Fergefri E39 – Kryssing av Bjørnafjorden

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Design Basis – Geotechnical design

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Design Basis for Geotechnical Design

1. Introduction

The Norwegian Public Road Administration is planning to build a ca. 5000 m long bridge across Bjørnafjorden, located south of Bergen. The fjord is 560 m deep at the crossing site and the proposed bridge will become the worlds' longest bridge built at such water depths. Some of the proposed bridge concepts require seabed anchoring; Alternative K12 bowed floating bridge with side anchoring, Alternative K13 straight floating bridge with side anchoring and alternative K14 side anchored floating bridge with curved floating part.

This design basis is valid for all bridge concepts in phase 5. Design criteria for seabed anchoring is provided in a separate document Design basis - Mooring and anchor ref. /22/.

2. Overview of available data, phase 3

2.1 Bedrock geology

The bedrock near Bjørnafjorden mainly stems from Paleozoicum, and got its current position and structure during the Caledonian orogeny, which took place in the Silurian-Devonian transition time, about 425-416 million years ago. The Bergen Arc is strongly influencing the bedrock in the Bjørnafjorden region. It is composed of metamorphic rocks, most gneiss and intrusive rocks, including anorthosite, gabbro-anorthosite and mangerites in the central, inner portion, and a mix of volcanic and sedimentary layers in the outer and inner parts ref. /1/ and /2/. There are very little knowledge on the submarine bedrock, but four rock samples at Flua show that this elevated feature consist of Greenstone.

2.2 Quaternary geology

The marine geological investigations and analyses conducted by DOF Subsea Norway AS provides a good overview of subsurface conditions in the Bjørnafjorden crossing area. Investigations carried out are ref. /2/:

- Mapping of seafloor topography
- Mapping of sediment thickness
- Sediment deposition and layering
- Mapping of previous subsea landslides with release- and deposition areas
- Description of rock protrusions on the rock wall from Flua and down to the basin
- Sampling in rock for caissons on Flua, total of 4 samples

Bathymetry data from Bjørnafjorden illustrates the variable seabed conditions. The fjord is asymmetrical with undulating seabed. On the northern side, there is more exposed bedrock, some submarine elevations and plateaus. In the south, the inclination down to the basin is steeper and less variable. Sediments appear in the basins and the troughs. In the central part of the fjord there are some raised areas due to undulating bedrock.

The sediment thickness in Bjørnafjorden varies from >60 m in the deepest areas to about 5-20 m at the mid to lover parts of the slope. On the shallower and steeper areas there are little or no sediments (Figure 2). The sediments are quite homogeneous throughout the crossing area and very little variation laterally and vertically. The sediments are mainly composed of clay in the deeper areas.

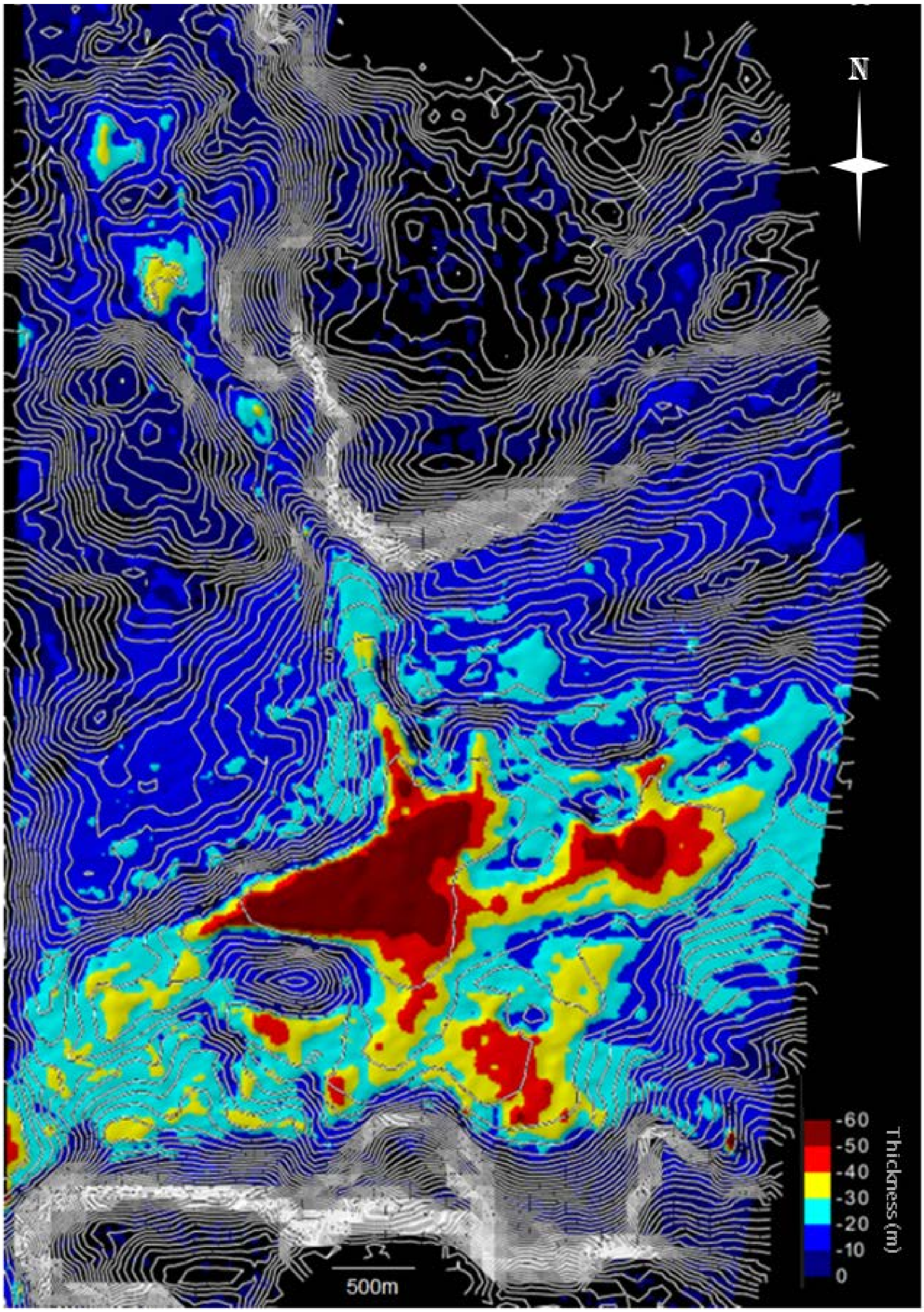

Figure 1: Isopach map with depth contours and coloured sediment thicknesses, ranging from 0 m at the shallow areas to 60 m in the deepest areas ref. /2/

Sub-bottom data suggest that the majority of the sediments in Bjørnafjorden were deposited during the last deglaciation. A somewhat lesser part of the sediments was deposited after the deglaciation, which is typical for most Norwegian fjords. The sub-bottom data provide evidence of slope failures (Figure 3). All of the identified slope failures have slipped at the interphase to assumed bedrock (acoustic basement) and it is not observed sedimentary slide planes ref. /2/.

