REPORT

Sudurøy subsea road tunnel, executive study

SUDURØY SUBSEA ROAD TUNNEL. EXECUTIVE STUDY FOR THE EARLY PLANNING PHASE

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Content

Summary and main findings

This report presents the findings of an executive study for the further planning of the Sudurøy road tunnel in the Faroe Islands.

The focus of this study is to address the feasibility of a cost-effective construction methodology similar to the previously constructed and currently under construction subsea road tunnels in the Faroe Islands. This means an approach with one-tube solution with rock support, water control and final lining technology similar to the recently constructed subsea tunnels, as well as an acceptable and functional safety concept with the expected traffic type and traffic densities.

Four main topics are addressed in this study:

- Tunnel alignment options
- Safety concept alternatives
- Rock engineering issues related to excavation method, rock support and water control
- Ground conditions and further required investigations to enable detailed planning and construction with acceptable uncertainty

Three main tunnel alignment options have been identified and considered in this study.

Tunnel alignment option 1: Two consecutive tunnels with lengths 8 995 and 17 160 m, with northern portal at Sandur, surfacing at Skugvøy which enables construction access totally at four excavation points. This gives the shortest maximum tunnel length. There is a significant uncertainty in construction and logistics costs related to the intermediate construction site at Skugvøy. This is likely the option with the highest cost, and cost uncertainty.

Tunnel alignment option 2: One tunnel with length 25 650 m passing under Skugvøy enabling the construction of a vertical shaft for ventilation purposes. The northern portal is foreseen north of Sandur. This option gives the most direct road connection from Sandur to Sudurøy. The ventilation issues during construction, normal operational condition and fire ventilation can be significantly mitigated with a ventilation shaft at Skugvøy.

Tunnel alignment option 3: One tunnel with length 22 180 m, passing to the East of Skugvøy with the northern portal at Skarvanes. This option gives the shortest total tunnel length. The ventilation issues during construction, normal operational condition and fire ventilation can be significantly mitigated with an intermediate tunnel to Skugvøy. This option will require road construction under challenging conditions between Sandur and Skarvanes.

The basic or "0" solution for safety and rescue is based on the standard Norwegian approach with longitudinal ventilation and self-rescue for a single tube tunnel. The two identified alternatives to the “0” -solution are:
Alternative 1: Rescue containers placed in extended niches. This solution can be integrated into the standard Norwegian approach by extending the safety niches at a relatively low additional cost.

Alternative 2: Continuous rescue channel physically separated from the traffic area along one of the sides of the main tunnel. This technical solution will in principle offer the same safety function as a separate rescue tunnel at a lower cost.

A risk analysis of the fire incident issues for the main safety options will be carried out in a separate study.

The tunnel can be excavated and supported with a proven methodology for drill-and-blast rock excavation, rock support with rock bolts and sprayed concrete, ground water control with pre-excavation grouting and a selective water drip protection on a functional need basis. The existing experience with construction cost and time can be used for the planning of the Sudurøy tunnel. The challenging issues posed by the very long excavation advance lengths, particularly construction ventilation, muck logistics and fire will very likely require special mitigation measures. Such rock engineering measures are well proven and represent minor uncertainties in cost. A rock support strategy based on a functional need basis and designed for long term durability is recommended.

The standard groundwater control strategy for subsea tunnels developed in Norway and used for the subsea tunnels in Faroe Islands could be directly adopted for the Sudurøy tunnel. However, for such a long tunnel, it is particularly important to achieve high degree of groundwater control efficiency as well as reliable results. This has motivated the analysis of options for process optimization presented in this report.

To reduce the negative effects of major hydraulic conductivity contrast along the pregrouting holes, this analysis shows that a two-round grouting approach may offer significant advantages. Approximately 40% reduction of required time for groundwater control execution, a realistic target water ingress limit closer to 10 than to 20 L/min/100 m of tunnel, as well as improved stable open time in zones of extremely poor ground can be achieved. A bonus effect will be reduced lifetime cost of water ingress pumping.

The topographical conditions are overall favourable for a subsea tunnel. Tunnel entrances can be established with relatively short open cut and sea depth is shallow between the islands. Based on the available information, the deepest rock levels are in the range of 80-130 mbsl. With a minimum rock cover of 50 m in critical parts of the tunnel, the low points of the tunnel(s) will be around 180 mbsl.

The tunnel in the southern part, between Sudurøy and Skugvøy, will be located in the Middle Basalt series which is regarded favourable for tunnelling. The whole or large parts of the tunnel north of Skugvøy will be located in the Upper Basalt Series. This Basalt Series has thicker and more frequent horizons of volcaniclastic sediments that represents challenging conditions regarding tunnelling.

Limited amounts of investigations have been performed. Several vertical boreholes have been drilled, but no core samples taken. Scattered lines of reflection seismics are performed at sea. Additional surveys have been done during the last year. It is recommended to carry out systematic geophysical surveys at sea and to perform refraction seismic lines on the sea bottom to get more accurate rock levels in the subsea part of the tunnel. At least two vertical core holes are recommended for detailed study of the rock and for laboratory analyses. The focus of the further investigations should be to detect any layers of sub-horizontal volcaniclastic sediments. Such layers may potentially affect the tunnel over large distances and thereby cause challenging conditions for tunnelling.

1 Background, purpose and scope of this report

The Sudurøy tunnel will provide a road connection between the Sandøy and Sudurøy islands in the Faroe Islands. The tunnel alignments which are considered will give a very long subsea length in the range of 22-27 km. Preliminary studies which previously have been carried out for this tunnel present different tunnel alignment corridors and main technical solutions.

The purpose of this study is to provide a basis for further planning of the tunnel in the form of main decisive issues including tunnel tube concept, tunnel alignment options with possible intermediate access or ventilation access points at the Skugvøy island. Strategic considerations within the important engineering issues, such as excavation methods, rock support and water control, and long-term durability are presented.

The study is based on the experiences with the already constructed subsea road tunnels in the Faroe Islands and is oriented towards the main success factors of these tunnel projects. A key success factor for realizing the Sudurøy tunnel will be to optimize cost and time budgets and minimize risk of overruns

The functional requirements, construction methods and special safety considerations for the Sudurøy tunnel for a one-tube solution are emphasized in this study.

2 Reviewed documentation

1. Alit um møguleikar, fyrimunir og vansar at gera ein Suduroyartunnel. Rapport Landsverk, juni 2019 (in Faroese)
2. Præsentation for Vejdirektoratet, R. Jakupsson, B. Sjurdarson, Landsverk, 08.08.2021(in Danish)
3. Suduroyartunnilin. Presentation av A.E. Steinholm, F. Heinesen, 04.09.2017 (in Faroese)
4. The high resolution marine seismic survey 2020. The Suduroyar Subsea tunnel: seismic processing report. Report to Landsverk, Jardfeingi, Uni K. Petersen, December 2020
5. Suduroyartunnil, Upprit um forkanninger. Dagført frå februar 2017, J.Nr. 17/00724-1, report Landsverk, 07.05.2020
6. Suduroyartunnilin. Geological report, status, ultimo 2018, report to Landsverk, Jardfeingi, fragreiding 2018, Nr 06

3 Executive considerations regarding main tunnel concepts

3.1 Main tunnel alignment options

In this study, three main alignment options have been considered:

- Option 1: A tunnel alignment which includes surfacing at Skugvøy. Hence, two separate tunnels will be constructed: Sandøy – Skugvøy and Skugvøy – Suduroy
- Option 2: A direct tunnel alignment Sandøy – Suduroy which passes below Skugvøy enabling the construction of a ventilation shaft and pumping shaft for drainage water
- Option 3: A direct tunnel alignment Sandøy – Suduroy, including a possible ventilation access tunnel at Skugvøy, enabling the shortest possible connection between Sandøy and Suduroy.

The alignment options are shown in figure 1.

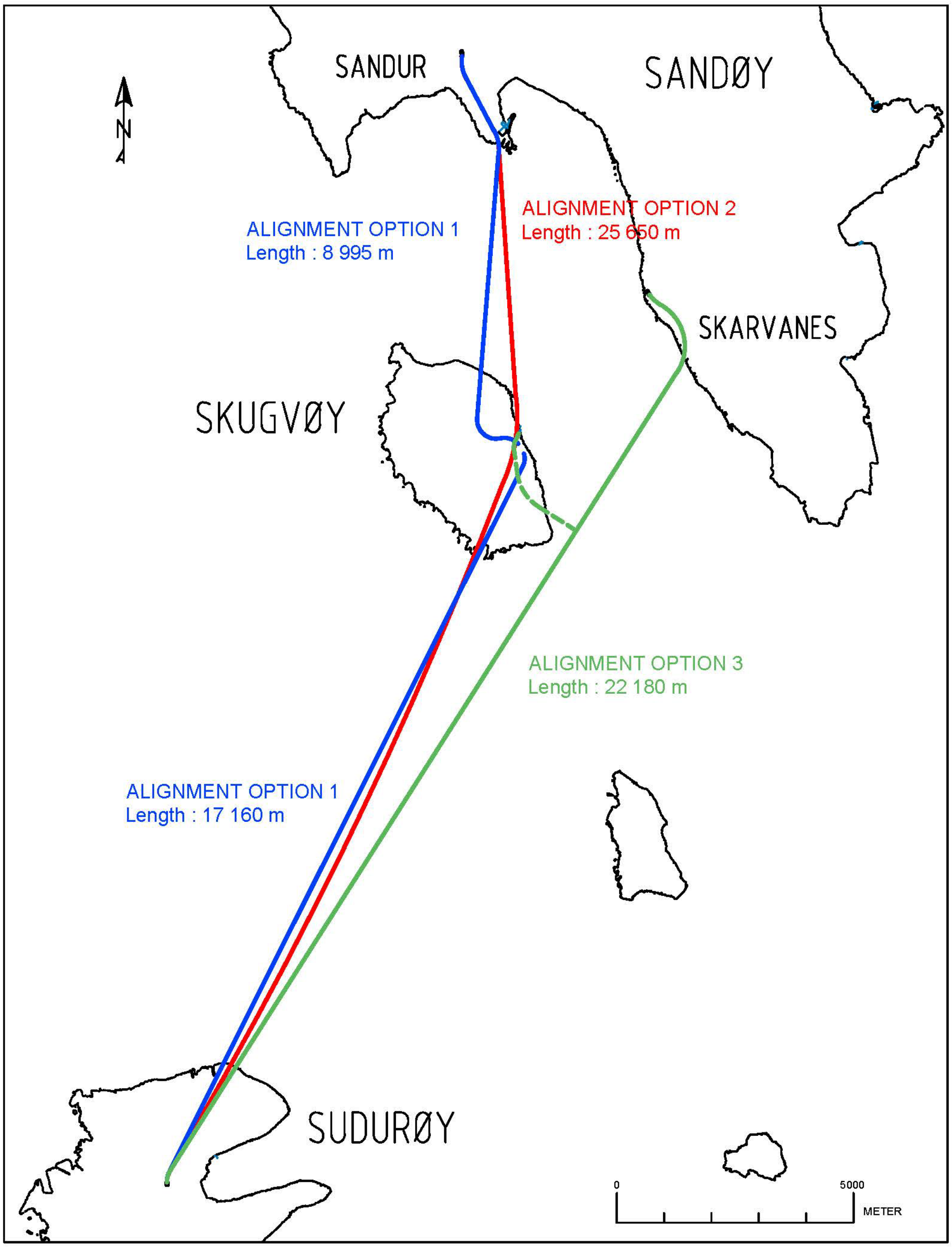


Figure 1. The three tunnel alignment options which have been considered in this study

A minimum rock cover of 50 m at the critical locations of the tunnel alignments has been assumed. This is in accordance with the Norwegian Code N500. In this study, only a one-tube solution has been considered. The feasibility from a technical and safety perspective of a one-tube solution is very likely a decisive factor for the financial feasibility of the entire project.

In the following the main features of the alignment options as well as main construction issues are described.

3.2 Alignment option 1: two successive tunnels, surfacing at Skugvøy

This option, illustrated in figure 2, involves the construction of two separate tunnels Sandøy (portal at Sandur) – Skugvøy and Skugvøy – Suđurøy with lengths approximately 8,5 and 17,5 km respectively.

This option represents a complete separation in two portions from a ventilation, safety and rescue perspective, both during construction and operational phase. This option involves relatively steep sections of the tunnel at Skugvøy with approximately 6% gradient with the given tunnel lengths. If 5% maximum gradient is a requirement, the tunnel alignment will be slightly longer.

The construction can be realized with access from four excavation advance points. This gives a minimum critical unilateral tunnelling advance length of approximately 8580 m for the longest tunnel.

A complete construction site facility including a harbour needs to be established in the remotely located Skugvøy. This will pose significant challenges, due to the need for reliable logistics of construction materials such as concrete components and explosives, as well as the environmental impact around Skugvøy.

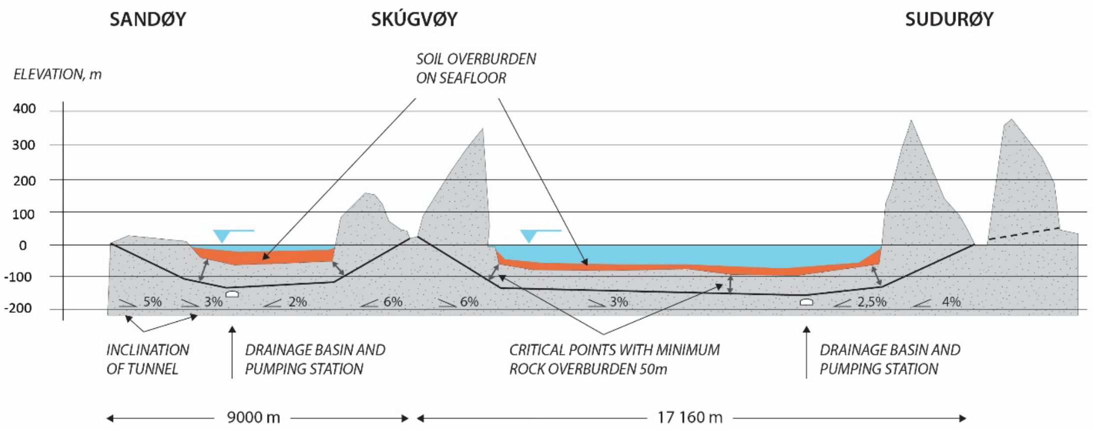


Figure 2. Tunnel alignment option 1 with two separate tunnels surfacing at Skugvøy. Longitudinal section.

