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Preface

The Äspö Expansion Project has gradually developed over a three-year period and will be completed in 2014. From the beginning, it was recognized that the design and construction of the TASU and TASP tunnels, as part of the Expansion Project, offered an opportunity to apply the Observational Method. While Observational Method has been used in geotechnical engineering since the 1940's, it is now formally adopted as one of the three Geotechnical Design methods in Eurocode 7. The method is typically applied where the costs of removing project risks to acceptable levels using more traditional design approaches is cost prohibitive.

For the Äspö Expansion Project, the overriding design constraint was maintaining the water pressure prior to construction to similar levels after construction. It was decided early in the project to control the water pressure using a grouting methodology and grout mix that had been developed by SKB over the past decade. The viability of this methodology, while successful at the research scale, needed adjustment and industrialisation to make it practical for the Äspö Expansion Project. Otherwise the costs for grouting could have also been cost prohibitive.

To apply the Observational Method to the Äspö Expansion Project required the help of many people at the Äspö HRL. Demands were made on their time and the project for a process, which was unfamiliar too many. In the end, the project was successful as the water pressures were controlled using the modified grouting methodology. This was the first time that SKB applied the Observational Method as a formal design procedure using Eurocode 7. This successful application of the Observational Method would not have been possible without the help of the project management for the Expansion Project and the contractor Strabag AB, as well as the Äspö HRL staff that provided coordination and services to this project.

Stockholm, 11 December 2013

Rolf Christiansson
Project Manager

Summary

The Final Repository for Spent Fuel must demonstrate that the methodologies and technologies necessary to construct the KBS-3 design are appropriate for the underground environment. To support the development of the methodologies and technologies necessary to construct the Final Repository for Spent Fuel, experiments will be carried out at Äspö Hard Rock Laboratory (HRL). These experiments required construction of two main tunnels and several large niches at the lower levels of the Äspö HRL (410–450 m level). The construction of these tunnels is referred to as the Äspö Expansion Project.

The overarching design requirement for the Äspö Expansion Project was that the long-term reduction in the existing hydraulic heads measured over the future tunnels would be less than 50 m at the end of construction. Based on past experience at the Äspö HRL, grouting was shown to be a cost effective solution for limiting water inflows to tunnels. In the Äspö Expansion Project, grouting was proposed as the primary means to meet the design objective of limiting the water drawdown.

This report describes how the Observational Method was implemented in the design and construction of the TASU and TASP tunnels, as part of the Äspö Expansion Project. The inflow tests that were carried out in the pilot holes before constructions started provided the data necessary for predicting the maximum inflow that would meet the drawdown requirement. This data was also used to design the grouting strategy and Stop Criteria. Comparison of the predicted inflows from the grout design with the measured inflows from the pilot holes and response tests demonstrated that there was an acceptable probability that the drawdown would be within the allowable design tolerances. A reference level for the groundwater head was established before construction started and the Hydro-Monitoring System (HMS) was used to monitor the impact of construction on the hydraulic heads. This monitoring system provided accurate information on both short-term and long-term drawdowns that resulted from the different construction activities. Contingency plans were developed as part of the grout design, in the event that the drawdowns exceeded the design predictions.

The risk associated with groundwater drawdown during the construction of the Äspö Expansion Project was identified during the design phase and successfully managed using the Observational Method. The project requirements were achieved and the measured drawdown relative to the accumulated inflow was in line with the predicted behaviour. Six months after the end of construction, the maximum drawdown was approximately 25 m, or 50% of the allowable drawdown. While the project objectives were met, the low-pH grout recipe used to control the inflows was not sufficiently robust for industrial application, and should be improved.

Sammanfattning

Slutförvaret för använt kärnbränsle ska demonstrera, att de metoder och tekniker som är nödvändiga för att genomföra KBS-3-designen är lämpliga. För att stödja utvecklingen av metoder och teknik som behövs för att bygga slutförvaret för använt kärnbränsle, kommer försök att genomföras vid Äspölaboratoriet. För att möta dessa behov utökades Äspölaboratoriets tunnlar 2012. Detta omfattade två huvudtunnlar och flera nischer på de lägre nivåerna i Äspölaboratoriet (410–450 m nivå). Uppförandet av dessa kallas Utbyggnad Äspölaboratoriet.

Det övergripande designkravet på Utbyggnad Äspölaboratoriet var att den långsiktiga minskningen av de rådande hydrauliska trycknivåerna uppmätt över de planerade tunnlarna skulle vara mindre än 50 m. Baserat på tidigare erfarenheter vid Äspölaboratoriet, har injektering visat sig vara en kostnadseffektiv lösning för att begränsa vatteninflöden till tunnlar. I Utbyggnad Äspölaboratoriet föreslogs injektering som den främsta åtgärden för att möta designmålet att begränsa grundvattensänkning.

Denna rapport beskriver Observationsmetodens tillämpning i designen och byggande av TASU- och TASP-tunnlarna. Inflödestester som utfördes i pilothålen, innan berguttaget påbörjades gav nödvändig data för att prognostisera det maximala inflödet, som skulle uppfylla kravet på avsänkning. Dessa data användes också för att utforma injekteringsstrategi och stoppkriterier. Jämförelse av de prognostiserade inflödena från injekteringsdesignen med de uppmätta inflödena från pilothålen och responstester visade, att det fanns en godtagbar sannolikhet att avsänkningen skulle ligga inom tillåtna designtoleranser. En referensnivå för grundvattentrycket fastställdes innan berguttaget påbörjades, och Hydro-Monitoring System (HMS) användes för att registrera förändringar i de hydrauliska trycknivåerna på grund av tunnelarbetena. Övervakningssystemet gav korrekt information om både kortsiktiga och långsiktiga avsänkningar till följd av olika byggaktiviteter. Beredskapsplaner utvecklades som en del av injekteringsdesignen, i händelse av att avsänkningen överskred designförutsättningarna.

Risken förknippad med grundvattenavsänkning under genomförandet av Utbyggnad Äspölaboratoriet identifierades under designfasen och hanterades framgångsrikt med tillämpning av Observationsmetoden. Projektets krav uppnåddes, och den uppmätta avsänkningen relativt det ackumulerade inflödet var i linje med det förväntade beteendet. Cirka 6 månader efter avslutade bergarbeten var avsänkningen ca 25 m eller 50% av den tillåtna avsänkningen. Medan projektmålen uppfylldes, var inte receptet med låg- pH injekteringsbruk tillräckligt robust för industriell tillämpning och bör förbättras.

Contents

1 Äspö Expansion Project

1.1 Background

The Final Repository for Spent Fuel must demonstrate that the methodologies and technologies necessary to construct the KBS-3 design are appropriate for the underground environment. To support the development of the methodologies and technologies necessary to construct the Final Repository for Spent Fuel, experiments are planned at Äspö Hard Rock Laboratory (HRL) for:

a. Full scale test of a low pH concrete plug.

b. Demonstration of the KBS-3H project.

c. Testing means and methods for detailed investigations (DETUM).

d. Testing of interaction "Concrete and Clay" for the low and intermediate waste programme.

e. Projects within the research foundation Nova and SKB International.

These experiments required construction of three tunnels and several large niches at the lower levels of the Äspö HRL. The construction of these underground openings is referred to as the Äspö Expansion Project. The area for test of the low pH concrete plug was chosen to be at the 450 m level in the expansion of the tunnel TASJ (Figure 1-1). The remaining experimental programs will be located at the 410 m level, in the new tunnels TASP and TASU and various niches of those tunnels (Figure 1-1). This report only addresses the design and construction of the TASP and TASU tunnels on the 410 Level.

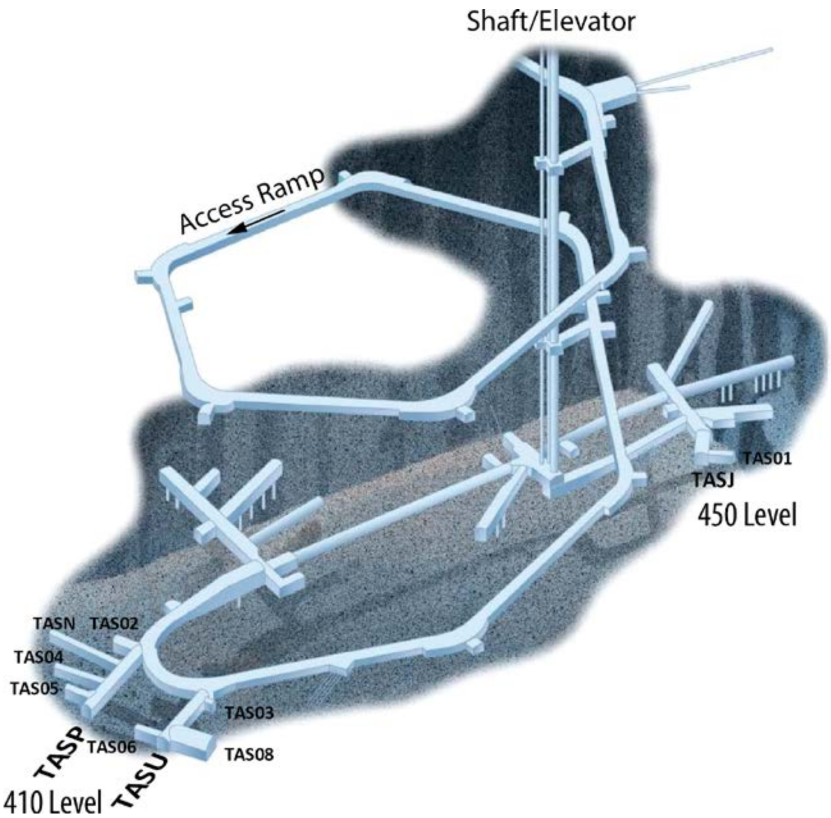


Figure 1-1. *General location of the Äspö Expansion Project. The location of the TASU and TASP tunnels is referred to as the 410 Level and the TASJ tunnel is located at the 450 Level.*

1.2 Design constraints

The experiments planned for the Äspö Expansion Project at the 410 m level (see Figure 1-1) had the following specific requirements for the host rock:

- **Concrete and Clay Project required a dry rock volume that did not require grouting:** The Concrete and Clay Project planned to study the interaction between concrete and bentonite in long-term borehole experiments. Hence there was a design requirement that there shall be no contamination of other cement material. Consequently grouting was not allowed during niche construction. This implies that the niche for the Concrete and Clay experiment would need to be located in a relatively dry rock volume that did not require grouting.
- **A maximum water pressure loss to the 410 Level equivalent to 50 m:** The KBS-3H project specified that the far-field water pressure above their niche must be similar to that expected at repository depth. This implied that the inflows to the KBS-3H niche must be kept as low as practical to maintain the high water pressure around the niche. In general, the Hydro Monitoring System (HMS) data from boreholes in the vicinity of the 450 m level indicated a head around 350 m. This drawdown was caused by the influence of the nearby elevator shaft. However, the HMS data from boreholes towards near the 410 m level indicated a pressure head of approximately 365 m, i.e. no significant drawdown attributed to the shaft. To take advantage of these higher pressures the collar of the new tunnels (TASU and TASP) were located to the 410 m level (Figure 1-2). This location met the hydraulic requirements provided the drawdown at the end of construction of the niche for the KBS-3H project was less than 50 m. It was therefore concluded that the design constraint of the TASU and TASP tunnel and niches would not be the inflow, but a maximum water pressure loss to the 410 Level equivalent to 50 m.

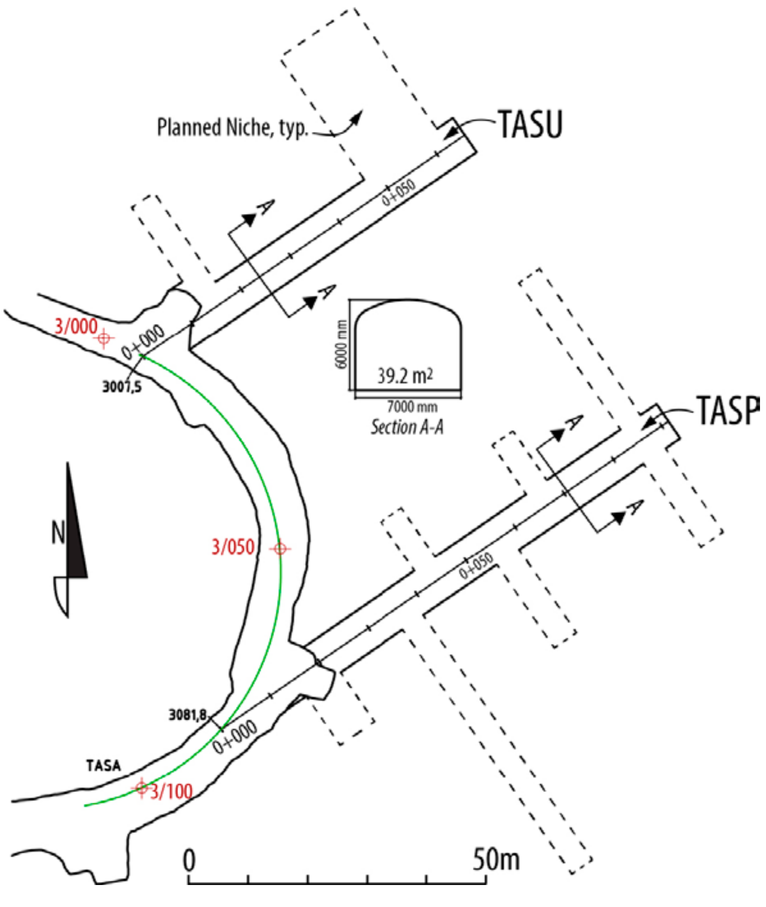


Figure 1-2. Location of the proposed tunnels (TASU and TASP) and the planned location of the Experimental Niches at the time of contract tendering.

1.3 Design and construction approach for the TASU and TASP tunnels

The Äspö Expansion Project like many underground construction projects involve uncertainty and risk. The uncertainty is associated with the actual rock mass conditions, and the risk is related to not being able to meet the design constraints imposed for the TASU and TASP tunnels.

The design and construction of the TASU and TASP tunnels provided an opportunity to:

- Apply the strategies for detailed investigations to reduce the rock mass uncertainty, outlined in the framework programme for detailed investigation (SKB 2010). This includes strategies and methods for characterization based both on borehole information and tunnel mapping.
- Demonstrate the Observational Method as a formal design methodology as outlined in Eurocode 7 Geotechnical Design.
- Apply grouting works as the primary tool for controlling groundwater drawdown.
- Implement Quality Assurance and Quality Control (QA/QC) procedures necessary for the grouting control programme.
- Implement QA/QC procedures necessary for the tunnel excavation control programme.

In most projects the selection of a particular means and method that can fulfil the design constraints is only made after formal engineering evaluation of the various alternatives and options. The selection of grouting as the primary means for controlling groundwater was based on past experience at the Äspö HRL. This past experience was related to grouting as a cost effective solution for limiting water inflows to tunnels. For example extensive grouting trails with silica sol and cement-based grouts were successfully used to limit the inflows to the TASS tunnel at the 450 Level (Funehag and Emmelin 2011). Those trials were primarily used as a research demonstration and the procedure and grout mix would have to be modified for implementation in the Äspö Expansion Project to meet the constraints of the construction schedule. In addition, there was no monitoring of the hydraulic heads on the previous grouting trials to link the inflows and pressure heads.

1.4 Objectives of this report

Eurocode 7 states that Geotechnical Design can be carried out using four general approaches:

1. Design using calculations.
2. Design based on prescriptive measures.
3. Load tests and tests on experimental models.
4. Observational Method.

A design strategy is normally chosen based on the complexity of the problem and associated risks. The Observational Method has been developed over the past 50 years and is appropriate when the prediction of geotechnical conditions/behaviour is uncertain. One usually chooses the Observational Method because the construction solution provided by "Design by calculation", results in prohibitive construction costs. The design based on the Observational Method, provides the formal process during construction for revising the means and methods necessary to meet the design objectives using cost effective solutions. The application of the Observational Method in the project design also provides a formal means of forecasting expected values and evaluating those values against the encountered conditions.

The objective of this report is to illustrate how the Observational Method was used in the Äspö Expansion Project to meet the design requirements of limiting the maximum water pressure loss to the 410 Level equivalents to 50 m.

2 Rock mass conditions at the 410 Level

2.1 Sources of information

The primary source of information for this section is Johansson et al. (2014).

The rock mass conditions for the proposed tunnels were obtained from the pilot holes KA3011A01 (TASU) and KA3065A01 (TASP) (Figure 2-1). In addition to the pilot holes there are a number of additional holes that existed prior to the drilling of the pilot holes (Figure 2-1). These holes provided the bases for the geological model.

