REPORT

BJØRNAFJORD SUBMERGED FLOATING TUBE BRIDGE

ENGINEERING GEOLOGY EVALUATIONS

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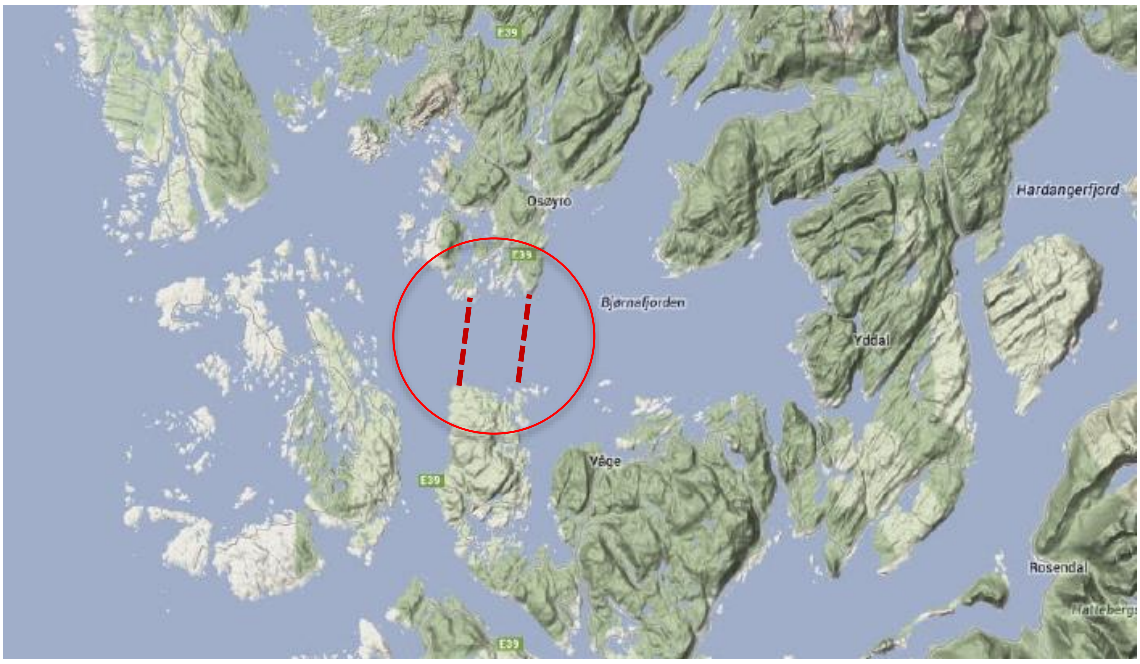
1 INTRODUCTION

1.1 Project description

The Norwegian Public Roads Administration (NPRA) has been commissioned by the Norwegian Ministry of Transport and Communications to develop plans for a ferry free coastal highway (E39) between Kristiansand and Trondheim. The 1 100 km long coastal corridor comprise today 8 ferry connections, most of them are wide and deep fjord crossings that will require massive investments and longer spanning structures than previously installed in Norway. For these fjords the Submerged Floating Tube Bridge (SFTB) is regarded as an attractive crossing solution. The applicability of the SFTB technology has recently been proven in a feasibility study for the 3.7 km wide and 1 300 m deep Sognefjord.

Following the Sognefjord study, considered the most difficult and challenging fjord to cross, crossing studies are initiated for other ferry connections along the route. One of the crossings for which a fixed link is to be assessed is the 5 km wide Bjørnafjord between Reksteren and Os as part of development plan for the E39 Aksdal – Bergen section. NPRA pursues the development of a permeant link over the Bjørnafjord through parallel studies comprising both floating and submerged floating bridge concepts. The assessment study for the SFTB is carried out by the design group REINERTSEN – Olav Olsen – Norconsult et al.

The objective of the SFTB assessment study is to consider both pontoon and tether stabilized SFTB solutions for two different crossing trajectories as identified by NPRA.



> *Figure 1 Relevant routes for crossing the Bjørnafjord between Reksteren and Os*

1.2 Objective and scope

This report sums up the available geological background material and results from ground investigations performed so far. Further, engineering geological challenges are discussed and particular further ground investigations are recommended.

2 GEOLOGY

2.1 Regional geological setting

The onshore areas at the southern connection, at Svarvhella on the island Reksteren, and at the northern connection, on the island Røtinga, are dominated by magmatic rock types described as:

- Reksteren (south)
  Greyish monzogranite, coarse grained, local occurrence with amphibole, some areas with red granite with biotite (*Håkre monzogranite*), as well as some dike intrusions of greenstone and basaltic lava with pillow structure.

- Røtinga (north)
  "Trondhjemite, tonalite and quartz-diorite"

However, the geological map indicates a boundary right at the shoreline of the island Reksteren, as can be seen on the smaller island of Svarvehelleholmen, just off the northern shore of Reksteren. The regional geology suggests that the seabed along the strait is composed of more complex geological structures, including:

- Green polymictic conglomerates, with lumps of gabbro greenstone, granitoids, diorite, and partly also marble and phyllite or greenish greywacke

- Mica schist, generally without garnet and amphibole, penetrated by tonalitic dikes (*Skotavik formation*)

- Conglomerates with quartz and quartzites

- Metamorphic sandstone

- Greenstone/basalt with pillow structure (*Gullfjell complex*)

2.2 Engineering geological mapping

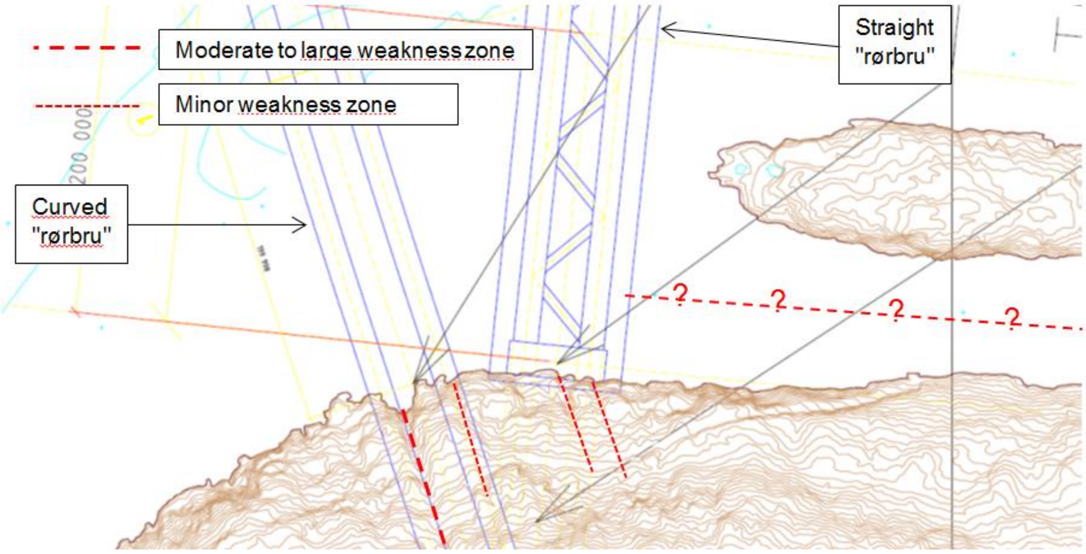
18th March 2015 a site visit and engineering geological mapping of available areas was carried out by Norconsult. The main observations and measurements from this work are summarized in the following. Reference is also made to the fieldwork report "*Engineering geological mapping of landfall areas*" dated 2015-03-26 (Appendix 1).

2.2.1 Landfall south (Reksteren)

The geology of the landfall area is composed mainly of a greyish, coarse grained monzogranite with local occurrence of amphibole and some areas with biotite-rich red granite. Just off the shore a narrow dike intrusion of greenstone and basaltic lava with pillow structure is present. Beyond this it is expected that the sea bed is composed of green polymictic conglomerates and phyllite, as can be observed on the small island Svarhelleholmen just off the northern shore of Reksteren.

The "landfall", i.e. the interface between submerged tube bridge and hard rock tunnel is planned to be at about 30 m depth off shore. As the geological mapping reveals a distinct boundary just along the shore line of Reksteren, it is expected that the landfall will be in geological setting hidden below the water, but expected to be similar to the basalt visible at the very edge of the northern shore of Reksteren, and to the phyllitic type rocks present at Svarvehelleholmen.

Some local weakness zones are indicated on Figure 2 below. These are observed within the granitic rock on Reksteren, thus it is not certain that these protrude in the same direction under water, but with the present information there is no reason not to expect this.



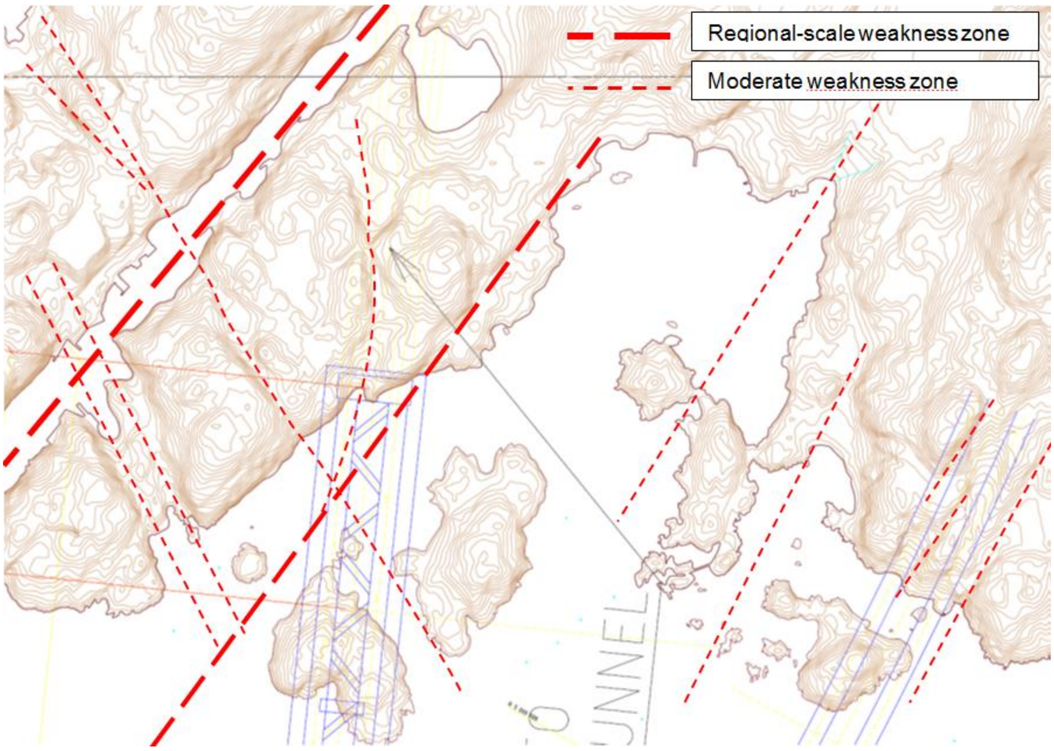
> *Figure 2 Topography and main weakness zones/lineaments at the southern landfall area.*

2.2.2 Landfall north (Røtinga)

The northern landfall area (Figure 3) consists of three main rock types:

- Easternmost (including the islands Kobbholmane and Fløholmen) a foliated, garnet bearing trondhjemite
- West of the trondhjemite, there is an approximately 100 m wide belt consisting of greenstone and basaltic lava with pillow structure (similar formation as visible on the northern shore at Reksteren)
- To the west the dominating rock type is conglomerate of the same formation as the one observed at Svarvehelleholmen.