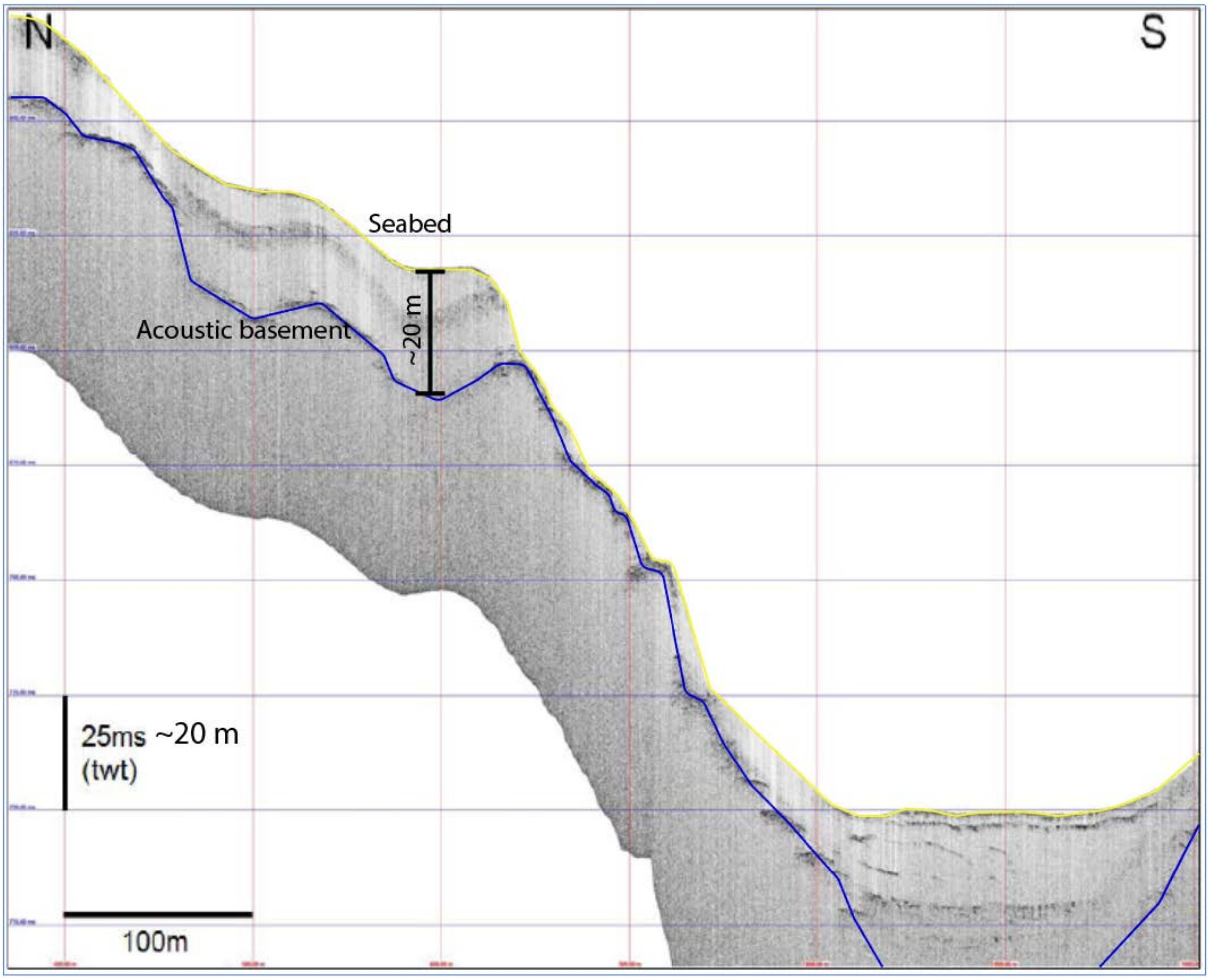

Figure 2: Example of a sub-bottom profile showing sediments on the slope above acoustic basement. The unit is up to 20 m (25 ms TWT) on the slide headwall (modified from ref. /2/).

The acoustic survey revealed 45 slides in the survey area ref. /2/. In 2016, one core was collected from the Bjørnafjorden basin by University of Bergen (Figure 4 and 5). Two recent slides have been radiocarbon dated showing that two slide events in Bjørnafjorden occurred about 500 and 1200 years ago ref. /3/ and /4/. The source areas are not known since the youngest debris deposit was too thin to be identified in the subbottom profiles; the debris deposit at about 500 yrs BP were 30 cm thick. The oldest debris lobe deposited at 1200 yrs BP were 1.5 m thick and may be possible to identify in the SBP data (work in progress). The largest slide lobe seem to have appeared before the two dated slides. The sediment core collected from Bjørnafjorden contain undisturbed sediments at 2.8 m depth. These sediments have been sent for radiocarbon dating and results are in progress. This may indicate a maximum age of the large debris lobe (Figure 4).

