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Holmestrand Underground Railway Station - Analysis of Groundwater Inflow and Methods for Water Sealing

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Earth Sciences and Petroleum Engineering

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Methods for Water Sealing

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Samandrag

Den planlagde nye stasjonshallen i Holmestrand som er meint å liggje under eit svært basaltplatå synast å gjeve store utfordringar mtp. injeksjon og vass-og frostsikring. Frå injeksjonsarbeid i dei nærliggjande tunnelane Snekkestad og Sjøskogen i tillegg til tilkomsttunnelane til stasjonshallen vil det verte prøvd å gjennomføre ei slags prognose for injeksjonssituasjonen for stasjonshallen.

Undersøkjingar gjennomførde før utbyggjing slik som CPTU, ødometer, piezometrer og elektrisk tomografi har så og seie ikkje avdekt nokre innlekkasjar. Nedbørsmønsteret førar til variasjon i grunnvassnivået avhengig av årstid. Hydrogeologisk vert Holmestrandplatået naturleg drenert av den alt eksisterande vegtunnelen i området som vert i teksten referert til som Holmestrandtunnelen. Holmestrandplatået er bygd opp av søylebasalt, og diverse lausmassar dekkjer toppen. Det finst ei veik sone som kan føre til problem for stasjonshallen, dvs. at injeksjonen vil verte omfattande for å halde på innlekkasjekravet. Vha. ein formel for å rekne ut den forventa innlekkasjen for stasjonshallen ser ein at det vert strenge krav samanlikna med ein vanleg tospors jernbanetunnel.

Injeksjonen vil verte gjennomført som forinjeksjon med eit innlekkasjekrav på $5 \frac{L}{\frac{min}{100 m}}$.

Dersom det møtast på vanskelege geologiske forhold vert kontroll-, ekstra- og etterinjeksjonsskjermer aktuelt å nytte. Vasstaps- og innlekkasjemålingar vert gjennomførde vha. byggjing av demningar og å direkte måle innlekkasjen frå borehol. Alternativ til injeksjon omfattar full utstøyping, osv. Sannsynlege vass- og frostsikringsmetodar vert PE-skum, betongelement. Sprøytbare membranar så som t.d. BASF Masterseal 345 kan også laust vurderast. Sistnemnde metoden har vorte testa i eit frostillaboratorium og i Gevingåstunnelen, der dei viktigaste resultatane er at fryseindeksen ikkje har noko å seie for kor djupt nullisotermen kan trenge inn i berget, og at temperaturmønsteret ikkje er kumulativt. Tunnelresultata syner ingen tydelege lekkasjar. Den sprøytbare membranen BASF Masterseal 345 har også svært høge deformasjonslasteeigenskapar.

Resultat frå injeksjonen syner høgst varierende mønster i samband med geologien, når det vert sett på fordeling av injeksjonsmasse per borehol og trykket per borehol per skjerm, i tillegg til den totale mengda injisert masse og gjennomsnittstrykket per injeksjonsskjerm. I område av tverrsnittet med dårleg bergmasse ser trykket ut til å minke og mengda injisert masse aukar. Innlekkasjekravet kontrollerar ikkje alltid mengda av injisert masse, sidan overdekning og overflateinstallasjonar i tillegg til bergmassen og sprekkjesystem har noko å seie her. 3D-målingar som syner bergspenningsfordelinga kan også vere ei vegleining for injeksjonen, saman med den loddrette sprekkjesituasjonen for stasjonshallen. Vass-og frostsikringsmetoden vert sannsynlegvis anten PE-skum eller betongelement. Forfattaren hevdar likevel at eit alternativ som sprøytbare membranar, t.d. BASF Masterseal 345 syner høvelege og gode verdiar utifrå diverse forsøk, men at det trengst fleire og meir omfattande testar før ein kan bruke det på slik eit prosjekt.

Abstract

The planned new railway station in Holmestrand whose location will be inside a basalt plateau poses challenges concerning the grouting and water and frost protection. Based on grouting works in adjacent tunnels Snekkestad and Sjøskogen as well as the entrance tunnels of the station hall, a loosely attempt to predict the grouting situation for the station hall will be made.

Pre-construction investigations including CPTU, oedometer, piezometers and ERT tomography have hardly revealed any signs of inflows. The precipitation pattern causes seasonal fluctuations in the groundwater. Hydrologically, the Holmestrand plateau is naturally drained by the already existing road tunnel, referred to in this text as the Holmestrand road tunnel. The Holmestrand plateau consists of column basalt and various soil masses cover its top. There exists a weakness zone that may cause problems for the station hall, meaning that grouting will have to be extensive to maintain the inflow criterion. By the aid of a formula of calculating the expected inflow for the station hall, a strict requirement emerges when comparing this to a standard double-lane railway.

Grouting works will take place at an inflow criterion of $5 \frac{L}{\frac{min}{100 m}}$ by pre-grouting fans. Should difficult geological conditions be encountered, control fans, extra fans and post-grouting fans may be applied. Measuring water loss and inflow may happen, both by construction of dams in the tunnel and directly measuring the inflow from boreholes. Alternatives to grouting include concrete lining, etc. Likely water and frost protection solutions will be PE foam plates, concrete elements. Sprayable membranes such as the BASF masterseal 345 may also be subjected to consideration. The latter has been tested in a frost laboratory and the Gevingåsen tunnel, where the most important conclusions are that the freezing index has no effect of how deep the zero-degree isotherm penetrates into the rock mass and that the temperature pattern is not cumulative. The tunnel results show no significant leakages. Also, the BASF Masterseal 345 shows high deformation load capacities.

Results from the grouting show highly varying trends according to the difference in geology, both when studying the distribution of grout amounts and the pressure per borehole for each grouting fan and also the total amounts of grout mass and the average pressure per grouting fan. In areas of the cross-section with low-quality rock the pressure seems to decrease and the amounts of grout mass increase. Also, the inflow criterion does not always control the amounts of injected mass, as the overburden and surface installations, as well as the rock mass and joint system situation play an important role. 3D measurements giving the rock stress distribution may also serve as guidelines for the grouting works, as will the vertical joint situation of the station hall. The water and frost protection solution is likely to be either the PE foam plates or the concrete elements. The author argues that an alternative solution like sprayed membranes, such as the BASF masterseal 345 shows decent qualities but requires more testing before being applied at such a project.

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1 Introduction

1.1 General

The Norwegian National Rail Administration is currently hosting several new railway projects through the county of Vestfold in Eastern Norway. The long-term idea is to build a high-speed railway from Oslo to Kristiansand. As a part of this project, several new tunnels are being excavated. One of these tunnels, located in the town of Holmestrand, will contain the new railway station which will be a giant station hall inside a basalt plateau adjacent to the town centre of Holmestrand.

This master's thesis is based on a project thesis carried out by the same author during autumn of 2011. The title of the project thesis is Analysis of Engineering Geological Conditions for the Holmestrand rail Station with main Emphasis on Water inflow. Description of engineering geological investigations and an assessment of the inflow situation for the new rail station in Holmestrand have been carried out in that assignment. (Ryningen 2011)

The focus in this text will be oriented towards the water inflow situation for the new Holmestrand railway station in relation to the grouting and water and frost protection situations. Currently, the project is in the early stages of excavation; the first blast was performed on December 21st, 2011. The entrance tunnels are being excavated prior to the railway station itself. Since the station hall is not likely to be excavated within the time limit set aside to this assignment, the grouting and inflow conditions for two adjacent tunnels, the Snekkestad and Sjøskogen tunnels, along with the entrance tunnels of the station hall will be studied. The idea is then, on basis of the grouting works already carried out in the adjacent tunnels, to make a prediction for the grouting in the station hall. See section 1.2 for more information regarding the latter. The inflow criterion of the station hall is $\leq 5 \frac{L}{\frac{min}{100 m}}$.

(Jernbaneverket 2011) For means of comparison, different methods of both grouting and water and frost securing will be discussed. To the author's knowledge, the newly built tunnel at Gevingåsen contains two different methods of water and frost securing support. Experiences from this project will also be discussed, before a prediction of the water and frost protection for the station hall will be attempted. The practical part of this master's thesis will be a stay in Holmestrand to collect data from the performed grouting and also an excursion to the Gevingås tunnel.

1.2 Location and description of the project

The entire project consists of 5 different tunnels, going from north to south. The tunnels directly connecting to the station hall are the Sjøskogen tunnel in the north and the Fibo tunnel in the south. Excavation works on the latter tunnel are at a very early stage, thus the need to study the southernmost tunnel of the entire project which connects with the Fibo

tunnel, the Snekkestad tunnel. This tunnel has been excavated from an entrance tunnel and the main tunnel part has at the time of writing recently been reached. The Snekkestad tunnel, along with the Fibo and Sjøskogen tunnels traverses through the same rock mass as the station hall. Figure 1 shows the location of Holmestrand in Norway.

The reason why the new railway station is being excavated inside rock mass is mainly due to strict geometrical requirements which force the railway to be built in a tunnel. Also, the beginning and end locations of the new railway have already been determined and cannot be moved.

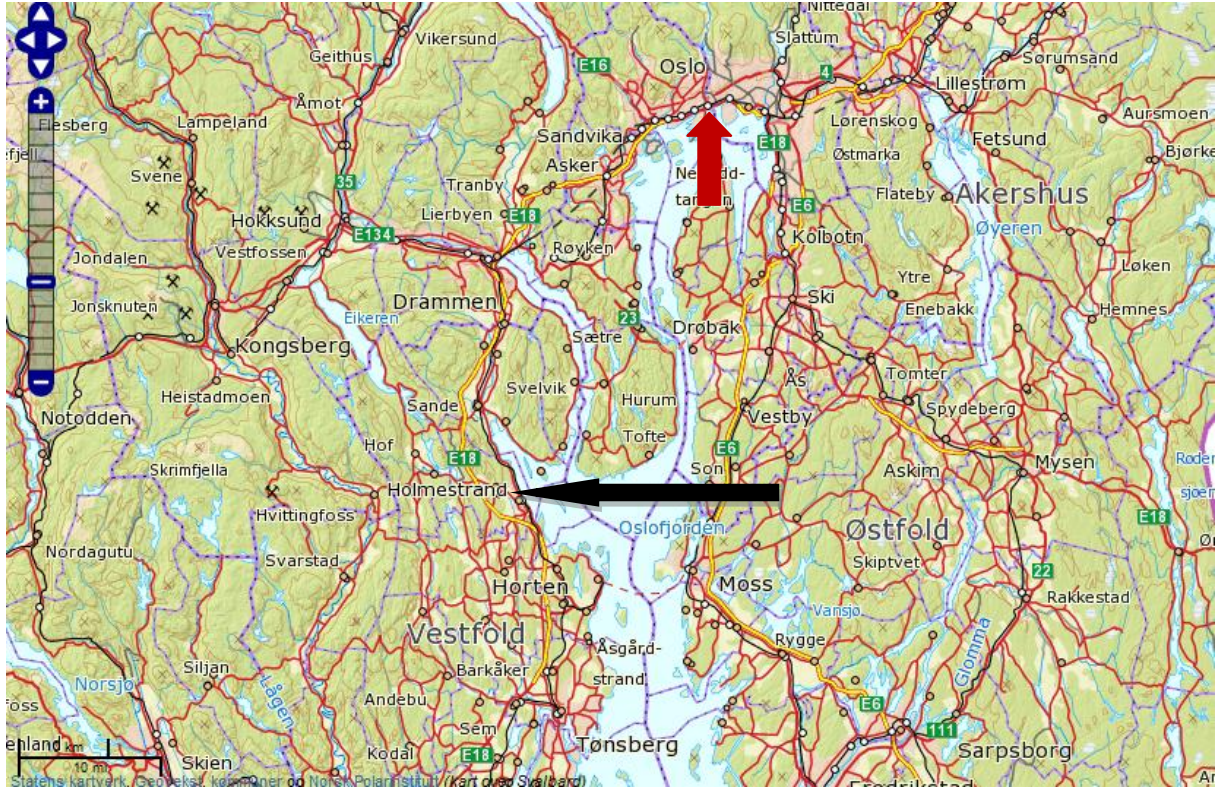


Figure 1. The location of Holmestrand in Norway (black arrow), finding itself southwest of Oslo (red arrow). (Kartverket 2012)

An overview of the locations of the various tunnels which are to be studied can be seen on figure 2. Note that the Fibo tunnel is located between the station hall and Snekkestad.

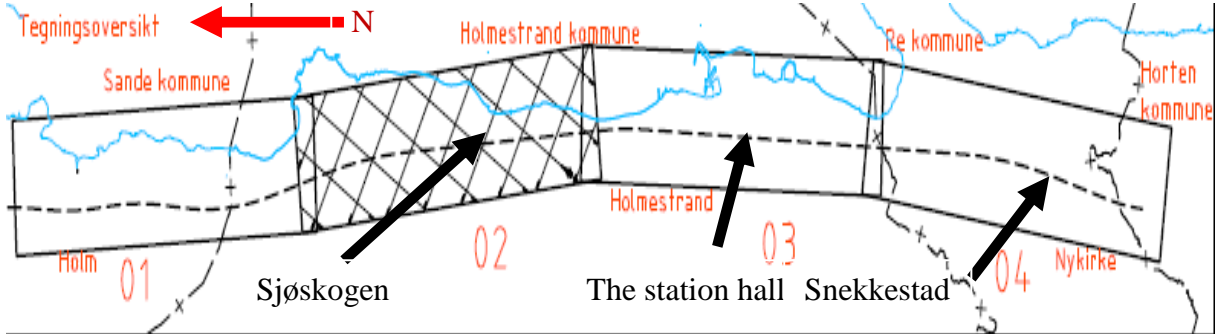


Figure 2. The tunnels studied in this text at their respective locations. (Jernbaneverket 2011)

1.2.1 The Sjøskogen tunnel

Constituting the Sjøskogen tunnel project are the main tunnel part itself and an entrance tunnel which will later on serve as an escape tunnel, named R4. The location of this project can be seen on figure 3. The inflow criterion for both the R4 tunnel and the main tunnel part is $5 - 10 \frac{L}{\frac{min}{100 m}}$. (Jernbaneverket 2011) Appendix A shows an engineering geological length profile of the site, while appendix B contains an engineering geological map of the surroundings.



Figure 3. Location and sketch of the R4 escape tunnel and the main tunnel part of the Sjøskogen project (red dotted lines). (Jernbaneverket 2011)

1.2.2 The Snekkestad tunnel

The Snekkestad project consists of the R13 escape tunnel which at the time of writing serves as an access tunnel for the tunnel works. The main tunnel part is in its early stages. Figure 4 shows a sketch of the Snekkestad area. Inflow criteria for the R13 tunnel at Snekkestad vary, and can be seen in table 1. For the main tunnel part focused on herein, the inflow criterion is $\leq 5 \frac{L}{\frac{min}{100 m}}$. (Jernbaneverket 2011) Appendices C and D show an engineering geological length profile of the tunnel and an engineering geological map, respectively.

Table 1. Inflow criteria for the R13 tunnel at Snekkestad. (Jernbaneverket 2011)

Section (m)	Length (m)	Inflow criterion ($\frac{L}{min}$) 100 m
0 – 175	175	≤ 5
175 – 330	155	5 – 10
Crossroad area next to main tunnel part	15	≤ 5



Figure 4. The Snekkestad project, composed of the R13 entrance tunnel and the main tunnel part (red dotted lines). (Jernbaneverket 2011)

1.2.3 The station hall and the entrance tunnels

An overview of the Holmestrand plateau with the location of the entrance tunnel to the station hall and the approximate tunnel line traversing below the plateau can be seen on figure 5. Appendix E displays an engineering geological length profile of the station hall and appendix F shows an engineering geological map over the surroundings of the station hall.



Figure 5. An aerial overview of the current railway station with the entrance tunnels, R7 and R8 (yellow arrows) and the approximate tunnel line (yellow dotted line). (Sagen 2011)

A sketch of the cross-section of the station hall can be seen on figure 6.

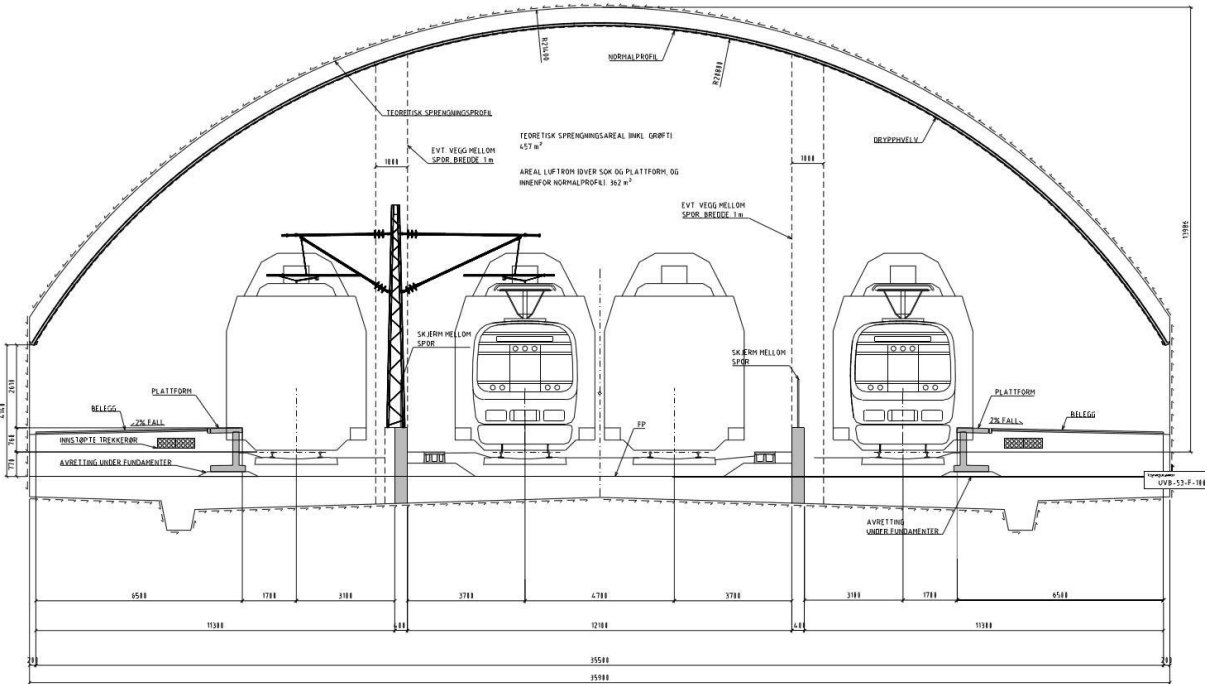


Figure 6. A sketch of the cross-section of the station hall. (Sagen 2011)

There will be two entrance tunnels to the station hall, where one of the entrance tunnels will start inside the Holmestrand road tunnel which is described later in this text.

The station hall is planned to be 16 m tall, 34,5 m wide and 260 m long. It will have a cross-section area of approximately 487 m². (Kveldsvik, Grøneng et al. 2011) The station hall will have an overburden altering between 50 – 170 m. (Norconsult 2010)

The layout of the station hall including entrance tunnels R7 and R8 (from left to right) is visualised in figure 7. Also, to the far right is an escape tunnel named R9. The inflow criterion for all tunnels; the station hall itself and tunnels R7, R8 and R9 is $\leq 5 \frac{L}{\frac{min}{100 m}}$.

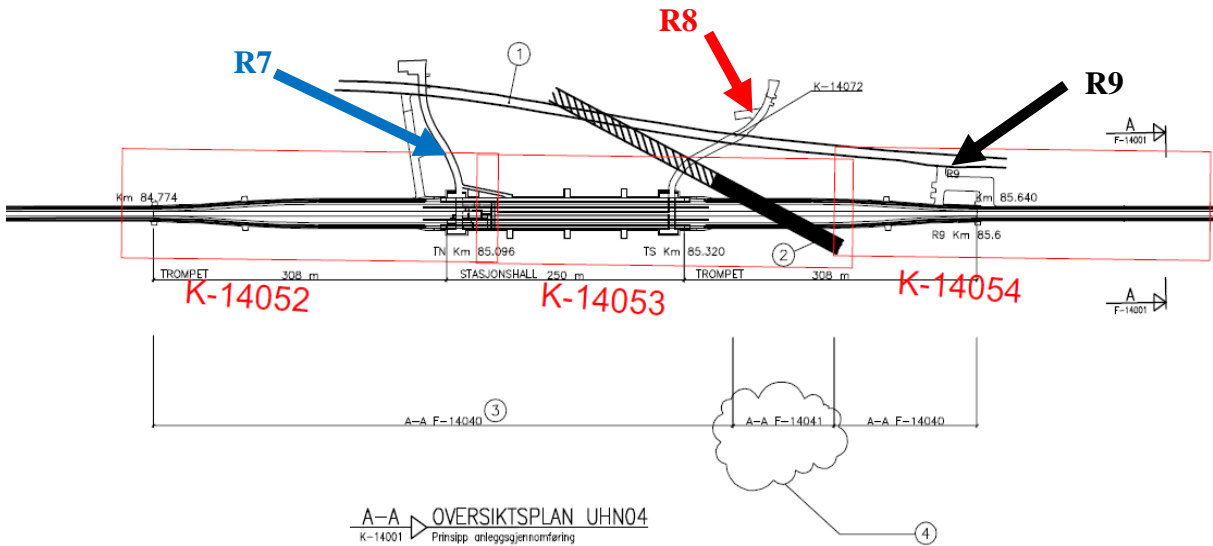


Figure 7. Vertical overview of the layout of the station hall, including the entrance tunnels R7, R8 and the escape tunnel R9. (Jernbaneverket 2011)

Additionally, there already is another tunnel in the vicinity of the planned station hall. The road traversing parallel to the Holmestrand plateau enters a road tunnel, referred to in this text as the Holmestrand road tunnel. This tunnel was first built in 1981 and was excavated through virtually the same rock mass. (Ryningen 2011) Specifics about this tunnel will not be discussed in this text, but for more information the source referred to can be examined.

Figure 8 shows the location of the Holmestrand road tunnel in comparison with the station hall.

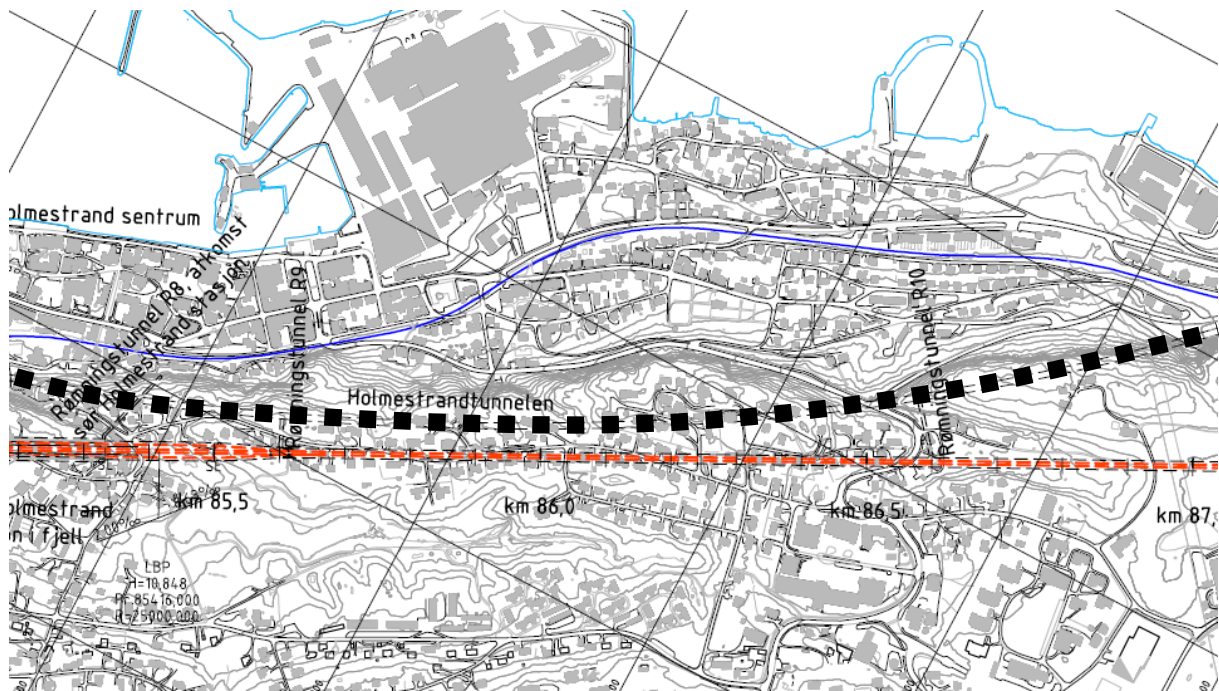


Figure 8. The Holmestrand road tunnel (black dotted line) across the town of Holmestrand, with the station hall and the railway tunnel (red dotted line) next to it. (Jernbaneverket 2011)

2 Pre-construction investigations

2.1 General

Prior to excavating the tunnel and the station hall, several engineering geological investigations have been carried out. These include core drillings, Lugeon tests, hydraulic fracturing, 3D stress measurements, etc. (Ryningen 2011) Most of these tests already have been discussed, in the following section investigations related to soil masses and groundwater conditions in particular will be reviewed.

Figure 9 shows the area where several different investigations have been performed.

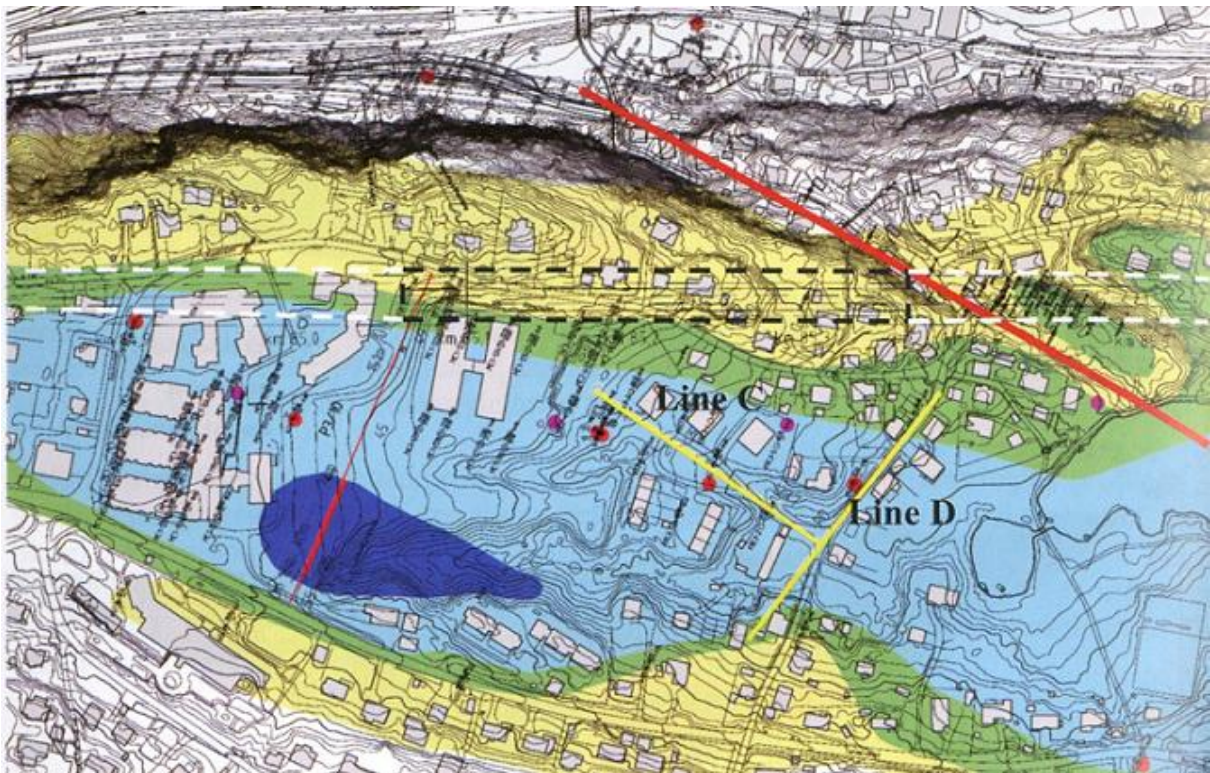


Figure 9. The area above the station hall with various investigations; yellow lines indicate resistivity measurements, red dots indicate pore pressure measurements, purple dots indicate wells in the rock mass. The red solid line at the upper right indicates the weakness zone intersecting the very edge of the station hall. The station hall is marked by the black, dotted lines. (Langford, Kveldsvik et al. 2011)

2.2 CPTU and oedometer measures

CPTU measures have been performed on top of the Holmestrand plateau at the same locations as the pore pressure measurements to assess the undrained shear strength, s_{ua} . The method itself consists of a cylindrical pressure device containing a conical top. The device is then being penetrated into the soil at a constant rate, where forces acting on the conical top (q_t), the friction forces ($\tan \phi$) as well as the pore pressure in the soil (u) are measured. From these measurements it is possible to assess the undrained shear force of the soil masses.

As a support test, the oedometer has been utilised. In such a test pressure is exerted vertically on the test specimen, under drained conditions. The vertical stress is measured, as is the pore pressure, the latter measured at the bottom of the sample. From this test, it is possible to determine p'_c , permeability and general compression abilities. (Langford, Kveldsvik et al. 2011)

Figure 10 displays both the CPTU and the oedometer equipment, respectively.

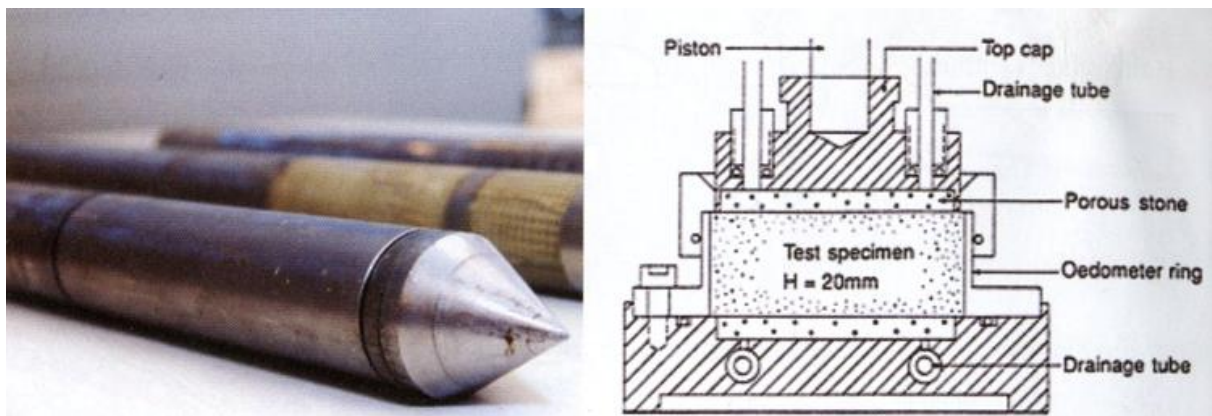


Figure 10. CPTU cylinder (left) and oedometer test (right). (Langford, Kveldsvik et al. 2011)

The determination of s_{ua} stems from various correlations established which are based on comparison of results from advanced laboratory tests at high-quality block samples of clay. It is also possible to assess the pre-consolidation pressure of the clay by the aid of relations between the unconsolidated shear pressure of the clay and over consolidation rate, OCR ;

$$(I) \quad OCR = \frac{p'_c}{p'_0}$$

where

p'_c = pre-consolidated pressure (the maximum pressure sustained in the past) (MPa)

p'_0 = in-situ pressure (the current pressure) (MPa)

Utilising (I), the desire is to assess p'_c . To do so, the following equation may be suitable:

$$(II) \quad s_{ua} = \frac{q_t - \sigma_{v0}}{N}$$

where

q_t = force acting at the top of the cone (kPa)

$\sigma_{v0} = p'_0$ = in-situ pressure (kPa)

N = Bearing capacity

By the aid of (II) it is possible to visualise the s_{ua} as a function of depth. The OCR is known from (I). (Langford, Kveldsvik et al. 2011)

Figure 11 shows results from several boreholes in which the CPTU and oedometer tests have been executed.

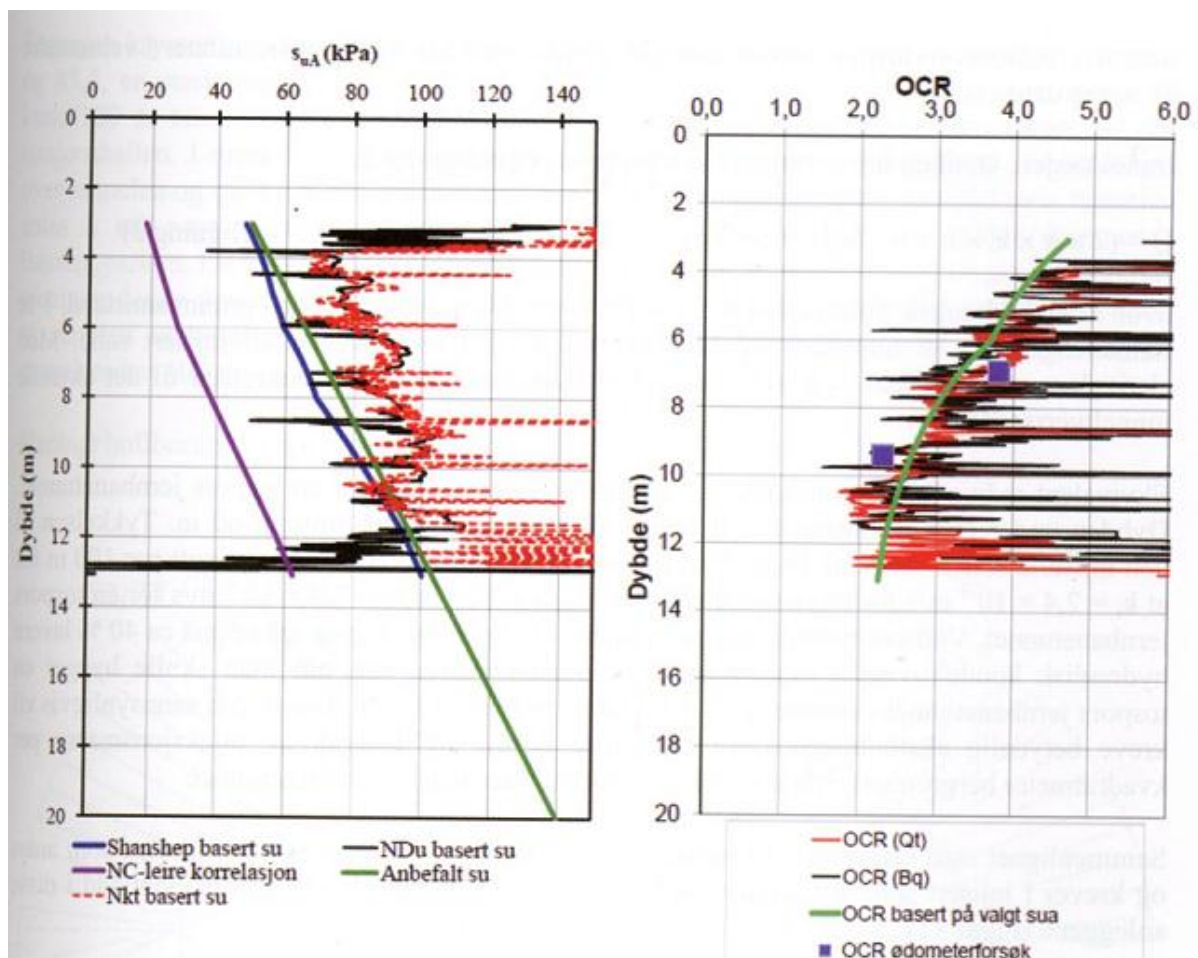


Figure 11. Results from the CPTU measurements above the station hall; s_{ua} (left diagram) and OCR (right diagram) as functions of depth. (Langford, Kveldsvik et al. 2011)

At the sinistral diagram in figure 11 the purple line indicates the trend of s_{ua} if there had been no pressure in the past (normal consolidated clay), the red dotted line is the actual s_{ua} based on q_t and the black line indicates the pore pressure reduction response. As seen on the diagram,

the clay measured is clearly pressured in the past, showing a difference of 20 – 30 KPa. The green, linear line is the recommended value of s_{ua} .

The dextral diagram shows the OCR , evaluated from the interpreted s_{ua} in the CPTU measurements, as a green line. OCR as a function of q_t is visualised as a red line, with the black line being OCR as a function of B_q , the pore pressure relationship which is defined as $\frac{\Delta u}{q_n}$. The purple squares are results from samples from the same borehole performed in an oedometer. (Langford, Kveldevisk et al. 2011)

2.3 Electrical resistivity tomography

There have been conducted electrical resistivity tomography (ERT) measures above the station hall on the Holmestrand plateau. To map the thickness of the soil layers down to the rock mass, refraction seismic was thought to constitute the only means at first. Due to technical reasons like the lack of mapping of electrical wires in the area, the main emphasis has been put on 2D resistivity measures. As a means of controlling the seismic works a seismic profile was also mapped using ERT. The results yielded from both methods were in principle the same.

Locations of the two ERT profiles can be seen on figure 9. The results from the two profiles are presented on figure 12.

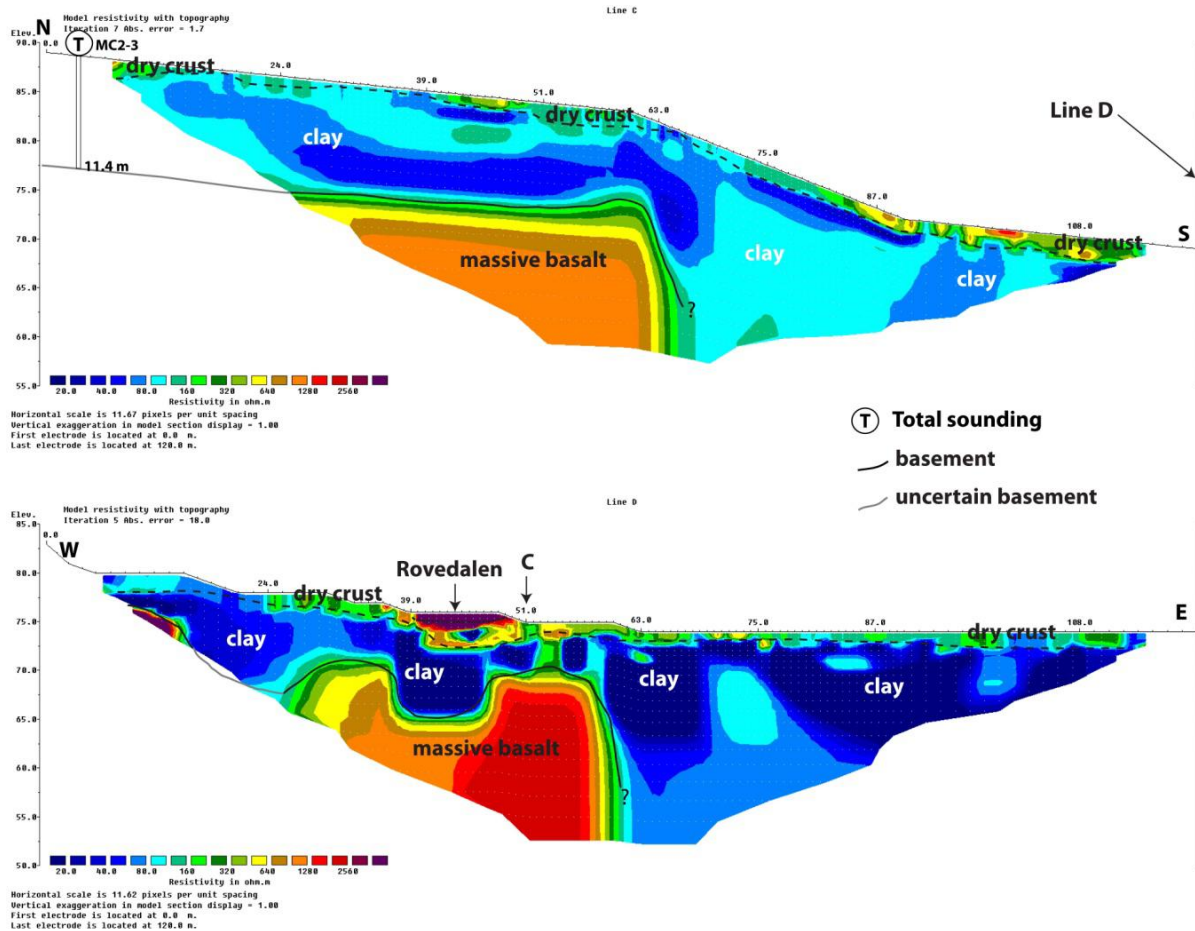


Figure 12. Results from the ERT measurements on the Holmestrand plateau. Profile C on top, profile D at the bottom. (Bazin, Pfaffhuber et al. 2011)

In these figures the various colours indicate different abilities and properties of the rock masses. Blue indicates poor quality of the rock mass which contains low resistivity (0 – 10 Ω m) while red indicates good quality with a high resistivity (16.000 – 20.000 Ω m). (Ganerød, Dalsegg et al. 2009) Studying the colours, it appears as if the basalt quality ranges from fair to good in these two profiles.

Regarding the RMS of the profiles, it is very good for profile C (2%) and fair for profile D (18%). The latter result probably stems from the fact that the measure was taken at a rainy day, being affected by changes in the surface resistivity because of water saturation variations and dynamic changes in the temperature. Highly anomalous IP measurements (which unfortunately are not available) in profile D support this assumption.

The dry crust layer is possible to be detected in both of the profiles. At the bottom of the layers, an increase in resistivity allows interpretation of the bedrock depth to be 12 – 20 m in profile C and 7 – 20 m in profile D. (Bazin, Pfaffhuber et al. 2011)

2.4 Pore pressure measurements

2.4.1 General introduction

The pore pressure measurements have been performed using piezometers which have been inserted into several boreholes throughout the area. This method constitutes an automatic way of logging the pore pressure changes in the soil masses over time as the tunnel works progress. The idea is to note differences in the pore pressure before and after the excavation took place and compare these with natural changes, affected by seasons, precipitation periods, etc.

In its simplicity, the piezometer utilised to assess the pore pressure in the soil mass is a cylinder which logs the level of the groundwater in the borehole and calculates the pore pressure, σ , using the following formula:

$$(III) \quad \sigma = \rho \cdot g \cdot h$$

where

ρ = density of the soil mass ($\frac{kg}{m^3}$)

g = acceleration of gravity ($\frac{m}{s^2}$)

h = height of the groundwater in the borehole (m)

By noting the level of the groundwater below the terrain surface the pore pressure can be calculated using (III). However, for practical reasons the pore pressure measurements are rather given in metres. When presented graphically this also gives a visual image over the variations and changes in the groundwater level. Thus is the pore pressure presented as time passes.

There are in total 8 piezometers which are relevant for the pore pressure changes due to the excavation of the station hall. The locations of these are presented in figure 13.