Local weakness zones, and some assumed more regional zones, observed during the field mapping and from the study of topographical maps and satellite images, are included on the map in Figure 3.

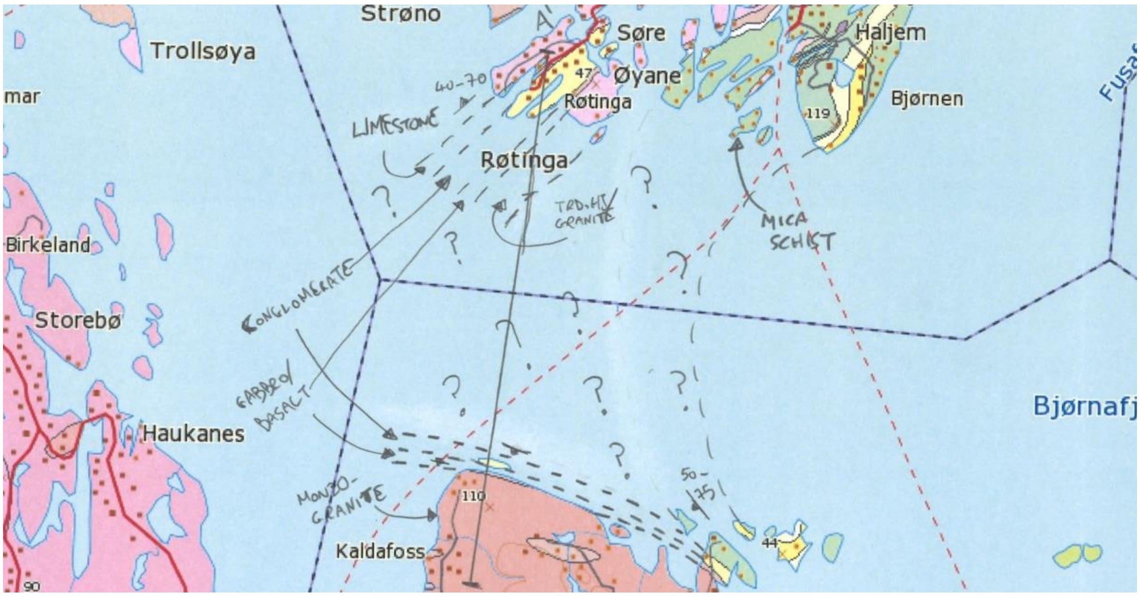


> *Figure 3 Topography and main weakness zones/lineaments at the northern landfall area.*

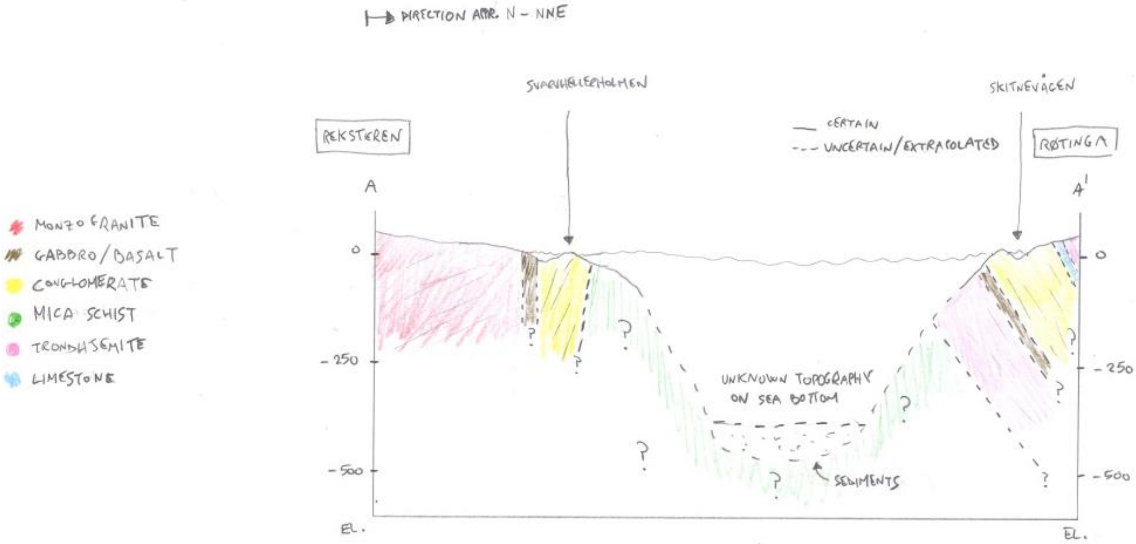
2.2.3 Conditions along the bridge alignment

The geological conditions along the bridge alignment(s) are unknown. A study of the geological setting on shore of surrounding islands and mainland may give some indication of the general conditions that may be expected, but the setting is too complex to indicate this with any certainty. Anticipated rock formations on the sea bed along the alignment are interpreted in Figure 4 and Figure 5 below.

More information on the geological formations along the seabed could be obtained through a detailed survey of the seabed, including core drillholes to collect rock samples. For the purposes of this project though, this type of information is not considered to be important enough to justify the cost of exploration – thus such investigations are not recommended at this point.



> *Figure 4 Anticipated seabed geology, interpreted along the strait crossing.*



> *Figure 5 Anticipated seabed geology, interpreted along a section of the strait crossing.*

2.2.4 Sediments

On shore there are no, or only insignificant sediments/loose material present.

Along the shoreline, in shallow water, it is expected that beach/shore sediments are present in the form of sand, mud and organic deposits, generally in the order of less than 5 m. However, in some local trenches the thickness may be more, maybe up to 10-20 m.

Further out on medium to deep water the types and thickness of sediments is less predictable. A survey of acoustic profiling (reflection seismic) has been performed at an earlier stage indicating the general conditions. However, this survey includes somewhat simplified data on the conditions, particularly where the topography is steep (due to limitations of the survey method).

In the deepest part of the strait the available survey indicates sediment thickness up to over 50 m, typical of deeper water.

More details on the available information on sediment thickness is discussed in the Geotechnical Design Report (KTR 6.2). Further, reference is made to the survey reports listed in section 2.3.

2.3 Available ground investigations

Acoustic profiling
An acoustic survey by "sparker" (reflection seismic survey) was performed along the strait crossing area in 2012; "*Bjørnafjorden – Bruforbindelse, Løsmassekartlegging for vurdering av ankringsforhold", GeoMap March 2012*.

Another series of "boomer" profiling in the shallow waters near the shoreline was performed in 2014; "*E39 state municipal plan Aksdal-Bergen, Acoustic profiling with boomer to map bedrock horizon", GeoMap November 2014*.

Test samples of clay material
A total of 4 sample series from the clay from the sediments in the deep part of the fjord have been recovered and tested.

More details on this can be found in the Geotechnical Design Report, ref. [1]. Reference is also made to the test report; *"Bjørnafjorden, Bunn- og grunnundersøkelser", Multiconsult June 2012*.

3 ENGINEERING GEOLOGICAL EVALUATIONS

3.1 Landfall south (Reksteren)

3.1.1 Tunnel on land

The tunnel on land is not part of the scope for this design team. However, the following evaluations have been made:

The hard rock tunnel on the island Reksteren is expected to pass through igneous rock types, observed (in the northern parts of the tunnel) to be of fair to good quality.

Some weakness zones striking near parallel to the alignment of the tunnel may influence the tunnel. Particularly when approaching the shoreline, these zones and/or the affected surrounding rock mass may be expected to have a higher potential for leakage of seawater. The alignment of the tunnel probably leaves little flexibility for avoiding these zones, and it is anyhow expected that several similar zones with the same trend may be found on either side of the planned alignment.

The conditions for tunnelling are generally considered to be good.

3.1.2 Underwater ditch

The main solution for establishing a connection point will be to excavate a trench in the shallow water towards land until a water depth of about 10 m. At this point a near vertical rock wall shall be excavated where the piercing from the hard rock tunnel will come out. The tube bridge shall be installed in the ditch and later completely or partly backfilled.

Excavation of such a ditch will, due to the rather large dimensions of the installations, be an extensive task. Subsea blasting and dredging is a relatively time-consuming operation, but to the relevant water depths for this ditch may be performed by relatively common and well proven technology. The length of the dich depends on the direction of the tube alignment (different alternatives) but may be in the range 150 – 200 m (based on available bathymetrical data). Onwards the sea bed falls rapidly and steeply down to water depths in the range 250 – 500 m.

The geological conditions for subsea excavation of the ditch at the southern connection point is considered to be good although the relatively open water may render work challenging at times with less favourable weather/wind-conditions. It is considered more favourable to locate this work as far to the east as possible, where the partial shelter of Svarvehelleholmen may provide calmer water. With respect to this, it is considered that the eastern landfall solution (i.e. the straight bridge alignment) may prove to be marginally favourable.

3.1.3 Submerged tunnel piercing

The detail design of a submerged piercing of the hard rock tunnel through to the ocean bed (prepared rock wall in the excavated ditch) is not considered part of the scope of the bridge designer. However, it is evident that the solutions must be developed in close cooperation between the tube bridge and the tunnel designers.

For now the evaluation of this element is limited to a consideration that this solution, though challenging, is feasible within common practice and proven technology.