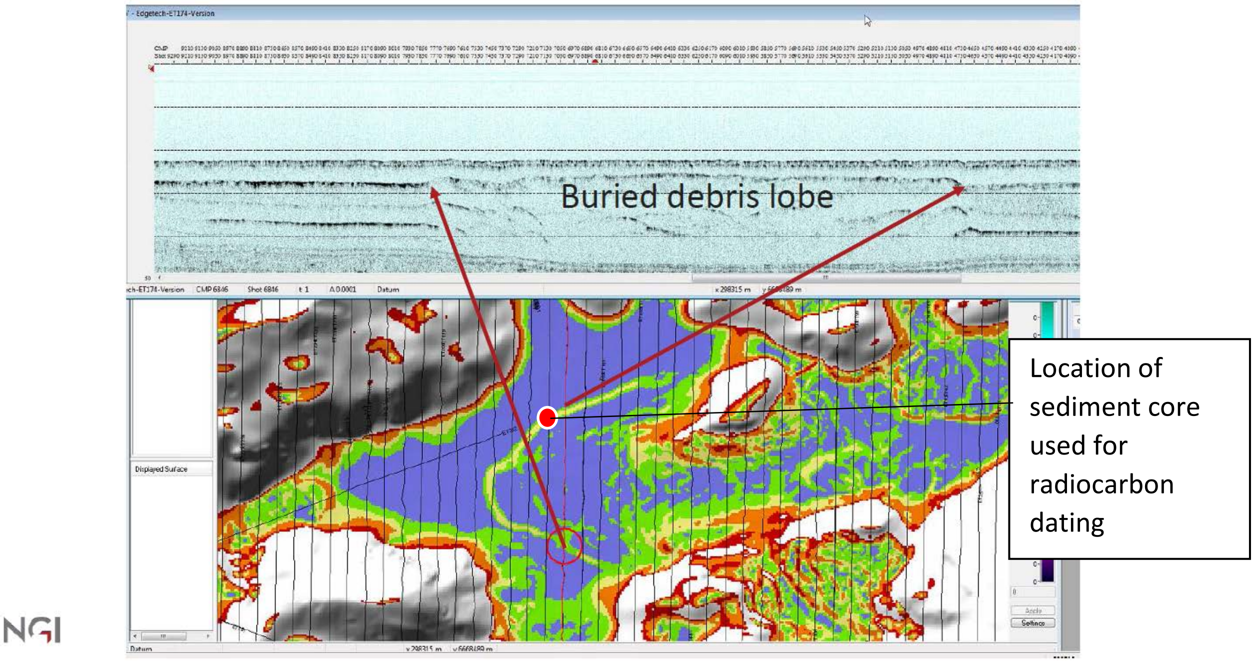

Figure 3: illustrating the biggest debris lobe observed in the Bjørnafjorden basin, the approximate thickness is indicated in the sub bottom data, suggesting a thickness of about 7-8 meters (Modified figure from M. Vanneste, NGI)

2.3 Geotechnical properties

Several geotechnical cores and analyses give a good impression of the general properties of the sediments in the area. See Figure 5 and 6 for geotechnical sampling locations.

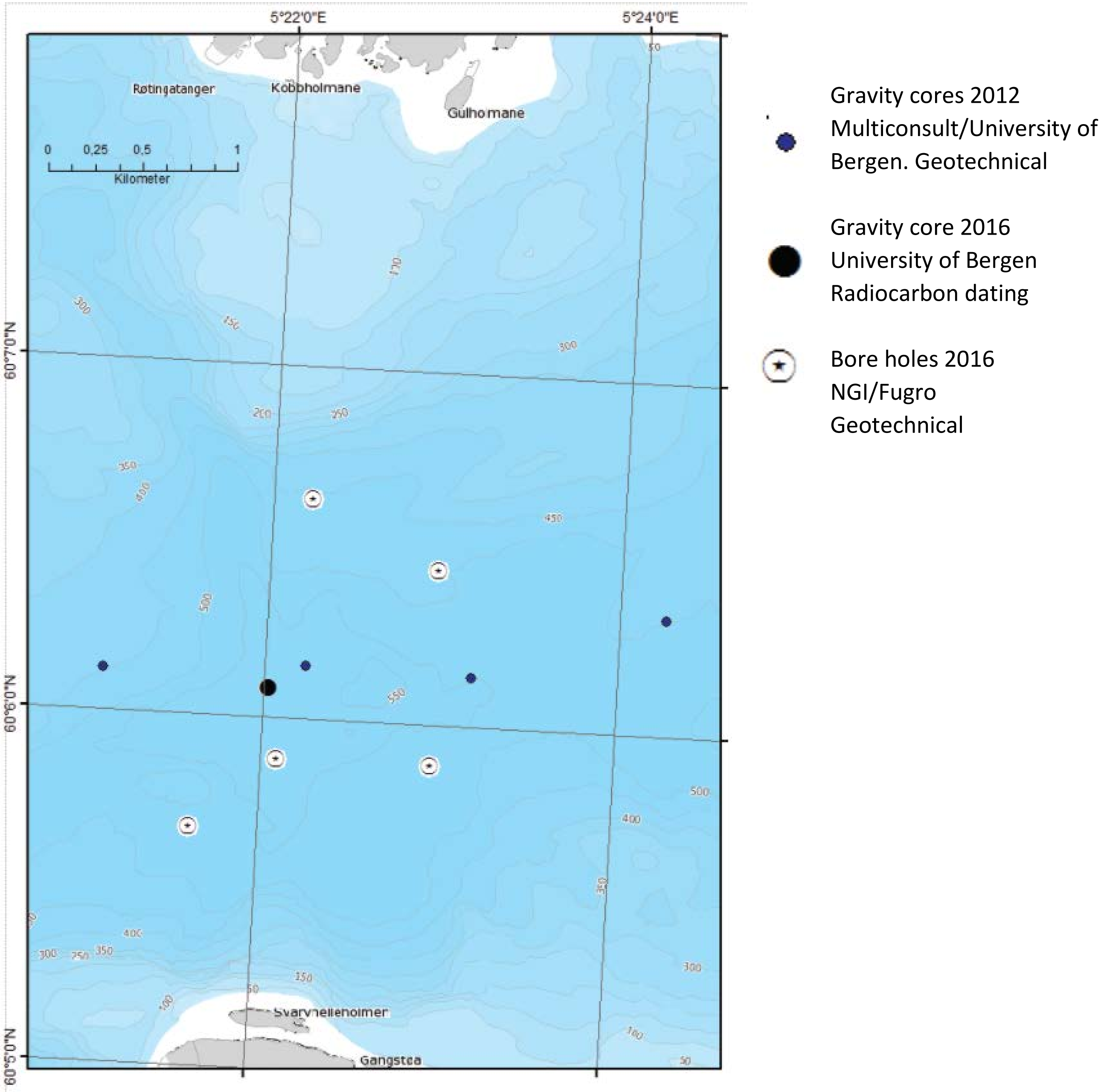

Figure 4: Core locations and bore holes from the central part of Bjørnafjorden ref. /6/, /7/, /8/ and /9/.

