Technical Report

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Äspö Hard Rock Laboratory

Characterisation methods and instruments

Experiences from the construction phase

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Foreword

The aim of this report is to present the investigation methods used in the Äspö Hard Rock Laboratory (HRL) during the construction phase. The methods are described and discussed with regard to usefulness for detailed characterisation and modelling of the Äspö rock volume, including the feasibility of the methods for surveys, measurements, sampling and tests in conjunction with tunnelling work, both in connection with drilling and blasting as well as TBM construction. Many of the methods are also discussed in many other reports compiled during the course of the Äspö HRL construction phase, to which relevant references are given in this report. Comments and recommendations given in this report yield important information to the planning of future detailed investigations for a deep repository. The report is also aimed to be a useful guide for further development of investigation methods and instruments.

The evaluation of the methods concerning the usefulness of collected data was directed to the finalisation of Äspö stage goal number one: Verify pre-investigation methodology; demonstrate that the investigations at the ground surface and in boreholes provide sufficient data on essential safety-related properties of the rock at repository level.

In this work valuable contributions were obtained from the principal investigators in Geological-structural model (Geology) – Roy Stanfors; Ground water flow (Geohydrology) – Ingvar Rhén; Groundwater Chemistry – Peter Wikberg; Transport of solutes (Geohydrology) – Ingvar Rhén; Mechanical stability models (or rock mechanics) – Roy Stanfors. The judgement of methods in Chapter 13 is mainly made by the principal investigators.

Contributions to the evaluation of methods were given by a number of specialists in the various fields. Many of their findings and results can be found in TR-reports, Äspö PR-, HRL- and ICR-reports.

In practical handling of the methods underground important contributions and feedback came from the characterisation team; Allan Stråhle, Christian Annertz, Bengt Gentzschein, Robert Gass and Katinka Klingberg.

Contribution to the tunnel surveying procedure came from Johannes Heikkilä. The database SICADA was described by Mats Ohlsson and Ebbe Eriksson. Important contributions to the report came from many other persons at universities, companies and laboratories, all of them in one way or another involved in the investigations. Just to mention a few; Olle Olsson, Gunnar Gustafsson, Ingemar Markström, Göran Nilsson, Gunnar Ramqvist, Eva-Lena Tullborg, Seje Carlsten, Christer Gustafsson, Berndt Johansson, Calin Cosma, Eric Gustafsson, Christer Ljunggren, Ann-Chatrin Nilsson, Göran Nyberg, Stig Jönsson, Valter Didriksson, Bengt Stillborg, Kent Hansson, Lennart Ekman and Jan Roymar.

Many thanks to You all, involved in the contribution to this report.

The main body of the reports covered in this work was published in 1997. The descriptions and the evaluation of the methods are based upon what was published then.

Finally we also want to thank Mikael Erlström who finalised the manuscript, and Anders Lindblom who finalised the illustrations.

Karl-Erik Almén *Leif Stenberg*

Karl-Erik Almén was in charge of instruments for SKB at that period of time and Leif Stenberg was Characterisation manager for the Characterisation Team at Äspö HRL.

Sammanfattning

Rapporten beskriver olika undersökningsmetoder som användes under byggandet av Äspölaboratoriet. Bygget påbörjades under 1990 och avslutades 1995. Undersökningsmetoderna beskrivs avseende utförande, felkällor, osäkerheter och användbarhet i bestämda, analyserade och/eller beräknade parametervärden eller annan typ av geovetenskaplig information. I övrigt kommenteras och diskuteras de olika metodernas praktiska genomförbarhet som är en betydelsefull faktor eftersom huvuddelen av undersökningarna genomfördes parallellt med byggandet av Äspölaboratoriet. Underjordsmiljön i sig själv medför speciella insatser från personalen sida, även för personal som inte direkt var involverade i det aktiva tunnelarbetet i så motto att rätt saker utförs på rätt sätt.

Med referens till de två första etappmålen för Äspölaboratoriet har i synnerhet verifiering av förundersökningarna och förundersökningsmetodernas tillämpbarhet utvärderats genom att jämföra prediktioner baserade på förundersökningsmodeller med data och resultat erhållna från byggskedet och därpå följande geovetenskapliga modeller. En generell utvärdering av förundersökningarna har rapporterats i ett antal rapporter under 1997.

Undersökningsmetoderna utvärderas i denna rapport med hänsyn till deras tillämpbarhet rörande undersökning av de geologiska, geohydrologiska, grundvattenkemiska samt bergmekaniska egenskaperna. Rapporten beskriver vår uppfattning om undersökningsmetoderna efter genomfört byggskede, dvs på samma kunskapsplattform som 1997 års resultat- och modellutvärderingsrapporter.

Utvärderingen av metodernas användbarhet struktureras i enlighet med de nyckelparametrar som användes för modellering och prediktionerna baserade på förundersökningsdata, dvs geologisk strukturmodell, grundvattenflöde (geohydrologi), grundvattenkemi (hydrokemi), transport av lösta ämnen samt mekanisk stabilitet (bergmekanik). I rapporten behandlas speciellt de undersökningsmetoder som använts under byggskedet för karaktärisering av de nyckelparametrar som var kopplade till de prediktioner som sattes upp före byggandet. Några av parametrarna har ändrats något under resans gång medan andra har tillkommit. De senare har direkt bäring på det andra etappmålet för Äspölaboratoriet, att fastställa detaljundersökningsmetodik.