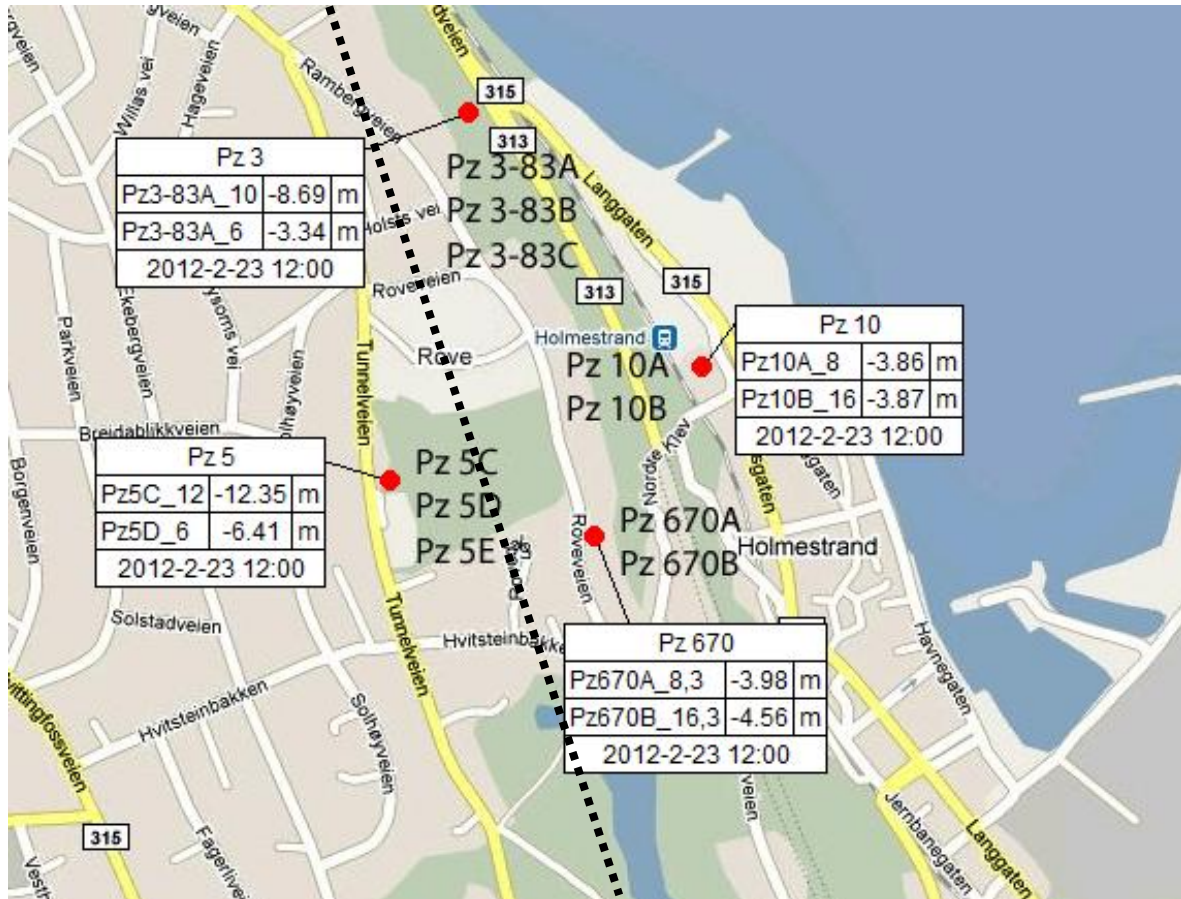


Figure 13. Locations of the piezometers around the station hall. The Pz 10 group is not placed on top of the plateau and is therefore not relevant for discussion. The black, dotted line indicates the tunnel axis. (Jernbaneverket 2011)

As seen on figure 13, Pz 5, Pz 670 and Pz 3–83 are the groups of piezometers which are most relevant for evaluation. Various graphic presentations of several of the boreholes within each group will be presented. By studying different time intervals of each borehole, the idea is to spot possible sudden changes in the groundwater level.

2.4.2 Measurements over one year

Pore pressure variations over one year have been logged and are presented for piezometers Pz 3–83A, Pz 5C, Pz 670A in figures 14 through 16. Note that the tunnel excavation works at the time of registering have been going on for approximately 2 months. Unfortunately, due to technical reasons, it is not possible to obtain data from the piezometer measurements solely prior to the excavation works.

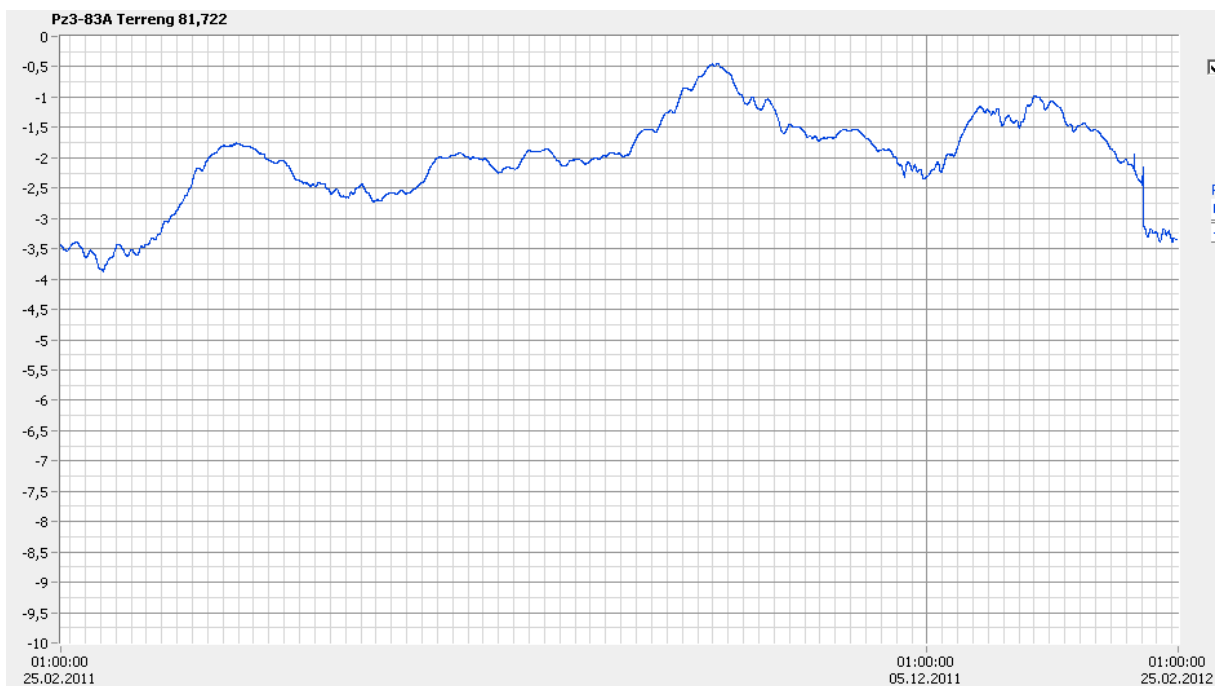


Figure 14. Piezometer measurements for Pz 3–83A. Measurements range from 25.02.11 – 25.02.12. (Jernbaneverket 2011)

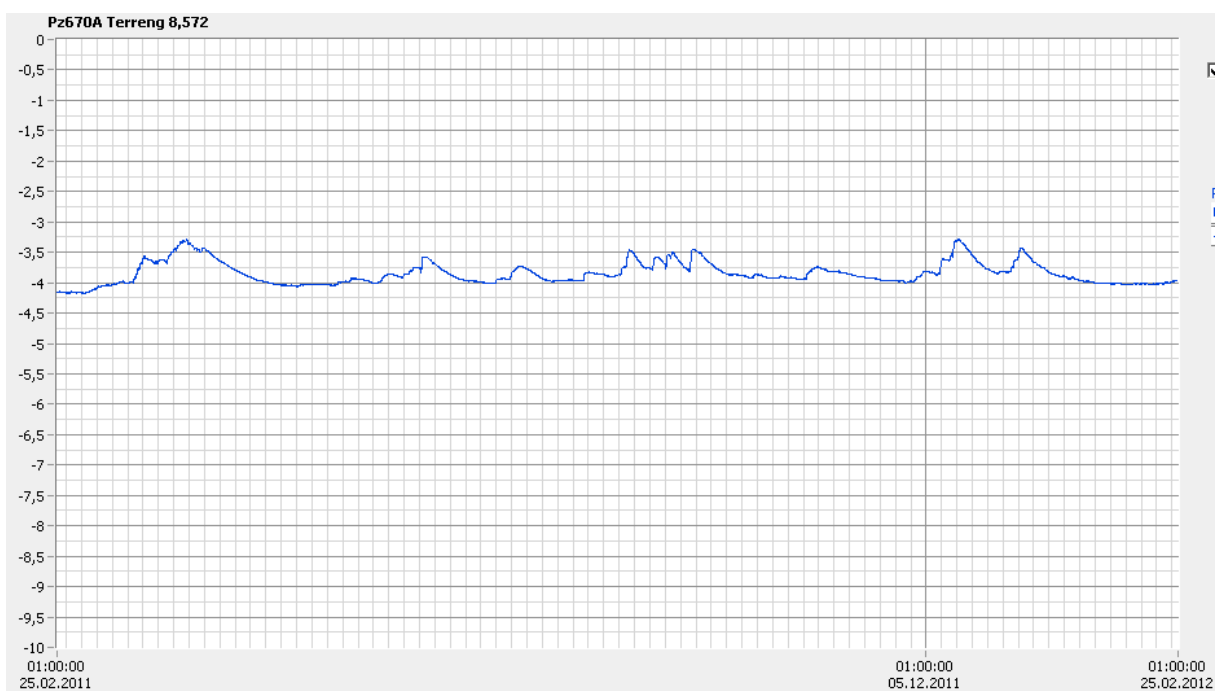


Figure 15. Piezometer measurements for Pz 670A. Measurements range from 25.02.11 – 25.02.12. (Jernbaneverket 2011)

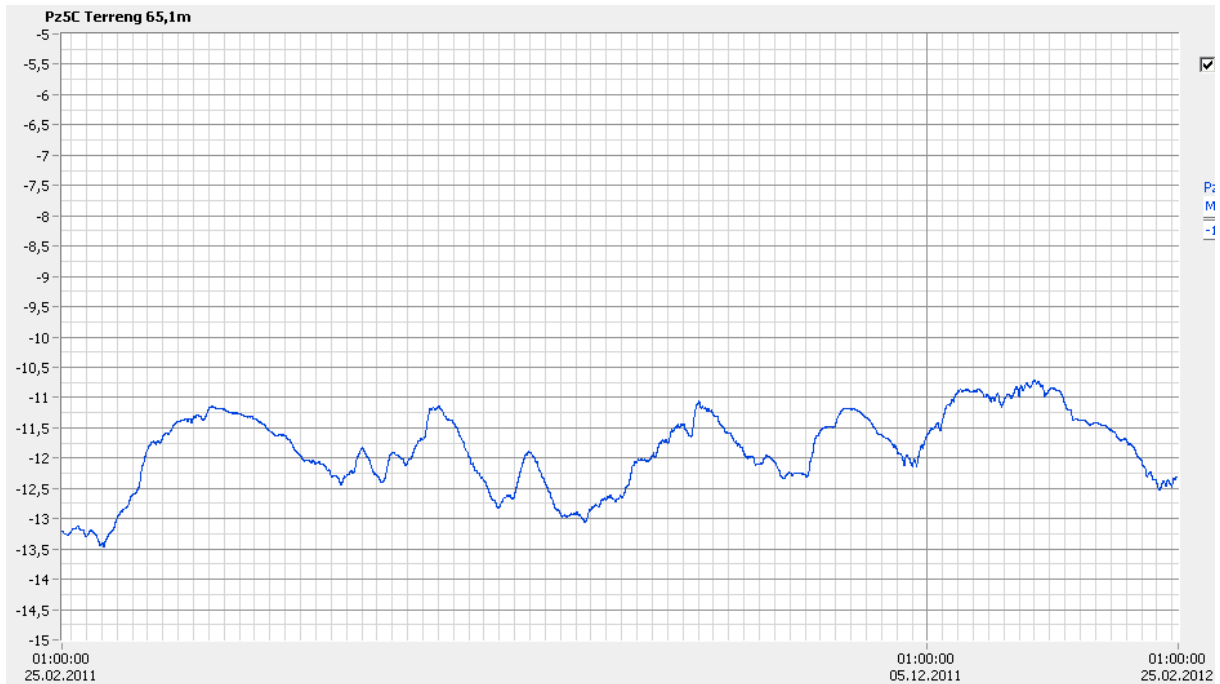


Figure 16. Piezometer measurements for Pz 5C. Measurements range from 25.02.11 – 25.02.12. (Jernbaneverket 2011)

2.4.3 Measurements over 2 months

Results from the latest 2 months at the time of writing for Pz 5D, Pz 3–83B and Pz 670B are graphically presented in figures 17 through 19.

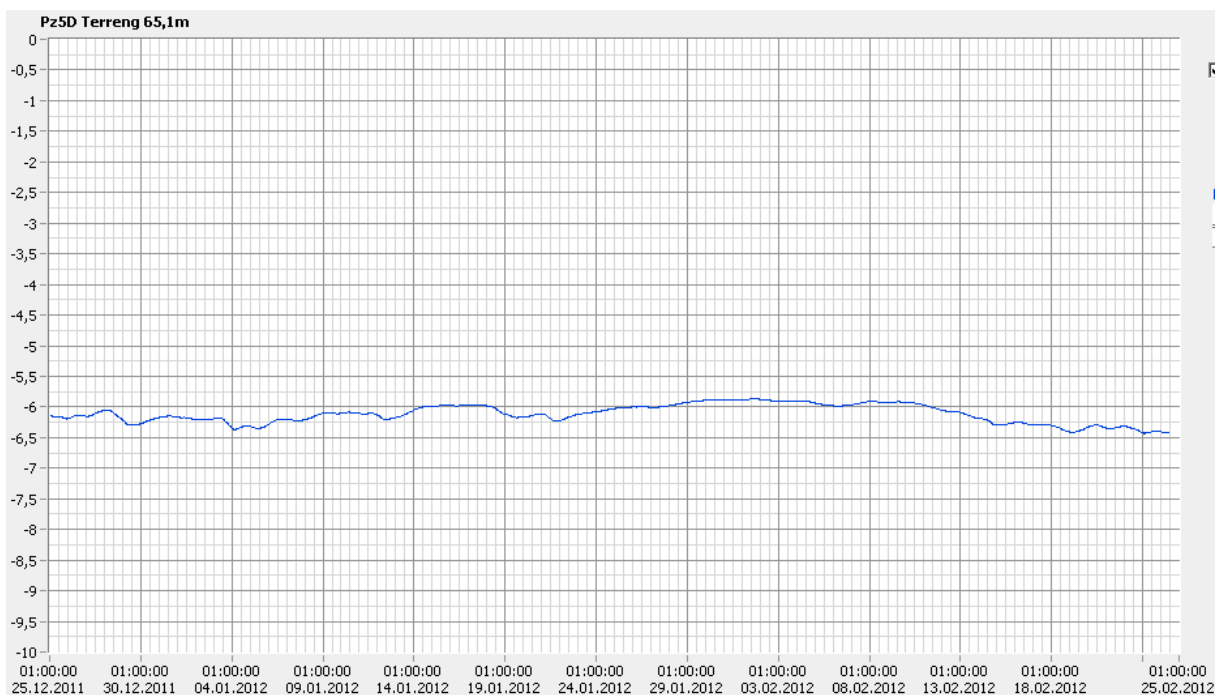


Figure 17. Piezometer measurements for Pz 5D. Measurements range from 25.12.11 – 25.02.12. (Jernbaneverket 2011)

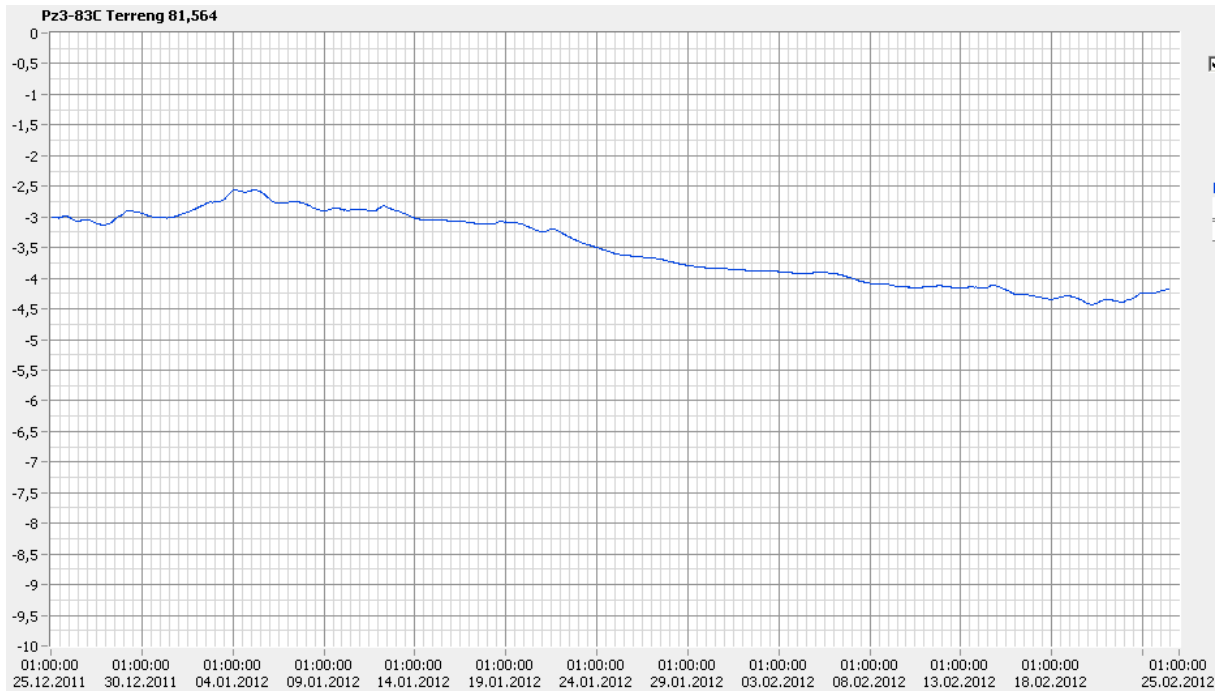


Figure 18. Piezometer measurements for Pz 3–83B. Measurements range from 25.12.11 – 25.02.12. (Jernbaneverket 2011)

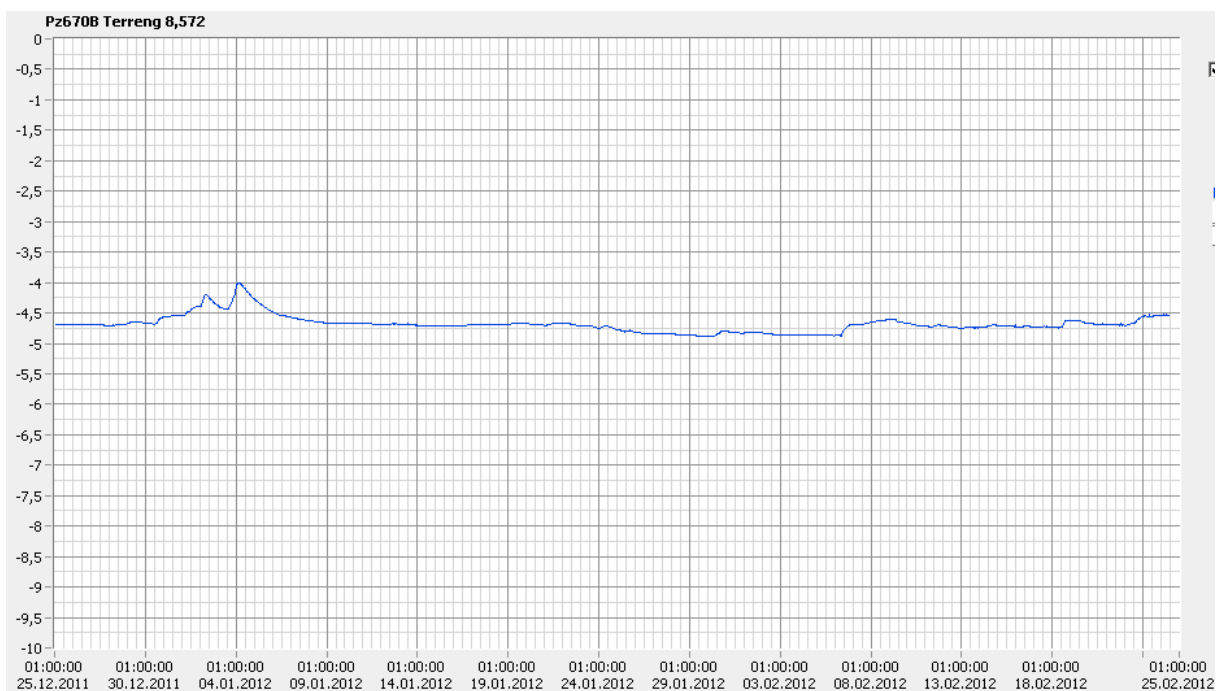


Figure 19. Piezometer measurements for Pz 670B. Measurements range from 25.12.11 – 25.02.12. (Jernbaneverket 2011)

When studying the different charts of the piezometers ranging over one year the values are more or the less the same after the whole year has passed. Naturally, due to physical and geological circumstances the groundwater level differs from hole to hole. Grain size, thickness and distribution of soil masses and joint frequency in the rock mass are some of the parameters that affect the groundwater fluctuation.

Locally, the most natural cause of groundwater fluctuation is precipitation, which can clearly be seen in the diagrams. Initially after the start of the measuring period the groundwater table increases in all diagrams, with different intensity. This is most likely related to the spring thaw where snow and ice melt and cause water to penetrate into the ground. Precipitation falling as rain will also increase the groundwater level.

Later on the year the groundwater table drops to reach a minimum in summer. This is because most of the meltwater from the spring thaw penetrates into the ground or evaporates. At the same time the precipitation is at its lowest, with mild and arid weather. Then, when approaching late summer and early autumn, the groundwater table starts moving up due to increase in the precipitation. This continues throughout the autumn then culminates in the late stages of October where the precipitation is at its peak. As time goes by and the temperature drops so does the groundwater table, reaching practically the same level at the start of the measuring period. This trend is particularly visible in Pz 3–83A.

Tiny, local peaks are found throughout the graphs in the different holes over the years. These are probably related to local geological differences. Pz 670A differs significantly more from the other holes in the one-year period but still follows the trend with the peaks and bottoms of the groundwater level. Pz 5C is the hole having most variations, however this is probably due to natural reasons and not the tunnel excavation works.

With respect to the last two months which are measurements from the very beginning of the tunnel excavation, the groundwater table remains at a rather constant level. This corresponds well to the one-year measurements which claim that the groundwater table is reaching the minimum level at this time of the year. In the chart for Pz 3–83B this is especially visible, but it is probably not a representative move for the trend of the lowering of the groundwater table. Most likely the sudden drop in the groundwater table is due to natural reasons.

In general it would seem based on these piezometer values, that the tunnelling has not affected the groundwater level in any particular way as the seasonal fluctuations remain. This is probably due to the natural draining effect discussed in chapter 4.2 and the grouting process discussed in chapter 8.

3 Geological and geotechnical conditions

3.1 Rock types

The most characteristic geological feature in the area is the long-stretched basalt wall which will house the station hall. It is from previous geological investigations known that the rock types in the area are mainly sandstone (also known as the Ringerike sandstone), the Asker group (sandstone, schist and conglomerate) and basaltic lava flows containing layers of red silt- and sandstone, tuff, agglomerate and lava conglomerate, the latter named the B1 group. The Holmestrand plateau is part of this basalt formation. The lava peaks in the B1 group may contain gas cavities and an abundance of joint sets.

There are also several dykes in the area that mainly consist of syenitic porphyry, diabase and rhomb porphyry. These follow the main joint directions in the area and have intruded in the various faults as well. The dykes are often more jointed than the country rock, housing calcite or clay zones which traverse parallel to the border of the intruded country rock. (Norconsult 2010)

For more information regarding the rock types and the geology in the area, see the project thesis referred to in the introduction.

3.2 Geological formations

Miscellaneous faults exist along the planned railway tunnel, varying in size. These faults compose most of the weakness zones of the tunnel. Some of these weakness zones are expected to contain clay. One of these, a zone with a width of 2 m intersects the Holmestrand road tunnel and is expected to intersect the southernmost entrance tunnel to the station hall. Figure 20 displays the location of the weakness zone.

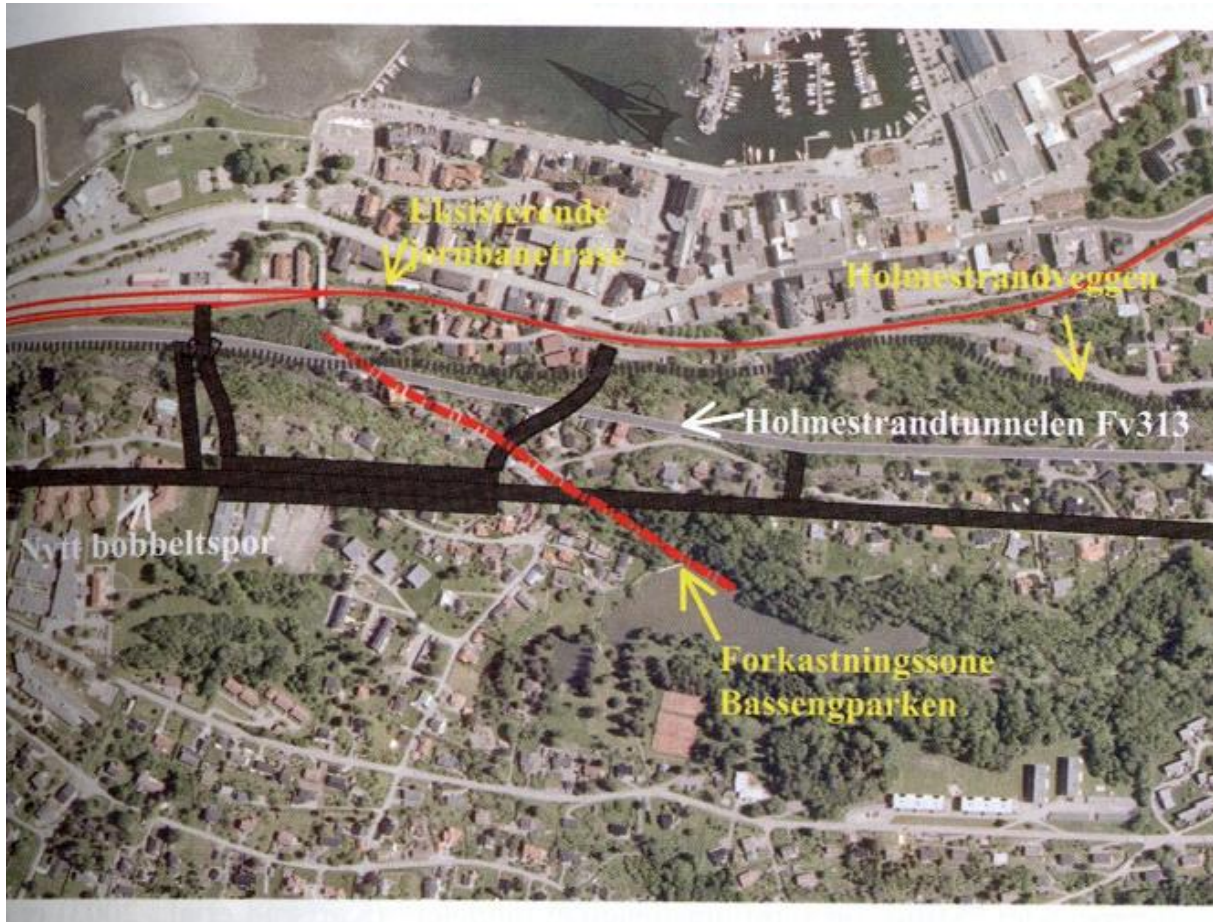


Figure 20. Overview at the major weakness zone (red solid line) intersecting the entrance tunnel to the station hall and the Holmestrand road tunnel. (Langford, Kveldsvik et al. 2011)

3.3 Soil masses

Most of the soil masses on top of the Holmestrand plateau are found below the marine border, hence the soil masses are composed of marine clay. The thickness of the soil masses varies and at certain areas naked basalt outcrops are found. A distinct feature of the Holmestrand plateau is parallel rock outcrops with interlaying valleys of soil masses. (Norconsult 2010)

In general, the greatest source of worrying comes from the risk of subsidence of houses related to pore pressure reduction in the soil masses when lowering the groundwater table.

4 Hydrogeological conditions

4.1 General introduction

Most rock types in Norway have a low permeability and thus only are able to transport water by secondary structures like channels and joints being formed during metamorphosis. The Norwegian rock mass is a joint aquifer and the significance of pore rooms can be neglected. Geological differences and history determine the joint systems.

Water in the rock mass mostly follows the lava peaks and the joint zones in the B1 formation as well as faults in the area. This means that the groundwater table may have an irregular flowing pattern, and there might be more than one groundwater table. Previous core drillings close to the Ringerike railway suggest that the lava flows are sealed at a depth of more than 100 m. Fault zones may still contribute to inflows. Local dykes in the rock mass may also increase the inflow activity. (Løset 2006)

The most important and significant source of groundwater in the Holmestrand area is precipitation. Parts of the groundwater is being evaporated, this is due to several reasons such as topography, temperature, air moisture, wind and wind direction, etc. The annual evaporation in the Oslo graben is around 50%. (Multiconsult 2008) Also, permeability in the soils and in the rock mass, magazine properties of the soils and the rock mass and other water sources like creeks, rivers, lakes are parameters which determine the groundwater behaviour. (Hognestad, Fagermo et al. 2010) There have been completed precipitation measurements at the closest meteorological stations to Holmestrand. In figure 21, these are presented.

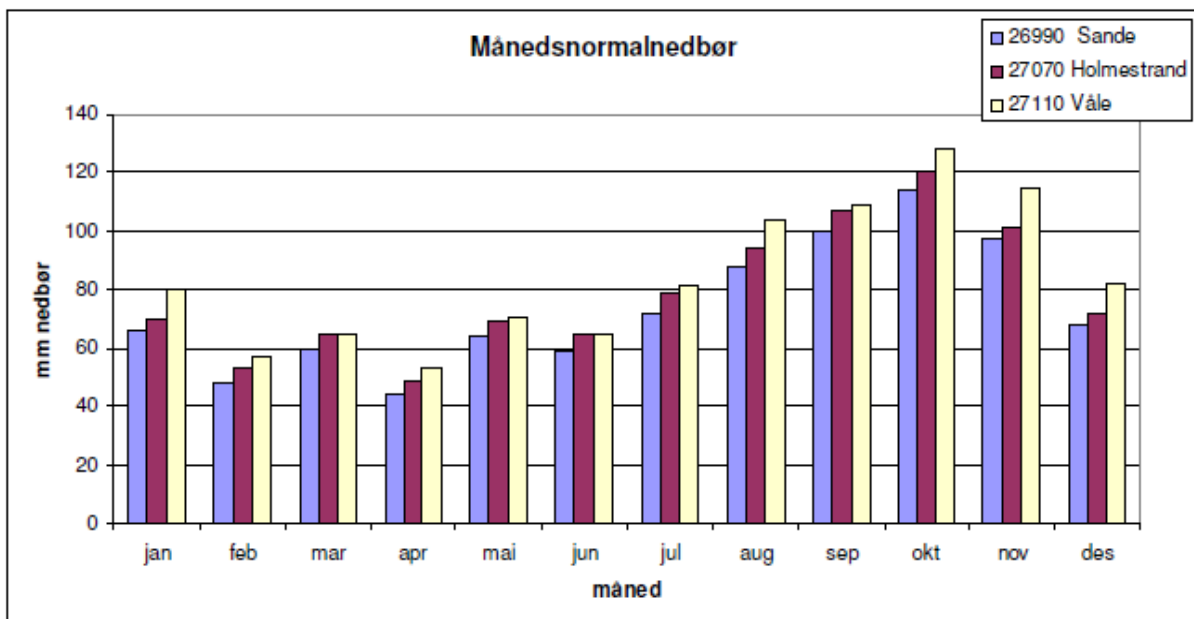


Figure 21. Diagram showing the annual precipitation in the Holmestrand area (red columns). (Multiconsult 2008)

Temperature measurements have been performed as well, these are shown in figure 22.

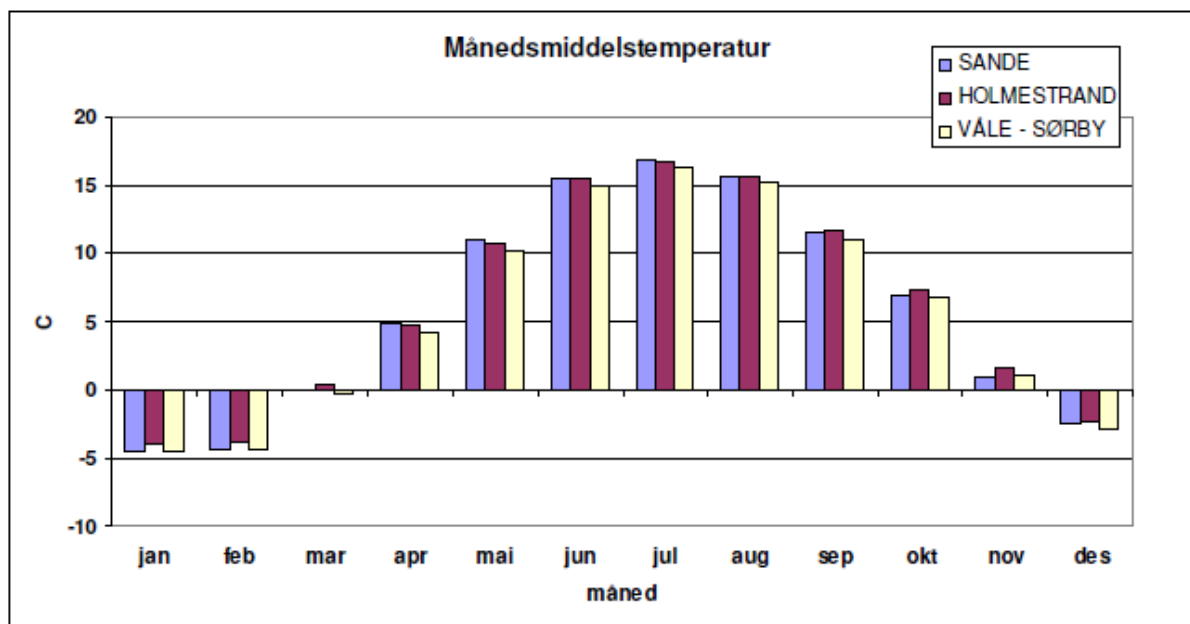


Figure 22. The annual temperature trend in the Holmestrand area (red columns). (Multiconsult 2008)

Studying the two figures the development is as expected in a common Norwegian climate; the greatest precipitation takes place in autumn, from September to November, with the temperature reaching a peak in summer; from June to August. With July having the temperature peak and intermediate precipitation, the amount of groundwater accumulation would be expected to be fairly low at this time of the year. Reversely, the month of October with a low average temperature, but still well over the freezing point and a high precipitation rate would lead to a groundwater accumulation maximum.

There are no great or significant rivers or streams receiving the precipitation at the top of the Holmestrand plateau. Hence, the groundwater flows via tiny, local creeks into the Holmestrand fjord. Occasionally several local ponds are located, including the Bassengparken area in the town centre of Holmestrand. (Multiconsult 2008)

4.2 Drainage situation

It is assumed that the Holmestrand plateau is naturally drained and that the groundwater table has been leveled down to the tunnel floor of the Holmestrand tunnel. This is because the tunnel has not been grouted. This assumption has also been confirmed by several measurements taken at the plateau, west of the town centre of Holmestrand. (Langford, Kveldsvik et al. 2011) A sketch of how the drainage functions interact can be seen on figure 23.

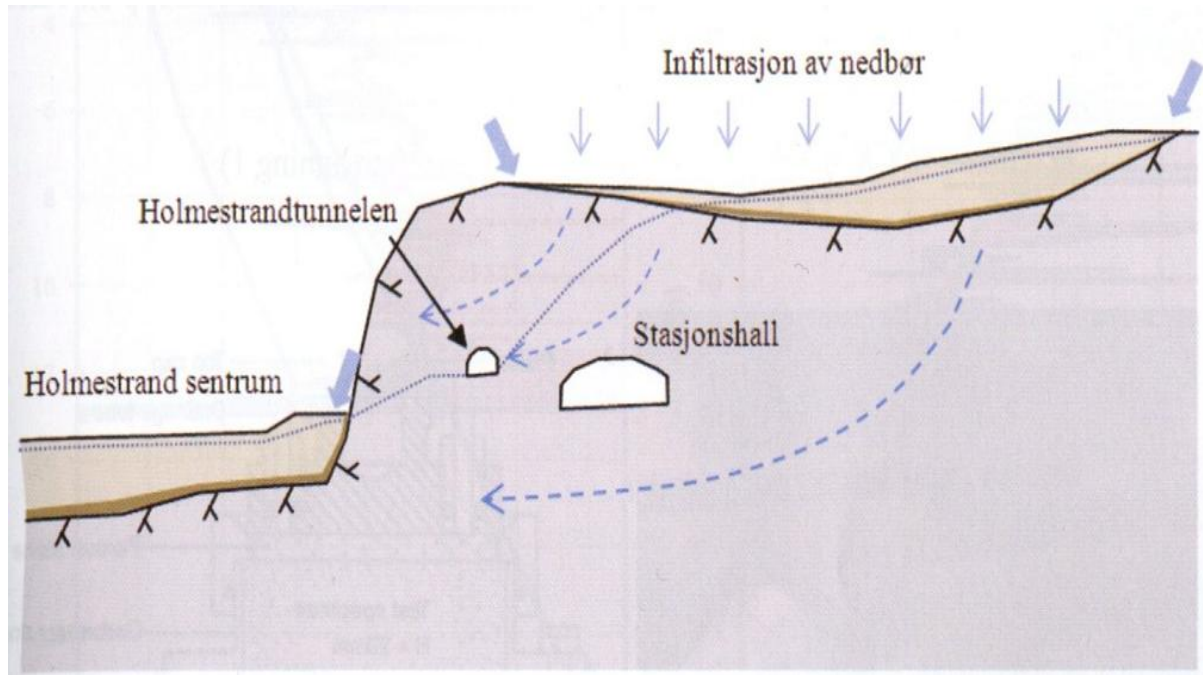


Figure 23. The drainage situation at the Holmestrand plateau. (Langford, Kveldsvik et al. 2011)

From the figure above it seems as if the draining of the groundwater poses no convincing threat to the tunnel project, since the Holmestrand tunnel already takes care of the draining. It is also known that the tunnels on the E18 expressway at the other side of the Holmestrand plateau are excavated without being grouted and that there have been no significant problems with the groundwater situation.

4.3 Impact on soil and groundwater

It is generally known that inflows occurring when tunneling through a rock mass with an overlaying soil mass will cause pore pressure reduction. Depending on the nature and quality of the soils, the scope and significance of the pore pressure reduction will vary. Should there be any soft clay composing the soil masses the risk of damages on buildings and other installations along the tunnel axis will increase. This point is especially valid when soil masses are marine clay which is fairly abundant throughout most of Eastern Norway. (Karlsud, Erikstad et al. 2003)

Figure 24 shows the effect of tunneling below soil masses.

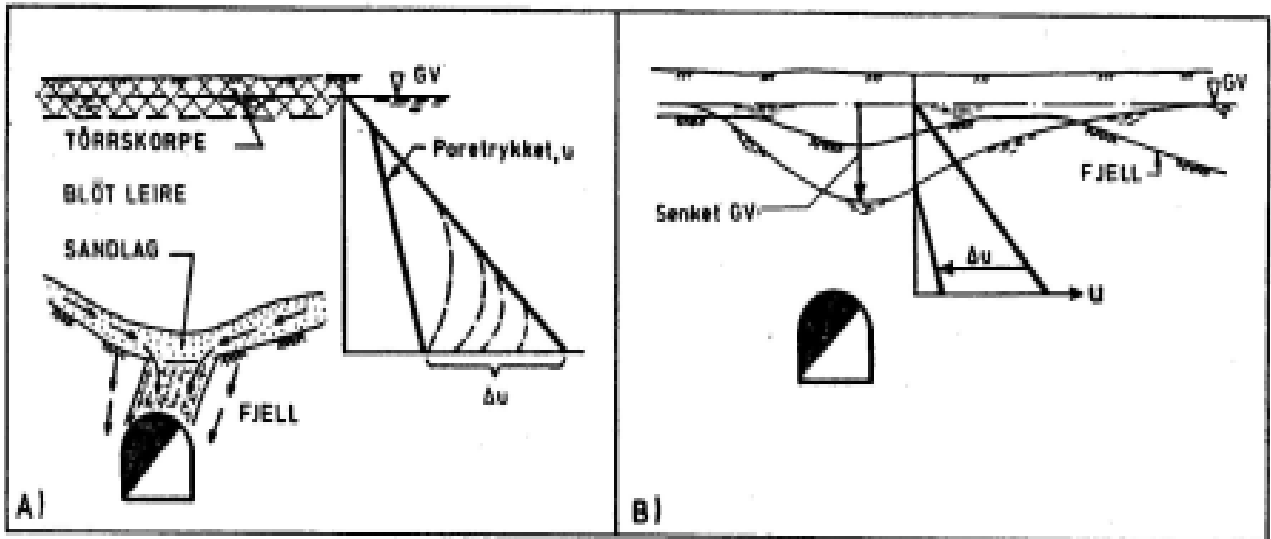


Figure 24. The effect of tunnelling below soil masses and the effect on the groundwater level. (Karlsud, Erikstad et al. 2003)

Studying figure 24, the consequence becomes clear. Lowering the groundwater table leads to greater effective stresses in the soil, increasing the risks of subsidence. Hence the reason to carefully monitor the groundwater by the aid of wells.

Usually, the pore pressure in cases like these will vary linearly 2 – 4 m under the groundwater level or 2 – 4 m below the lower part of the dry crust and down to the rock mass. The reason for this is that the dry crust or the upper weathering zone has a significantly higher permeability than the non-weathered underlying clay zone. If there are clay layers with varying permeabilities, the stationary distribution will deviate from the linear. (Hognestad, Fagermo et al. 2010) This can also partially be seen on the sinistral part of figure 11.

5 Inflow criteria related to grouting

5.1 Inflow requirements

Inflow criteria for a tunnel including the station hall are based on several factors, depending whether they are for the excavation works or the installation. The most relevant factors are aggregated in table 2.

Table 2. Various factors for assessing the inflow criteria related to surroundings and the excavation works for the station hall. (Hognestad, Fagermo et al. 2010)

Reasons for determining inflow criteria related to the excavation works	Reasons for determining inflow criteria related to the station hall purpose	Reasons for determining inflow criteria related to the surroundings
Prevent greater water leakage	Avoid ice formation	Possibilities of draining and subsidence of adjacent soil masses
Problems and costs related to pumping out the water	Corrosion and wear of technical installations	Damages to agriculture when lowering the groundwater level
Stability of the rock room/cavern/tunnel	Sever high conduction in electrical installations	Environmental affection of the surface and the biological diversity
Working environment	Moisture affections on quality, lifespan and maintenance costs	Reduction of groundwater magazines in wells
Reduced quality of lining works	Seepage from the tunnel	Possibilities of pollution from escaping fluids
Determine the maximum inflow with respect to execution of support like bolting, shotcrete, lining, etc	Traffic and safety aspects	
Determine the maximum inflow with respect to drilling blast holes		

Naturally, these parameters are more relevant and important when tunneling through a populated area than at a remote location. The Holmestrand plateau contains the major part of the inhabitants in Holmestrand and may be considered as a sub-urban and moderately populated area. Because of the risk of damages to houses should subsidence occur along with lowering of the groundwater table and loss of magazines in wells, there can be virtually no visible leaks at the tunnel face. Also, depending on the inclination of the tunnel floor the water pump rate will have to be adjusted and the costs may vary.