3.1.4 Foundations / anchoring

The tube bridge at the southern connection will need horizontal anchoring (in addition to vertical foundation/anchoring). The rock mass is expected to be generally fair to good, allowing sufficient anchoring capacity.

There are several possible solutions to obtain the necessary anchoring. However, as a hard rock tunnel shall be excavated up to the point of the connection, it is expected that a solution where anchor chambers, excavated from inside the tunnel, will be favourable. Such solution may include one or several chambers at both sides of the tunnel, and a drillholes towards the sea. Cables may be pulled through holes into the chambers, and connected to a wedge or a hook, and later the complete chamber and hole may be completely grouted, for permanent anchoring and corrosion protection.

3.2 Landfall North (Røtinga)

3.2.1 Tunnel on land

The tunnel on land is not part of the scope of this design team. However, the following evaluations have been made:

The tunnel will pass several weakness zones, presumably of more regional character. The crossing of some of these will have a rather low angle, and it is expected that the tunnelling conditions close to and through some of these zones will be challenging.

The tunnel will also cross several less predominant zones. As the tunnel will be passing below sea level, and partly under water some challenges related to permeability should be expected. As for the southern connection, it is also here expected that similar zones appear on either side of the planned alignment, thus there are no other areas that stand out as significantly better or worse alternatives than those planned.

3.2.2 Underwater ditch

As for the southern connection point, a trench shall be excavated in the shallow water towards land until a water depth of about 10 m.

The area of shallow water depth is larger at the northern connection, thus the excavation of a trench will be more extensive at the northern end of the tube bridge. Based on current bathymetrical data, the extent of such a trench may be in excess of 250 m. Beyond this point, the water depth remains shallow, but slightly deeper than the lower limit of the tube construction. Onwards another 300 m the subsea hill top of "Flua" is reached, where a gravitational foundation and fixation point for the tube construction is planned.

The geological conditions for subsea excavation of the ditch at the northern connection are expected to be good, although the relatively open water may render work to be challenging at times of less favourable weather/wind-conditions. It is not expected that there will be any significant difference in either geology or other conditions between the different alternative alignments presented so far.

3.2.3 Submerged tunnel piercing

Similar considerations as for the submerged piercing at the southern landfall are also valid at the northern connection.

3.2.4 Foundations / anchoring

A similar solution for horizontal anchoring, as for the southern connection may be considered. The method and conditions for anchoring will also be similar.

3.3 Flua

At "Flua" it is planned to establish a gravitational foundation where the tube construction may be fixated both vertically and horizontally.

Different alignments, thus locations of the anchoring point at Flua are being considered. There exists little detailed geological information of the sea bed at Flua, but generally the planned solution is considered feasible and not very sensitive to varying geological conditions within the expected range.

Available bathymetrical information indicate that the water to the east, south and north of Flua remains rather shallow, while it falls rather steeply to the west. As the current alignments for the tube favour locations towards the west, it is important to perform more detailed bathymetrical mapping in this area, as well as evaluation of rock mass and sediments with regards to stability of a potential rock fill foundation in this western slope.

If the final location stays to the west of Flua (current plan), it is anticipated that the works in the area may include rock fill and possibly some dredging. A more eastern location will probably also necessitate some extent of subsea blasting.

3.4 Subsea operations at shallow and deep water

Along the middle stretch of the tube bridge the bridge will be anchored in either "hanging" floating pontoons or anchored down to the sea bed. The latter option necessitates sea-bed anchors at regular intervals.

On rock surface (or in areas of limited overburden) such anchor points at the sea bed may be comprised of gravitational anchors or tensional anchors, or a combination of these. In areas of thick layers of sediments there may also be an option of suction anchors (this option is discussed in the geotechnical design report (12149-OO-R012)

Gravitational foundation may be placed on a stable and fairly flat surface (of necessary dimensions/extent). However, in the expected steep topography between the shallow waters and the deep sea it may be necessary to prepare a suitable flat area by either filling or blasting (or a combination).

Tensile anchors must involve some form of drillhole where an anchor may be installed. Performing such operations at great water depths will render any solution challenging (at least expensive and time consuming) potentially with some risk of unsuccessful results.

3.4.1 Blasting in deep water

Subsea blasting is in principle performed in the same manner as on dry land. The process involves drilling of a hole - insertion of explosives with detonators - detonation. The main differences below water involve the following:

- Drilling operations
  - From above water line on a rock fill; down to 5 m depth
  - Platform standing on the sea bed; down to 15 – 20 m

- From a floating vessel; difficult operation, but can possibly be an alternative down to 30-40 m depth
- Divers with handheld equipment; conventionally down to 30 m, more advanced diving down to 45 m. (deeper diving is possible, but must be considered by specialist companies)
- Remotely operated machinery with drilling equipment; modified equipment may be designed to operate at least to 500 m depth

- Explosives, detonators and connections must be water proof to the relevant pressures for a suitable amount of time
  - Conventional explosives and detonators;
    low risk down to 10 m, is regularly used down to 20m
  - Special (but conventional) explosives; down to 40 - 50 m
  - Very special or specifically designed explosives; down to at least 450 m (For example *OWE80* (oil well explosives) with a rating of 48 bar for 24 hours)

- Necessary specific charge (amount of explosives pr. m^3 rock to be blasted) increases significantly with increasing water pressure.
  A method for estimation of necessary charge (Q) is as follows (*reference: Stig O Olofson*):
  - Q = 1,0 + 0,01 x [*water depth*] + 0,02 x [*sediment thickness*] + 0,03 x [*bench height*]

- At 350 m depth with a excavated height of 5 m (no sediments) this formula would suggest 4.65 kg/m^3

- Above mention OWE80 is delivered as standard in 1 ¼" cartridge - if this dimension explosive was to be used one would in this case require a drillhole pattern of less than ½ m hole spacing.

- More realistically one would drill a larger hole and; either use larger dimension explosive cartridges or re-pack explosives in plastic tubes to the desired dimensions.
  Considering 4" drillholes (which is a rather large dimension drillhole), one would in the same case as calculated above require a drillhole spacing of about 1.5 m

- Dredging of sediments and blasted rock may be performed by:
  - Mechanical excavators; limited water depth (2 – 3 m)
  - Lowering a dredge from a platform of floating vessel; possibly lower limits in the order of 30 m
  - Subsea suction equipment; conventional equipment that can operate down to more than 1000 m

3.4.2 Grouted piles/anchors

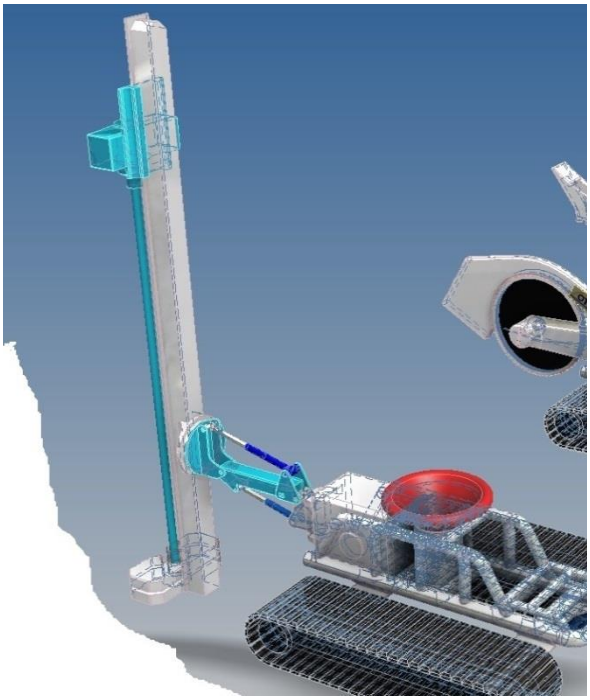
Anchoring through drilled and grouted piles or anchors is a well proven technology on dry land. Performing the same under water should in principle be an equally sound solution, however the underwater operation will make this more complicated.

There is conventional equipment available for performing drilling operations. However, no reference of such work for similar purpose and with comparable capacities has been found. It is considered feasible, at least in fairly shallow waters (down to 100 m), though it is probable that special solutions may need to be developed or modified for the particular requirements for this project. At greater water depths similar methodology should in principle be applicable, though higher water pressures will lead to more costly equipment and probably more time consuming procedures for the works.

In principle there are 3 different methods for drilling the hole for the anchor:

- Drilling from a surface drill rig (floating or standing on the sea bed)
- Drilling from a subsea template that is lowered in position from a floating vessel
- Drilling from a remotely operated machine, with some sort of self-propulsion system

A principal sketch of the latter is shown in Figure 6.



> *Figure 6 Principle sketch of subsea machinery for drilling and dredging.*

Further, it is considered that a requirement for this type of anchoring would need to be the possibility of non-destructive quality testing of each anchor (pull-test) – which to the relevant capacities necessary for this project may be a rather challenging operation in itself.

4 PRELIMINARY EVALUATION OF FEASIBILITY AND RISK

The main operations relevant for the engineering geological study/design and preliminary evaluation of feasibility and risk are discussed below. Also qualitative considerations of time and cost are mentioned.

4.1 Feasibility and risk

Preparation of ditch in landfall area

Preparation of ditch involves subsea blasting and dredging of sediments and blasted rock, down to depths in the order of 50 m. Generally, the methods for performing such works are well known and proven, thus the operations are considered feasible with a relatively low risk. However, the large dimensions of the excavation works will require a rather extensive operation that will be both costly and time consuming.

The stability of the cuts during and after excavation is a special element with regards to work safety during construction. Some large scale mechanical scaling may possibly be performed by machinery from rafts or work platforms, but it is not considered feasible to install traditional rock support under water (at these depths). Thus it is generally considered that the necessary safety is achieved by other means – generally through limited use of personnel (divers) in potential problem areas.

Anchoring/foundation

Anchoring may be achieved through drilled and grouted tensile anchors, or by gravitational anchors.

Tensile anchors involve drilling a hole of necessary depth and diameter – installing anchor – grouting of anchor. Further, it is expected that there will be a requirement for pull testing to at least the final load.
Gravitational anchors may be completely or partially pre-fabricated - and lowered in place on a pre-prepared foundation area. A "box"-construction may be filled by ballast rock after installation, in order to achieve the required weight of the anchor.