I rapporten diskuteras även metoder som är mer eller mindre kopplade till konceptuella modeller. Flertalet metoder, positionering (t ex koordinatsystem och lägesbestämning av objekt), utvärdering av data och rapportering, provhantering, kemiska analyser av vattenprov, datainsamlingssystem och inlagring i databasen (SICADA) diskuteras översiktligt. Det är viktigt för utvärderingen (i termer av kvalitet och tid för utvärdering) att dessa stödjande funktioner fungerar bra. Till viss del kan uppdatering och förfining av modeller ske separat från undersökningsarbetet men det föreligger alltid ett antal frågor som berör detaljer i modellerna som är viktiga för utformning (design) av tunnel(ar). Detta kräver en snabb uppdatering av modellerna och snabba beslut om hur nya undersökningshål skall borras på bästa sätt. Undersökningsmetoderna som behandlas är också verktygen för att få en bättre integrerad utvärdering av modellerna för nyckelfrågorna; geologisk strukturmodell, grundvattenflöde (geohydrologi), grundvattenkemi (hydrokemi), transport av lösta ämnen samt mekanisk stabilitet (bergmekanik).

Abstract

This report describes the different investigation methods used during the Äspö HRL construction phase which commenced 1990 and ended 1995. The investigation methods are described with respect to performance, errors, uncertainty and usefulness in determined, analysed and/or calculated parameter values or other kind of geoscientific information. Moreover, other comments of the different methods, like those related to the practical performance of the measurements or tests are given. The practical performance is a major task as most of the investigations were conducted in parallel with the construction work. Much of the wide range of investigations carried out during the tunnelling work required special efforts of the personnel involved. Experiences and comments on these operations are presented in the report.

The pre-investigation methods have been evaluated by comparing predictions based on pre-investigation models with data and results from the construction phase and updated geoscientific models. In 1997 a package of reports describe the general results of the pre-investigations. The investigation methods are in this report evaluated with respect to usefulness for underground characterisation of a rock volume, concerning geological, geohydrological, hydrochemical and rock mechanical properties. The report describes out opinion of the methods after the construction phase, i.e. the same platform of knowledge as for the package of reports of 1997. The evaluation of usefulness of the underground investigation methods are structured according to the key issues used for the pre-investigation modelling and predictions, i.e. Geological-structural model, Groundwater flow (hydrogeology), Groundwater chemistry (hydrochemistry), Transport of solutes and Mechanical stability models (or rock mechanics). The investigation methods selected for the different subjects for which the predictions were made are presented. Some of the subjects were slightly modified or adjusted during the construction phase while other (not predicted) subjects or parameters were added (measured/determined) during the construction phase. These added subjects/parameters were more or less directly related to the refinement of detailed characterisation methods, the second stage goal of the Äspö HRL.

Methods for more or less direct observation of features coupled to the conceptual model are also presented. A number of methods such as positional information (i.e. coordinate system and positioning of objects), data evaluation and reporting, sample handling, analysis of water samples, data acquisition and data base system (SICADA) are briefly described. It is, however, essential that these basic systems are working well to be able to achieve a high confidence with the pertinent data of each individual observation and sample.

To some extent the updating and refining of models can be performed “separately” from the excavation work. But there will, however, always be a number of issues concerning details in the models that are important for the design of the tunnel(s). This requires fast updating of the models, which facilitates decision taking concerning e.g. how to drill investigation holes in an optimum way. The methods which are evaluated are also the tools to get better integrated evaluation of models for the key issues “Geological structural model”, “Groundwater flow”, “Groundwater chemistry”, “Transport of solutes” and “Mechanical stability”.

Extended summary

This report describes the different investigation methods used during the Äspö Hard Rock Laboratory (Äspö HRL) construction phase, which commenced 1990 and ended 1995. The investigation methods are described with respect to performance, errors, uncertainty and usefulness in determined, analysed and/or calculated parameter values or other kind of geoscientific information. Moreover, comments related to the practical performance of the measurements or tests are made. The practical aspect is of great importance as most of the investigations were conducted in conjunction with the construction work. In order to conduct the right thing in the right manner, the underground setting often called for special efforts, frequently also by personnel not directly involved in the active tunnelling work.

The pre-investigation methods have been evaluated by comparing predictions based on pre-investigation models with data and results from the construction phase and updated geoscientific models. In 1997 a package of reports describe the general results of the pre-investigations. The investigation methods are in this report evaluated with respect to usefulness for underground characterisation of a rock volume, concerning geological, geohydrological, hydrochemical and rock mechanical properties, based on our status of knowledge and experiences as of 1997.

The evaluations of usefulness of the different investigation methods are structured in the report according to the key issues used for the pre-investigation modelling and predictions i.e.:

- Geological-structural model.
- Groundwater flow (hydrogeology).
- Groundwater chemistry (hydrochemistry).
- Transport of solutes.
- Mechanical stability model (or rock mechanics).

The investigation methods were used for monitoring, detecting and measuring the different subjects on which the predictions were presented before the construction phase was started. Some of the subjects were slightly modified or adjusted during the construction phase while other (not predicted) subjects or parameters were added (measured/determined) during the construction phase. These added subjects/parameters were more or less directly related to the refinement of detailed characterisation methods, the second stage goal of the Äspö HRL.

Methods for more or less direct observation of features coupled to the conceptual model are also presented. A number of methods such as positional information (i.e. coordinate system and positioning of objects), data evaluation and reporting, sample handling, analysis of water samples, data acquisition and data base system (SICADA) are briefly described. It is, however, essential that these basic systems are working well to be able to achieve a high confidence with the pertinent data of each individual observation and sample.

To some extent the updating and refining of models can be performed "separately" from the excavation work. But there will, however, always be a number of issues concerning details in the models that are important for the design of the tunnel(s). This requires fast updating of the models, which facilitates decision taking concerning e.g. how to drill investigation holes in an optimum way. The methods which are evaluated are also the tools to get better integrated evaluation of models for the key issues "Geological structural model", "Groundwater flow", "Groundwater chemistry", "Transport of solutes" and "Mechanical stability".