2.2 Rock type, fractures and groundwater flow

The main rock types encountered in the pilot holes were provided in Johansson et al. (2014) and are summarized as:

TASP : Between 0 and about 47 m the dominate rock type is Ävrö granodiorite.
Beyond 47 m the dominate rock type is Äspö diorite.

TASU: Between 0 and about 72 m the dominate rock type is Ävrö granodiorite.

2.3 Open fractures

The dominant steeply dipping open fractures at Äspö tend to strike NW-SE. This is also true for the open fractures observed in the pilot holes (Figure 2-2). There are two other dominant fracture sets at the HRL; one steeply dipping striking NNE – SSW (Rhén et al. 1997) and one gentle dipping. Both these sets are almost parallel to the pilot holes. However, they are quite tight and have limited importance to the hydraulic conditions at site.

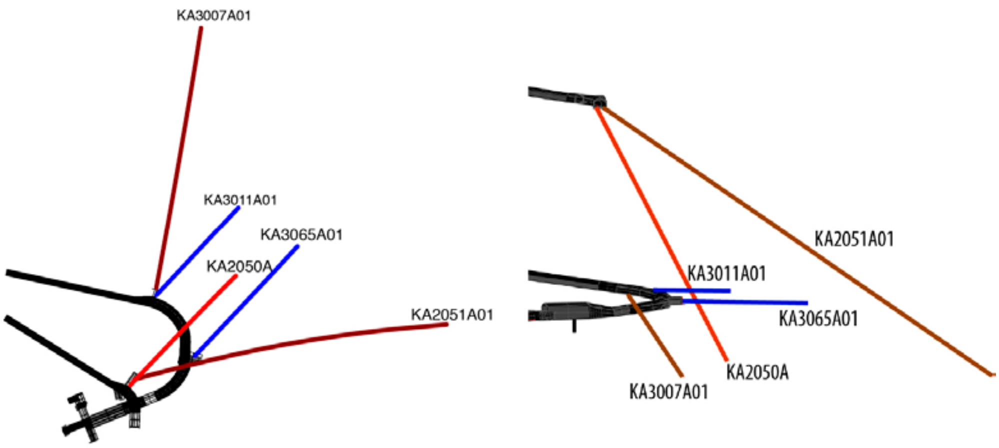


Figure 2-1. *Illustration (plan and section) of the location of existing boreholes KA3007A01, KA2051A01 and KA2050A, and the pilot holes KA3011A01 (TASU) and KA3065A01 (TASP). Note that KA2050A contains piezometers and penetrates the pillar separating TASU and TASP.*

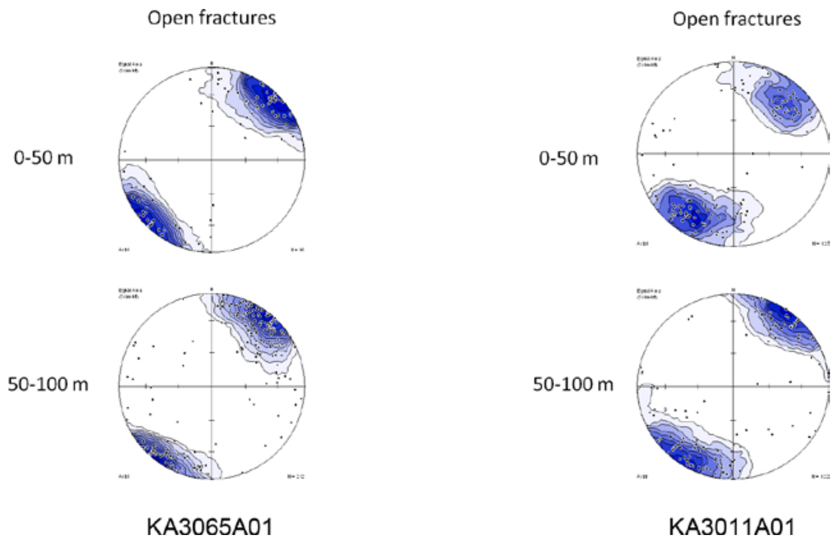


Figure 2-2. *Open fractures measured in KA3065A01 (TASP) and KA3011A01 (TASU).*

The single-hole interpretation of the drill core suggested that a possible deformation zone (DZ1) was identified in KA3011A01 between section 14–35 m. The core (central portion) of the deformation zone is located between 18.75 and 20.10 m surrounded by a transition zone ("damage zone") on both sides. The orientation of this deformation zone is similar to the general trend of the steeply dipping open fractures which are shown in Figure 2-3. Based on this information the deformation zone is interpreted to have a NW-SE-to WNW-ESE orientation and be an extension of DZ1 in KA3007A01. Hydraulic information supports that DZ1 in KA3007A01 and the interpreted DZ1 in KA3011A01 are part of the same structure.

No deformation zone was identified in KA3065A01. However, the DZ1 identified in KA3011A01 would intersect KA3065A01 in the range of 35–45 m.

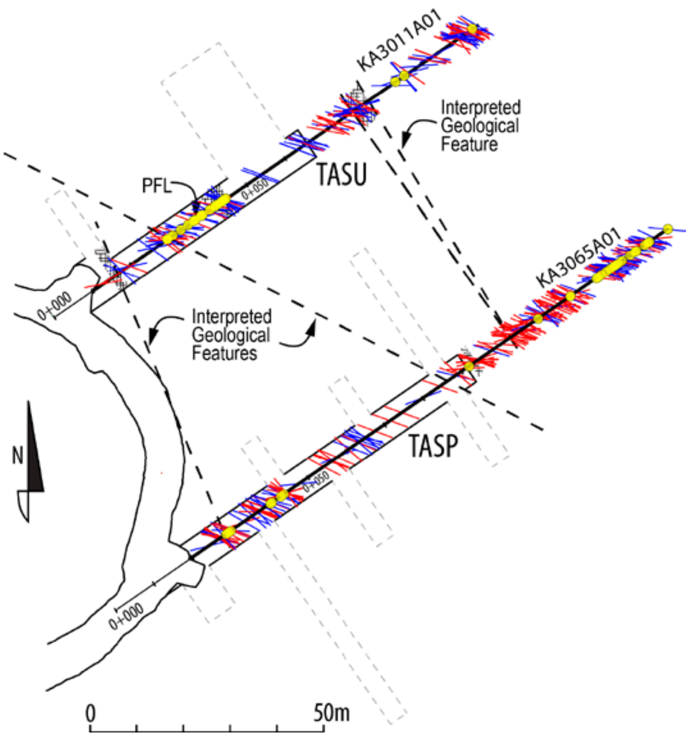


Figure 2-3. *The orientation of the open fractures in the general NW-SE direction and the PFL locations (yellow circles). Also shown is the interpretation of possible geological features.*

The measured inflows during drilling and the calculated transmissivity values assessed from the Posiva Flow Logger (PFL) data along KA3011A01 and KA3065A01 are summarized in Figure 2-4. Additional values from boreholes in the surrounding rock volume are provided in Appendix A (Borehole hydraulic tests).

A number of response and interference tests were performed in the boreholes surrounding the proposed tunnels and the pilot drill holes in order to try to understand the hydraulic connectivity. The tests were performed over the most transmissive portions of the drill holes. A significant hydraulic connection was observed between about 0 and 40 m and from about 80 m to the end of the borehole in KA3011A01 and KA3065A01. The area between borehole depths of 40 and 80 m in these two pilot holes shows poor hydraulic connectivity.

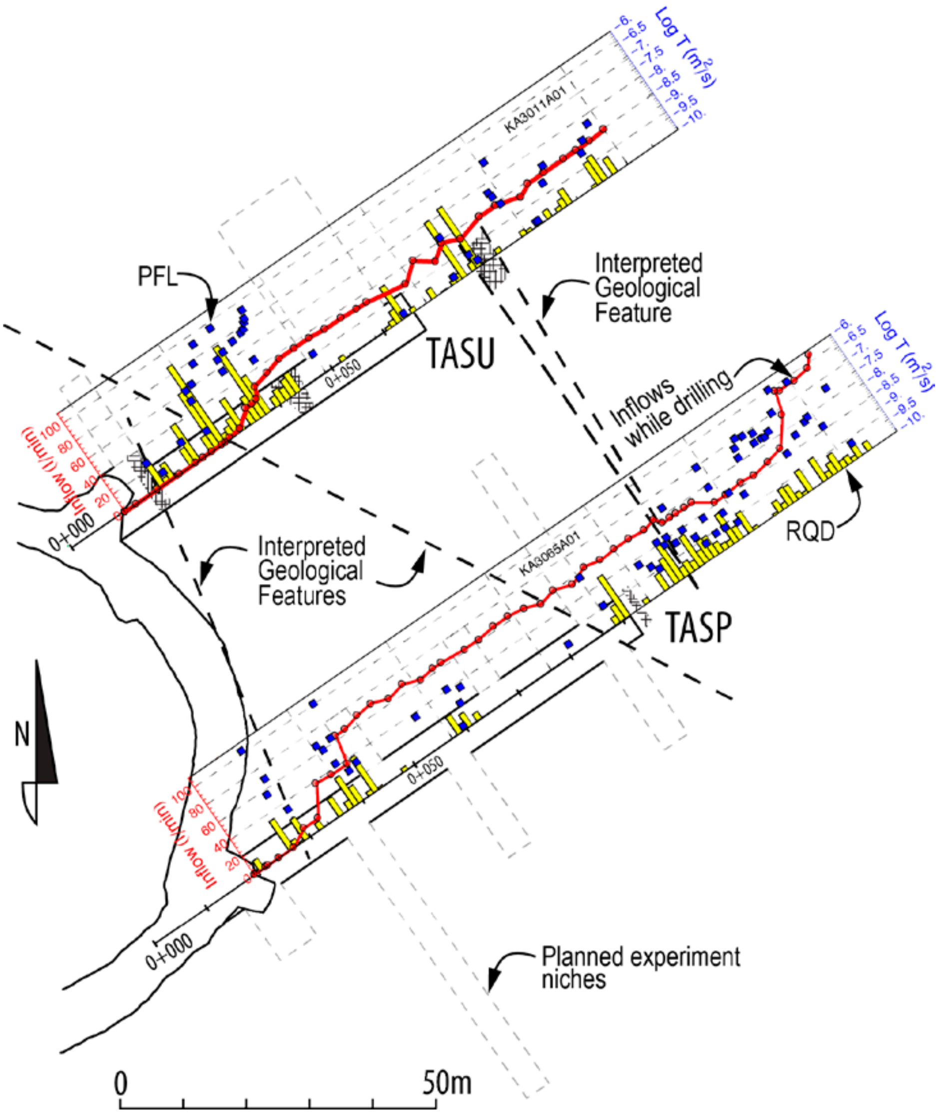


__Figure 2-4.__ Comparison of inflows (l/min) measured while drilling, the transmissivity measured using the Posiva Flow Logger (PFL), the Rock Quality Designation (RQD) plotted as 100-RQD and the interpreted geology. In order to obtain the corresponding tunnel length along TASU and TASP, 9.83 m is added to the length of borehole KA3011A01 and 18.99 m to KA3065A01.

2.4 Groundwater and monitoring

Äspö has an extensive hydro monitoring system (HMS) that has been in place since the facility was completed. The system monitors the ground water pressures in boreholes using packer-piezometers and datalogging technology. The system can be adjusted for any frequency of readings and produces standard plots of pressure and time for any selected borehole and monitoring interval (piezometer).

Groundwater pressures in the vicinity of the tunnels are monitored using piezometers installed in the boreholes given in Table 2-1. The pressures along the pilot holes KA3011A01 and KA3065A01 range from about 3900 kPa away from the Äspö main tunnel (TASA) to about 3600 kPa where the borehole intersects TASA.

Borehole KA2050A was drilled from the Äspö ramp above the Expansion. It penetrates the middle of the pillar separating TASP and TASU and contains 3 piezometric monitoring sections. KA2050A is 211 m long and encountered very large inflows totalling approximately 500 l/min during drilling. At borehole depths between 123–130 m and 178–185 m inflows of 160 l/min and 250 l/min were recorded, respectively. Injection of 2.5 m^3 of grout between 120–211 m and 2.3 m^3 between 118–211 m was carried out in an attempt to reduce the inflows.

During the drilling of the pilot holes KA3065A01 and KA3011A01, the piezometers in KA2050A gave the largest response. It was decided based on an evaluation of all the piezometers in Table 2-1, that the piezometers in KA2050A would be the primary instrumentation for monitoring the drawdown response during the driving of the TASP and TASU tunnels and niches.

Table 2-1. Water pressures in the monitoring piezometers in the vicinity of TASU and TASP prior to tunnel construction.

Borehole	Number of monitoring piezometers	Borehole length (m)	Water pressure (kPa)
KA2048B	4	184.45	1310–1790
KA2050A	3	211.57	2515–2650
KA2051A01	10	319.84	2135–2630
KA2858A	1	40.77	3155
KA2862A	1	15.98	3050
KA3005A	4	50.03	3330–3020
KA3010A	1	15.06	3605
KA3011A01	10	100	3625–3890
KA3065A01	5	125	3660–3900
KA3065A02	4	69.95	3580–3715
KA3065A03	1	11.80	3688
KA3067A	4	40.05	3716–3740
KA3068A	1	16.85	3700
KA3105A	5	68.95	3090–3630
KA3110A	2	26.83	3070–3130
KXTT1	3	28.76	2720–3420
KXTT2	4	18.30	2560–3330
KXTT5	1	9.81	3320

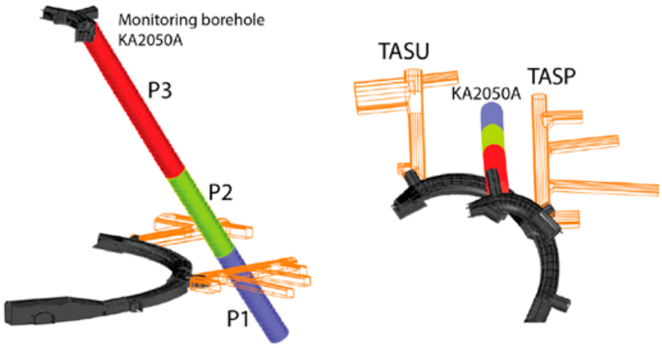


Figure 2-5. *Location of the hydraulic monitoring borehole KA2050A, showing the piezometers P1, P2 and P3.*

3 Geotechnical design and the Observational Method

3.1 Background

According to Nicholson et al. (1999), *"the Observational Method in ground engineering is a continuous, managed, integrated, process of design, construction control, monitoring and review which enables previously defined modifications to be incorporated during or after construction as appropriate. All these aspects have to be demonstrably robust. The objective is to achieve greater overall economy without compromising safety."*

While the Observational Method was first proposed for managing the risks associated with uncertain ground conditions (Peck 1969), the principles of the Observational Method can be applied to many complex projects as a formal methodology for managing project risks (Le Masurier et al. 2006). The essential steps that should be carried out at the early stages of design are: (1) Identify all potential failure modes for each major component of the project/system, (2) conduct sufficient analyses to establish the uncertainties and risk with each failure mode, and (3) establish a risk registry for the project to document the identified risks and the proposed design solution to mitigate the risk to acceptable tolerances. As the project proceeds many of the risks identified in the risk registry will be managed using approaches specified in EuroCode SS-EN 1997-1:2005. However, when the uncertainties and risks cannot be mitigated to acceptable levels the Observational Method should be specified.

While the Observational Method is a risk reduction tool for projects involving challenging ground conditions; the method itself is not without its own risks. There have been well-documented case histories that have demonstrated the failure of the Observational Method because of inadequate and inappropriate project management (HSE 2000). In this section the relationship between the Observational Method and Geotechnical Design is summarized from the Eurocode guidelines. Implementing the OM takes serious effort and as noted by Peck (1985), if not careful, the OM can easily be discredited by misuse. Peck (1985) warned that:

"The Observational Method, surely one of the most powerful weapons in our arsenal, is becoming discredited by misuse. Too often it is invoked by name but not by deed. Simply adopting a course of action and observing the consequences is not the Observational Method as it should be understood in applied soil mechanics. Among the essential but often overlooked elements are to make the most thorough subsurface investigations that are practicable, to establish the course of action on the basis of the most probable set of circumstances and to formulate, in advance, the actions to be taken if less favourable or even the most unfavourable conditions are actually encountered. These elements are often difficult to achieve but the omission of anyone of them reduces the Observational Method to an excuse for shoddy exploration or design to dependence on good luck instead of good design. Unhappily, there are far too many instances in which poor design is disguised as the state of the art merely by characterising it as an application of the Observational Method".