3.3 Alignment option 2: one tunnel below Skugvøy with shaft access

This option involves the construction of one tunnel Sandøy (portal at Sandur) – Suðurøy, passing below Skugvøy. This option is illustrated in figure 3. The alignment would pass under Skugvøy at a location which enables the construction of a shaft system for ventilation and pumping of drainage water. This system could consist of two separate shafts or one shaft to cover this purpose. The shaft would be vertical and constructed with pilot holes drilled from Skugvøy followed by raiseboring. All the excavated rock material will then be taken out through the main tunnel portals. The construction facilities on Skugvøy will then be limited to the shaft boring site.

This option includes the possibility for sectionizing of the tunnel from a ventilation and fire perspective. This could be realized in the form of a facility constructed in the tunnel at the shaft location, with gates, turnaround structure and ventilation guidance installations.

There is also the possibility of including a pipeline in the shaft for pumping of seepage water for a portion of the tunnel.

During construction the shaft can be utilized for construction ventilation, hence providing a significant improvement of the ventilation situation for the portion of the tunnel from Skugvøy towards Suðurøy.

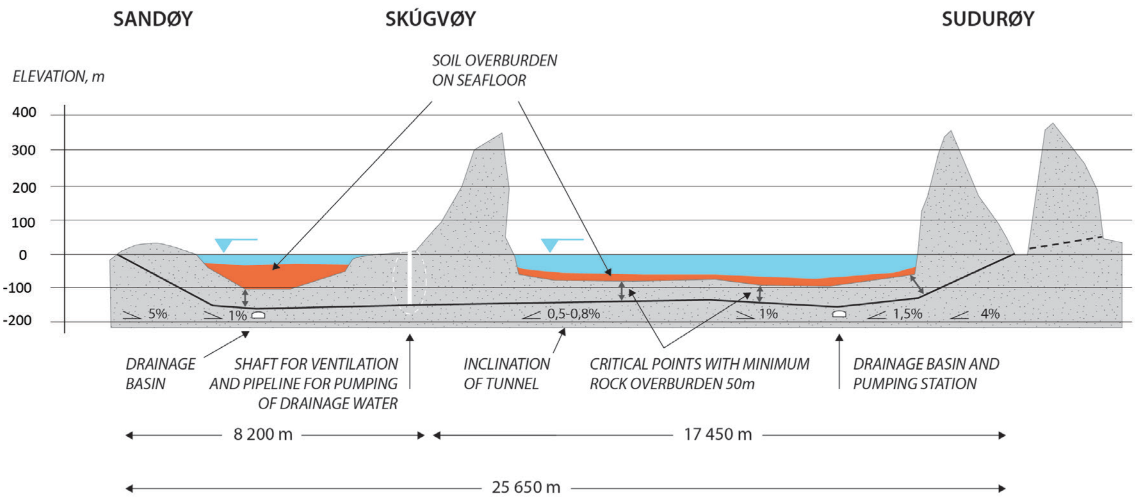


Figure 3. Tunnel alignment option 2 with one long tunnel passing under Skugvøy with ventilation shaft. Longitudinal section.

3.4 Alignment option 3: one tunnel with shortest possible length

This option, illustrated in figure 4, involves the construction of one tunnel with the shortest possible alignment Sandøy (portal at Skarvanes) – Suðurøy, optimizing the alignment irrespective of passing under Skugvøy. The tunnel alignment passes slightly East of Skugvøy. Hence, there is a possibility to realize a ventilation tunnel or shaft with access to Skugvøy also for this option. A ventilation tunnel can also be used as an escape tunnel.

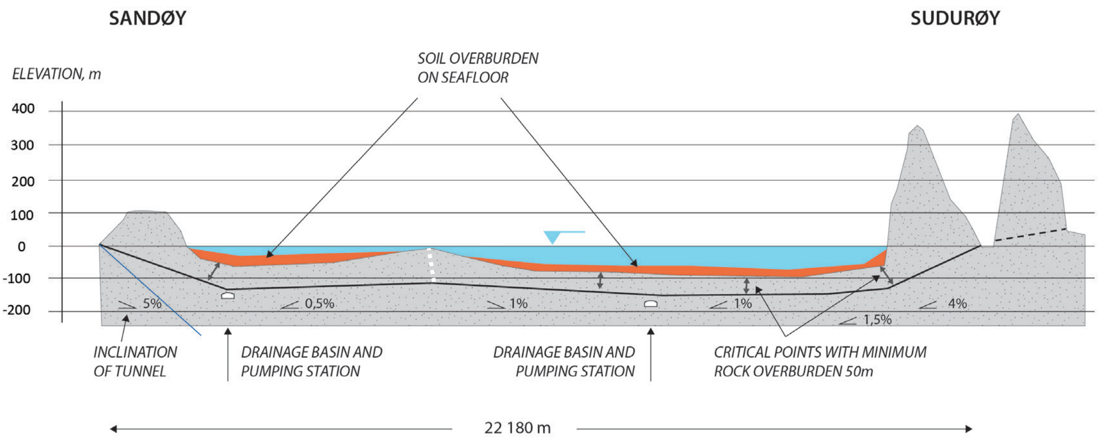


Figure 4. Tunnel alignment option 3 with one long tunnel passing East of Skugvøy with possibility for ventilation tunnel. Longitudinal section.

3.5 Main safety concepts

Three different safety concepts are considered in this study.

Table 1. Main safety concepts identified for a one-tube tunnel solution.

Safety concept	Technical solution
0	Existing approach in the Faroe Islands, according to Norwegian code N500
1	Extended safety niches with rescue containers every 500 m
2	Continuous rescue channel in the main tunnel, separated from the traffic area

3.5.1 Safety concept 0, standard technical solution according to Norwegian code N500

The Norwegian code N500 gives requirements for tunnel classes based on traffic density and tunnel length. For traffic density between 300 and 4000 vehicles per day up to 10 km tunnel length, tunnel class B, the Norwegian code allows two-way traffic in one tunnel. For tunnels longer than 10 km the term "special consideration" is used. The safety concept 0 in this study is the standard solution for tunnel class B in Norwegian tunnels and is based on longitudinal ventilation in one tube and self-rescue. The Sandøyartunnel, currently under construction, will use this safety concept. No extraordinary rescue and safety measures are included in this option. On this basis single tube is considered also for the Sudurøy tunnel.

3.5.2 Safety concept 1. Rescue containers in extended safety niches

This safety concept is an adjusted version of the standard solution for tunnel class B in Norwegian tunnels, by including rescue containers in every second safety niche (every 500 m). The rescue containers will be equipped with life sustaining equipment for survival during a certain period of time in the case of a fire incident. This concept is illustrated in figures 5, 6 and 7.

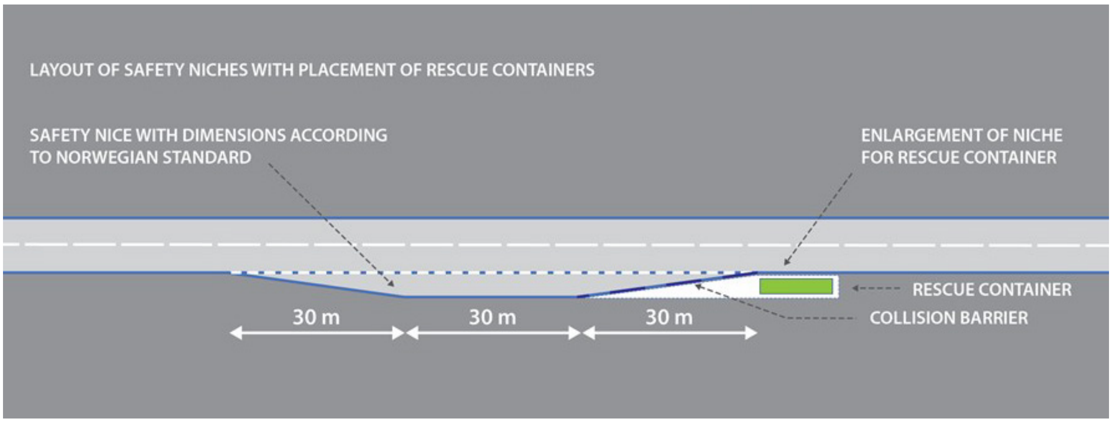


Figure 5. Safety concept 1 with rescue containers in extended safety niches. Planar view.

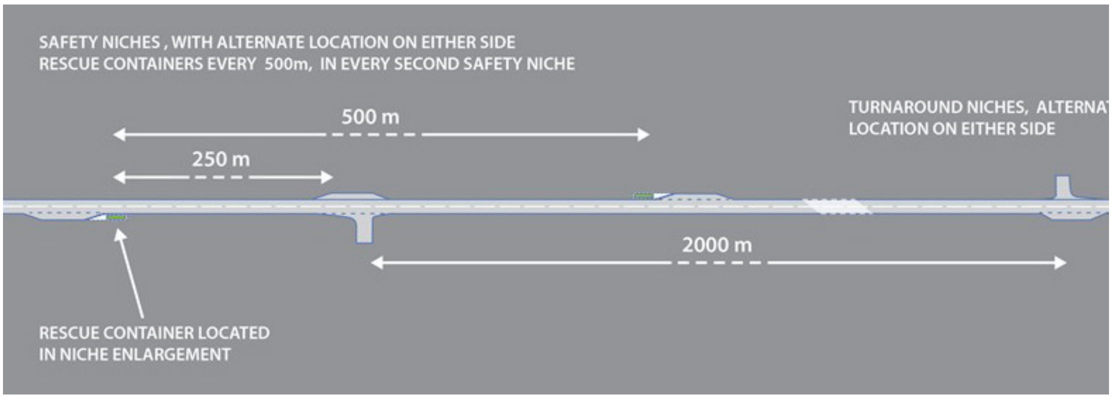


Figure 6. Safety concept 1 with rescue containers in extended safety niches, based on tunnel class B, Norwegian code N500. Planar view.

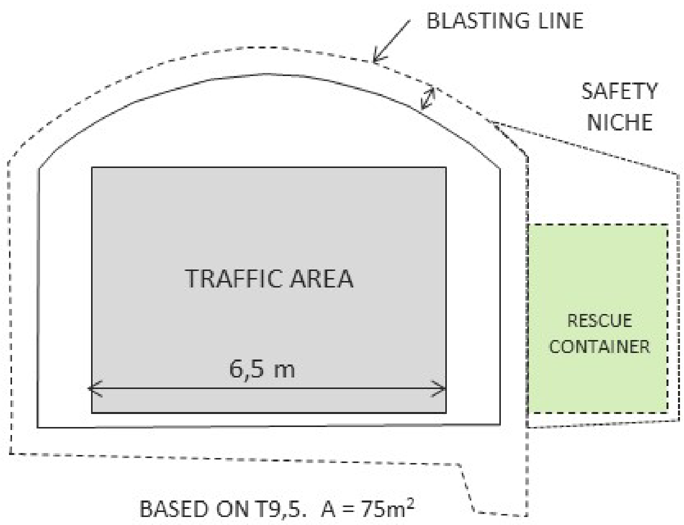


Figure 7. Safety concept 1 with rescue container in extended safety niche. Cross section

3.5.3 Safety concept 2, continuous rescue channel

This safety concept is based on a continuous rescue area in the form of a channel along the entire tunnel. The channel is an area physically separated from the main tunnel. Access from the traffic area to the rescue channel will be through doors with close spacing. Hence, access to the rescue channel will be granted practically everywhere along the tunnel. The air in the rescue channel will be subject to a separate ventilation circuit and have a slight overpressure compared to the traffic area. The excavation area can be based on the standard Norwegian section T10,5 GS+ which is originally foreseen for a walk- and bikeway alongside the traffic area for vehicles. Safety concept 2 is illustrated in figures 8 and 9.