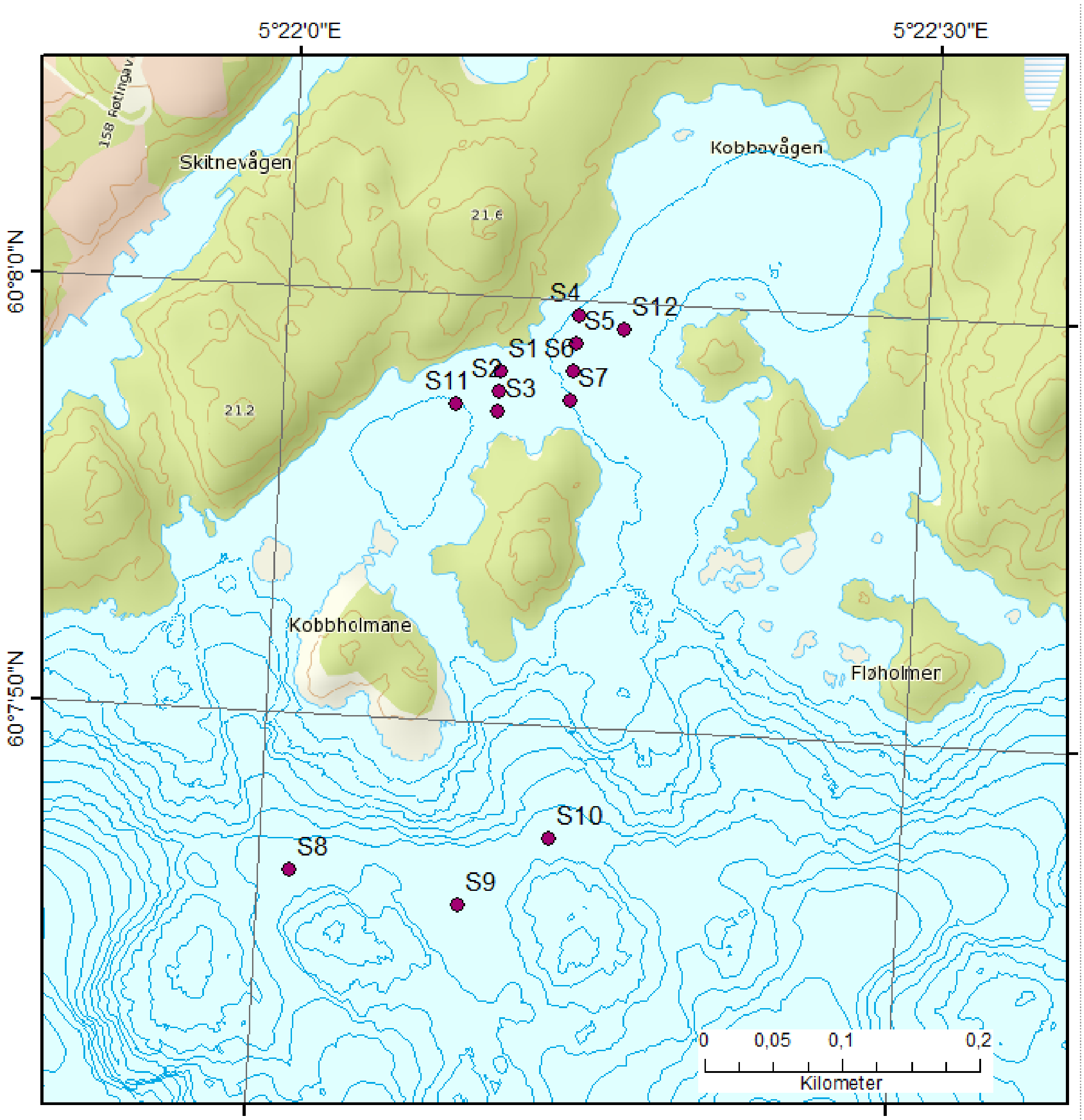

Figure 5: Shallow geotechnical boring locations ref. /10/, /11/ and /12/. Contour lines in 5 m increments.

The geotechnical results show that the sediments consist of normal consolidated clay with extremely low to high shear strength (Figure 7). Just above the assumed bedrock, a relatively thin layer of sandy, silty clayey material is observed. The preformed geotechnical analyses are listed below with references to the relevant reports:

- 4 sediment cores of about 3 m length from the basin, retrieved in phase 1 ref. /6/
- 5 boreholes in the deeper flat areas: Combined CPT and coring down to assumed bedrock, retrieved in phase 2. Drilling depths ranges between 22.5 m to 45.9 m below the seabed. Water depth at the drilling locations varies between 463.8 m to 561.2 m. ref. /7/, /8/ and /9/
- 12 CPT in the northern near-shore areas, retrieved in phase 2 ref. /10/, /11/ and /12/

Soil material may generally be described as normal- or moderately over-consolidated clay with very low to high strength [NGF melding no. 2]. Water contents varies from 13 % to 92%. In the bottom of these boreholes a layer of sand silty clayey material is encountered. This interpretation is based on

CPTU soundings, description of specimens and laboratory analyzes conducted both offshore and onshore. Offshore sampling comprises a total of 31 bag samples / plexiglass samples, 21 "Waxed subsamples" and 9 undisturbed cylinder samples for material classification and advanced laboratory tests. Performed laboratory tests onshore includes triaxial tests, cyclic direct shear tests, oedometer and index test.

The CPTU data indicate that the top 10-30 m of soil is essentially normally consolidated clay to silty clay. Beneath this clay layer the geophysical data shows an acoustic basement, which is interpreted as a stiff material. The data shows that the total unit weight varies from 15.6 kN/m^3 to 18.0 kN/m^3.

The undrained shear strength values (s_{uC}) from the CPTUs are based on $N_{kt} = 14$ and $\Upsilon = 16.5$ kN/m^3. The undrained shear strength increases linearly with depth, from values of 0-10 kPa near the seafloor, to 50-60 kPa at 25 meters below the seafloor.

Results from the laboratory test show strain- softening in some of the soil samples.

Geotechnical results from subsea investigations in 2016 ref. /7/, /8/ and /9/ shall be used for geotechnical assessments in concept verification phase 5.