3.2 Geotechnical design process

This section is an extract from Eurocode SS-EN 1997-1:2005, Eurocode 7: Geotechnical design – Part 1: general rules, with focus on underground construction (tunnels and caverns).

Eurocode 7 stipulates that for each geotechnical design situation it shall be verified that no relevant limit state is exceeded. When defining the design situations and the limit states, the following factors should be considered for ground conditions:

- Site conditions with respect to overall stability.
- Nature and size of the structure, including any special requirements such as the design life.
- Conditions with regard to the structure's surroundings.
- Regional seismicity.
- Influence of the environment.

Limit states can occur either in the ground or in the structure or by combined failure in the structure and the ground. Limit states should be verified by one more design methods using:

1. Adoption of prescriptive measures.
2. Use of calculations.
3. An observational method.

An overview of the three different design methods is presented in Figure 3-1. Preliminary analysis of the initial problem will lead to the selection of the appropriate design method. Regardless of the design method selected, the limit states for the specific design requirements and restrictions must be assessed. A suggested flow chart for the problem analysis is given in Figure 3-2. A brief description of the three design methods follows.

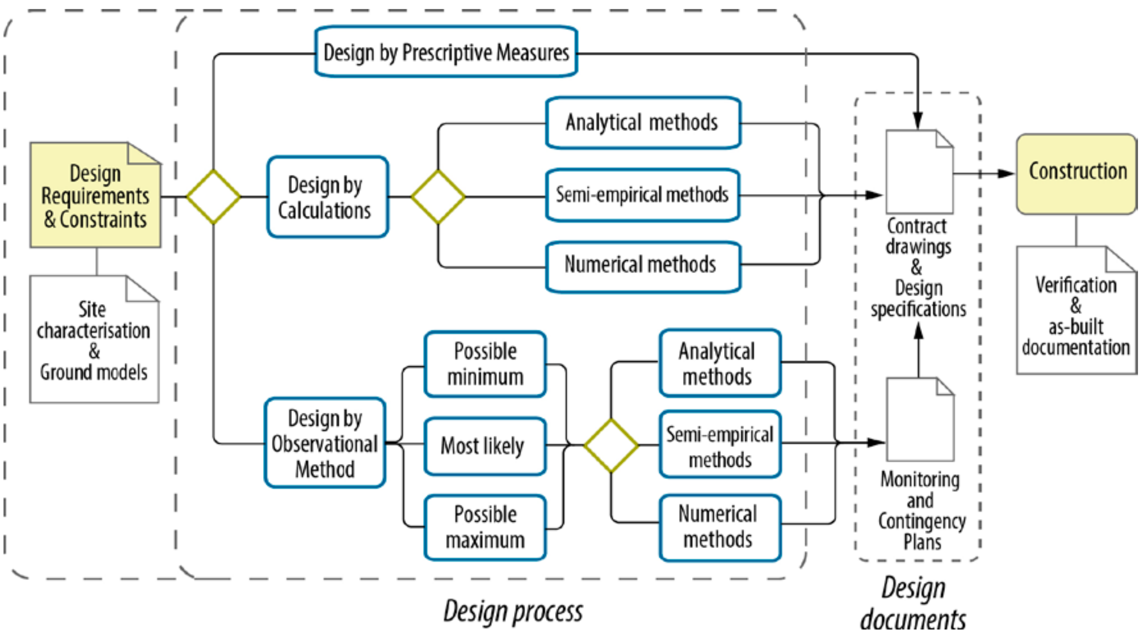


Figure 3-1. *Process chart for the design process with the different methods to verify the limit state.*

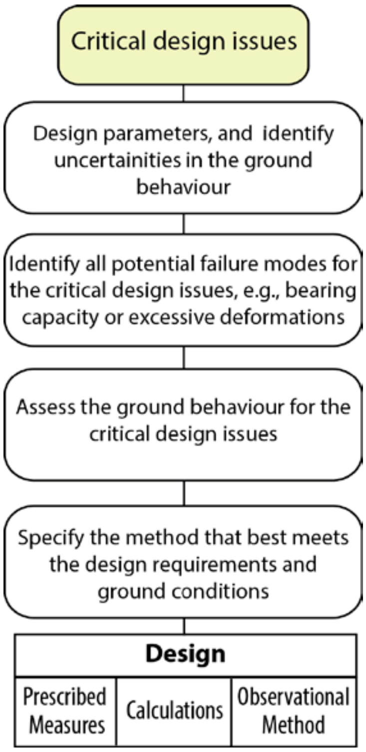


Figure 3-2. *Schematic flow chart for choice of design method (modified from Holmberg and Stille 2007).*

3.2.1 Design by prescriptive measures

In design situations where calculation models are not available or not necessary, exceeding limit states may be avoided by the use of prescriptive measures. These involve conventional and generally conservative rules in the design, and attention to specification and control of materials, workmanship, protection and maintenance procedures. Design by prescriptive measures may be used where comparable experience makes design calculations unnecessary. Prescriptive measures are defined as: "*Documented or other clearly established information related to the ground being considered in design, involving the same types of rock and for which similar geotechnical behaviour is expected, and involving similar structures. Information gained locally is considered to be particularly relevant*."

3.2.2 Design by calculations

While much of the design can be completed using prescriptive methods and/or Observational Method, in some situations the design will need to be verified by calculations. Design by calculation involves:

- Actions, which may be either imposed loads or imposed displacements, e.g. from ground movements.
- Properties of soil, rocks and other material.
- Geometrical data.
- Limiting values of deformations, crack widths, vibrations etc.
- Calculation models.

The calculation model shall describe the assumed behaviour of the ground for the limit state under consideration. The calculation method can be analytical, semi-empirical or numerical, depending on the design issue.

3.2.3 Design by Observational Method

In all underground design and construction, uncertainties with regard to site conditions must be anticipated. The uncertainties that will influence the final layout are the spatial location and the variability of the geological setting and the rock mass response to excavation, rock support and grouting measures. The scope of the design tasks will be primarily limited to the following five requirements of the Observational Method stated in Eurocode SS-EN 1997-1:2005, Section 2.7:

1. Acceptable limits of behaviour shall be established.
2. The range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits.
3. A plan for monitoring the behaviour shall be devised, which will reveal whether the actual behaviour lies within the acceptable limits. The monitoring shall make this clear at a sufficiently early stage, and with sufficiently short intervals to allow contingency actions to be undertaken successfully.
4. The response time of the monitoring and the procedures for analysing the results shall be sufficiently rapid in relation to the possible evolution of the system.
5. A plan of contingency actions shall be devised which may be adopted if the monitoring reveals behaviour outside acceptable limits.

The Observational Method has several caveats. All five requirements must be met before construction is started. The most likely design situation as well as the extremes (minimum and maximum) has to be analysed. The calculation method has to be chosen for each one of these design situations, see Figure 3-1. One must be able to provide a technical solution or define an action plan for every design situation or possible adverse condition based on current site understanding. The method cannot be used if a predictive model for the behaviour cannot be developed, i.e. one must be able to establish a model that can estimate the parameters that will subsequently be monitored during construction. This is not a trivial problem as often we can measure what we cannot calculate and vice versa. This means that the monitoring plan must be chosen very carefully with a good understanding of the significance to the problem.

During construction, the monitoring shall be carried out as planned. The results of the monitoring shall be assessed at appropriate stages and the planned contingency actions shall be put into operation if the limits of behaviour are exceeded. Monitoring equipment shall either be replaced or extended if it fails to supply reliable data of appropriate type or in sufficient quantity.

3.3 Summary

In the previous section the background on the development of the Observational Method and its use in Eurocode 7: Geotechnical Design was briefly reviewed. In the following sections the application of the Observational Method to the Äspö Expansion Project is described.

4 Observational Method: implementation

4.1 Preliminary design

The design started with the evaluation of existing piezometric data in the Äspö HRL to establish the pressure heads that existed prior to construction. A volume located NE of the ramp was identified with minimum undisturbed groundwater conditions, i.e. maximum pressure heads. Two pre-investigation boreholes were core-drilled to characterise this area and establish the relationship between pressure heads on borehole inflows (see Section 2). Based on those results a preliminary design was developed with proposed locations for TASP and TASU, and preliminary locations for the experimental niches. Two pilot holes were then core-drilled within the contours of TASP and TASU. These holes confirmed the location of TASU and TASP as well as provided input data for the grouting design and rock support. The locations of the experimental niches were to be adjusted based on the information collected while excavating TASP and TASU.

Preliminary predictions of the drawdown of the groundwater head, due to rock excavation, were an essential part of the preliminary design. These predictions of the groundwater responses were based on the observed inflows during pilot-hole drilling. The grouting strategy was developed based on the use of cement grouts and on the inflow measurements that was done during pilot hole drilling. The PFL transmissivities were not available at the time the grouting design was carried out. The design analyses showed that using cement grout would be sufficient to maintain the groundwater head within given restrictions, but the project would not deliver dry tunnels. The scope of the grouting design is presented in Appendix B.

The bulk of the preliminary design covered the standard contract templates for civil works, technical specifications with drawings and site-specific constraints for the works in the HRL, such as limitations in blasting times, restrictions on vibrations etc. Special effort was focused on the grouting design with respect to maintaining the hydraulic head (Section 1.2). The technical specifications covered:

- Requirements on drilling.
- Prescribed grout.
- Strategy for pre-grouting.
- Stop criteria.
- Requirements on the contractor's documentation.
- Quality control.

4.2 Project Control

Figure 4-1 illustrates the flow of information that is used to determine if the grouting results are acceptable. The Design Engineer evaluates the data and decides if the results meet the design criteria. Members of the Reference Group are available for helping with data review. The Reference Group meet as necessary to review the projects progress. The following describe the monitoring programme and the grouting control programme.

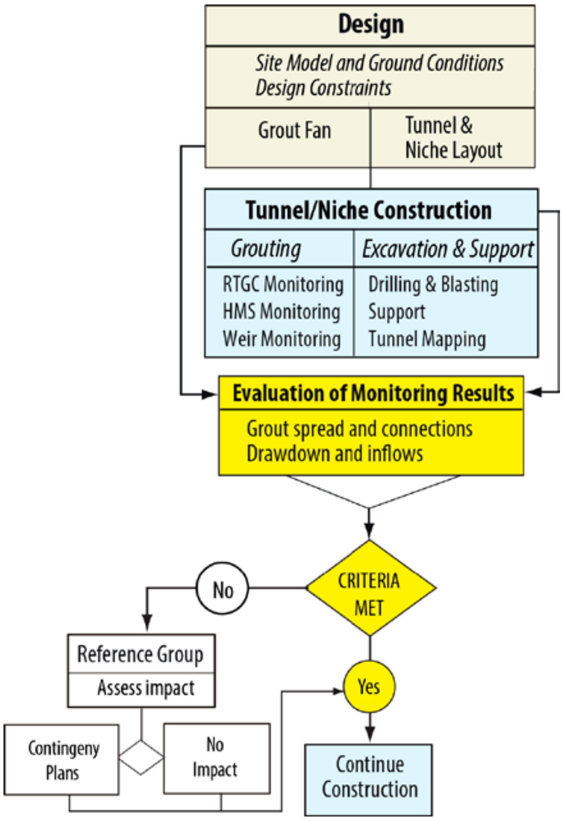


Figure 4-1. *Illustration of the decision making process used to evaluate the results after each grout fan and each tunnel excavation round.*

The design approach described in Appendix B was implemented using:

- Real time Grout Control (RTGC).
  The RTGC method can be used for verifying, planning and controlling the grouting process. A prerequisite is that the rheological properties and the penetrability of the grout mix have been measured in the laboratory (smallest fracture to be penetrated). The primary application of the RTGC in this project was to verify the stop criterion. This contains the following steps (Stille 2012):
  1. Volume, flow and time is measured during grouting.
  2. Dimensionality of the grout flow is estimated (2D radial flow, 1D channel flow).
  3. The aperture of largest grouted fracture are estimated.
  4. The grout propagation is calculated in the largest and smallest fracture the grout can penetrate.
  5. The grout propagation is checked with the targets values and if necessary the stop criterion are modified.
- The Äspö HRL maintains a Hydro Monitoring System (HMS) at the site. This includes multiple packer installations in boreholes with pressure transducers connected to data loggers in the packed-off sections. This provides the possibility to follow on-line hydraulic response to disturbances such as tunnelling works.
- Weirs to measure inflow of water were constructed at the collar of the TASU and TASP tunnels early during the construction works. However, it is well known that there are large uncertainties related to measure water inflow in weirs during tunnelling due to the large volumes of water used for drilling etc. The project only measured inflow on Monday mornings.

The method of analyses, the design calculations and the stop criteria for implementing the Observational Method are presented in Appendix B. The overall grouting strategy that was formulated based on these design analyses is summarised in Figure 4-2. In the following sections some of the results from this analyses are repeated to clarify how and when they are applied to the principles of the Observational Method.

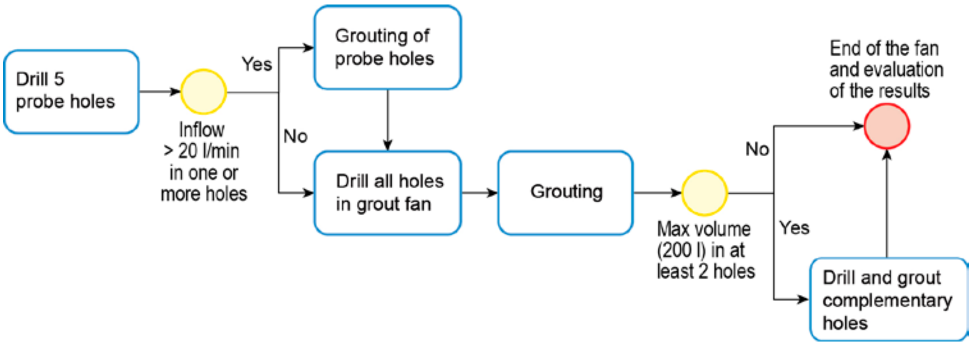


Figure 4-2. *Overall grouting strategy.*

4.3 Acceptable limits of behaviour

The acceptable limit of behaviour is defined by the overall project requirement, i.e. to maintain high hydraulic head in the vicinity of TASP and TASU. The maximum allowable drawdown was set to 50 m relative to a reference level in the monitoring hole KA2050A.

4.4 Range of possible behaviour

Assessments of the most likely hydraulic conditions of the rock mass before grouting and the expected inflow after grouting are shown in Table 4-1 for TASU and TASP, respectively.

Table 4-1. Assessed transmissivity and maximum hydraulic aperture before grouting are relative to the given tunnel sections. Calculated inflow to the TASU and TASP tunnel relate to conditions after grouting.

Tunnel Section	Before Grouting		After Grouting
TASU [m]	Transmissivity [m²/s]	Hydraulic aperture [μm]	Accumulated inflow [l/min]
10–20	0	0	0
20–21	4×10^{-9}	18	0
21–24	9×10^{-6}	230	2
24–26	1×10^{-8}	26	2
26–29	0	0	2
29–30	3×10^{-8}	33	3
30–31	3×10^{-8}	35	4
31–33	6×10^{-8}	44	6
33–36	4.3×10^{-7}	95	9
36–38	8×10^{-8}	48	11
38–39	5.6×10^{-7}	91	14
39–42	2.1×10^{-7}	66	16
42–45	2.7×10^{-9}	71	19
45–69	0	0	19
20–31	4.3×10^{-7}	83	3
31–34	0	0	3
34–37	1.4×10^{-6}	123	6
37–43	0	0	6
43–44	1.5×10^{-6}	125	9
44–46	0	0	9
46–49	3×10^{-7}	73	11
49–52	2×10^{-7}	60	14
52–55	0	0	14
55–58	1×10^{-7}	57	16
58–61	0	0	16
61–64	7×10^{-8}	45	18
64–73	0	0	18

The assessed inflow after grouting to TASU and TASP was 19 l/min and 18 l/min, respectively. However, this was considered as a conservative judgement based on that previous experiences in similar hydrogeological conditions indicate that the assessed inflows after grouting are conservative. Inflows may in reality be smaller due to that the calculation model does not consider that fractures may interact through a network and that sealing of a water bearing fracture may close off or reduce supply of water to other fractures in such a network.

The typical layout for a grout fan is illustrated in Figure 4-3. The planned locations of each individual grout fans and the expected grout takes are summarised in Table 4-2. The expected grout take for TASU and TASP tunnels were about 1900 l and 900 l, respectively. However, this was considered as an optimistic judgement and in practice the occurrence of larger grout takes could not be excluded due to the following reasons. There is uncertainty about the fracture frequency. The statistical approach using lognormal distribution of the transmissivity do not exclude that fractures with larger aperture may be encountered. Both research and practical tests have pointed out that the physical fracture aperture is likely to be larger than the hydraulic aperture back-calculated from water loss measurements, see e.g. Holmberg et al. (2012). In the event that the total transmissivity is higher than expected, the total grout takes increases. The assessed grout take will increase by about 100% in TASU and 50% in TASP if the total transmissivity over the tunnel lengths becomes twice the expected.