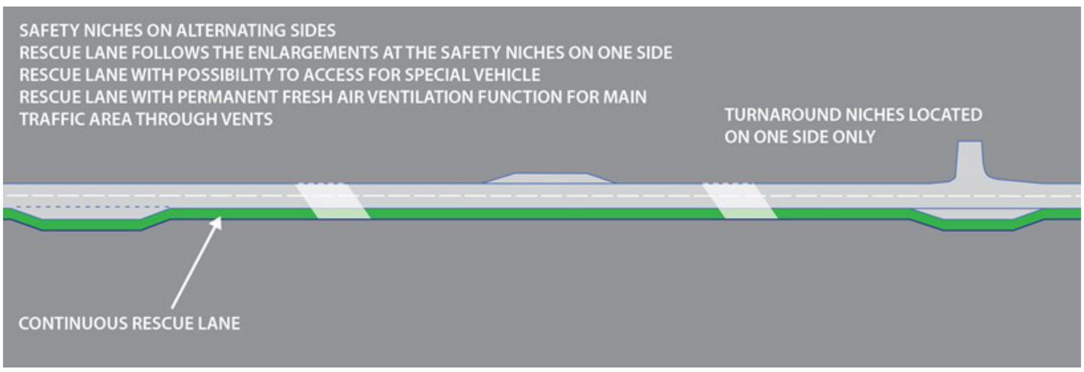


Figure 8. Safety concept 2 with continuous rescue channel

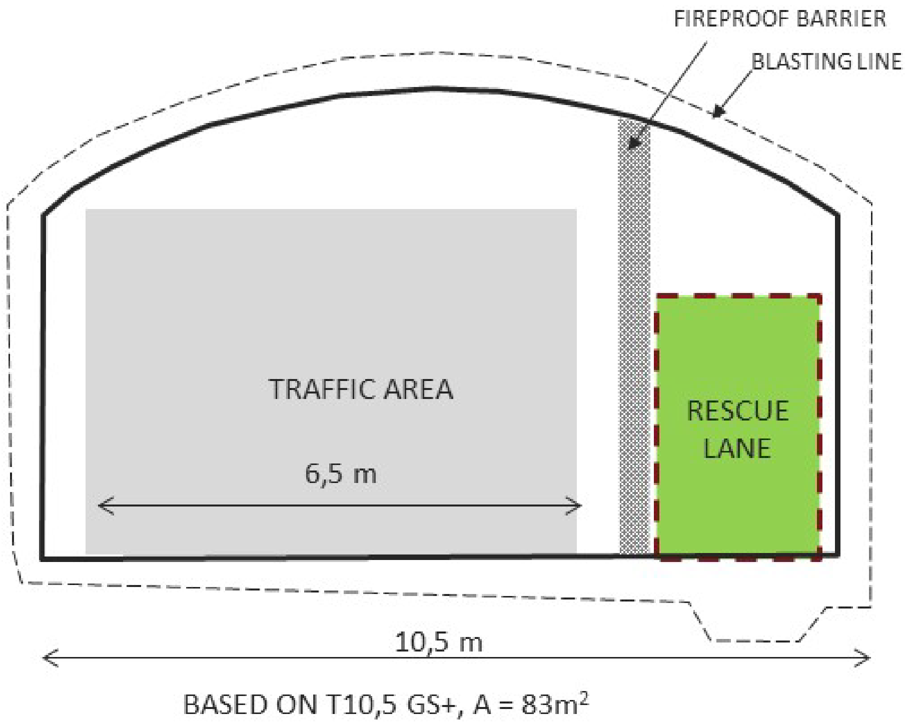


Figure 9. Safety concept 2 with continuous rescue channel

3.6 Evaluation of alignment options and safety concepts

3.6.1 Tunnel alignment options

The main identified advantages and disadvantages of the three tunnel alignment options are summarized below.

<u>Alignment option 1.</u>

Main advantages	Main challenges and disadvantages
- Intermediate construction access point at Skugvøy - Favourable access points on either side - Shortest maximum length excavation section from two headings - Ventilation and safety: Two shorter tunnels instead of one long. - Lighter safety solution possible in the northern tunnel	- Environmentally sensitive area at Skugvøy - Intermediate construction site at Skugvøy involves a large additional investment for logistics - Uncertainty in time and cost for intermediate site at Skugvøy - Additional total tunnel length - Highest cost of the three options - Operational phase: one additional incline and decline, potentially increased risk of incidents - Different safety concepts for the two tunnels may pose challenges

Alignment option 2.

Main advantages	Main challenges and disadvantages
- Favourable access points on either side - Tunnel will be sectioned in two parts from a ventilation perspective - Uncertainty in construction costs can be reduced - Ventilation and safety: sectioning in two separate tunnels (with shaft connection to surface) - Shaft solution at Skugvøy with minor impact at surface - Alternative to shaft: intermediate tunnel excavated from main tunnel (length approximately 2 000 m with 8% gradient) - Operational phase: only one incline and decline	- Excavation of 25 650 m of tunnel from two headings - Challenging logistics, ventilation, fire incident risk during construction - Critical safety points during construction, low points and risk of flooding and entrapment of personnel in the tunnel - Operational phase: safety concept for this tunnel length

Alignment option 3.

Main advantages	Main challenges and disadvantages
- Shortest total tunnel length (22 180 m) - *Possible* intermediate tunnel, alternatively combined with shaft to Skugvøy - Ventilation and safety: *Possible* sectioning in two parts from a ventilation perspective - (either shaft or tunnel) - Operational phase: only one incline and decline	- Excavation of 22 180 m of tunnel from two headings - Challenging logistics, ventilation, fire incident risk during construction - Critical safety points during construction, low points and risk of flooding and entrapment of personnel in the tunnel - Operational phase: safety concept for this tunnel length - Additional construction costs related to access at Skarvanes (portal area and road construction, approximately 6-7 km). Land tunnel may be necessary.

3.6.2 Safety and rescue alternatives

The safety and rescue concepts will be subject to a detailed risk analysis, possibly including a cost benefit analysis. This will be published in a separate report. The main identified advantages and challenges related to the two considered safety alternatives are summarized below.

Technical solution with rescue containers

Main advantages	Main challenges and disadvantages
- Can be integrated into Norwegian Code N500 solution with safety niches - Low additional cost compared to additional tunnel with escape tunnels to main tunnel - Number of rescue containers can be increased also after commissioning	- Rescue point spacing may give up to 400-450 m distance for escape - Will require maintenance and service of the facilities in the containers - Access for emergency services in the event of an incident is dependent on possible access in main traffic area - The concept with rescue containers is not approved by EU - Rescue time from rescue containers may be long

Technical solution with continuous rescue channel

Main advantages	Main challenges and disadvantages
- Offers a continuous safe area in the entire length of the tunnel, with access points possible at very short distance (100-150 m) - Can be accessed by vehicle inside the rescue lane - Relatively low cost compared to additional tunnel with escape tunnels to main tunnel - Low maintenance cost - Fresh air in the rescue lane can be integrated into the ventilation concept for the main tunnel	- Will require a larger excavation area (approximately 10-15 m^2 larger excavation section throughout the entire tunnel - Higher construction cost - The solution requires further development - Rescue time from safe rescue channel may be long

4 Rock engineering and construction issues: strategic considerations

4.1 Excavation method

4.1.1 Drill-and-blast excavation

Drill-and-blast excavation has been used for the previously constructed subsea road tunnels in the Faroe Islands, as well as all subsea road tunnels in Norway. Drill-and-blast excavation represents a well proven approach to hard rock subsea tunnelling for highways and any other type of tunnel requiring strict groundwater ingress control. An important part of the construction methodology for subsea road tunnels is how to handle the groundwater control, starting with systematic percussive probe drilling ahead of the tunnel face. Measured leakage in the probe drilling holes and defined trigger levels are used for deciding about pre-excavation grouting.

Direct access to the tunnel face allows for many different techniques for dealing with challenging rock mass conditions with the drill-and-blast excavation method compared to the TBM alternative (see below). Hence drilling works ahead of the tunnel face for probe drilling, pre-excavation grouting and forepoling/spiling are always at hand and collared with unlimited choice of locations and directions. Reduced excavation rounds can be utilized and immediate support can be installed after excavation. If necessary, support work at the tunnel face can also be used.

4.1.2 Excavation with Tunnel boring Machine (TBM)

In this case a TBM excavation method with an open gripper TBM has been considered on a principal basis. A shielded TBM of Earth Pressure Balance (EPB)-type with the continuous installation of a pre-cast concrete segmental lining has not been included in this study. Such a technical solution is expected to have a too high cost which would be unrealistic for this project. The nature of the rock masses encountered in the Faroe Island are generally favourable for TBM excavation due to the low abrasivity and strength of the rock types and potentially high net advance rates that can be achieved.

The main advantage of a TBM is the possibility to achieve high advance rates over longer portions of the tunnel when the geological conditions are favourable. With unilateral excavation lengths of more than 8 km as in the case of the Suðurøy tunnel, a TBM solution can be found to be cost-effective. TBM excavation creates a circular section. Hence, the layout of the different areas needs to be adjusted accordingly. The carriageway will be placed on a concrete floor or a gravel backfill. Drainage and possibly a rescue channel can be placed in the invert area below the carriageway. This is illustrated figure 10.

Drilling ahead of the tunnel face for probing, pre-excavation grouting and forepoling/spiling need to be collared a certain distance behind the face, normally 2-3m. This has been realized on a number of TBM projects and is illustrated in figure 12.
An important limitation of a TBM excavation is the access to the tunnel face for different works such as immediate support, and the handling of water inrushes. Rock support works can be effectively executed immediately behind the roof shield, which will be approximately 3-4 m behind the tunnel face. The areas for rock support are normally classified in the L1, L2 and L3 areas. L1 corresponds to the area in front of the grippers, and L2 is the area behind the grippers and the back-up. Rock support works carried out on the back-up is classified as L3. This is illustrated in figure 11.

Poor ground, especially in combination with high hydraulic conductivity can be detrimental to TBM excavation in a subsea situation. The vulnerability of TBM excavation to difficult and water bearing rock masses is therefore a critical issue to consider in the choice of excavation method. The uncertainties related to ground conditions need to be significantly reduced for TBM excavation to be selected compared to drill-and-blast excavation. Hence, a larger amount of extra pre-investigations will be required before TBM-excavation can be decided.

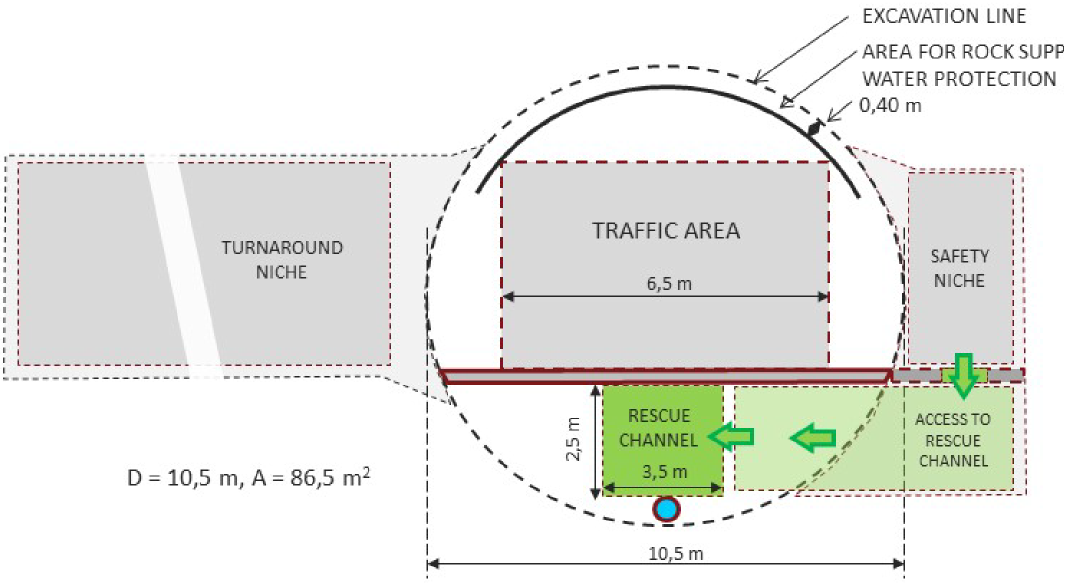


Figure 10. Option with TBM excavation, excavation section indicating the necessary additional areas for safety niches, turnaround niches and possible location of rescue channel. Vertical cross section.

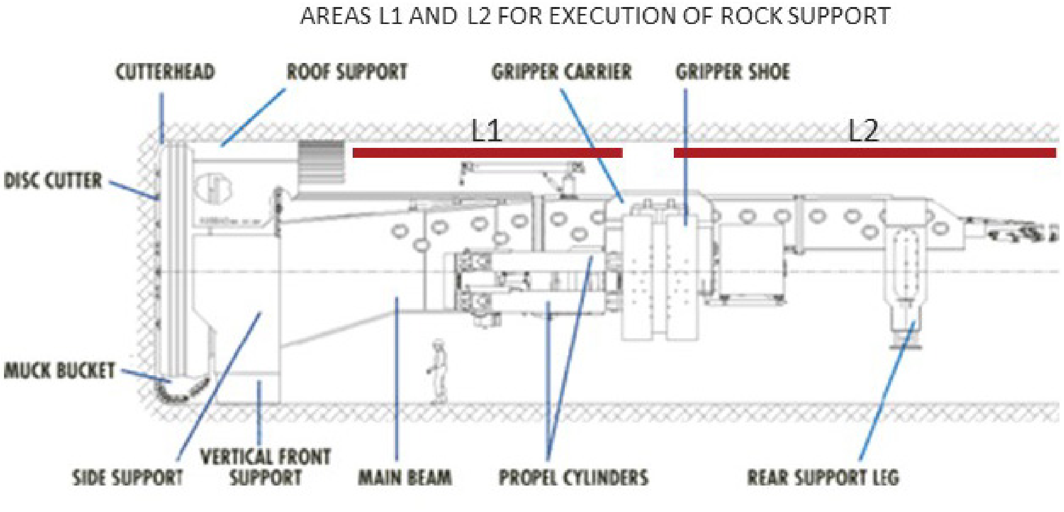


Figure 11. Configuration of open gripper TBM with indication of possible areas for execution of rock support L1 and L2. Vertical longitudinal section.

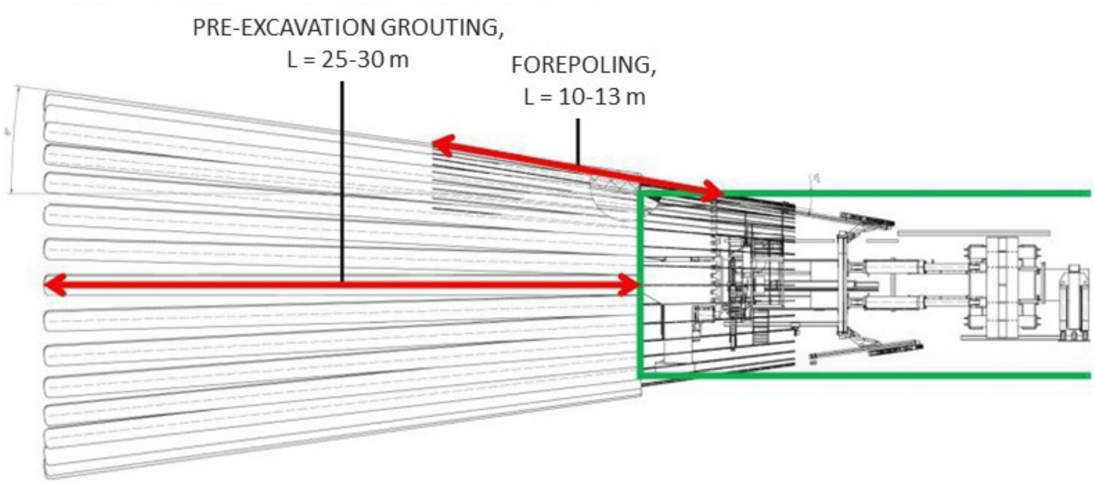


Figure 12. Open gripper TBM and schematic layout of drilling ahead of the tunnel face for pre-excavation grouting and forepoling. Vertical longitudinal section.