Shallow waters

Drilling and installation of tensile anchors is considered feasible at "shallow" waters. However, due to the very high loads it is considered that the required capacity of the anchor may be on the limit or beyond conventional available equipment and materials for this type of anchoring. Though considered feasible, it is expected that there is some level of risk involved with regards to the availability (or development) of equipment and materials. Further, the testing procedures for the installed anchors will also be challenging, and little experience with similar pull test on grouted anchors exist.

On the other hand, gravitational anchoring is a well-known method, and it is expected that the necessary capacities may be achieved, given sufficient dimensions of a suitable and stable foundation for the anchor. Thus, the anchor itself is considered to include a relatively low risk, and what remains to be determined is whether suitable and sufficiently large stable and "flat" areas can be found or created at the necessary anchoring points. It is expected that the answer to this last question may be answered through continued survey and ground investigations.

Steep topography
Unless a natural "shelf" at an appropriate location may be found it will be challenging to establish a stable and sufficiently large foundation area in areas with steep topography. In such areas it is expected to be necessary to perform extensive excavation through blasting and dredging – possibly also in combination with filling.

As discussed in section 3.4.1, it is expected that the high water pressure will require very large amount of explosives – thus the operation of drilling holes for the blasting will be very extensive. Further, explosives and detonators will usually be certified to withstand a certain pressure for a limited amount of time. Due to the expected time consuming process of charging holes under water (at great depths) it is expected that the maximum size of each blast will be very limited.

Though considered feasible, blasting at greater water depths is expected to be very costly and time consuming. Also worth mentioning is that the high amount of explosives required will result in a very large safety radius, for moats, swimmers, fish and marine mammals.

The creation of an anchor point in steep areas must also consider the stability of the slope above/around the foundation and the foundation-shelf itself. From experience it is known that evaluation of the stability of natural slopes and blasted cuts under water is very difficult and the conclusions are uncertain. Thus, there will always be a significant risk with concern to this. The best solution for design would be to consider as large dimensions of the foundation/anchor construction as possible, to limit the damage a potential slide from above may cause on the construction. With regards to stability of the shelf where the foundation is placed, the best solution may be either to find a sufficiently stable, large and suitable location, or to create the same through "over excavation" of the shelf.

All in all it is expected that anchor points in the steep topography will be a challenging task for both design and construction and include a considerable risk.

At great depths
At greater depths the available bathymetrical data indicate that the topography is more level. Though these data must be verified by more detailed survey, along with evaluation of sediment types and thickness, it is expected that conditions for foundation of anchors will be favourable compared to the steep sections.

If the sediment thickness is not very significant, the same considerations as for the "shallow waters" are expected to be valid – with the exception that all works will be performed from floating vessels and/or remotely operated vehicles/machinery.

The risk for the methodology is also considered comparable with the ones at "shallow water", although one would naturally expect that the risk is somewhat higher, due to the fact that all operation must be performed and observed through remote operation.

If the sediment thickness is significant suitable foundation/anchoring is discussed in the Geotechnical Design Report (12149-OO-R-012).

Connection and horizontal anchoring of tube bridge
One of the main challenges in this area is that the connection point will need to be water proof. It is considered feasible to design a solution where the actual connection will be sufficiently water tight but there is a risk that water may leak through fractures in the rock mass surrounding the hard rock tunnel.

The natural permeability of the rock mass is one element that must be handled. However, the extensive blasting works for preparation of the ditch and the final piercing of the rock tunnel through to the sea will possibly create new fractures in the rock mass. Sealing of the rock mass may be performed prior to the establishment of the ditch and tunnel – but due to the continued blasting one must also expect some extent of sealing after the excavation works.

Pre-grouting of the rock mass, for performing such sealing, is well-known and proven method; however, the necessary extent of the work remains uncertain. Though this work does include some risk it is considered that this will be minor compared to the risk of the other main elements of this project.

4.2 Preliminary recommendations

The main challenge related to engineering geology is considered to be the construction of anchoring of the tube. The challenges will be present along the entire alignment but the main issues are expected to be:

1. Anchor points in the steep slopes leading down to the deep waters
    a. Risk related to slope stability above anchor point
    b. Risk related to stability of the "shelf" at the anchor point
    c. Complicated subsea operations in the range 100 – 400 m water depth
2. Anchor points at great water depths (where topography requires preparation of the surface)
    a. Complicated subsea operations in the range 300 – 500 m water depth

Regarding the first issue, a main recommendation is to try to avoid anchor points within this steep slope. A study of maximum spacing between anchor points will need to be performed in order to limit the number of anchor points in these areas, or avoid it completely.

For both issues, with respect to feasibility and risk, it is generally considered favourable to avoid drilling operations. This means that gravitational anchors would be more favourable than tensile anchors, and further that the need for surface preparation that involves excavation of rock should be limited or avoided. The latter requires that suitable level areas may be found within the limits of the required position for anchor points.

Further, it is also generally considered that gravitational anchors may be favourable as this method allows the risk of insufficient anchoring capacity (or degradation of capacity over time) to be eliminated through the design of the anchor construction.

5 RECOMMENDED FURTHER STUDIES

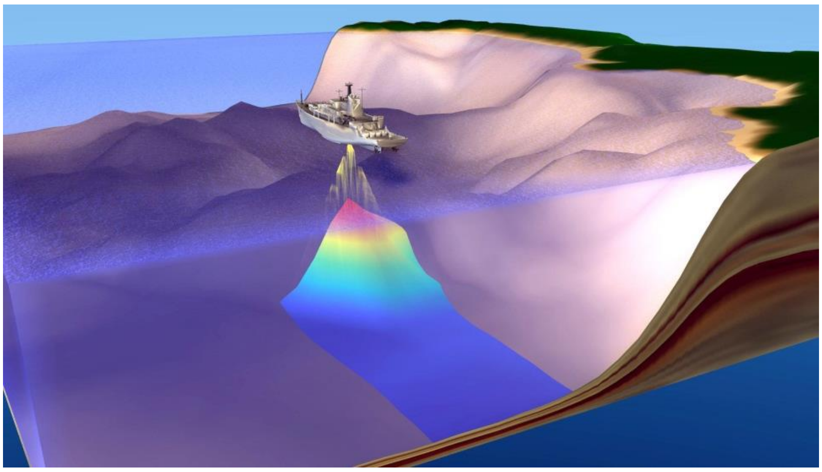
5.1 Ground investigations

5.1.1 Topographic survey

Detailed information of the sea-bed topography is required for evaluation of possible and suitable methods for anchoring along the tube-bridge alignment. Detailed survey of the sea-bed is easy to perform with a fairly high resolution and degree of accuracy.

A 3-dimensional terrain model of the sea-bed should be established along a corridor below the bridge alignment (all alternative alignments), with special attention to high resolution data at the planned locations of the anchoring points. Such a model will allow for a better understanding of the sea-bed topography, and to a certain extent also the conditions of rock outcrops, and types and expected thickness of sediments. The model may also reveal structures that may influence the stability of the ground at the anchoring points.

Specialists within marine surveying may advise on optimal methods for surveying. It is expected that an advanced multi-beam echo sounding may be a suitable method to obtain or supplement data for a detailed 3D terrain model. A principal sketch of such survey method is shown in Figure 7.



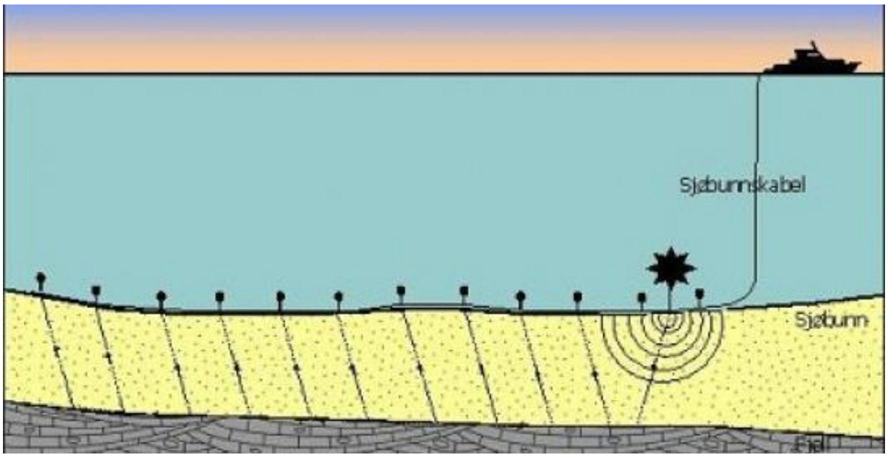
> *Figure 7 Principle sketch of subsea topographical survey by multi-beam echo sounding.*

5.1.2 Thickness of sediments

Some data on the thickness of sediments exist from preliminary acoustic profiling (reflection seismic survey). These investigations indicate that in the shallow areas close to shore and the steep sections where the water depth increase down to 3-400 m, there are little or no sediments, while in the deep sections in the middle of the strait, the thickness of sediments may in some area be more than 50 m.

This type of acoustic profiling is an efficient way of mapping, as large areas may be covered in relatively short time and at a moderate cost. However, a downside is that the signals for the survey have a relatively large scatter, thus the resolution will suffer in deeper waters, and there are particular challenges related to side reflections in steep terrain. The latter is indeed also the case for the existing survey, as it is indicated on the plots that the data is inaccurate in some areas due to side reflections. The existing mapping is considered very useful for evaluation of concepts and planning of further investigations. For further detailing of the design principles it is recommended to perform more detailed and accurate mapping and investigations along the final alignment(s), particularly at the locations of anchoring.

More detailed survey of the local areas for foundations/anchoring may possibly be done by refraction seismic survey. A limitation for this survey method is however the water depth. It is assumed that practical limitations allow for this type of survey to be performed down to about 300 m. In this case, the northern 1/3 of the alignment may probably be surveyed by this method. In the south the topography is quite steep and the water depth increases to over 300 m after a very short distance from the shore. It is however assumed that the conditions for foundations/anchoring within this steep section, if applicable, may be investigated by refraction seismic. A principal sketch of subsea refraction seismic survey is shown in Figure 8.



> *Figure 8 Principle sketch of sea bed refraction seismic survey.*

5.1.3 Rock mass quality

In areas where the anchor point falls in rocky terrain, it will be necessary to collect information on detailed 3D topography. Depending on the topography, it may also be necessary to collect information on geology; rock mass quality and amount and quality of any loose sediments. In particular it is of interest to detect any weakness zones in the rock mass at the anchor point or in the near vicinity where such conditions may affect the stability of the area. This is applicable whether the design includes grouted anchors in rock or gravitational anchors.