Table 4-2. Predicted grout takes in TASU and TASP.

Grout fan TASU Sections [m]	Grout take TASU [l]	Grout fan TASP Sections [m]	Grout take TASP [l]
10–30	1750	20–40	740
25–45	80	35–55	60
40–60	0	50–70	30
55–70	60	65–85	60

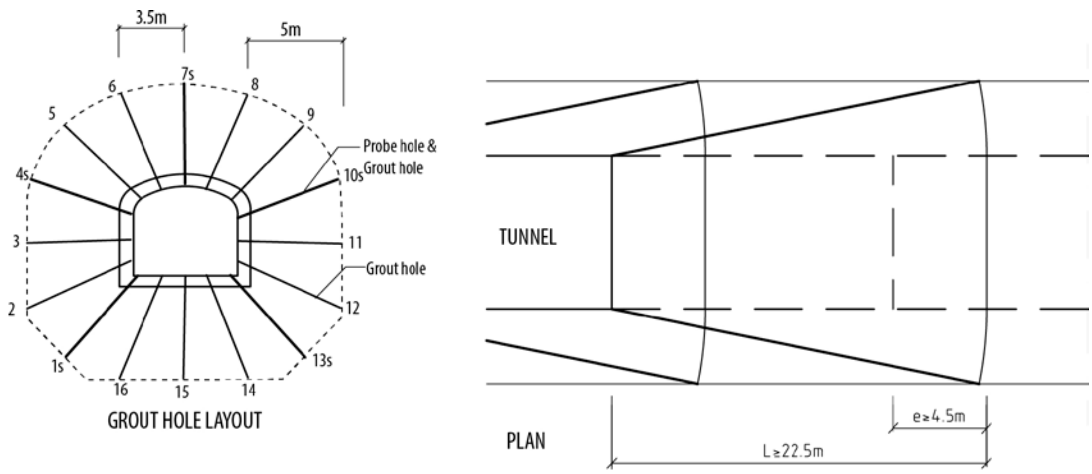


Figure 4-3. Layout of the probe holes and grout holes used for the grouting operation, section and plan view.

The following assumptions were the bases for assessing the grout take.

- The spatial distribution of water bearing fractures and their apertures were evaluated from results of inflow measurements carried out in sections that were 3 m long in average, see Appendix A and B. It was assumed that the back-calculated transmissivity are representative for the behaviour of a single water bearing fracture.
- The grout take depend on the number of representative fractures that intersects the grout holes and their hydraulic properties. The hydrogeological model indicated that the orientations of the water bearing fractures were approximately perpendicular to the tunnel alignment, see Figure 2-3. It was therefore assumed that grout holes would intercept all identified water bearing fractures.
- The grout spreads radially from the borehole along the fracture planes.
- The effective grout pressure and grouting time are in accordance with the stop criteria shown in Appendix B.
- The experience from Swedish rock conditions and tunnels located in good rock quality is that about 30% of the grout holes will essentially have no grout takes. Hence the grout volume in such holes is equal or less of that required for hole filling.

The reason why grout holes have no grout take may be one or more of the following.

- In reality all grout holes do not intercept water bearing fractures or water bearing sections in the fracture.
- The fracture aperture is less than the critical aperture required for grout penetration.
- Fractures may already be sealed by grouting carried out in adjacent holes.

Each grout fan was assumed to cover 15 m distance of the tunnel, i.e. 20-m-long grout holes with a 5 m overlap between grout fans. The expected spread of grout varies between 4 m and 14 m, depending on the hydraulic aperture for the representative water-bearing fracture.

4.5 Acceptable probability that actual behaviour is within the acceptable behaviour

During the drilling of the pilot holes KA3065A01 and KA3011A01, inflows were recorded and drawdowns were monitored with piezometers in three packer intervals in the nearby monitoring hole KA2050A, see Figure 2-5. The drawdown was recorded during drilling of each pilot hole. In addition, drawdown tests were also carried out after the pilot holes were completed.

Figure 4-4 shows for various flow rates in the pilot holes the short term drawdowns for the piezometer that gave the greatest response in borehole KA2050A. The relationship between inflow and drawdowns were established for inflow rates of approximately 40 l/min in KA3011A01 and between 60 to 70 l/min in KA3065A01. Since the tests were performed in one pilot hole at the time and were short-term tests, there is uncertainty relative to the long-term drawdown response that would occur after both tunnels were excavated.

Based on the short-term inflow tests it was determined that the allowable drawdown of 50 m would not be exceeded if the total inflows occurring in both tunnels after grouting were less than 40–50 l/min. The predicted inflow to both tunnels after grouting was assessed to be around 40 l/min. As this assumption was judged to be conservative the likelihood of that the inflow would impede on the drawdown requirement was considered to be small. Hence, the probability that the actual behaviour would be within the limits established for acceptable behaviour was acceptable.

4.6 Monitoring plan

The Hydro-Monitoring System (HMS) provided real time data on the hydraulic response during excavation of the TASU and TASP tunnels. The results were used to monitor the efficiency of each grout fan as well as the long-term drawdown. In addition, the water inflow was measured in the probe holes that were part of the grouting strategy. Any inflows during the drilling of the remaining holes for the grout fan were only noted and not measured.

Overflow weirs were constructed near the entrance of each tunnel. The water flow was normally recorded after the weekends before construction activities commenced Monday morning.

The grouting equipment was furnished with a data acquisition unit that provided data for carrying out back-analysis of the grouting operations using the RTGC concept.

Hence, the monitoring plan provided the adequate time to respond to any unforeseen ground conditions and to adjust the grouting operation in accordance with the contingency plan.

4.7 Contingency plan

The design requirement defined the acceptable drawdown to be maximum 50 m. As shown in Figure 4-4 considerable drawdown could be expected during the drilling of a grout fan. The inflow tests carried out in the pilot holes also showed that the drawdown quickly recovered after completion of the tests. Hence, some uncertainty prevailed about the relationship between the permanent inflow to the tunnels after grouting and the long-term drawdown. Table 4-3 highlights the contingency measures for various scenarios should the grouting efficiency and the drawdown trend become unacceptable.

There was uncertainty whether the required grouting efficiency could be achieved with one grouting round in the event that water-bearing fractures with relatively large aperture or connected holes were encountered. The inflow to the pilot holes and the geological conditions indicated that these events could occur and the contingency measure was to carry out two grout rounds in accordance with the overall grouting strategy and the standard drill and grout plan. The first grout round consisted of grouting the five probe holes with a thick grout mix and the second grout round were carried out with the regular grout mix and covered the remaining grout holes in the drill plan, see Figure 4-3.

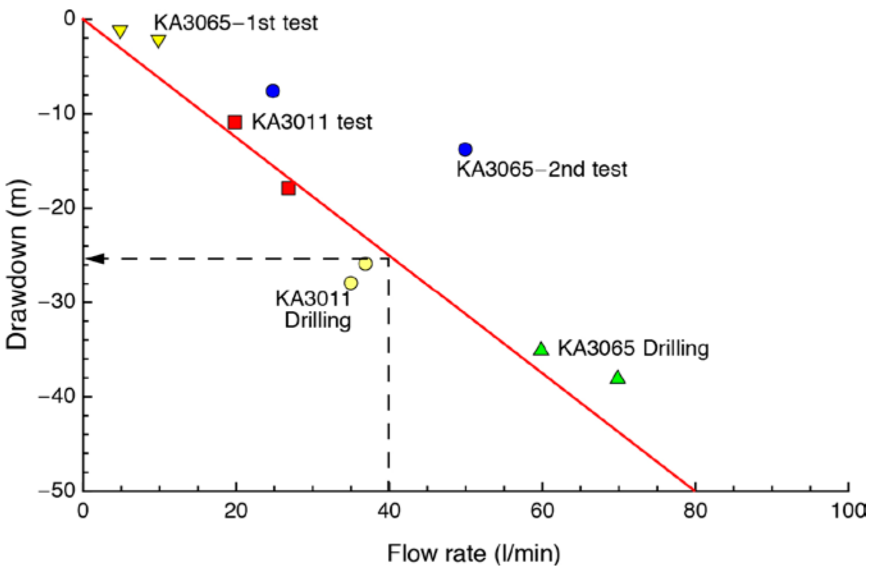


Figure 4-4. *Drawdown in borehole KA2050A versus flow rate based on the data from the drilling of KA3065A01 and KA3011A01 prior to the start of construction.*

Table 4-3. Indicators, monitoring system, likely causes and mitigation measures to be implemented due to occurrence of unacceptable water inflow and/or groundwater drawdown.

Indicator	Monitoring System	Likely cause	Mitigation measure
Inadequate sealing effect	Inflow measurements	Water bearing fracture with large aperture	Execute two grouting rounds in accordance with the specified drill and grout plan
Inadequate sealing effect	HMS shows unacceptable drawdown and/or unacceptable inflow to tunnel is recorded. Geological mapping indicate occurrence of water leakage in fractures after grouting	Water-bearing fractures are connected in a network to a significant extent	Grouting by means of a primary and a secondary grout round
Inadequate sealing effect		Grouted zone is not wide enough. The grout spread is not sufficient	Increase grouting time. Change grout mix to achieve a better penetrability through reduced viscosity
Poor sealing effect in a rock mass with small fracture apertures		High frequency of water bearing fractures with apertures smaller than the critical required for grout penetration	Change grouting method from cement grouting to solution grouting. e.g. silica sol
The quality of the grouting works is not the expected	Contractor documentation or other observations	(Not the scope here)	Execute supplementary grouting. additional holes in the current grouting round or a secondary grout round

5 As-built tunnel geometry, grouting and geology

5.1 Tunnel Excavation

The TASU and TASP tunnels were excavated using drill and blast technology. The excavation rounds were approximately 4 m long and were drilled with a modern computerized Sandvik two-boom drill jumbo (Figure 5-1). As shown in Figure 5-2 excellent perimeter control was obtained using the drill-and -blast technology. This facilitated the mapping of both the water bearing fractures and the dry open fractures after grouting.

a) Side view of the Sandvik drill Jumbo

b) View from the operator's cab of the Sandvik drill jumbo. The computer screen with the blast hole layout can also be observed.

Figure 5-1. *Drill Jumbo used for the tunnel excavation and drilling of the grout holes.*

a) Example of the tunnel profile quality achieved in the TASU and TASP Tunnels.

b) Photo of the completed niche located at the end of TASU that will be used for the KBS-3H experiments (TAS08).

Figure 5-2. *Illustration of the drill-and blast quality achieved during the excavation of the TASU and TASP tunnels and the niches. Note the prominence of the perimeter holes suggesting minimum blast-induced damage.*

The locations of the individual blast rounds are listed in Table 5-1 and Figure 5-3. Mapping of the blast rounds was carried out to identify all the open dry and water bearing fractures as well as grouted fractures to help identify which fractures sets could be related to the grout take. This data was also used to assess if blast-induced fractures were associated with water inflows.

Table 5-1. Chainage and number of each blast rounds round for TASP and TASU.

Round No. TASP	Start Chainage (m)	End Chainage (m)	Round Length (m)	% Pull	Round No. TASU	Start Chainage (m)	End Chainage (m)	Round Length (m)	% Pull
1	19.40	23.72	4.32	85	1	13.2	17.41	4.21	75
2	23.72	27.84	4.12	93	2	17.41	21.1	3.69	90
3	27.84	31.94	4.10	92	3	21.1	25.32	4.22	87
4	31.94	35.92	3.98	86	4	25.32	28.85	3.53	93
5	35.92	39.95	4.03	89	5	28.85	33.12	4.27	90
6	39.95	43.69	3.74	96	6	33.12	37.32	4.2	99
7	43.69	48.13	4.44	87	7	37.32	41.66	4.34	84
8	48.13	51.88	3.75	84	8	41.66	45.32	3.66	90
9	51.88	56.63	4.75	103	9	45.32	49.35	4.03	87
10	56.63	60.44	3.81	81	10	49.35	53.3	3.95	78
11	60.44	64.14	3.70	91	11	53.3	56.94	3.64	87
12	64.14	67.97	3.83	76	12	56.94	60.82	3.88	90
13	67.97	71.63	3.66	81	13	60.82	65.02	4.2	–
14	71.63	75.18	3.55	98					
15	75.18	79.71	4.53	–					

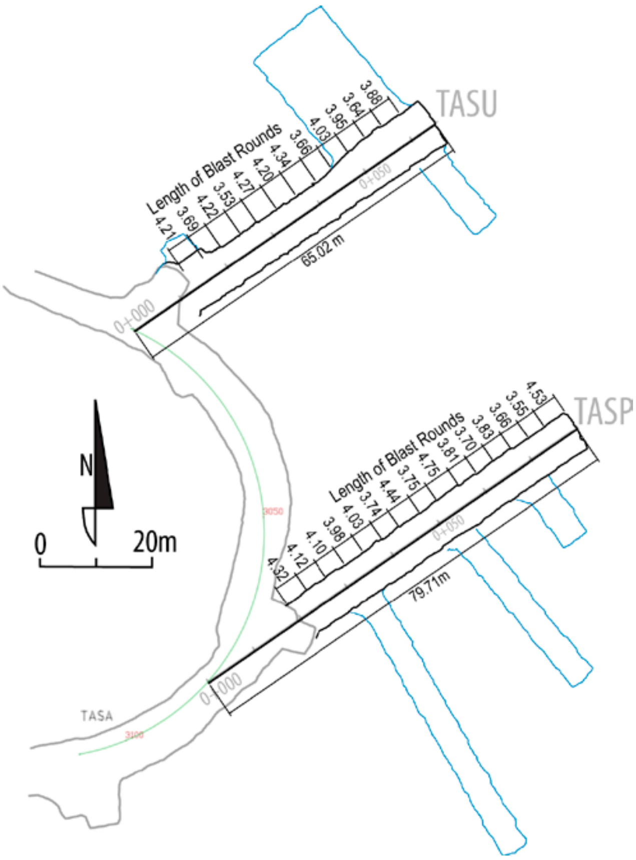


Figure 5-3. Location of the blast rounds in TASP and TASU tunnels. The locations of the individual blast rounds are listed in Table 5-1.

5.2 Grout plant

Each grout round required the drilling of 5 probe holes. Any inflow, such as that illustrated in Figure 5-4, was measured. The grout mix and injection was carried out using the Atlas Copco Unigrout Equipment (Figure 5-5). Packers for the grouting operations were the GMA – GX single-use packer illustrated in Figure 5-6.

Figure 5-4. *Example of a water flow encountered in a probe hole prior to grouting.*

Figure 5-5. *The Unigrout Equipment (EH22 200-140 AWB-SS) used in the Äspö Expansion.*

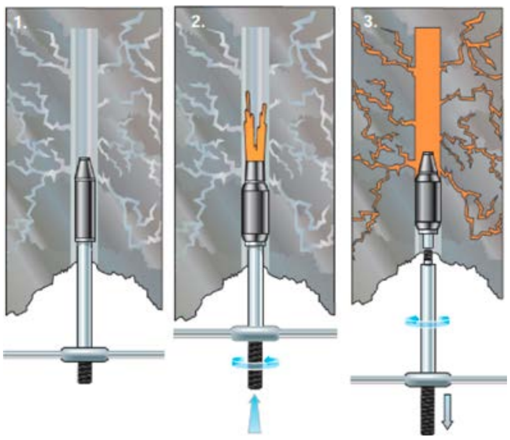


Figure 5-6. *Illustration of the single-use GMA GX packer used in the tunnel grouting.*

5.3 Water bearing fractures

Once grouting and excavation was completed geological mapping was carried out. All fractures, sealed, open, grouted and water bearing longer than 0.5 m was mapped using the ROCS System and photogrammetric techniques. The mapped water bearing fractures are shown in Figure 5-7.

The location of the individual grout fans and the number of grouted holes are shown in Figure 5-8. The actual chainage of each fan and the number of holes is also given in Figure 5-8. Prior to the construction of each niche, probe hole drilling at the niche location was carried out to assess the rock mass permeability. Where no grout fans are shown the permeability of rock was assessed to be sufficiently low, so that grouting was not required.