4.1.3 Summary of risk considerations related to excavation method

Risk issues related to drill-and-blast excavation:
For all alternatives there will be very long unilateral excavation drives with logistic challenges related to mucking and transport of rock. Excavation logistics and exhaust from trucks can be improved by the use of a conveyor belt system instead of truck transport.

Long drives will require large diameter ventilation ducts and booster stations along the tunnel to get enough fresh air to the tunnel frontGases from the blasting rounds will need very long time to escape the long tunnels and may hamper work.

Collapse of the tunnel at the tunnel front is a serious event, but as discussed in chapter 4.1.1, this should be a very unlikely event since so many tools are available to prevent it to happen.

Fire during construction may cause large consequences.

Risk issues related to TBM excavation:
For a TBM alternative, the use of conveyor belt for transport of the rock mass from the tunnel front is now the main practise. The need for ventilation is not as critical as for dill & blast since blasting is need for niches and technical rooms only.

The method is much more sensitive for large ingress of water and adverse ground mass conditions, since the possibilities of mitigation measures are limited.

Passing of weakness zones will take much more time than for D&B and there will be a high risk for getting the machine stuck in these situations. It may take weeks to get the machine going again. If the TBM enters a subhorizontal layer/horizon that follows the tunnel over hundreds of meters, this could be extremely time consuming and costly.

Large ingress of water is also much more difficult to handle. Pre-excavation grouting (PEG) may be done, but as the possibilities of adjusting direction and number of holes are limited, this is much more difficult to control and solve than doing the same for D&B.

4.2 Water control strategy

4.2.1 Main functional requirements, considerations

Introduction

Groundwater control execution for sub-sea tunnels has normally more than one purpose and the basic situation for sub-sea tunnels is different to most tunnels above sea level:

- All water ingress must be pumped out of the tunnel both during construction time and in long-term operation. Long sections of the tunnel will be excavated on a decline and water ingress therefore runs toward the tunnel face.
- Ingress water is salty ocean water, causing special corrosion conditions and in case of any major face area inrush, the water supply is unlimited, even though the steady state rate of ingress after typically some days will be lower than the initial ingress.

Functional requirements

The most important functional requirements are:

- The peak water ingress at tunnel face area must be less than the installed pump-out capacity (even during power-outages) to avoid submerging the excavated part of tunnel.
- The residual ingress to the tunnel must be limited to an acceptable practical level during tunnel construction and the target value must be optimized for lowest possible lifetime project economy. This means that the planned sum of investment into pre-excavation grouting (PEG) until opening time of the tunnel, should amount to about the same as the discounted value of the cash-flow stream during e.g. 100 years for the permanent pump-out installation, operation and maintenance.
- The current targeted residual ingress to sub-sea road tunnels in the Faroe Islands seems to be 20 L/min/100 m of tunnel, while similar Norwegian tunnels have commonly used 30 L/min/100 m of tunnel as the target value. Actual tunnel values will deviate from these target values both in average for a whole tunnel and along tunnel sections. A closer analysis of this balance is recommended.
- Since there is no simple or accurate way of linking resource allocation for PEG and the resulting residual ingress to a tunnel, the optimization task becomes complicated. What we know, is that less water will be beneficial to construction

time and cost, reduce maintenance of installations in the tunnel as well as the mentioned pump-out installation, operation and maintenance cost. At the same time, targeting less and less residual water ingress will eventually face a limit due to declining return on investment.

- In this context, it must be emphasized that tunnel sections requiring post-excavation grouting resulting from any 'cost-saving' effort, may become extremely expensive to mitigate.
- Lastly, groundwater control and rock support through extremely poor ground are frequently viewed as two separate issues, thus overlooking the crucial interaction between the two. When the face is entering into a shear zone with this kind of ground quality, the control of water becomes particularly important and sometimes demanding. Most clay-filled complex shear zones will offer a fair amount of cohesion, producing sufficient stable open time (for 1-2 m of stepwise advance) to apply sprayed concrete and rock bolts, but a rapid face collapse may develop if water causes erosion or hydraulic collapse. Proper groundwater control may from experience be the difference between losing 6-12 months or more due to a face area collapse or excavating and supporting through the zone with short blasting rounds, and support steps.

4.2.2 Pre-excavation grouting (PEG), strategy and methodology

Executed sub-sea road tunnels in the Faroe Islands have produced residual ingress levels around 20 L/min/100 m of tunnel and avoided any serious flash-flows through the face area. The execution has also been quite effective and efficient at spending just the necessary time to reach targeted results. This is well demonstrated by weekly advance rates between 40 and 90 m with the average around 60 m.

Most of the tunnelling approach has been based on systematic probe drilling and PEG when detecting water above the trigger limit, also called grouting on demand. The limit has typically been 8 L/min from the probe holes, regardless of 2-3 or 4 probe holes drilled. Along sections with frequent switching between triggered PEG and sections of no grouting, indications are that more use of systematic *grouting* would have performed better.

The normal strategy has been based on achieving satisfactory groundwater ingress reduction by executing only one round of grouting. This has been done by drilling enough grouting holes around the tunnel profile (sometimes also including holes at the tunnel face) giving spacing of about 1.5 m at the collaring location. To allow for sufficient time for cement setting, one round of blasting has been the next step, followed by control holes to check the grouting result. Mostly, one grouting round has been enough, but occasionally, a total of 2 to 4 rounds have been required. It seems that no post grouting has been executed and areas of water drips have been diverted to the walls by roof-mounted "umbrellas".

The normal stop criteria for cementitious grouting per hole has been 55 bar stop pressure or 5000 L of grout, if stop pressure was not reached first. The typical average consumption of cement per meter of overall tunnel length has been about 1000 kg.

Grouting has been done using a local rapid hardening Portland cement, supplied in big-bag and by bulk, to feed up to 4 grout pumps on the grouting rig. Mostly, 3 pumps have operated simultaneously, achieving an average output of 2500 to 3000 kg/h of pumping time.

It should be noted that the main high-yield joints and channels have been sub-vertical with observed apertures up to 10 cm. It has been a surprising observation that backflow of cement grout through the face or even to several meters behind the face has occurred, mostly without any water flow. Grouting ahead of the face, sometimes creates new water leakage behind the face, especially along sections without systematic PEG.

4.2.3 Potential for optimization of the grouting methodology for the Sudurøy tunnel

To develop a strategy for groundwater control for the Sudurøy tunnel, it is first necessary to clarify what is the targeted level of acceptable residual water ingress. Experience from Faroe Island subsea road tunnels so far indicate that 20 L/min/100 residual ingress can be a reasonable choice, even though sections of tunnel have given as high as 35 L/min/100 m measured ingress. The current target value and execution of groundwater control has satisfied the requirement to avoid major flash-flows in the face area. The remaining questions could therefore be:

1. Is 20 L/min/100 m the optimal residual ingress target considering lifetime cost?
2. Will it be possible and beneficial to reduce the significant variation in actual local section ingress (between 15 and 35 L/min/100 m)?

Question 1 probably needs an in-depth economic analysis of the balance between cost of groundwater control, compared with discounted cost of pump-out system investment, operation and general maintenance works during ca. 100 years of operation. Currently, it may be postulated that the optimum target value could be as low as 10 L/min/100 m because of the cost benefit of less water to pump during the tunnel lifetime and practical benefits during construction as well as reduced general maintenance during operation.
Question 2: If analysis of Question 1 leads to a reduced residual ingress target, it will be reasonable to expect fewer local sections with ingress well above the average.

By combining a lowered targeted residual ingress limit and efforts to reduce the high-ingress tunnel sections there will also be benefits during tunnel construction from less disturbance during sprayed concrete application, less road maintenance, less water to handle at the face etc. A reduction of the target ingress limit will not prevent need for drip protection, but it will reduce the necessary area for such mitigation measures. If the number of drip locations gets sufficiently reduced, this might also open the possible usage of spray-on membrane for spot drip protection.

It is important to realize that a stricter residual ingress target without optimization of the groundwater control execution, could increase execution cost more than the value of long-term benefits of less water to handle. Conversely, if a significant reduction of overall water ingress can be achieved by no or marginal increase in cost of groundwater control, there would be good reasons to work through an optimization effort. The basic task therefore will be to optimize the groundwater control execution. This needs to be expressed in a detailed method statement for groundwater control in the Sudurøy tunnel.

Facts of life in pre-excavation grouting (PEG)

To identify potential for optimization, it is necessary to establish what we know, what we think we know as well as what are targeted expected effects of changes of execution. This means moving from aspects of execution that most will agree to (based on previous experience), into less clear and obvious mechanisms and effects of suggested changes. Because of the length of tunnel and the substantial cost of groundwater control regardless of strategy and method of execution, it must be pointed out that the built-in uncertainty of modifications of standard methodology can be reduced by pre-construction testing, which is therefore one important recommended action to take.

Some important aspects of groundwater control by grouting that we do know:

1. The bulk of groundwater control must be done by cementitious grouts through pre-excavation grouting (PEG) and to make it crystal clear, post grouting is not an alternative (only an occasional local supplement).
2. Conductivity contrast along individual one-meter sections of boreholes about 25 m long, can run into several orders of magnitude.
3. Grout pumped will always take the path of least resistance.
4. Sufficient quantity of grout must be distributed in a controlled way to where it is needed for successfully achieving the targeted result.
5. Large aperture joints and channels may accept grout spreading to 100’s of meters away from the borehole if pumping is not properly terminated.
6. Sufficient grout spread into medium and small aperture joints and channels will require high enough pumping pressure and enough pumping time to achieve this.
7. Available maximum pumping pressure of modern grout pumps (typically 100 bar or more) can produce hydraulic jacking of the rock mass.
8. Cement maximum particle size and particle size distribution influence what minimum size apertures that the grout may enter and penetrate.
9. A grout with high bleed-value has also typically poor pressure filtration coefficient according to API Method RP 13B-1, this being negative for the ability to penetrate.
10. There is a huge range of available cement types and qualities, also when limiting the search to those deemed suitable for grouting in rock tunnels. Setting time is extremely important for PEG and final set can range from ‘too fast’ to between 15 and 20 hours.

If the overall goal is to satisfy a decided level of residual ingress to produce the lowest lifetime cost, smallest resource allocation and shortest construction time, the above 10 items do already present some high potential options for optimization.

Optimization suggestions and rationale

Starting with Item 10 about various cement types, the stop time after end of grouting until drilling for blasting or any long hole drilling ahead of the face is currently set to 5 hours in Faroe Island subsea tunnels. By selecting a more suitable cement and possibly modifying its setting properties, the stop time could be reduced to 1 hour or less. The overall influence on critical path construction time will depend on the percentage of the tunnel length that turns out to require grouting, but a likely scenario could produce seven weeks difference alone if added up as continuous waiting time.

The conductivity contrast mentioned in Item 2 and the dominant grout flow along path of least resistance in Item 3, have several effects when grouting first round holes into virgin ground. When aiming at satisfying the ingress limit by just one round of grouting, the hole spacing necessary will be small (1-1.5 m) and a relatively large percentage of these holes will cross large aperture features. Because of this, pumping pressure in those holes will be low and flow rate will be high. Some of the grout will flow into other holes in the round and the rest will very soon take off outside of the rock volume targeted for treatment. Standard approach is then to gradually reduce w/c-ratio and increase viscosity to increase pumping pressure. When pressure finally increases, apertures smaller than the dominant ones have been blocked by developed dry-plugs at the opening to the borehole. The high pumping pressure targeted, when finally reached, has effect limited to a very short distance around the borehole and *only* in the large aperture channels where grout is flowing, so penetration into smaller apertures will be significantly limited. Any borehole that does *not* cross any large aperture feature, will more quickly produce pumping resistance (high pumping pressure) and penetration into medium and small size apertures will take place as intended. However, there will be an obvious risk that some of these holes will show preferential connection to a nearby borehole (not yet grouted), which will disturb the general grout spread into the surrounding rock mass.

The technical result of grouting can be improved by using a different strategy, which is based on the same logic and practical experience available from decades of dam foundation grouting. If one third to one half of the number of holes normally drilled for the one-round approach is drilled as a first round, then grouted and followed by a second round, the overall total number of holes could be about 80% of the standard approach. The first round can be grouted quickly and efficiently to partly stop on pressure, or to stop (mostly) on a reasonable stop quantity, without any modification of viscosity in the process. The grout must have zero bleeding, should demonstrate thixotropy and fast initial and final set. Fast enough to allow start of drilling of the second round in less than one hour. The drill-jumbo stays in place with the grouting rig outside of it during the two rounds of drilling and grouting. This way, most of the holes in second round will stop on pressure (without any thickening of grout viscosity) and provide a better general distribution of grout into medium and small size channels. The 20% reduction of number of boreholes could alone (for a likely scenario) reduce grout consumption by about 200000 liters. Even more important is the likely reduction in overall grout consumption, assumed to be 20% reduced compared to the one-round strategy.

Items 4, 5 and 6 are about controlling the placement of grout to where it is needed and to avoid loss of grout to outside of where it can do any good for the tunnel. The large aperture features must be stopped on quantity, regardless of which pressure has been reached. For this to be efficient, the grout must be non-bleed, preferably thixotropic and with short setting time. Provided that backflow to the tunnel is avoided, water and grout in channels of the rock mass are in a static situation when stopping the pump and channel volume filled will remain 100% filled and will set up to completely block those channels. High pressure is not necessary or beneficial and limiting the volume is just a matter of common-sense decision-making regarding stop quantity.

In the second round of grouting, conductivity contrast has been significantly reduced by the grout filling of the largest aperture joints and channels during first round grouting. Consequently, most of the holes will quickly start building pumping pressure and grout is forced into smaller size apertures.