Geological structural model

For the subject *lithological units, rock composition* and *rock boundaries* the methods used were:

- Geological mapping
- Probe boreholes and percussion boreholes (measurements while drilling, MWD)
- TV-logging
- Core boreholes – core logging
- Geophysical borehole logging

For the subject *rock type* characteristics the methods used were:

- Geological mapping
- Core boreholes – core logging
- Geological analysis of rock samples
- Geological analysis of rock samples and mineralogical investigation of fracture fillings

For the subject *small scale fracturing* the methods used were:

- Geological mapping
- Core boreholes – core logging
- TV-logging
- Geophysical borehole logging

For the subject *major fracture zones, minor fracture zones* and single *open fractures* the methods used were:

- Geological mapping
- Core boreholes – core logging
- Percussion boreholes (drilling and measurements while drilling)
- TV-logging
- Geophysical borehole logging
- Radar methods
- Seismic methods

The most useful method for updating the geological model is continuous geological mapping in combination with scan-line mapping of minor fractures. This mapping between every round gives very good information on rock composition, rock boundaries, structures and fracture zones on different scales. In addition, underground core drilling is the best method for investigating the extension and character of fracture zones outside the tunnel. Percussion boreholes provides also additional geological information outside the tunnel when combined with TV (BIPS) logging and geophysical borehole logging.

Borehole radar and tunnel radar measurements are useful methods for detecting and orientating, in particular minor fracture zones and single open fractures.

The Vertical Seismic Profiling (VSP) and Horisontal Seismic Profiling (HSP) methods were found to be useful as a complement to the tunnel radar data for determination orientation of fracture zones.

Groundwater flow (hydrogeology)

For the subject *hydraulic conductivity, water-bearing zone* and *conductive structures* the methods used were:

- Measurements during probe hole drilling
- Pressure build-up tests in probe holes
- Measurements during drilling of investigation holes
- Pressure build-up tests in investigation holes (single packer and double packer system)
- Groundwater monitoring during drilling and tunnelling
- Flow-meter logging
- Interference tests

For the subject *boundary conditions* and *pressure in the rock volume* the methods used were:

- Monitoring in surface boreholes
- Monitoring in tunnel boreholes

For the subject *flow into tunnel* the methods used were:

- Water flow into tunnels and shafts (Dams (collecting the water flow along the tunnel) and weirs (measurement of the flow rate))
- Water flow in pipes
- Vapour transport by the ventilation air (temperature, humidity, air velocity)
- Electrical conductivity (of the water flowing into tunnels and shafts)

For the subject *groundwater flux* the methods used were:

- Groundwater flow measurements (Dilution method)

For the subject *point leakage* the methods used were:

- Hydrogeological mapping

For the subject *disturbed zone* the methods used were:

- Monitoring in probe holes of water pressure
- Water flow into tunnels
- Pressure build-up tests in probe holes

Most of the applied methods characterising subject related to groundwater flow are considered useful for the updating and refining of the hydrogeological model.

Improvements of the applied methods concerning measurement of water flow in pipes, vapour transport by the ventilation air (temperature, humidity, air velocity) and electrical conductivity are discussed in the report.

There are also improvements in methodology that have to be considered when measuring, water flow into the tunnel, flow logging and measurements of large drawdown. The delay of construction of dams and installation of weirs for the flow measurements was a problem for part of the construction period. The monitoring system that should be in operation for a long time should also be designed for that. Possibly the monitoring system shown in this report has to be improved in that respect. There are also alternatives of how to sample hydrogeological (and groundwater chemical) information that should be analysed.

The suggested alternative (long investigation holes ahead of tunnel face) has positive as well as negative sides compared to the main strategy (short probe holes) used during the excavation. The performance of hydraulic tests can probably also be improved. A great improvement would be if a flexible test-rig is constructed that shorten the time for set-up and measurement. It is recommended that a few standardised investigations should be performed in a consistent manner over an entire borehole or along a tunnel. Specially designed tests may in addition be conducted in, for example, parts of the borehole where e.g. a hydraulic conductor domain is assumed to occur. This flexibility increases the possibility to perform re-interpretations and modifications to the models made. Grouting is another obstacle that has to be considered when designing the investigation programme. Most of the applied methods require un-disturbed (un-grouted) rock volumes as to give useful data for the hydrogeological model. However, grouting is occasionally needed to complete the drilling operation. In such a case a minimum test program must be performed before grouting. The grouting technique used over the borehole sections with high flow rates and/or low stability can also probably be improved. On the other hand, mapping of grouted fractures gives very useful information of the active hydraulic system.

High water flows in boreholes occur at several sites and the investigation strategy must take that into account. Drilling and tests should be performed in such a way that the problems for the contractor are minimised. The testing methods (most of them at least) must also be feasible in boreholes with high flow rates.

From the hydrogeological point of view the hydraulic tests made during the excavation with the TBM was more or less a failure. If TBM is to be used for the excavation it must be better designed for drilling investigation holes and hydraulic testing. Drilling long cored boreholes ahead of the TBM may be a solution but still the testing possibilities from the TBM should be improved considerably.

The measurement frequency concerning monitoring of the water pressures in space and time is judged to be mainly sufficient. It would, however, been better to have some kind of more reliable measurements of the natural conditions. To some extent the natural conditions were disturbed by the investigation itself. It is also likely that the investigations within a regional area have to be somewhat more extensive than what was the case for the Äspö HRL. This would greatly improve the confidence of the boundary and initial hydraulic conditions, groundwater chemical characteristics and properties of the rock mass in a regional context.

Groundwater chemistry (hydrochemistry)

For the subject *groundwater chemistry in major fracture zones* the methods used were:

- Sampling within the documentation programme
- Sampling within the monitoring programme

For the subject *hydrochemistry in low conductive rock* no specific methods were used except for the packer system.

Quality changes in e.g. the REDOX experiment is discussed. The instrumentation was simple, robust and useful for the purpose. The flexibility was good which was needed as the focus of investigation changed during the duration of the experiment.

Groundwater samples were collected from probe holes and drips in the tunnel roof. None of these methods turned out to give valuable information, since the hydraulic disturbance by the tunnel had already played a significant role on the sample representativity, i.e. un-disturbed conditions. Based on these experiences improvements could be made. Useful methods are:

- groundwater sampling during drilling of long pilot holes,
- arrangements for sampling of probe holes in the tunnel.