In general when determining inflow requirements for any tunnel such as the station hall several factors and aspects are necessary to review. Examples are consequences of leakages related to surroundings, the excavation works and the tunnel itself. To survey these parameters, several investigational methods exist, such as water loss tests prior to and after the grouting, leakage measurements in the rock mass, pore pressure reduction and subsidence measurements. (Hognestad, Fagermo et al. 2010)

5.2 Inflow assessment

For a tunnel finding itself in a homogenous, isotropic rock mass with a constant permeability value at a given depth the inflow into the tunnel can be calculated using the following equation:

$$(IV) \quad Q = \frac{2 \cdot h \cdot l \cdot \pi \cdot k_i}{\ln \left(\frac{r_e + t}{r_e} \right)}$$

where

h = depth below the groundwater level (m)

l = tunnel length (m)

k_i = hydraulic conductivity in the grouted zone ($\frac{m}{s}$)

r_e = equivalent radius of tunnel (m)

t = thickness of the grouted zone (m)

Input parameters to be used in (IV) and the calculated result for both the station hall and an ordinary double railway tunnel can be found in table 3.

Table 3. Calculation and comparison of the hydraulic permeability for both the station hall and a regular double railway tunnel. (Langford, Kveldsvik et al. 2011)

Holmestrand station hall		Regular double railway tunnel	
h (m)	60	h (m)	60
l (m)	100	l (m)	100
Q ($\frac{L}{\frac{min}{100 m}}$)	10	Q ($\frac{L}{\frac{min}{100 m}}$)	10
r_e (m)	14	r_e (m)	7
t (m)	10	t (m)	10
k_i ($\frac{m}{s}$)	$2,4 \cdot 10^{-9}$	k_i ($\frac{m}{s}$)	$3,6 \cdot 10^{-9}$

Studying table 3 it becomes evident that the demand for hydraulic permeability is approximately 40% lower for the station hall than for a regular railway tunnel with the same inflow demands. This will likely lead to several more grouting holes and greater amounts spent of grouting mass. (Langford, Kveldsvik et al. 2011)

To compare this need for hydraulic conductivity, a graphic presentation of several lately excavated and constructed Norwegian tunnels has been visualised in figure 25. When studying the figure, it can clearly be seen that the hydraulic conductivity assessed for the station hall in (IV) falls below the average and joins the level of the stricter and more difficult achievements reached.

The tunnel crosses a major weakness zone as displayed in figure 20 between km 85,4 and 85,5 which constitutes a distance of approximately 80 m. This is at the time of writing regarded as the most difficult part of the tunnel when it comes to grouting. In the vicinity the Bassengparken area is found containing a groundwater table 4 – 5 m higher than the rest of the rock mass. This is about 45 m above the tunnel floor. The groundwater table in this part of the station hall is assumed to find itself close to the terrain surface. The weakness zone continues towards the south beneath the pond in the Bassengparken area. Geological and excavating conditions are thought to pose major difficulties at this point, especially achieving an inflow criterion at $Q = 10 \frac{L}{\frac{min}{100 m}}$. This means that full concrete lining might be necessary to fulfill the requirements. (Langford, Kveldsvik et al. 2011)

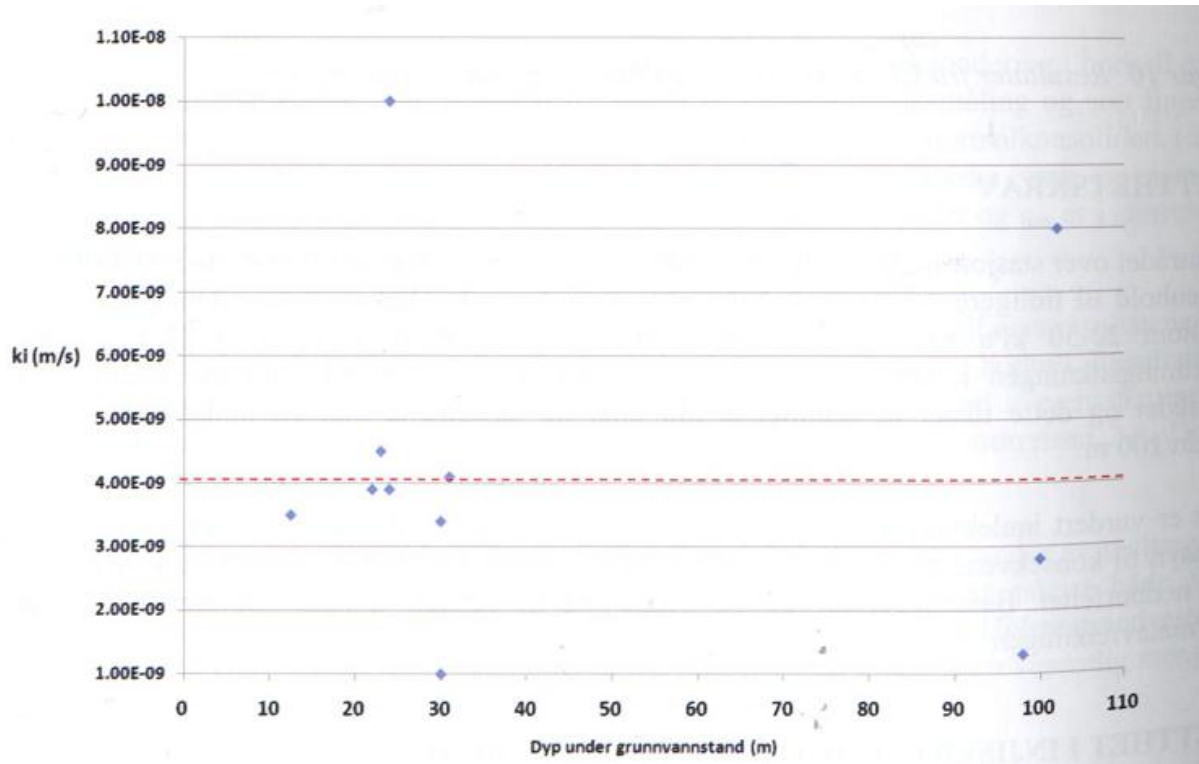


Figure 25. Hydraulic conductivity values in the grouted zone for various newly constructed tunnels. The red dotted line indicates the average. (Langford, Kveldsvik et al. 2011)

6 Grouting

6.1 General introduction

To reduce the inflow into the tunnel to an absolute minimum, grouting procedures are often necessary. By the aid of several thick fluids filling cracks and joint systems which serve as water flowing channels, the aim is to keep the tunnel free of water.

Physically, the grouting procedure consists of a drilling machine drilling a determined number of holes with a certain angle relative to the tunnel face; the holes are then being filled by injecting one or several types of fluid. Hole lengths usually are within the range of 18 – 24 m. (Hognestad, Fagermo et al. 2010) The pressure at which the fluid is injected has to be determined from various factors such as the joint sets in the rock mass and the stress situation. Typically, the pressure must be at least 15 bar. (Hognestad, Fagermo et al. 2010) When using cement or micro cement as grouting agents the relationship between water and cement, in this text referred to as the w/c-ratio is another matter that needs to be considered. The finalising pressure and methods for stoppage must be cleared before starting the grouting round.

Figure 26 shows the basic principles of how a grouting fan is drilled.

Other fluids suitable for grouting are chemical compounds. While it can be argued that chemicals constitute a hazard to both the environment and the health of tunnel workers as well as being outright expensive compared to cement based grouting agents, the correct usage of chemical compounds will prevent water inflows. The latter point applies especially to tunnels with strict inflow regulations. Also, when experiencing severe leaks and immediate sealing is necessary, chemical grouting compounds compose one of the better options. Post-grouting fans are very often based on chemical grouting agents. In most cases though, the chemical compounds will be used in combination with cement based agents and constitute a lesser volumetric part. In the later years, micro cement has taken over parts of the chemical compound usage. (Davik, Kveen et al. 2002)

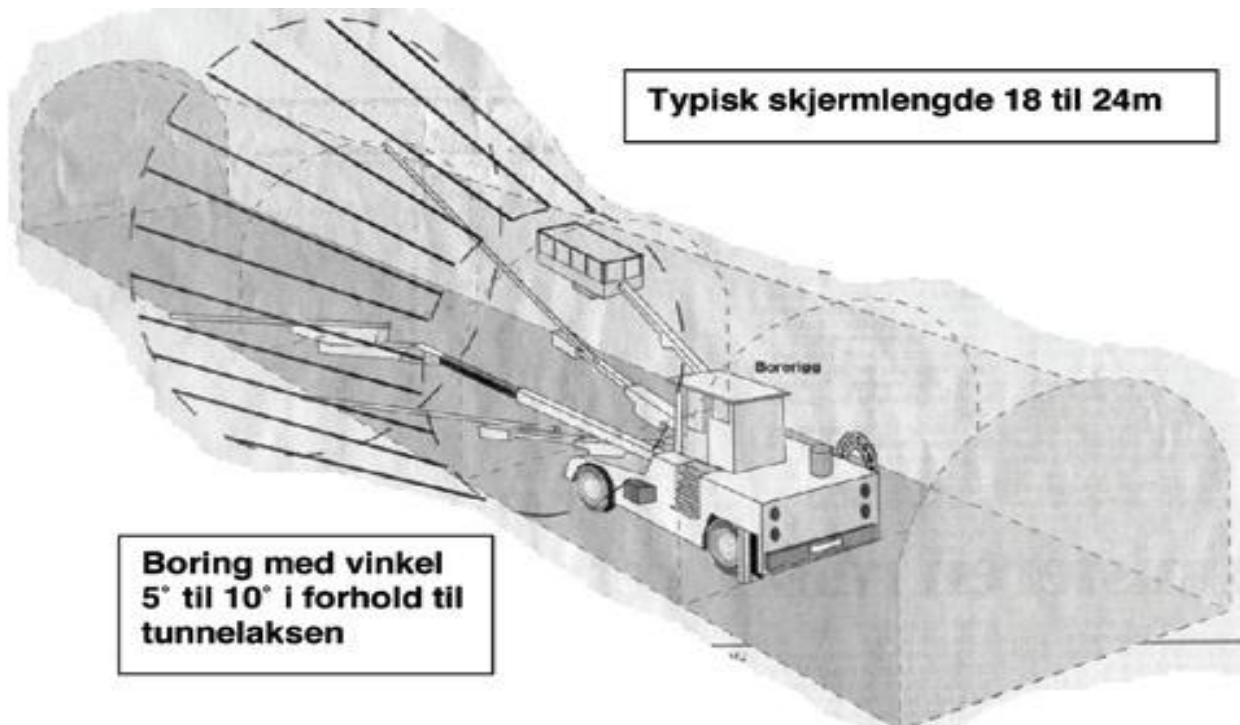


Figure 26. A sketch of the drilling of a grouting fan in a tunnel. (Hognestad, Fagermo et al. 2010)

A basic pre-grouting fan carried out in the R7 tunnel can be seen on figure 27.



Figure 27. Grouting rods being inserted at the tunnel face in the R7 entrance tunnel.

6.2 Inflow limits for the station hall

Assessing the precise number of requirements for grouting inside a tunnel is difficult, next to impossible. However, it is possible to determine a representative number which serves as an inflow limit. Based on several projects in the past, an inflow limit can be determined for most of the tunnels today. The Holmestrand tunnel is being excavated in a sub-urban environment, meaning that high-class inflow requirements will be necessary. It is already determined that class A will be utilised in the entire station hall. This makes the inflow limit being $5 \frac{L}{\frac{min}{100 m}}$.

Based on several investigations and surveys of different tunnels and underground installations in Eastern Norway a model for predicting the risk and scope of pore pressure reductions and risk of subsidence has been assessed. This model is presented in figure 28.

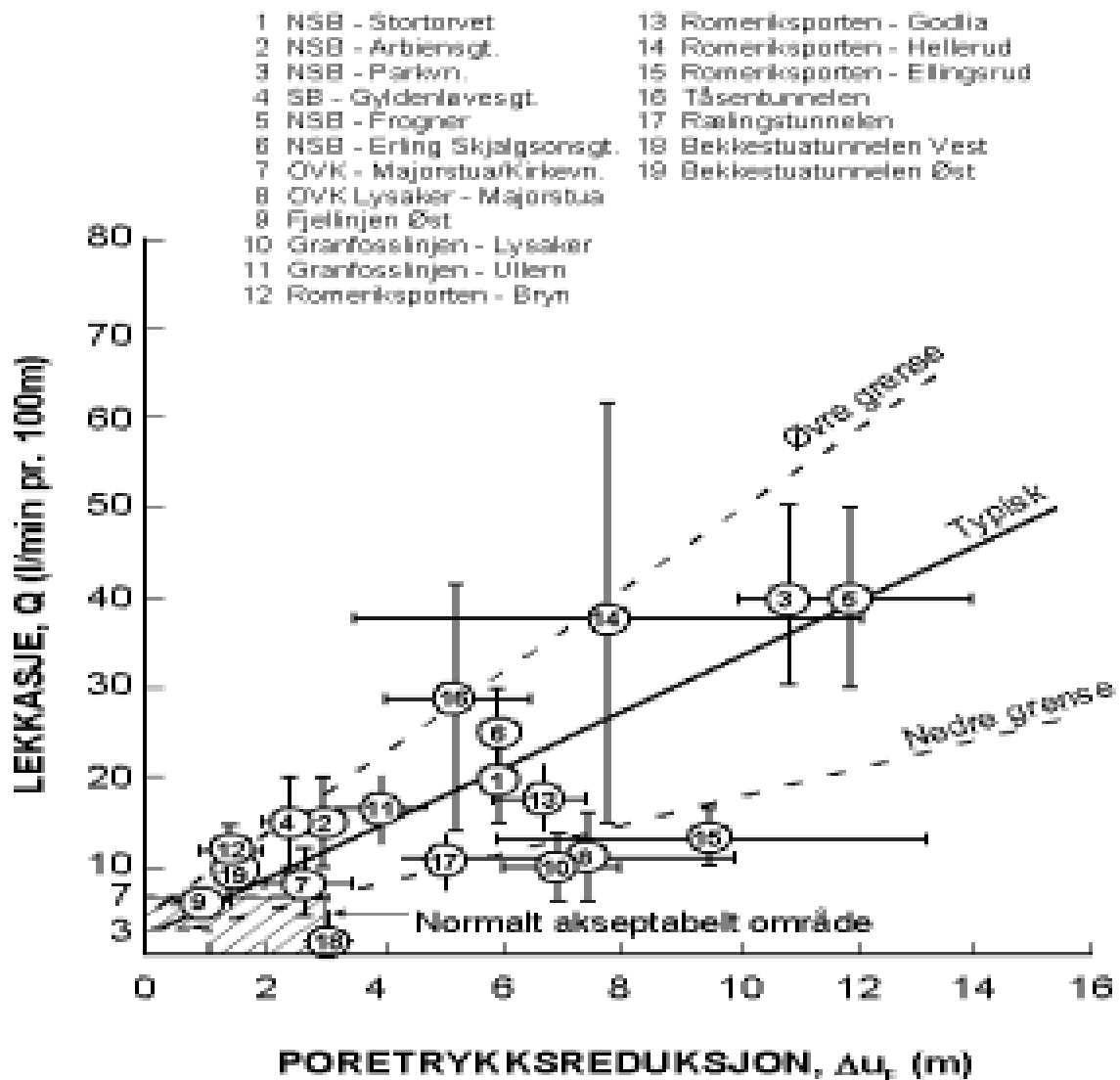


Figure 28. A statistic model showing the relation between inflow and pore pressure reduction. (Karlsud, Erikstad et al. 2003)

When studying figure 28, the "normal acceptable area" is within the range of $3 - 7 \frac{L}{\frac{min}{100 m}}$. This means pore pressure reductions of 1 – 3 m which further lead to subsidence at several cm. This is usually acceptable at most cases and since there have been revealed no great risks of harming any installations at the surface of the Holmestrand plateau, the inflow criterion is at times when geological conditions are satisfactory being allowed to fluctuate between 5 – 10 $\frac{L}{\frac{min}{100 m}}$. As seen, such is the case at the Sjøskogen tunnels and partially the R13 entrance tunnel at Snekkestad.

Considering the situation of the soil masses, the pore pressure reduction referred to in figure 28 will lead to reductions of 20 – 50 KPa when operating with the current inflow criterion. The stress increase in the clay when reducing the pore pressure is within the over consolidated range (see figure 0011), causing minimal and harmless subsidences. (Langford, Kveldevisvik et al. 2011)

6.3 Types of grout used in the station hall

There are mainly two grouting blends which are to be used for grouting in the station hall. The contractor has the right to determine the composition and the types of grouting agents used for every grouting round in the tunnel. Additional grout compounds might be used upon approval of the contractor. Under no circumstances may Bentonite be used as an additive. The grouting blends can be seen in table 4.

Table 4. The two different blends used for grouting in the station hall. (Jernbaneverket 2011)

Blend 1		Blend 2	
A. Cement type	Standard injection cement. Grain size D95 ca. 35 µm.	A. Cement type	Micro cement (Portland). Grain size D95, ca. 12 µm.
B. Super plasticiser	Must be adapted to the cement type used, and other blends. Must keep its dispersing property for at least 30 minutes.	B. Super plasticiser	Must be adapted to the cement type used, and other blends. Must keep its dispersing property for at least 30 minutes.
C. Micro silica	Micro silica and similar additives must have a grain size lesser than the micro cement and have better penetration ability in joins and sand columns. The micro silica must compose at least 5 – 20% of the entire cement weight.	C. Micro silica	Micro silica and similar additives must have a grain size lesser than the micro cement and have better penetration ability in joins and sand columns. The micro silica must compose at least 5 – 20% of the entire cement weight.
D. w/c ratio	0,5 – 1,0	D. w/c ratio	Maximum 2,0
E. Free water	Free water must not exceed 2% in the mix in column samples.	E. Free water	Free water must not exceed 2% in the mix in column samples.
F. Viscosity	Maximum flowing time in marsh-funnel: 40 – 45 s.	F. Viscosity	Maximum flowing time in marsh-funnel: 40s.
G. Accelerator	Normally used in the last blend.	G. Accelerator	Normally used in the last blend.

Regarding water-reactive polyurethane, only a polyurethane system with 4,4 – diphenylmethane di-isocyanate may be used. (Jernbaneverket 2011)

All cement types and grouting blends will have to undergo approval from the contractor before being licensed to usage. The control is accomplished through investigating the grain size of the cement in dry state and in the blend. The density of all blends must also be documented before usage.

Before every grouting round the entrepreneur is obliged to measure the density of the suspension blend using a Barold mudweight. The “bind-off time” must also be measured before grouting. Occasionally, the contractor will perform test to check if the w/c-ratio and the density of the grouting suspensions are within the current regulations. (Jernbaneverket 2011)

6.4 Pre-grouting fans

6.4.1 General description

The basic idea behind pre-grouting fans is to reduce all ways of inflow into the tunnel, clogging all water flowing channels in the rock mass, reducing most changes in the groundwater level. Should some of the boreholes not be satisfactory filled, post-injection might occur. This applies also for bolt holes that experience water leakage.

Pre-grouting fans are to be utilised throughout the entire station hall, including the entrance tunnels. It is planned to execute the grouting by means of a primary fan and several control fans, based on how the primary fan clogs the rock mass channels. Pre-grouting fans must imbricate each other, meaning that every new pre-grouting fan will have to be drilled within a previously grouted area of the tunnel. (Multiconsult 2008) The imbrication distance for the pre-grouting fans is usually at around 9 m. (Jernbaneverket 2011) Depending on the quality of the rock mass and the state of the tunnel the frequency of which the control fan shall be used may vary after negotiations between the contractor and the entrepreneur have taken place. Normally, the control grouting fan will be taking place after the first blast after the pre-grouting fan. (Jernbaneverket 2011) A sketch of a planned typical pre-grouting fan for the station hall can be seen on figure 29.

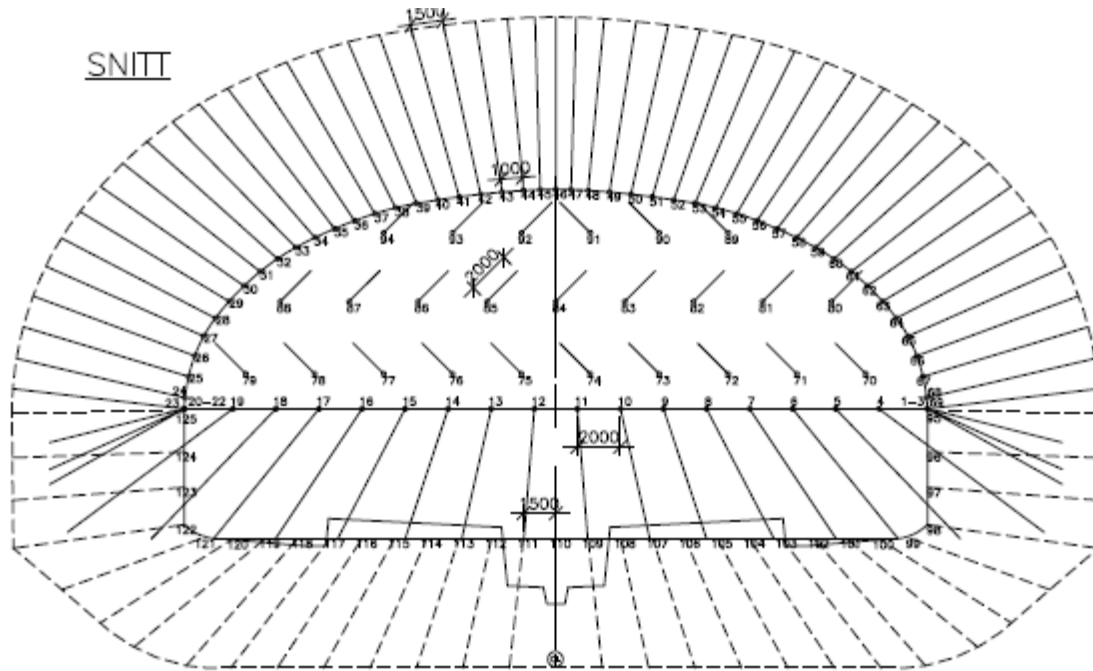


Figure 29. The basic hole distributions for pre-grouting fans in the station hall. Holes are drilled from the top of the cross-section and then around the entire tunnel profile. Prior to blasting out the lower part, holes are drilled around this as well. (Jernbaneverket 2011)

At weakness zones, the grouting fan will be partially altered. The different situation can be seen on figure 30.

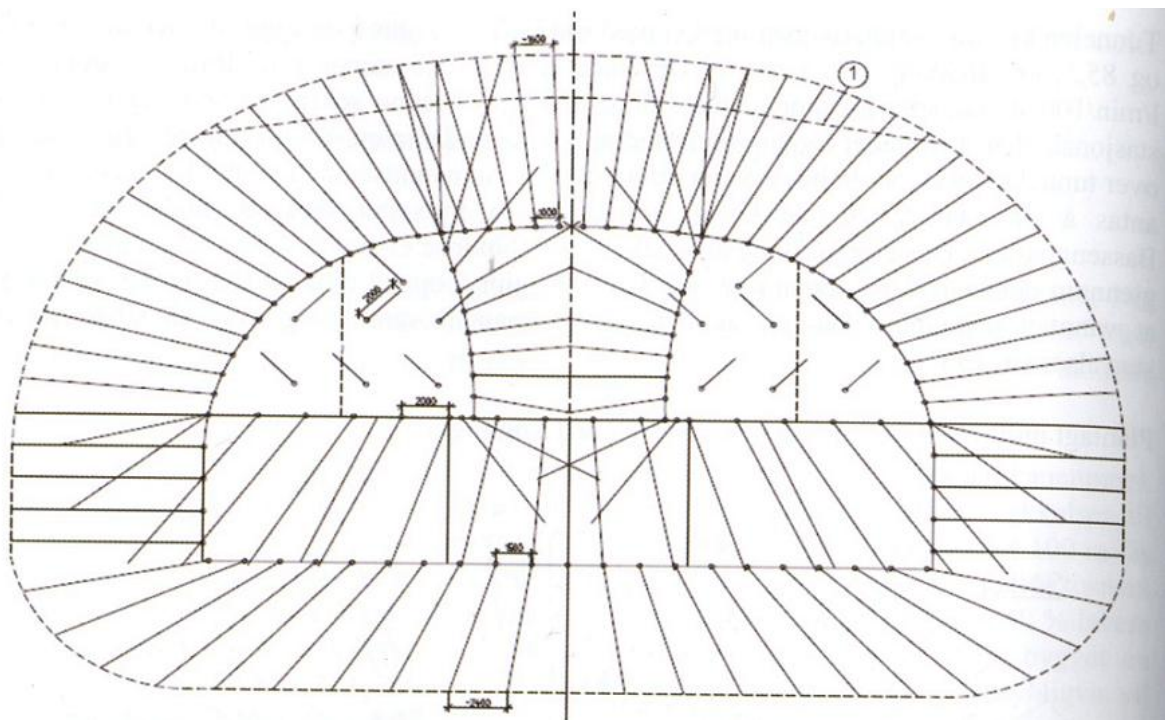


Figure 30. Sketch of the grouting scenario at a weakness zone. Holes are drilled in the tunnel and partially in the rock mass which later is to taken out before each of the partial cross-sections are excavated. (Langford, Kveldsvik et al. 2011)

6.4.2 Accomplishment and stoppage criteria

Generally, as seen on figures 29 and 30, the pre-grouting fans are created by an abundant number of boreholes. Should there be demands or necessities that require additional holes, the contractor will have to decide on this matter, after thoroughly discussions with the entrepreneur. The distance between the boreholes and the tunnel face is supposed to be 5 m in the tunnel ceiling and the tunnel walls and 5 – 6 m in the tunnel floor. The distance between the pre-grouting fans is supposed to be 15 m. Boreholes at the tunnel floor are injected firstly, then the grouting is being performed upwards.

Prior to grouting inflows are to be registered. Should inflows occur, a water loss measurement must be carried out. After the registration of the inflow is complete, packers must be inserted into the boreholes once they are cleaned. As soon as possible after insertion of the packers and drilling of the grouting holes, the injection procedure has to start. All injection holes must have a starting blend with high penetration rate. Changes in the grouting blend (such as viscosity, etc.) and stoppage procedures of the borehole must be assessed by individual consideration of the blend in every hole.

Should there be any greater cavities in the boreholes, sand with a grain size of 0 – 2 mm may be added to blend 2. Holes are to be finalised with an accelerator. The stoppage pressure must not be greater than the rock mass stress otherwise deformations may take place. The pressure must also be controlled and determined in coordination with the nature of the rock mass. All holes are to be filled, even though in a dry state. Maximum stoppage pressures are found in table 5. The grouting pressure has to be reduced should there be outside seepages, at cave-ins, etc. (Jernbaneverket 2011)

Table 5. The maximum stoppage pressure in boreholes at different locations in the tunnel with different overburdens. (Jernbaneverket 2011)

Overburden	Maximum pressure	
	Boreholes in tunnel ceiling and at tunnel walls	Boreholes on the tunnel floor and at the tunnel face
0 – 5 m	20 bar	30 bar
5 – 10 m	40 bar	60 bar
>15 m	80 bar	80 bar

The values in table 5 are only theoretical, and the stoppage pressure is subject to change based on the water pressure in holes, joint frequency, etc. (Jernbaneverket 2011)

Mechanical single-purpose packers are to be utilised and inserted approximately 2,5 m inside the grouting hole. The position of the packer is subject to change based on the geology, etc. Should there be necessities to partially grout the borehole, the packer might be placed 15 m into the borehole. This may happen if there are significant inflows or leakages in a borehole. To this purpose, hydraulic multi-purpose packers must be used.

Grouting must take place at a fairly low flow, such as $20 - 25 \frac{L}{min}$, and end with a lower flow value than what was initially used. The w/c-ratio, pressure, additives, etc. are subject to change based on the conditions of the hole. Filling of the hole is complete when the determined pressure is reached and has a constant value at a time span at least more than two minutes at a low flow, value reaching zero.

When there has been injected more than 250 L grout mass at the lowest possible w/c-ratio in a hole and the stoppage pressure has not been reached, a break shall take place in the grouting works for the actual hole and the hole is left in its current state. If a total amount of 1200 kg cement is injected to a hole in the tunnel ceiling and 1600 kg on the tunnel walls and the tunnel floor and the stoppage pressure still has not been reached, the hole shall be finalised by means of an accelerator. This does not apply for the neighbouring hole as long as the hole has not been filled with the maximum amount of cement. Holes undergoing injection works are not to be finalised before the stoppage criterion has been reached.

Should the overburden be low, the amount of grout inside the holes has to be agreed upon between the contractor and the entrepreneur.

Hardening time after grouting works are complete depends on the injection course and it might be necessary to delay the following works in the tunnel such as probe drilling or blast hole drilling. Over time, as more knowledge and experience is gathered of the rock mass, criteria for stoppage pressures, flow, blending properties are subject to change.

(Jernbaneverket 2011)

6.5 Control fans and extra fans

The control fan length is supposed to be adapted to the length of the pre-grouting fan and the direction is determined by the contractor. If desired, control holes might be positioned elsewhere than on the regular drawings. Standard injection cement, like industrial cement is usually to be used for a control fan. The stoppage criteria are the same as for the pre-grouting fan.

Extra fans may be suitable for usage where there is a fairly low overburden (less than 12 m). At such locations, an extra fan may be injected in the tunnel ceiling serving as a means of aid for the common pre-grouting fan. Depending on the cross-section of the tunnel, 5 – 8 holes are drilled and grouted prior to placing the pre-grouting fan. Holes are drilled with such an angle that they constitute an extra fan. Also, extra fans may be utilised at spots containing

special geological properties such as weakness zones, or where there is significant inflow in the boreholes in the pre-grouting fan. Normally, standard injection cement (industrial cement) is to be used with stoppage criteria being the same as for the pre-grouting fans. (Jernbaneverket 2011)

6.6 Post-grouting

Generally, post-grouting procedures are difficult to perform, and time should be allowed for the injection works. Experiences have shown that in tunnels with strict inflow requirements post-injection frequently has a noticeable effect. In a post-grouting procedure, holes are drilled at the position of the leakages, and are injected with grout mass to stop the inflow.

Apart from observing and noting the seepage points, the orientation of the water-conducting joint systems should be surveyed. The boreholes should be oriented so that they reach the water-conducting joints 5 – 10 m outside the tunnel profile. The first boreholes to reach the leak should be used as holes for lowering the pressure. The boreholes being at the greatest distance away from the leakage should be the ones which are firstly grouted based on the principle that leakages move. The latter point is also the reason why there should be a significant distance on either side of the leakage to the post-grouting holes. This distance depends on the overburden and the joint frequency of the rock mass. (Hognestad, Fagermo et al. 2010) At times, bolts which have punctuated the pre-grouting fan, a Thorbolt adapter should be used. Usage of water reactive polyurethane should be limited only to bolts experiencing water leakages. (Jernbaneverket 2011) Figure 0031 shows the basic principle when post-grouting over great surface extents.

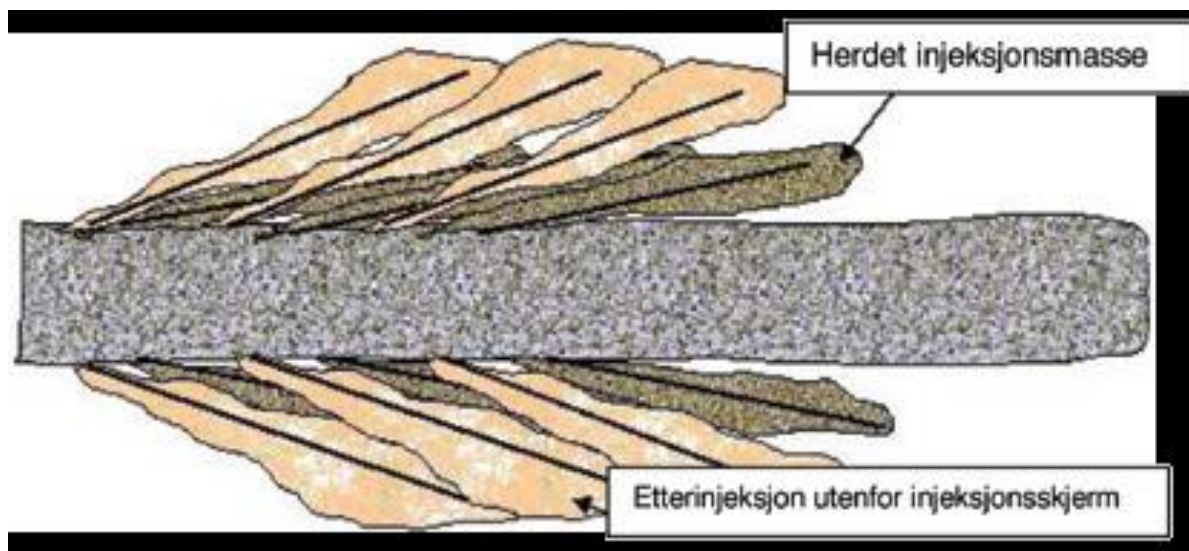


Figure 31. During the performance of post-grouting over a vast surface, holes are drilled in the opposite direction of the holes of the pre-grouting fan. Dark slabs show the hardened pre-grouting fan while the beige slabs indicate the post-grouting. (Hognestad, Fagermo et al. 2010)

6.7 Probe drillings

Probe drillings may take place to determine the quality of the rock mass finding itself in front of the tunnel face. Locations which contain a weakness zone are always to be probe drilled at the tunnel face and the probe drilling results are to be further assessed. At the same locations probe drillings, including core drillings may also be necessary. The latter is only to be performed by request from the contractor.

Typically a probe drilling fan consists of five holes; one in the middle of the tunnel ceiling, one in the abutments between the tunnel wall and the tunnel ceiling on either side and one at the border between the tunnel wall and the tunnel floor on either side. The probe drilling hole length must be 24 m. The probe drilling length is subject to change by request from the contractor and can be expanded up to 30 m. (Jernbaneverket 2011) It is also the contractor who determines the hole positions, their angle, etc. All probe drilling holes are to be grouted and filled once having served their purpose. (Jernbaneverket 2011)

6.8 Measures of inflow and water loss

Should an inflow or seepage in a borehole occur it is necessary to determine the flow rate of the leakage. This will mainly be performed by using a container such as a plastic bucket or a plastic bag and measure the time it takes to fill the container. At a flow rate $\leq 1 \frac{L}{min}$ the measure must last for 5 min. At flow rates not suitable for measuring, an assessment must take place. Water loss measures in the form of Lugeon tests will be necessary to execute using single purpose packers. The pressure used in the hole must be determined prior to performing the test, usually there are two rounds taking place. One round at 10 bar and the other round at approximately 60% of the maximum allowed pressure in the rock mass. Both test pressures are subject to change based on the quality of the rock mass, joint frequency, overburden or any other geological property. Each measure has a minimum lasting time of 5 min. The measures are continuously repeated until two measures in a row give the same inflow value.

The formula used to assess the Lugeon value is:

$$(V) \quad L = \frac{Q}{t \cdot l} \cdot \frac{10}{p - p_0}$$

where

Q = water loss (L)

t = time (min)

l = effective borehole length (m)

p = pump pressure (bar)

p_0 = pore pressure in the rock mass (bar)

These water loss measures are to be performed until a relationship between the inflow and the water loss for miscellaneous conditions such as geological features, overburden, etc can be established. Temporary upper limits of the number of supplied holes in the control fan at pre-grouting and the regular pre-grouting fan depends on the both the water loss and leakage in single holes and a collection of holes. Over time, as experience and knowledge about the geological and injection conditions are gained, these values can be altered. (Jernbaneverket 2011) The values are found in table 7.

Table 6. Upper limit values for water loss and inflow in boreholes at different inflow requirements. (Jernbaneverket 2011)

Inflow requirement ($\frac{L}{\frac{min}{100 m}}$)	<5	<10
Water loss value ($Lugeon$)	0,3	0,5
Leakage in single hole ($\frac{L}{min}$)	0,6	0,9
Accumulated leakage in 3 holes ($\frac{L}{min}$)	1,0	1,8
Accumulated leakage in 6 holes ($\frac{L}{min}$)	1,7	3,0

6.9 Construction of dams and measurements of inflow in the tunnel

To measure a possible inflow from boreholes, joint systems, bolts, etc. several measuring thresholds in the form of dams are to be built inside the station hall. This is especially important at a border between different inflow criteria. The dams will be constructed in concrete. Prior to the construction of the dam loose slabs of rock have to be removed and the tunnel floor has to be sealed with Bentonite clay. A possible grouting of the dam itself once finally constructed must also be considered in every case. (Jernbaneverket 2011)

Measuring borders are to be installed both north and south of the station hall. Dams will be constructed on the border between the station hall and the entrance tunnels R7 and R8, as seen on figure 7. Behind the dams, concrete wells will be installed, at a lower level than the theoretical tunnel profile. The wells are marked at 2 spots so that the volume between these spots is known. The procedure itself consists of pumping water out of the wells to a point below the lower level to drain out the stored water. Time for refilling the area between the 2 spots is then taken and the leakage water is assessed. To isolate the station hall when locally measuring the leakage water, runoff water from the measuring borders must be temporarily pumped past the station hall. (Jernbaneverket 2011)

Should there be any problems with the measuring of water inflow from the dam, a container with a minimum volume of 1000 L has to replace the dam for the measuring works. Also, a pump has to be present so that the water in the tunnel ditch can be pumped into the water container. Usually, these water measurements are performed using a plastic bucket or a plastic bag over several minutes.

Experience has shown that measurements like these require planning. Since there are various types of works and operations in the tunnel, several regulations apply. The requirements can be seen in table 8. (Jernbaneverket 2011)

Table 7. Requirements for water measuring in the station hall. (Jernbaneverket 2011)

Requirement number	Description
1	Measures must be taken at a holiday or a long weekend, either Sunday afternoon/evening or early in the morning of Monday or the first day after a holiday
2	All injection rods must be inserted in place so that no seepage occurs and bolt holes sealed during the final day before the holiday
3	All hoses, rods and junctures are to be controlled at the final working day before measuring. The drainage ditch has to be cleaned at least 2 days before measuring, and 4 days at maximum. The tunnel operations requiring water must cease for at least 48 hours. No water under pressure must be present in the tunnel at this time. During these hours, inspection must take place by authorised personnel

6.10 Alternative and supporting methods to grouting

Often, the costs of grouting an entire tunnel to ensure no water inflow can reach a high level cause the demand for other methods such as full concrete lining which seals the tunnel inside a waterproof layer of concrete. Water infiltration may also serve as a support method for grouting and/or prevent fluids from escaping cavern storages, etc. (Davik, Kveen et al. 2002)

6.10.1 Concrete lining

Functioning more like a support method for grouting in itself, though perfectly able to constitute the lone method of waterproofing the tunnel, concrete lining is especially useful in urban areas with strict inflow demands and pore pressure reduction requirements. The costs of filling all joint systems to ensure that the inflow criterion is not violated may reach high levels thus welcoming the concrete lining to help fulfilling all demands. Before the lining takes place, the tunnel surroundings should be drained including the concrete structure itself. Drainage holes used to extract the water are later sealed when the concrete structure has gained adequate fortitude.

The waterproofing itself is materialised through the usage of different kinds of membranes like PVC, PE or asphalt/bitumen. The membranes, which are to be attached to the rock mass demand a smooth surface, and as an extra protection for the waterproof membrane, a coating will be firstly attached to the rock mass surface, before the membrane is installed. The protection coating also has a draining function, which in drained tunnels works as a permanent solution and in non-drained tunnels as a waterproof working solution before the concrete layer is being installed. This method requires utmost attention and precision as well as carefulness since the coating is prone to being damaged, especially when installing armed concrete covers.

The concrete is applied onto the rock surface by the aid of a special shield which is attached to a truck. Prior to the lining all slabs of loose rock must be removed. Bolts and shotcrete must be utilised as means of working protection. The concrete lining process start at the tunnel floor and is gradually directed upwards. (Sve, Elvøy et al. 2008) An example of a shield can be seen on figure 32.



Figure 32. A shield used for concrete lining in the entrance tunnel to the Melkøya tunnel. (Sve, Elvøy et al. 2008)

In combination with grouting this method proves especially worthy at weakness zones with significant leakages. By cladding the tunnel walls in concrete lining the leakage can be directed through the drainage holes in the concrete while injecting at a fairly high pressure in the rock mass since the lining works as a wall. (Davik, Kveen et al. 2002)

7 Water and frost protection

7.1 General introduction

Water inflow and dripping into the tunnels and the station hall will have great consequences at the electrical installations scattered around. Ice formation will also occur in winter, making train driving fairly dangerous. (Multiconsult 2008)

To ensure good maintenance of the water and frost protection, several features are required, which can be seen in table 9.

Table 8. Suitable features and properties of the water and frost securing of the station hall. (Multiconsult 2008)

Property number	Description
1	Inspection shafts on either side of the tunnel, easily accessible
2	Ladders on either side to reach the shafts
3	Necessary space between the vault and the tunnel profile.
4	Safety line attachments behind the vault, when inspecting the vault
5	Profile of the water and frost securing must reach the tunnel floor on either side, to the inclination plane (the skewed bottom of the tunnel floor, leading water into the drainage ditch)

Usually the inspection shafts are found at the bottom of the tunnel profile, close to the tunnel floor. For a human being to move and reach the top of the vault, a necessary amount of space is required. This is one of the reasons for having the theoretical profile 60 cm wider than the normal profile. Another reason is to make concrete lining possible without having to expand the profile, unless the tunnel floor is also to be lined. (Multiconsult 2008) To reach the upper part of the tunnel and study the tunnel ceiling a ladder is appropriate. Attachment points for the safety line are necessary to prevent causalities and injuries.

A sketch of a common water and frost securing profile can be seen on figure 33.