Even though the two-round strategy has less tendency to be disturbed by grout connections from one hole to one or more of the other holes, it will save time and improve results if all holes (both first and second round) have closed packers before start of grouting. The normal objection is that connections then cannot be *observed*. That is correct, but by keeping all packers closed, there will not be a gradient towards boreholes with open packers leaking water, inviting grout to enter such holes to contaminate them before planned start of pumping on those holes. This approach helps executing grouting from invert holes to roof holes as a fixed routine, all holes being stopped on the same criteria, thus simplifying and speeding up work progress.

Item 7 about maximum allowed pumping pressure is important. Provided that there is a fixed stop on quantity, the maximum allowed stop pressure should be as high as possible, while avoiding hydraulic jacking. Estimated stop pressure will depend on depth of tunnel and local rock cover, but modification as needed must be made based on evaluation of pressure-flow curves during grouting. Typically, most of the grouting should allow stop pressure in the range 50-70 bar at the pump, but work execution results will tell if a lower pressure limit must be used locally to avoid jacking.

Another recommended approach detail, regarding use of pumping pressure, is about boreholes not taking any grout. It may happen that some holes are just filled by grout and after that, the pressure increases directly to the specified local stop pressure. If this is recorded as a stop, the hole is basically wasted, since no grouting of the rock mass will take place. In such cases, pressure should be increased to create a very local hydraulic jacking, which often will open a connection to an existing local conductive channel close by the borehole. This will be demonstrated by a sudden drop in pump pressure and simultaneous start of grout flow. The local hydraulic jacking may happen 20-40 bar above the stop pressure but testing for the described break-through should be limited to 100 bar.

Items 8 and 9 covers properties of cement that are important for penetration in small aperture joints and channels, which must be given higher priority and focus, the lower the decided maximum residual ingress to the tunnel. Also, the typical ground conditions will influence how important the penetration ability of the grout will be. Generally, if a target residual ingress limit is set as low as 10 L/min/100 m, most of the execution time and cost of PEG will be linked to sufficiently penetrate the small aperture joints and channels. A cement grout with good penetration ability, may allow larger borehole spacing, thus saving time of drilling, reduce cement quantity and reduce overall execution time, while still satisfying the target result.

When considering the material cost of cement for grouting, it is normal in most projects to focus on the kg-price. When this partial cost element of PEG amounts to well below 5% practically irrespective of cement type, it is fair to say that the kg-price focus is misplaced. The overall cost of PEG is about 70% time related, so whatever the materials cost of a grout mix design that can save time, will be a better choice if it also satisfies the technical requirements. In other words, serious efforts should be made to find a cement type with low enough maximum particle size, favourable particle size distribution, allowing high w/c-ratio and low viscosity without bleeding and a favourable pressure filtration coefficient. To also fulfil requirements on setting time, the use of admixtures and additives must be carefully investigated in pre-construction testing with all candidate cement types, without considering the cement kg-price. It is highly likely that a version of microfine cement will be the best choice both from a cost perspective and to ensure satisfactory groundwater ingress reduction.

Outline of optimized methodology

Before presenting an outline of an optimized methodology, it must be stated that a set of assumptions have been made first. In a real project development, many parameters will play a role and they interact with each other. A balanced methodology will therefore have to be adjusted accordingly and is likely to deviate somewhat from this outline. The normal observational approach of feeding actual residual ingress measurements back into an implemented methodology for optimization purposes, will also typically lead to adjustments during execution.

Still, it is expected that the described strategy will turn out beneficial and that it can be implemented with some limited adjustments of the methodology outline presented below.

The target regarding groundwater control is to achieve a balanced lifetime cost between investment in pre-excavation grouting and installation and 100 years operation of pump-out system plus general tunnel maintenance.

With the stated target, it seems reasonable to implement a strategy including:

- Reduction of target residual ingress from 20 to 10 L/min/100 m of tunnel.
- Switching from an execution method based on one-round grouting, to generally using a two-round approach for achieving the decided residual ingress requirement.

- Identification of and use of one single micro-fine cement (MFC) with verified properties suitable for this strategy and ground conditions.
- Using dual stop criteria per grout hole of maximum allowed pumping pressure, or if not reached first, stop on quantity of grout regardless of pressure reached.
- Systematic probe-drilling to check for trigger of PEG, using 4 probe-holes drilled *within* the forward tunnel excavation volume.

The methodology for execution of drilling ahead and PEG must be detailed in a Method Statement (MS). The last valid version of the MS must be followed in the tunnel without exceptions, until a new version with adjustments or additions has been issued, this to avoid ad hoc improvisations making it hard to identify the likely reasons for any unexpected results of PEG.

The MS must describe the details of pre-construction testing of candidate cement types, based on a prioritized set of property parameters decided for this project. One product should be selected with one back-up product to ensure cement availability.

Probing- and grouting sections should be covered by 25 m long holes with a certain overlap. A 5 m overlap can be recommended if the probe drilling doesn't trigger grouting. If grouting is executed, a 7-8 m overlap is recommended. Drill 4 probe holes parallel to the centreline, within the forward excavation volume (placed in a rectangle about 1.5 m inside the blasting line). If PEG is not triggered, grouting of the probe holes is *not* necessary.

Trigger value to execute PEG: 5 L/min or more, from any hole, or sum of all probe holes. *Note that if any probe hole hits 300 L/min or more, at any depth drilled to less than 25 m, just drill 1 m more and stop. Procedure for such special case must be detailed separately.*

When PEG is triggered, drill 1st round of grout holes 25 m long at about 3.6 m spacing (around 9 holes) with a lookout angle of 10^{o}. Place closed packers in all holes (including the 4 probe holes) and grout bottom-up (invert to roof) with mix design for 1st round. Stop criteria: 70 bar or 3000 liters.

Drill 2nd round grout holes 25 m long at about 3.6 m spacing (around 9 holes) with a look out angle of 7^{o}. This will produce an overall hole spacing of about 1.8 m after both rounds. Execute grouting with mix design for 2nd round.

Drill control/probe holes as described above, but to avoid premature return to excavation without PEG, the trigger criterion for PEG should be reduced to 3 L/min or more, from any hole, or sum of all probe holes. If PEG is *not* triggered by 3 L/min or more, the probing trigger criterion for the next section is back to 5 L/min.

Note that any grout hole that goes straight to the pressure stop criterion with no flow, just after filling the borehole volume, should be tested by increasing the pressure (if necessary to 100 bar) to possibly create a break-through to any close by conductive feature. This would be the case if pressure suddenly drops to below the elevated test pressure and grout flow starts simultaneously. If so, keep on grouting the hole to normal stop criterion.

4.2.4 Indicative comparative outcomes of strategies

By calculating through 3 different strategies, using a set of likely basic assumptions, it is possible to compare the relative outcomes of these strategies. The numbers generated will certainly not accurately reflect reality, but the indicative tendencies are considered reasonably correct and may support motivation for further analysis and preconstruction testing.

The common basic assumptions used are the following:

- Overall tunnel length of 26000 m
- The time-critical heading length is 50% of the total, so 13000 m
- Systematic full length probe drilling is executed and 60% of the probing stations are assumed to trigger pre-excavation grouting (PEG), so the time-critical heading requires 7800 tunnel meters treated by PEG.

Comparison 1:

One standard round of grouting per station, as described above from typical Faroe Island sub-sea tunnels already finished, compared with just a change to splitting the 22 holes into 11 holes plus grouting and another 11 holes plus grouting.

This change will require 22% more groundwater control time (time from start probe drilling until start drilling for blasting) due to the 5 h turn-around from grouting to next drilling ahead. This applies when there is no need for any extra grouting round due to unsatisfactory verification of result.

If we include one extra grouting round in 20% of the stations for the standard case and 10% when using the two-round approach, the overall extra time for the two-round approach requires 11% more groundwater control time.

Comparison 2:

One standard round of grouting per station, as described above from typical Faroe Island sub-sea tunnels already finished, compared with again the one round approach, but now optimized to allow 1 hour turn-around from grouting to next drilling.

The two numbers given above will for this case be 18% saved execution time when no extra grouting rounds are required, and 16% time saved when including extra rounds.

Comparison 3:

One standard round of grouting per station, as described above from typical Faroe Island sub-sea tunnels already finished, compared with the two-round approach using microfine cement with short setting time.

The two numbers given above will for this case be 35% saved execution time when no extra grouting rounds are necessary, and 39% time saved when including extra rounds.

Summing up:

Comparison 1 shows primarily the significant effect of *not* changing the turn-around time when splitting the standard approach into two rounds with the same total number of grouting holes. The groundwater control time increases by roughly 10 to 20 %, which in this example would be 7 to 11 weeks extra.

Comparison 2 shows that if *not* changing to a two-round strategy, but just optimizing the standard approach for 1 h turn-around time, about 17% execution time may be saved, which in this example would amount to 10 weeks saved.

Comparison 3 shows the effect of a two-round strategy optimized both for the 1 h turn-around time, 18% reduction of number of grout holes and 20% reduction of cement consumption, with an assumed reduction in cases of having to add one extra round of grouting, that may ultimately save 39% of the execution time. In this example this translates into 23 weeks saved.

How to translate the above indications into a final strategy choice and detailed Method Statement should await a later stage, when more geologic information is available and closer analysis of activities in the tunnel can be performed. As mentioned, the economic evaluation of lifetime cost and value of reduced residual ingress will be important for selection of the targeted residual ingress limit.

Recommendations:

Generally, the two-round strategy will deliver more flexibility to adapt to variations in ground conditions and thus produce improved groundwater ingress reduction. It is considered highly likely that this strategy will reduce cost and time, while also improving the technical results. It is therefore recommended to base further project development on the above outlined two-round strategy and to ensure and verify the important strategy presumptions by necessary lab- and pre-construction grouting in the field.

The target ingress limit should probably be between 10 and 20 L/min/100 m. This can be decided at a later stage. Since fulfilling the decided limit is not critical outside of the tunnel and some variation in amount of drip locations that will need treatment is easily managed by existing methods, adjustment of execution and target ingress limit can await further project analysis and even practical experience from the tunnel.

4.3 Rock support and water drip protection

4.3.1 Main functional requirements

The functional requirement for the rock support follows the Norwegian approach for rock support in traffic tunnels. This approach has been followed for the previously constructed subsea tunnels in the Faroe Islands.

The level of rock support is required to provide a global rock mechanical stability of the tunnel and a sufficient detailed stability of the rock surface. Rock surfaces which are located directly over the carriageway traffic area or critical installations are recommended to be supported with the required rock bolting and the minimum sprayed concrete thickness. Rock surfaces which are located in walls or not directly over the carriageway traffic area, can be left without sprayed concrete when the rock mass class or detailed stability allows for this.

4.3.2 Rock support design and methodology

The rock support design which has been successfully used on all earlier tunnel projects in the Faroe Island will be suitable for the Sudurøy road tunnel.

The rock support design utilizes the rock mass as the structurally bearing element. The function of the rock support is to reinforce the rock mass and provide detailed stability of blocks and rock fragments. Hence, both global and detailed stability of the rock mass surrounding the tunnel is given. The rock support design utilizes fully grouted steel bar rock bolts in combination with fiber reinforced sprayed concrete.

Rock mass classifications according to the Q-system (NGI, 2015) and empirically based rock support classes will be a cost-effective approach. In the good rock classes (e.g. Q > 4) the rock support classes shall be adapted to the functional need. In the poorer rock classes (Q < 4) it is recommended to use the rock support charts for tunnel construction currently in use in the Faroe Islands (Landsbyggifelagid, 2020)

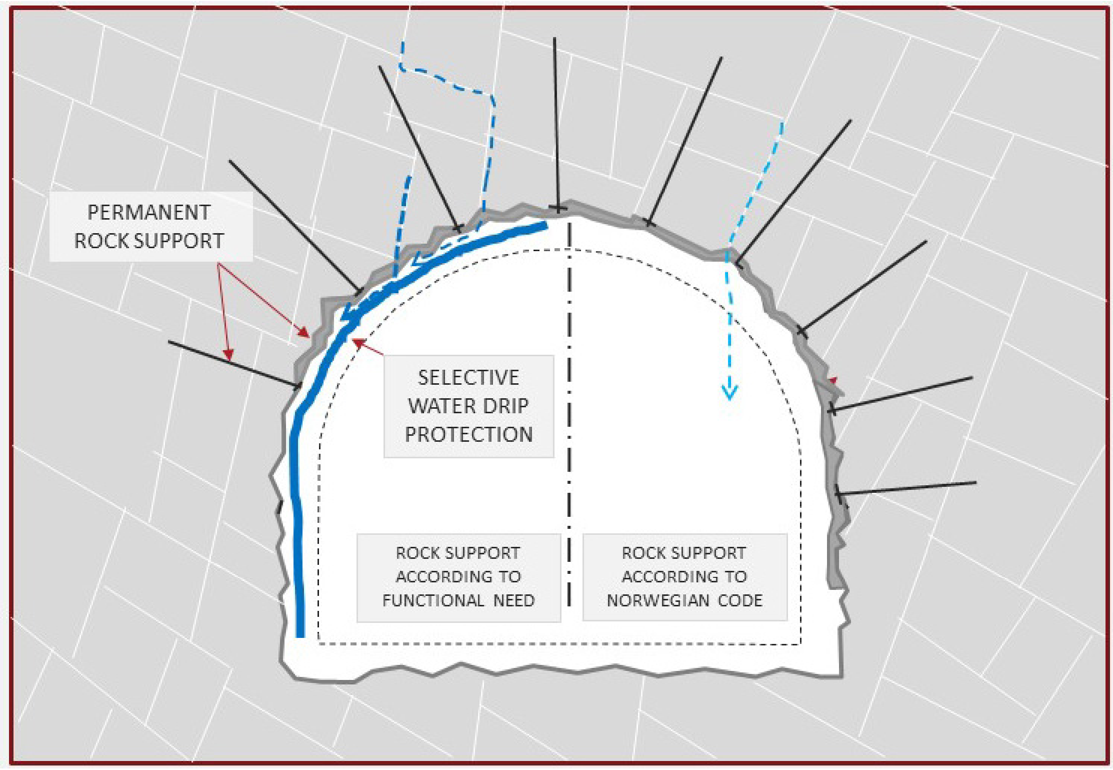


Figure 13. Rock support and water drip protection, cross section. Principle of selective approach based on functional need.