The results from the sampling, in the pilot holes give the unperturbed conditions while the probe hole data is used to give the changes caused by the tunnel.

Transport of solutes

For the subject *flow paths* and *arrival time* the methods used were:

- Large scale tracer tests

For the subject *saline interface* the methods used were:

- Sampling within the documentation programme
- Sampling within the monitoring programme
- Monitoring in surface boreholes – electrical conductivity
- Flow into tunnel – electrical conductivity

For the subject *natural tracers* the methods used were:

- Sampling within the documentation programme
- Sampling within the monitoring programme
- Special sampling programme

The methods involving measurements of flow into the tunnel and monitoring in surface boreholes are presented in the report, as well as discussions concerning improvements of methods, as the method of measuring the electrical conductivity.

A few attempts were made during the pre-investigation and construction phases to estimate the flow porosity of the rock mass. For example, prior to construction a combined long-term pumping and tracer test (LPT-2) was conducted to test the hydraulic connectivity of hydraulic conductors and to derive estimates on flow porosity. During the construction phase some efforts were directed to the use of other types of natural tracers as well as to derive transport parameters for non-sorbing transport. A large scale tracer test was performed in fracture zone NE-1 before the tunnel was excavated through the zone in order to estimate the flow porosity and with the purpose to get information useful for the design of the grouting operations.

The groundwater flow and chemistry have been carefully followed during the construction of the HRL tunnel. The experiences from this work indicate that there have been major changes of the conditions due to the tunnel excavation and inflow to the tunnel. It is therefore not possible to observe the undisturbed condition and the changes thereafter in short probe holes as done in the HRL tunnel. Long probing holes for each tunnel leg may be useful for future investigations, as the means to investigate the dynamic groundwater conditions during an excavation. Short probe holes can be used for sampling when the long time changes around the constructed tunnel are to be monitored.

It is important for modelling purpose to have a sampling strategy that gives a reasonable number of points in space where time series are established for natural conditions as well for the construction phase of the important chemical constituents.

A few deep boreholes for sampling of groundwater and performing hydraulic tests are essential to support the modelling of transport of solutes, but also groundwater flow and groundwater chemistry. It is not sufficient to just take samples in boreholes close to the surface and in boreholes from the tunnel level.

Mechanical stability (rock mechanics)

For the subject *rock quality* the methods used were:

- Tunnel mapping (RMR)

For the subject *rock stress and stability* the methods used were:

- Rock stress measurements, overcoring method
- Stability observation

Within the subject *mechanical characteristics* and *fracture surface properties* the parameters determined and methods used were:

- Uniaxial compressive strength   Unconfined compressive test and Empirical references
- Elastic moduli   Unconfined compressive test and Empirical references
- Poisson's ratio   Unconfined compressive test and Empirical references
- Brittleness ratio   Unconfined compressive test and Empirical references

The most useful method for updating the Rock Quality subject of the model is mapping of the RMR- parameters in conjunction with the geological tunnel mapping after each round. Mapping of fracture zones was used for updating of rock quality on different scales.

Rock stress measurements, by use of the overcoring method, were valuable to provide information on the variation of rock stresses in the rock mass.

Observations of rock burst problems like cracking and tendency of spalling is the best method to indicate stability problems due to high rock stresses.

Contents

1 Introduction

1.1 Background

An extensive research programme was initiated by SKB (the Swedish Nuclear Fuel and Waste Management Co) in 1977 to demonstrate the suitability of using deep geological formations for the disposal of high-level nuclear waste. The Swedish concept for final disposal involves an excavated repository at a depth of about 500 m in crystalline rock, see Figure 1-1. The spent fuel will be encapsulated in steel/copper canisters, which will be placed in deposition holes drilled in a system of tunnels. Blocks of swelling bentonite clay will surround the canisters in the holes. Upon sealing of the repository the tunnel galleries will be backfilled with a mixture of crushed rock and bentonite.

The geoscientific research being conducted by SKB concerns crystalline rock and the proposed disposal concept. The most important properties of the rock volume hosting a nuclear waste repository are /SKB, 1991/:

- Long-term mechanical stability.
- Long-term chemical stability.
- Low ground-water flow and radionuclide transport capacity from the repository up to the biosphere.

During the period 1977-1986, study sites were investigated in order to characterise different rock types with a view to waste disposal. These study site investigations only involved measurements from the surface and in boreholes drilled from the surface.

During the period 1977-1992, research on rock properties and development of underground investigation methods were carried out in the Stripa Mine /Olsson et al. 1992/.

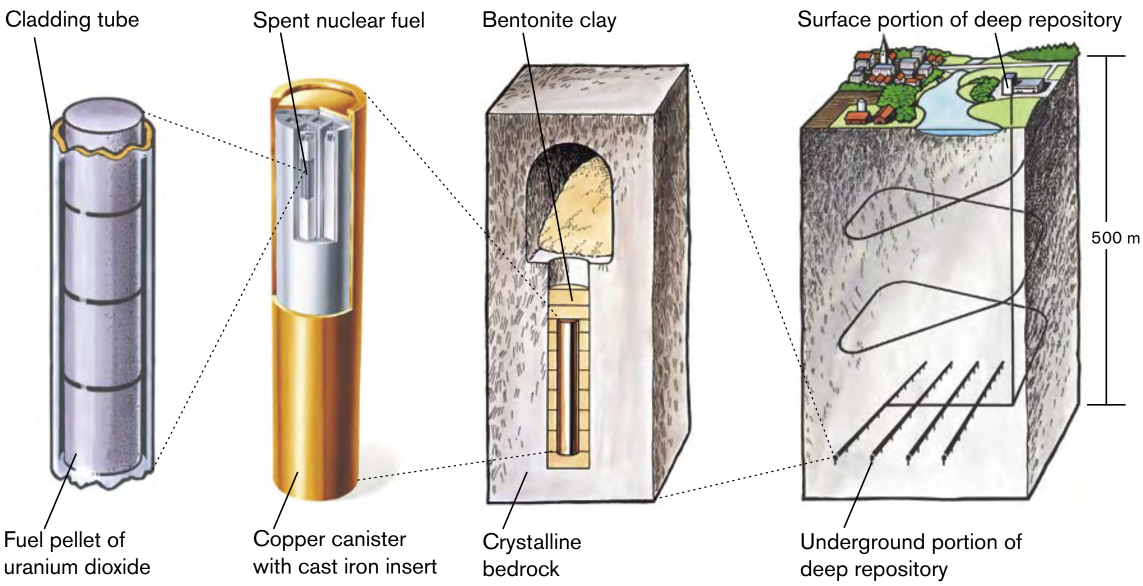


Figure 1-1. *Conceptual Deep Repository Design.*

The need to compare directly the results obtained from surface and borehole investigations by systematic observations from shafts and tunnels down to the depth of a deep repository was the main object for the Äspö Hard Rock Laboratory (HRL).