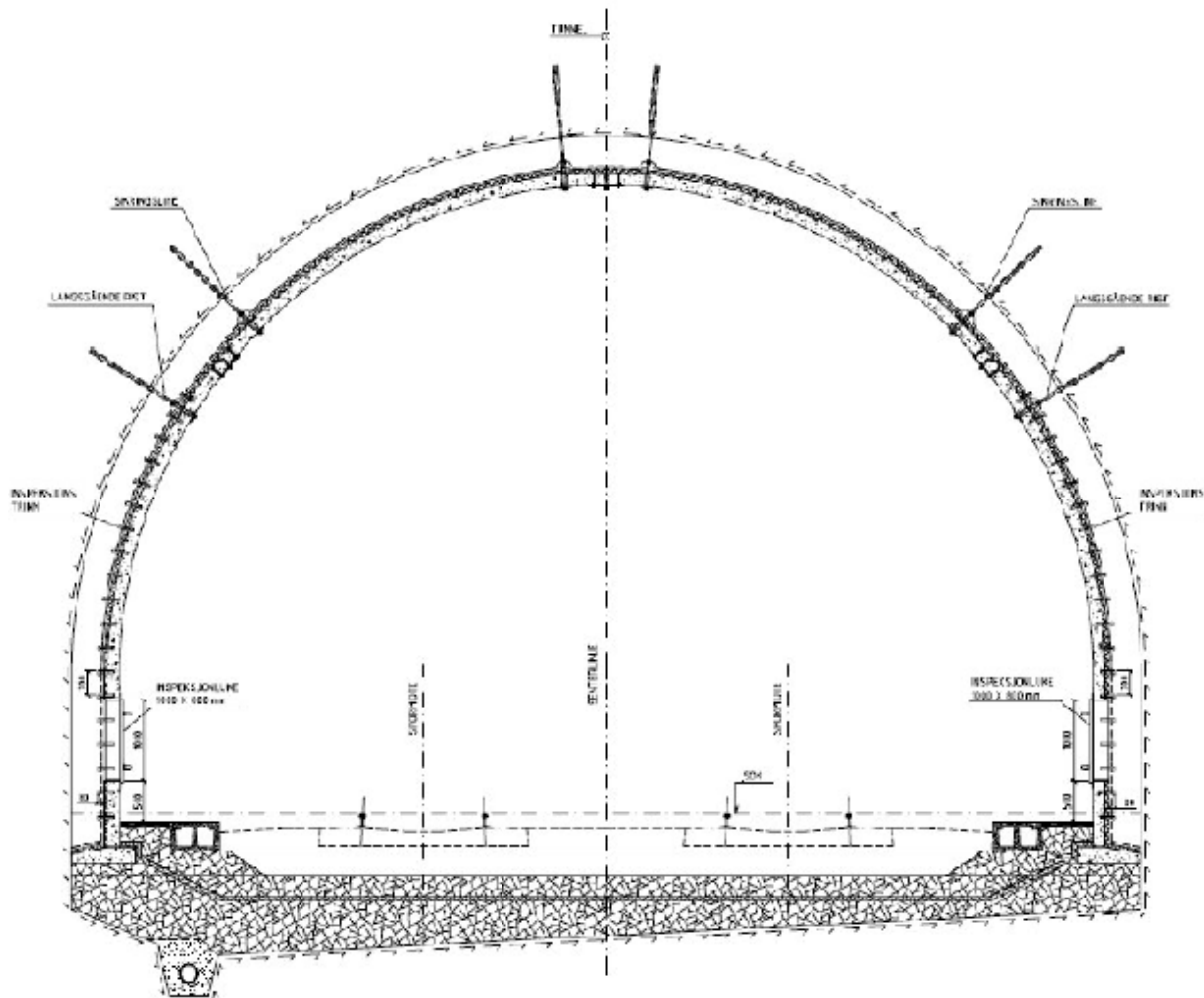


Figure 33. Sketch of a common water and frost profile for the station hall. At the bottom of either side there are inspection shafts, with safety lines behind the vault. (Multiconsult 2008)

For the Holmestrand station hall, PE foam plates and concrete elements are both eligible. At the time of writing no decision on whether to only use one method or both has yet been made. The author assumes that either one or both in combination will be chosen as the method for water and frost protection.

7.2 PE foam plates

This method will cover the whole profile of the station hall, from the sinistral to the dextral part of the inclination plane. The normal profile will be the outer border to the railway and station area with a quadratic bolt pattern having a distance of 1,2 m between each bolt. This outer border will serve as a shield being composed of fibre reinforced shotcrete with a minimum size of 70 mm. This outer layer of shotcrete will compose the fire protection wall against the PE foam plates which are found behind this layer. Thickness of the PE foam plates are approximately 55 mm.

Table 9. Positive and negative properties of the PE foam plate solution. (Multiconsult 2008)

Advantages	Disadvantages
Lower cost	Must be controlled frequently to prevent concrete from falling down, especially at joints
Easier to adapt the geometry at changes in the cross-section	Difficult to maintain and cleanse, time and cost consuming
Easier to repair when minor damages occur (in other solutions, such as concrete elements, these damages will usually not occur, since they are more robust)	PE foam is punctuated by a great number of stays. This causes seepages which further lead to icicle formations and the need to salt
	At times too great span between dilatation joints, causing them not to resist movements, leading to damages, making concrete slabs fall out, leading to seepages. Most dilatation joints still have been installed without crack indicators

Though the PE foam plates pose the problem of being a construction prone to inflow because the lack of strength and fortitude compared to other solutions, it is generally a flexible method. Should the current installation not be satisfactory during an inspection, additional parts of the tunnel can be secured at a later occasion. This means that the securing installation does not have to be chronological, it can be scattered throughout the entire tunnel. However, maintenance will be cheaper when the whole tunnel length is equipped with the same waterproofing solution. (Multiconsult 2008)

To the author's knowledge, PE foam plates in railway tunnels have also been widely used in Norway during the construction of new tunnels in the later years. It is expected that both the contractor and the entrepreneur of the station hall know this solution very well and that previous results of this method have proven satisfactory. This is probably the major reason why this support method has been picked out amongst the other ones.

7.3 Concrete elements

When using concrete elements, the station hall will be totally covered in these along the whole tunnel line where water and frost securing is needed. Pre-fabricated concrete elements will constitute this support method. The elements consist of armed concrete which has a thickness of 150 – 180 mm. Should frost isolation be necessary, a 50 mm XPS or PE plate will be inserted. The water securing is a homogenous PVC membrane that will be suspended in on the rock bolts and welded in place before the concrete elements are installed. The concrete element solution is visualised on figure 35.

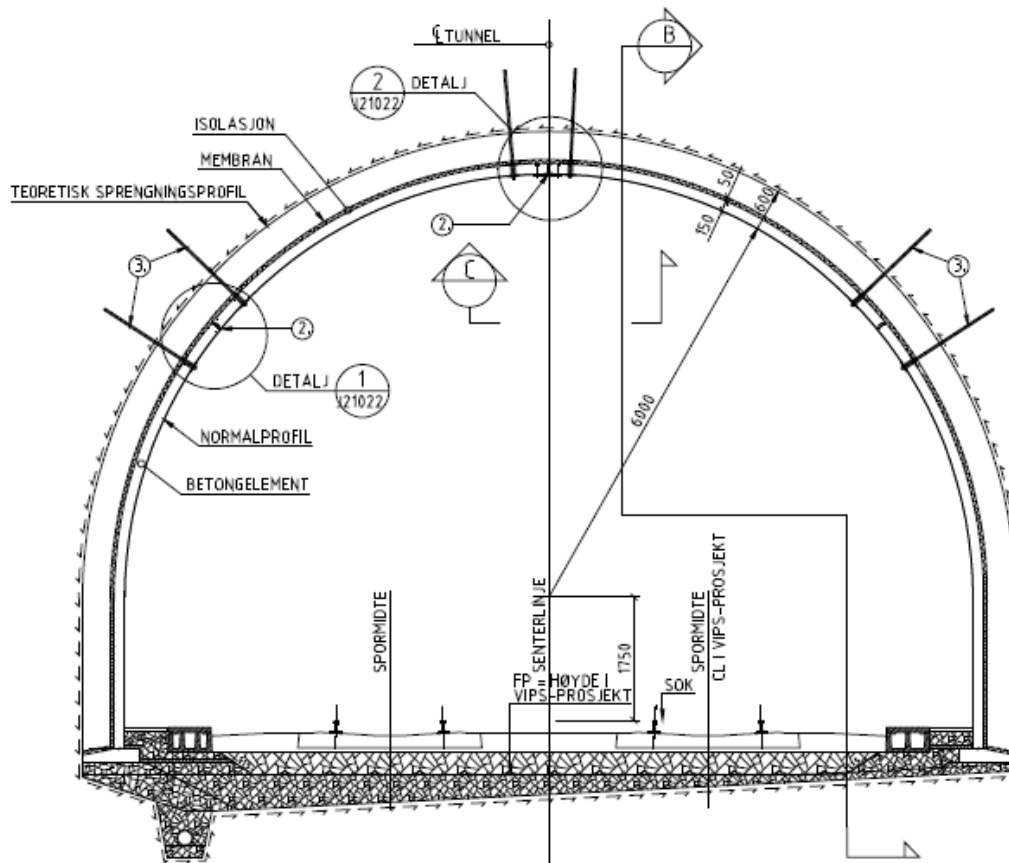


Figure 35. Sketch of the water and frost protection method using the concrete elements. (Multiconsult 2008)

The tunnel profile will be divided in 4 parts where the concrete elements cover the tunnel walls at approximately 3,5 m over the railway on either side. The tunnel vault is to be coated with two concrete elements in the middle of the tunnel ceiling with a joint separating the elements. The wall elements are 5 – 6 m wide while the tunnel ceiling elements will be half of that. The lower part of the wall element is to be attached to a fundament while the upper part is being screwed in place with 2 rock bolts. In the joint between the wall element and the tunnel ceiling element a neoprene band will be inserted. All concrete elements are being held together by steel ribs making no rock bolt critical considering the suspension of the elements. The distance between the theoretical profile and the concrete elements will be 400 mm where no significant rock support is needed. (Multiconsult 2008) A collection of advantages and disadvantages for this method are located in table 11.

Table 10. Advantages and disadvantages when using concrete elements. (Multiconsult 2008)

Advantages	Disadvantages
Aesthetic interior of tunnel, bright surface	Thorough controlling of bolts and stays are necessary
Easy cleaning procedures	At replacing works, the working amounts will increase. Special orders are needed for concrete elements
Elements can be replaced should the need be present, though proving both time and cost consuming. Small probability that this will happen in a railway tunnel.	Numerous joints to maintain. The joints have gotten better over the years but still constitute a major work load
Water leakages do not pose a problem	Inspection works will have to take place at a reduced space. This has previously been a general problem, however it is expected that by increasing the distance between the theoretical and normal profile to 60 cm, circumstances will be more comfortable

7.4 Other water and frost protection methods

7.4.1 Sprayed membranes

Various sprayed membranes exist today. One of these is the BASF masterseal 345 which is an impermeable plastic membrane that can be embedded within a shotcrete structure. With a high bonding strength the membrane bonds to the shotcrete on both sides. The main idea is to regard this as a composite structure. In this description, the membrane used is an etyl-vinyl-acetate co-polymer. Simply using a handheld nozzle, the membrane is sprayed onto one of the layers of shotcrete. It is then left to harden for several days before a new layer of shotcrete is applied onto the membrane. In this case, tensile bonding is critical to achieve full confining in the sandwich structure. Due to this reason, the mechanical properties of the substrate surface are important in order to gain this mechanical interaction. It also means that the surface will have to be clean. (Holter, Nermoen et al. 2011)

Additionally the roughness of the surface needs to be minimalised, making the bonding of the membrane to the shotcrete layers easier. Figure 36 shows such a composite sandwich structure.

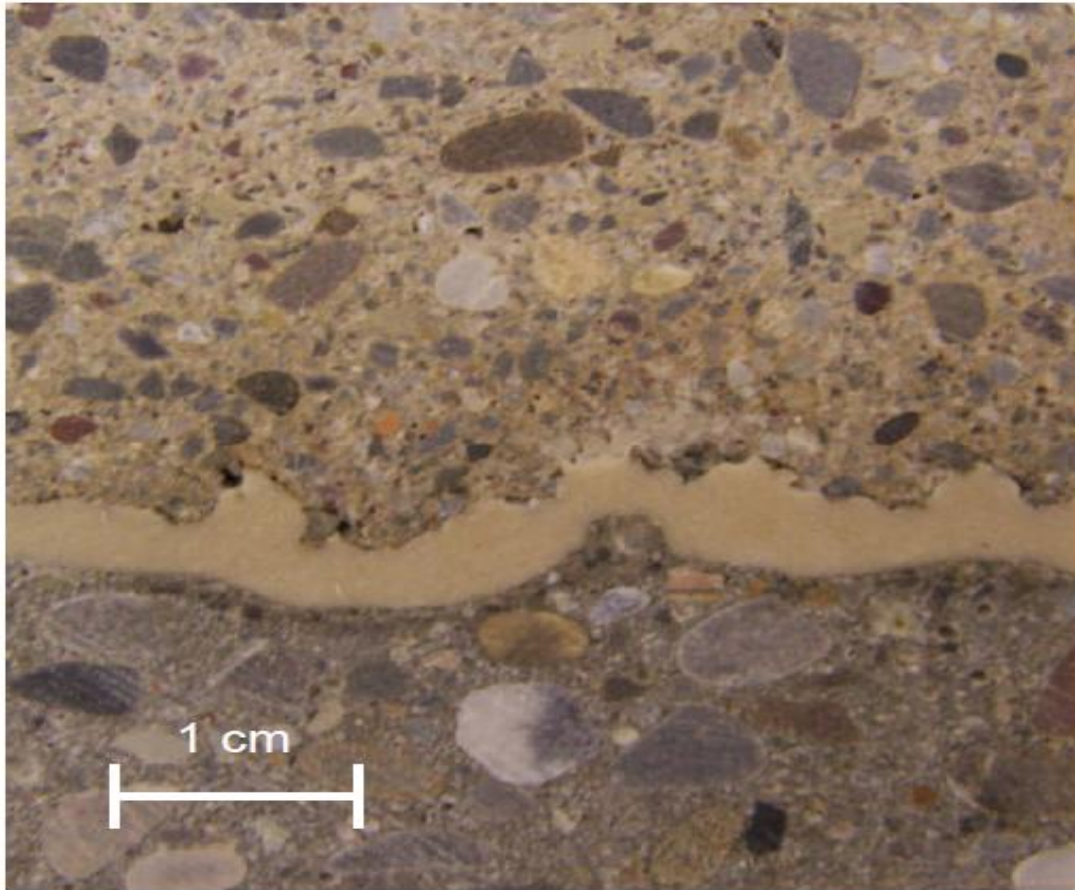


Figure 36. Photograph of the sprayed membrane sandwich structure. The upper part shows the rock support layer of shotcrete, the lower part is the inner lining. The membrane, in the middle, bonds chemically and mechanically to either side of the shotcrete. (Holter, Nermoen et al. 2011)

In order for the membrane to work properly, it needs to have a thickness of at least 2 mm. As previously mentioned, the tensile bonding on both sides of the membrane is important. The tensile bonding of the membrane creates certain features making it different from other methods of water and frost securing, like PE foam sheets and drainage geotextile.

A cavity or imperfection in the membrane does not necessarily mean a seepage into the tunnel since the point of cavity must coincide with a seepage point in the shotcrete. Also, since water cannot migrate along the interface of the membrane, a possible seepage point in the membrane can be fixed locally exactly at the point of seepage. This is so because this point corresponds to the seepage channel in the concrete behind the membrane.

One of the main technical features of this system is to regard it as a mechanically continuous structure. It is also viewed upon as a composite structure, since it consists of 3 different layers. Some of the main technical properties of the composite structure are the mechanical strength parameters which relate to the bonding interfaces between sprayed concrete and the membrane.

Usually it is possible to achieve a higher tensile bonding strength than 1 MPa. Reduction of the roughness of the shotcrete surface may yield a tensile bonding strength of up to 2 MPa. This causes the membrane between the shotcrete layers to be impermeable, hence preventing any fluid from flowing into the tunnel. (Holter, Nermoen et al. 2011)

Design of the tunnel is another matter that comes into hand when determining the composition of the structure. Virtually it can be distinguished between two cases, namely a drained and an undrained situation. Figure 37 displays the two different situations.

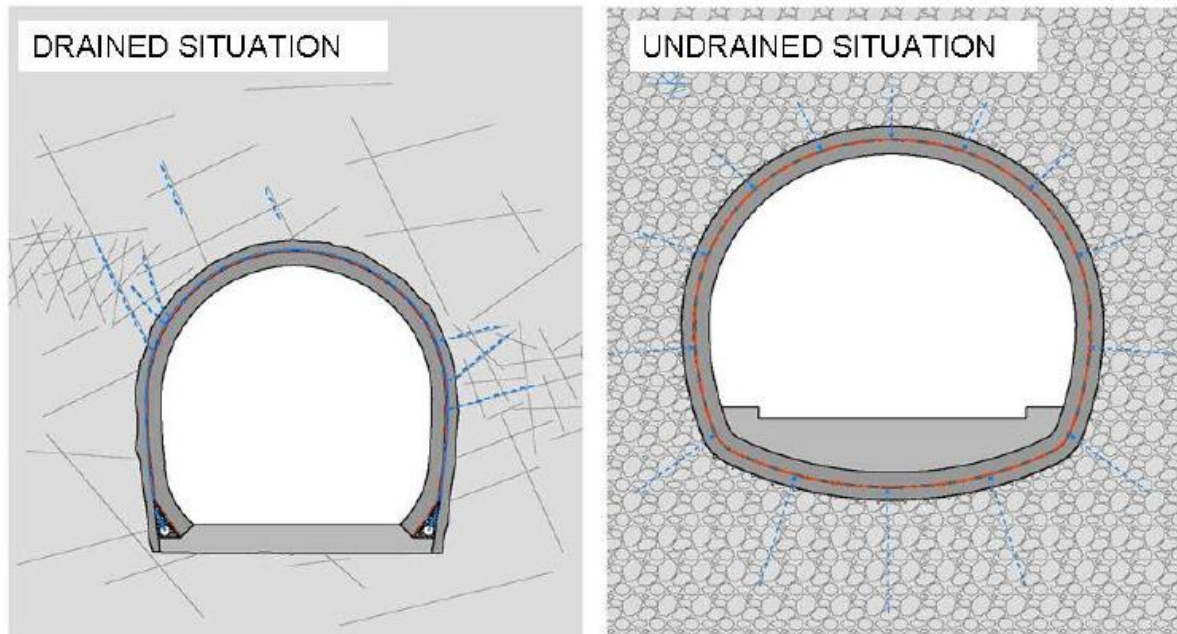


Figure 37. A sketch of a drained and undrained situation, and the securing application of both of the cases. (Holter, Nermoen et al. 2011)

The undrained situation makes the need for support and securing greater than that of the drained one. The latter case means that the water is left to migrate into the drainage channels of the tunnel. The rock mass is thus being reinforced, working as the support means itself. The first case, the drained situation, causes the tunnel to be exposed to full hydrostatic pressure. This makes the entire tunnel lining waterproofed.

The ability of the membrane to carry load has been tested. This is due to the fact that the composite structure is being regarded as the final support means of the tunnel. One way of determining the strength of the structure is to carry out energy absorption tests of circular samples of shotcrete. This gives a realistic load simulation of what can be expected as a load scenario for a shotcrete support.

Using a sample of pure fibre reinforced shotcrete and the sandwich structure with the sprayable membrane, energy absorption tests were carried out on both samples. (Holter, Nermoen et al. 2011)

Figure 38 shows the results of these tests.

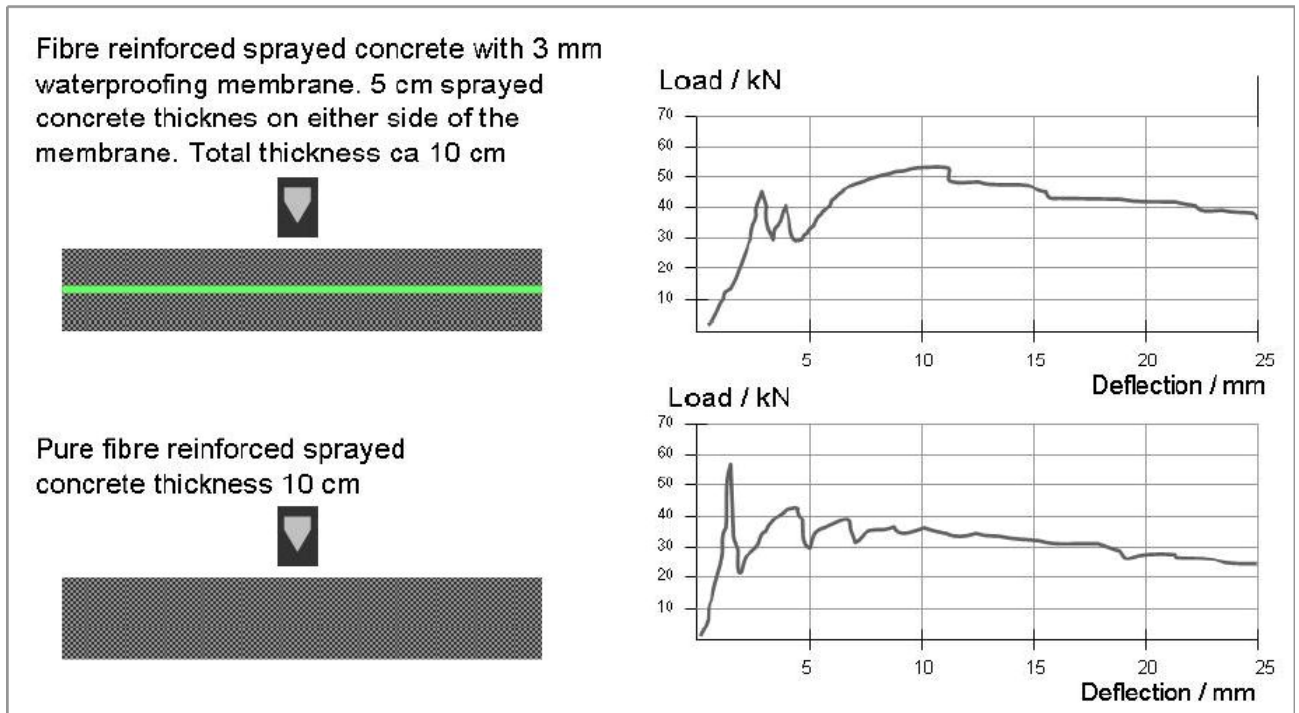


Figure 38. The results from the tests. In total, three samples of pure fibre reinforced concrete and the composite structure were tested. (Holter, Neramoen et al. 2011)

When studying the results of this test, there are slight differences between the pure fibre reinforced shotcrete and the composite sandwich structure. The fibre reinforced shotcrete may handle a greater load, thus failure occurs at a greater value but at less deformation. When studying the behaviour of the samples after 5 mm of deformation, several interesting aspects occur. The composite structure is able to handle a greater load than the fibre reinforced shotcrete, and the load maximum is actually greater than the load at the initial failure. From a mechanical perspective, it is possible to see the composite structure as a means of final tunnel support. (Holter, Neramoen et al. 2011)

7.4.2 WG tunnel sealing membrane

The WG tunnel sealing membrane is a PVC membrane developed for the sealing of humid rock walls and ceilings. The system is simply attached to pipes in the tunnel ceiling and upper tunnel walls using lifts and manpower. The membrane has a thickness of approximately 0,54 mm and a weight of $600 \frac{g}{m^2}$. In areas where F_{10} does not surpass $3000 \text{ h}^\circ\text{C}$, handbook no. 163 from NRA states that non-isolated constructions may be used. This is the upper limit of which the WG tunnel membrane has been approved for usage. A correct and undamaged WG tunnel membrane will shut the air between the rock mass and the membrane, working as an impermeable layer, making it a partially isolating ability. (Grøv 2011)

7.4.3 Ørsta composite solution

The producer of rebar bolts, Ørsta, is working at a water and frost securing method consisting of 70% glass and a bonding material. Each cartridge which is made of the composite material, has 3 mm walls and a thickness of 5 – 6 mm and can be sliced to a preferable length. At either end is an opening suitable for inserting isolation material like rock wool. A common length is 3 m with a height of 60 mm. The cartridges are to be attached to arches with c/c about 3 m and the arches are anchored with rock bolts c/c approximately 2 m. Using 10 mm bolts the cartridges are attached to the arches. The construction has yet to pass the fire test as well as several other tests and is currently not in production. Mainly to be utilised for road tunnels the construction may also be installed in railway tunnels but must be reinforced to withstand the pressure and sucking forces occurring. The cost rate will probably be some levels above the PE foam solution. (Multiconsult 2008)

7.4.4 Water fans

In tunnels with less strict inflow demands, PE foam plates with or without shotcrete, pre-fabricated concrete plates, pre-fabricated vaults with steel or concrete structures as bearing devices. The water is then drained behind the constructions and should the need for prohibition of ice formation remain present, the constructions will have to be isolated. (Davik, Kveen et al. 2002)

7.4.5 Water infiltration

The method of water infiltration is strictly a support method for the grouting works and has predominantly two purposes; prohibiting a pore pressure reduction which can cause subsidence and preventing fluid leakages from a cavern storage room. (Davik, Kveen et al. 2002) Only the first purpose will be described in this text.

In order for this method to perform properly, the geological conditions must be well known and documented. By creating a low-permeable zone around the tunnel through grouting, the best effect is reached, though this sometimes proves unnecessary in areas with low-permeable rock types. An important aspect to notice is that the long-term effect of the impermeable fan may be reduced because of particles in the water which will lower the infiltration capacity, thus the need for an infiltration filter. The infiltration fan may be applied in both rock mass and soils.

The water infiltration fan should be ready for use before the tunnel is fully excavated, though it is possible to lift the groundwater table up to its previous state if the water infiltration processes have begun after the excavation works are completed. The pressure of the water infiltration usually is from 0,5 – 10 bar over the original groundwater pressure. The lower overpressure applies to densely populated areas with soil sediments. (Davik, Kveen et al. 2002)

7.5 SINTEF frost laboratory tests

To better understand how the sprayed membrane solutions function in a cold Norwegian climate, a frost laboratory has been built. The knowledge about frost amount and the interaction of cold air between the rock mass and the support in the tunnel is not very well documented. An interesting aspect to study is how far into the rock mass will the so-called zero-isotherm reach and how heat between the rock mass is and the tunnel transferred through the water and frost securing. (Grøv 2011)

The frost laboratory itself consists of four rooms. One room to simulate the tunnel, containing a freezing device, one room filled with the BASF masterseal 345 sprayed membrane solution, one room to simulate the rock mass behind the tunnel support and one room to cool down the rock mass room. With the desire to have fairly homogenous material granite from Støren composes the rock mass room. A sketch of the frost laboratory can be seen on figure 39.

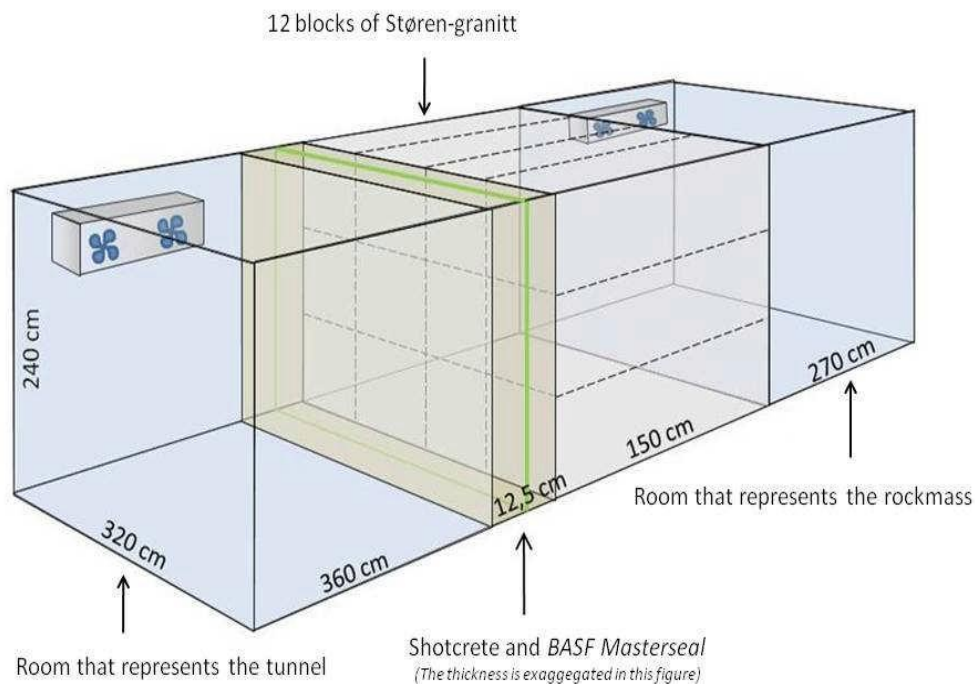


Figure 39. Sketch of the frost laboratory of SINTEF containing all rooms. (Holter and Neramoen 2011)

On figure 40, the sprayed membrane solution along with the granite from Støren is sketched more detailed.

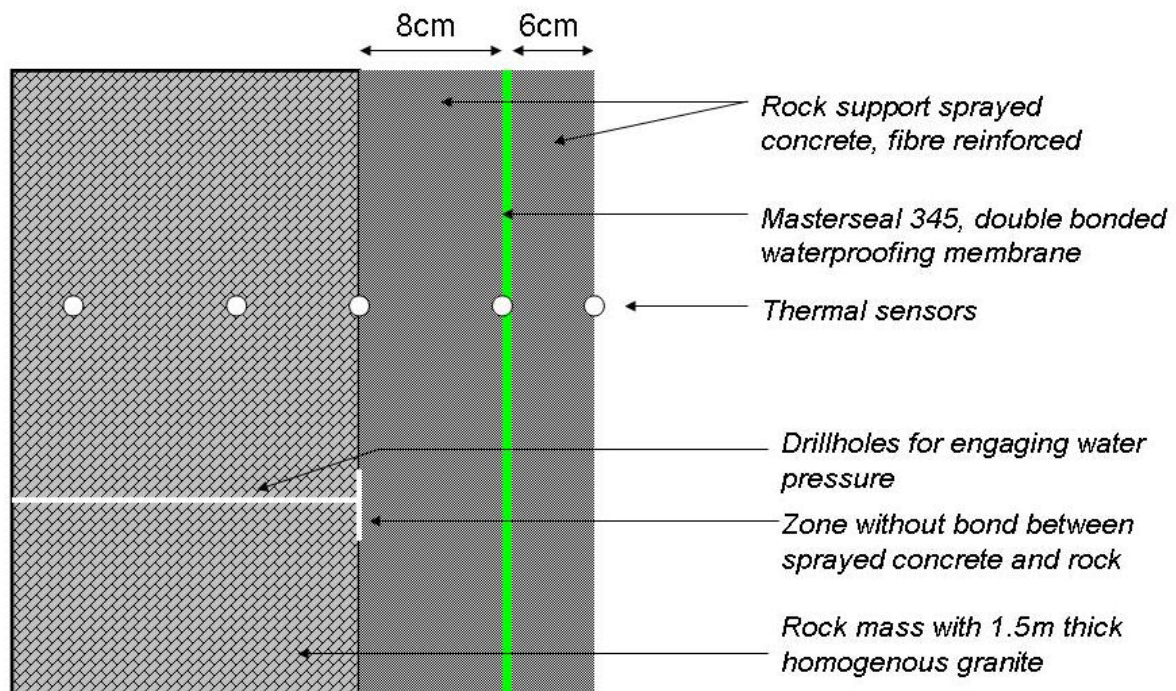


Figure 40. Sketch of the BASF masterseal 345 membrane and shotcrete package installed on the rock mass. (Holter and Neramoen 2011)

The first tests in the frost laboratory showed that the model built reflected very well the actual situation found in a real tunnel. The temperature changes in the model occur instantly, whilst the actual temperature changes in a real tunnel occur gradually. Still, this does not affect the model in any significant way. (Grøv 2011)

Initial tests also revealed that the temperature development in the rock mass was nearly identical in all blocks. Thus, block 2 was picked as a reference block for the tests. This block contains several holes in which the temperature is being measured. The block hole numbers along with their respective depths can be seen in table 12. (Holter, Neramoen et al. 2011)

Table 11. Overview of the measuring depth of all measure points in block 2. (Holter, Neramoen et al. 2011)

Block number	Depth of measuring (mm)
B2-1	20
B2-2	40
B2-3	80
B2-4	160
B2-5	320
B2-6	640
B2-7	1280

There were conducted several tests including some short-durability tests and some long-durability tests. For every time the temperature in the tunnel room was dropped, the frost amount in the rock mass was assessed based on the temperature measured in the holes in the rock mass. The first test took place during 310 hours of freezing. The results can be viewed on figure 41.

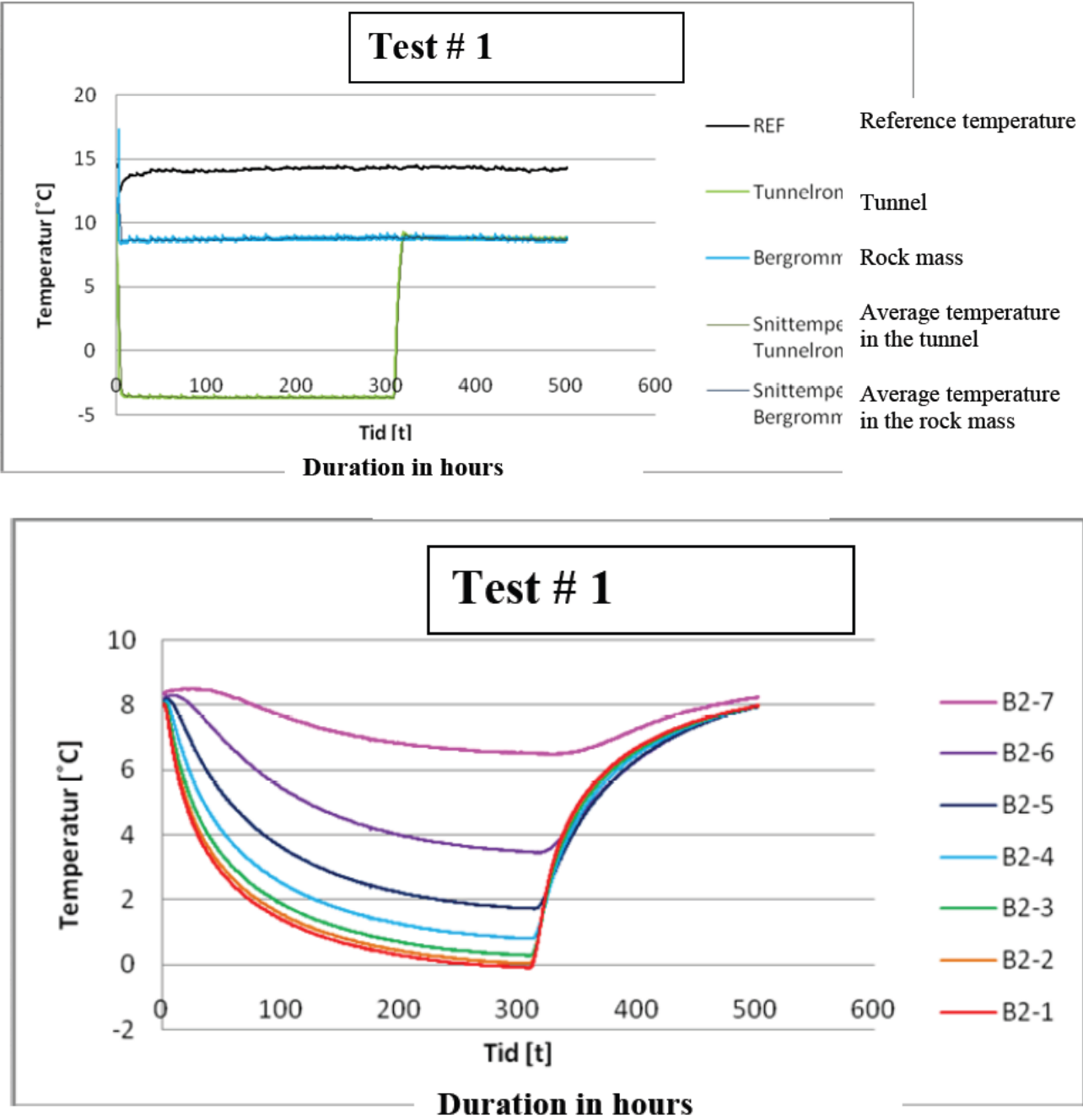


Figure 41. Results from the first test with the temperature in the tunnel room plotted in the upper diagram, and the temperature in the rock mass plotted in the lower diagram. (Holter, Neramoen et al. 2011)

As seen on the diagrams after approximately 310 hours, the test was reversed. At that moment the temperature had begun to stabilise. There was not any reason to believe that the temperature would have decreased any more, thus the test result is valid. As seen on the lower diagram the lowest temperature in the rock mass was in block B2-1, at $-0,1\text{ }^{\circ}\text{C}$. This shows that the freezing index has no effect of how deep the zero-degree isotherm penetrates into the rock mass.

A long-term test was conducted during the summer of 2010 where the temperatures fluctuated between $+5$ and $-8\text{ }^{\circ}\text{C}$ in repeated cycles. In total, the zero-degree isotherm penetrated past B2 -5, but not as far as B2-6 . The results also showed that it takes time for the temperature to reach a stable state in the rock mass. (Holter, Nermoen et al. 2011) The results can be viewed in figure 42.

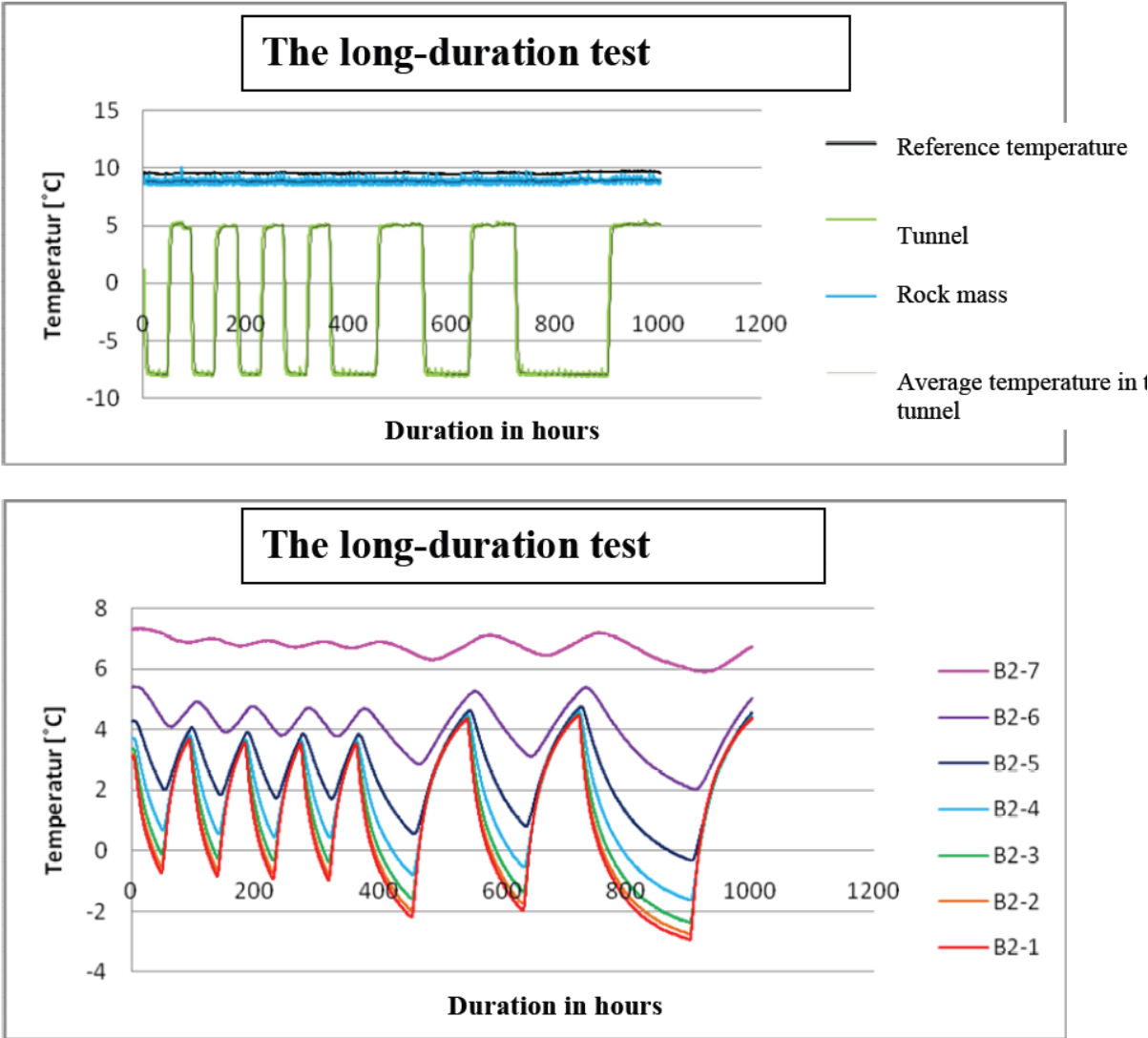


Figure 42. The results from the long-term test, with the temperatures in the tunnel room in the upper diagram, and the lower diagram representing the temperature in the rock mass. (Holter, Nermoen et al. 2011)

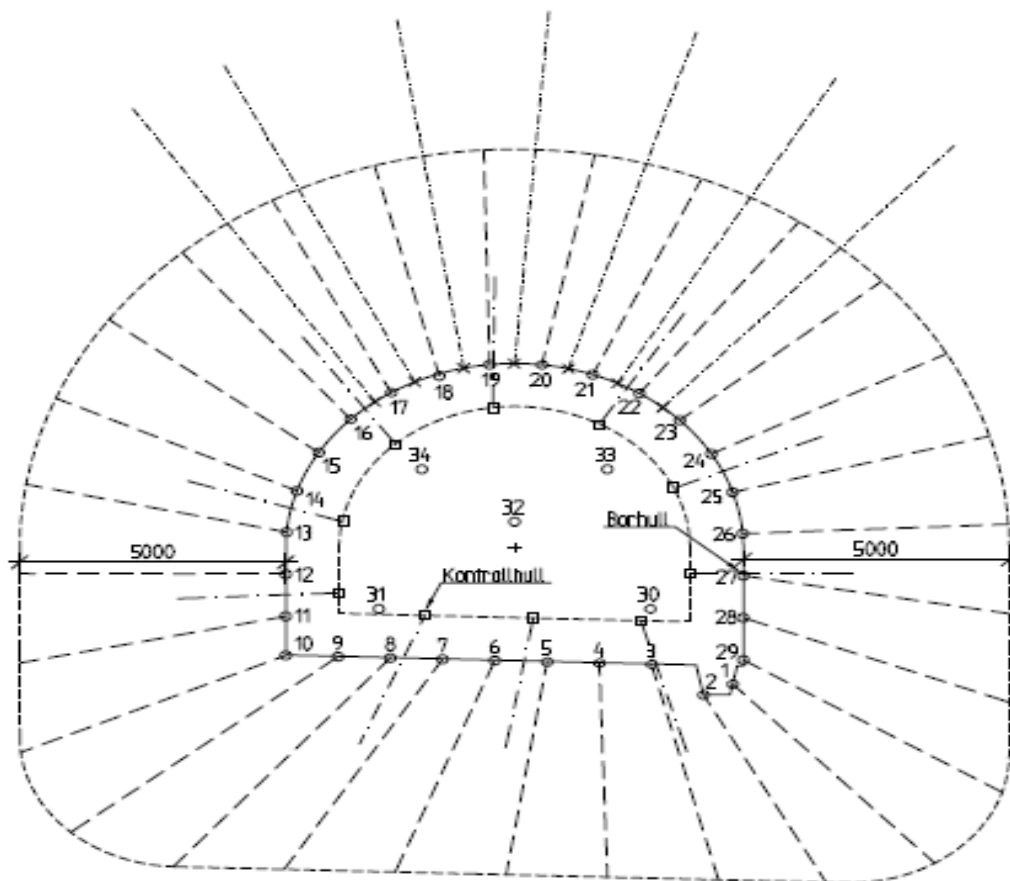
As seen on the diagrams, the temperature in the rock mass returns to its original state once the temperature in the tunnel room is altered between the two temperature limits. A slight delay occurs since it takes time for a temperature to reach a stable state in a rock mass. Though performing repeated cycles, the lowest temperature does not penetrate deeper than the lowest temperature in the previous cycle. However, when the duration increases, the penetration goes deeper. This shows that the temperature pattern is not cumulative when performing the test at the same duration period. In the final and longest cycle, a stable temperature in the rock mass was not reached, it is thought that the test would have to continue another 100 – 150 hours before a stable temperature of approximately $-3\text{ }^{\circ}\text{C}$ in the rock mass would have been reached, with the temperature in the tunnel room being $-8\text{ }^{\circ}\text{C}$. (Holter, Nermoen et al. 2011)

8 Results from tunnel excavations and grouting

Presentations of the grouting results are divided into four parts for each tunnel; total injected grout mass per grouting fan, average pressure per grouting fan, distributed grout mass per borehole for each grouting fan and distributed pressure per borehole for each grouting fan. Subsequently, results from an excursion to check out the sprayed membrane solution in the Gevingås tunnel follow.