4.3.3 Water drip protection

An approach based on functional need can be cost-effective for the Suduroy tunnel. This approach has been used for both the Vaga and Nordøy subsea road tunnels.
In principle this approach means:

- Definition of a trigger level for the use of water drip protection, in the form of a minimum number of drips and drip density
- Areas of the tunnel which have an amount of water drips below the trigger level, are left without water drip protection

4.4 Service life time and durability

4.4.1 Planned service lifetime

The planned service lifetime for traffic tunnels in Norway (Bane NOR and NPRA) is 100 years. The material quality specifications for rock support materials and layout of the rock support aims at 100 years of service lifetime.

The Norwegian code N500 requires an average thickness of 100 mm for steel fibre reinforced sprayed concrete and triple corrosion protection of rock bolts for subsea tunnels for a 100 years service lifetime. The required durability of the sprayed concrete is reflected in the maximum allowed water/binder ratio of the sprayed concrete of 0,4. The main degrading feature in a subsea tunnel is the exposure to running seawater. Hence, significantly improved durability of the rock support materials and the water drip protection structure can be achieved by reducing the exposure to seeping water.

Detailed research data from the rehabilitation work in subsea tunnels in Norway will be available in spring 2022. This can provide a basis for a precise assessment of the sprayed concrete thickness and target seepage into the tunnel in operational condition. As a basic recommendation the average applied thickness of sprayed concrete should be 100 mm. With this requirement a service lifetime of 100 years can be expected. Only local areas exposed to seeping water may experience degradation within within the service lifetime. Such areas can be repaired selectively in a cost-effective manner.

4.4.2 Maintenance strategy within service lifetime

The maintenance strategy within the service lifetime should aim at the following main issues:
Rock support:

- Target service lifetime 100 years.
- Every 25 years: Inspection and evaluation according to updated and new regulations, with assessment of condition and need for rehabilitation works
- Every 5. year: main inspection in conjunction with scaling of bare rock surfaces, assessment of possible degrading and need for local rehabilitation works

Water drip protection:

- Target service lifetime 50 years, with planned complete reconstruction
- Every 5. year, main inspection in conjunction with rock scaling, assessment of local maintenance and repair

5 Geological conditions and pre-investigations

5.1 Main current base and uncertainties

5.1.1 Geology

The most recent update of geological information is found in the Geological report, status ultimo 2018, prepared by Jardfeingi. The report refers to one tunnel alignment only, where the tunnel starts in Sandur, goes up in Skugvøy and a new tunnel continues to Sudurøy.

The basalt flows in general represent good conditions for tunnelling. However, in between the basalt flows there are volcaniclastic layers that may contain ash and sediment layers that represents weaker rock mass and layers of high permeability. In the photo taken on the west coast of Sandøy, these layers are marked with different colours in the column. Thickness of these layers vary from dm`s to tens of meters (C-horizon). The main horizons have specific names, see figure 14.

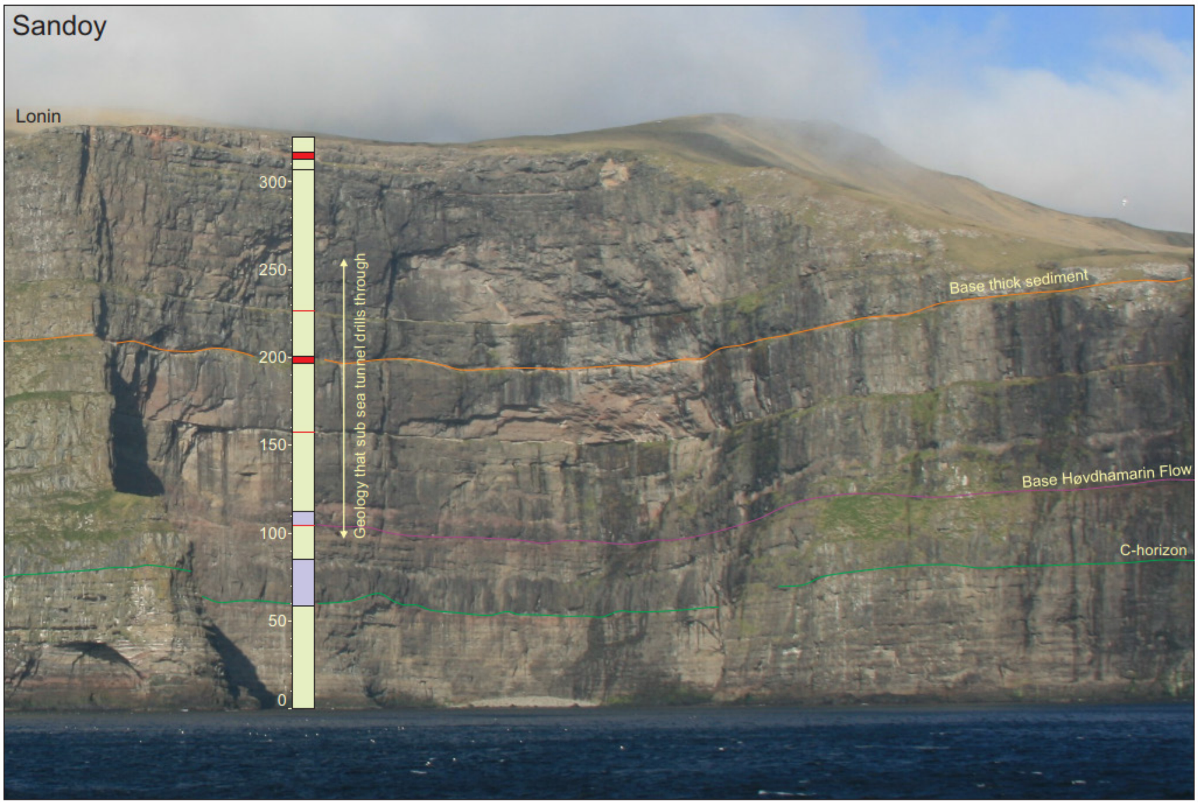


Figure 14. Photo of west coast of Sandøy. Three horizons are marked in the photo. These horizons have rock mass quality that are poorer than surrounding basalt layers and will cause stability problems if encountered in the tunnel roof.

The dip and strike of the layers are found from different locations around the island. Roughly the Base thick sediment has a dip of 2.8° towards East. Based on this, Jardfeingi has produced a contour map showing the Base thick sediment location around the region. For a subsea tunnel starting north of Sandur, the tunnel floor will be around 100 mbsl as it enters the subsea part South of Sandur. From figure 15 the Thick Sediment is around 80 mbsl in this location. The tunnel alignment shown in Figures 15 to 17 is the same as Option 1 in this study.

Going southwards, and assuming the tunnel around the same depth, the Hovdhamarin Flow must be crossed by the tunnel before it goes up to Skugvøy. When the tunnel goes down again from Skugvøy, also the C-horizon will have to be crossed. From there to Suðurøy the tunnel will be located in the Middle Basalt Formation.
The volcaniclastic sediments that are deposited on top of each basalt flow, represent a hazard and challenge to tunnelling, and should thus be avoided or have the shortest possible contact with the tunnel. The geology report states that the Thick Sediment represents a several meter thick volcaniclastic layer. The horizons Hovdhamarin and C-horizon may represent similar horizons.

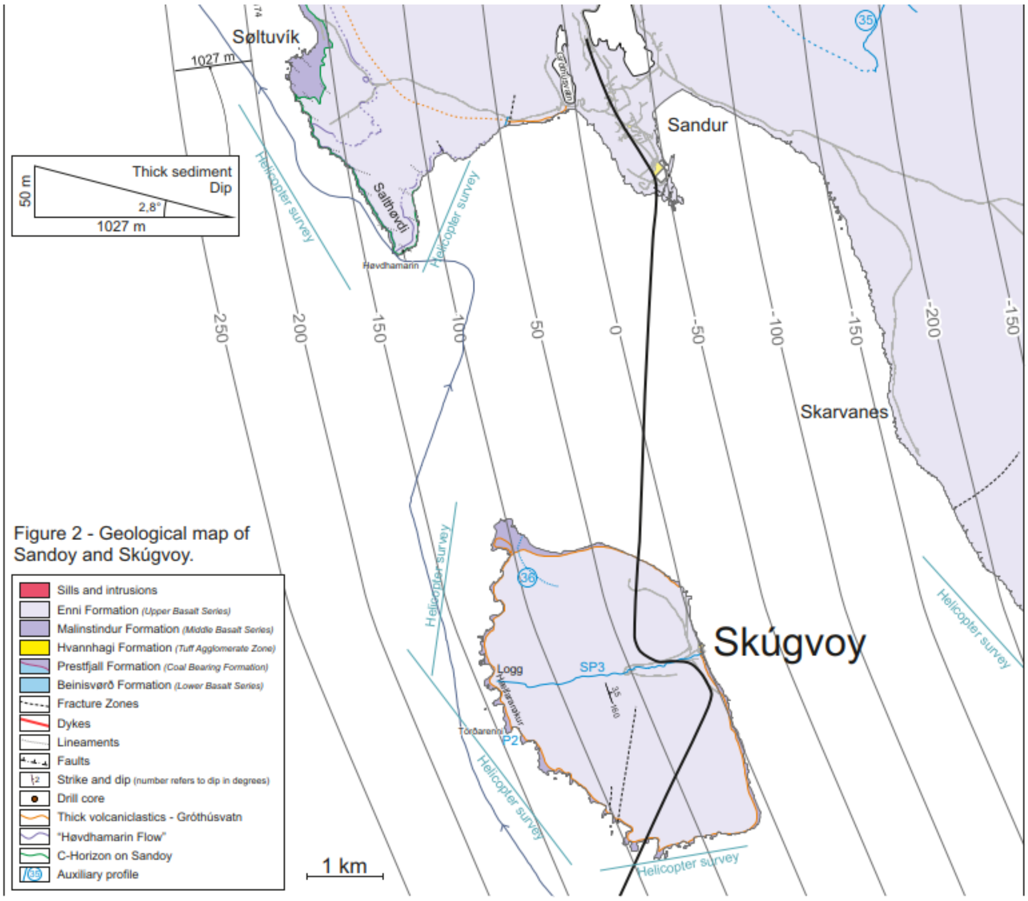


Figure 15. Geological map of Sandøy and Skúgvøy. The contour lines show the level (masl) of the Thick volcaniclastic or Base thick sediment.

Figure 16 shows a longitudinal section along a tunnel line that goes up to Skugvøy. The C-horizon is indicated with a number of question marks. Figure 15 indicates that the C-horizon is deeper on the northern side of Skugvøy than shown in Figure 16, so that the tunnel is above the C-horizon for the northern tunnel.

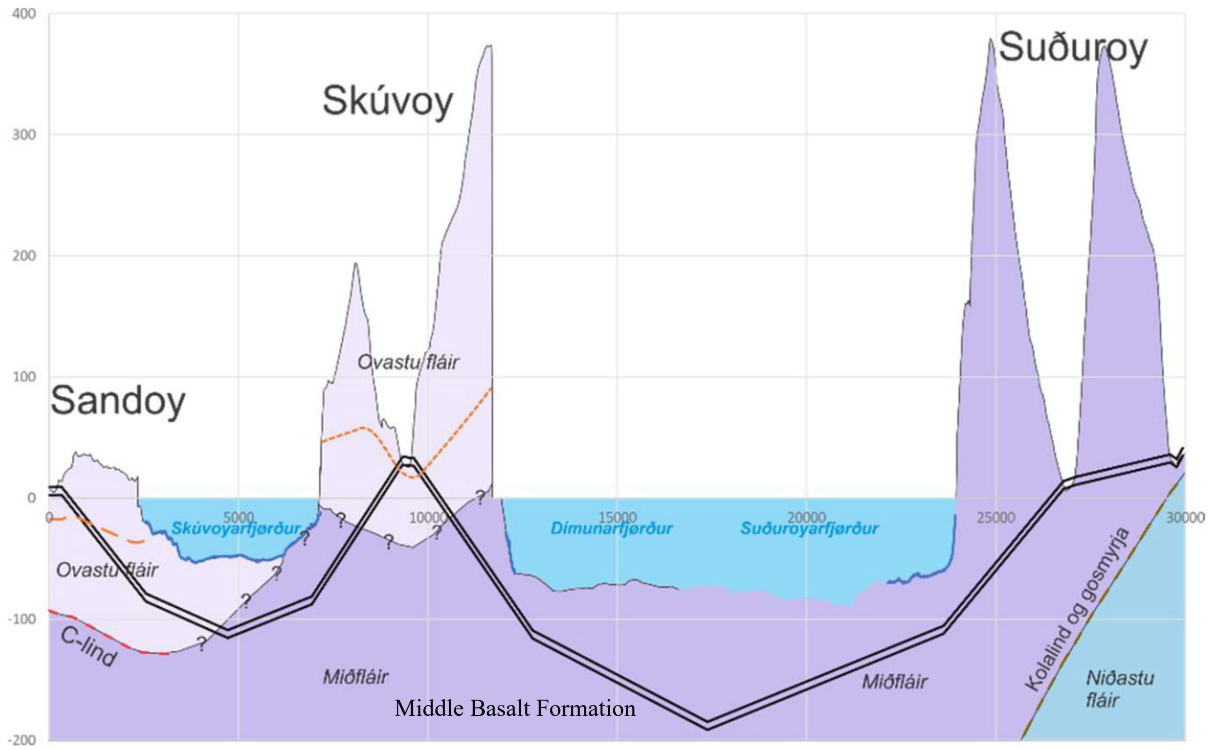


Figure 16. Geology along the tunnel alignment through Skugvøy. Jardfeingi 2017. C-horizon is "C-lind" on the profile. Orange dotted line indicates the Grotusvatn horizon.

The longitudinal profile shows that the tunnel south of Skugvøy is in the Middle Basalt Formation.