The results of study site investigations and the research carried out in the Stripa Mine and in the Äspö HRL will form a platform of knowledge with regard to geological characterisation on which future site investigations and detailed investigations for the deep repository will be based.

1.2 The Äspö Hard Rock Laboratory

1.2.1 General

The Äspö HRL is an important part in the work of developing a deep repository and adjoining testing methods for investigating and licensing a suitable site. The plan to build an underground rock laboratory was presented in Programme 86 /SKB, 1986/. In the autumn of 1986, SKB initiated the field work for the siting of the underground laboratory in the Simpevarp area of the municipality of Oskarshamn. At the end of 1988, SKB decided to site the laboratory on southern Äspö, about 2 km north of the Oskarshamn Nuclear Power Plant, see Figure 1-2. Construction work commenced in the autumn of 1990 and was finished in 1995.

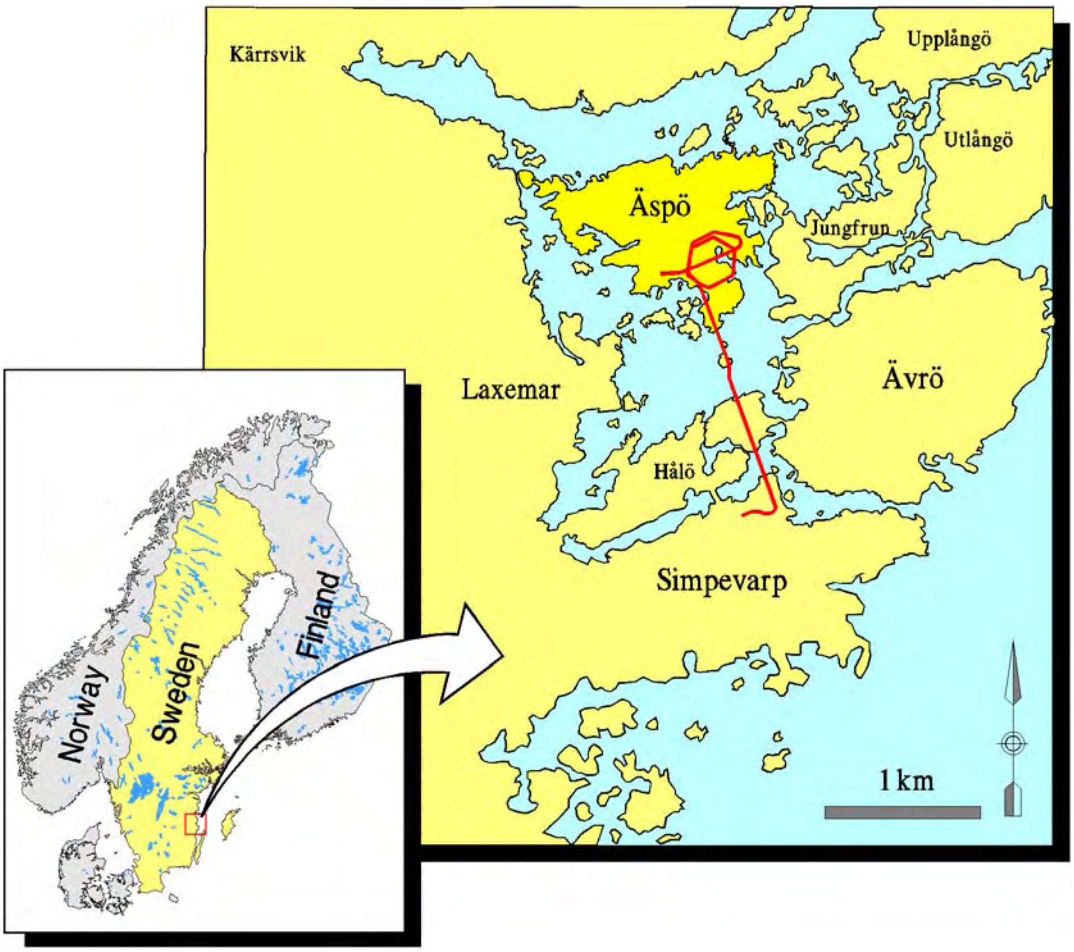


Figure 1-2. Location of the Äspö Hard Rock Laboratory.

The Äspö HRL was designed to meet the projected needs of the planned research, development and demonstration activities. The underground part takes the form of a tunnel from the Simpevarp peninsula to the southern part of the island of Äspö, see Figure 1-3. Below Äspö, the tunnel runs in two turns down to a depth of 450 m. Total length of the tunnel is 3,600 m. The first part of the tunnel was excavated using the drill-and-blast technique. The last 400 m were excavated by a tunnel boring machine (TBM) with a diameter of 5 m. The underground excavations are connected to the surface facilities by a hoist shaft and two ventilation shafts.

The work at the Äspö HRL was divided into three phases: the pre-investigation phase, the construction phase and the operating phase. The pre-investigation phase, 1986–1990, involved siting of the Äspö HRL. The natural conditions in the bedrock were described and predictions were made with respect to the hydrogeological and other conditions that would be observed during the construction phase, /Gustafson et al. 1988/, /Gustafson et al. 1989/, /Wikberg et al. 1991/ and /Gustafson et al. 1991/. Planning for the construction and operating phases was also carried out.