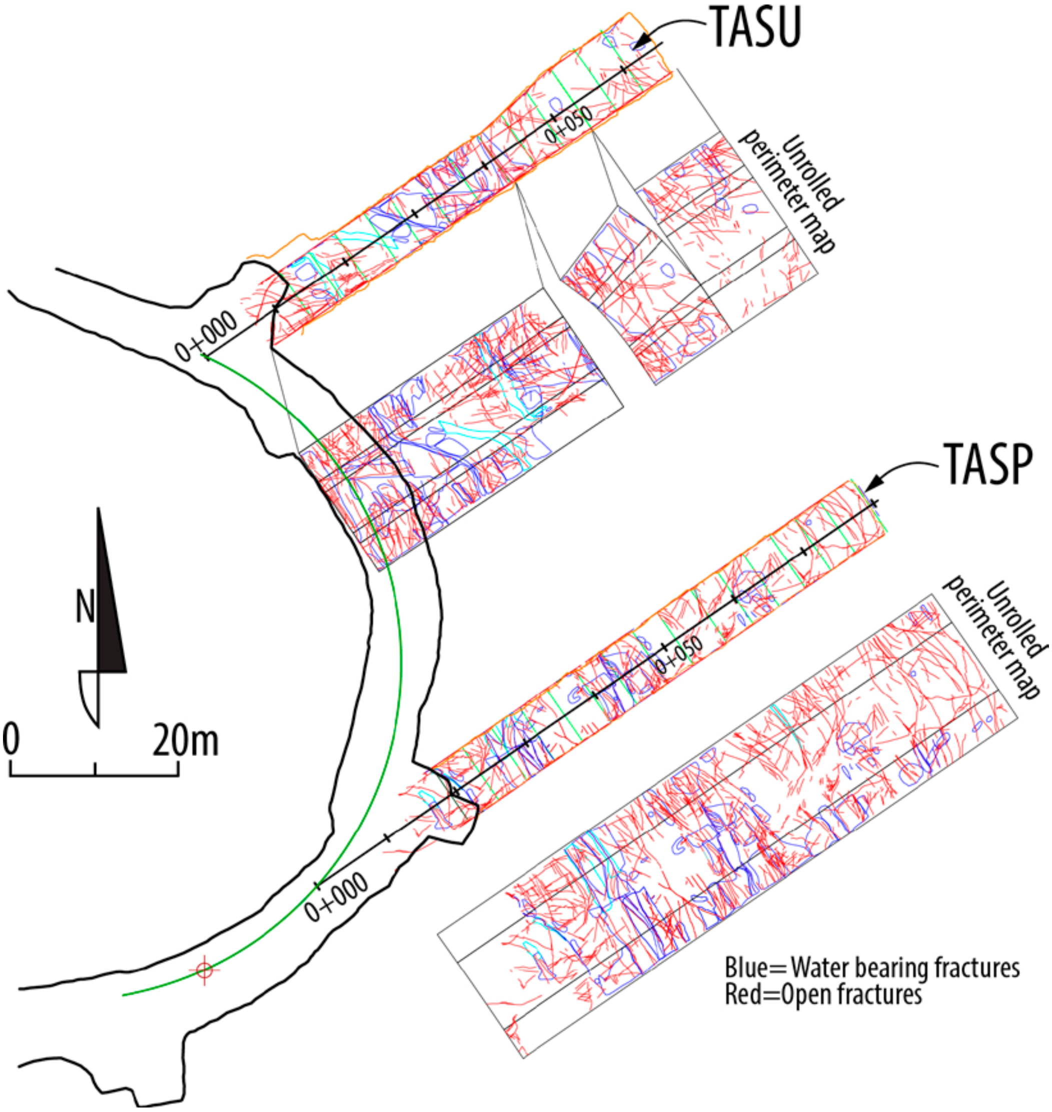
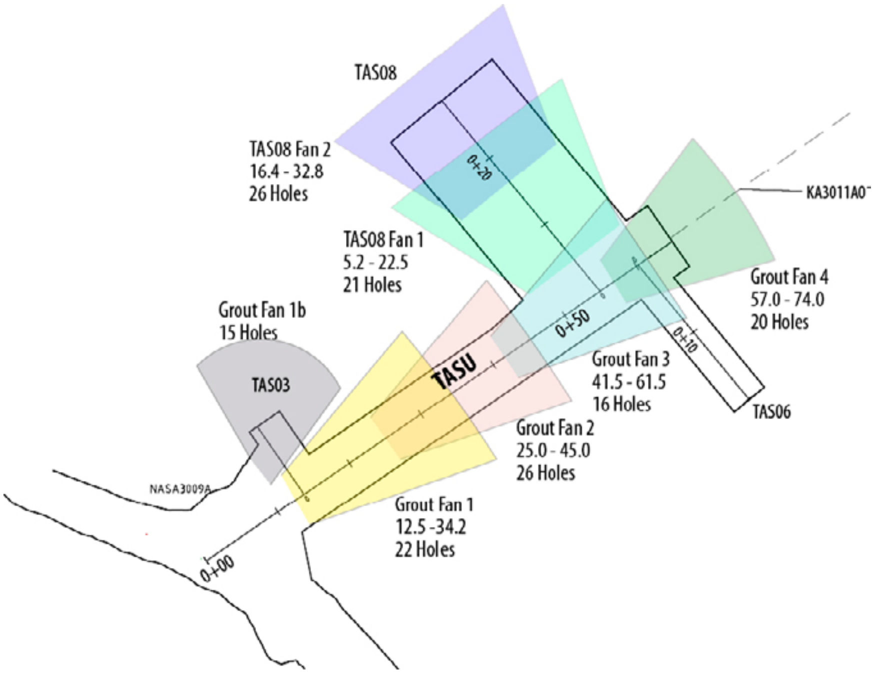
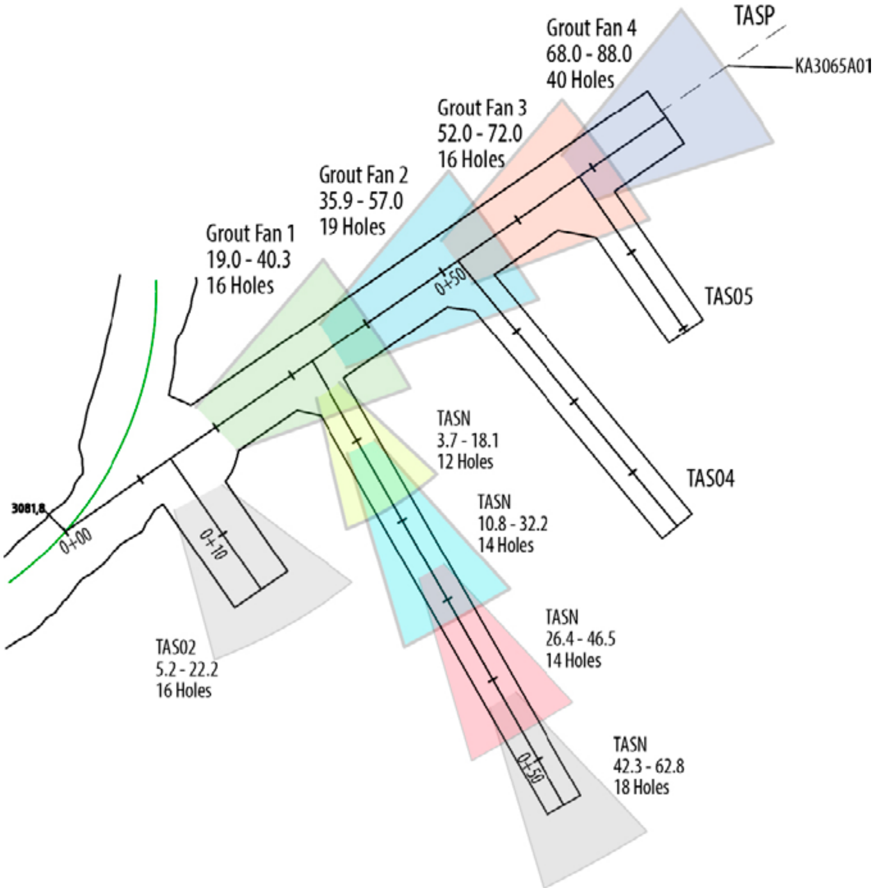


Figure 5-7. *Location of the "as-built" water bearing fractures (shown as blue lines) that were mapped after grouting. The blue enclosed areas are wet surfaces. The red lines are dry open fractures. The green lines indicate the location of the blast rounds.*



a) TASU Tunnel grout fans



b) TASP Tunnel grout fans

Figure 5-8. *Grout fans used for sealing TASP and TASU tunnels and the experimental niches.*

The time required for carrying out the individual grout fans is shown in Table 5-2. The inflow that was measured or observed during drilling and the grout take for each individual grout fan is shown in Table 5-3 and Table 5-4. The grout takes are excluding hole filling. It was difficult to assess the actual grout takes in holes where connection between holes and leakage around the packers occurred.

The standard geometry of a fan called for 16 holes with five probe holes (1, 4, 7, 10, 13). However for fans 1 and 2 in TASP and for fan 1 in TASU, the probe holes were 1, 5, 7, 9 and 13. This was changed in order to obtain a better distribution of the probe holes. At tunnel completion, each tunnel face was grouted using four evenly distributed grout holes.

In accordance with the design intention the results of the first grouting fans in TASP and TASU were evaluated using RTGC. The evaluation verified that the grout spread met the design requirements and confirmed visual observations of hydraulic connections between several grout holes. The evaluated grouting performance in combination with drawdown response provided by the HMS system gave confidence that the grouting strategy were fit for purpose. The succeeding grout fans (no. 2) fulfilled criteria for executing complementary grout holes. During this grouting sequence grout mix were pushed out from previously grouted holes, something which revealed that the strength development of the grout mix were slower than expected. Considering the uncertainty relative to inadequate sealing efficiency as well as the workers safety, the project decided to adjust the grouting strategy and omit the inflow criteria of 20 l/min in probe holes. The remaining grouting operation was executed in two steps. The first contained grouting of the probe holes and the second contained drilling and grouting of the remaining grout holes. Considering that a more conservative grouting operation was implemented it was decided to postpone RTGC analysis to after project completion. A detailed RTGC analysis is currently underway.

Table 5-2. Duration of grouting activities in the TASP and TASU tunnels.

TASP Grout Fan	Start Chainage (m)	End Chainage (m)	Date /Time Start	Date /Time Finished	Grouting time (2)	COMMENTS
1	19.1	41.3	20130319 / 14:26	20130319 / 16:40	2 t 14 min	Grouting of probe holes
			20130321 / 16:01	20130321 / 19:57	3 t 56 min	Grouting
2	35.9	57	20120601 / 13:43	20120601 / 18:42	4 t 59 min	Grouting
			20120604 / 12:06	20120604 / 12:33	0 t 27 min	Complementary holes
3	52	72	20120810 / 14:04	20120810 / 15:35	1 t 31 min	Grouting of probe holes
			20120814 / 15:06	20120814 / 19:18	4 t 12 min	Grouting
4	68	88	20120830 / 19:06	20120830 / 21:04	1 t 58 min	Grouting of probe holes
			20120903 / 14:34	20120903 / 22:16	7 t 32 min	Grouting
4–2 (1)	68	88	20120906 / 14:37	20120906 / 16:02	1 t 25 min	Grouting of probe holes
			20120911 / 09:13	20120911 / 12:21	3 t 8 min	Grouting

(1) Complementary fan drilled in between fan 1.

(2) Grouting time includes time for transition from hole to hole, which is between 1 and 5 minutes between two holes.

TASU Grout Fan	Start Chainage (m)	End Chainage (m)	Date /Time Start	Date /Time Finished	Grouting time (1)	COMMENTS
1	12.5	34.2	20120323 / 15:51	20120323 / 18:39	2 t 48 min	Grouting of probe holes
			20120326 / 16:32	20120326 / 20:39	4 t 7 min	Grouting
			20120327 / 09:06	20120327 / 09:46	40 min	Complementary holes
1b			20120427 / 15:57	20120427 / 17:42	1t 45 min	Grouting (2)
			20120501 / 08:52	20120501 / 17:42	6t 10 min (3)	
2	25	45	20120531 / 09:02	20120531 / 15:49	6 t 47 min	Grouting
			20120604 / 19:25	20120604 / 20:52	1 t 27 min	Complementary holes
3	41.5	61.5	20120813 / 16:39	20120813 / 18.48	2 t 9 min	Grouting of probe holes
			20120815 / 17:22	20120815 / 21:57	4 t 35 min	Grouting
			20120817 / 14:45	20120815 / 15:16	31 min	Complementary holes
4	57	74	20120911. 17:02	20120911. 18:18	1t 16 min	Grouting of probe holes
			20120912. 14:20	20120912. 20:23	6t 3 min	Grouting

(1) Grouting time includes time for transition from hole to hole, which is between 1 and 5 minutes between two holes.

(2) Holes 13–15 and 19–20 were grouted on 2012-04-27. Holes 1–2. 5–12 and 21–22 on 2012-05-01.

(3) Grouting stopped for a while.

Table 5-3. TASP Tunnel inflows and grout takes. Refer to Figure 4-2 for hole locations.

	INFLOWS/GROUT TAKE BY HOLE NUMBER																				
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**Total**
FAN 1-Inflow (l/min)	1.05				5	*Dripping*	26.4	*Dripping*	25.2	*Dripping*	*Dripping*		6.25		< 5	< 5					
FAN 1-Grout (l)	15	29	0	5	36	3	0	0	200	24	0	48	24	42	87	208					720
FAN 2-Inflow (l/min)	9.8				6.4		1.26		18				16.8								
FAN 2-Grout (l)	0	26	185	0	0	167	0	179	38	168	0	241	198	4	5	64	0	170	8		1452
FAN 3-Inflow (l/min)	0.03			0			0.34		~ 2	0.06			0.11								
FAN 3-Grout (l)	1	5	0	0	0	0	10	41	7	0	2	2	0	2	3	2					74
FAN 4-Inflow (l/min)	2.8	*Dripping*		0			0.2	*Dripping*	*Dripping*	0.06			0								
FAN 4-Grout (l)	28	3	4	0	0	2	5	13	21	2	10	1	2	1	2	2	0	0	0	6	100

Grout take excluding hole filling. But volumes can include leaking and connected holes.
Inflow values in italic are estimated during drilling. Others are measured.
An empty cell in the table means no measurement.

	INFLOWS/GROUT TAKE BY HOLE NUMBER																				
	21	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**	**31**	**32**	**33**	**34**	**35**	**36**	**37**	**38**	**39**	**40**	**Total**
FAN 4-2- Inflow (l/min)				0			1	*0.2*	*0.1*	1			0			0					
FAN 4-2-Grout (l)	2	4	3	0	0	0	0	4	0	6	1	1	1	2	2	2	18	4	12	18	80

Grout take excluding hole filling. But volumes can include leaking and connected holes.
Inflow values in italic are estimated during drilling. Others are measured.

Table 5-4. TASU inflows and grout takes. Please see Figure 4-2 for hole location.

	INFLOWS/GROUT TAKE BY HOLE NUMBER																										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**Total**
FAN 1- Inflow (l/min)	27.3		*<5*	*<5*	74	*<5*	92		7.8				51														
FAN 1-Grout (l)	397	101	72	118	398	90	400	35	56	0	43	82	399	145	142	201	206	188									3072
FAN 1b- Inflow (l/min)																											
FAN 1b-Grout (l)	4	6	216	237	2	28	15	2	0	23	201	193	0	202	205				204	0	153	19					1710
FAN 2- Inflow (l/min)	10			4			2.5			17.5			0.3														
FAN 2-Grout (l)	102	122	0	45	115	13	21	6	94	209	201	174+274	6	0	13	0	0	0	204	5	0	8	102	15	42	84	1855
FAN 3- Inflow (l/min)	0.16	*~0.5*		0.01			0.17			0.06	*~3*	*~1*	0.95														
FAN 3-Grout (l)	2	2	7	0	2	5	3	17	10	2	8	5	2	0	4	9											79
FAN 4- Inflow (l/min)	0			0			0			0			0														
FAN 4-Grout (l)	3	3	4	0	14	4	0	17	4	0	2	23	2	4	0	4	4	0	0	10							97

Grout take excluding hole filling. But volumes can include leaking and connected holes

Inflow values in italic are estimated during drilling, others are measured.

6 Observational Method: Evaluating results

6.1 Quality assurance

A lot of effort was put into the quality control of the grouting operation. The contractor had responsibility to document the works in accordance with a control plan, which included pre-defined protocols and check lists. All data collected are handled following SKB's routine for data processing.

6.2 Monitoring plan

As stated in Section 4.4 changes in groundwater head in borehole KA2050A was followed on-line. Before construction started a reference level for the groundwater head was established by looking at one-year time series identifying 1-week undisturbed measurements. There were ten other boreholes having HMS instrumentation in the vicinity of the expansion area that also were checked on a weekly basis.

