite Stability or Plaxis 2D depending on the complexity and what is regarded to be most appropriate. The weight will be at the greatest after installation, but before hookup. As this might be a long-term situation the calculations will be done for undrained and drained conditions.

To control local stability of underwater fillings with respect to anchors, a slope shall not be steeper than 1:3 as stated in the design basis ref. /2/.

5.3.2 Global slope stability

Similarly, the preliminary global slope stability will be evaluated by using the static FoS map. The Plaxis 2D slope stability results from the last phase, ref. /13/, will also be used. Where considered required, global slope stability will be checked with GeoSuite Stability or Plaxis 2D. Where relevant, these calculations will be conducted with undrained and drained parameters.

5.3.3 Landslide dynamics

We will aim to find locations where static slope stability is not critical. Effects of landside dynamics will be evaluated on an overall level, but no detail analysis will be performed.

5.3.4 Seismic condition

Preliminary seismic slope stability will be checked using the permanent shear strain maps described in chapter 5.1. Analysis for seismic conditions will be conducted with pseudo-static analysis in GeoSuite Stability if deemed to be a critical issue. If the safety factor requirement is not met with a pseudo-static analysis, a dynamic analysis in Plaxis 2D will be executed. The 8 acceleration time series described in Appendix A ref. /13/, will then be used in the slope stability calculations. For dynamic analysis cyclic degradation shall be taken into account according to /8/. Cohesive materials will experience tall rate effects and degradation. According to /8/ the rate effect will typically give an increased shear strength of 30-40%, while degradation will decrease shear strength by 15-25% depending on the seismic class. In these preliminary studies we will assume that these effects cancel each other out.

5.3.5 Liquefaction

The evaluations presented in ref. /14/ indicates that some layers will liquefy for the given earthquake scenarios. However, NGI states four reason why significant liquefaction will not occur, which we agree upon. One of the reasons stated by NGI in the report is that many of the soil layers which are predicted to liquefy are more clayey than sandy and thus not a problem. It’s herein assumed that liquefication with regards to anchor design and slope stability is not an issue. Note that additional soil investigation at the proposed anchor sites will reveal whether this is a real issue or not.

6 ANCHOR DESIGN PHILOSOPHY

6.1 Anchor types

The type of anchor depends on the soil conditions, soil thickness, variability and bathymetry, seabed inclination and practicality with respect to marine operation and installation. The anchor criteria used for anchor site evaluation is summarized is presented in Table 6-1.

> *Table 6-1 Limiting conditions used in screening for the different anchor types used in this phase*

Anchor type	Maximum seabed slope [deg]	Soil thickness [m]
Gravity anchor	< 5	< 5
Combined anchor	< 5	5 < x < 15
Suction anchor	< 7.5	> 10
Mixed anchor	< 7.5	5 < x < 20
Plate anchor	< 10	> 15

6.1.1 Gravity anchor

For areas near the bedrock it's proposed to use gravity anchors. In the Design Basis ref. /2/ the preliminary criteria is set to 10 m soil thickness. From an economic and installation perspective this is reduced to 5 m. Before installation, the soil on top of the bedrock should be removed and exchanged with crushed rock/gravel. This is to ensure an even bedding surface for the gravity anchor. The simplest kind of gravity anchor is one of rectangular shape where plates are welded to a steel frame. This allows for some flexibility with respect to production. To ensure sufficient vertical and horizontal capacity, the anchor is filled with crushed rock, olivine and/or steel lumps. The choice of filling material is a matter of practicality and cost. It's assumed that the gravity anchors can be installed with high precision.

6.1.2 Combined anchor

Combined anchor is here defined as gravity anchors with additional measures, example skirts. The idea behind this anchor is to be used in areas with soil thickness between 5 and 15 meters. Like the gravity anchor the seabed should be flat. If placed on in-situ soil mass one should expect settlements. However, depending on the distance to bedrock, primary consolidation can be achieved before hook-up if properly planned. It's also assumed the failure mode is shallower when compared to foundation area. This in contrast to the mixed anchor which is described in 6.1.4

6.1.3 Suction anchor

The capacity of suction anchors is highly dependent on the diameter and skirt length, and thereby also dependent on the soil thickness. High precision regarding installation is assumed and tighter tolerances can thus be achieved. The soil thickness should be more than 10 m as stated in the Design Basis. According to Design Basis ref. /2/ the maximum seabed inclination for suction anchors is set to 10 degrees. For anchor site evaluation the criteria is set to 7.5 degrees such that higher holding capacity can be achieved.

6.1.4 Mixed anchor

Mixed anchor is defined as a suction anchor with additional measures. For instance, filling on the passive side or on top of the anchor to reduce skirt depth and increase capacity. The anchor type will primarily be used in areas where the soil thickness is between 5 and 20 meters. The main difference from the combined anchor is that the foundation area is smaller and thereby resulting in a deeper failure mode.