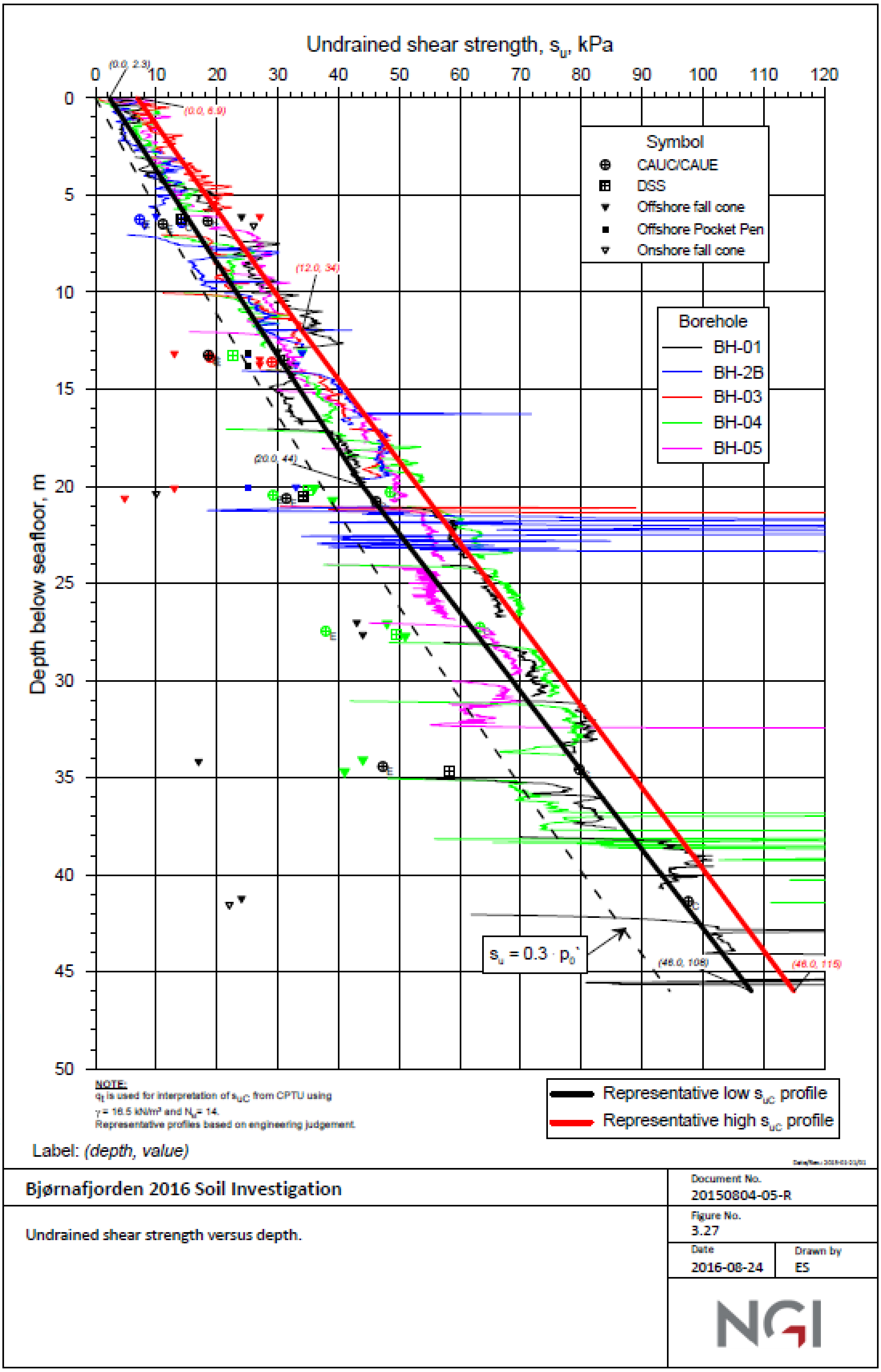

Figure 6: results from geotechnical boreholes down to assumed bedrock at water depths ranging from 450-560 m ref. /7/

3. Regulatory documents and standards

Handbooks published by Statens vegvesen (see list below) are to be used as basis for geotechnical design wherever applicable. If there are strong arguments to deviate from the “handbook” regulations, this needs to be approved by the Road Directorate before it is used further. If there is any uncertainty related to the interpretation of the regulations, these uncertainties need to be raised and clarified with Statens Vegvesen.

General rules are as described below, listed as prioritized.

1) Handbook N200: Vegoverbygning (General rules for road construction, 2018)
2) Handbook N400: Bruprosjektering (Rules for bridge design, 2015)
3) Handbook V220: Geoteknikk I vegbygning (Guidelines for geotechnical design, 2014)
4) Handbook V221: Grunnforsterkning, fyllinger og skråninger (Guidelines for Ground improvement, fillings and slopes, 2014)

Other design requirements, which are not covered by these handbooks, will be referred to following design codes wherever relevant

1) NS-EN 1990:2002+A1:2005+NA:2016 Eurocode 0 Basis for structural design
2) NS-EN 1992-1-1:2004+NA:2008 Eurocode 2 Design of concrete structures: General rules and rules for buildings + Amendment NS-EN 1992-1-1:2004/A1:2014 + Corrigendum NS-EN 1992-1-1:2004/AC:2010
3) NS-EN 1992-2:2005+NA:2010 Eurocode 2 Design of concrete structures: Bridges
4) NS-EN 1993-1-1:2005+A1:2014/AC:2015 Eurocode 3: Design of steel structures
5) NS-EN 1993-5:2007+NA:2010 Eurocode 3: Design of steel structures - Part 5: Piling + Corrigendum NS-EN 1993-5:2007/AC:2009
6) NS-EN 1997-1:2004+A1:2013+NA:2016: Eurocode 7: Geotechnical design – Part 1: General rules
7) NS-EN 1997-2:2007+NA:2008: Eurocode 7: Geotechnical design - Part 2: Ground investigation and testing + Corrigendum NS-EN 1997-2:2007/AC:2010
8) NS-EN 1998-1:2004+A1:2013+NA:2014: Eurocode 8 Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings
9) NS-EN 1998-2:2005+A1:2009+A2:2011+NA:2014 Eurocode 8: Design of structures for earthquake resistance - Part 2: Bridges
10) NS-EN 1998-5:2004+NA:2014 Eurocode 8: Design of structures for earthquake resistance - Part 5: Foundations, retaining structures and geotechnical aspects

In addition to above design codes, special design requirements for offshore structures shall be covered by following standards

1) ISO 19901-8:2014 Marine soil investigations
2) ISO 19901-4:2016 Geotechnical and foundation design considerations in offshore construction
3) Offshore geohazards: NORSOK standard Z-013. Risk and emergency preparedness analysis

Other design requirements, which are not covered by any of above, shall be determined in agreement with Statens vegvesen.