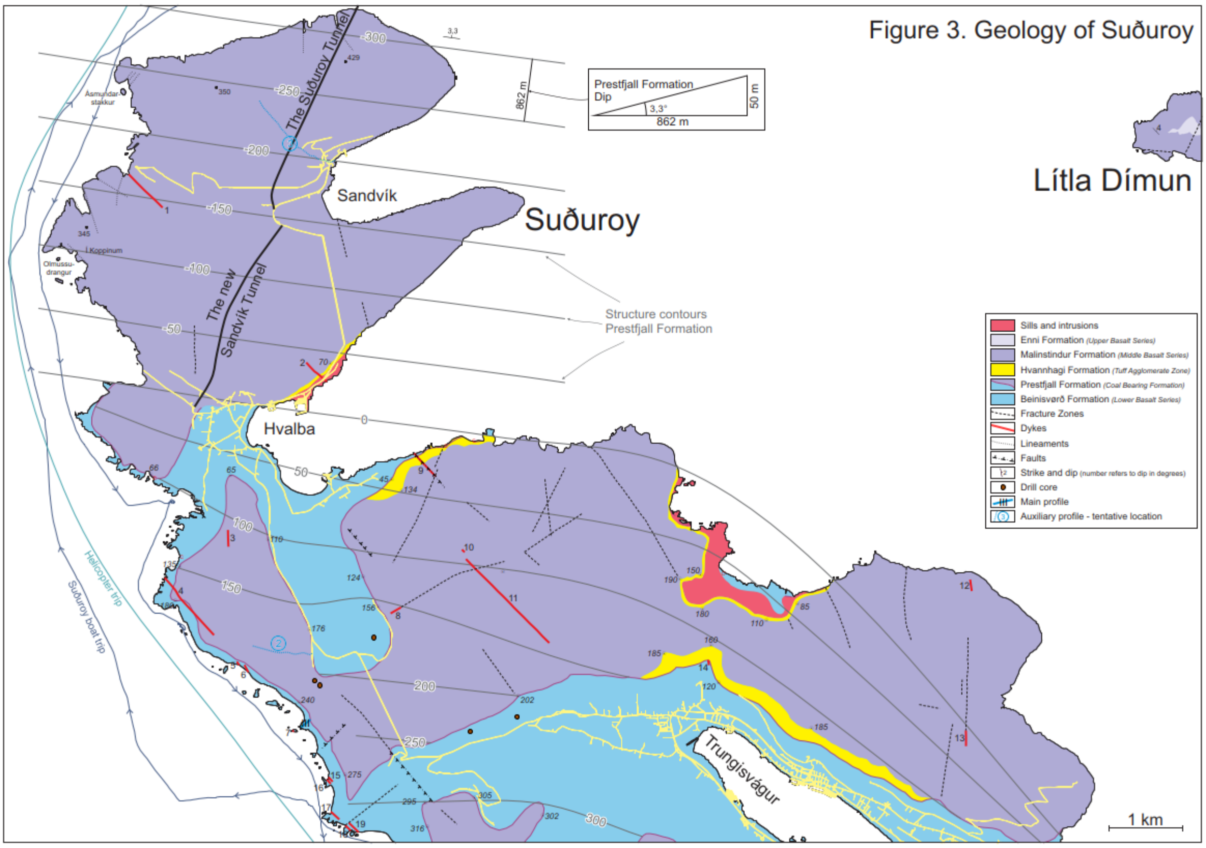


Figure 17. Geology of Sudurøy. The contour lines show the elevation (masl) of the Prestfjall formation, which is the contact between Malinstindur (Middle Basalt Formation) and Beinisvørd formation (Lower Basalt Formation).

5.1.2 Borehole investigation

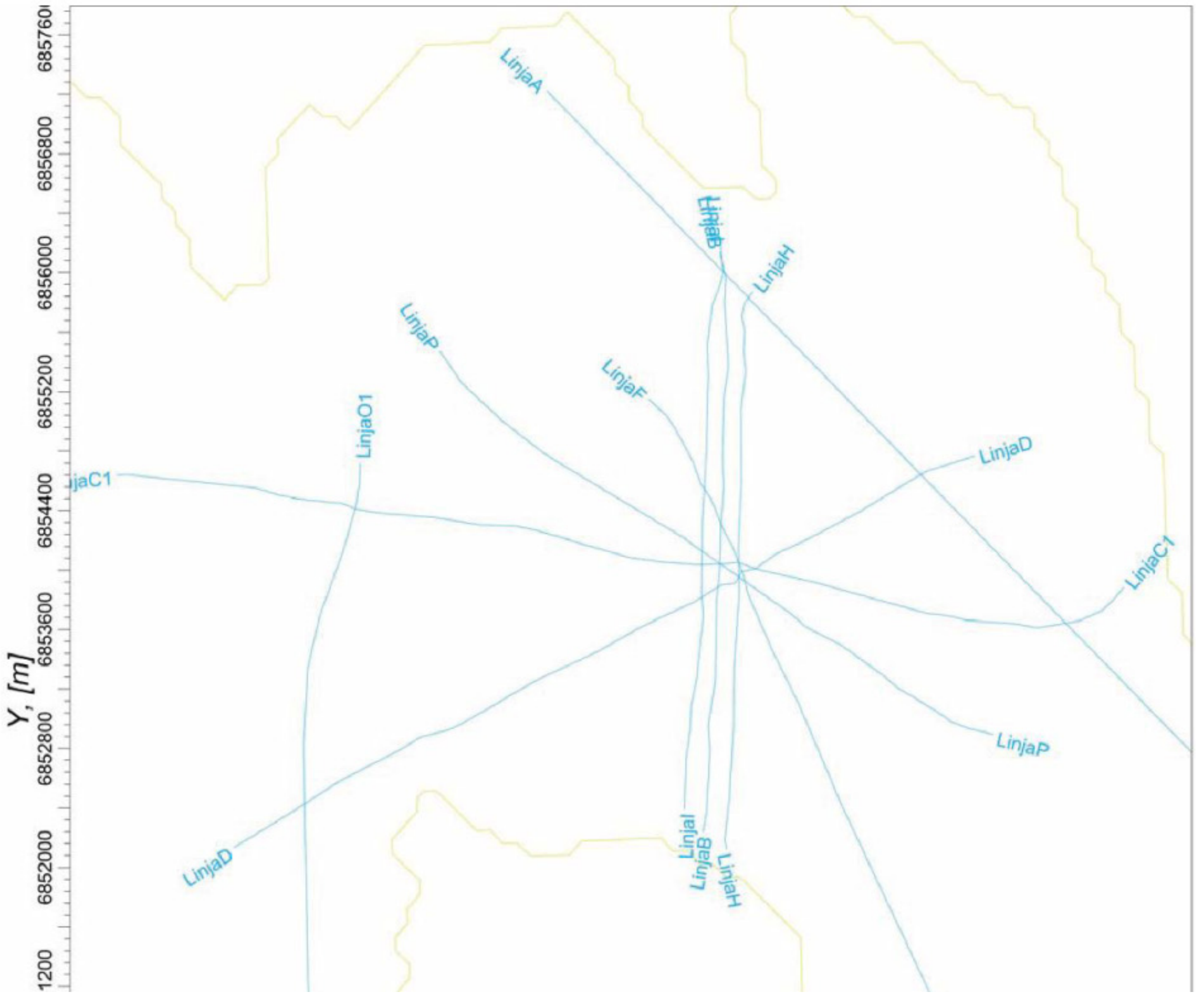
A high number of bore holes have been drilled for investigation purposes both in Sandøy and in Sudurøy. In figures 1 and 10 of the geological report, locations are indicated for more than 30 boreholes. These holes are drilled no deeper than to sea level. No cores have been sampled, so interpretation of formations and horizons must have been done based on logging with optical televiewer, caliper and electric log.

5.1.3 Geophysical surveys

A geophysical survey was performed in July 2020, and the report includes processing of individual seismic lines. The report is not compiling the results into maps since:

- No detail seabed contour map is produced.
- No map is produced showing thickness of soil deposits.

Reflection seismic profiling (2020) has revealed thick soil deposits, up to 50 m, approximately midway between Sandøy and Skugvøy. The thickness has been taken from velocity models derived from refraction seismic analyses of the reflection seismic survey. The profile lines can be seen in figure 18.



LinjaC1

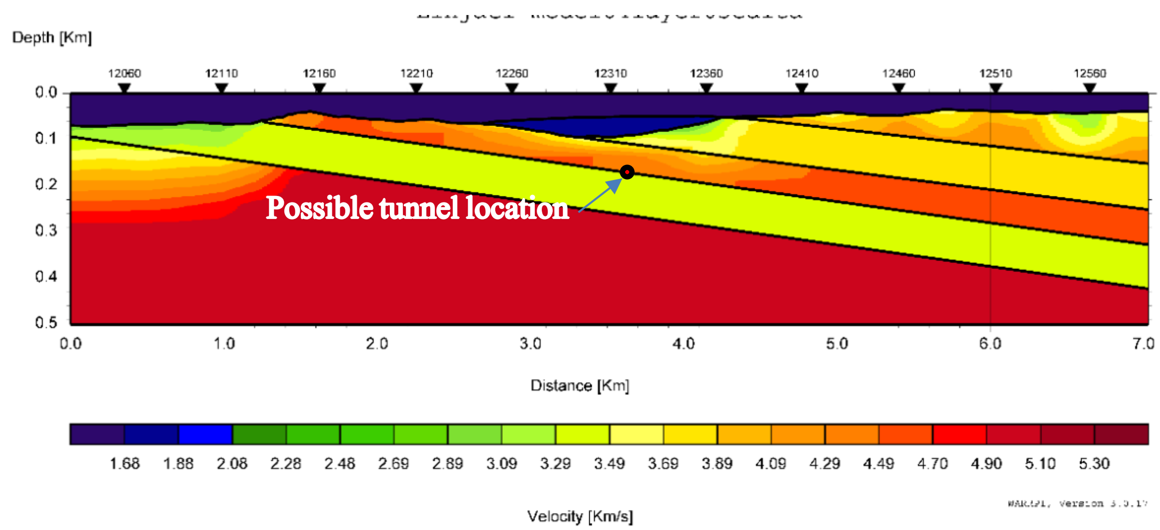


Figure 18. Geophysical survey lines 2020. Section shows Linja C1

In profile LinjaC1, the rock surface is located at approximately 105 mbsl. Thickness of sediments (blue) is close to 50 m maximum.

Jardfeingi (Turid Madsen) has informed (June 2021) that investigation work has been ongoing also during the last year. Figures 20 and 21 show bore holes that have been completed and also holes planned for execution in the present year. No reports of the findings have been produced to date.

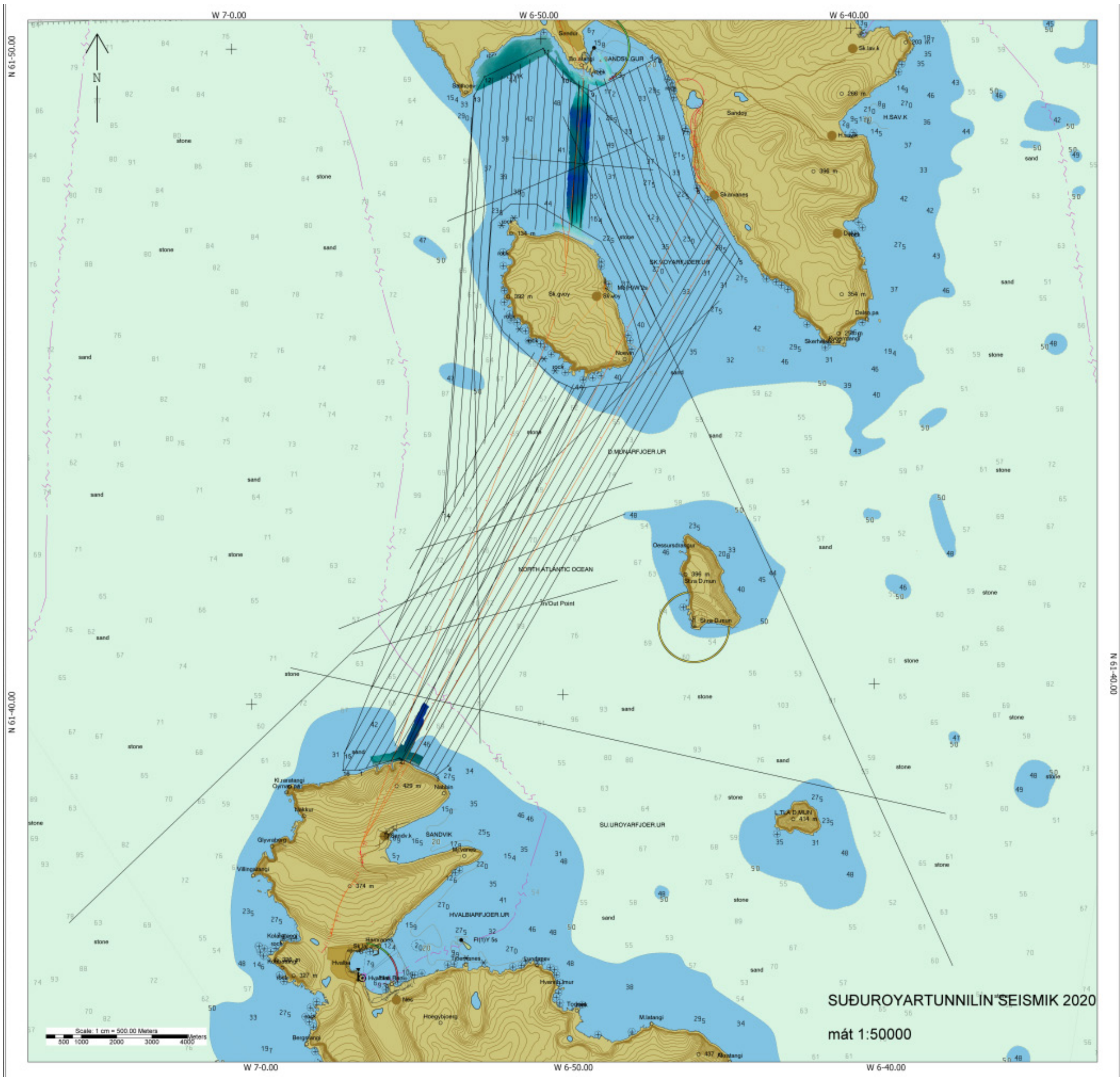


Figure 19. Seabed map of the project area. The systematic, parallel lines are planned lines for multibeam seabed mapping. Other lines are seismic lines from the 2020 survey by Jardfeingi.