8.1 The station hall entrance tunnels

At the time of writing, grouting works have only been performed in R7 and R9. A sketch of a typical grouting fan for both of these tunnels can be seen on figure 43.



Snitt

Figure 43. Sketch of the grouting fan for the R7 and the R9 tunnels. (Jernbaneverket 2011)

8.1.1 Grouting results from R7

The R7 entrance tunnel is approximately 105 m long, with the first grouting fan starting after 30 m of excavation at section 75. There is one grouting report missing thus the leap from section 46,6 to section 18. At the time of writing there are no further grouting reports present but the author theorises that there should be at least one more report from approximately section 0 present. Figures 44 and 45 display the total amounts of injected grout mass and the average grouting pressure per grouting fan in R7. It is worth mentioning that due to the approaching of the wall of the station hall, the final grouting fan of R7 contains 4 additional boreholes.

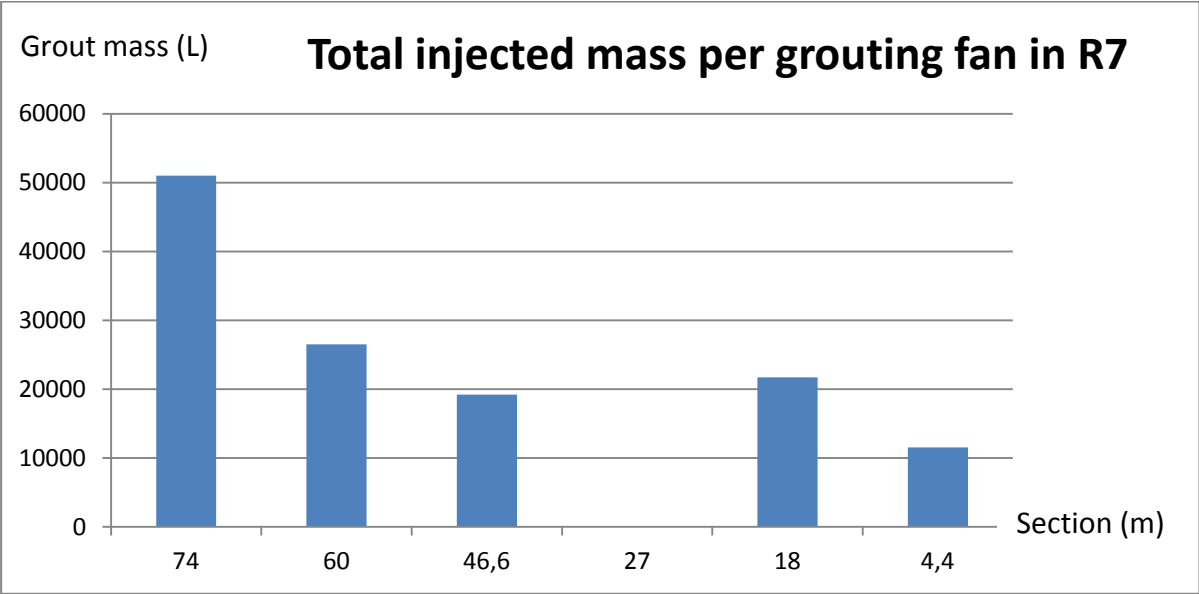


Figure 44. The total amounts of grouting mass per grouting fan in R7.

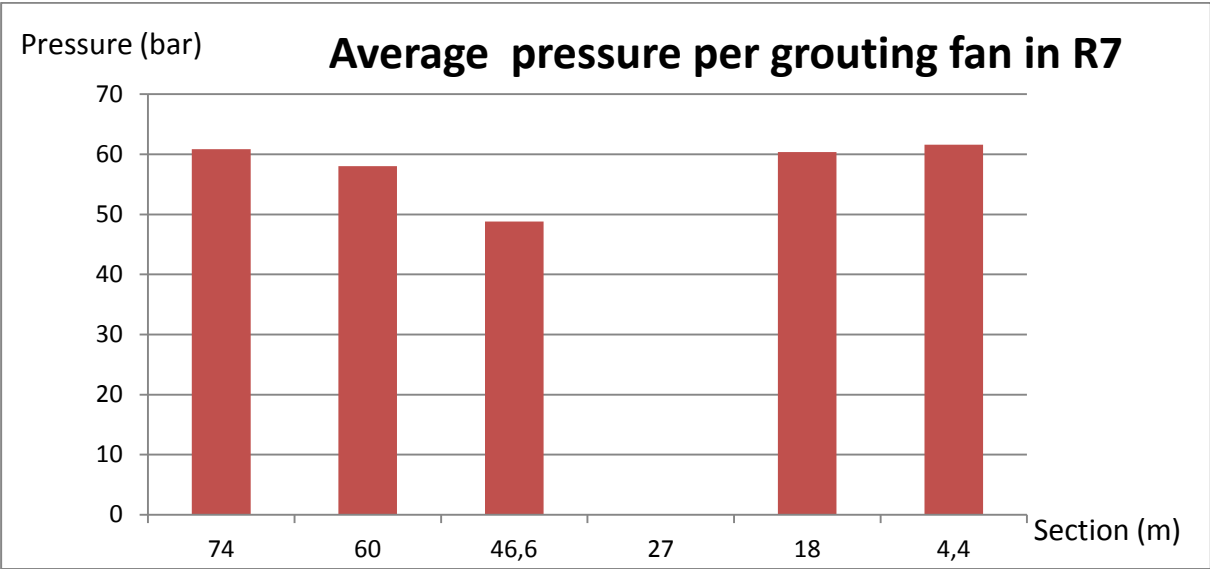


Figure 45. The average grouting pressure per grouting fan in R7.

Figures 46 and 47 display the distributed amounts of grout mass and the grouting pressure per borehole per grouting fan for R7.

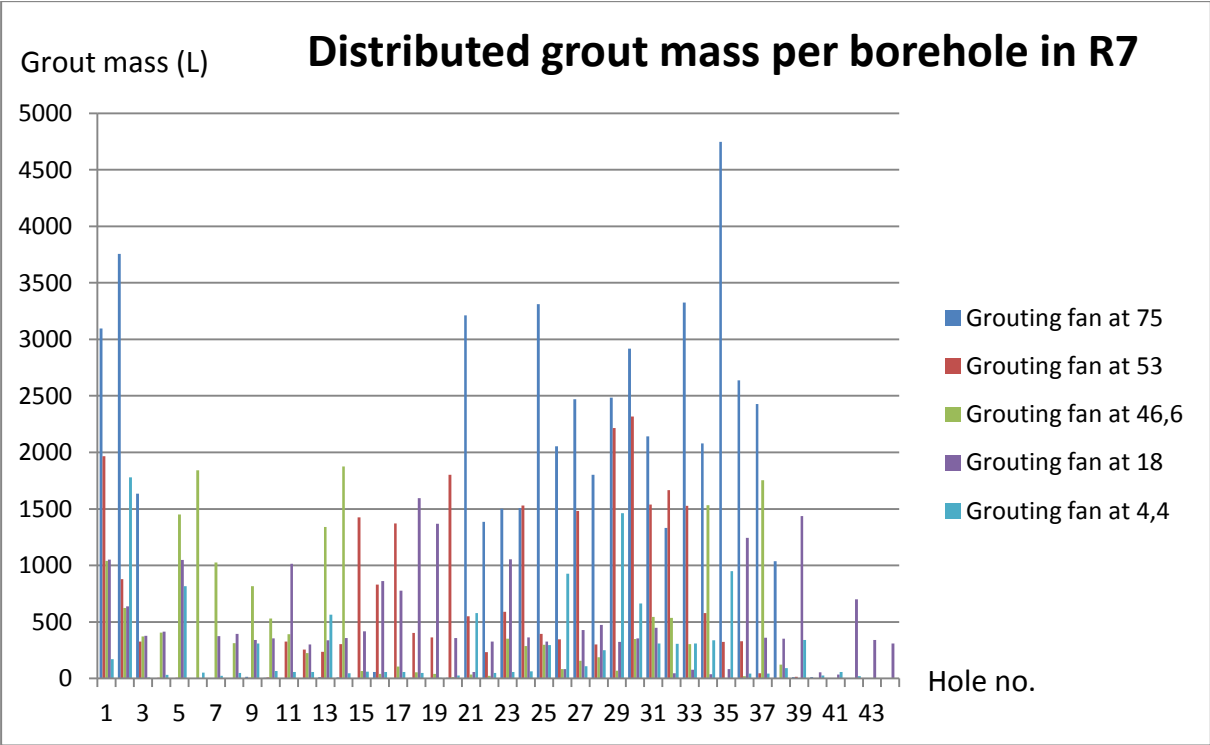


Figure 46. Distributed amounts of injected mass per borehole per grouting fan in R7.

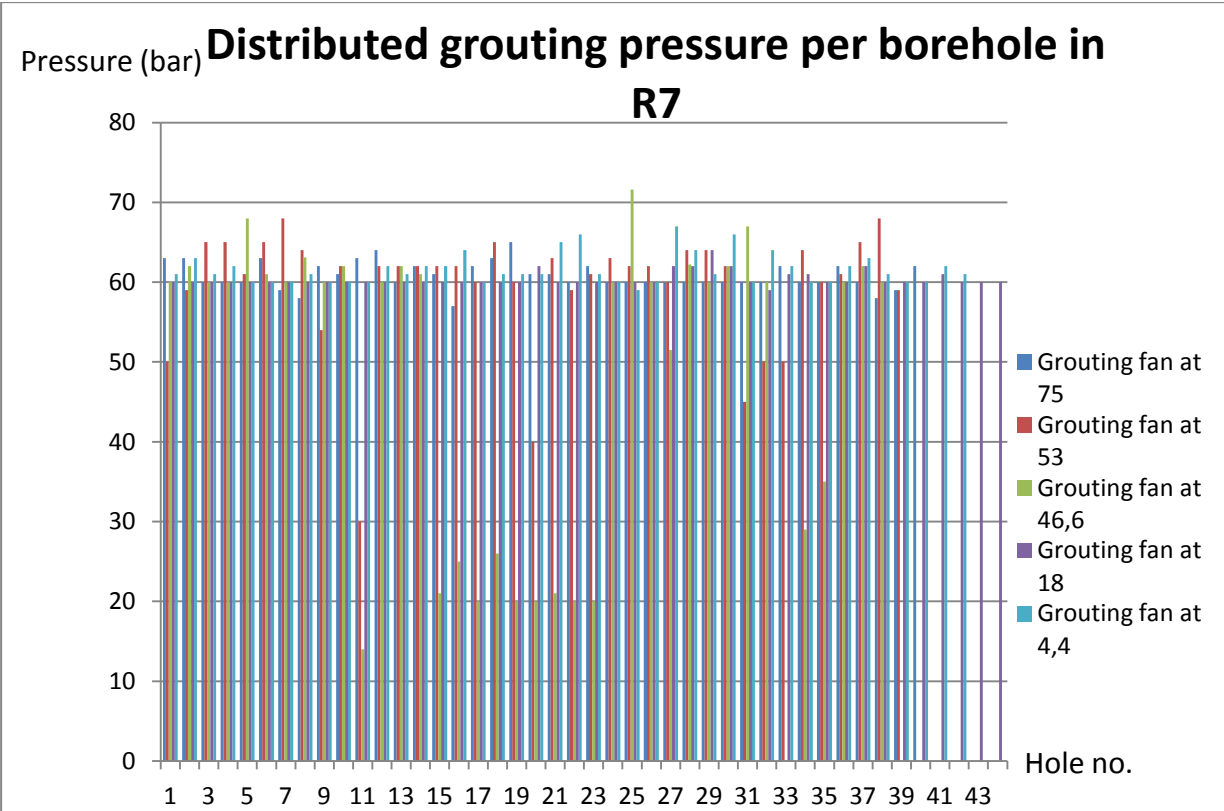


Figure 47. Distributed amounts of grouting pressure per borehole per grouting fan in R7.

8.1.2 Geology during excavation of R7

Rock conditions in all of the entrance tunnels are well. Due to unfortunate and practical circumstances, mapping of the Q-values has been rather scarce. There has been no mapping performed in R8 and R9, however there has been some registered Q-values in R7. Appendix G displays a representative, calculated Q-value from the R7 tunnel.

8.1.3 Grouting results from R9

Due to certain difficulties and complications in labeling each grouting report with their respective section numbers, the visualised results from the tunnel sections might not actually reflect the real position of the tunnel section of the grouting fans as the author has had to rely on oral information and personal adjustments. Still, the locations of most of the grouting fans are adequately accurate. Figures 48 and 49 show the total amounts of injected grout mass and the average grouting pressure per grouting fan in R9, while figures 50 and 51 display the distribution of the grout mass and the grouting pressure per borehole per grouting fan in R9.

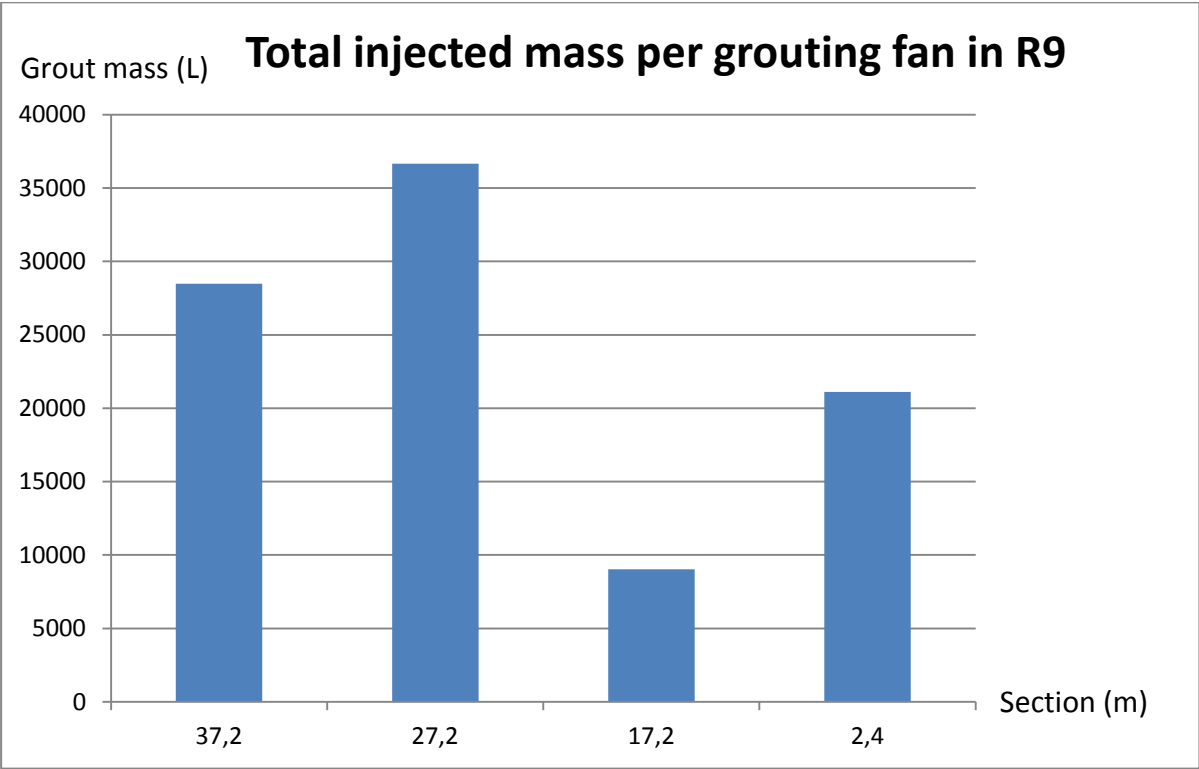


Figure 48. Total amounts of injected grout mass per grouting fan in R9.

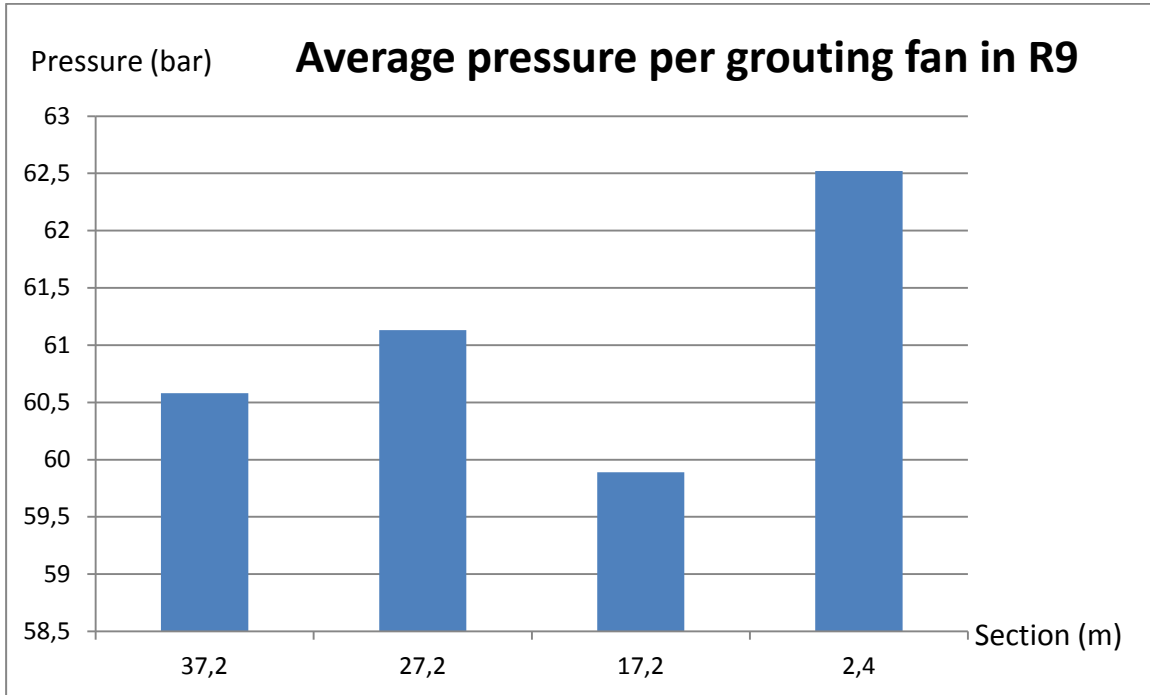


Figure 49. The average grouting pressure per grouting fan in R9.

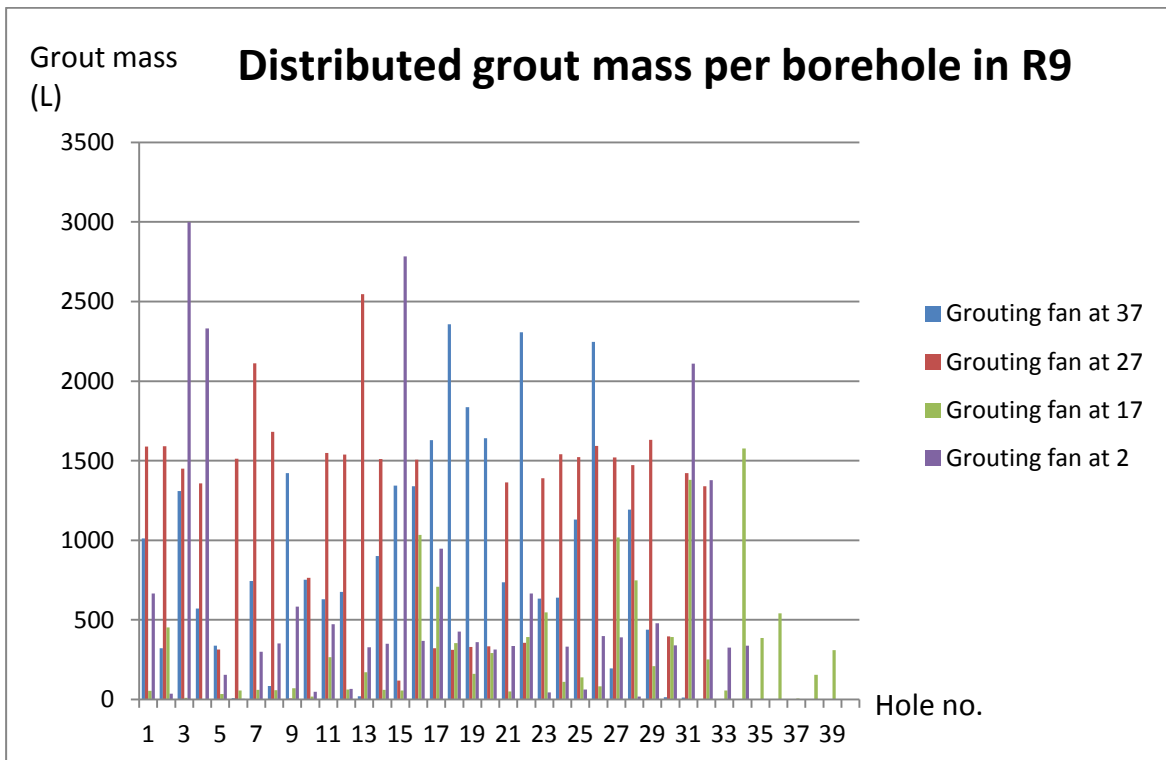


Figure 50. Distributed amounts of injected grout mass per borehole per grouting fan in R9.

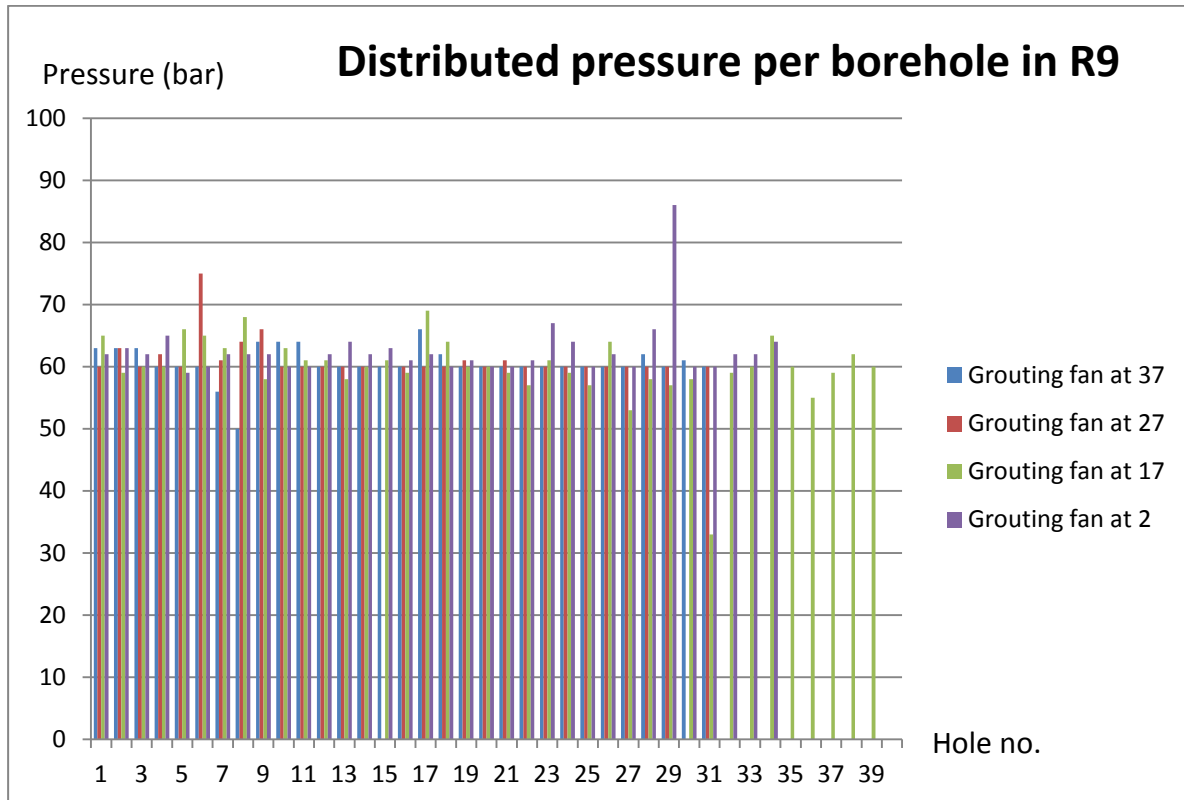


Figure 51. Distributed grouting pressure per borehole per grouting fan in R9.

8.1.4 Geology during excavation of R9

Due to lack of information and documentation of the excavation of R9 there is not much to reveal about the geological features. There have been Q-values obtained that range from 3 – 4 other than that, the basalt in the tunnel really poses no problem. Based on the Q-value and oral information, the rock quality is decent. It is estimated that the rock mass quality is virtually the same as in the R7 tunnel.

8.2 Sjøskogen

In here, only the entrance tunnel and the tunnel direction towards Holmestrand and the station hall will be discussed. This is due to a vast abundance of data and because this direction is more relevant considering the rock mass quality. A sketch of the planned pre-grouting fans with each hole labeled with its unique number, for both the R4 entrance tunnel and the main tunnel section can be seen on figures 52 and 53.

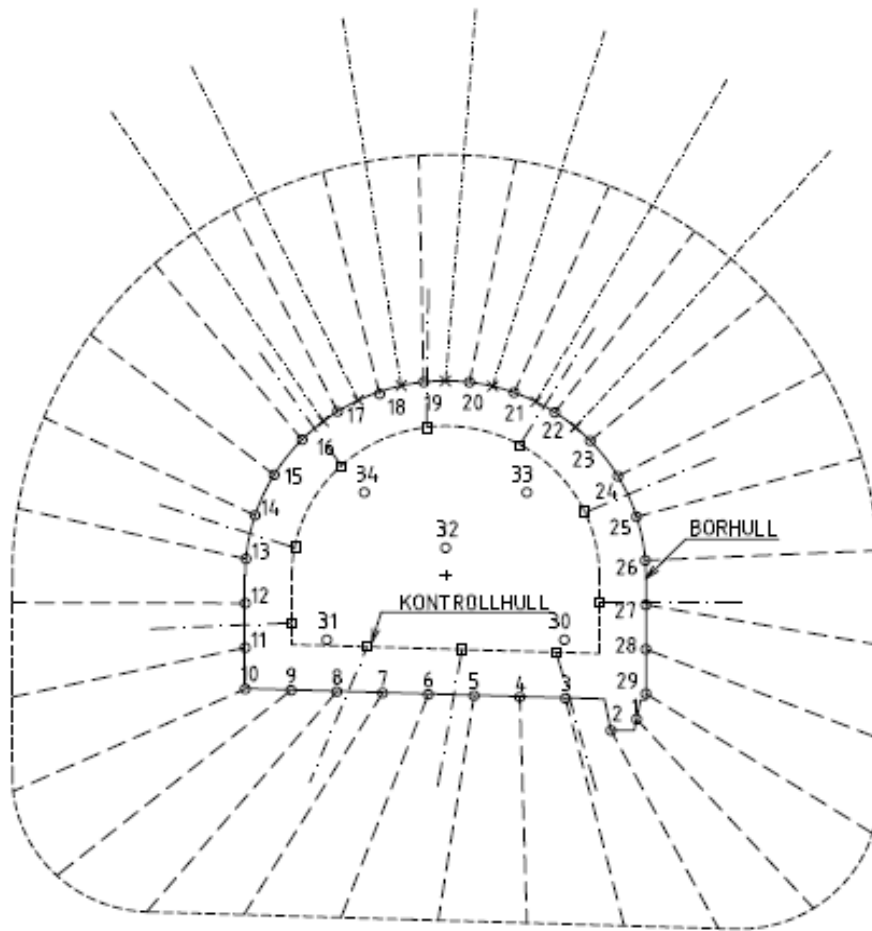


Figure 52. The planned pre-grouting fan for the R4 entrance tunnel at Sjøskogen. (Jernbaneverket 2011)

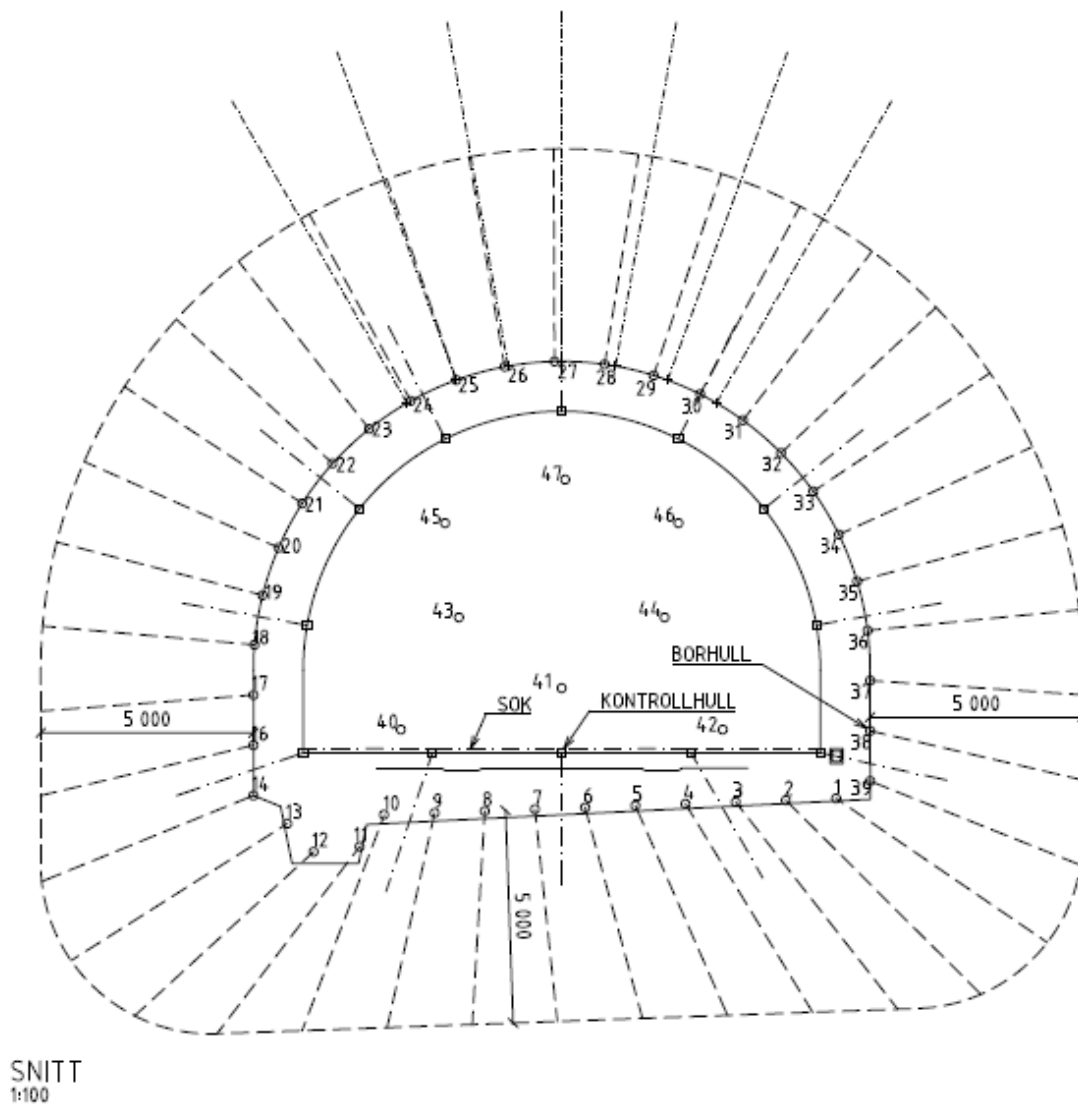


Figure 53. The pre-grouting fan for the main tunnel at Sjøskogen. (Jernbaneverket 2011)

8.2.1 Grouting results from R4

Note that there are no data available of grouting performed from section 3 – 127 due to missing grouting reports.

Figures 54 and 55 display the total amount of grout mass and the average pressure per grouting fan of the R4 entrance tunnel.

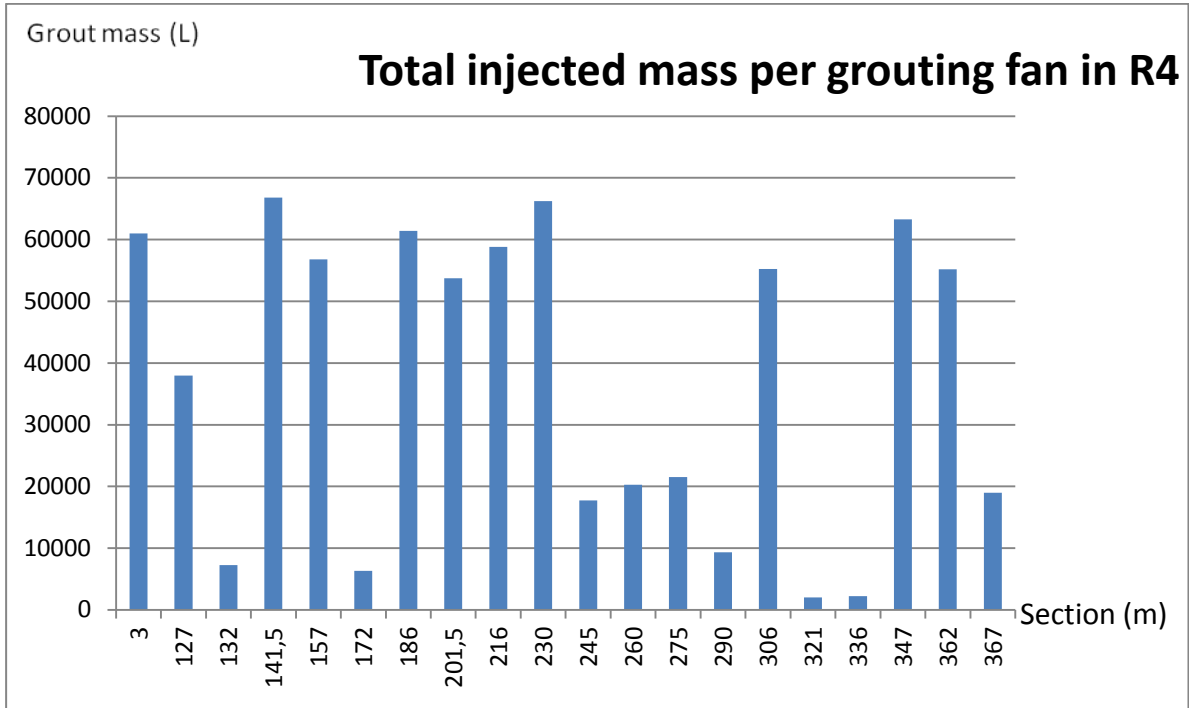


Figure 54. Total amount of grout mass per grouting fan in the R4 entrance tunnel.

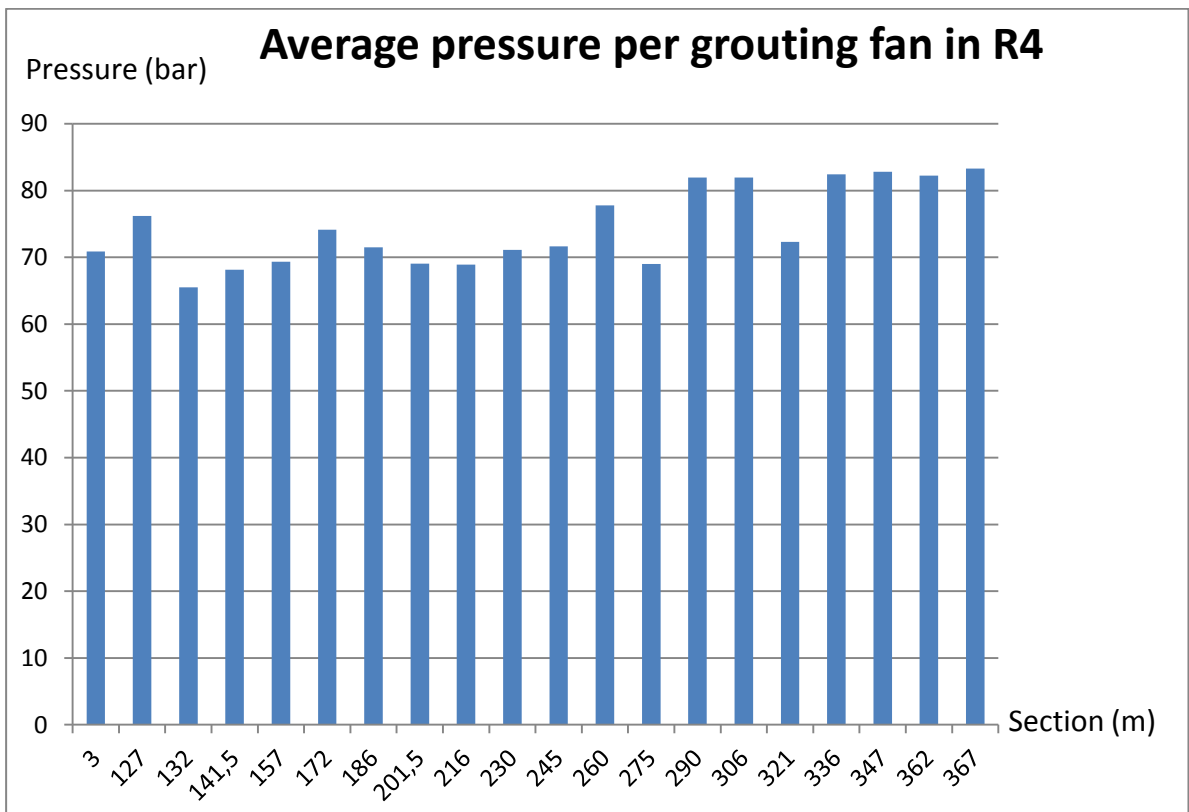


Figure 55. The average pressure per grouting fan in the R4 entrance tunnel.

In figures 56 and 57, the amounts of grouting mass and the pressure in every borehole per grouting fan in R4 are displayed.

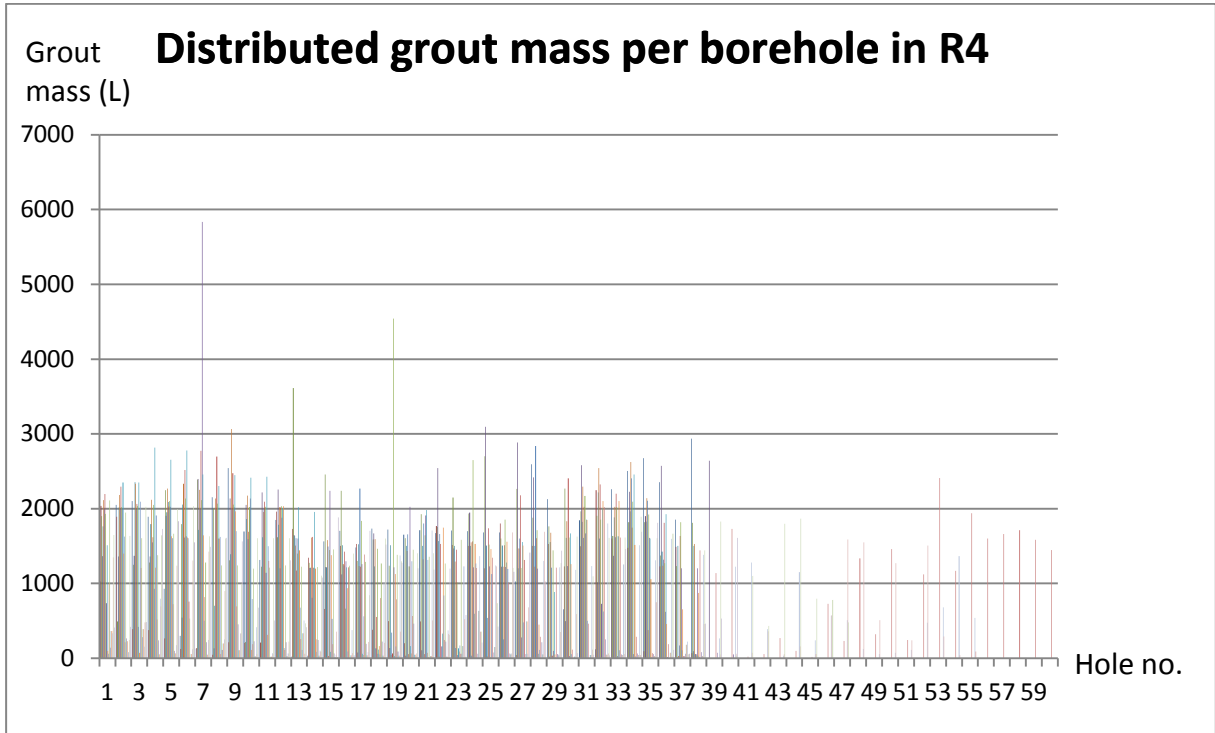


Figure 56. Grout mass per borehole per grouting fan in R4.

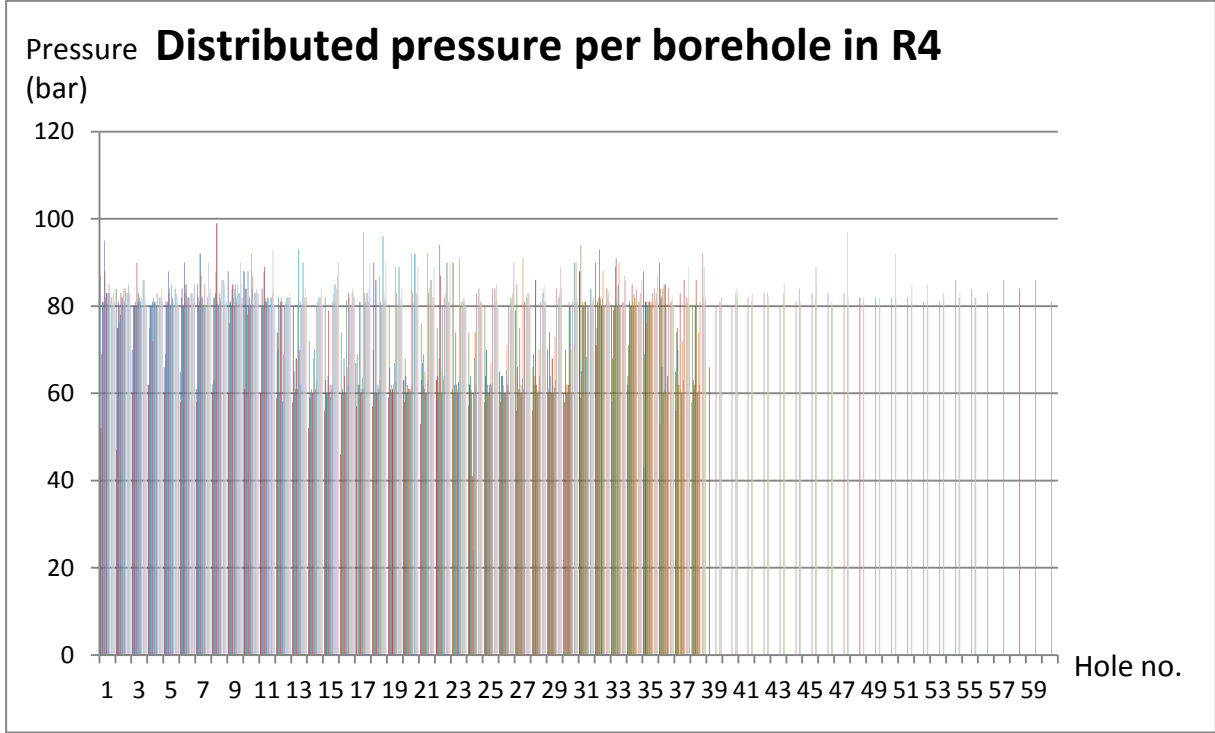


Figure 57. Grouting pressure per borehole per grouting fan in R4.