As all on-going activities in the Äspö HRL were reported in a daily log it was possible to relate drawdowns to construction activities and to follow the recovery of the system. An example of the response from the HMS system is shown in Figure 6-1. It shows temporary drawdown of the groundwater head that occurred due to inflow in boreholes that were drilled during a grouting round.

The accumulated water inflows that were measured in weirs during construction are illustrated in Figure 6-2. The total inflow from both tunnels is around 15 l/min. The one measurement that shows a peak occurred after drilling of flowing probe holes and before their grouting. Inflow in weirs increased from October due to excavation of experimental niches (TASN, TAS04, TAS05 and TAS06). Another source to accumulated inflow was that drilling for rock bolts occasionally intersected flowing fractures in the non-grouted experimental tunnels. Water leakages also occurred in a few bolt holes when installing permanent support. These holes were grouted and re-drilled as part of the quality assurance plan.

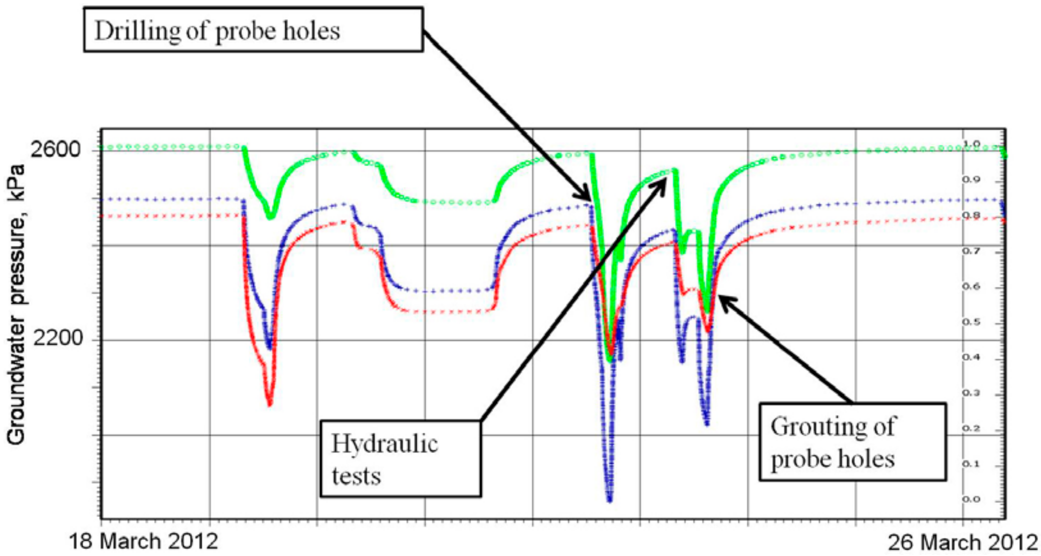


__Figure 6-1.__ Drawdown observed in KA2050A related to ongoing construction activities such as probe hole drilling, hydraulic testing and grouting.

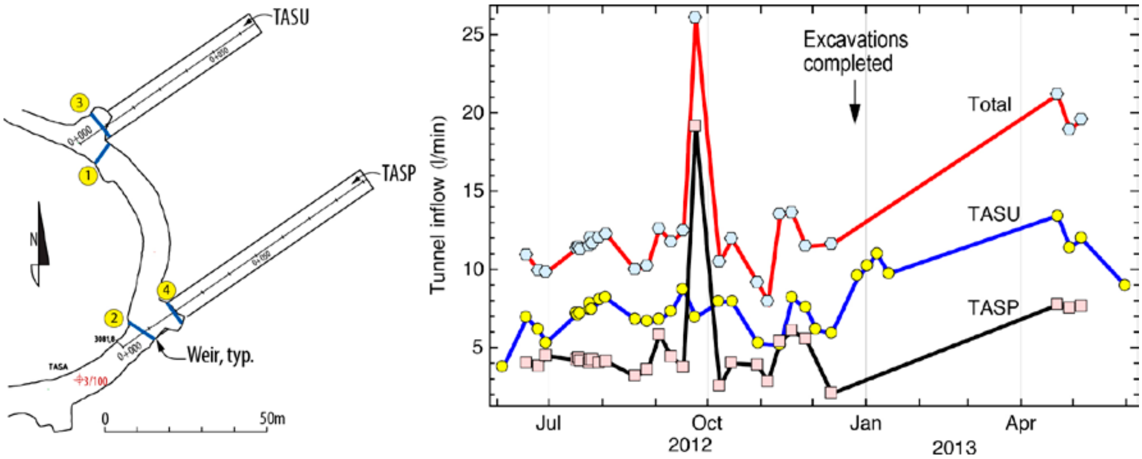


Figure 6-2. *Inflows in l/min measured in each weir for TASU and TASP during construction.*

6.3 Predicted and actual behaviour

Figure 6-3 shows that the total drawdown at the end of construction was about 25 m, which is below the acceptable drawdown that was set to 50 m. The accumulated inflow from both tunnels was around 20 l/min. The long-term drawdown is as expected larger than the predicted behaviour, which was determined by the short-term tests shown in Figure 4-4. The short-term tests predicted around 15 m drawdown for an inflow of 20 l/min.

The possible behaviour for the total inflow from both tunnels after grouting was assessed using conservative upper bound assumptions. The prediction was that 40 l/min could be expected after grouting while the actual value was around 20 l/min. This implies that grouting was more efficient then assumed during the design.

The possible behaviour for the grout take was also assessed using conservative assumptions. The prediction was about 900 l and 1900 l for TASP and TASU, respectively. The recorded grout take (excluding hole filling) for all grouting fans has been calculated as 2400 l for TASP and 5600 l for TASU. Figure 6-4 shows the distribution of predicted and actual grout takes in TASU and TASP compared to the inflows and PFL transmissivities measured in the pilot holes KA3011A01 and KA3065A01.

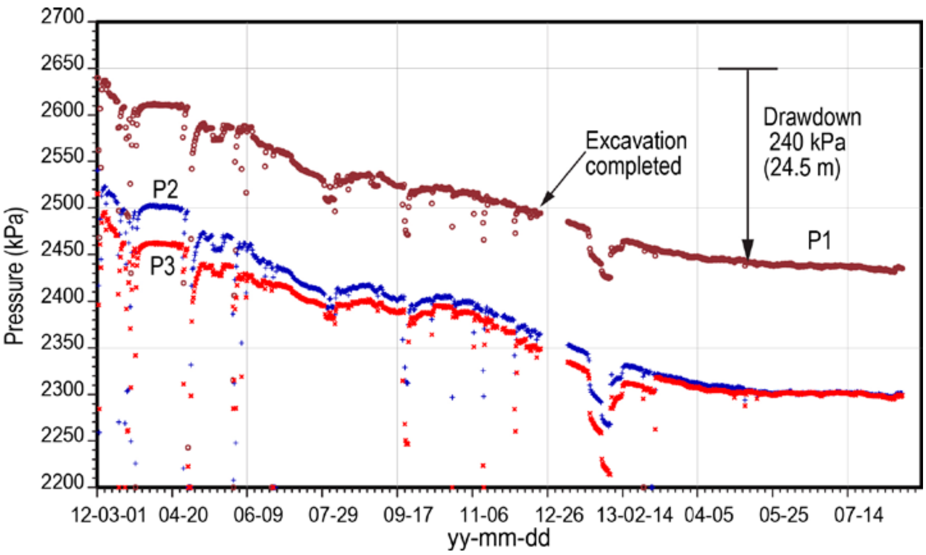


Figure 6-3. *Drawdown observations in KA2050A in the three piezometers P1 (brown), P2 (blue) and P3 (red). The drop in pressure at the end of 2012 is related to rock bolting in the roof of Niche TAS05, which was not grouted (see Figure 5-8 for location).*

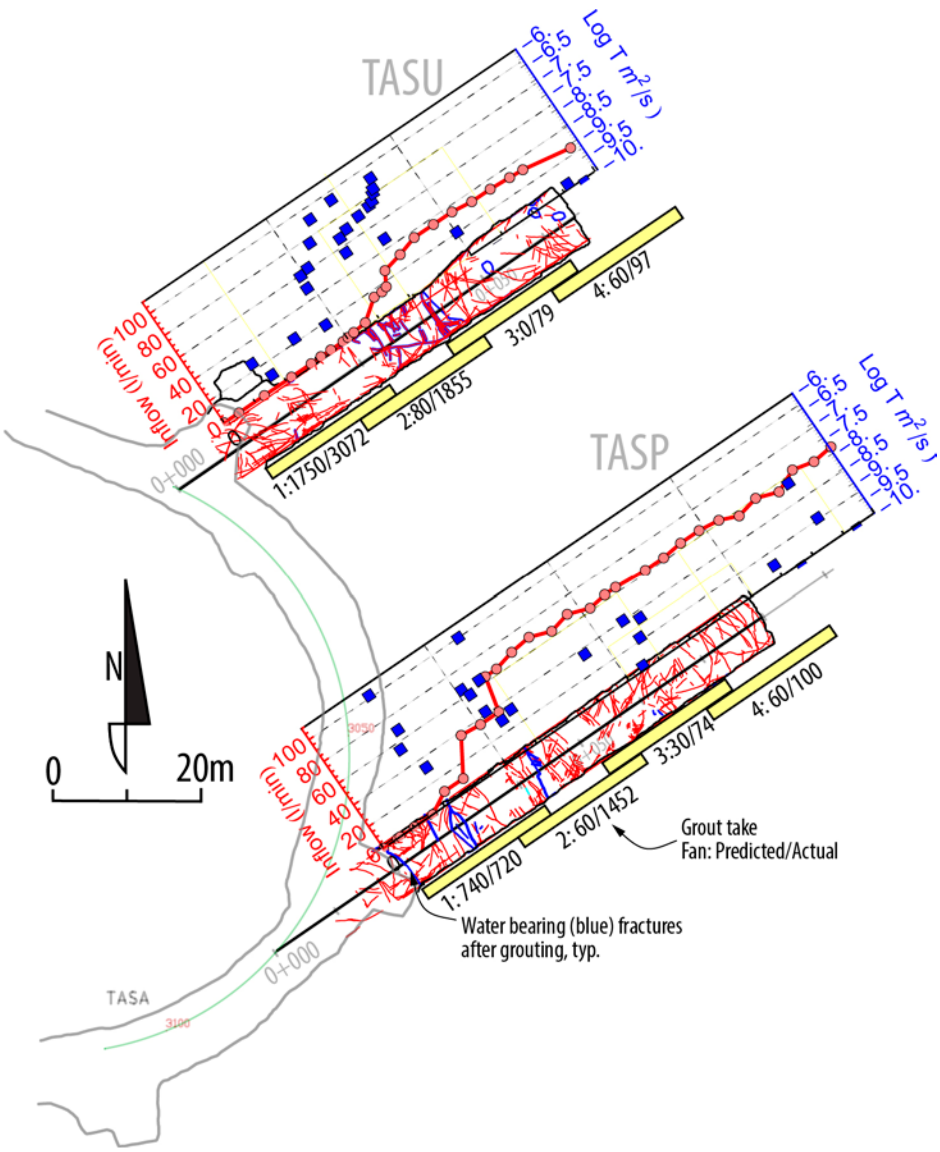


Figure 6-4. *Grout takes in TASU and TASP compared to the inflows and PFL transmissivities measured in the pilot holes KA3011 and KA3065.*

The theoretically predicted grout takes were lower than the actual grout takes. However the actual grout takes were for some grout holes difficult to assess. This was caused by the occurrence of hydraulically connected grout holes, due to packers coming loose during grouting and from occasional grout leakage to the rock surface. It is likely that wasted grout mix were included in the summation of the actual grout take. In addition theoretical model errors may have contributed to underestimation of the grout take.

6.4 Contingency measures during construction

The frequency of hydraulically connected grout holes was unexpectedly high both in the first and the second grout round in both tunnels. In connection with grouting of complementary grout holes in the second grout round, in accordance with the grouting strategy, it was discovered that the strength development of the grout mix were slower than expected. In order to handle the uncertainty relative to inadequate sealing efficiency the contingency measure of executing a primary and secondary grout fan were implemented for the remaining grout rounds.

6.5 Conclusions

The acceptable limit of behaviour for the Äspö Expansion project was defined by the overall project requirement, i.e. the maximum allowable drawdown was 50 m relative to the initial conditions in the monitoring hole KA2050A. The inflow tests that were carried out in the pilot holes before constructions started provided a model for predicting the maximum inflow that would meet the drawdown requirement. The inflow after grouting was predicted in design calculations. By comparing this design prediction with results from the inflow tests it was established that there was an acceptable probability that the actual behaviour (drawdown) would be within the acceptable limits, as stipulated by the principles of the Observational Method.

A reference level for the groundwater head was established before construction started and the Hydro-Monitoring System (HMS) was used to monitor the hydraulic heads during construction and provided accurate information on both short-term and long-term drawdowns that resulted from the different construction activities. Hence, the system provided reliable data for controlling that the actual behaviour remained within the acceptable limit.

The overall conclusion is that the application of the Observational Method was successful. The project requirement was respected and the actual drawdown relative to the accumulated inflow was in line with the predicted behaviour. The lesson learned is that provisions must be made in a project using the Observational method for following-up and if necessary revise criteria that govern implementation of contingency measures.

7 Summary and recommendations

The construction of the tunnels TASU and TASP within the Äspö Expansion Project was carried out over a 8 month period commencing in March 2012. The overarching design requirement for the Project was that the long-term reduction in the existing hydraulic heads measured over the future tunnels and niches would be less than 50 m at the end of construction.

7.1 Project management

While the Äspö Expansion Project provided an opportunity to demonstrate the application of the Observational Method to achieve the overall design requirement, the project management structure that is typically used at Äspö HRL project was not modified to accommodate the application of the Observational Method. This may not have been optimum, but demonstrated the Observational Method is not limited to a particular management strategy. The form of the contractual arrangement with the excavation and grouting contractor in part facilitated this.

According to Nicholson et al. (1999), there is more interaction between designers and constructors when the Observational Method is used as the risk mitigation strategy for a project, than in a project that utilize more traditional design approaches. This interaction needs management and co-ordination. The commitment of the members of the project team and their willingness to "own" and solve problems are therefore of critical importance. In general the Observational Method should not be initiated unless the critical design issues have been considered thoroughly and an effective management system has been set up for implementing the solutions to those issues. If the Observational Method is applied under a management regime that does not take into account the full range of complexity associated with the project and provide the necessary integration of processes, then significant risks can be introduced into the project.

The application of the principles of Observational Method for managing project risks provides a framework for the management of the various interacting sources of uncertainty, not only those associated with the ground but also organisational uncertainties. Figure 7-1 illustrates some of the factors that project management must consider when implementing the Observational Method to minimise the organisation risks on major projects.

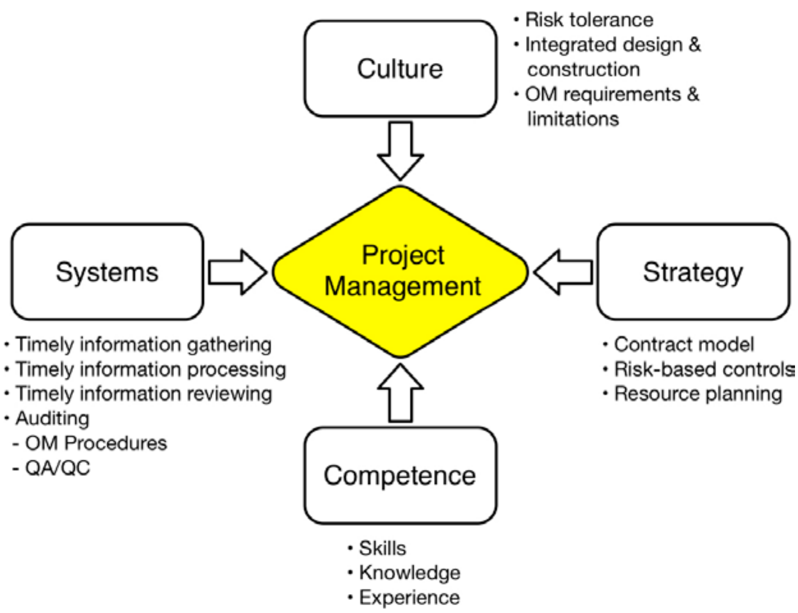


Figure 7-1. *Example of the factors that a project management team must evaluate when implementing the Observational Method. Adapted from Nicholson et al. (1999).*

7.2 Project summary

The risk associated with maintaining the groundwater pressures during the construction of the Äspö Expansion Project were successfully managed using the Observational Method. The steps that were used to manage the risk are summarised below.