4. Geotechnical project category

4.1 Consequence class

A general consequence class for the project is defined in Design basis – Bjørnafjorden floating bridge ref. /21/. Selected consequence class is CC3, which is described as "Large consequence in form of loss of human lives, or extremely large economic, social or environmental consequences". Structures with consequence class 3 correspond to geotechnical category 3

Konsekvensklasse	Geoteknisk kategori
CC1	1
CC2	2
CC3	3

Figure 7: Definition of geotechnical category (Handbook N200)

For other components of the project, which are not critical for global stability of the bridge, a lower consequence class can also be assessed.

4.2 Reliability class

Reliability class is defined in accordance with Handbook N200. Rreliability class is selected to be RC3. Same also applies to subsea fillings.

Konsekvensklasse	Pålitelighetsklasse
CC1	RC1
CC2	RC2
CC3	RC3/RC4 [1)]

1) Vanligvis vil CC3 gi RC3. Spesielle vegprosjekter med ekstremt store konsekvenser, kan vurderes plassert i pålitelighetsklasse RC4.

Figure 8: Selection of reliability class (Handbook N200)

For other components of the project, which are not critical for global stability of the bridge, a lower reliability class can also be assessed.

4.3 Control/review of geotechnical design

Reliability class together with consequence class CC3/RC3 gives control class PKK3.

Pålitelighetsklasse (RC) / Geoteknisk kategori	1	2	3	4 [1)]
Geoteknisk kategori 1	PKK1	PKK2		
Geoteknisk kategori 2	PKK2	PKK2	PKK3	
Geoteknisk kategori 3		PKK2	PKK3	Skal spesifiseres

Figure 9: Requirements for design review (Handbook N200)

PKK3 requires extended/independent control. An independent control of the work in phase 5 of the project shall be carried out.

In case of other consequence class and reliability class is selected, relevant control class will be assessed in collaboration with Statens vegvesen

5. Geotechnical material/partial factors

The project shall use material/partial factors defined on basis of definitions given in Handbok N200 and relevant Eurocodes.

For design of anchors in ground for Side Anchored Floating Bridge, material/partial factors defined in relevant DNVGL codes shall be used (see Design basis - Mooring and anchor ref. /22/).

5.1 Deterministic analysis

Handbook V220 describes the method of selecting material/partial factors for deterministic slope stability analysis as following

"Partial factors shall be selected with regard to how soil strength is determined, how the failure mechanism is working and what is recognized practice. Note that the partial factors shall increase when the risk of progressive failure development in the brittle failure materials (Sprøbruddmaterialer) are considered to be present, and when it is required to bring it in accordance with the recognized practice of the analysis method and the problem to be addressed."

Partialfaktorer for $\gamma_{M,\varphi'}$ og $\gamma_{M,c'}$ ved effektivspenningsanalyser

Konsekvensklasse	Bruddmekanisme		
	Seigt, dilatant brudd	Nøytralt brudd	Sprøtt, kontraktant brudd
CC1 Mindre alvorlig	1,25	1,3	1,4
CC2 Alvorlig	1,3	1,4	1,5
CC3 Meget alvorlig	1,4	1,5	1,6

Partialfaktorer for $\gamma_{M,cu}$ ved totalspenningsanalyser

Konsekvensklasse	Bruddmekanisme		
	Seigt, dilatant brudd	Nøytralt brudd	Sprøtt, kontraktant brudd
CC1 Mindre alvorlig	1,4*	1,4*	1,4
CC2 Alvorlig	1,4*	1,4	1,5
CC3 Meget alvorlig	1,4	1,5	1,6

* Eurokode 7 krever at $\gamma_{M,cu} \geq 1{,}4$ ved totalspenningsanalyser

Figure 10: Partial factor Υ_M for effective stress and total stress analysis (Handbook N200)

For material with contracting behavior (CC3/RC3) N200 (see figure 10) sets partial factor **1.6 for local slope stability** in foundation area. For the side anchored bridge, the partial material factor for **global static stability shall be set to 1.4** for areas where a submarine slide can result in one or multiple anchor failure. These material factors applies to both drained and undrained analysis.

An illustration of safety philosophy regarding local and global stability is shown at figure 11.

The partial factors described here govern local and global stability in the soil, but not the anchor capacity calculations. These material factors are also applicable to stability analysis with dead weight of anchors and rockfills under gravity anchors.

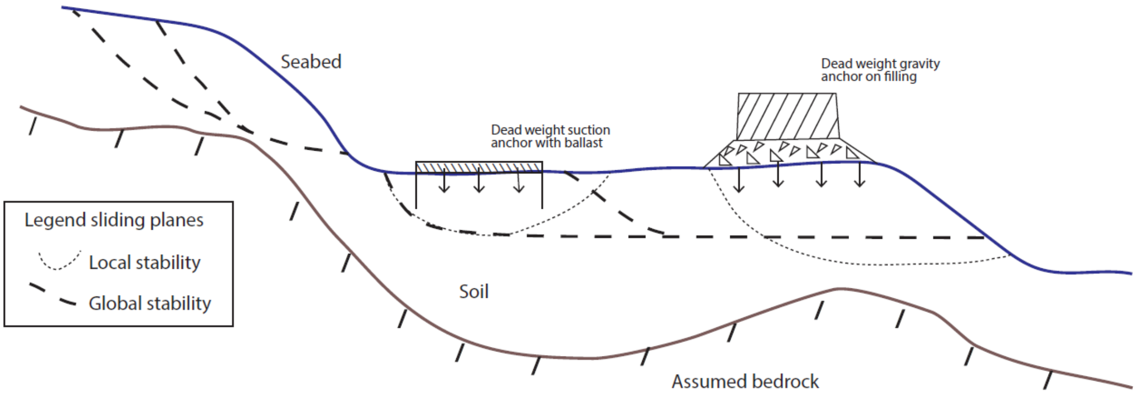

Figure 11: Illustration of safety philosophy for sub-marine slope failure (Ref. NIFS Report no. 15-2016)

5.2 Probabilistic analysis (reliability-based design)

Probabilistic analyses can be considered if seems necessary. In such case, required safety level for each analysis shall be assessed in a way that global safety of the structure is achieved as intended. These analyses shall be carried out using recognized industry standards and best practices.