Refraction seismic survey, discussed in the previous section, is considered a suitable method to also collect information on rock mass quality and as a tool to detect potential weakness zones in the rock mass. Information on both sediments (type and thickness) and rock mass quality is collected in the same operation by refraction seismic survey.

5.1.4 ROV video inspection

In due time it will probably be necessary, or at least useful, to perform a "visual" inspection of certain key areas. Such areas may be connection point for tube bridge/rock tunnel, areas for foundation points and particular areas in steep topography where slope stability may be an issue.

In shallow waters such inspection may be performed by divers with video camera, probably down to 20-30 m depth. In deeper water (which will be the major parts of the project) inspection may be performed by ROV. Suitable equipment for such inspections is conventionally available and may be used down to at least 500 m depth.

6 REFERENCES

[1] REINERTSEN, Dr.techn Olav Olsen, Norconsult, et al. Bjørnafjorden Submerged Floating Tube Bridge – Report 12149-OO-R-012, Marine geotechnical design, Rev. 01.

To: Statens Vegvesen

From: Norconsult

Date: 2015-03-26

Engineering geological mapping of landfall areas
Fieldwork report

1. INTRODUCTION

As part of new a new highway connection between Bergen and Stavanger the Norwegian Public Road Administration (SVV) are studying different possibilities for crossing the Bjørnafjorden strait, replacing the current solution of ferries. Norconsult, Reinertsen and Dr. techn. Olav Olsen are examining a solution consisting of a submerged tube bridge (rørbru) connecting two rock tunnels on each side of the fjord. Both a curved and a straight tube bridge are considered.

In particular, Norconsult is responsible for the engineering geological aspects of the project. Engineering geological field mapping of the landfall areas was carried out on 18th March 2015 by Thomas K. Mathiesen and Erik Martinelli. The main observations and measurements from this work are summarized in the following.

All photographs from the fieldwork referred to in the text are presented in chapter 4, for the sake of readability. Rose-diagrams illustrating the jointing structure at each locality are presented in chapter 5.

2. LANDFALL SOUTH (REKSTEREN)

2.1 Overall rock type distribution

The northern part of the Reksteren peninsula consists mainly of a greyish, coarse grained monzogranite with local occurrence of amphibole and some areas with red, biotite-rich granite (Håkre monzogranite). Along the northern shore, a narrow dike intrusion of greenstone and basaltic lava with pillow structure is present. The small island Svarhelleholmen consists of green polymictic conglomerates, with lumps of gabbro greenstone, granitoids, diorites, and partly also marble and phyllitic or greenish greywacke.

A geological map of the southern landfall prepared by the Norwegian Geological Survey (NGU) is shown in Figure 1. Based on the observations in the field, the boundary between the monzogranite and the greenstone and basaltic lava intrusion seems to be closer to the shore than what appears on the map.

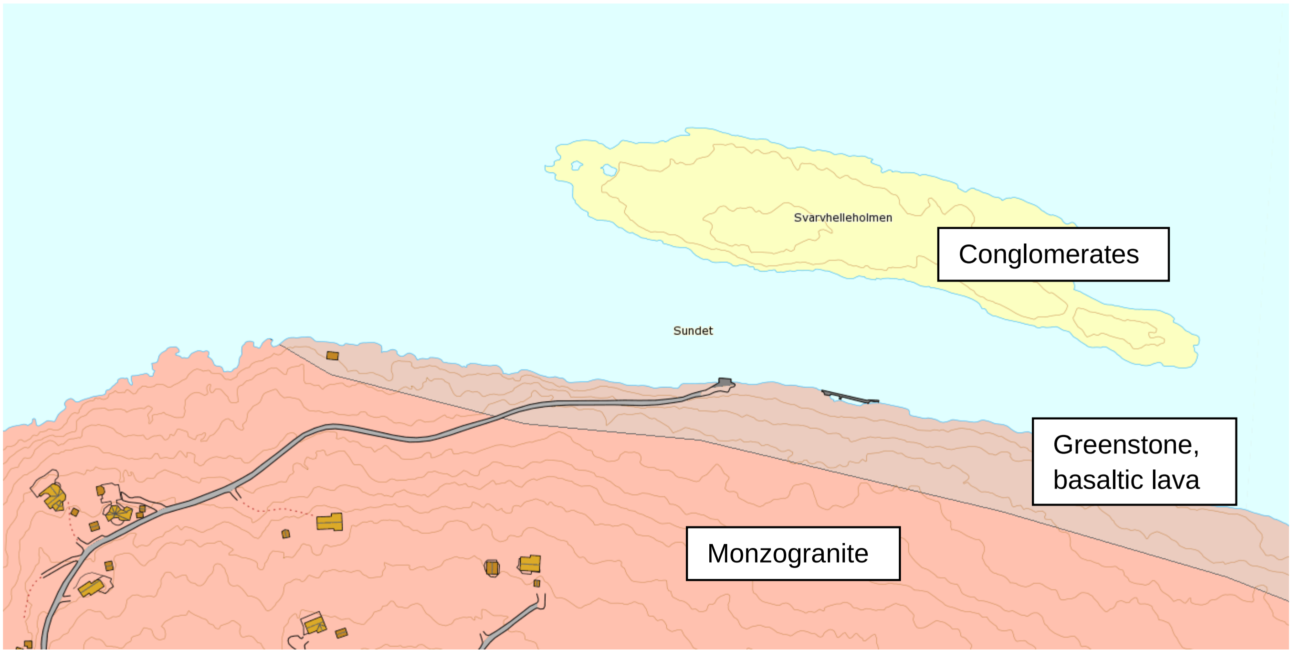


Figure 1 - Geological map of the southern landfall area

2.2 Geological mapping

Detailed geological mapping was carried out at the 4 localities shown in Figure 2.

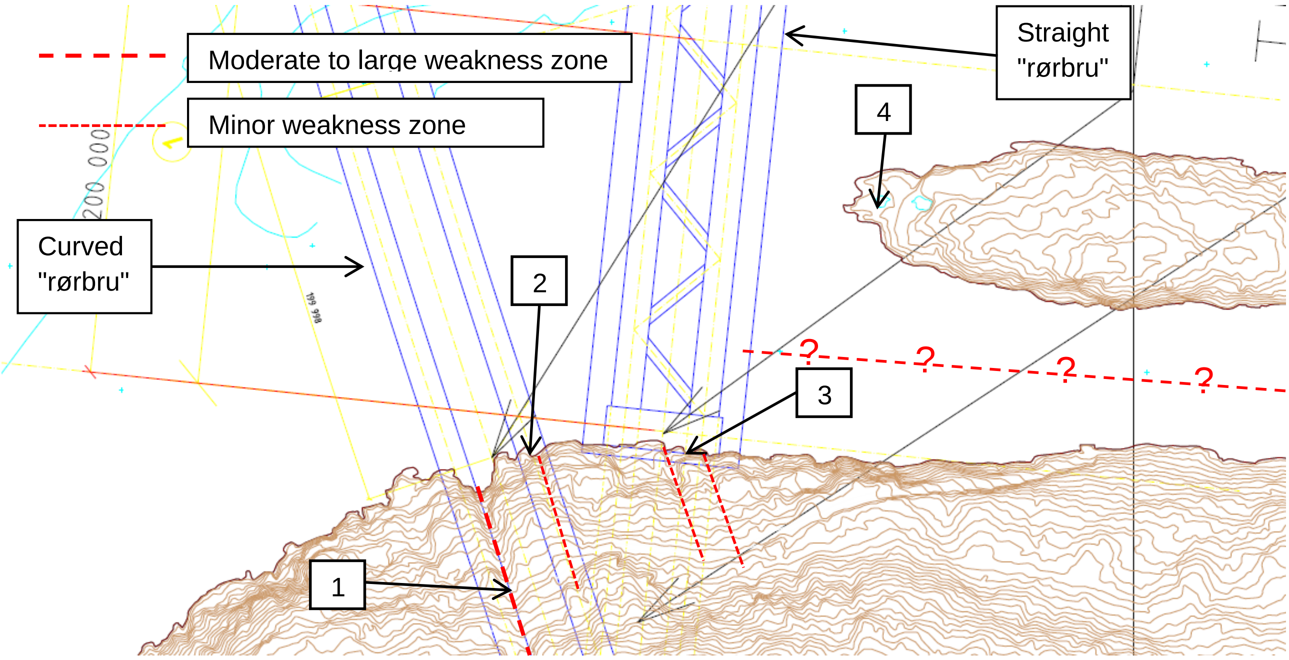


Figure 2 - The 4 locations for detailed geological mapping in the southern landfall area

1) Location 1 lies approximately between the tubes of the curved rørbru alternative, about 70 m away from the shore. The most pronounced joint sets can be described as follows:

Joint set nr.	Avg. dip/dip direction	Joint surface	Spacing	Trace length
1	70-80°/70°	Planar, rather smooth	0.5 m	> 10 m
2	10°/80-90°	Planar to slightly undulating, rather smooth	0.5 - 1 m	1 - 10 m
3	75-80°/150-160°	Planar, slightly rough and irregular	0.5 - 1 m	1 - 10 m
Sparse joints	90°/0°	Planar, rough	2 m	1 - 10 m

The dominating rock type is monzogranite of fairly good overall quality. The rock mass in general appears as blocky, with an average block size of about 0.25 m^3 (Figure 5). Higher degrees of jointing and surface weathering were observed in the first 1-2 m in the exposed rock mass. Below this depth, the rock mass appears as relatively fresh and unweathered.

2) Location 2 lies approximately in the path of the eastern tube of the curved rørbru alternative. The most pronounced joint sets can be described as follows:

Joint set nr.	Avg. dip/dip direction	Joint surface	Spacing	Trace length
1	80°/345°	Planar, slightly rough	2 m	> 10 m
2	80°/80°	Planar, rough	1 - 1.5 m	1 - 10 m
3	0-5°/330°	Planar, slightly rough	1.5 m	1 - 10 m

The dominating rock type is monzogranite. The greenstone/basalt intrusion emerges from the sea approximately about at this location (Figure 6).

3) Location 3 lies approximately in the path of the straight rørbru alternative. The most pronounced joint sets can be described as follows:

Joint set nr.	Avg. dip/dip direction	Joint surface	Spacing	Trace length
1	75-80°/100°	Planar, slightly rough and irregular	1.5 m	1 - 10 m
2	20-30°/150°	Slightly undulating, slightly rough	2 m	> 10 m
3	65°/345°	Planar, slightly rough	0.5 m	1 - 10 m

The dominating rock type is monzogranite, permeated by numerous syenitic dykes, up to 1 m wide (Figure 7). Joint frequency increases towards the sea. Greenstone/basalt is present on the first few meters of the shoreline.