8.2.2 Geology during excavation of R4

Excavation works for the R4 entrance tunnel began in the autumn of 2010 and reached the final stages in February 2011. The entrance tunnel serves as both an access and escape tunnel to and from the main railway tunnel. The rock type is the B1 basalt that will house the station hall and the Snekkestad tunnel. The tunnel is approximately 360 m long.

At the initial excavation works certain brown-coloured layers of weathered rock mass were found also causing several local inflows. At approximately 100 – 200 m inside the R4 tunnel, water was abundant along with fractured rock mass. Mostly, this occurred on the right side of the tunnel face close to the tunnel wall. The fractured rock mass consisted of agglomerate and reddish silt- and sandstone. At around 230 m, water inflow occurred in the tunnel roof where also the rock mass was fractured. Further on into the tunnel at 250 m, several diabase intrusions were present bringing with them water which came out when drilling boreholes. At 300 m water became present in the tunnel floor. At the final stages of the entrance tunnel, at 350 m a loosely calculated water inflow of $20 \frac{L}{min}$ occurred in 4 boreholes. (Jernbaneverket 2012)

8.2.3 Grouting results from the main tunnel at Sjøskogen

The part of the main tunnel which is relevant for studying is the part which eventually will connect with the station hall tunnel. The tunnel has been nearly 500 m excavated at the time of writing. Due to a great abundance of data, it is not possible to display all of the grouting fans on one chart, and the total amount of injected grout mass and the average pressure per grouting fan will therefore be displayed on 4 charts. The main tunnel part studied goes from 82091 – 82631.

Figures 58 and 59 display the total amount of injected grout mass per fan.

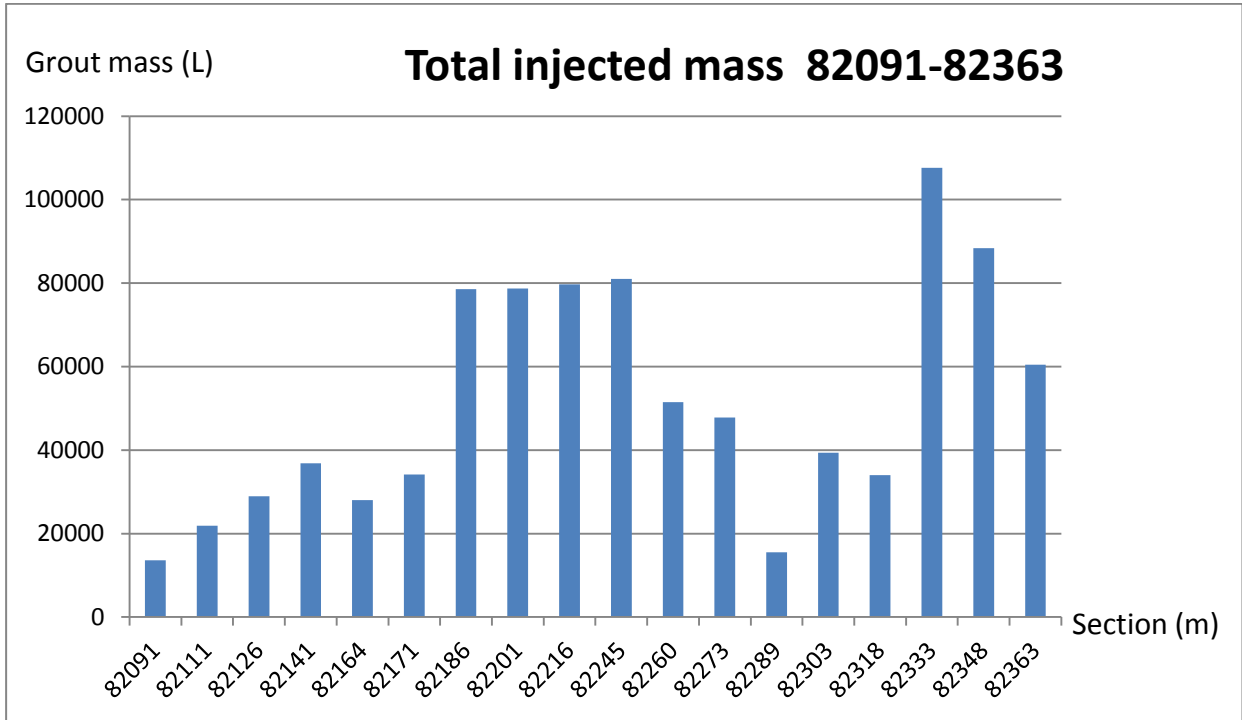


Figure 58. Total amount of injected grout mass per grouting fan in the main tunnel at Sjøskogen from section 82091 – 82363.

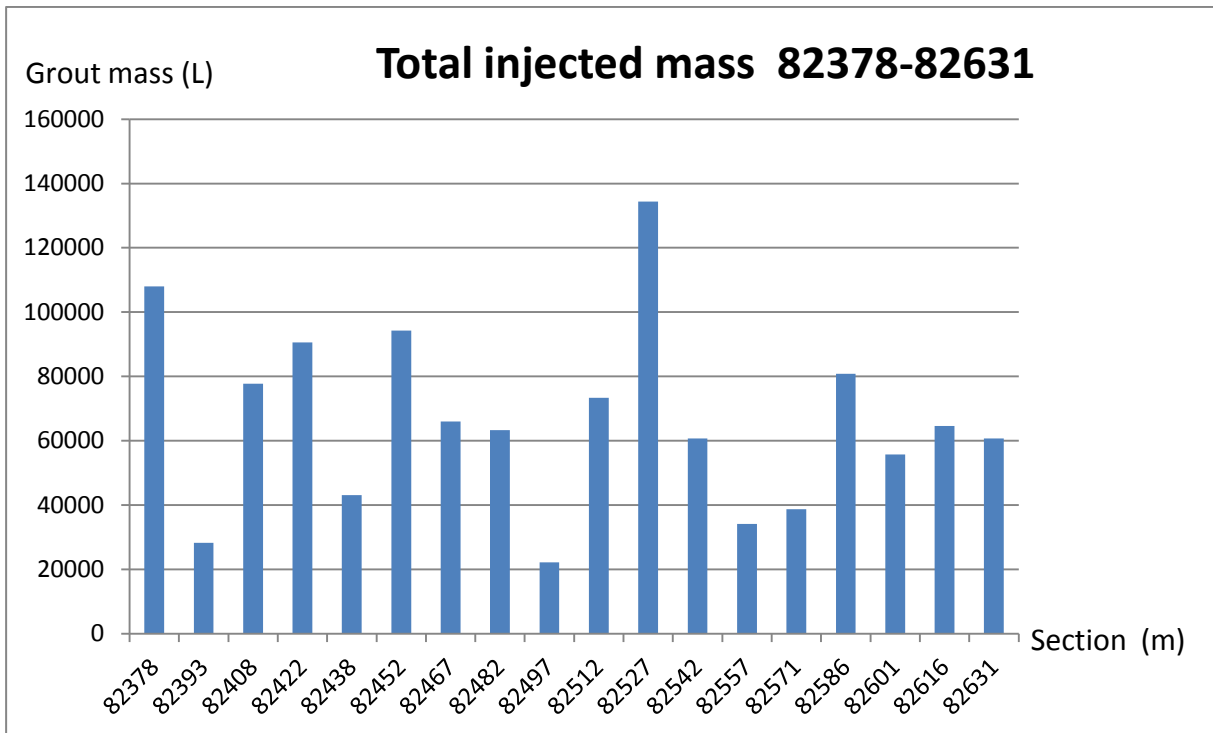


Figure 59. Total amount of injected grout mass per grouting fan in the main tunnel at Sjøskogen from section 82378 – 82631.

Figures 60 and 61 show the average pressure per grouting fan in the main tunnel.

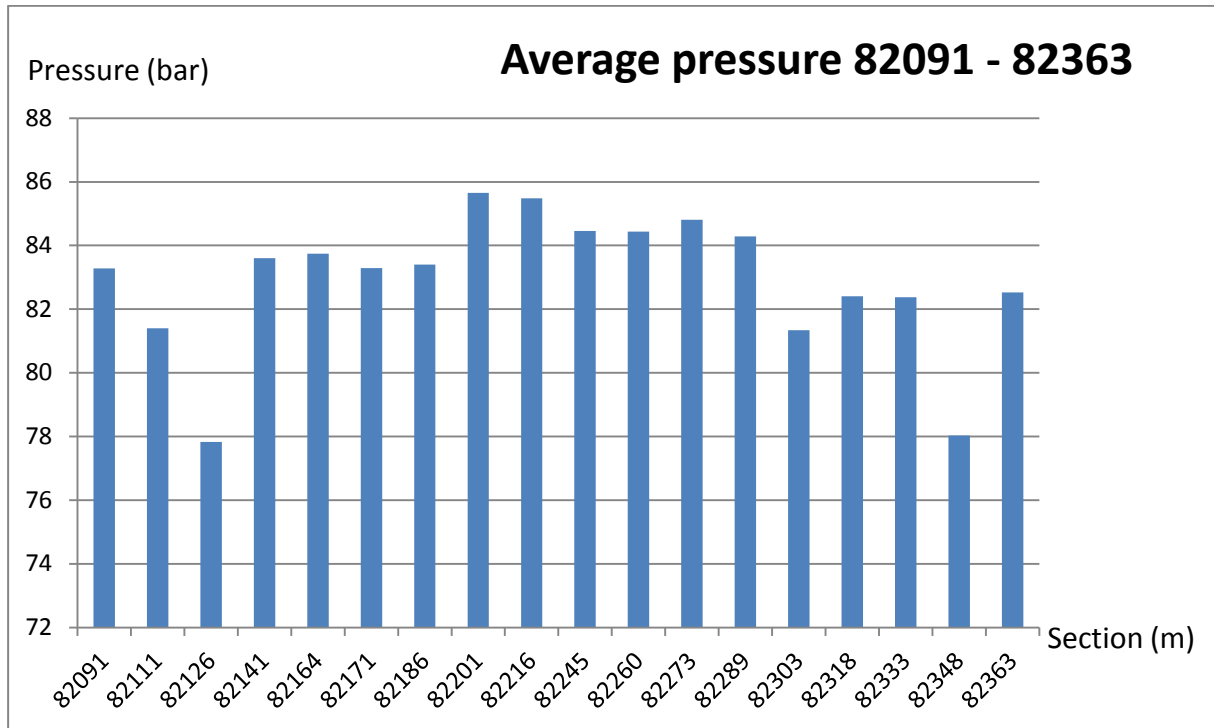


Figure 60. The average pressure per grouting fan in the main tunnel at Sjøskogen from section 82091 – 82363.

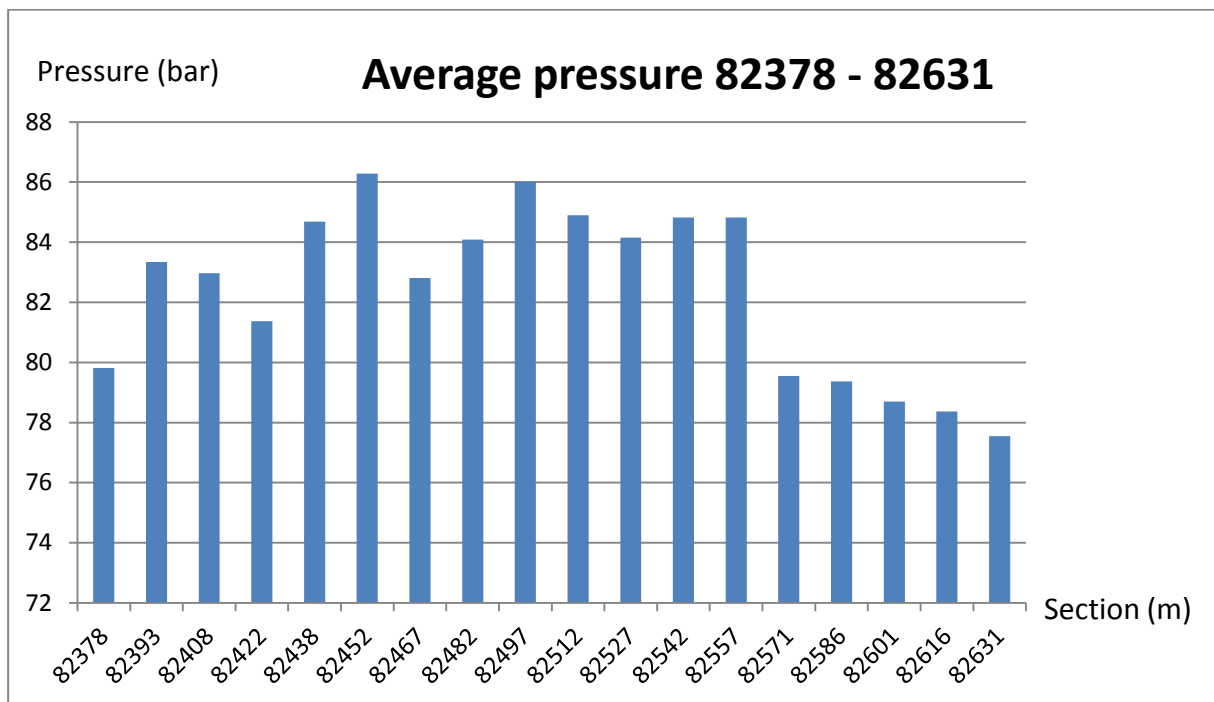


Figure 61. The average pressure per grouting fan in the main tunnel at Sjøskogen from section 82378 – 82631.

The injected grout mass and the grouting pressure per borehole per grouting fan throughout the main tunnel are displayed in figures 62 and 63.

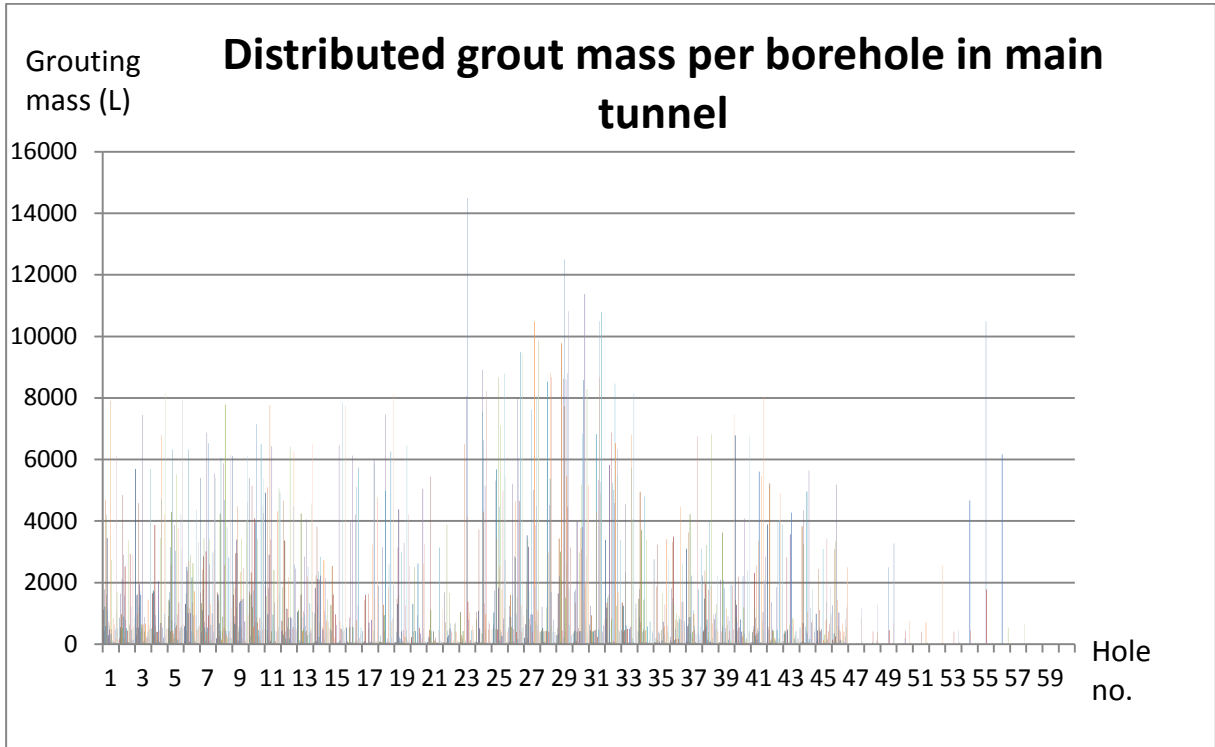


Figure 62. Injected grout mass per borehole per grouting fan in the main tunnel at Sjøskogen.

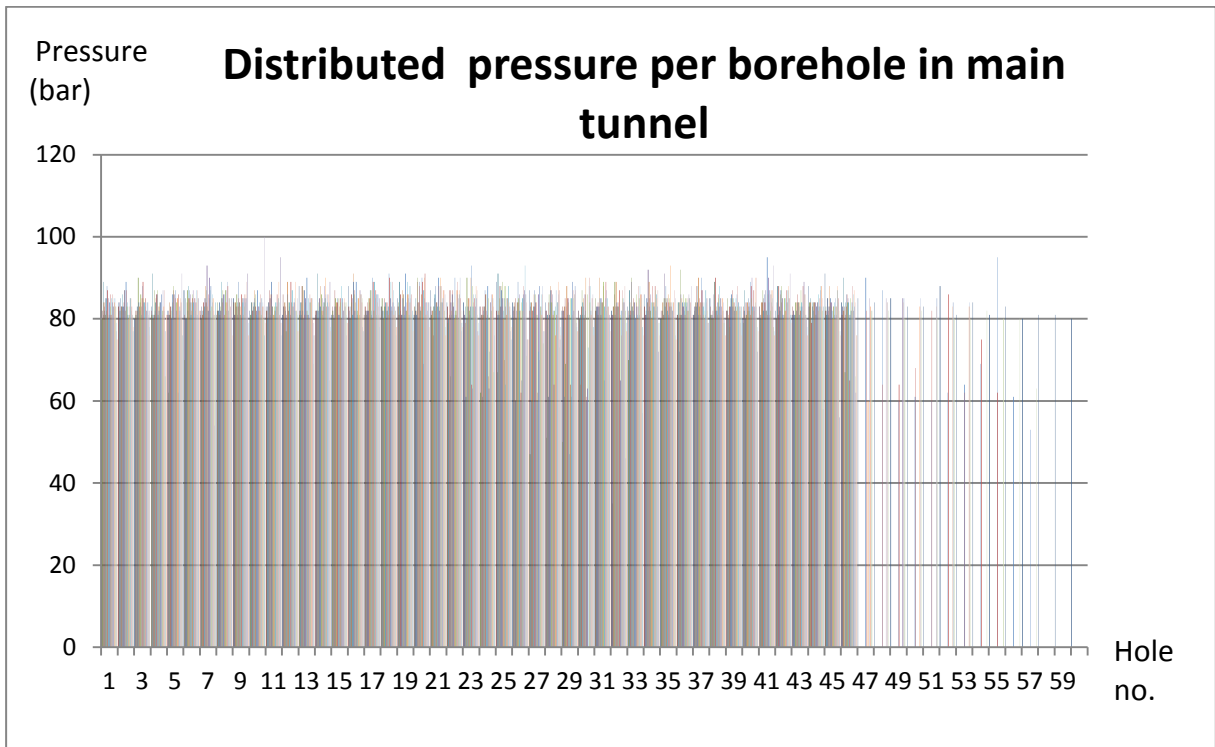


Figure 63. Grouting pressure per borehole per grouting fan in the main tunnel at Sjøskogen.

8.2.4 Geology during excavation of main tunnel at Sjøskogen

The data collected and referred to ranges from the period of February 2011 – March 2012. During this period approximately 600 m has been excavated.

Initially, several cracks in the area between the tunnel ceiling and the tunnel walls were discovered along with some fractured rock mass. At 100 m a lava top in the upper right abutment became visible and remained present as the tunnel works moved on. The lava top consisted of fractured rock of mediocre quality and came with some inflows. At 181 m the lava top suddenly appeared on the upper left abutment. The lava top then disappeared. After 200 m of tunneling a clay zone containing fractured poor quality rock caused the tunnel face to be coarsely shaped. This tendency continued for ca. 20 m with significantly fractured rock mass.

A section of good rock then followed. Still, occasional inflows occurred due to several diabase intrusions emerging. At times coarse blocks appeared in the tunnel ceiling of the tunnel face. A sinistral porphyry intrusion appeared after 350 m which then turned dextral at 400 m. The intrusion, naturally contained fractured rock and several inflows occurred. A new section of fair quality followed until after 500 m a fractured zone appeared at the tunnel floor. From 556 m and onwards, an increasing number of vertical and sub-vertical joint sets could be spotted. (Jernbaneverket 2012)

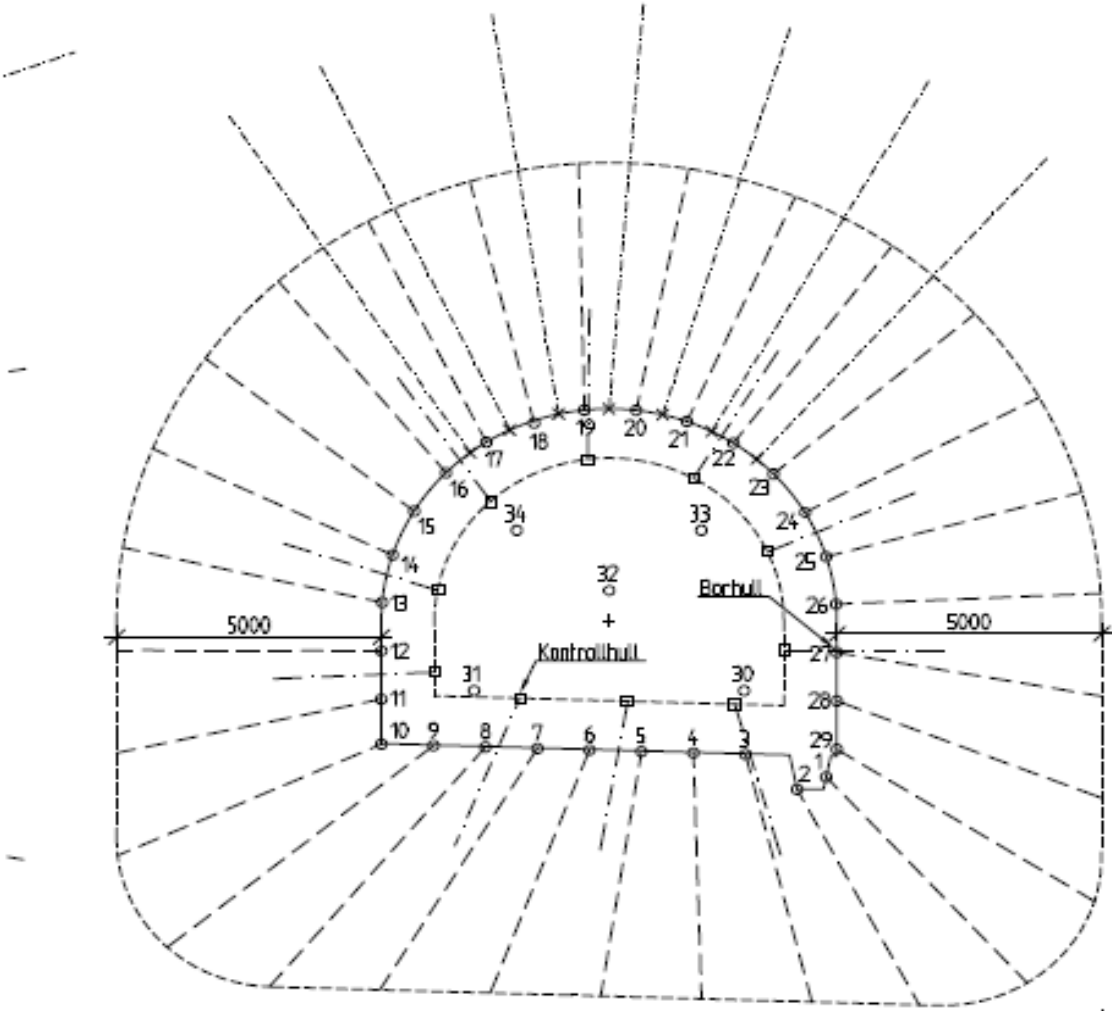
As previously mentioned, the rock mass of the Holmestrand plateau consists of column basalt with 3 vertical joint sets. Two of these one vertical and one sub-vertical have already been observed in the main tunnel part by the author, more information can be found in (Ryningen 2011).

To the author's knowledge, large open joint spaces were found at the tunnel face subsequent to blasts (and therefore also grouting) at times containing abundance of water inflows. Unfortunately this oral information obtained could not point out the exact section number but the author suggests that the section numbers with the highest injected grout mass values probably indicate such large joint spaces that required enormous amounts of grout.

8.3 Snekkestad

For the Snekkestad project as with the Sjøskogen project only the tunnel towards Fibo and thus Holmestrand will be discussed, along with the escape tunnel which now functions as an entrance tunnel to the main tunnel. The escape tunnel is named R13. Figures 64 and 65 show the sketches for the pre-grouting fans of the R13 escape tunnel and the main tunnel.

In the main tunnel at the time of writing, only one single grouting fan has been carried out and due to other priorities in the tunnel works no further grouting fans will be performed in the near future. The author is thus forced to analyse the grouting situation based on this fan alone.



Snitt

Figure 64. Sketch of the pre-grouting fan for the R13 escape tunnel. (Jernbaneverket 2011)

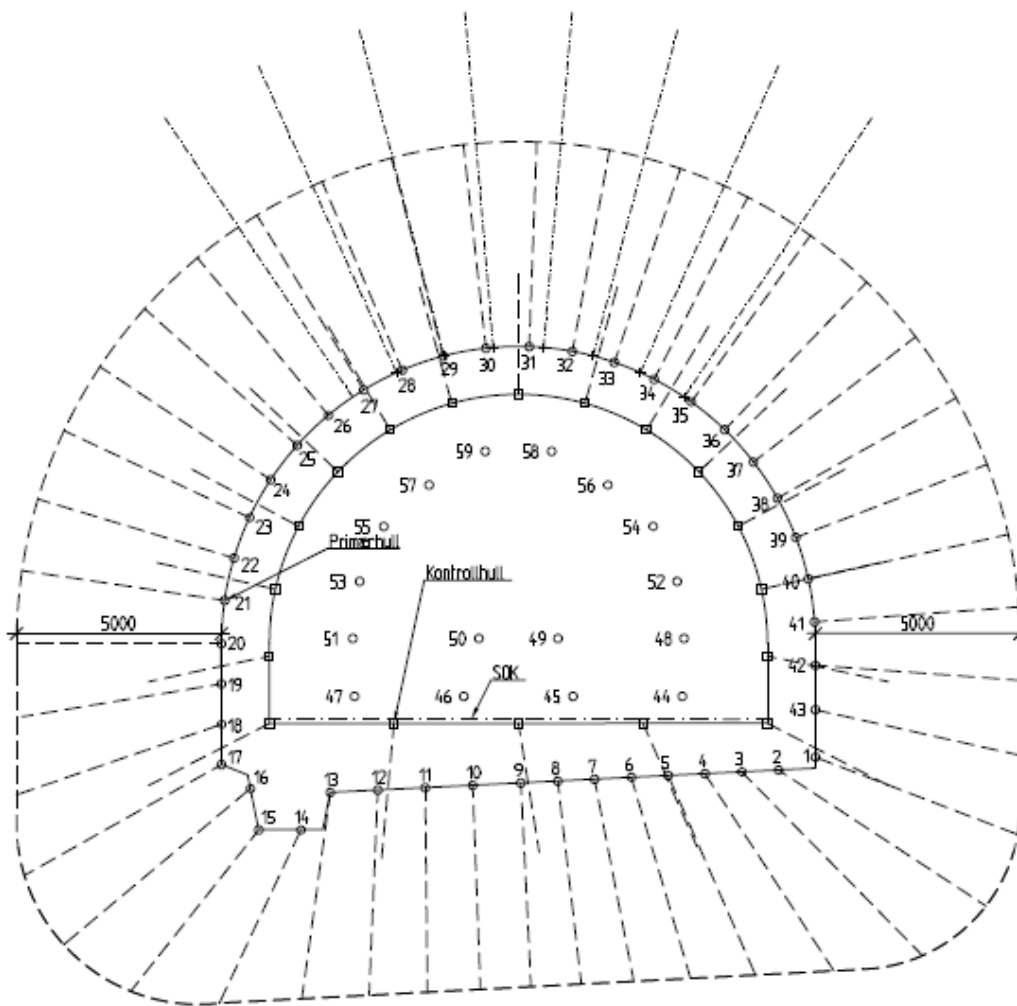


Figure 65. Sketch of the pre-grouting fan for the main tunnel at Snekkestad. (Jernbaneverket 2011)

8.3.1 Grouting results from R13

Figures 66 and 67 display the total amount of grout mass and the average grouting pressure per grouting fan for the R13 escape tunnel.

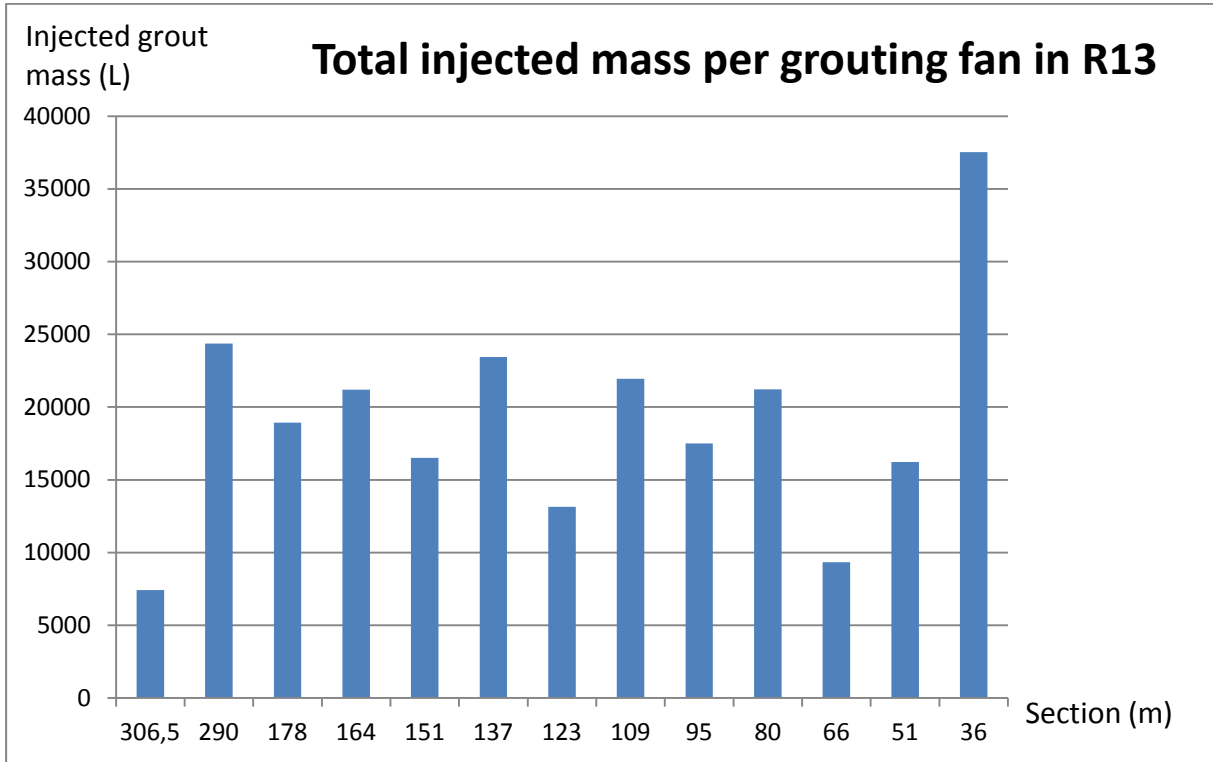


Figure 66. Total amount of grout mass per grouting fan in R13.

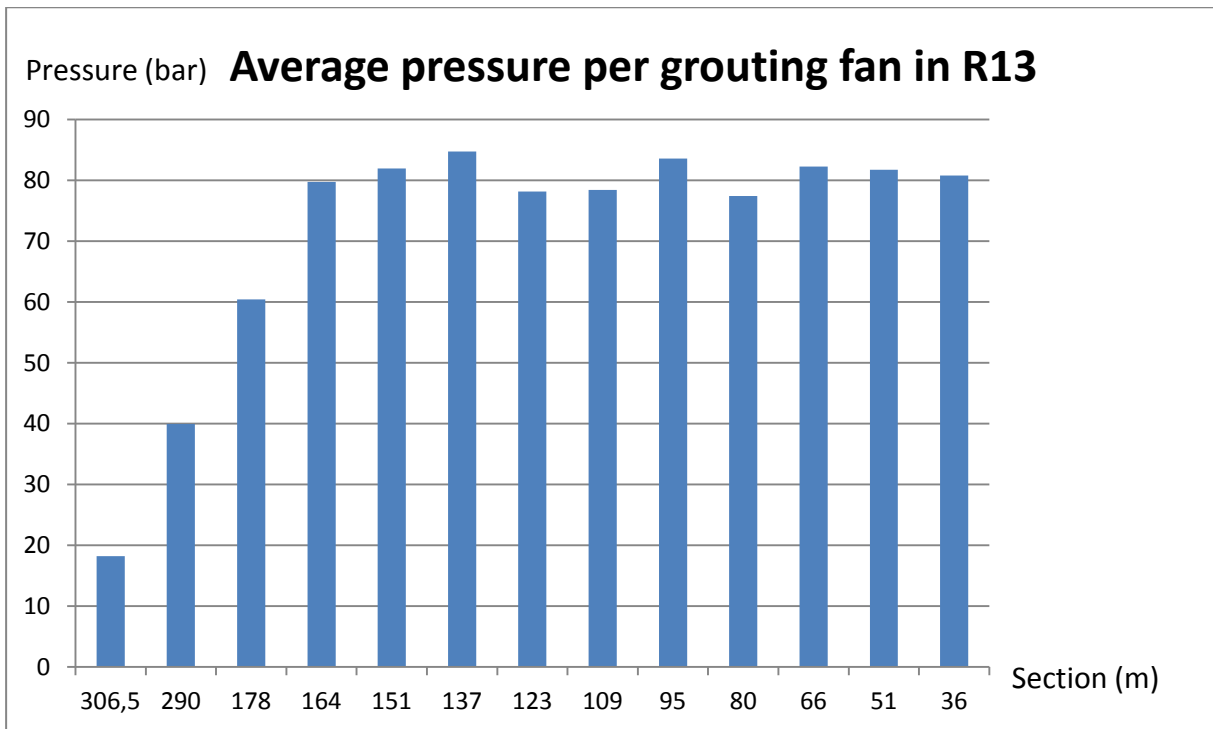


Figure 67. Average grouting pressure per grouting fan in R13.

Figures 68 and 69 display the amount of injected grout mass and the pressure per borehole in the R13 tunnel.

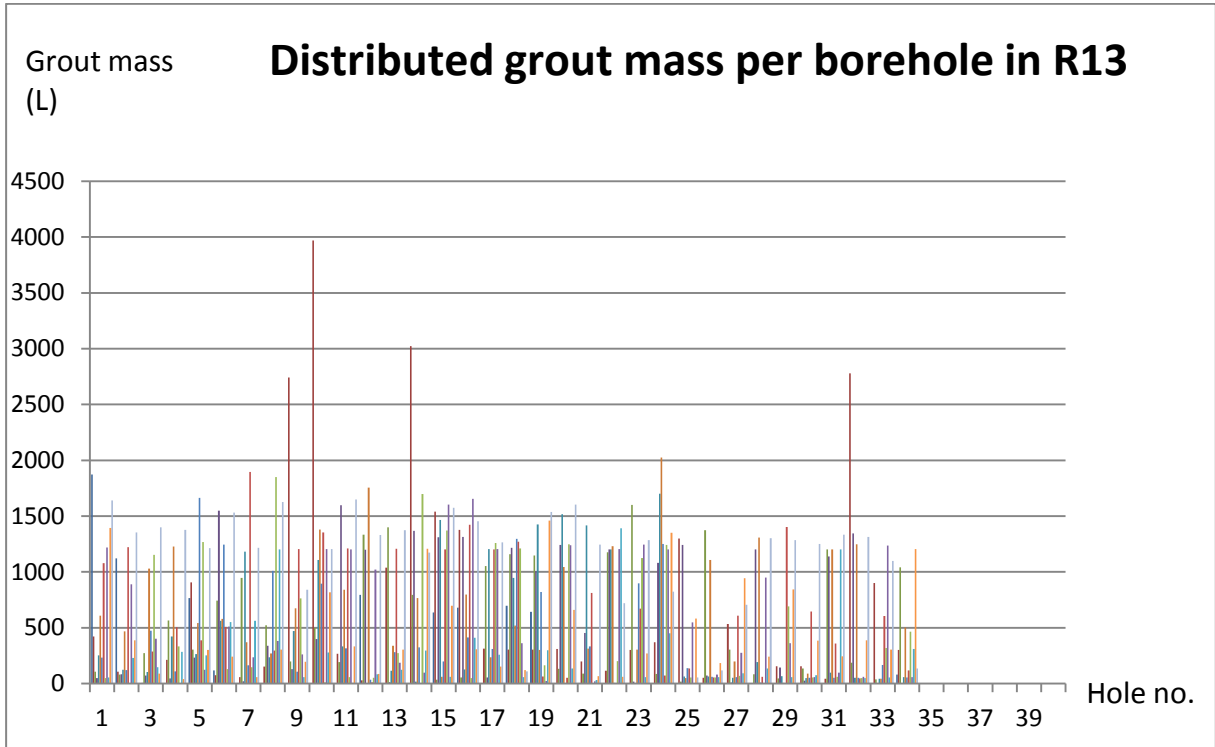


Figure 68. Distributed grout mass per borehole per grouting fan in R13.

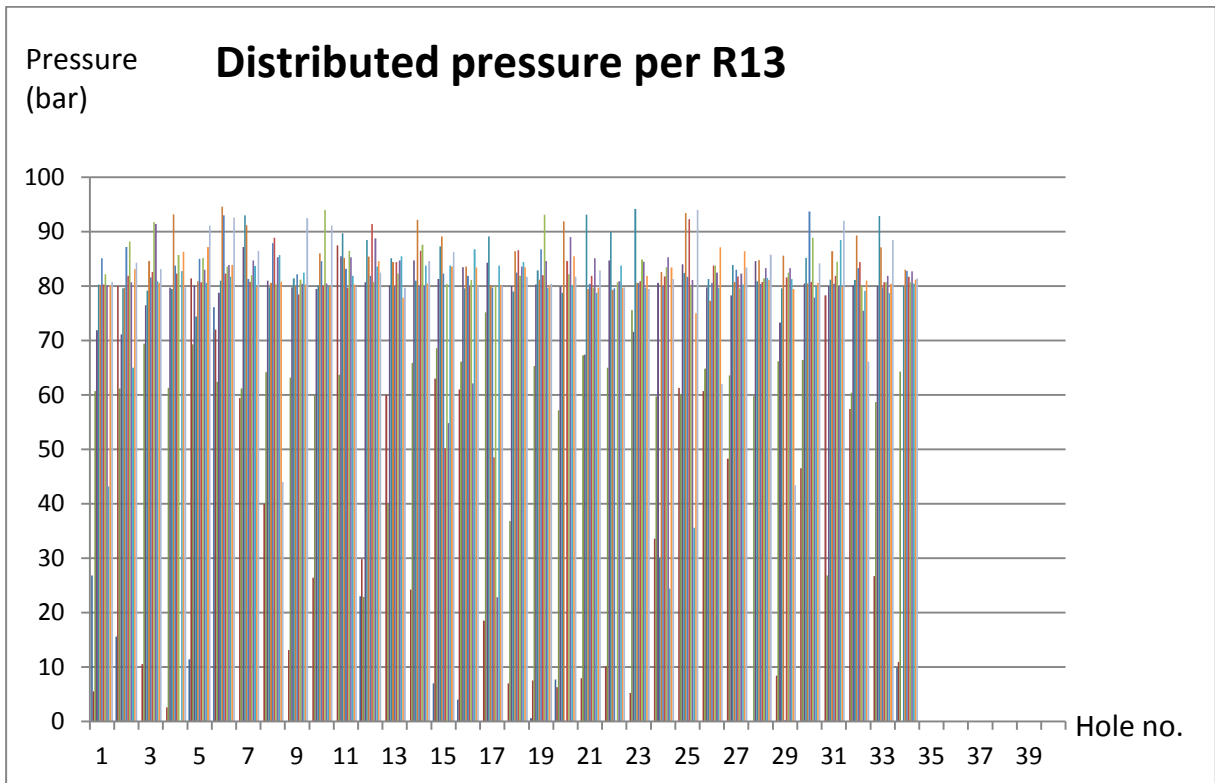


Figure 69. Distributed grouting pressure per borehole per grouting fan in R13.

8.3.2 Geology during excavation of R13

Initially various horizontal joint sets are found yielding Q-values of 0,42. After 70 – 80 m, various blocks can be seen in the tunnel ceiling but otherwise good rock quality. After 85 m certain tendencies of an intrusion were spotted. The rock mass at this point had 3 distinctive joint sets. At 90 m, this intrusion now appeared as a clay zone contacting poor rock quality forcing the need for fibre reinforced shotcrete arches. The weakness zone, consisting of clay continues at 120 m then disappears. At 100 – 120 m, water is starting to emerge from boreholes, which probably are probe drilling holes. From 180 – 250 m, the rock quality increases with Q-values edging 30. The final part of the R13 tunnel shows good quality of the rock as well. (Jernbaneverket 2012)

8.3.3 Grouting results from the main tunnel

Figure 70 and 71 show the total amount of grouting mass and the average pressure for the single pre-grouting fan performed in the main tunnel.