During the construction phase, 1990–1995, extensive investigations, tests and experiments were synchronously carried out with the civil engineering activities, mainly to check the reliability of the pre-investigations. The tunnel was excavated to a depth of 450 m and construction of the Äspö Research Village was completed and taken into service during the summer of 1994. The underground civil engineering works were mostly completed in the summer of 1995.

The operating phase began in 1995. A programme for this phase was presented by SKB in 1995 /SKB, 1995/.

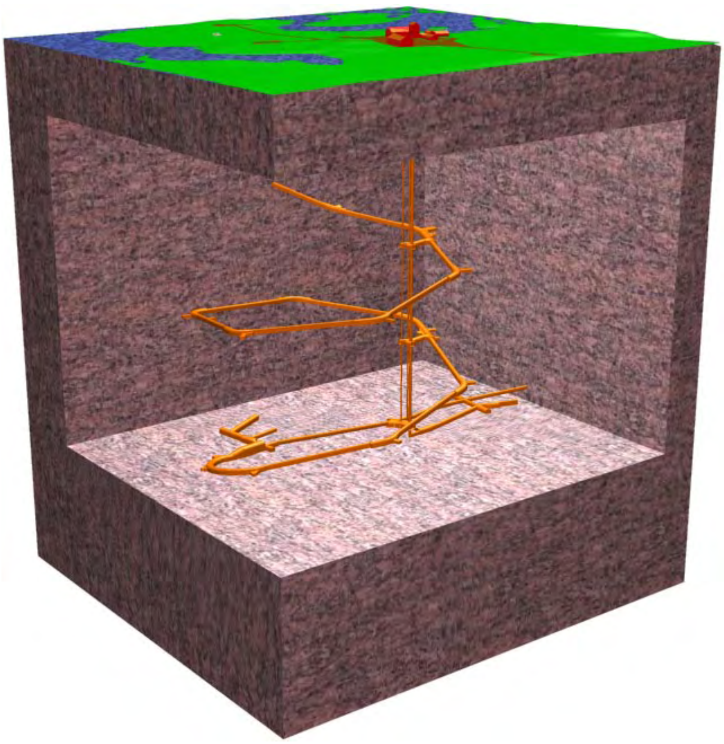


Figure 1-3. *General layout of the Äspö Hard Rock Laboratory (as of 1998).*

1.2.2 Goals

One of the main motives for SKB's decision to build the Äspö HRL was to provide an opportunity for research, development and demonstration in a realistic and undisturbed bedrock environment down to the depth planned for a future deep repository.

Main goals

The main goals of the research and development work at the Äspö Hard Rock Laboratory (HRL) are to:

- Test the quality and usefulness of different methods for characterising the bedrock with respect to conditions of importance for a final repository.
- Refine and demonstrate methods that can adapt a final repository to the local properties of the rock in connection with planning and construction.
- Collect material and data of importance concerning safety of the final repository and for confidence in the quality of the safety assessments.

Stage goals

The following stage goals are guiding the activities at the Äspö HRL (after revision of /SKB, 1995/):

1. Verify pre-investigation methodology.
   - Demonstrate that the investigations at the ground surface and in boreholes provide sufficient data on essential safety-related properties of the rock at repository level.
2. Finalise detailed characterisation methodology.
   - Refine and verify the methods and the technology needed for characterisation of the rock in the detailed characterisation of a site.
3. Test models for description of the barrier function of the rock.
   - Refine and test at repository depth methods and models for describing groundwater flow, radionuclide migration and chemical conditions during the repository's operating period and after closure.
4. Demonstrate the technology for and function of important parts of the repository system.
   - Test, investigate and demonstrate on a full scale different components of importance for the long-term safety of a deep repository system and show that high quality can be achieved in the design, construction and operation of system components.

1.3 Approach for verification of pre-investigations

The investigations during the construction phase are related to stage goals 1 and 2 (Verify pre-investigation methodology and Finalise detailed characterisation methodology). The approach for verification of pre-investigation methodology in the Äspö HRL is shown below, see also Figure 1-4. The verification activities are also related to the second stage goal.

- Pre-investigations were conducted from the surface and in boreholes drilled from the surface using the best available methodology /Stanfors et al. 1991/ and /Almén and Zellman, 1991/.
- Collected data were analysed and evaluated and geoscientific models, on different scales, were developed /Wikberg et al. 1991/.