4) Location 4 lies on the western end of the small island Svarhelleholmen, some 50-100 m east of the straight rørbru. The most pronounced joint sets can be described as follows:

Joint set nr.	Avg. dip/dip direction	Joint surface	Spacing	Trace length
1 (foliation)	70-80°/175°	Undulating, rough and weathered	0.2 - 0.3 m	1 - 10 m
2	90±10°/120°	Undulating, rough and weathered	1.5 m	1 - 10 m
3	50°/70°	Undulating, rough and weathered	1.5 m	> 10 m
4	0-20°/250°	Undulating, rough and weathered	0.5 - 1 m	< 1 m

The dominating rock type is conglomerate with defined schistosity along joint set 1 and several veins of quartz, syenite and greenstone, mainly striking in the same direction as the foliation (approximately E-W). The rock surface is heavily weathered/eroded, and has a bladdery texture (Figure 8).

2.3 Weakness zones

Based on map studies and observations in the field, the rock mass in the southern landfall area appears to be cut by some weakness zones (faults and/or zones with higher joint frequency or crushing), apparently dipping steeply towards ENE, as shown in Figure 2.

A 10-20 m wide depression runs from the bay where the curved rørbru is planned to hit the shore and into the peninsula striking about N160°E, along the same direction as the foliation planes. This indicates the possible presence of a persistent weakness zone. The core of such zones are usually found to be limited to less than one meter in width, but rock mass on either side is usually disturbed, with a higher degree of jointing, normally in the range of 5 – 10 m to each side of the core.

Other zones with high joint frequency and small depressions were observed. Most likely, these also represent weakness zones. Compared to the depression described above however, these seem to be of minor character, and it is difficult to exactly determine their continuation into the peninsula (it seems however reasonable to assume that they will follow the orientation of the assumed larger weakness zone, striking about N160°E). Only the most evident of such zones are indicated in the figure.

The increase in jointing frequency towards the sea observed at location 3 and the presence of the depression that separates the island Svarvhellerholmen from the Reksteren peninsula, indicate the presence of a fault and/or weakness zone somewhere in the straight between Svarvhellerholmen and Reksteren, possibly following the boundary between the greenstone/basalt intrusion and the conglomerates.

3. LANDFALL NORTH (RØTINGA)

3.1 Overall rock type distribution

The northern landfall area consists of three main rock types. Easternmost in the area (including the islands Kobbholmane and Fløholmen) a foliated, garnet bearing trondhjemite (a variety of tonalite enriched in oligoclase) represents the dominating rock type. West of the trondhjemite, there is an approximately 100 m wide belt consisting of greenstone and basaltic lava with pillow structure (the same formation visible on the shore at Reksteren). To the west of this belt, the dominating rock type is conglomerate of the same formation as the one observed at Svarvhelleholmen. A geological map of the northern landfall prepared by the Norwegian Geological Survey (NGU) is shown in Figure 3.

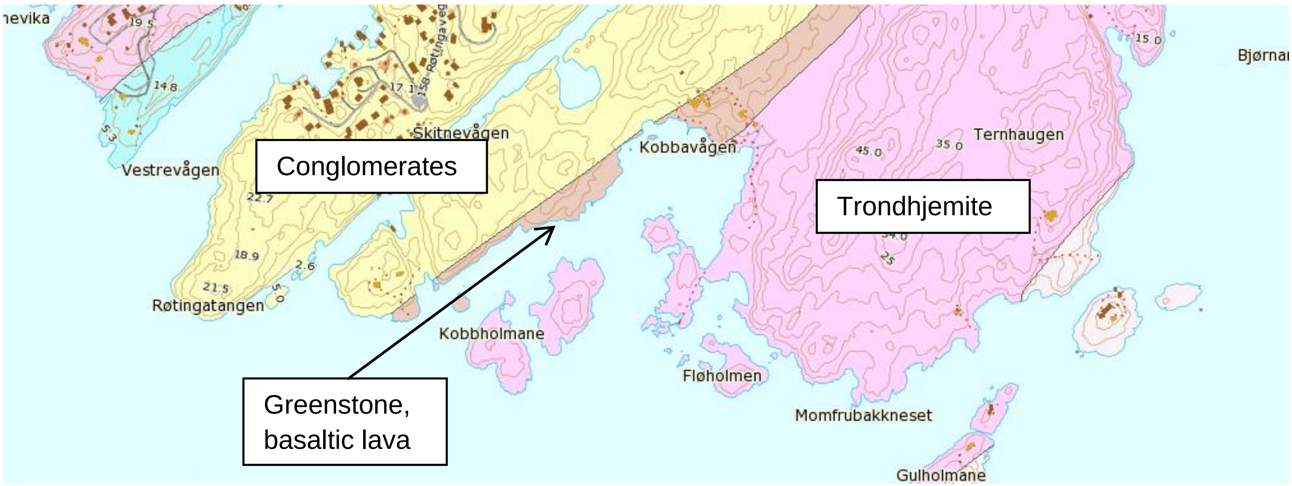


Figure 3 - Geological map of the northern landfall area

3.2 Geological mapping

Detailed geological mapping was carried out at the 7 localities shown in Figure 4.

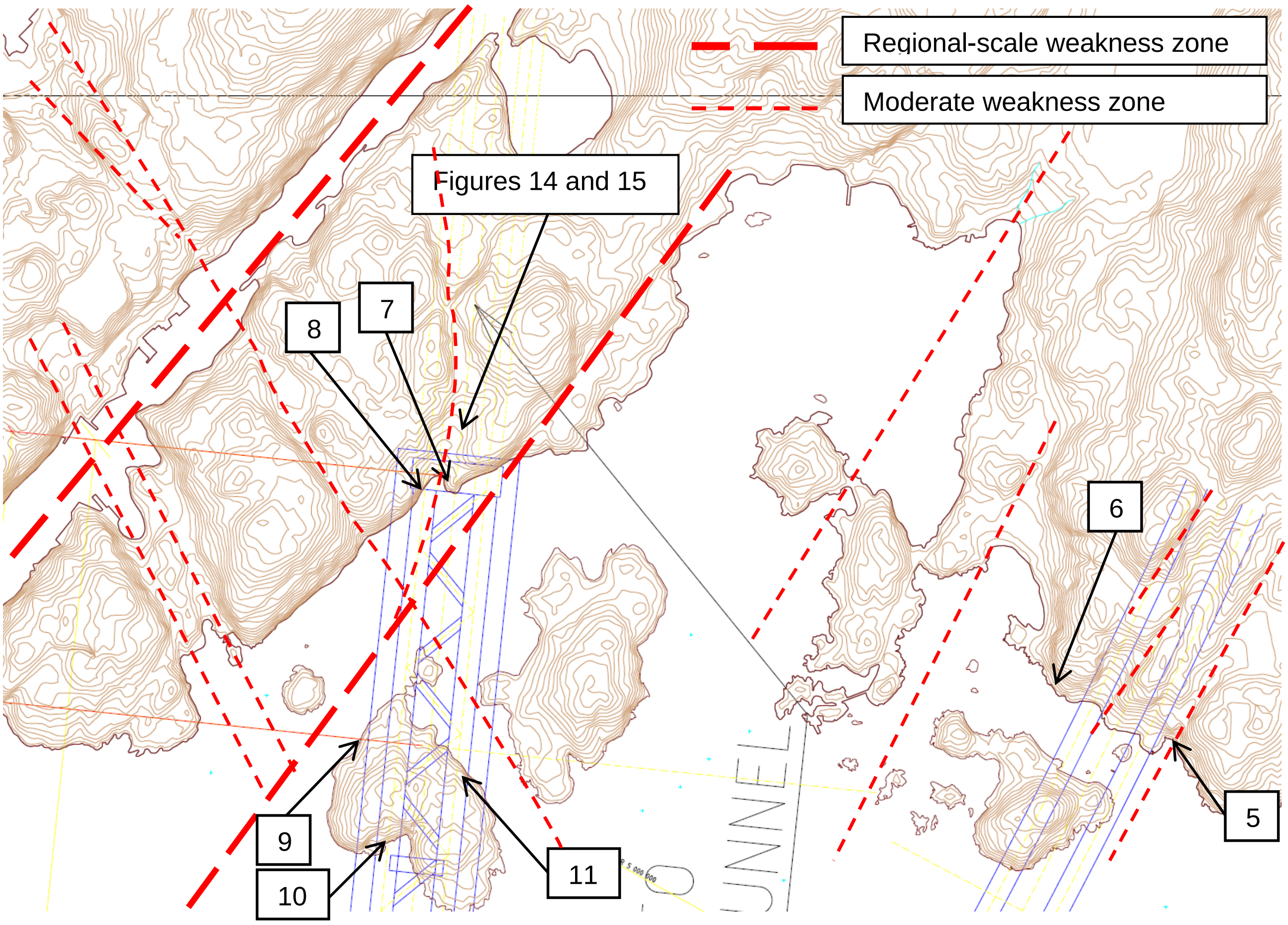


Figure 4 - The 7 locations for detailed geological mapping in the southern landfall area

5) Location 5 lies just east of the planned path for the curved rørbru. The most pronounced joint sets can be described as follows:

Joint set nr.	Avg. dip/dip direction	Joint surface	Spacing	Trace length
1 (foliation)	80-85°/300°	Planar, rough/irregular	1 m	> 10 m
2	45-55°/190-200°	Planar to slightly undulating, rough	0.5 - 1 m	1 - 10 m
3	55°/60°	Planar, slightly rough and irregular	0.5 - 1 m	1 - 10 m
4	35°/30°	Planar, rough to irregular	0.5 - 1 m	> 10 m

The rock mass at this point consists of trondhjemitic gneiss with a well-developed foliation along joint set 1 and good overall rock mass quality. A visible weakness zone runs through the bay east of this point and into the terrain (weakness zones are discussed in detail in chapter 3.3). The rock mass is significantly more densely jointed up to 15 m on each side of this zone (Figure 9).