6. Considerations for seismic loads

Design under seismic conditions shall be in accordance with Eurocode 8. Seismic condition shall be considered in both ultimate limit state (ULS) and accidental limit state (ALS).

Seismic class shall be determined in accordance with Table NA.4 (902) in Eurocode 8-1 for each structure, and for bridges specifically according to Table NA.2 (901) in Eurocode 8-2.

The bridge structure is placed in **Seismic Class IV** (structure with total length over 600 m ref. Table NA.2 (901) in Eurocode 8-2). This seismic class IV is also valid for associated structures which can affect the global stability of the bridge. A lower seismic class can be considered for the structures planned to be built independent of the main structures and fulfill the criteria for a lower seismic class according to Eurocode 8.

Ground type and seismic amplification factor with parameters corresponding response spectrum shall be determined in accordance with Eurocode 8-1 and SVV earthquake guideline ref. /19/ .

Ground type for the ground where clay sediments are present shall be established via calculation of $v_{s,30}$ for the actual soil profile in question. **Ground type A** shall be used for foundations directly on rock (ref. NS-EN 1998-1:2004+A1:2013+NA:2014, table 3.1).

Selection of earthquake loads for ULS and ALS conditions shall be in accordance with SVV handbook N400 and Eurocode 8.

Bedrock accelrations for assessment of exclusion criteria and/or any earthquake calculations shall be based on NORSAR report Probabilistic Seismic Hazard Analysis for Bjørnafjorden ref. /20/.

For analyses, where it is required to calculate a response spectrum, it is referred to Eurocode 8 and PSHA report for Bjørnafjorden ref. /20/.

7. Geohazard analysis

For geohazard analysis risk analysis framework presented in NORSOK standard Z-013 Risk and emergency preparedness analysis can be used. A specific Geohazard risk analysis framework based on NORSOK Z-013 is presented in International Association of Oil and Gas Producers report no. 425.

Geohazards analysis can be carried out according to guidelines provided in OGP report no. 425 as presented in figure 13.

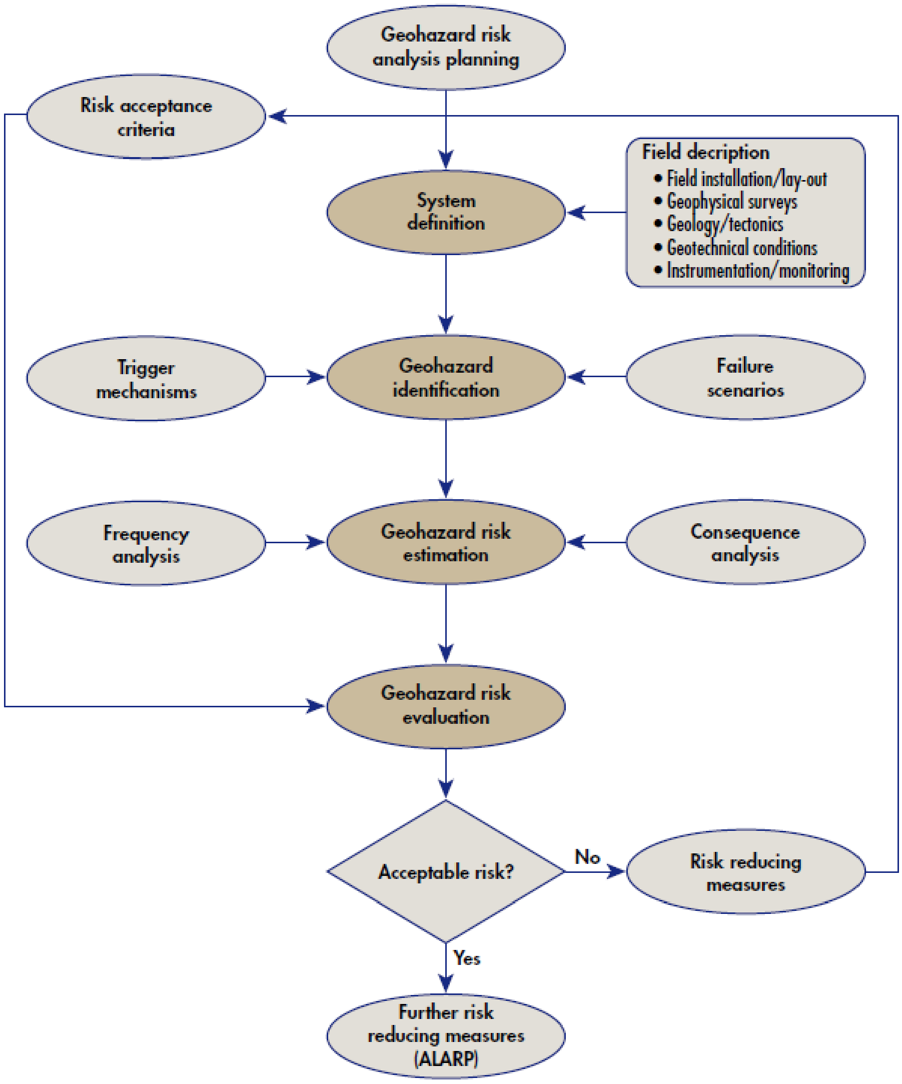

Figure 12: Example of geohazard risk analysis framework (OGP report 425)

Geotechnical assessments related geohazard analysis for earthquake generated slides shall be carried in accordance with Eurocode 8. Seismic class and ground type for each foundation area shall be assessed according to ground condition described in geotechnical investigation reports ref. /6 to 13/.

8. Slope stability

Stability of the seabed shall always be checked when either stability of a natural slope can be affected by foundation or environmental loads or a failure in natural slope can hit a foundation and cause destruction of anchor(s). Deterministic slope stability analysis shall be carried out to fulfil safety requirements given in section 5.1 of this document.