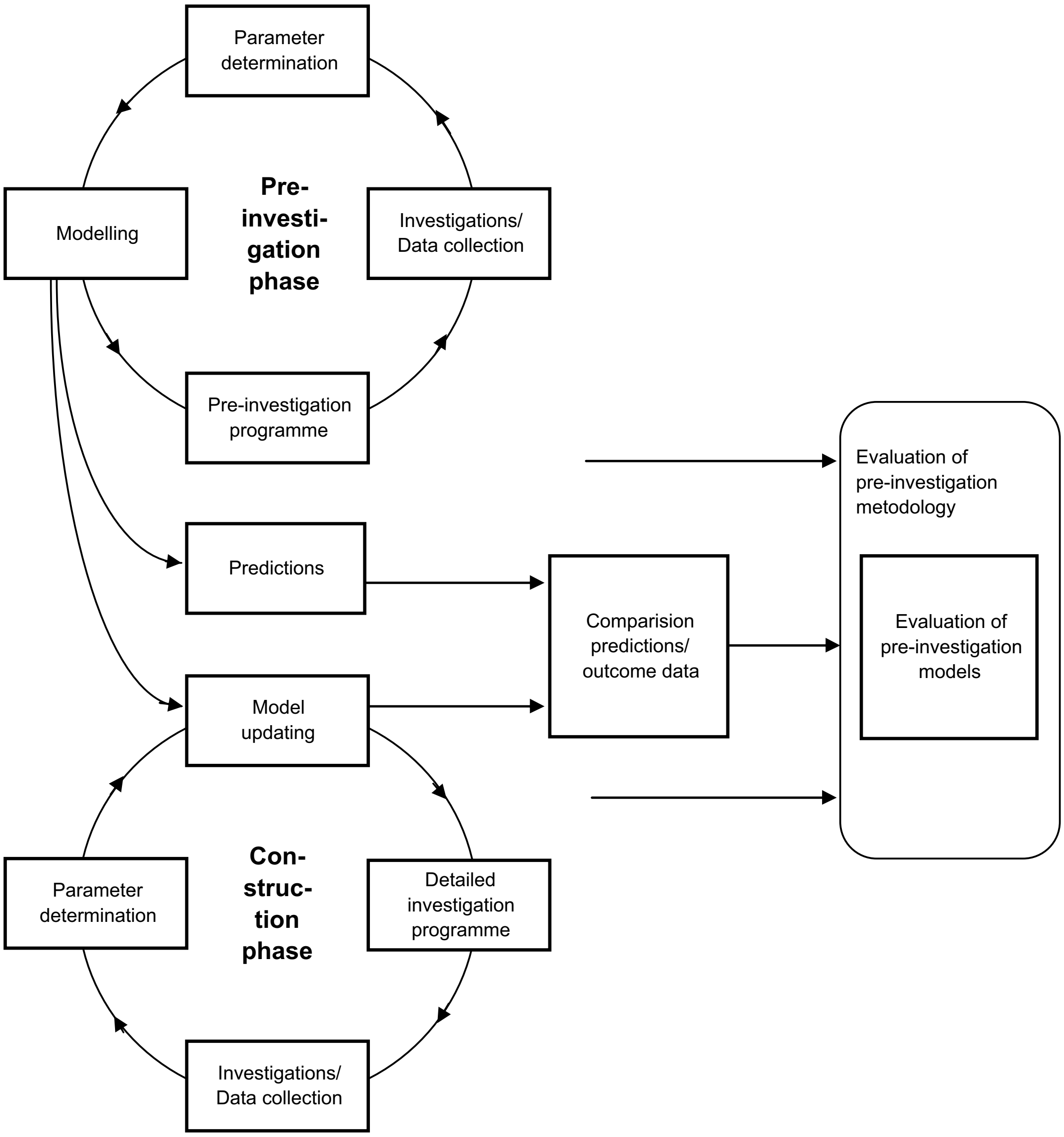


***Figure 1-4.** Approach for verification of pre-investigation methodology.*

- Based on these models and for a given tunnel layout, predictions were made for a number of subjects (features, parameters etc) related to key issues of relevance to the design, performance and safety of a deep repository for nuclear waste, see Table 13-1 /Gustafson et al. 1991/.
- Tunnel documentation, detailed characterisation and monitoring were carried out during the construction of the Äspö HRL, see Figure 1-5 /Stanfors et al. 1997a/.
- Collected data and parameters determined for the subjects were compared with the previously made predictions and the pre-investigation models were validated /Stanfors et al. 1997b/ and /Rhén et al. 1997b/.
- The geoscientific models were updated, now also based on the additional information from the investigations from the construction phase /Rhén et al. 1997c/.
- Finally the pre-investigation strategies and methods underwent evaluation (verification) /Rhén et al. 1997a/.
- The general information flow and reports for the verification process during the construction phase are shown in Figure 1-6.

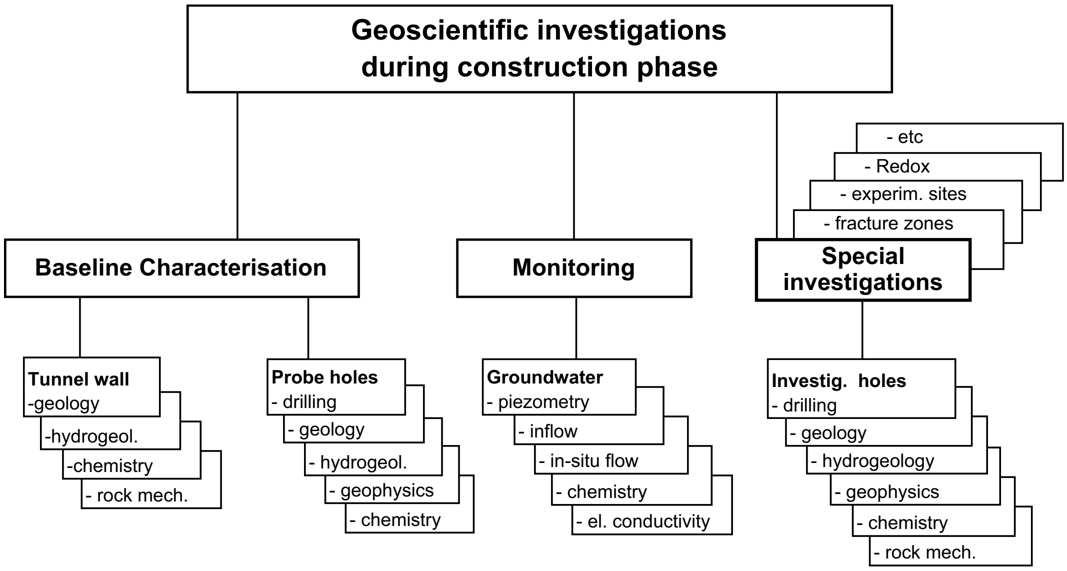


Figure 1-5. *Outline of investigations performed during the Äspö HRL construction phase.*