6) Location 6 lies just west of the planned path for the curved rørbru. The most pronounced joint sets can be described as follows:

Joint set nr.	Avg. dip / dip	Joint surface	Spacing	Trace length
1 (foliation)	80°/315°	Plane to undulating, rough	0.5 m	1 - 10 m
1	80°/60°	Planar, slightly rough	0.5 - 1 m	1 - 10 m
3	30-40°/240-270°	Undulating, rough	0.3 - 0.6 m	1 - 10 m

The rock mass at this point consists of trondhjemitic gneiss (Figure 10).

7) Location 7 lies between the two tubes of the straight rørbru where these hit the shore at Kobbavågen. The most pronounced joint sets can be described as follows:

Joint set nr.	Avg. dip/dip direction	Joint surface	Spacing	Trace length
1	85°/310°	Undulating and folded, rough	0.05 - 0.2 m	1 - 10 m
2	90°/340°	Undulating, rough	2 m	1 - 10 m
3	70-80°/100°	Undulating, rough	1.5 m	1 - 10 m

This location is on the boundary between the greenstone and basalt intrusion and the conglomerates. The rock mass is heavily folded and metamorphosed, which makes it difficult to accurately measure the orientation of the joint sets (Figure 11). The values listed in the table above must therefore be considered as rather uncertain mean values. A weakness zone runs between location 7 and 8, striking approximately NE-SW.

8) Location 8 lies about 30 m west of location 7. The most pronounced joint sets can be described as follows:

Joint set nr.	Avg. dip/dip direction	Joint surface	Spacing	Trace length
1 (foliation)	75-80°/175°	Undulating, rough to irregular	0.5 m	1 - 10 m
2	70°/65-75°	Planar, smooth to irregular	0.5 m	1 - 10 m
3	55-60°/250°	Planar, irregular	2 m	1 - 10 m

The rock mass consists of phyllitic shale with intrusions of conglomerates, basalt and diabase. Like location 7, the jointing structure is irregular due to the heavy foliation and intrusions (Figure 12).

9) Location 9 lies on the north-western side of the small island Kobbholmane, in the path of the western tube of the straight rørbru. The most pronounced joint sets can be described as follows:

Joint set nr.	Avg. dip/dip direction	Joint surface	Spacing	Trace length
1 (foliation)	65-70°/315-325°	Undulating, slightly rough	0.1 - 1 m	> 10 m
2	65°/235°	Undulating, rough	1 m	1 - 10 m
3	50-60°/80°	Planar, rough	1 - 5 m	1 - 10 m

The rock mass at this point consists of trondhjemitic gneiss with a well-developed foliation along joint set 1. The jointing frequency increases towards the boundary between the trondhjemite and the basalt, indicating the presence of a weaker zone under the sea between the island and the mainland – possibly a fault or maybe only a weaker zone as a result of the contact zones of different rock formations.

10) Location 11 lies on the southern side of the small island Kobbholmane, in the path of the straight rørbru. The most pronounced joint sets can be described as follows:

Joint set nr.	Avg. dip/dip direction	Joint surface	Spacing	Trace length
1 (foliation)	70°/310°	Undulating, slightly rough	0.5 - 1 m	> 10 m
2	80-90°/250°	Planar to slightly undulating, slightly rough	1 - 2 m	1 - 10 m
3	70°/160°	Planar, slightly rough	1 - 2 m	1 - 10 m

The rock mass at this point consists of a trondhjemitic, garnet bearing gneiss with a well-developed foliation along joint set 1 (Figure 13). The orientations of the joints sets vary consistently within this area, due to folding.

11) Location 11 lies on the eastern side of the small island Kobbholmane, in the path of the eastern tube of the straight rørbru. The most pronounced joint sets can be described as follows:

Joint set nr.	Avg. dip/dip direction	Joint surface	Spacing	Trace length
1 (foliation)	50-60°/310°	Undulating, rough	1.5 m	> 10 m
2	55-60°/75-80°	Planar to undulating, rough to irregular	0.5 - 1 m	1 - 10 m
Sparse joints	40°/15°	Planar, slightly rough	1 - 2 m	< 1 m
Sparse joints	70°/235°	Planar, rough	2 m	> 10 m

The rock mass at this point consists of a trondhjemitic gneiss with a well-developed foliation along joint set 1 and good overall rock mass quality.

3.3 Weakness zones

Based on map studies and observations in the field, the rock mass in the northern landfall area appears to be cut by several weakness zones (faults and/or zones with higher joint frequency or crushing), varying in dimension from regional-scale faults to jointing zones of a few meters. The most evident weakness zones are shown in Figure 4.

On a regional scale, several large depressions strike approximately NE-SW, creating elongated bays like Kobbavågen (the eastern of the two regional-scale weakness zones), Skitnevågen (western regional-scale weakness zone) and Vestrevågen (not included, lies west to the area shown in the picture). Several moderate to large depressions (mostly 10 to 20 m wide) also have the same orientation. Figures 14 and 15 show the weakness zone between locations 7 and 8, looking north and south respectively. Measurements of exposed rock surfaces in several of these depressions indicate that the weakness zones dip steeply (60° to 90°) towards W-NW.

In addition, several moderate to large weakness zones strike approximately SE-NW and dip steeply towards SW. These appear to be most recurrent in the western part of the area shown in Figure 4, where the straight rørbru alternative is planned to hit the shore.

4. FIGURES

Figure 5 - Location 1, blocky granitic rock mass

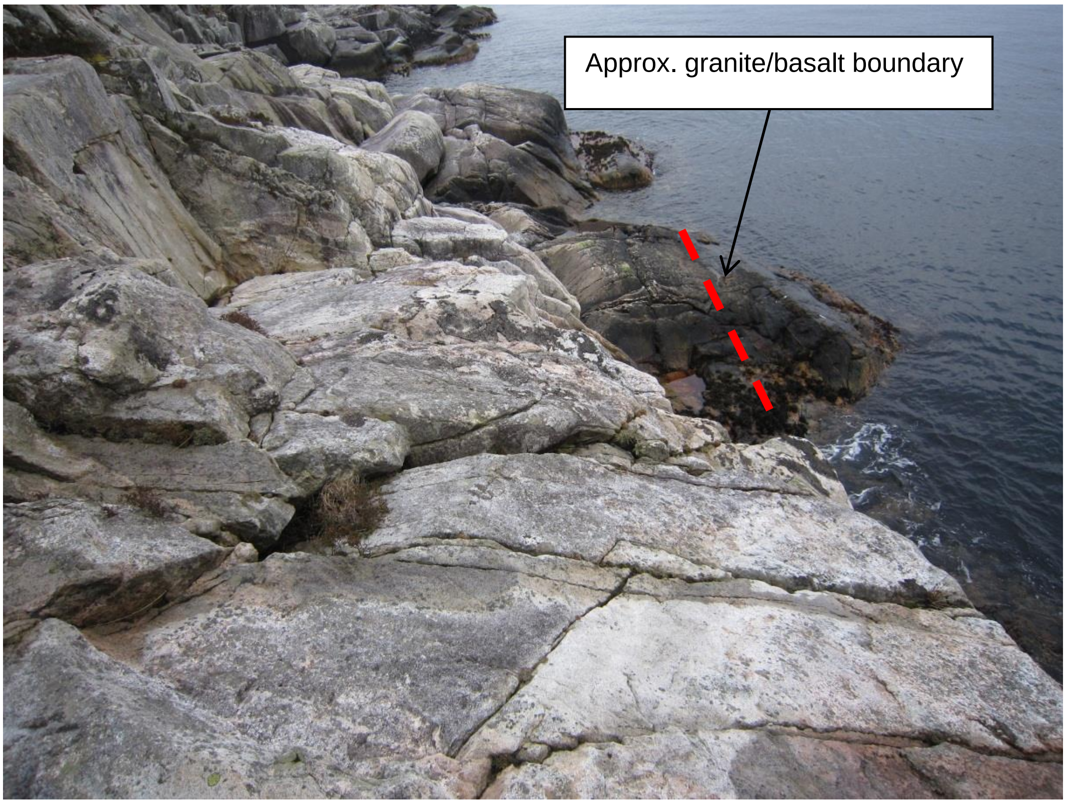


Figure 6 - Location 2, mainly monzogranite

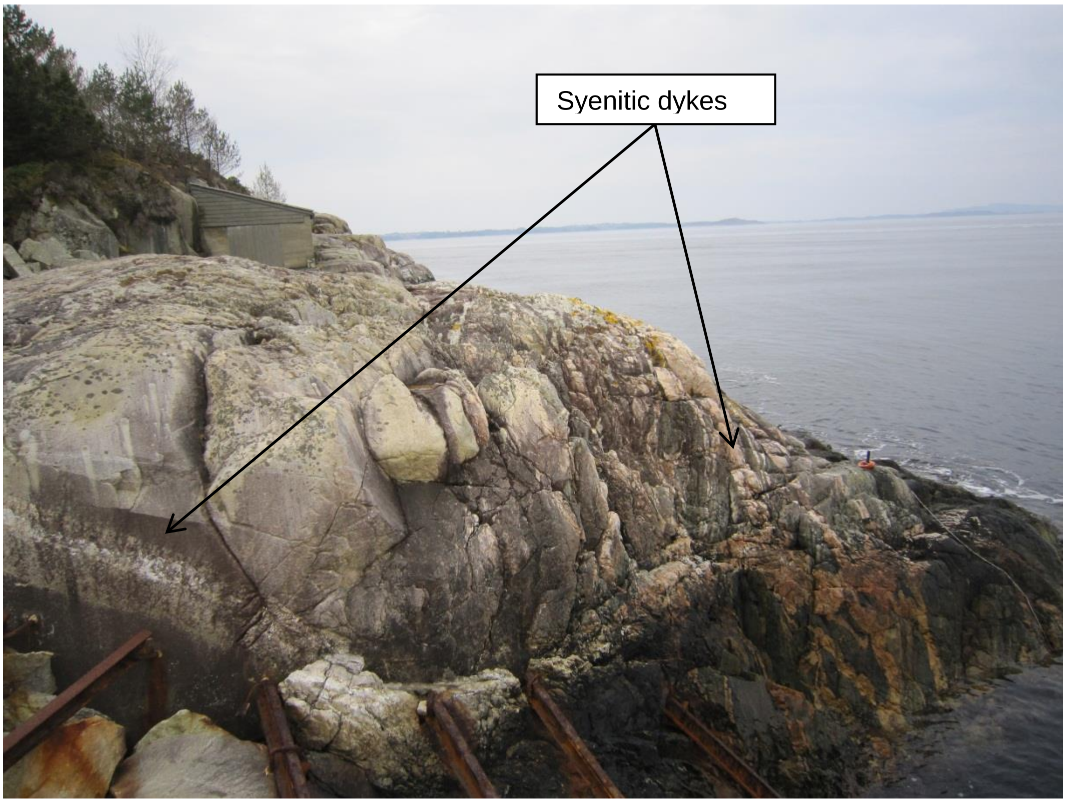


Figure 7 - Location 3, monzogranite with syenitic dykes

Figure 8 - Location 4, surface-weathered conglomerate with several intrusions and dykes