6.1.5 Plate anchor

Plate anchors can be either drag-in or push-in. Due to the required holding capacity and anchor size, the soil thickness should be at least be 15 m. Drag-embedded anchors requires an "airstrip" area with similar bathymetry when it comes to installation. Higher tolerances should therefore be accounted for. Assuming push-in installation of the SEPLA type, the bathymetry and installation tolerances are of less importance. The plate anchors are also less influenced by the seabed inclination and is thus set to 10 degrees in accordance with the design basis ref. /2/.

6.1.6 Anchor choice philosophy

Regarding priority of anchor choice, an evaluation based on robustness, installation practicality and cost will be done. Gravity anchors on bedrock, which is believed to be the most predictable, will be proposed whenever suitable. Given adequate topography conditions and sufficient soil thickness plate anchor will be recommended. If the topography is variable or difficult, suction anchors will be considered. Lastly for anchor sites with difficult topography and soil thickness, combined or mixed anchor will be evaluated. Based on the above assumptions a final evaluation with regards to cost will be performed with special regards to bundle discount due to similar anchors and marine operations.

6.2 Loads

6.2.1 General characteristic and design loads

It's specified in the geotechnical design basis, ref. /1/, that design of anchors shall be in accordance with DNVGL codes. The partial load factors from ref. /23/ are shown in the tables below and the design tension is estimated as $T_d = T_{c-mean} \cdot \gamma_{mean} + T_{c-dyn} \cdot \gamma_{dyn}$. Consequence class 2 as defined in DNVGL-OS-E301 is used in the design. The characteristic mean line tension is due to pretension and the mean effects from environmental loads, while the dynamic characteristic component is tension due to wave- and low-frequency cyclic loading. The environmental load components of T_{c-mean} and T_{c-dyn} are taken from the global response where the dynamic component is determined from envelope curves and corrected for use of different extreme value distribution. The pre-tension is estimated based on the local analysis of the mooring system.

The combined characteristic line loads are primarily dominated by the eigenmodes of the bridge and thus dependent on both the concept and anchor layout. Generally, the lowest modes for the different concepts is less than 0.1 Hz, i.e. low-frequency deformation. The anchors will therefore be subjected to one-way load cycling.

Furthermore, the design load is adjusted for the loss of two mooring lines and is either determined directly from calculations or estimated if global results are not available.

Table 1 Load coefficients for ULS [1)]

Consequence class	*Type of analysis*	γ_{mean}	γ_{dyn}
1	Dynamic	1.10	1.50
2	Dynamic	1.40	2.10
1	Quasi-static	1.70	
2	Quasi-static	2.50	

1) If the characteristic mean tension exceeds 2/3 of the characteristic dynamic tension, when applying a dynamic analysis in ULS consequence class 1, then a common value of 1.3 shall be applied on the characteristic tension instead of the partial load factors given in Sec.10 Table 1, ref. DNVGL-OS-E301. This is intended to ensure adequate safety in cases dominated by a mean tension component. The partial safety factor on the characteristic anchor resistance given in Sec.10 Table 1 is applicable in such cases provided that the effects of creep and drainage on the shear strength under the long-term load are accounted for.

Table 2 Load coefficients for ALS

Consequence class	*Type of analysis*	γ_{mean}	γ_{dyn}
1	Dynamic	1.00	1.10
2	Dynamic	1.00	1.25
1	Quasi-static	1.10	
2	Quasi-static	1.35	

> *Figure 6-1 Load coefficients from DNVGL*

6.2.2 Cyclic and rate effects

As stated in the previous chapter the global response is mainly governed by the eigenmodes of the bridge, which has a low-frequency. This would theoretically imply some cyclic degradation. However, during normal operations the loads are much lower than in ULS and ALS load case, and thereby probably resulting in negligible soil degradation. The low one-way cyclic response can possibly also increase the holding capacity as mentioned in the anchor design report from the previous phase, ref. /19/.

Furthermore, for most limit states the dynamic component is irregular with the peak load acting over a short duration. Due to rate effects the holding capacity may thus increase. As a simplification the cyclic and rate effects are neglected for anchor design in this phase. As previously stated, for earthquake loading the cyclic degradation and rate effect cancels each other’s out and therefor also neglected.

6.2.3 Seismic loading

Since the anchors are placed on a seabed with less than 10 degrees slope inclination, it is assumed that the elastic response described in chapter 4.3.2 is still valid. As a simplification the maximum acceleration will be used to calculate the resulting anchor forces. Kinematic interaction will also be neglected as recommended in ref. /8/. Where relevant, the added water mass is assumed to be 25% /19/ of the anchor volume which is above the seabed.