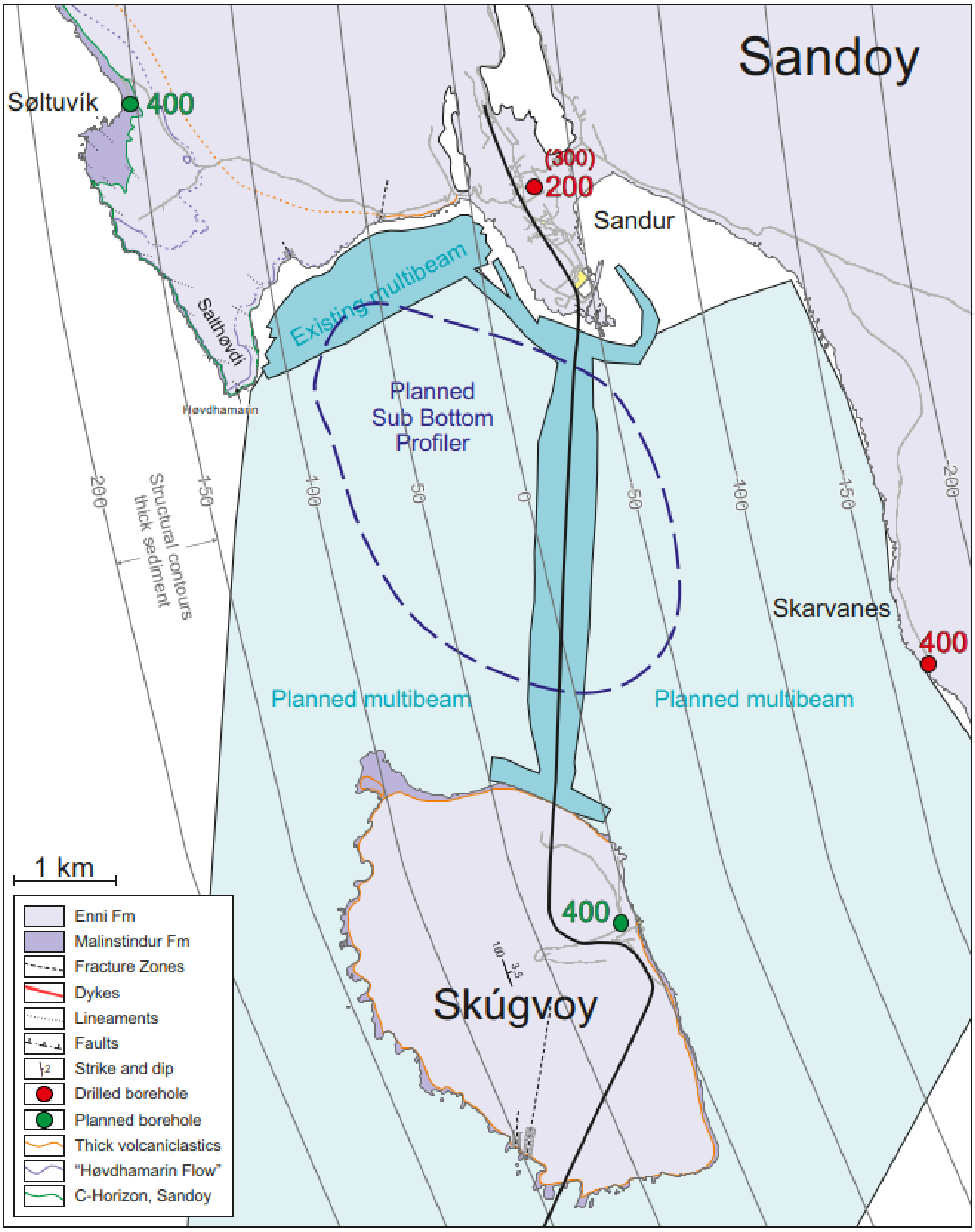
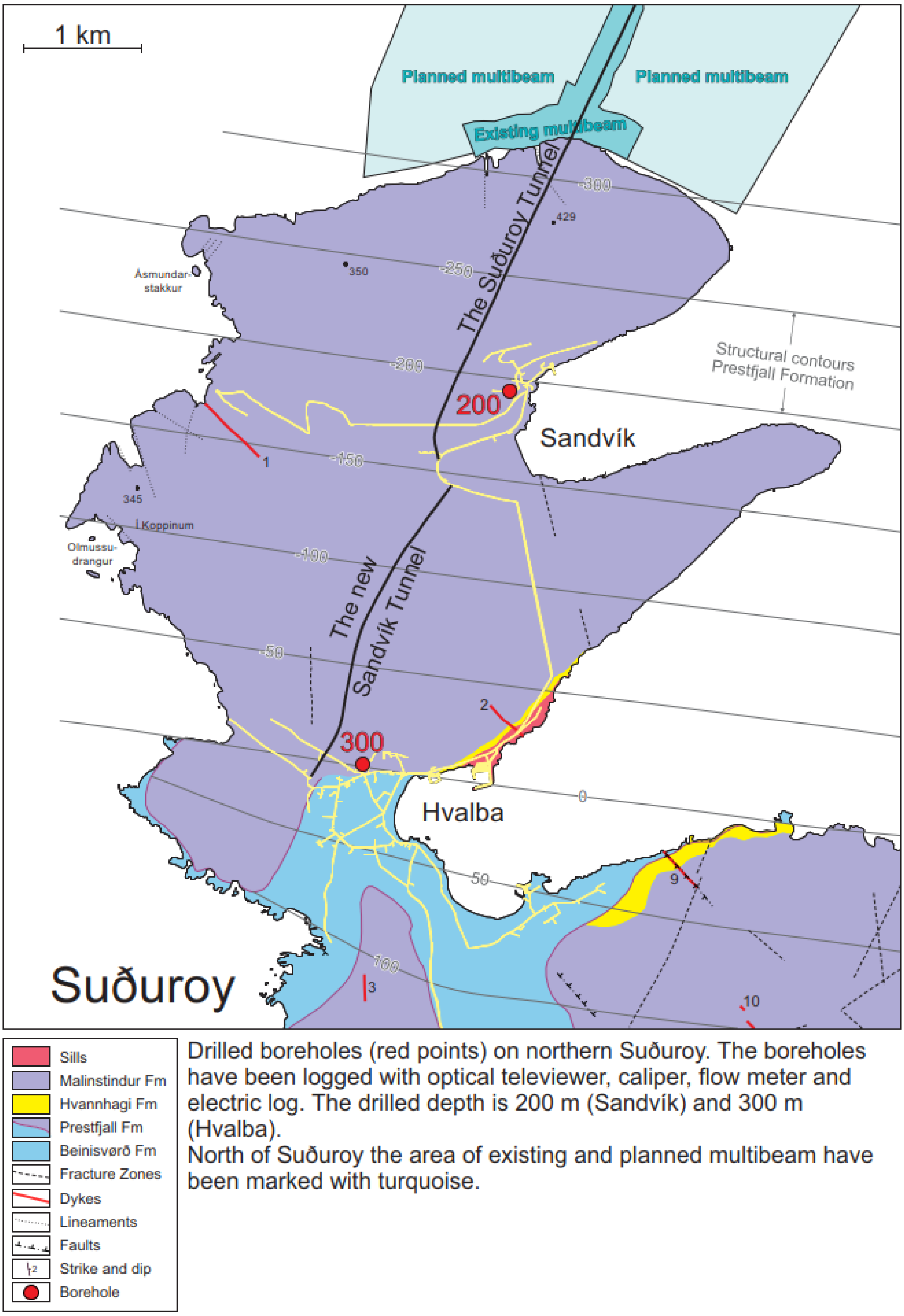


Figure 20. Boreholes performed last year and planned boreholes. Planned multibeam echo sounding and sub bottom profiler.



Drilled boreholes (red points) on northern Suðuroy. The boreholes have been logged with optical televiewer, caliper, flow meter and electric log. The drilled depth is 200 m (Sandvík) and 300 m (Hvalba).
North of Suðuroy the area of existing and planned multibeam have been marked with turquoise.

Figure 21. Boreholes drilled during the last year. Planned areas for multibeam echosounding.

5.1.4 Rock overburden and excavation challenges in the basalt formations

Sea depths are relatively small (< 100 m) in the entire subsea area for the tunnel. This is favourable for the project. In the Skuvøyarfjørdur the seismic investigations have revealed soil deposits up to 50 m in thickness. This is not critical for the project, but top of rock surface has to be verified through new investigations.

A rock overburden of min. 50 m is recommended for all parts of the tunnel. Critical areas may be where the tunnel is close to shore. These areas must be documented well related to rock overburden during the pre-investigation phase.

The basalt in the Middle Basalt Series seems to be generally homogeneous without flows of sediments with open structures. Experience from tunnelling in this Series confirms this. The Upper Basalt Series (above the C-horizon) has numerous flows that represent low quality rock mass and may be challenging both in respect of rock mass stability and ingress of water.

The geological strike of these flows follows roughly the alignment of the tunnel, which may cause tunnel contact with such a weak layer over large distances (up to km). This situation is indicated in Figure 16, which shows the tunnel location close to a horizon.

5.2 Further geological information

The geological report and the geophysical survey have been concentrated on a tunnel alignment in an approximately straight line between Sandur in Sandøy and Suduroy, Option 1. Other alternatives, going further to the east are also possible routes for the subsea tunnel. Also, a straight line west of Skugvøy is possible. Further geological investigations must be planned in a way so that all possibilities are considered, within a defined corridor. The planned corridor of multibeam echo sounding, see figures 19, 20 and 21 define a sufficient corridor for all relevant alternatives, see Figure 1.

Having studied the geological and geophysical reports produced up to now, the parts of the tunnel that will be located in the Middle Basalt Formation is likely to create less problems for the tunnel excavation than a location in the Enni Formation (Upper Basalt Series). The strategy for further geological investigation should therefore be to

- Establish the location (level) of the contact between the Middle and Upper Basalt Series (C-horizon) and
- Get detailed information about the volcaniclastic sediment layers that are recognized in the Upper Basalt Series and are likely to effect tunnel excavation and stability in a negative way.

Location of the planned, new boreholes at Sandur, Skarvanes, Skugvøy and Sandvik are likely to give the geologists a good understanding of the location of the flows. However, no cores will be sampled from these boreholes. It is recommended to make core sampling to 200 mbsl at the locations Sandur and Skugvøy. Quality of sampling increases with diameter of core, especially in zones of poor quality, which are the layers of most importance for tunnelling. It is therefore recommended to have minimum 75 mm core diameter.

Seabed depth and soil deposits on the seabed does not seem to have large impact on the tunnel alignment. However, a detailed and high-quality seabed map is a first priority for the investigations. To achieve min. 50 m of rock cover at all locations along the tunnel, this also requires that thickness of soil deposits have been investigated to a certain level. Table 2 gives the priority and purpose of further geological investigations. Local geologist must follow the investigations and update the information that is gathered. Observations of rock cuts at the different islands must be held up against the new investigations. The cuts at Stora Dimun should be included in this work.

Table 2: Priority and type of further investigations

Pri-ority	Type of investigation	Purpose	Amount of investigation	Presentation of results
1	Mulibeam echosounding	Seabed map, accuracy ±1 m	A corridor wide enough for all alternatives.	Seabed map
2	Sub bottom profiling (boomer, sparker)	Detect and give approximate thickness of soil sediment at sea bottom. Detect anomalies in the bedrock (special flows in the basalt)	Cover corridor of appr. 300 m width on both sides of each alternative route. c/c 50 m of survey lines, crossing profiles c/c 200 m.	Maps showing thickness of soil deposits with contour lines 5 m. Profiles showing soil deposits and special flows in the basalt.
3	Refraction seismics (cables with hydrophones c/c 5 m on sea bottom)	Thickness of soil deposits (±10% of layer thickness). Sound way velocity in bedrock (indication of rock mass quality). Identification of subvertical weakness zones or highly jointed rock	Shall be carried out in critical sections of the tunnel alignment where min. rock cover is questioned. For the selected alignment, a min. of 1000 m of profiles in each fiord should be executed. Normally executed in two rounds; spread profiles first stage and "confirmation" of selected line, last round.	Seabed maps with location of profiles. Section of each profile, scale 1 : 500.
4	Bore without coring	Detection and description of basalt flows and sediment layers	Vertical holes at Sandur, 300 m Skarvanes, 400 m (or into the Middle BL) Skugvøy, 300 m Sandvik, 100 m	Geological borehole logs. Update of the geology and definition of location of the different flows and sediments.
5	Core holes	Sampling of rock and possibly also the sediment layers. Water loss tests in the bore holes at selected sections. Lab. analyses on samples (rock, soil and clay)	Vertical holes at Sandur, 200 m Skugvøy, 200 m	Geological borehole logs. Laboratory results

From the geological investigations it seems that a tunnel location in the lower part of the Upper Basalt Formation, including the C-horizon, may create more problems for tunnelling and rock support than a location in the Middle Basalt Formation. This has most impact on the part of the tunnel from Sandøy to Skugvøy.

If the tunnel is located East of Skugvøy, the tunnel will be located all the way between Sandur and Skugvøy in the Upper Basalt Formation. With a location west of Skugvøy, the tunnel may soon enter the Middle Basalt Formation and continue all the way to Suðurøy if the tunnel does not go up to Skugvøy. As stated in chapter 3.2 it is essential to establish the location of these flows and layers and thereafter consider the consequences for tunnelling and rock support.

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