- A pilot hole drilled at the approximate centre of the TASU and TASP tunnels were used to establish the relationship between inflows and drawdown. With this relationship, acceptable limits of behaviour, i.e. the potential drawdown was established.
- The inflow-drawdown relationship, and the hydraulic apertures of the fractures associated with the inflows provided sufficient information for developing a grouting design and strategy. Design calculations of the predicted inflow after grouting demonstrated that there was an acceptable probability that the actual inflows will be within the acceptable limits necessary to keep the hydraulic heads within the design target, i.e. <50 m of drawdown.
- A plan for monitoring the drawdown was implemented that provided real-time responses of the hydraulic heads in the rock mass above the tunnels. This monitoring system was also used to monitor the response in the pilot holes drilled prior to tunnel constructions. In addition to the borehole monitoring, the inflows to each tunnel were also monitored in weirs constructed at the entrance of each tunnel.
- The monitoring data was accessible to all decision-making personnel and provided adequate warning to allow contingency actions to be undertaken quickly.
- A contingency plan with potential cause-and-effects was developed and used to establish action required for each grout fan. If the results from the grouting were not acceptable, i.e. the head loss was greater than predicted, a procedure was in place to stop work and have the design reviewed by a senior expert group.
- Grouting methodology
  - The low-pH (pH<11) grout mix that was used in the project did not have an acceptable strength development. This event resulted in that a more conservative grouting strategy using two grout rounds was implemented. As a consequence the potential for executing an efficient grouting operation could not be fully utilised. The grouting strategy provided means for handling hydraulically connected holes. A inflow criteria, here 20 l/min was established that triggered grouting of the probe holes, However in similar conditions, initially a more conservative approach should be considered with provisions for adjusting the criteria based on the actual conditions.
  - The predicted grout takes were lower than the actual grout takes. Occurrences of hydraulically connected grout holes and leakage due to packers coming loose during grouting have likely led to that some of the wasted grout mix was included in the summation of the actual grout take. In addition the theoretical model may have contributed to underestimation of the grout take. The theoretical model should be refined to accommodate for the difference between the hydraulic fracture aperture and the actual fracture aperture.
  - The predicted inflow to the tunnels after grouting was higher than the actual inflow. The theoretical model assume that all water-bearing fractures are independent of each other, while in reality such fractures may form networks. Previous experience has shown that grouting seems to seal the network, something which leads to a better sealing effect than what is predicted by the theoretical model.
  - RTGC was used to develop Stop Criteria. They were based on maximum grouting time, maximum grout take and minimum grout flow, the latter was to be applied for negligible grout take. Stop Criteria are always conservative as they have to encompass expected variations of the hydrogeological conditions. The used Stop Criteria have been working well with respect to that the project requirement was respected and in line with the predicted behaviour.

7.3 Recommendations

As with all projects of this nature there were lessons learnt that should be borne in mind when planning for underground construction projects that require the implementation of the Observational Method. These are outlined below:

- The successful implementation of the Observational Method requires significant interaction between designers and constructors. This interaction needs management and co-ordination. The commitment of the members of the project team and their willingness to "own" and solve problems are therefore critical to the success of the Observational Method. A project management structure that is not focused on problem solving and a team strategy will hinder the implementation of the Observational Method.
- The information and data that is required for decision making during construction is typically transient in nature and consequently of little value in establishing the initial state of the rock mass, e.g. the flow from a probe hole. A clear distinction needs to be made between data and information needed for construction purposes and implementing the Observational Method. and the data and information needed for other purposes. These data streams should be treated separately so that the construction-only data is delivered in the most efficient means possible. The success of the Observational Method relies on collecting and evaluating monitoring data expeditiously. If this information cannot be provided as required, the Observational Method will simply not work.

While the project objectives were met, the low-pH grout recipe used to control the inflows was not sufficiently robust for industrial application, and should be improved.

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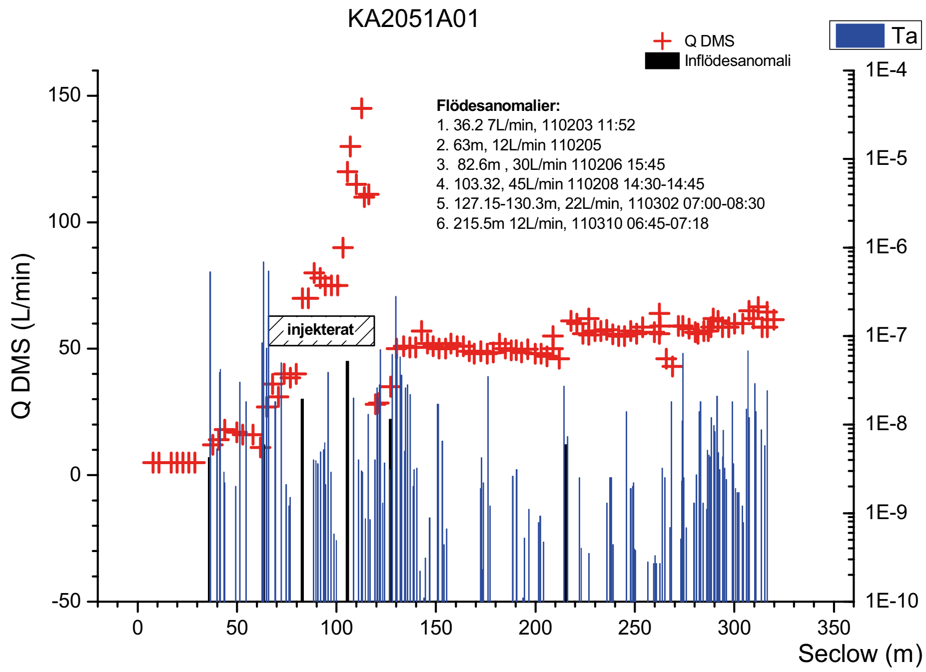
Borehole hydraulic test results



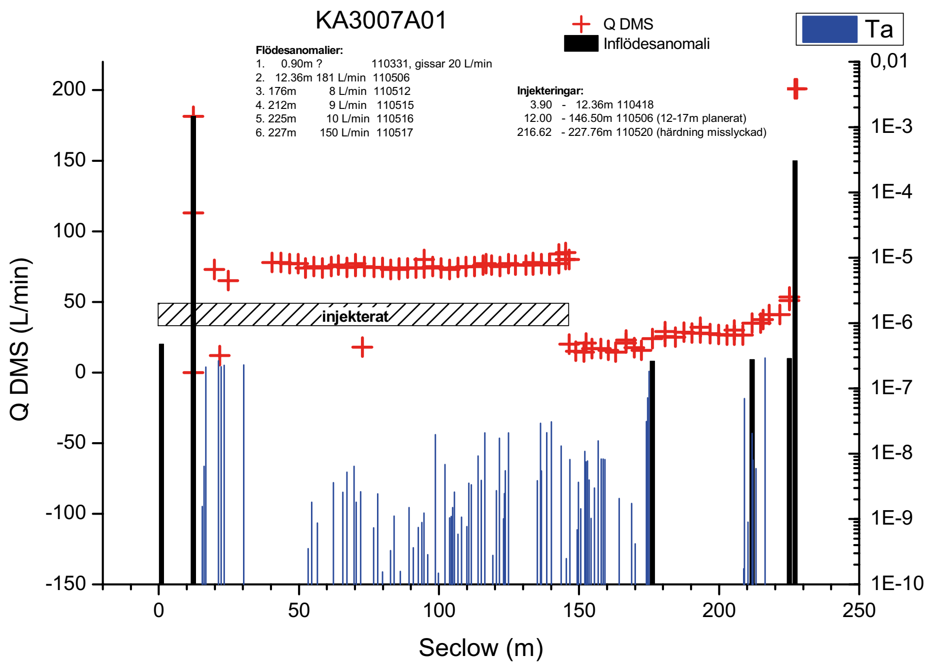


Figure A-1. *Measured inflows during drilling (l/min) and transmissivity values determined from the Pfl difference flow logging (m^2/s) along KA2051A01 and KA3007A01.*

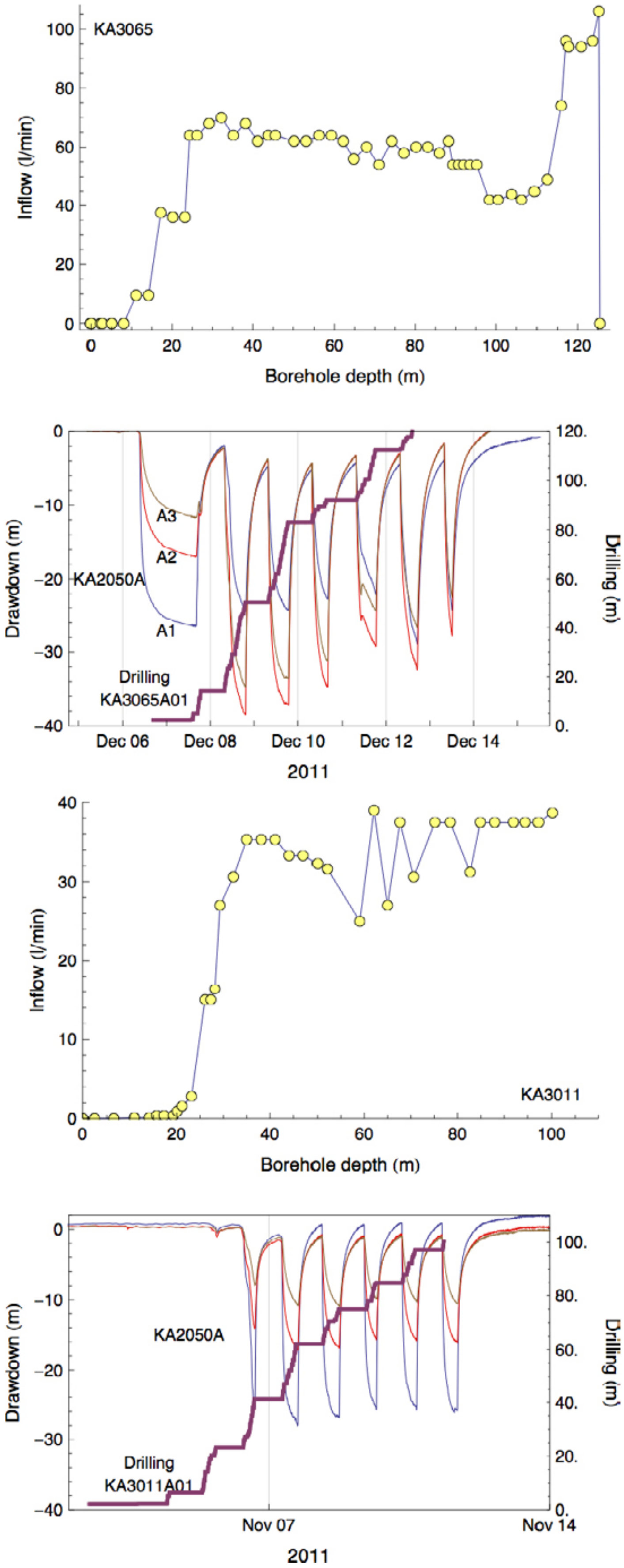


***Figure A-2.** Drawdown response in KA2050A while drilling KA3065A01 and the drawdown response in KA2050A while drilling KA3011A01.*

Appendix B

Basis for Grouting Design

B1 Introduction

This Appendix presents the basis for grouting design, i.e. the method of analyses and the results from the design calculations needed to implement the Observational Method.

Theoretical models for calculating the sealing efficiency and grout spread around tunnels were presented by e.g. Dalmalm and Stille (2003) and Gustafson and Stille (2005). The theories form the basis for the real time grouting control method (RTGC). The results from RTGC calculations are the grout flow and the grout spread in fractures with given apertures as a function of the grouting time. The input data for RTGC calculations are the effective grouting pressure and the rheological properties of the grout mix. By considering the distance between grout holes and the hydrogeological conditions in terms of maximum and minimum fracture apertures, RTGC can be used to develop stop criteria for the grouting operation. The assumption for RTGC calculations is that water flow is concentrated in discrete individual and independent fractures, and that the grout penetration is governed by the physical aperture of the water bearing fractures, the grouting time and the grouting pressure. RTGC can be used as a basis for grouting design as well as to analyse and evaluate the performance of pre-grouting operations (Kobayashi and Stille 2007, Kobayashi et al. 2008, Stille et al. 2009).

B2 Definitions

Grouting time is the time period when grouting is in progress at the prescribed grouting pressure, i.e. planned and unplanned stoppages are not included in the grouting time. The grout take is the consumed grout volume excluding hole filling. The grouting operation is illustrated in Figure B-1.

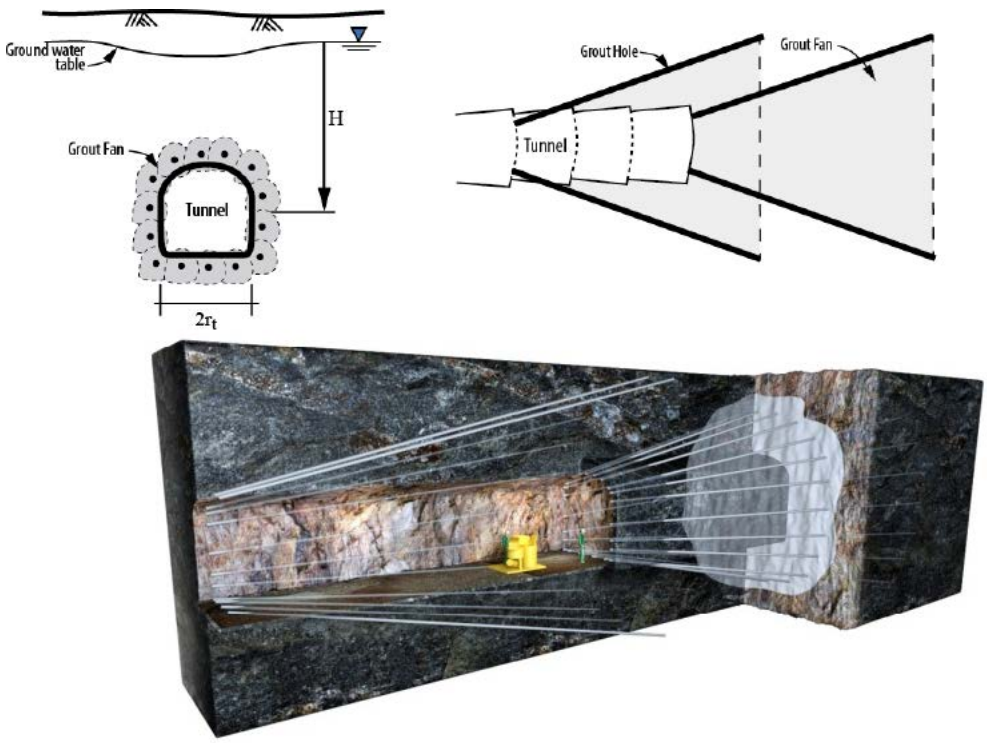


Figure B-1. *Illustration of the grouting operation including the grouting fan.*

B3 Design calculations

During drilling of the pilot holes in the alignment of the tunnels the water inflow was recorded regularly at documented borehole depths, see Appendix A. Water inflow was measured through the drill string as the drill hole was advanced. Based on the measured accumulated inflow the total transmissivity $\sum T$ was evaluated using Moye's formula[1] (Moye 1967):

$$\Sigma T = \frac{\Sigma Q}{2\pi H}\left(1 + \ln\frac{L}{d}\right) \qquad \text{(B2-1)}$$

where

$\sum Q$ = measured inflow of water [l/min].

H = groundwater head, here 350 m.

L = the total borehole depth up to where a measurement was executed [m].

d = diameter of the borehole, here 0.076 m.

The transmissivity between successive measurements in the borehole was assessed by evaluating the difference in total transmissivity as:

$$\Delta T = \Sigma T_i - \Sigma T_{i-1} \qquad \text{(B2-2)}$$

The hydraulic aperture, b_{hyd}, corresponding to the transmissivity evaluated with eq. (B2-2), was calculated using the "cubic law" (Snow 1970):

$$b_{hyd} = \sqrt[3]{\frac{12\,\mu\,\Delta T}{g\,\rho}} \qquad \text{(B2-3)}$$

where

ΔT = transmissivity between successive measurements in the borehole [m^2/s].

μ = viscosity of water, 0.001 [Pa s].

g = gravitational constant, 9.81 m/s^2.

ρ = density of water, 1000 kg/m^3.