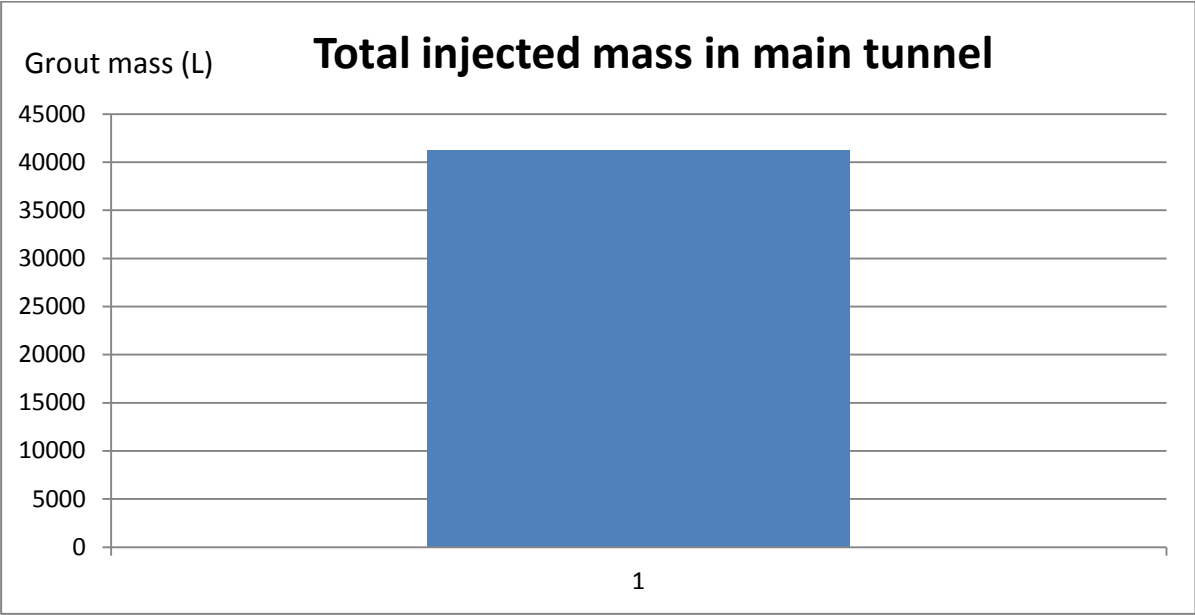


Figure 70. Total amount of grout mass for the single grouting fan in the main tunnel at Snekestad.

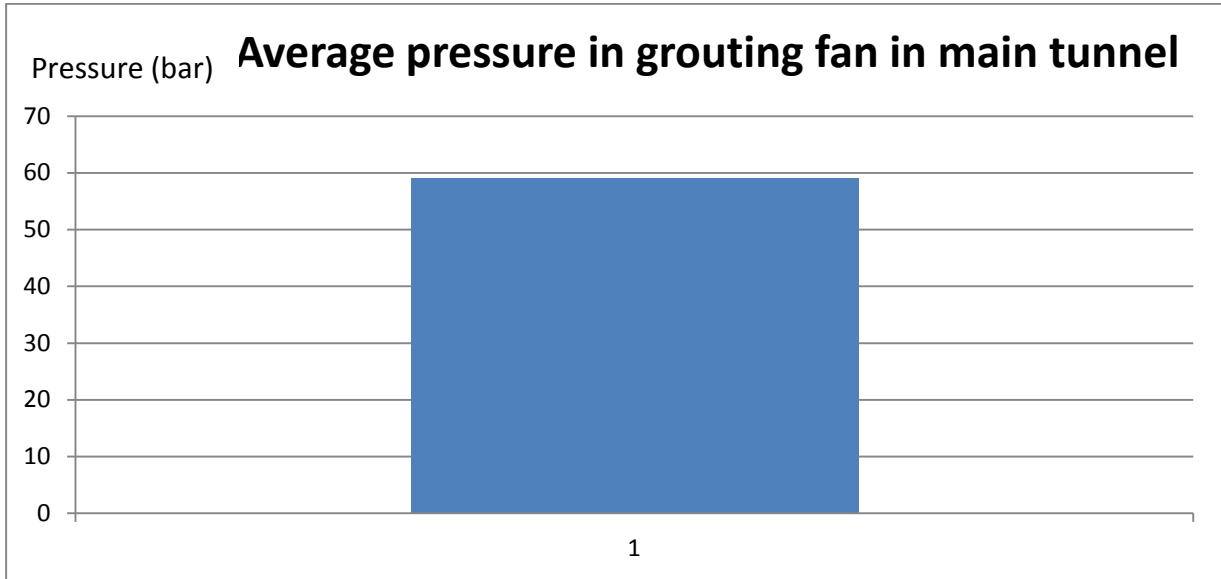


Figure 71. Average pressure for the grouting fan in the main tunnel at Snekkestad.

Figures 72 and 73 display the amount of grout mass and grout pressure distributed for the grouting fan in the main tunnel. Note that the number of boreholes differ from that on the sketch of the grouting fan on figure 65 due to additional control holes being drilled for this fan. Holes 59 – 66 constitute the extra control holes.

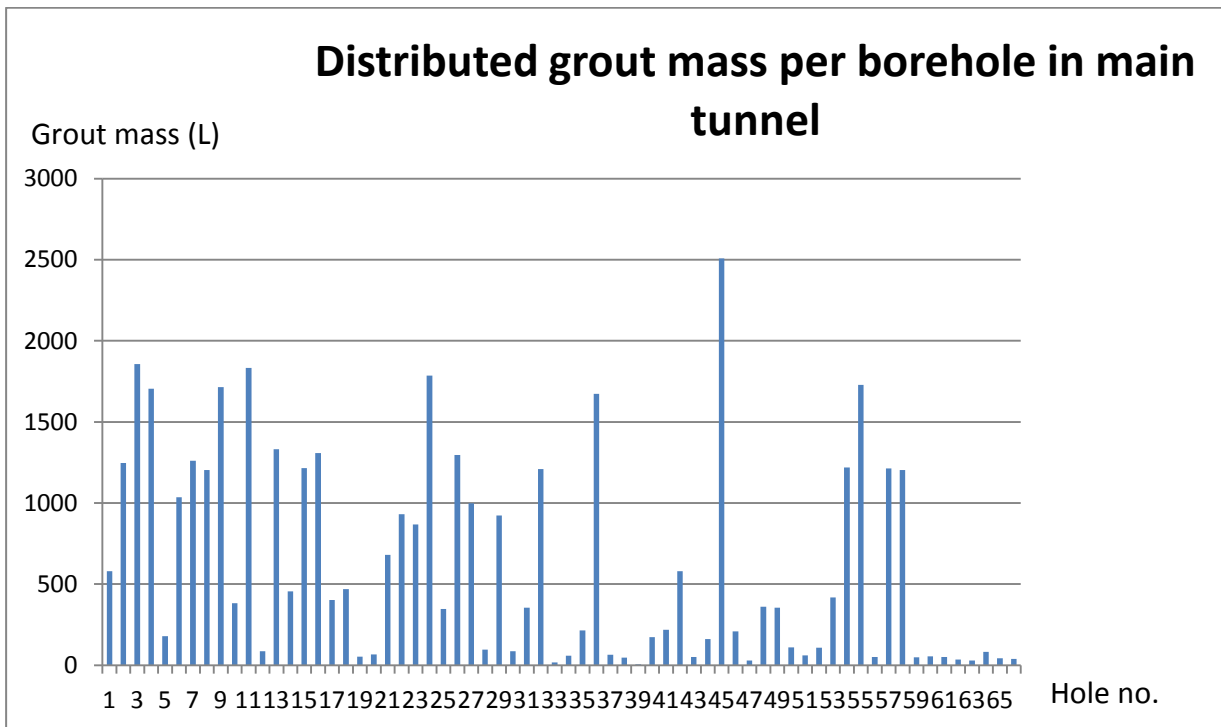


Figure 72. Distributed amounts of grout mass per borehole for the grouting fan in the main tunnel at Snekkestad.

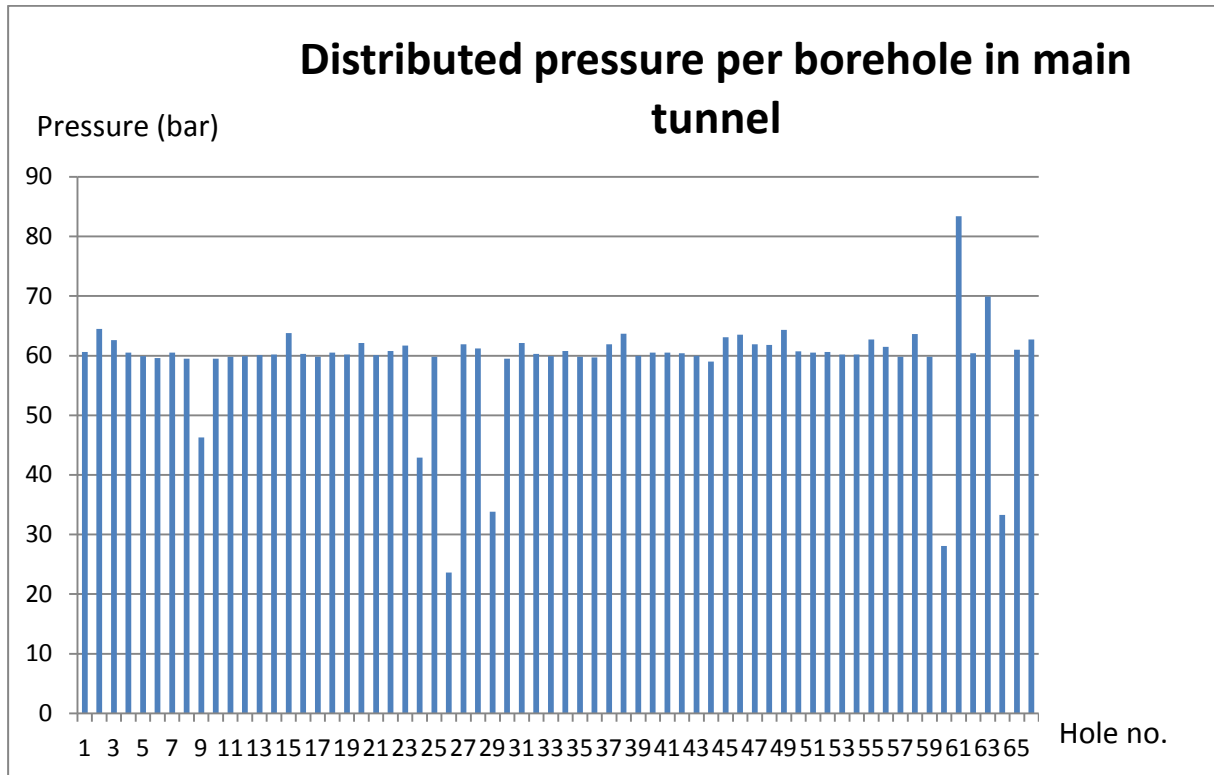


Figure 73. Distributed grouting pressure per borehole for the grouting fan in the main tunnel at Snekkestad.

8.3.4 Geology during excavation of main tunnel

Following the excavation works of the R13 escape tunnel the main tunnel also has decent quality of its rock mass. However, the Q-values decrease and at the wall of the main tunnel a Q-value of 7 is obtained. Certain foliation features are also visible in the rock mass. At the time of writing a lesser number of blasts have been performed yielding Q-values more than 10. No water leakages or weakness zones have been spotted. (Jernbaneverket 2012)

8.4 Excursion to the Gevingås tunnel

The author has conducted an excursion into the Gevingås railway tunnel located north of Trondheim. This tunnel utilises both the traditional PE foam plates and the sprayed membrane solution, namely the BASF Masterseal 345 composition as means of water and frost protection. Since this is one of the first times the sprayed membrane solution has been used in a Norwegian tunnel, a somewhat careful approach has been taken to ensure that the areas being subjected to the least amount of freezing were equipped with the sprayed membrane solution. During the excursion several spots were noticed that contained seepage. Although post-injection in the rock mass had sealed some of the seepage points, there was still some humidity on the surface of the shotcrete layer closest to the open tunnel space. Figure 74 shows one point of seepage in the tunnel section where the sprayed membrane solution had been applied.

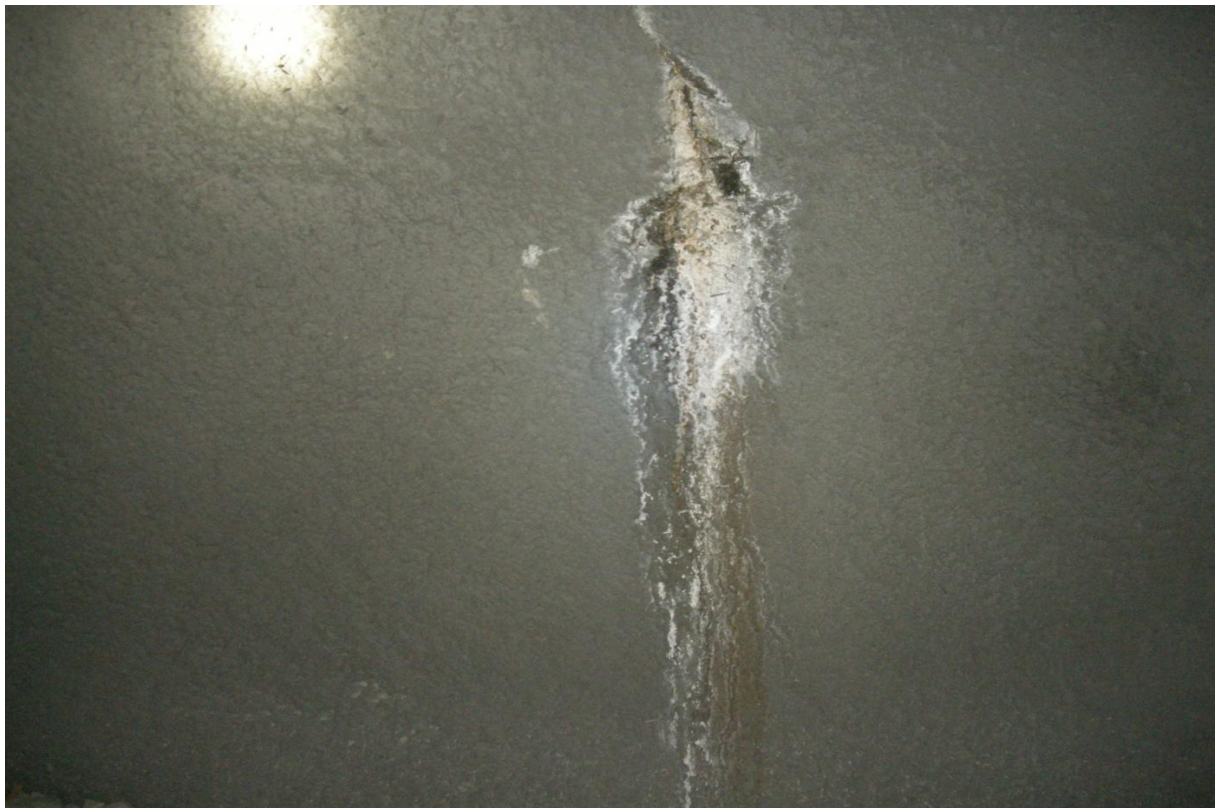


Figure 74. Photograph of a seepage point on the surface of the outer layer of the shotcrete in the Gevingåsen tunnel, at 26.01.12.

Also, several temperature measuring profiles had been installed in the composite layer. In total, there were three holes at 20 mm, 60 mm and 130 mm. The latter hole thus reached the rock mass of the tunnel. Two such profiles were installed at one point; one at the base of the tunnel floor and one close to the upper abutment near the tunnel ceiling. The temperatures of these holes can be seen in table 13.

Table 12. Temperature measurements in the frost measuring profiles in the Gevingås tunnel, at 26.01.12

Lower profile		Upper profile	
Hole depth (mm)	Temperature (°C)	Hole depth (mm)	Temperature (°C)
0 (inside tunnel)	0,8	0 (inside tunnel)	1,9
60	1,9	60	1,9
230	3,3	230	2,1
430	5,3	430	4,1

Studying the table, the tendency seems to be that the temperature increases as the profile penetrates deeper and deeper into the rock mass, reaching a fairly stable temperature in the rock mass. This corresponds well with the tests performed in the SINTEF frost laboratory tests where the graphs representing the temperature in the rock mass show the trend of converging after a while.

Naturally, the most important result during this observation is that all temperatures inside the rock mass are above zero, prohibiting the water from freezing. This shows that in the Gevingås tunnel, the heat capacity of the rock mass constitutes the frost protection method, causing the sprayed membrane to only take care of the water protection. Regrettably, only the water sealing properties of the sprayed membrane solution have been shown in this tunnel and not the frost protection features.

In general, the results from the Gevingås tunnel show that single point leakages mainly caused by rock bolts compose the most significant inflows in a tunnel utilising the sprayed membrane. Results from other tunnels also show this tendency.

The difference in the temperatures at the outer layers in the two profiles is probably because of air with a higher temperature will travel upwards. It would be assumed however, that the temperature in at the innermost point of both profiles had a similar value. Also, the difference in the noted values is probably due to several sources of error like immediate heat affection of the temperature apparatus when being touched, etc.

9 Evaluation and discussion

9.1 The Station hall entrance tunnels

9.1.1 R7

When studying figure 44 there is over all similar amounts of grout mass injected save for the very first grouting fan. When taking into consideration that the R7 tunnel was excavated some 30 m prior to the first grouting fan, there is no reason to believe that the higher amounts of grout mass found at section 74 had anything to do with overburden changes or surface weathering of the rock. It is probably likely that local factors constitute this difference such as joint sets, joint spacing, joint frequency, etc. Also, there seems to be a trend in which the amount of injected grout mass drops as R7 approaches the station hall. This might be due to geological reasons such as more compactness in the rock further inside the Holmestrand plateau.

Figure 45 displaying the average grouting pressure per grouting fan keeps a somewhat steady development, except at section 46,6 where the average drops slightly. It then increases as the tunnel approaches the station hall. Since there is no solid geological description of the excavation it can be assumed that local geological features once again are responsible for this drop. However such a drop would likely indicate that the rock mass quality goes down since the compactness cannot handle such a pressure level as for the previous and subsequent grouting fans. Greater abundance of joints, joint frequency and less joint spacing would be a logical assumption. Also, the RQD value would probably be slightly lower.

Before studying the distribution of grout mass and pressure per borehole it is important to notice that sections 18 and 4,4 contain additional boreholes related to the approach of the station hall, thus the need for extra amounts of grout. Since participating in too few grouting fans, these extra holes will not be subject for discussion.

Distributed injected mass and grouting pressure per borehole in figures 46 and 47 reveal a partial pattern. On figure 46, it can be argued that holes 1 – 2 and 25 – 33 have more grouting mass injected when considering the thickness of the columns on the chart. However, it is dominated by a random distribution and thus a conclusion on this can hardly be made. Oppositely, it hazily appears that holes 3 – 19 contain the least amounts of grout mass. When consulting figure 43 it becomes somewhat evident that the left side of the tunnel floor demands less amounts of grout mass. This would likely indicate solid rock mass, few joint sets, and a low joint frequency.

Figure 47 shows vaguely that holes 11, 19 – 23 and 33 – 35 demand lower pressure values than the rest of the holes. Whilst it can be argued that this is due to a rock mass less solid than the rest the author firmly believes that this is a pattern of randomness and that a wider collection of data must be present before jumping to any conclusions. Reversely it seems easier to assert that holes 1 – 9, 25 – 33 and 35 – 39 have the highest pressure values. Thus, it is also more safe to argue that the rock mass may be slightly more compact in these holes. Figure 43 shows that holes 1 – 9 constitute most of the tunnel floor, holes 25 – 33 compose the right tunnel wall and the tunnel face and holes 35 – 39 are also found in the tunnel face. Should these tendencies continue it could be argued that the tunnel ceiling and the left tunnel wall contain weaker rock than the rest of the tunnel.

9.1.2 R9

Firstly, when studying figure 48 it appears as if the amounts of injected grout mass increase as the tunnel progresses then drop before increasing again. Due to the lack of width in the data collection it would be much too early to speak of tendencies in the development of the amounts required to seal every grouting fan. However, it would seem as if this sudden decrease indicates the presence of a zone containing good quality rock with few joint sets and great joint spacing. Overall though it is most likely another randomised distribution.

Taking the grouting pressure into consideration at figure 49 reveals a varying pattern of the grouting pressure with mostly random average values save for the very last fan where suddenly the average value increases greatly. There is not enough data to discuss or even consider tendencies in the variation, but such an increase would likely mean a zone with rock of decent quality with few joint sets. Comparing figures 48 and 49 against each other reveals a somewhat interesting section, namely 17,2, with low values of grouting pressure and low amounts of grout mass injected. It may seem as if the rock mass is a bit weary and cannot handle great pressures before bursting, but still requiring low amounts before reaching the stoppage pressure and preventing water from flowing in. Various geological differences along imbrications of previous grouting fans might also explain such low values.

When studying figure 50, holes 16 – 31 predominantly contain more grout mass per grouting fan than the rest, when taking the thickness of the columns into consideration. Holes 6, 12, 33 – 34 seem to contain the least amount of injected mass, however it is very difficult to make an assumption as the amounts mostly seem to alter between fans. Consulting figure 43 for aid at finding the location of these holes the tunnel ceiling, the dextral tunnel wall and the tunnel face seem to demand more grout mass to achieve stoppage pressure. Thus, it could be vaguely assumed that there are more joint systems and joint frequency present in this part of the tunnel. The areas requiring less amounts of grout mass seem to be virtually scattered around the tunnel and constitute a pattern of randomness leaving no concrete proof that one specific section of the tunnel profile demands less grout mass due to its compactness and fortitude.

Figure 51 reveals few or no patterns of any holes with significantly higher or lower grouting pressure values. Save for the grouting fan at section 17 which has additional boreholes, there

is not any specific pattern emerging from this chart. The pressure distribution is being kept at a constant level at 60 bar. The two single boreholes which deviate from the others are boreholes 6 at section 27 and 29 at section 2, with a value of approximately 75 and 86 bar, respectively. Numerously, there is no any other hole in the vicinity of these, making them exceptions. The high values are probably related to abundance of joints and joint frequencies.

9.2 The Sjøskogen tunnels

9.2.1 R4

For the R4 entrance tunnel, and the amounts of injected grout mass, most of the tunnel from the very beginning to section 230 with some minor exceptions has fairly high values of injected grout mass as seen in figure 54. The amounts then drop from section 245 – 306 before increasing decreasing and increasing again at the end of the tunnel at section 367. The pressure distribution remains fairly constant, with low variations, and a tendency of increase as the tunnel progresses.

In comparison with the geology, from 100 – 200 m there is water present in great quantities. This is one of the areas on figure 54 where the amounts of injected grout mass are greatest and is probably related to the need to prevent the water inflow. On figure 55 the grouting pressure is fairly low making sense when noting that the rock mass is fractured in this part. Injecting cement at a high pressure would likely fracture the rock mass even more. The diabase intrusions which appear at 250 m, does not affect the general picture of the rock mass when regarding the amounts of grout mass injected, since the amounts decrease from 250 – 300 m meaning that the rock mass at those sections does not require lots of grout mass to stabilise itself and keep water out of the tunnel. With some exceptions, the grouting pressure increases from section 250 – 300. The local bottoms are probably due to geological differences in the rock mass.

Water is again appearing at section 300, where also the grouting mass amounts increase. This is also probably related to poor rock quality. The rock quality then increases a little before worsening as water appears in several boreholes, demanding more grout injected into the rock mass. At the same time the grouting pressure increases slightly indicating that the strength of the rock also increases. Sections 275 and 321 are the only sections with low grouting pressure values which are probably caused by local geological features, such as fractured rock, joint systems, etc.

The average pressure per grouting fan on figure 55 finds itself steady at around 70 bar during most of the tunnel excavation, however it increases fairly when entering the final 7 fans save for one fan. The pressure seems to increase to 80 bar. This is probably due to the increased overburden as the R4 tunnel inclines downwards.

When studying the distribution of the grout mass per borehole in the grouting fans in figure 56, there can be spotted a trend in which holes 3 – 9 and partially holes 25 – 34 receive more amounts of grout mass than the rest of the holes. There still are variations between each grouting fan and so this pattern is not very distinct. Holes 13 – 20 seem to require less amounts of grout mass. Holes 39 – 60 are not always injected and fall outside the main trend of the chart. These added holes are located at the tunnel wall and thus are not displayed on the pre-grouting fan sketch at figure 52. When looking at the pre-grouting fan holes 13 – 20 constitute the left part of the tunnel ceiling which seems to be the more compact part of the tunnel. This is partially consistent with the geology during the excavation as it is mostly the right tunnel wall and occasionally the tunnel floor and the tunnel ceiling that give the greatest inflows. Holes 3 – 9 constitute the tunnel floor save the drainage ditch, while holes 25 – 34 are found on the right tunnel wall and at the tunnel face. There were some inflows in the tunnel floor at section 300, which seems reliable concerning the situation. In the rest of the tunnel, especially in the first part, seepages and weathered rock quality constituted the right tunnel wall which is probably why this part of the tunnel profile required more amounts of grouting mass. Since holes 30 – 34 are found on the tunnel face itself it can be argued that these holes communicate with the holes on the right tunnel wall.

For the grouting pressure distribution at figure 57 it seems like holes 13 – 25, save holes 17 – 19 and 22 – 23 require lower pressure values than the rest of the holes. Grouting pressure value in this part of the tunnel face remains at approximately 60 bar while the rest of the holes in the tunnel profile have a more or less constant value of 80 bar. Holes 13 – 25 are found mainly in the abutments on both sides and the tunnel ceiling. Save the exceptions, it may be that the rock mass in the ceiling is a little bit less compact than the rest of the rock mass. This could be due to rock stresses. It could also mean that there are just random geological features such as less joint sets and less joint frequency, making the grout mass hardening at a lower pressure. Since there are no stress measurements present at hand, there can be many theories and it is difficult to provide a solid answer.

At the final stages of the R4 tunnel, water became abundant in the tunnel floor. Holes 1 – 10 cover most of the tunnel floor as seen on figure 52. There are no visible signs of this in figure 57 other than the variation between fans. When studying the figure with respect to the location of the picked fans, no signs of decrease of the pressure appear either. This would probably indicate that despite the water inflow the rock mass is still able to withstand pressure.

9.2.2 Main tunnel at Sjøskogen

From section 82091 – 82363 on figure 58 the lava top which appeared after 100 m approximately at section 82190 will probably require extra amounts of grout mass since the fractured rock mass containing water inflows demands sealing. As expected when studying the chart on figure 58 the amounts increase and last for 70 m, before decreasing again. This is most likely due to increasing rock quality and the disappearance of the lava top. At 240 m the

amounts dramatically increase without any explanation being offered in the description of the geology. It can be assumed that this is most likely for various geological reasons such as local diabase intrusions which go away after the subsequent blast.

On figure 60 the grouting pressure in this part of the tunnel varies from fan to fan, however not with any significant differences other than what would be expected. Still, a pattern can be seen with tendencies of higher pressure from 50 – 200 m on the very same figure. When taking into comparison that the various lava tops and local weakness zones appear quite occasionally in this area, the tunnel face is in general quite compact and a major part of the tunnel face consists of good quality rock. The rest of the tunnel face, consisting of various weakness zones, thus demands more amounts of grout.

From section 82378 – 82631 on figure 59 the situation changes. The porphyry intrusion which appeared after 350 m and lasted for approximately 50 m can be seen at section 82452 when the grout mass amount increases. The following tendencies of the grout mass indicate different geological conditions with no explicit signs of any weakness zone. The rock quality is fairly good until 500 m where a fractured zone occurred in the tunnel floor. On figure 59 this is barely visible at section 82586, where the injected grout mass amount increases slightly. As the tunnel progresses, the amounts decrease, indicating better rock quality.

The grouting pressure in this area displayed on figures 60 and 61, increases from 300 – 350 m, and then suddenly drops at 350 – 370 m. This may be due to the porphyry intrusion which demanded less grouting pressure. However, such a decrease indicates that all grouting fans are being executed with low pressure. The grouting pressure then increases from 400 – 490 m, which is from section 82497 – 82557. This is in an area with rock mass not specifically noted, thus good rock mass. When reaching 500 m the pressure goes down, most likely due to the fractured zone in the tunnel floor. The pressure is then kept at a stable level, probably due to certain weakness zones.

It is known that the entrepreneur asked specifically to reduce the grouting pressure in this part of the tunnel. It remains unknown whether the desire has been approved, however the latter tendencies in the grouting pressure on figure 61 would suggest so.

The distribution of the grout mass per borehole on figure 62 offers certain patterns. Holes 20 – 22 are notoriously quickly filled with the least amounts of grout mass. Also, holes 35 – 41, with slightly more local variations demand less amounts of grout mass. Holes 23 – 35 once again with varying values, seem to require overall more amounts of grout than the rest. Holes 1 – 20 vary but require intermediate amounts of grout mass when compared to the extremes. When taking a look at figure 53 the sketch of the pre-grouting fan indicates that the upper left abutment requires small amounts of grout, while the entire tunnel ceiling demands the most grout mass. The right hand tunnel wall and the tunnel face itself require less grout again. Finally the holes on the tunnel floor require intermediate amounts of grout. Based on this it would seem that most of the weakness zones appear in the tunnel ceiling and also, partially at

the tunnel floor. Geological reasons probably make the demand for grout mass less in holes 20 – 22 and 35 – 41.

On figure 63, the distribution of the grouting pressure per borehole, it can very loosely be argued that holes 27 – 29 require a slightly lower value of the grouting pressure than the rest. Other than this, there is really no explicit pattern in the pressure other than local variations, the pressure being stable at approximately 80 bar. There is not really any specific part of the tunnel profile, save holes 27 – 29 which are found in the tunnel ceiling whose injected grout mass hardens at a lower pressure. The author argues that only two holes with so weak tendencies are not enough to constitute a pattern of lower grouting pressure. It is a rather random trend.

In addition, the uncovering of the tendencies of column basalt and the wide joint space openings must be responsible for some of the amounts of grout mass injected.

9.3 The Snekestad tunnels

The R13 tunnel traverses below a road tunnel, namely the Brekke tunnel on the E18 expressway. Due to pipe systems and various other installations in the Brekke tunnel, there were no grouting performed when the R13 tunnel passed under the Brekke tunnel. This section lasts for about 80 m. This is why there are no grouting data in this range.

The section labeling system is reverse from that at R4. The R13 is 320 m. The tunnel starts at section 320 and ends at section 0. The first grouting fan was performed at section 306,5. The zone without grouting ranges from section 290 – 207. However there are no grouting reports present from section 207 – 178, any possible report appears to be lost. Thus, grouting data are at a halt from section 290 – 178.

9.3.1 R13

Starting off with figure 66, the amounts of injected grout mass are at a fairly low level at the first grouting fan before rapidly increasing as the tunnel is being farther excavated. The geological description shows a weakness zone emerging at 90 – 120 m, and at 100 – 120 m, water is starting to emerge from certain boreholes. On figure 66 this is not possible to notice, as the grouting free zone ranges from section 290 – 178. There are no distinct geological features other than that which are important, the rock quality is fairly good, and the variations in the amounts of injected mass are most likely caused by local geological differences and also technical incidents on the equipment.

The displayed average grouting pressure per grouting fan on figure 67 offers a significant increase as the tunnel progresses into the rock mass. A fairly low overburden at the beginning of the excavation which increases over time is probably the main reason for this. The pressure varies somewhat from section to section but overall keeps a stable average value of

approximately 80 bar. However at section 123, which is nearly 200 m into the tunnel the grouting pressure goes down and the amount of injected grout mass also decreases. This somewhat strange behaviour suggests a fairly compact mass which obviously does not need high pressure of the injection, probably due to geological features such as few fairly huge joint sets. At section 66 the grouting pressure is low while the holes of the grouting fan demand numerous amounts of grout mass, probably due to certain weak features in the rock. This is partially inconsistent with the geological description, and could also be because of a rock mass not demanding high values to reach the stoppage pressure.

The distributed amounts of grout mass per borehole on figure 68 show some partial patterns as holes 10 – 12 and 25 – 34 demand the least amounts of grout mass. Still, the variations between sections play an important role here. When comparing this to the pre-grouting fan on figure 64 it is the dextral part of the tunnel ceiling and the tunnel wall, along with the middle of the tunnel face that demand the least amounts of grout. In no way is this reflected in the geological description, leaving numerous reasons to reflect upon. A suggested theory would simply be that the rock mass in this part of the tunnel face is of better quality than the rest. The rest of the holes show tendencies of demanding more grout mass, but once again this is strictly sectional variations. It is difficult to find a distinct pattern in the distribution without a broader width of data.

On figure 69 where the distribution of pressure per borehole is displayed, there are virtually no holes which show a significant amount of less or more pressure when disregarding the variations between sections. It could however be argued that holes 12 and 18 – 24 somewhat show tendencies of lower pressure values, but this is also strictly at random. Reversely, holes 11, 19 and 31 seem to have the highest pressure values. When taking this into consideration and comparing with figure 64 it would seem that the tunnel ceiling at times demands lower pressure than the rest of the tunnel. The holes with the highest overall pressure values are more scattered, though. The author argues that despite all this, there is really no distinctive pattern here.

9.3.2 Main tunnel at Snekkestad

There are in general few details about the general geological situation of the main tunnel in the direction towards Holmestrand, since due to reasons of priorities, there has only been but one single grouting fan performed. Still, it is possible to somewhat analyse the grouting situation in the main tunnel.

Figures 70 and 71 depict the total amounts of grout mass and the average pressure of the main tunnel, respectively. Since there are no other fans to compare the columns with and this lone grouting fan contains additional control holes, comparing this with figures 66 and 67 would serve no real point. Loosely, a comparison with figures 58 – 61 is somewhat logical due to the fact that these figures depict the amounts of total grout mass and average grouting pressure per grouting fan in the main tunnel at Sjøskogen. Still, it is important to keep in mind that even though these tunnels are being excavated in the same rock mass, there are significant

differences in the geological conditions, such as the joint systems, the water inflow situation, the presence of intrusions and their natures, etc.

Studying figure 70 in comparison with 58 and 59, it can be seen that even when including the additional control holes in the grouting fan, the amounts of injected grout mass are slightly less than what would be the average in the main tunnel at Sjøskogen. Since there is but one grouting fan, it would be meaningless to blame the quality of the rock mass for this, as there are no tendencies of development at Snekkestad. However, should this be a usual amount injected, it could be argued that the rock mass at Sjøskogen has slightly better quality, since it demands less grout mass injected.

The average pressure for the grouting fan at figure 71, when compared to figures 60 and 61, shows that the grouting fan at Sjøskogen demands lower pressure. Whereas the average pressure of all grouting fans performed in the main tunnel at Sjøskogen is approximately 80 bar the grouting fan at Snekkestad only has 59 bar. One argument could be that the rock mass is more compact, demanding less pressure for all holes to be sealed, however it is again no point to speak of tendencies as there are no one at Snekkestad. However, as was the argument with figures 70 compared with figures 58 and 59, should this value be representative for the subsequent grouting fans in the main tunnel, it would seem that Snekkestad has a better rock mass quality.

It is also important to keep in mind that the main tunnel is being excavated next to a technical storage room which in the tunnel excavation period serves as a storage room, and since these are being excavated simultaneously, the already injected grout mass in the walls around the storage room would surely affect the outcome of the pre-grouting fans performed in adjacency to the storage room.

The distribution of grout mass and grouting pressure per borehole graphed on figures 72 and 73 could also be compared to figures 62 and 63. The situation is as with the rest of the data for the main tunnel; no tendencies, thus no reason to speak of similar development. When regarding at figures 72 and 62, there are a few similarities such as holes 18 – 20 on figure 72 and holes 20 – 23 on figure 62. All of these holes have fairly low amounts of grout mass injected into them and are found in approximately the same area of the grouting fan; the left tunnel wall and the upper left abutment. Other than that, there are virtually only random distributions.

Figures 73 and 63 could be argued as being fairly similar to each other. However, on figure 69 the average grouting pressure is approximately 60 bar while being 80 bar on figure 63. Holes 22, 26 and 29 on figure 73 have lower pressure values as compared to holes 25 and 27 on figure 63, the latter figure with slightly more variations depending on the section number. Though these holes are found in almost the same area of the grouting fan in both tunnels; the upper left abutment and the left side of the tunnel ceiling, the author argues that this is strictly arbitrary.

9.4 Inflow criteria and grouting

Considering the inflow requirements at the different tunnels, Sjøskogen immediately appears as the project with the least strict criterion, where both the R4 and the main tunnel part have an inflow criterion of $10 \frac{L}{\frac{min}{100 m}}$. Theoretically, this would require less amounts of grout mass injected per grouting fan as opposed to for example the sections of the R13 tunnel at Snekkestad with inflow criterion of $5 \frac{L}{\frac{min}{100 m}}$. From figures 54 and 66, it can be seen that despite the virtually same number of boreholes injected with occasional variations, the R4 tunnel overall demands more grout mass injected per section number than the R13 tunnel does in the areas where the inflow criterion is $5 \frac{L}{\frac{min}{100 m}}$. The rock mass quality of the R4 and the R13 tunnels is fairly good, with perhaps the R13 tunnel just slightly being a little better. This might be a reason for the higher amounts injected at Sjøskogen.

However, when comparing table 1 and figure 66, it can clearly be seen that the strictest criterion in R13 applies for the first half of the tunnel; approximately from section 320 – 145, when taking into consideration that the R13 tunnel starts at section 320 and that $320 - 175 = 145$. Within this part of the tunnel is the non-grouted zone traversing the Brekke tunnel found, and since there are plans to post-grout this area it will become interesting to see if the amounts of injected grout mass per borehole increases once the post-grouting is complete. Still the injected sections show significantly less amounts of grout than the R4 tunnel, and since the rock mass quality more or less is the same for both of these tunnels, the author is left to argue that this is purely due to geological differences, predominantly joint systems, joint frequency, occurrences of weathered rock, etc. Also, the different grout blends utilised in addition to several additives may cause the grout mass to harden at a later time.

When studying the rest of the R13 tunnel all the way to the fork section at which the main tunnel part starts, the amounts of grout mass remain at more or less the same level as was the case for the section with the inflow criterion of $5 \frac{L}{\frac{min}{100 m}}$. This rather odd behaviour, when compared to the R4 tunnel at Sjøskogen is not easily explained. Disregarding the distance constituted by sections 132, 172, 245 – 290 and 321 – 336 which contain a great drop in the amounts of grout mass, the R4 tunnel has considerably more grout mass injected into it than R13 does. It is difficult to find a single answer for this as the rock mass is fairly compact in both of the tunnels with the R13 tunnel slightly showing a better quality at times. As theorised previously, various complex and undulating joint systems and altering joint frequencies along with appearances of weathered rock zones may help understanding why the R4 tunnel requires more grout mass than the R13 tunnel does, over a distance with same inflow criterion.

Otherwise, it may be argued that the difference in a pre-grouting fan for a tunnel with these inflow requirements is mainly founded on the number of boreholes, but since the difference of the pre-grouting fans for R4 and R13, seen on figures 52 and 64 is not very big, the argument

of local joint properties in the rock mass seem to be the most reasonable. Also, to the author's knowledge the determination of an inflow requirement does not solely rely on the overburden and the quality of the rock mass, but also on the number of houses and other installations on the surface, possible bogs or other wet areas prone to drainage, nature conservational sites, etc. Save the first 100 m, the R4 tunnel is found at a rather scarcely populated area in the Holmestrand area and there are no significant installations or other features on the surface which demand a strict inflow criterion, it appears thus that the criterion has been elevated. Reversely at Snekkestad, there is a nature conservational area in the vicinity and also an initially low overburden. The R13 tunnel also traverses below the Brekke road tunnel. These are factors that have likely forced the inflow criterion to remain at $5 \frac{L}{\frac{min}{100 m}}$ for the first half of the tunnel.

The injected grout mass amount per section in the main tunnel parts at Sjøskogen and Snekkestad, displayed at figures 58, 59 and 70, respectively may also be discussed when bearing in mind that there has been but one pre-grouting fan carried out at Snekkestad, and that the way of comparing figure 70 with the others is by considering that possible future tendencies will remain at virtually the same level as the single grouting fan carried out. By studying these figures it can be seen that the injection pattern varies significantly for the main tunnel part at Sjøskogen, some sections containing over 100 000 L, while others drop down to under 20000 L.

Despite the very undulating tendency it can be argued that overall there is more injected grout mass per grouting fan in the main tunnel at Sjøskogen than at Snekkestad, considering the sole grouting fan is a representative value. Once again, when taking into account that the inflow criterion for Snekkestad is $5 \frac{L}{\frac{min}{100 m}}$ and $10 \frac{L}{\frac{min}{100 m}}$ for Sjøskogen, it seems that the same situation applies for the main tunnel parts as for the R4 and R13 tunnels. Since these tunnels are found in mainly the same locations, the inflow requirements are based on virtually the same factors, and to the author it seems reasonable to claim that the reason for the somewhat strange distribution of injected grout mass between the two main tunnels parts is mainly due to joint systems, joint distribution, etc. The large open joint spaces that were discovered at the tunnel face are also probably responsible for requiring great quantities of grout mass.

9.5 Gevingåsen water and frost protection results

At the time of writing, the water and frost protection in the Gevingåsen tunnel has been installed and in use for a little more than one year, thus there cannot be any talks of longevity or long-time duration. However, the results spotted and obtained may still be discussed and reflected upon.

The very coat of BASF Masterseal 345 installed in the Gevingåsen railway tunnel showed little or no signs of seepage save for various scattered spots of moisture in the tunnel walls. At no points there were wet areas on the railway. There were some wet areas, naturally frozen as the

excursion took place in winter, but these were found on the pavement at the side of the railway. A typical seepage has been depicted and can be seen on figure 74.

Assuming that the methods used for water and frost protection for the station hall will either be concrete elements or PE foam plates, it could be interesting reflect at how the sprayed membrane solution would perform. To the author's knowledge, the very seal itself does not tolerate water prior during the installation phase, meaning that the layer of shotcrete which the sprayed membrane solution will be installed on must be completely dry.

This also means that the entire tunnel must be drained, something that takes a significant amount of time and cost. With such a great geometrical contour that the station hall has, this will indeed prove a huge task to accomplish. Especially the weakness zone in the southern part of the station hall would demand significant amounts of drainage works before even considering installing the sprayed membrane solution due to the expected low quality state of the rock mass. Also, taking into consideration that the rock mass is column basalt, the vertical joint sets might easily conduct water down to the station hall, making the drainage task even more difficult.

On the other hand though, once finally installed it appears as if the sprayed membrane solution performs very well and should the present results from the Gevingås tunnel remain, this water and frost protection system seems to handle its mission decently. Not installing huge concrete elements or PE foam plates makes the installation process fairly simple. The preliminary results from the temperature measurements also indicate that there is no freezing occurring as long as the membrane is whole and not punctured. It is also important to remember that a hole in the membrane does not equal an inflow into the tunnel, since this point has to coincide with a seepage point in the shotcrete. Water is unable to travel along the interface of the membrane, thus a seepage point can be fixed locally without worrying about inflows. This is a strength of the sprayed membrane solution, however it causes another practical problem since the outer layer of shotcrete must be removed before fixing the damaged part of the membrane.

10 Concluding remarks

Based on the aggregated data and the discussion regarding the already performed pre-grouting fans in the tunnels at Sjøskogen, Snekkestad and the R7 and R9 tunnels it would be interesting to perform a declaration made in advance and also a conclusion about the possible theoretical outcome of the grouting fans for the station hall, including the weakness zone. The rock mass of the planned station hall is the same basalt as is found at Sjøskogen and Snekkestad. The only slight difference is that the basalt in the station hall is column basalt containing at times three vertical joint sets and one joint set parallel to the foliation of the rock mass. For more information regarding the geological situation of the station hall, see (Ryningen 2011) . Also the water and frost protection situation for the station hall will be discussed.