As given in section 5.1, slopes affecting anchors shall normally be designed with a factor of safety of 1.4. However, lower factor of safety can be accepted under special cases with an assessment of potential factors which could worsen the stability of the slope during the design life of the anchor(s) which is 100 years. In such cases if it is concluded that the stability of the slope shall not be reduced during design life of anchor(s) then the lower value of global safety factor can be accepted. This implies that a factor of safety of 1.25 for an effective stress stability analysis and a factor of safety of 1.3 for a total stress stability analysis, see figure 13. This is applicable only to static slope stability analyses.

	Effective stress analysis	**Total stress analysis**
Normal situation	1,4	1,4
Special case	1,25	1,3

Figure 13: Requirement for safety factor for static slope stability

Effective stress (drained) static slope stability analyses shall be carried out for those slope which are not effected by any external loading causing an undrained situation.

Identification of critical profiles shall be selected from spatial investigation that identifies potential release-areas around anchor areas.

8.1 Identification of critical profiles

Areas around the anchor locations shall be assessed for potential slope failures, which can directly or indirectly affect the anchoring structures.

Critical profile(s) shall be selected from careful investigation of the identified areas.

8.2 Selection of static design shear strength profile

Design profile for active shear strength shall be selected considering all available data from geotechnical investigations. Quality of gathered data shall be kept in focus and only good quality data from field and laboratory investigations shall be used for selection of design shear strength profile. Guidelines provided in ISO 1991-8:2014 shall be used for such process.

As a part of the soil investigation performed in phase 2 of the project NGI prepared a soil parameter report ref. /7/ that contains recommended static design shear strength profile which can be used for further analysis.

8.3 Selection of material model for stability and deformation analysis

Material model for modelling and analysis shall be selected keeping in view the soil behavior defined by the results of the field and laboratory testing and possible loading scenarios. Sample quality and subsequent result shall be kept in view for selection of material model.

8.4 Consideration of 3D effects for 2D analysis

Stability analysis shall be carried out by 2D plain strain analysis program. In cases where results from the 2D analysis gives marginally low safety, 3D effects of the analyzed geometry shall be considered and documented.

8.5 Landslide dynamics

In those areas where static slope stability analysis show critical stability condition around anchor location which could directly or indirectly affect the ancho, a landslide dynamics study shall be carried out.

Landslide dynamics parameters such as runout mechanism, flow velocity, flow height, impact velocity, ploughing depth and impact energy etc. shall be calculated and used for further anchor design.

8.6 Effect of sedimentation on long-term stability of slopes

It is assumed that there will be a natural sediment accumulation process on the slopes. The effect of this on the stability condition needs to be examined. For this, a sedimentation rate of 30 cm per 100 years shall be considered in the analysis.

8.7 Dynamic slope stability (seismic condition)

Slope stability analysis under seismic condition shall be in accordance with Eurocode 8 part 5 (NS-EN 1998-5:2004+NA:2014).

Limit state condition for slope(s) under seismic loading according to 4.1.3.1(2)P is defined as "unacceptably large permanent displacements significant for both structural and functional effects on structures under study", which in this case are anchors on the seafloor.

Geotechnical analysis to check this limit state can be carried out by following methods

1. Pseudo-static analysis
2. Dynamic analysis

Safety factor requirements for simplified pseudo-static method are given as following:

According to NA.3.1 (3) in Eurocode 8-5 (2014), the material / partial factors on soil strength are required for seismic slope stability are:

For total stress analysis: $\boldsymbol{\gamma_{cu} = 1.1}$ for clays

For effective stress analysis: $\boldsymbol{\gamma_{a\phi} = 1.2}$ for fill materials and **1.1** for other materials

In such cases where above given safety factor criteria for post-earthquake stability is not satisfied, stability of slopes can be verified by dynamic analysis. Safety requirements for dynamic analysis are given in term of maximum allowable permanent shear strains $\boldsymbol{\gamma_p}$ which in this case is **3%** ref. /19/.

According to NA.4.1.4 (11) in Eurocode 8-5 (2014) will parameter $\boldsymbol{\lambda = 0.80}$ for saturated cohesionless materials that may get into the liquefaction. This corresponds to a requirement for safety / material factor of 1.25 against such behavior could occur.

9. Foundations in shallow water and on land

This section applies to fillings in water (sea) and on the land on south and north side of main bridge.

9.1 Road filling in shallow water

Road fillings in water (sea) are classified in geotechnical category 3 according to handbook N200. Maximum slope of filling in the water is dependent upon stone quality, stone size and method of filling given that filling is founded on competent ground. Maximum slope for underwater filling given according to figure 2-3-3 in handbook V221 is 1:1,3 (for water depth upto 10 m). For filling in water depth more than 10 m, slope of filling shall not be steeper than 1:1,8. Slope angle shall be verified by stability analysis.

9.1.1 Stability of filling

Stability of filling in sea shall be documented by stability analysis.

Geotechnical category 3 with CC3/RC3 for material with dilatant behavior sets partial factor γ_M 1.4 for effective stress analysis. This applies to stability of filling in water and on land.

9.1.2 Settlements in fillings

Maximum allowable settlements in longitudinal and cross-sectional direction for road filling shall be calculated according to section 205.1 in handbook N200. Fixed value in this case cannot be given as it depends upon the various variables.

Requirements for settlements for structure on land shall be considered according to section 11.1.7 handbook N400.

Settlements and time required for settlements for the fillings shall be calculated and documented. Creep settlements in the rockfill itself shall be considered.

9.2 Southern tower foundation

Design of tower foundation shall be carried out according to relevant Eurocodes.

Design shall be carried out for all relevant limit states for foundation design stated in Design basis – Bjørnafjorden floating bridge ref. /21/. Stability of foundation under seismic loading shall also be checked.

9.3 Northern Abutment

Design of tower foundation shall be carried out according to relevant Eurocodes.

Design shall be carried out for all relevant limit states for foundation design stated in Design basis – Bjørnafjorden floating bridge ref. /21/. Stability of foundation under seismic loading shall also be checked.

Design of required permanent and temporary anchoring for abutment caisson shall be accordance with chapter 8 in Eurocode 7 NS-EN 1997-1:2004+A1:2013+NA:2016. Partial factors for materials, resistances and creep limit criteria according to national annexure NA.A.6. Anchors shall be designed to withstand both static and dynamic loads.

Loss of post-tensioning due to creep in steel material and/or between grout and rock material shall be taken into account.

10. References

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