To assess the water inflow before and after grouting it was assumed that the transmissivity along a section of the tunnel, prior to grouting, was equal to the value calculated for the same section of the pilot hole. The inflow to the tunnel both before and after grouting was assessed with (Hawkins 1956, Gustafson 2009):

$$\Delta Q = \frac{2\pi\, H\, \Delta T}{\ln\left(\frac{2H}{r}\right) + \left(\frac{\Sigma T}{\Sigma T_g} - 1\right)\ln\left(1 + \frac{t}{r}\right) + \zeta} \qquad \text{(B2-4)}$$

where

ΔT = transmissivity along a section of the tunnel (for section lengths, Table B-5) [m^2/s].

r = equivalent tunnel radius, here 3.5 m.

$\sum T_g$ = transmissivity of the grouted rock mass was assumed to be 2 x 10^{-8} m^2/s, which corresponds to the smallest fracture aperture that could be penetrated by the grout mix. For ungrouted rock $\sum T_g = \sum T$.

t = thickness of the grouted zone, assessed with the calculated grout spread in a fracture with a hydraulic aperture equal to b_{hyd} [m]. For un-grouted rock t=0.

ζ = Skin factor[2], here = 0.

[1] The motive for using Moye's equation instead of Thiem's ditto is that it provides an estimate of the radius of influence over which drawdown occurs.

[2] The basis for the empirical skin factor is uncertain, especially for grouted rock, and therefore a conservative approach was used.

Based on the results from pre-testing the grout mix, the following properties for the grout mix were used as a basis for grouting design.

- Yield strength, τ_g: 8 Pa.
- Viscosity, μ_g: 0.065 Pas.
- Minimum fracture aperture that could be penetrated by the grout mix, b_{min}~30 µm.

B4 Stop criteria and drill plan

Stop criteria were developed to match the range of expected hydrogeological conditions in the tunnels. The stop criteria takes account of the grout mix properties, effective grouting pressure and tip distances between boreholes. Furthermore, the stop criteria also accounts for requirements on the acceptable grout spread in the smallest fracture aperture that can be penetrated by the grout mix and the maximum acceptable grout spread in predicted fracture apertures. Both maximum and minimum fracture apertures may occur in the same grout hole and the requirement on minimum grout spread was the governing parameter.

The grout mix properties, the grouting pressure and the requirements on grout spread that were used to determine the stop criteria are summarized in Table B-1. Stop criteria that apply for the effective grout pressure of 4 MPa (above existing groundwater pressure) are summarized in Table B-2. A waiting time of at least 6 hours was prescribed before any other activity could take place at the grouted tunnel front.

The stop criteria given in Table B-2 can be summarised as follows:

- Grouting should be discontinued when the injected volume reaches 200 l regardless if grouting time or prescribed grouting pressure has been reached (10 m ≤ grout spread ≤ 14 m).
- Grouting should be discontinued after 5 min grouting time if the grout take is less than 3 l, which for the grout hole corresponds to $\sum Tg \leq 2\times10\text{–}8\ m^2/s$.
- For holes with grout take exceeding 3 l after 5 min grouting time, the grouting should be stopped after 15 min grouting time if the grout take is less than or equal to 5 l(grout spread ~4m).
- For holes with grout take exceeding 5 l after 15 min grouting time, the grouting should be stopped after 20 min grouting time (4 m ≤ grout spread ≤ 14 m).

The stop criteria were developed based on the drill plan for the grout fan shown Figure B-2.

Table B-1. Grout mix rheological properties, pressures and estimated grout spread for different hydraulic apertures.

Grout mix properties		Grouting pressure	Grout spread per hydraulic aperture		
Viscosity	Yield strength	Total / Effective	> 50 µm	160 µm	230 µm
0.065 Pas	8 Pa	~8 MPa / ~4 MPa	Grout Spread 4 m	Grout Spread 10 m	Grout Spread 14 m

Table B-2. Stop criteria for grouting with 4 MPa effective grouting pressure.

Stop Criteria – Grout Take	Stop Criteria – Grouting Time
Grout Take = 200 l	Grout Time < 20 min
Grout Take < 3 l	Grout Time = 5 min
3 l < Grout Take ≤ 5 l	Grout Time = 15 min
5 l < Grout Take ≤ 200 l	Grout Time = 20 min

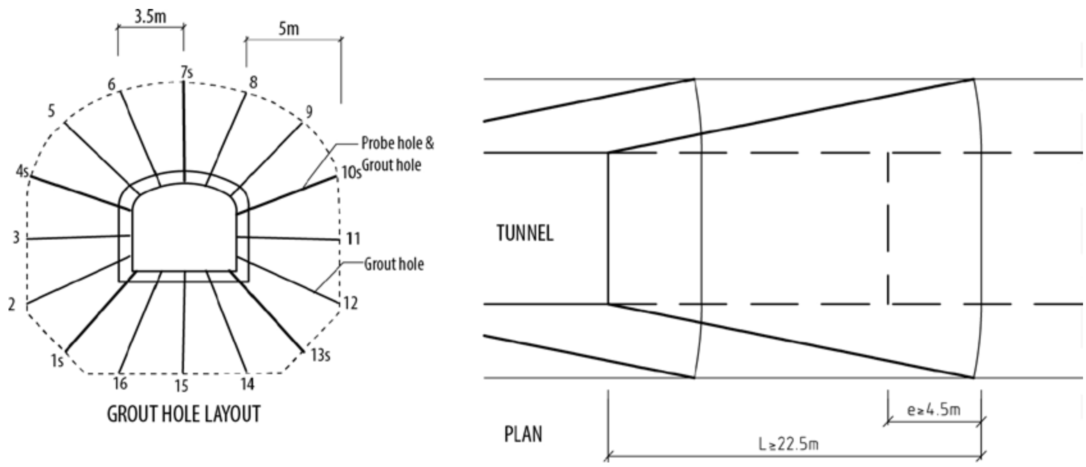


Figure B-2. Layout of the probe hole and grout hole used for the grouting operation, section and plan view.

B5 Assessment of hydraulic apertures

The assessment of hydraulic apertures was based on results from the water inflow tests, e.g. shown in Appendix A.

Maximum recorded inflow in the pilot hole KA3011A01 located in TASU, was approximately 35 l/min. However, there is uncertainty about the inflow as it may have been influenced by previous grouting operations that were carried out in nearby a borehole KA3007A01. The water inflow to KA3007A01 was measured to about 180 l/min at about 12.5 m depth (borehole depth). The inflow was distinct as the hole was dry when drilling had reached about 10.5 m depth. The hole was grouted at this depth something which may have affected the hydraulic conditions in KA3011A01.

For the grouting predictions, the maximum recorded inflow to KA3007A01 was used. Consequently, the expected maximum hydraulic aperture in TASU was assessed to approximately 230 microns and occurring between 20 and 30 m depth. Beyond 30 m to approximately 65 m depth, the largest hydraulic aperture was assessed to approximately 95 microns.

Maximum inflow to the pilot hole KA3065A01 located in TASP was approximately 70 l/min. For TASP the maximum hydraulic aperture was assessed to be about 160 microns and occurring between 35 and 45 m depth. Beyond 45 m depth to the planned end of the tunnel, at about 90 m depth, the largest hydraulic aperture was assessed to be approximately 75 microns.

The back-calculated transmissivity follows approximately a log-normal distribution, see Figure B-3. The probability that the total transmissivity for a 3 m long section in one of the tunnels will be larger than the estimated maximum hydraulic aperture was assessed to be less than 5%.

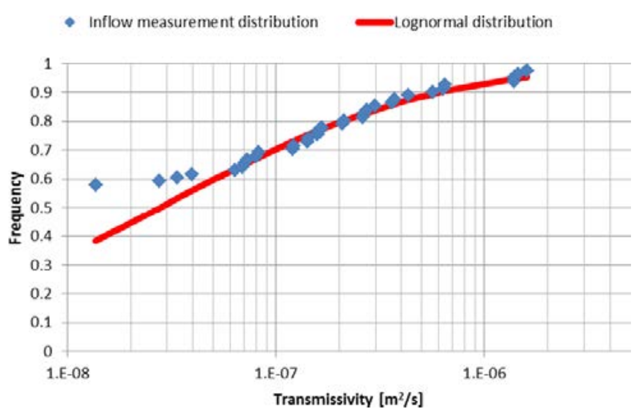


Figure B-3. Empirical distribution of the inflow measurements in the pilot holes and a best-fit lognormal distribution.

B6 Grout mix design

The design constraint for a low pH grout required optimizing the grout mix for properties that are compatible with a construction environment. The design grout must have properties that allow for penetration of fractures with a minimum hydraulic aperture. The selected grout mixes and their properties are given in Table B-3.

B7 Assessment of inflow to the tunnel after grouting

Assessments of the total transmissivity, hydraulic apertures before grouting and water inflow to the tunnel after grouting into each tunnel, TASU and TASP, are shown in Table B-4 and Table B-5, respectively. The assessed inflow after grouting to TASU is 19 l/min and to TASP 18 l/min.

The assessed inflow values are the inflow before excavation of niches. However, most niche positions will be established after the tunnels are completed and aimed to be located in rock volumes with low water bearing capacity. The planned niche for the horizontal deposition (KBS3H) is about 25 m long. Based on the assessed inflows for TASU and TASP, an initial assessment suggested that the water inflow after grouting to this niche would be around 10 l/min.

Table B-3. Properties of the Regular and Stiff grout mixes for the Äspö Expansion Project.

	Cement (kg)	Silika fume (kg)	Melcrete (kg)	Water (kg)	After 10 minutes	After 30 minutes
Stiff Grout mix	40	54.8	2.8	66.8		
Yield stress (Pa)					7.13–8.71	7.35–8.84
Viscosity (mPas)					54.6–68.6	59.9–64.8
Temperature (°C)					13.3–13.5	15.9–17.3
b_{min} / $b_{critical}$ (µm)					35–50/64–120	35–50/64–145
pH					12.8	12.8
Regular Grout mix	40	68.5	3.5	83.5		
Yield stress (Pa)					5.51–6.79	5.39–6.84
Viscosity (mPas)					46.6–54.1	42.1–54.4
Temperature (°C)					13.4–13.7	15.2–15.9
b_{min} / $b_{critical}$ (µm)					35/65–75	35/65–77
pH					12.8	12.8

Table B-4. Assessed transmissivity and hydraulic aperture before grouting and calculated inflow to the TASU tunnel after grouting.

TASU	Before Grouting		After Grouting
Tunnel Section [m]	Transmissivity [m²/s]	Hydraulic aperture [µm]	Total inflow [l/min]
10–20	0	0	0
20–21	4×10^{-9}	18	0
21–24	9×10^{-6}	230	2
24–26	1×10^{-8}	26	2
26–29	0	0	2
29–30	3×10^{-8}	33	3
30–31	3×10^{-8}	35	4
31–33	6×10^{-8}	44	6
33–36	4.3×10^{-7}	95	9
36–38	8×10^{-8}	48	11
38–39	5.6×10^{-7}	91	14
39–42	2.1×10^{-7}	66	16
42–45	2.7×10^{-9}	71	19
45–69	0	0	19

Table B-5. Assessed transmissivity and hydraulic aperture before grouting and calculated inflow to the TASP tunnel after grouting.

TASP	Before Grouting		After Grouting
Tunnel section [m]	Transmissivity [m^2/s]	Hydraulic aperture [μm]	Total inflow [l/min]
20–31	4.3×10^{-7}	83	3
31–34	0	0	3
34–37	1.4×10^{-6}	123	6
37–43	0	0	6
43–44	1.5×10^{-6}	125	9
44–46	0	0	9
46–49	3×10^{-7}	73	11
49–52	2×10^{-7}	60	14
52–55	0	0	14
55–58	1×10^{-7}	57	16
58–61	0	0	16
61–64	7×10^{-8}	45	18
64–73	0	0	18

B8 Grout take

The assessed grout take for each planned grout fan are shown in Table B-6. The expected grout take for TASU and TASP tunnels are about 1900 l and 900 l, respectively. The expected spread of grout varies between 4 m and 14 m, depending on the hydraulic aperture for the representative water-bearing fracture.

It is worth pointing out that the first grout fan in each tunnel is expected to require the most extensive grouting. It is likely that the inflow in each of the first five probe holes in the first grout fans will be large in TASU and TASP. Based on actual inflow in the investigation holes discussed in Section B5, the inflow to individual grout holes may be around 180 L/min and 30 L/min, respectively.

Table B-6. Assessed grout take in TASU and TASP for 20 m long grout fans.

Grout fan		Grout take	
TASU Section [m]	TASP Section [m]	TASU [l]	TASP [l]
10–30	20–40	1750	740
25–45	35–55	80	60
40–60	50–70	0	30
55–70	65–85	60	60

B9 Previous experience from TASQ tunnel

Cement grouting was carried out in connection with excavation of the TASQ tunnel (Emmelin et al. 2004). The grouting campaign consisted of two grout fans, each being around 20 m long. A water inflow of 75 l/min was recorded in a pilot hole that was drilled ahead of the excavation. It was assessed that the pilot hole intercepted six water-bearing fractures and that the hydraulic apertures were about the same size as the largest occurring in TASU and TASP. The water inflow to the tunnel after grouting was approximately 1 l/min over the 20 m tunnel length that was grouted.

B10 Uncertainties

Uncertainties may be large for both the assessed grout take and water inflow after grouting due to underlying assumptions.

It is likely that the assessed inflow to the tunnels after grouting is conservative. In practice inflow may be smaller as the calculation model does not take into account that fractures may be connected and sealing of a water bearing fracture may reduce supply of water to other fractures.

It is likely that the assessed grout take is optimistic and that in practice will be larger. Based on a statistical approach fractures with larger aperture may be encountered. Research has pointed out that fracture apertures are likely to be larger than the hydraulic aperture based on for instance water loss measurements, e.g. Holmberg et al. (2012). In the event that the transmissivity is higher and increases an order similar to a water bearing fracture with a maximum hydraulic aperture, the assessed grout take increases by about 100% in TASU and 50% in TASP.

There is uncertainty whether the required grouting efficiency can be achieved with one grouting round if water bearing fractures with relatively large apertures or hydraulic connected holes will be encountered. These events can be expected to occur and should therefore be handled by the regular grouting strategy and not by a contingency measure. It is recommended that probing is included in the grouting strategy and when high inflows occur, that an initial grouting round including the probe holes only are carried out. Based on the inflow tests it is suggested that an inflow above 20 L/min in one probe hole trigger grouting of the probe holes. After completing the regular grout fan there should be a provision to carry out complementary grout holes if there are grout holes with large grout takes. Complementary holes should be carried out if two holes or more have grout takes equal to the maximum volume of 200 l.

B11 Observations and measures

In accordance with the Project control plan groundwater pressures will be monitored in KA2050A during rock excavation. A reference pressure, which represents “undisturbed” conditions, should be defined at the onset of the project and the drawdown during the excavation works should be checked against this reference pressure. It is deemed reasonable to accept an average drawdown of 3 m between each grout fan, i.e. along 15 m of tunnel excavation. Should the difference between the reference pressure and the groundwater pressure after recovery exceed 3 m or 6 m, depending on if the rock excavation is carried out in both tunnels in parallel, the technical designer should be informed to take decisions whether mitigation measures are required or not. The bases for decision are the RTGC analysis of the executed grouting operations, follow-up of expected grouting performance and time series documentation of the draw-down and recovery processes that is monitored during grouting operations and rock excavation. A contingency plan that summarise indicators, monitoring system, likely causes and proposed mitigation measures are given in Table B-7.

This contingency plan is developed to handle uncertainties that relate to the understanding of the hydrogeological setting and cover extreme behaviour. The contingency plan is developed for and shall be handled by the Client´s supervisor on site. As long as the grouting performance progresses in accordance with the likely behaviour and the grouting strategy the measures contained in contingency plan will not be implemented.

Table B-7. Suggested indicators, monitoring system, likely causes and mitigation measures to be implemented due to occurrence of unacceptable water inflow and/or groundwater drawdown.

Indicator	Monitoring System	Likely cause	Mitigation measure
Inadequate sealing effect	Inflow measurements	Water bearing fracture with large aperture	Execute two grouting rounds in accordance with the specified drill and grout plan
Inadequate sealing effect	HMS shows unacceptable draw-down and/or unacceptable inflow to tunnel is recorded. Geological mapping indicate occurrence of water leakage in fractures after grouting	Water-bearing fractures are connected to a significant extent	Execute grouting using a primary and a secondary grout round
Inadequate sealing effect		Grouted zone is not wide enough. The grout spread is not sufficient	Increase grouting time. Change grout mix to achieve a better penetrability through reduced viscosity
Poor sealing effect in a rock mass with small fracture apertures		High frequency of water bearing fractures with apertures less than the critical required for grout penetration	Change grout mix and grouting method from cement grouting to solution grouting. e.g. silica sol
The quality of the grouting works is not the expected	Contractor documentation or other observations	(Not the scope here)	Execute supplementary grouting. additional holes in the current grouting round or a secondary grout round