For anchors on top of ridges the maximum acceleration S_e is multiplied with an amplification factor S_T as described in Annex A Eurocode 8 - 5. The total line force is calculated as

$$F = (m_{anchor} + m_{added}) \cdot S_e \cdot S_T$$

where m_{anchor} is dry weight of the anchor structure and soil within, and m_{added} is the added water mass.

7 ANCHOR HOLDING CAPACITY AND DEFORMATIONS

7.1 Holding capacity

Soil spatial variability, installation tolerances and variation in seabed and bedrock will in this phase not be addressed. It's also assumed that the time between hook-up is sufficient such that full set-up effects may develop. The time required will depend on the anchor size and geometry, although 1 year is assumed to be a conservative estimate. Cyclic effects are handled as described in chapter 6.2.2. Furthermore, it's assumed there are no cracks between the soil and anchor.

7.1.1 Gravity and combined anchors

Design of gravity anchors and combined anchors shall be done in accordance with DNVGL-OS-C101 and DNVGL-OS-E301, ref. /23/ and /24/. Depending on the soil thickness, slope inclination, model complexity and required accuracy simple bearing analysis, Plaxis 2D and or Plaxis 3D will be performed.

If a gravity anchor is placed on crushed rock, the friction angle will be assumed to be 37° with zero in attraction. The filling material used inside the gravity anchor will be reduced with a factor of 0.9 as is customary in Eurocode. This is to account for possible unfortunate compaction and increased reliability.

For combined anchors both effective and undrained bearing capacity will be evaluated. Fillings must comply with the requirements given in chapter 5.3. The friction angle is assumed to be 37° and the attraction is assumed to be 0 kPa. The unit weight is reduced with a factor of 0.9.

7.1.2 Suction and mixed anchor

For design of suction and mixed anchor DNVGL-RP-E303, ref. /26/ will be used. The holding capacity will be calculated with simple limit equilibrium methods described in the DNVGL standard or Plaxis 2D using the same plane-strain model. To account for friction on the anchor sides, the direct undrained shear strength and a rectangular projection will be included in the calculations together with the appropriate set-up and material factor. In Plaxis 2D this calculation is included by reducing the input load.

The peak load is assumed to act over a short period, thus only undrained capacity is of relevance. In the case of Plaxis 2D, the NGI-ADP material model will be used where $G_{max}/S_u = 800$, $\gamma' = 16$ kN/m^3, $R_{inter} = 0.65$, which is taken from ref. /19/, and $S_{uC} = 4 + 2z$ as a simplification. The failure strains are roughly calibrated with soil tests and lab results at 13.25 m for DSS and 13.5 m for triaxial, giving $\varepsilon_{f,C} = 3.3\%$, $\varepsilon_{f,D} = 5\%$ and $\varepsilon_{f,P} = 10\%$

Fillings used in combination with suction anchors (i.e. mixed anchors) must comply with the requirements given in chapter 5.3. The friction angle is assumed to be 37° and the attraction is assumed to be 0 kPa. The unit weight is reduced with a factor of 0.9.

7.1.3 Plate anchor

Drag-in and push-in anchors will be design after DNVGL-RP-E302 ref. /27/. Simplified calculations will be carried out according to the standard. For complex calculations Plaxis 2D will be utilized.

For simplicity the kappa factor used in the standard is set to 1. The direct undrained shear strength is depth dependent and will be taken as the average shear strength within the failure zone.

7.2 Deformations

Deformations will be roughly calculated for a few chosen anchors which are assumed to be critical. What kind of calculations that will be done depend on the type of anchor, as described in the subchapters. The deformations will be compared to the deformation in the mooring lines, to check whether the anchors or the mooring lines represent the major contribution to the deformations.

7.2.1 Gravity and combined anchors

For gravity anchors it is assumed that most of the vertical deformations will occur in the rock fill after the installation and before the hook-up. After hook-up the weight of the anchor will be less and thus continued settlements is not considered an issue.

For combined anchors on top of clay simplified calculations of settlements due to consolidation will be carried out to estimate the magnitude of added pretension.

7.2.2 Suction and mixed anchor

The anchors will have the highest weight after installation, but before hook-up. Since there will be some time before the hook-up, the soil will consolidate. Some rough estimations of the remaining settlement after hook-up will be conducted.

Horizontal deformation due to consolidation and creep caused by pretension will be calculated by simple Janbu settlement models or in Plaxis 2D, depending on the complexity and what is regarded to be most appropriate

7.2.3 Plate anchor

Horizontal deformation due to consolidation and creep caused by pretension will be calculated by simple Janbu settlement model or in Plaxis 2D, depending on the complexity and what is regarded to be most appropriate

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