10.1 Inflow situation based on pre-construction investigations

With respect to the already performed investigations prior to the excavation stage on figure 9, there are no slight indicators of any potential significant inflow situation in the rock mass. The ERT profiles on figure 12 show various zones with low resistivity and some zones with high resistivity. High-resistivity zones contain rock of good quality while the opposite applies for the low-resistivity zones. The former is composed mostly of basalt while soil masses, mostly clay, constitute the latter. Pure clay masses not being water-conducting will thus not pose any significant threat to water inflow in the station hall but rather work as a sealing layer. However, it is very important to notice that impurities in the clay masses such as sand zones and fragments of weathered rock will take away this effect and open up water-leading channels and pore rooms.

The more compact part of the rock masses the basalt rock will also conduct water. Based on core drillings performed (Ryningen 2011), several joint sets have been obtained and registered. It is realistic to assume that these joint sets will be the dominating cause of possible water inflows into the tunnel. Though some joints and cracks may be filled up with sediments deposited during the ice ages which act as a seal, it is not likely that these sediments are composed of pure clay.

On figure 11, CPTU and oedometer measurements reveal a value of s_{ua} at some 20 kPa above the recommended value. OCR values are scarce but it can be argued that possible trends indicate a graph which develops and thus drops at a slightly slower pace than the green, recommended value. These differences however are not thought to compose any specific danger since they account for a fairly little deviation.

The various pore pressure measurements that are performed over one year have to be seen in comparison with the annual precipitation occurring in Holmestrand. Figures 14 – 16 which

show the pore pressure measurements in the soil masses above the station hall, all indicate a lower groundwater table in summer which correlates with figure 21. Oppositely when approaching the autumn, the groundwater table increases based on a growing amount of rain. Naturally due to local differences, the trends in development for the graphs on figure 14 – 16 are different. Especially figure 15 is somewhat difficult to interpret. The author still argues that the annual trend of the groundwater table sinking in summer and increasing in autumn is partially visible. The rest of the year, with the groundwater table being low in mid-winter and increasing again when approaching spring due to melting of snow and ice, is also visible.

When considering figures 17 –19 for the two-month period there is as expected, an ongoing negative trend in the groundwater table. This period represents the time of the year when average temperatures are below zero and precipitation is scarce. Additionally, the excavation began only some days prior to the first day of this measuring period. There are no sudden drops in these graphs that would indicate any significant affection of the excavation on the groundwater table.

Taking figure 22 into consideration it would therefore be logical to assume that the major part of the water inflow occurring into the station hall will take place in autumn when precipitation increases and average temperatures are above zero, and in spring when the melting of snow and ice along with average temperatures over zero will increase the groundwater table. Local differences such as a clay zone, a highly compact part of the basalt without joint sets, etc may cause the inflow pattern to deviate from this. In areas with the basalt containing multiple joints and the soil masses being coarse, more inflow may occur.

The drainage pattern of the Holmestrand plateau on figure 23 is subject to remain at its current state should things continue the way they are at the time of writing. Still it can be argued that the huge rock cavern that is the station hall will alter the drainage pattern in some way, possibly causing most of the water to be lead towards its tunnel ceiling before letting the water flow outwards to the fjord. The author argues that this will likely not cause any major consequences.

However, it would be still be wise to be aware of the drainage situation. Studying figure 24 drainage of soil masses may cause drops in the pore pressure level and lead to subsidence. Local differences such as undulating joint sets and scattered clay zones might lead to subsidence should the drop in the groundwater table be great based on a changing drainage pattern. Indeed this is a very pessimistic approach, however attention should be paid to the drainage pattern and pore pressure measurements as excavation works for the station hall begin.

10.2 Grouting situation for the station hall based on previous grouting

The grouting performed in the tunnels at Sjøskogen and Snekkestad along with the R7 and R9 tunnels reveal slight but passive patterns in the grouting processes. Prior to predicting any patterns for the station hall, it is important to keep in mind that not only will the rock mass, the column basalt be slightly different from what has been injected so far, but the size and the vastness of the station hall along with the fairly flat tunnel ceiling will make the grouting fundamentally special and different from the other tunnels.

For all of the tunnels excavated so far there have been partial definite patterns in the amounts of grouting mass injected per borehole, highly related to local geological differences and the consequences of previous grouting fans. Mostly, these are either found in the tunnel ceiling or on one of the tunnel walls. At times the tunnel floor also receives the maximum amount of grout mass. For nearly all of the cases this is consistent with the geological situation of the tunnel face, such as local weakness zones, abundance of joint sets and otherwise a situation which demands high amounts of grouting mass. All of these cases represent possible water inflow situations. Therefore whenever probe drillings reveal rock of low quality or the seepage water from the boreholes has a reddish colour, extra attention should be paid to these areas, and this will probably indicate requirements of extra amounts of grout mass. Noticeable joint sets which keep appearing in repeated patterns will likely indicate zones with demands for more injected grout mass.

The grout pressure distributed per borehole is fairly constant for most of the tunnels excavated, with an average value of 60 or 80 bar. A unique exception is the R4 entrance tunnel at Sjøskogen where holes 13 – 30 have an overall lower value than the rest. These holes constitute the tunnel ceiling and the right tunnel wall. This is consistent with what has been discovered during the excavation works; various weakness zones occurring on the right tunnel wall and at times in the tunnel ceiling. Should there be repeated appearances of weak rock such as numerous joint sets or a weathered zone it would be wise to lower the grouting pressure for that specific area. Other than that, it is strictly the geology itself in combination with previous grouting fans that will decide the grouting pressure for every grouting fan performed in the station hall. Also in-situ measurements of the stress situation of the rock mass has to be considered, since grouting at a higher pressure than σ_3 may disrupt the rock mass. Knowing that $1 \text{ bar} = \frac{1}{10} \text{ MPa}$, it would be wise to conduct several 3D stress measurements during and prior to the excavation of the station hall.

The amounts of injected grout mass for tunnels R7 and R9 on figures 44 and 48 remain, with some variations at the same level as for the R13 tunnel at Snekkestad. Due to good rock quality yielding fairly high Q-values it may partially help explain the reason for this trend. Also, since the R7 and R9 tunnels are decidedly shorter than the R4 and R13 tunnels the shortage of data when comparing the two pairs may be a reason for this deviating trend and it

would be interesting to see how the amounts of grout mass injected would have changed or not if R7 and R9 were as long as the R4 and R13 tunnels.

The total amounts of injected grout mass per grouting fan will predominantly rely on the geological situation and previous grouting fans and is subject to change for every grouting fan, thus making it nearly impossible to assess any accurate number of expected grout mass to be injected. Still, a simplified attempt can be made when regarding the previous grouting works. For reasons of simplicity, only the main tunnel will be considered and since the only grouting fan at Snekkestad in the main tunnel contains additional holes, this will be neglected. Thus, only the Sjøskogen main tunnel will be used. The following formula will be used:

$$(VI) \quad A = n \cdot \frac{\Sigma T}{n_b}$$

where

A = loosely assessed amount of grout mass per grouting fan (L)

n = number of boreholes for a common pre-grouting fan

ΣT = Average amount of grout mass for all grouting fans (L)

n_b = average number of boreholes for the main tunnels

Inserting the various numbers yields:

$$A = 125 \cdot \frac{82276,25}{60} = 171408,85 L.$$

Knowing that a common value for the tunnels at Sjøskogen is 82276,25 L for a total of 60 holes and that a common pre-grouting fan in the station hall will contain 125 boreholes, it is somehow realistic to assume a value in the vicinity of this when considering that there are no significant weakness zones present or any other geological or technical difficulty that demands more amounts of grout mass injected.

Calculating a theoretical amount of grout mass for a possible weakness zones includes so many determining and critical factors that the author sees no real point in doing so. Studying the already performed grouting works at Sjøskogen, Snekkestad, R7 and R9 shows that the amounts of grout mass is subject to vary greatly between normal pre-grouting fans in what is otherwise considered as good rock mass, therefore it will prove too challenging, time-consuming and diffuse to try to assess an amount of grout mass injected into a grouting fan at a possible weakness zone.

Also depending on how grouting fans imbricate each other, the result will vary. When tunnels R7 and R9 reach the wall of the station hall the tunneling will physically turn and begin progressing in the north – south direction. In the process of doing so boreholes for the grouting fans will be obliquely drilled, causing various layers of grout to be present in the rock mass at skew angles compared to the direction of the tunnel axis of the station hall. At this point it is likely that grouting fans going in the north – south direction will be affected by the skew ones, mostly in the tunnel walls. A possible outcome would perhaps be reduced amounts of grout mass in these locations of the grouting fans and also reduced grout pressure to prohibit disruption of the already injected grout mass.

The contour and shape of the station hall is immediately dominated by the very flat tunnel ceiling. Knowing that such a flat ceiling in comparison to high tunnel walls and a long width causes a delicate stress situation for the tunnel ceiling, injection procedures might want to specifically focus on this area. When keeping in mind that the overburden is 40 – 60 m, this may be a factor that can complicate grouting works even more. Reduced grouting pressures might help stabilising the situation for the tunnel ceiling, however reducing the pressure too much will cause the stoppage criteria to never be achieved, making it become a balancing task. The joint sets in the column basalt may be prone to conduct the grout mass upwards towards the surface on top of the Holmestrand plateau, however this is not likely to happen as long as the grouting pressure is being kept at an acceptable level. Giving an exact number on the grouting pressure necessary to keep the situation stable is next to impossible since the grouting situation changes for virtually every new blast.

When studying figure 7, the layout of station hall along with the expected weakness zone, it can be seen that the weakness zone will not be encountered in the station hall itself but rather in the southern end section. The weakness zone also traverses the R8 entrance tunnel which at the time of writing has not yet been fully excavated. The contour and shape of the station hall thus escapes the very weakness zone itself, however there might be rock mass of low quality adjacent to the weakness zone that will affect the grouting works. There might also be other types of weakness zones or low-quality rock mass that has not yet been shown on the various investigations which have been conducted.

A sketch of a typical grouting fan for the weakness zone in the southbound end of the station hall is shown on figure 30. It would be expected, as for a common grouting fan, that the tunnel floor is being injected at first and then the injection progresses up towards the tunnel ceiling. Most likely, such a grouting fan will consume significantly higher amounts of grout mass at a lower pressure than what has been a possible trend. The fractured zone with the weak rock mass, probably also containing abundant amounts of joint sets will also demand a lower grouting pressure, otherwise it may fail. The need for additional control holes may also be present, should the rock mass be in a severe state. Further on, when studying the grouting fan on figure 30, it can be seen that the upper cross-section is nearly split in half as the need for sub-fans to imbricate each other within the main grouting fan is important to retain the strength and stabilisation of the rock mass. It is likely to assume that this pre-grouting fan will

consume more time and cost due to its complexity and the presumed low quality of the rock mass.

In the previous grouting works which have been logged and registered, emphasis has not been put on the blends. However, it may be of interest to discuss this matter concerning the grouting situation of the station hall. The main difference between the blends is predominantly the usage of standard cement versus micro cement, more precisely the grain size. Micro cement is able to penetrate deeper into tiny cracks and joints than standard cement and it would be logical to assume that increased amounts of micro cement in the weakness zone and in other parts of the station hall where the rock quality goes down. At the same time, the grouting pressure has to be kept low in order to prevent the rock mass from failing.

The cost of the usage of micro cement is to the author's knowledge is mostly critical to determine whether it should be used or not, as prices for the micro cement is fairly higher than for standard cement. It can be argued that a mix of the two blends ought to be tried at first to see if the stoppage criterion is achieved, if not, then add more micro cement and see if the situation stabilises. Also, the w/c – relationship must be considered as lowering the value would decrease the viscosity of the blend, making it easier to flow and entering all tiny joints scattered around in the rock mass. Despite all this though, local geological differences and the in-situ situation at the tunnel face will prove crucial.

For every single grouting situation, decisions will have to be taken at the tunnel face at the time of the injection occurring. There can be guidelines and theories prior to the grouting, however since the situation can alter completely for each and every grouting fan in the same rock mass at otherwise similar situations, there is no possible way that an accurate prediction of the grouting works of any tunnel can be made. It is crucial and necessary that a preview of the rock mass is allowed to take place through probe drillings, as the information acquired through the MWD data will help the tunnel workers and the engineering geologist determining the maximum allowed pressure, the blend that will be used, etc. Also, information about the rock mass through carefully examining at the tunnel face may provide helpful information needed to assess the subsequent grouting fan.

10.3 Inflow criterion and injected grout mass

The inflow criterion for the station hall has been set to $5 \frac{L}{\frac{min}{100 m}}$. Based on what has been observed in the previous excavation and grouting works for the other tunnels studied hereunder, it would seem like the surface installations and properties in general constitute no major impact on the amounts of injected grout mass, since it has already been seen, from comparison of the Sjøskogen and Snekkestad tunnels that it is predominantly the rock mass itself that is responsible for the quantities of grout mass injected. A certain special situation arises for the station hall since the rock mass is composed of column basalt with 3 vertical joint sets. These joint sets may be highly water-conducting and therefore demand more grout mass than what would usually have been the case. Taking into consideration that 2 of the 3

vertical joint sets already have been spotted at Sjøskogen, this could mean that the calculation of expected amounts of grout mass injected for a pre-grouting fan under normal circumstances may be slightly underrated. Thus, it may be expected that more grout mass is required to seal the joints and to keep the station hall dry. Due to the sheer complexity of such joint sets and various other factors such as the angle of the borehole, etc, it is very difficult to try and calculate an expected average amount of injected grout mass per grouting fan, and so the author suggests caution and awareness, as well as preparation for fairly vast pre-grouting fans in the station hall.

10.4 Water and frost protection usage

Advantages and disadvantages of the two most likely methods of usage have already previously been discussed. A hypothetical situation where a sprayed membrane solution is being utilised may thus be discussed.

Already knowing that the installation and application of the membrane is fairly easy, the main disadvantage of the application of this solution is the drainage works. Drilling drainage holes which conduct the water flow outside the shotcrete and membrane layer might prove expensive and long-during should there be severe inflows taking place. Since the inflow pattern and intensiveness depends on the quality of the rock mass and the joint sets, joint spacing, joint frequency, etc, it is not possible to predict the extent of drainage holes in the tunnel ceiling until drilling works start.

Also, grouting performance will have a major effect on this type of water and frost protection. It is not possible to gain a completely dry tunnel with such a large contour, meaning that there will always be water dripping at some point of the tunnel or in the best case several spots of moisture. This means that the possibility of water dripping down onto the railway is present, and this should better be avoided, as water eventually will disrupt the technical and electric installations. Thus, a coat that prevents the water from flowing in may be a better solution than huge concrete elements or PE foam plates. Determining which method is the better one beforehand is not possible, however it can be somehow decided when looking at the inflow situation in the station hall subsequent to the grouting works.

Should there be numerous great seepages taking place and abundance of water flowing into the tunnel, drainage works will prove extensive and the sprayed membrane solution may be too much of a challenge to install, even if the application of the membrane itself is fairly easy. Also in such a situation knowing that the rock mass is highly water conducting, punctures in the membrane will likely lead to disruptive consequences. In a situation like that it is arguably better to utilise the PE foam plates or concrete elements. Perhaps the latter would prove the best since it is fairly easy to install however cleaning and maintenance procedures are easier for the concrete elements. Also, since water leakages normally do not pose a problem, the concrete elements probably will constitute the best solution.

In situations where the grouting has well prevented the water from flowing into the tunnel and where there are no significant inflows in the station hall the sprayed membrane solution might well be utilised, as long as the drainage works go smoothly. The author argues however, that this method requires more testing and more in-situ trials before being approved to usage for such a grand project like this. A more appropriate way of approaching this type of solution would be to test the sprayed membrane not only on parts of a tunnel like in Gevingåsen but rather throughout the whole tunnel before considering applying the method on the station hall.

Indeed, the deformation properties of the composite layer in which the sprayed membrane solution is included is an argument for considering this solution when regarding at the strength and deformation load perspective of the station hall. Though the composite layer may initially fail at a lower load than a common massive shotcrete layer the composite layer is able to withstand deformation at a higher workload than the shotcrete layer and even a higher load exerted than what made it initially fail. Should the excavation of the station hall prove that the rock mass is fairly deformable, the sprayed membrane composite solution would be of utmost aid.

However, it must be stressed that prior to the excavation works; it does not seem as if the B1 group contains highly deformable rock types. No indications of that have appeared in the engineering geological investigations (Ryningen 2011), and the author has not yet observed any indications of that during the excavation and securing of the R7 and R9 tunnels. There have been minor occurrences where petite slabs of rock have fallen down from the tunnel ceiling. This happened prior to shotcrete and bolts being applied onto the rock mass. In the tunnels at Sjøskogen and Snekkestad there have been no such incidents.

The author suggests that a somewhat careful approach could perhaps be to utilise a sprayed membrane solution in one of the entrance tunnels R7, R8 or R9. These tunnels have a less cross-section area and are lengthwise shorter, making them suitable for experimenting with such a solution.

Regarding the frost protection, based on the results from the various tunnels containing the sprayed membrane solution, it is the author's opinion that the heat capacity of the rock mass will most likely prohibit freezing of the water. However, this is naturally dependant on the rock type.

The other methods described herein are not likely to be subject for usage in the station hall save the concrete lining. In extreme cases, such as a highly permeable low-quality rock weakness zone lining the full profile may be an alternative if the other solutions not are providing the satisfactory results desired. In the weakness zone at the southern end of the station hall should the rock mass be so weak that it can hardly support itself covering the whole tunnel profile in a thick layer of cement may be necessary. Before such an extreme step though, the author argues that other solutions, including fibre reinforced shotcrete arches in combinations with particular thick layers of shotcrete may be considered and even tried before bringing in the concrete lining shield. Combining concrete lining with grouting should,

in the author's opinion definitely keep the station hall stable and dry satisfying the inflow criterion.

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12 Appendices

Appendix A: Engineering geological length profile of the Sjøskogen tunnel. (Jernbaneverket 2011)

Appendix B: Engineering geological map of the surroundings of the Sjøskogen tunnel. (Jernbaneverket 2011)

Appendix C: Engineering geological length profile of the Snekkestad tunnel. (Jernbaneverket 2011)

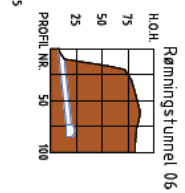
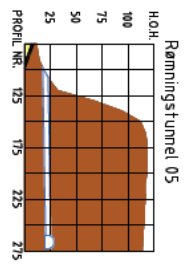
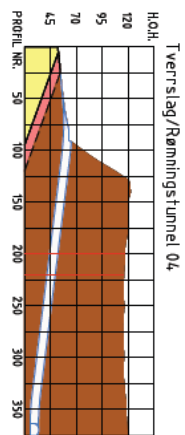
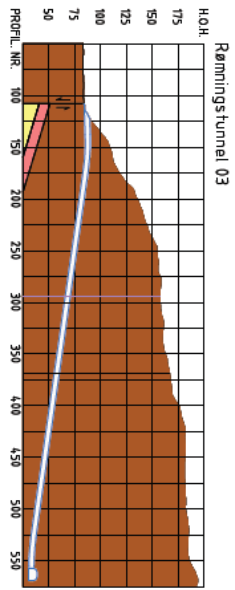
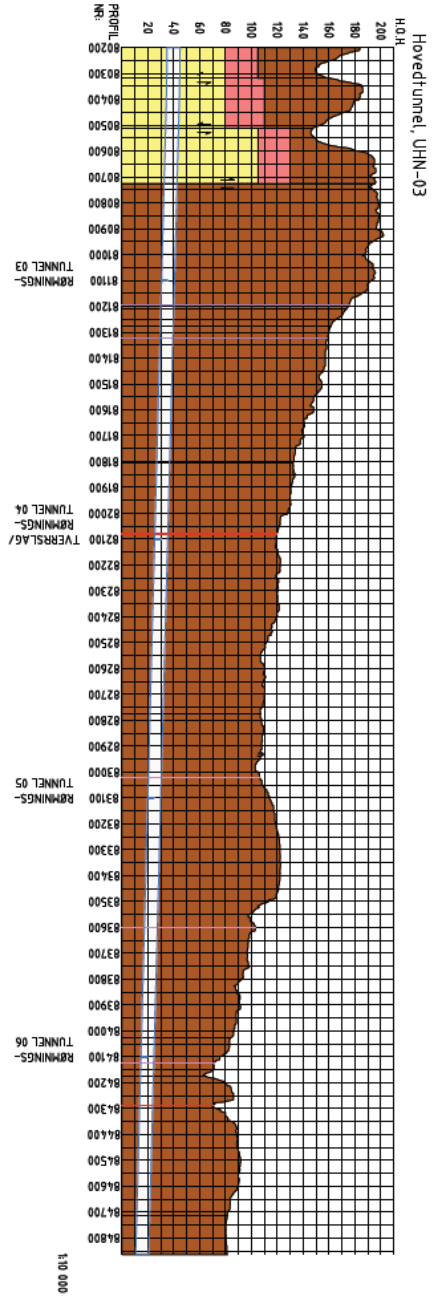
Appendix D: Engineering geological map of the surroundings of the Snekkestad tunnel. (Jernbaneverket 2011)

Appendix E: Engineering geological length profile of the station hall. (Jernbaneverket 2011)

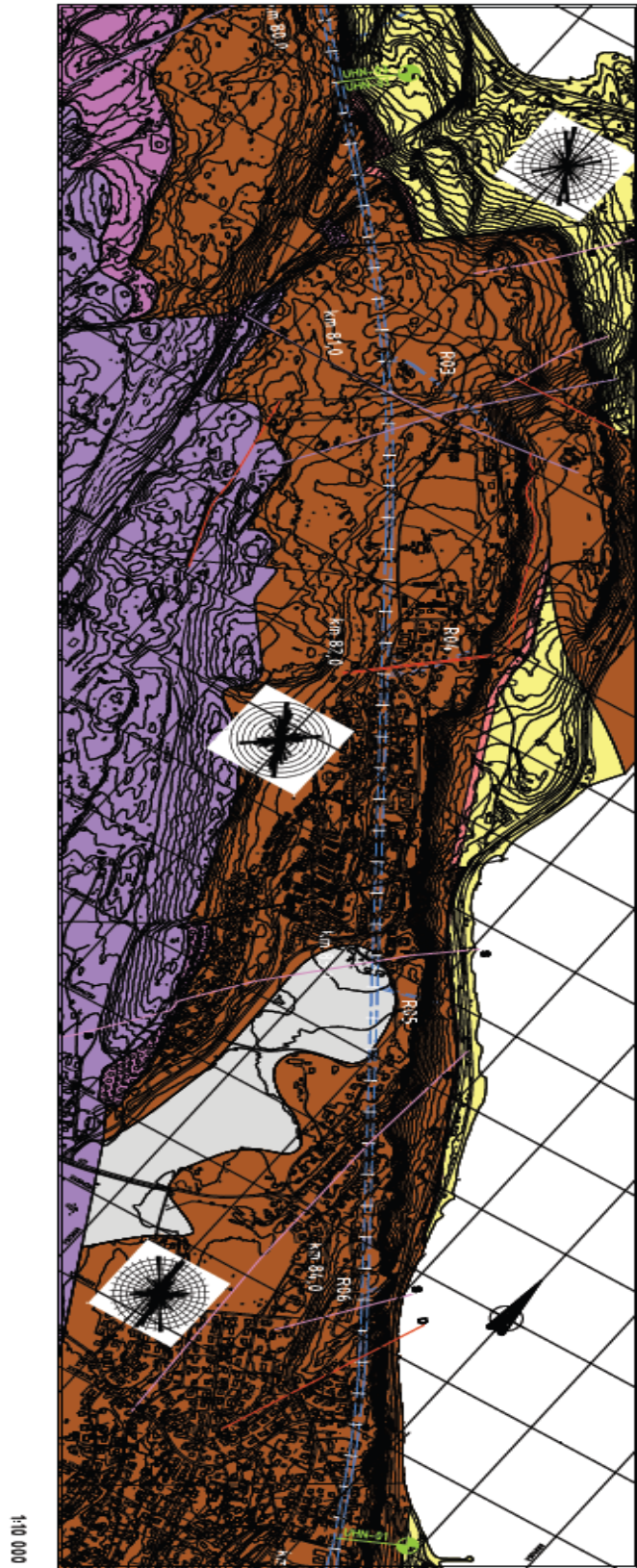
Appendix F: Engineering geological map of the surroundings of the station hall. (Jernbaneverket 2011)

Appendix G: Calculated Q-value of the R7 entrance tunnel. (Jernbaneverket 2012)

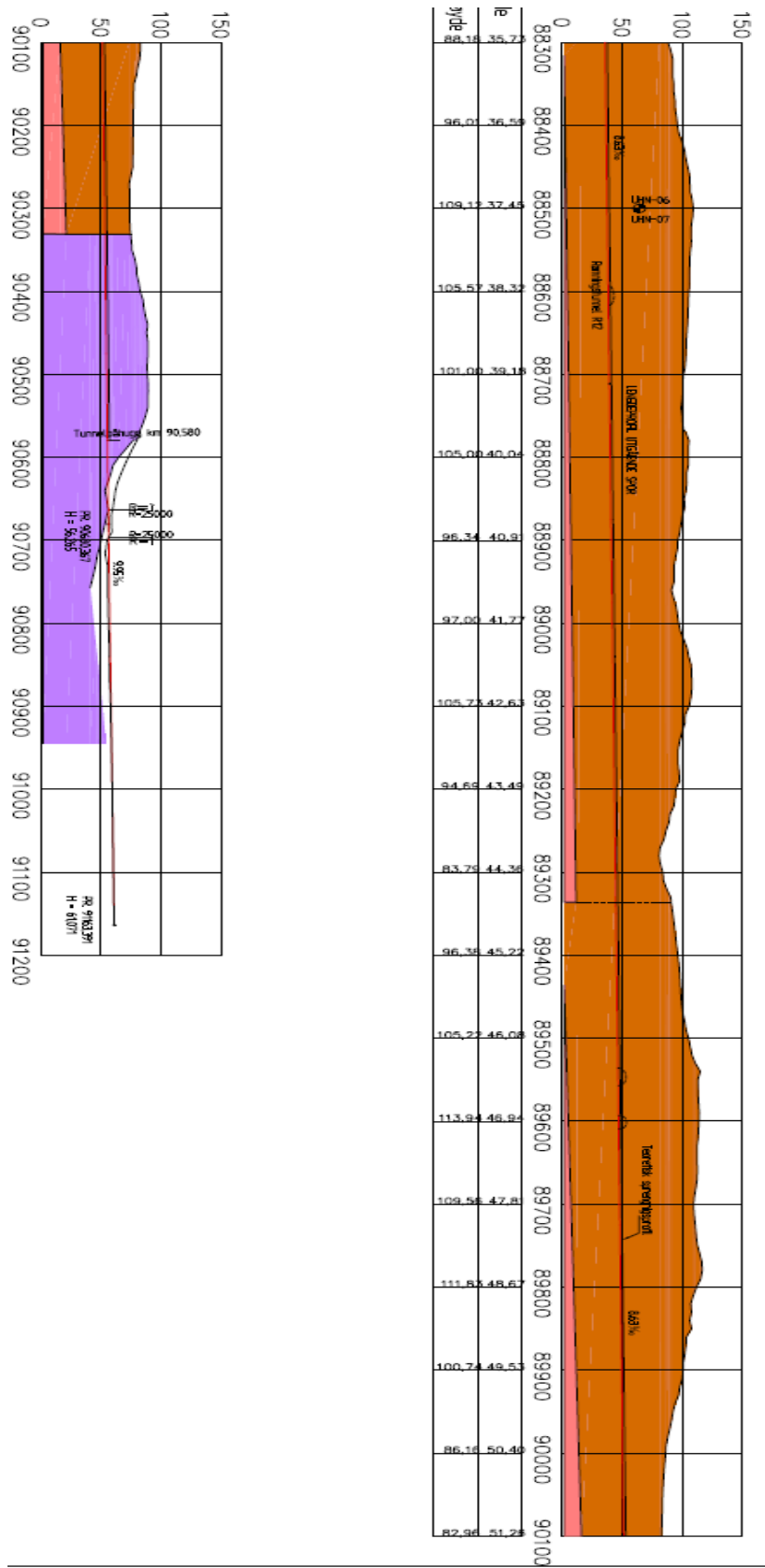
For appendices A – F: Colours indicate the rock types. Brown is the B1 basalt group, yellow the Ringerike sandstone, purple rhomb porphyry and red the Asker group; sandstone, conglomerate, etc.



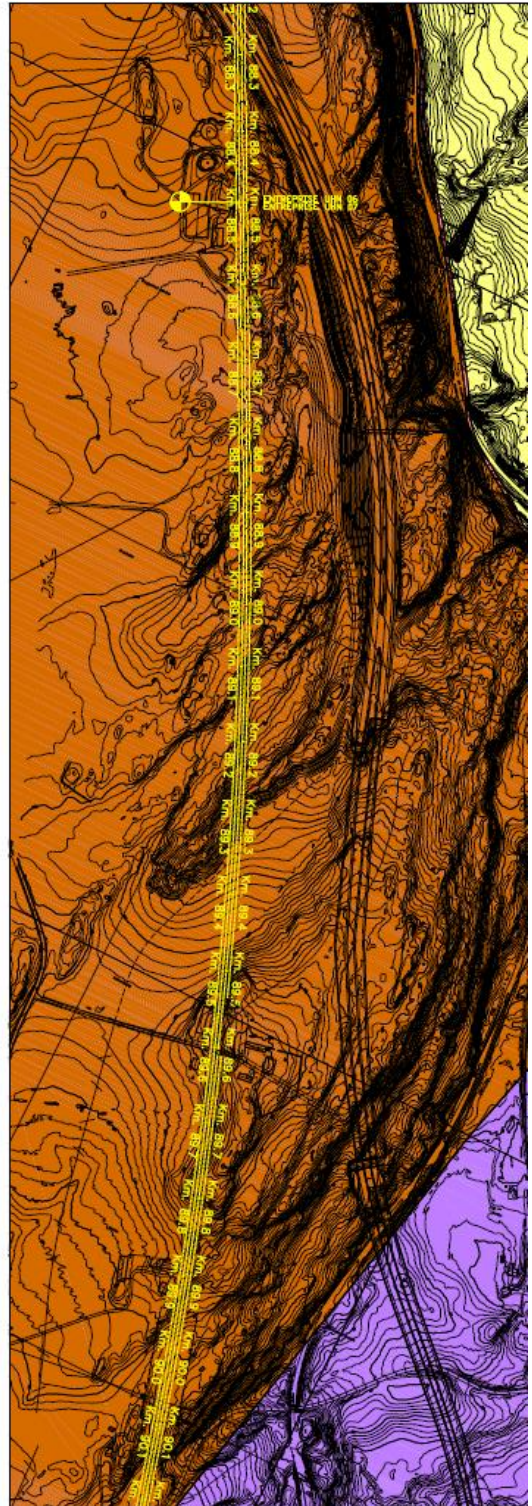
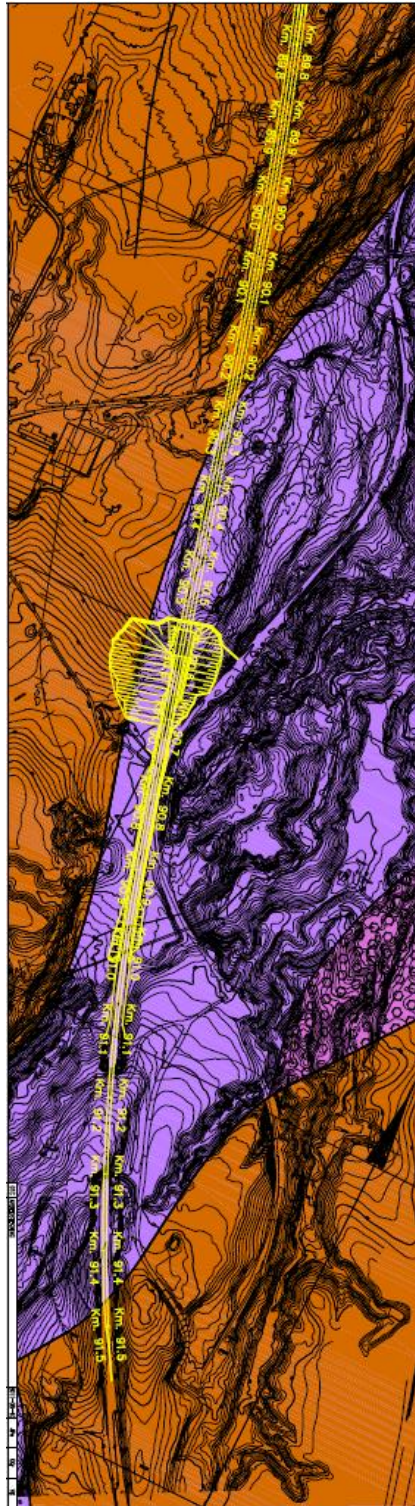
Appendix B



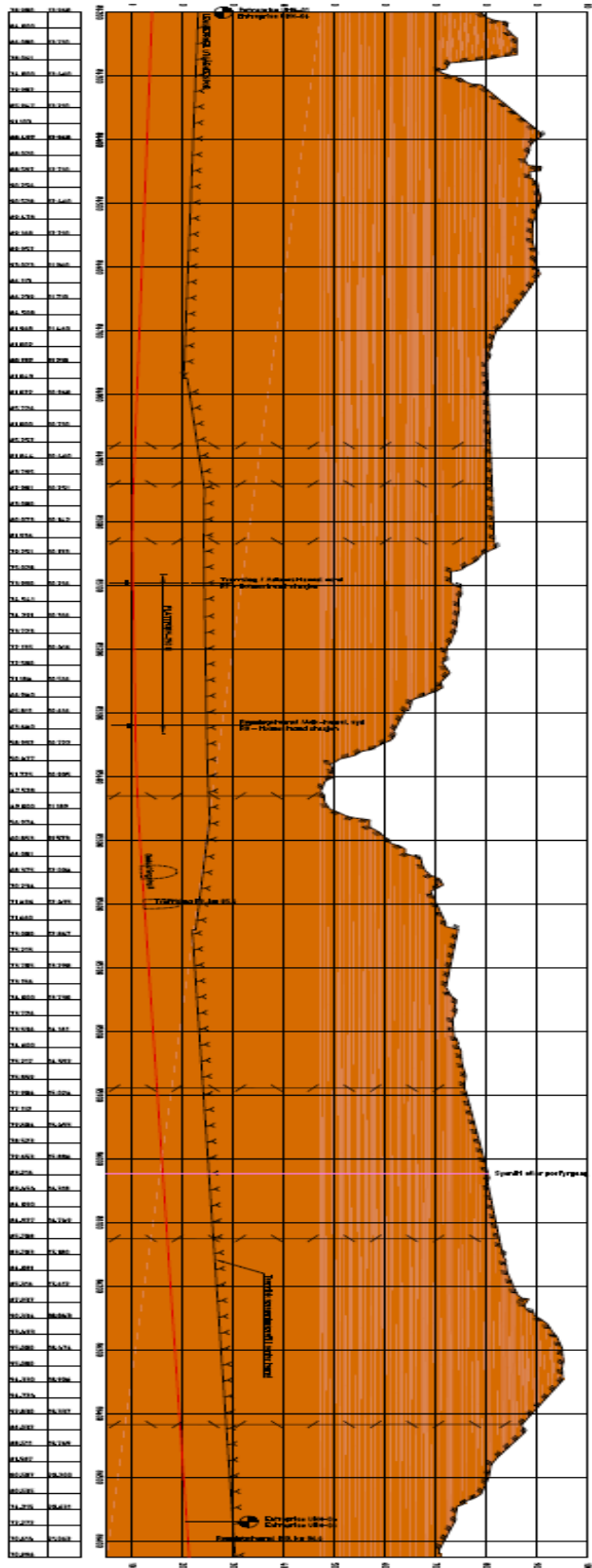
Appendix C



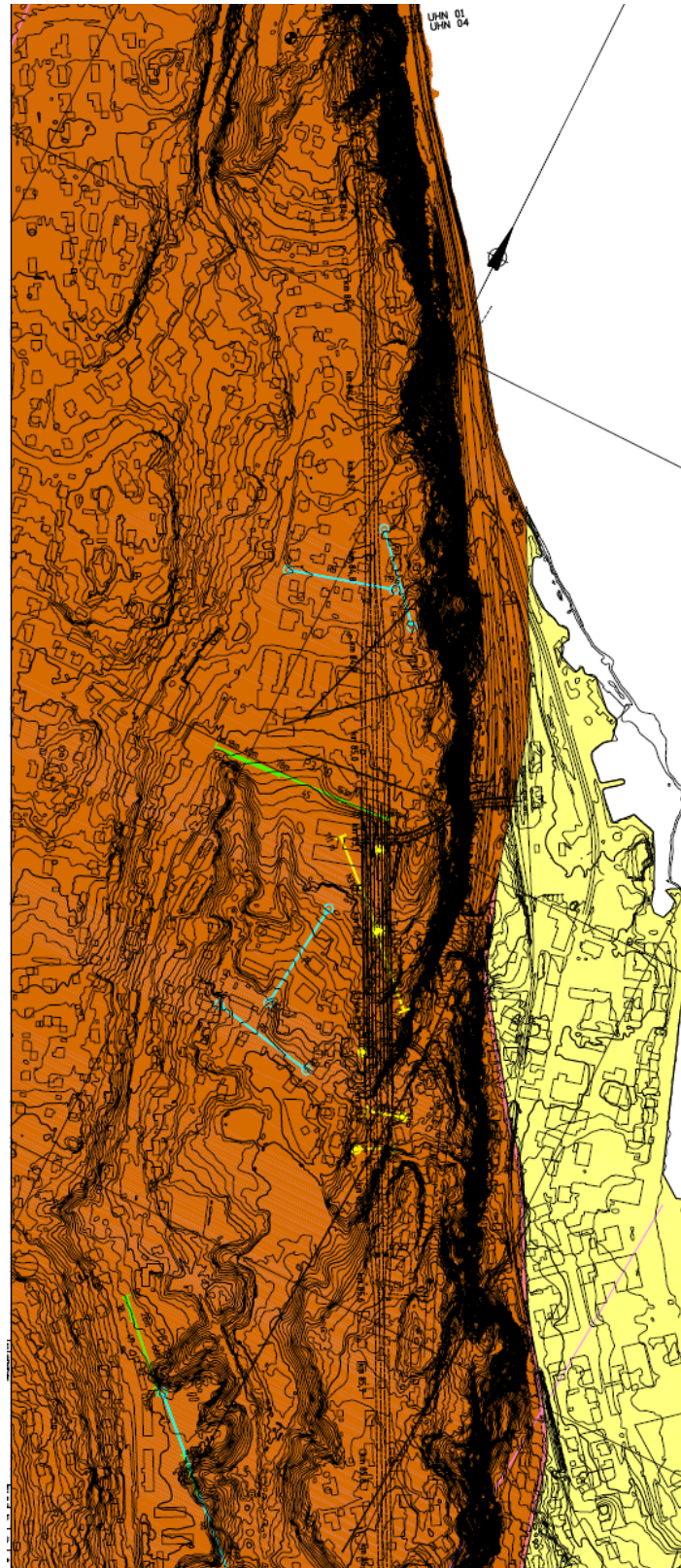
Appendix D



Appendix E



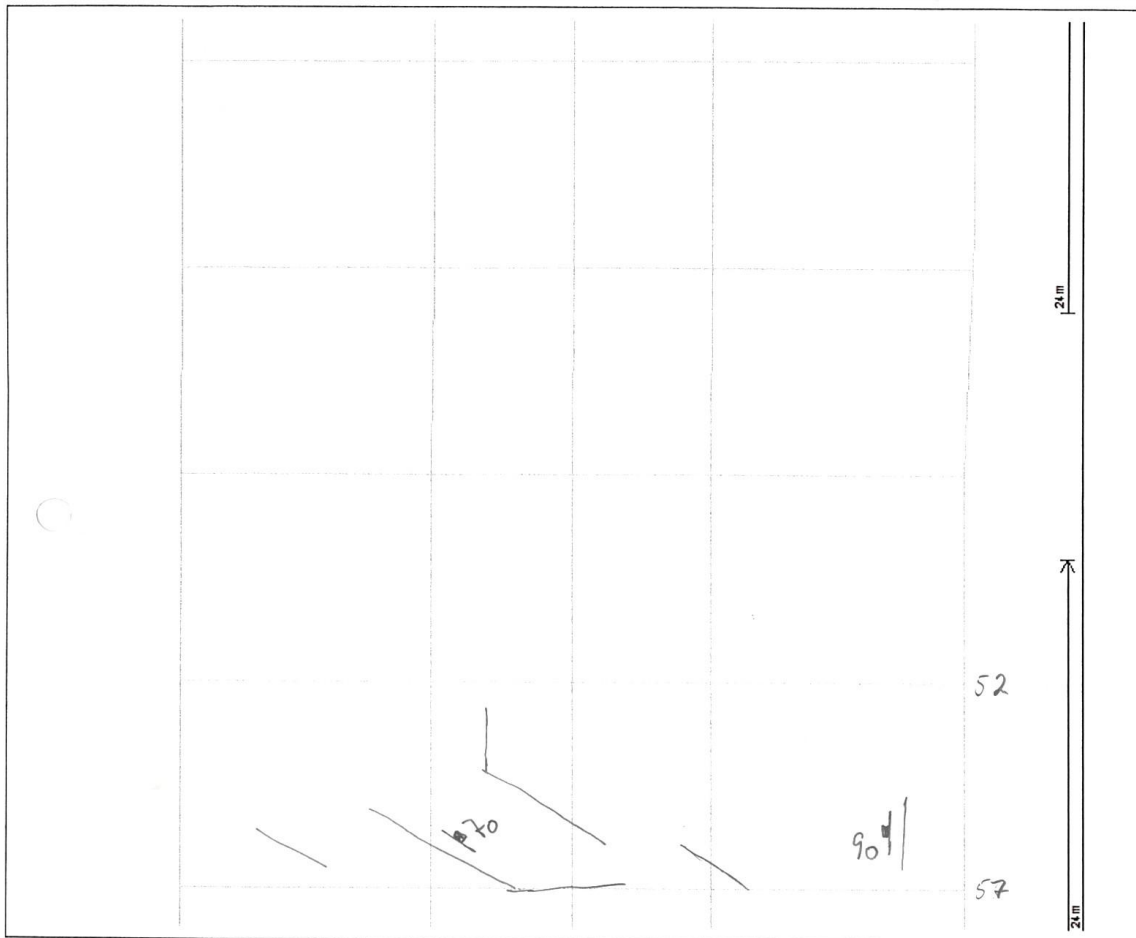
Appendix F



Appendix G

Geologi

Tunnelmodell R7	Dato 10.02.12	Registrert av L. E.	Til-fra prof.nr. 52 - 57
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Tilforklaring

	Diskont.		Svakhetsone < 1 m
	Foliasjon		Svakhetsone > 1 m.
	Sprekk	S	Avskaling etter over 1 time
	Sleppe	SS	Avskaling etter få minutter
		B	Intenst bergslag

Prof.nr.				
RQD	45			
Jn	4			
Jr	3			
Ja	2			
Jw	1			
SRF	1			
Q	16,8			