Figure 9 - Location 5, looking north towards the depression/weakness zone

Figure 10 - Location 6, blocky trondhjemite

Figure 11 - Location 7, rock mass with irregular structure and jointing

Figure 12 - Location 8, rock mass with irregular structure and jointing

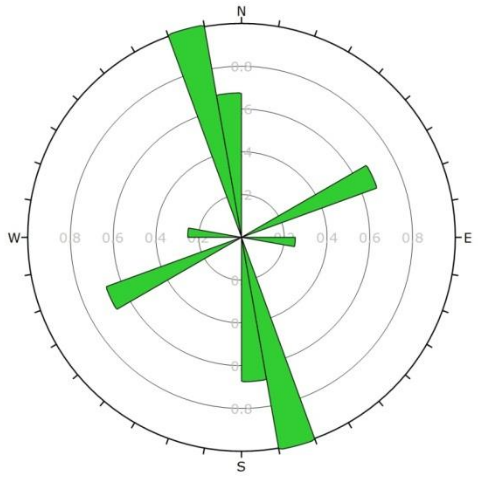
Figure 13 - Location 10, blocky and foliated trondhjemite on the island Kobbholmane

Figure 14 - Weakness zone between locations 7 and 8, looking north

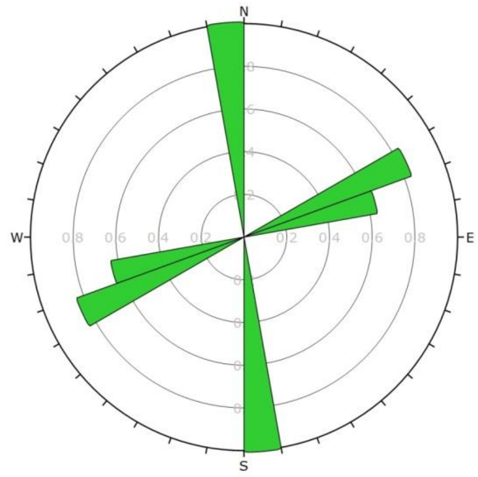
Figure 15 - Weakness zone between locations 7 and 8, looking south

5. ROSE-PLOT DIAGRAMS

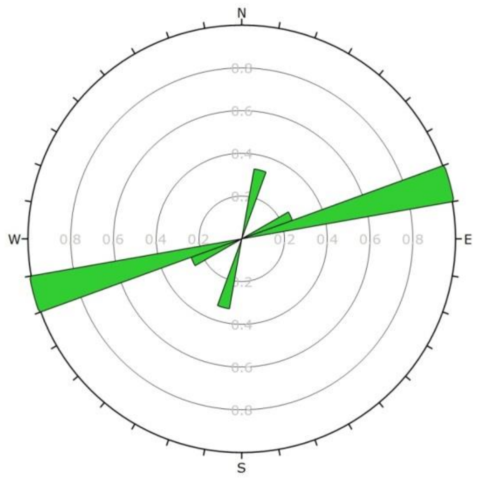
Rose-plot diagrams for each of the earlier discussed locations 1 - 11 are presented below. The diagrams are prepared with Dips 6.0 from Rocscience. The joint set with the highest frequency at each location has been given a quantity-value of 1, whereas the other joints sets are weighted against this value. E.g. for location 1, the sparse joints are on average four times less frequent than joint set 1 (the joint set with highest frequency). In the rose diagram therefore, joint set 1 has a relative value of 1, the sparse joints have a relative value of 0.25.



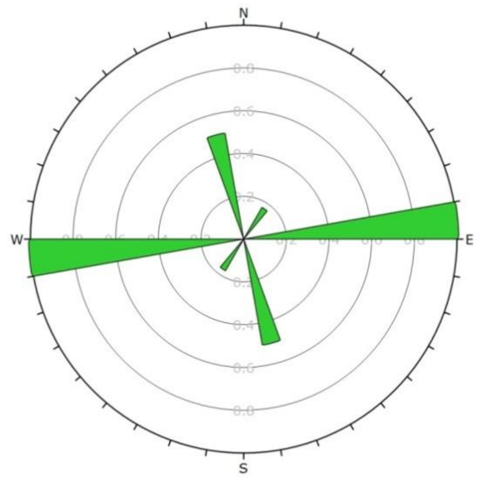
Rose diagram for Location 1



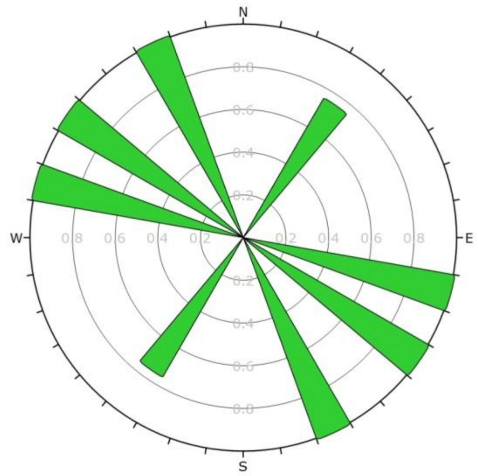
Rose diagram for Location 2



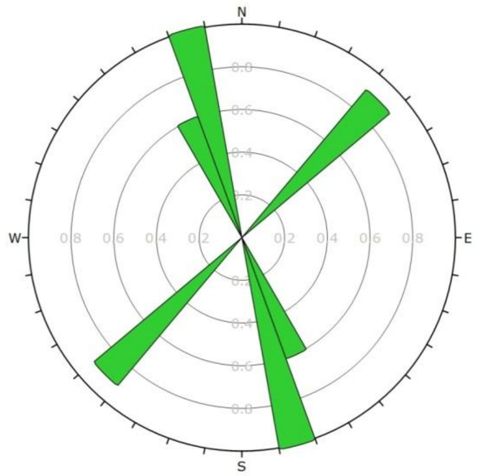
Rose diagram for Location 3



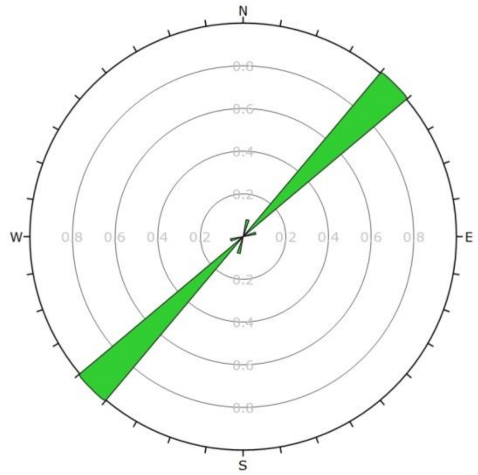
Rose diagram for Location 4



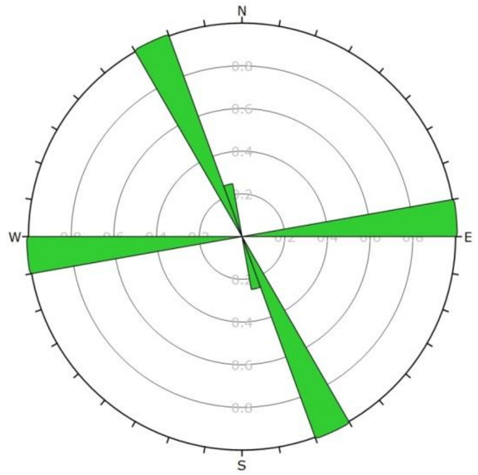
Rose diagram for Location 5



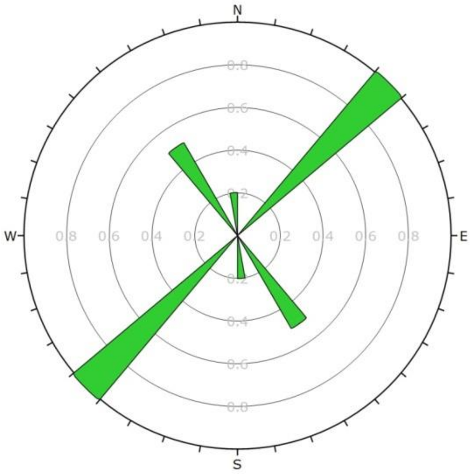
Rose diagram for Location 6



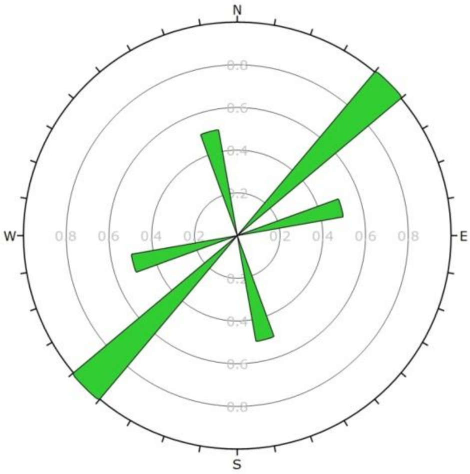
Rose diagram for Location 7



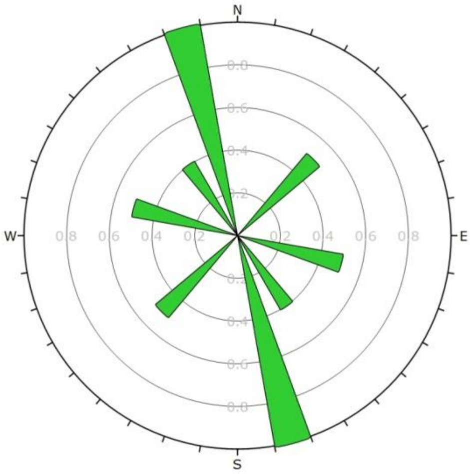
Rose diagram for Location 8



Rose diagram for Location 9



Rose diagram for Location 10



Rose diagram for Location 11

Sandvika, 2015-03-26

Prepared:

Erik Martinelli

Checked:

Thomas K. Mathiesen