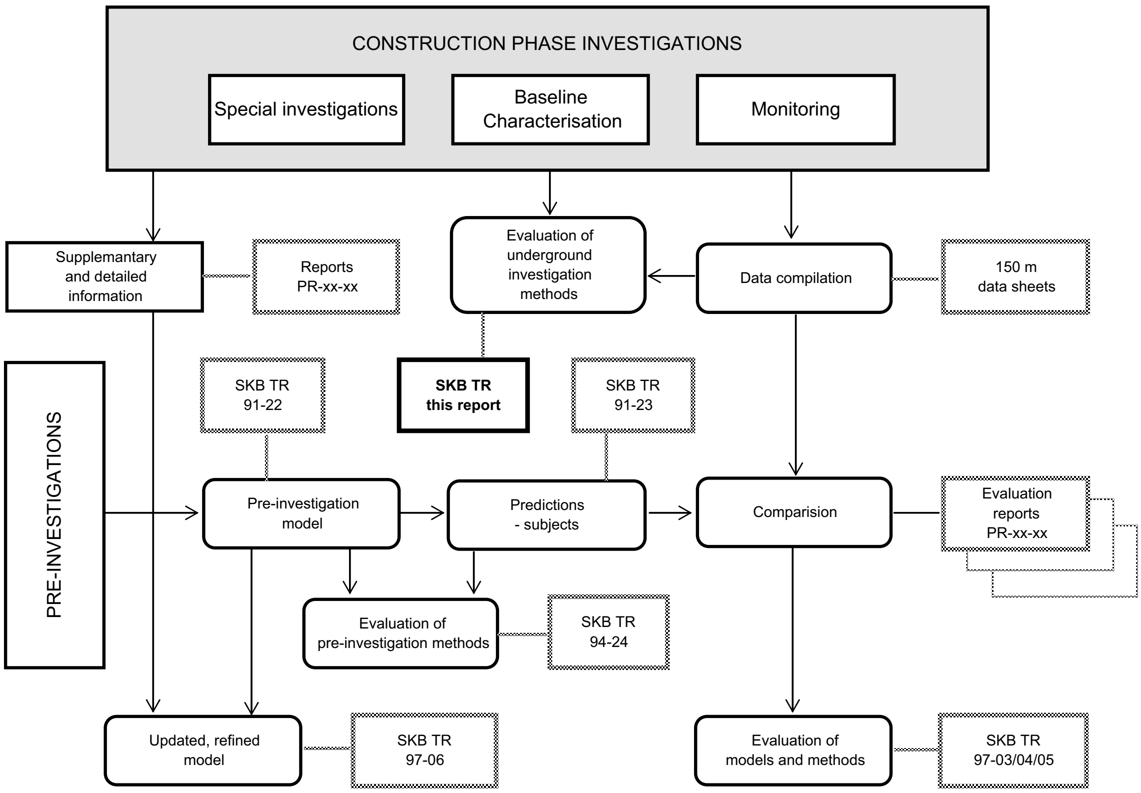


Figure 1-6. *General information flow and reports produced for verification of pre-investigations.*

1.4 Outline of this report

The aim of this report is to present all the investigation methods used during the construction phase (1990–1995) of the Äspö HRL. The methods are described and discussed with regard to usefulness for detailed characterisation and modelling of the Äspö rock volume (as they were evaluated 1997), not less the feasibility of the methods for surveys, measurements, sampling and tests in conjunction with tunnelling work. Many of the methods are also discussed in many other reports compiled during the course of the Äspö HRL construction phase, to which relevant references are given in this report. Comments and recommendations given in this report yield important information to the planning of future detailed investigations for a deep repository. The report is also aimed to be a useful guide for further development of investigation methods and instruments.

The outline of the report is as follows:

Chapter 1 gives a general introduction to the Äspö HRL Project.

Chapter 2 describes the construction phase with regard to stages and techniques of construction and the geoscientific investigation programme carried out together with the construction work.

In Chapters 3-12, the different investigation techniques are presented and discussed as follows:

Chapter 3	Positional information
Chapter 4	Characterisation of the tunnel
Chapter 5	Drilling and related activities
Chapter 6	Geological borehole investigations
Chapter 7	Geophysical borehole investigations
Chapter 8	Hydrogeological borehole investigations
Chapter 9	Hydrochemical borehole investigations
Chapter 10	Rock mechanical investigations
Chapter 11	Groundwater monitoring
Chapter 12	Database system

These chapters are structured in roughly the same way. First a general description of the specific method (or group of methods), including the purpose of the method is given, followed by a description of instrument and measurement methodology. The accuracy of the method is discussed and if possible quantified. Subsequently, comments on the technical performance of the investigation method are given in each of these chapters, sometimes with recommendations for improvements.

In **Chapter 13**, the usefulness of the investigation methods for geological characterisation is discussed based on the experience gained during the construction phase, i.e. from the evaluation/modelling team's point of view as presented in the package of reports 1997. Chapter 13 also includes brief comments and/or recommendations.

2 Outline of the construction phase

2.1 Overview of the construction work

Construction of the Äspö tunnel system was carried out in two main stages. The first stage involved excavation of tunnel and shaft down to 330 m depth and the second stage involved excavation of tunnel and shaft from 330 m down to the 450 m level. The tunnelling during the first stage was done by conventional drill-and-blast technique while a Tunnel Boring Machine (TBM) was used for part of the second stage. The principal contractor for the first stage was SIAB and SKANSKA for the second stage /SKB, 1996/.

An overview of the tunnel layout, as per autumn 1998, is shown in Figure 2-1. The underground construction on Äspö consist mainly of the A tunnel (named TASA) and the shaft. The minor tunnels TASG and TASK were excavated during the operating phase.

The tunnelling work started on the Simpevarp peninsula with an inclined and straight "access" tunnel to Äspö. The tunnel gradient is approximately 14% (8°). The first 500 m provided an opportunity to test construction and investigation procedures, but also (and equally important) to establish coordination routines between the construction and characterisation teams. At about 1,300 m tunnel length the fracture zone NE1 (regarded as a boundary to the Äspö rock volume) was passed at approximately 180 m depth. The straight tunnelling continued to about 1,500 m length where the general tunnel layout changed to a hexagonal tunnel spiral. Each leg in the tunnel spiral had a length of about 150 m. The lowermost approximately 600 m of the tunnel continues under the tunnel spiral area and extends 200 m further out to the west. The final 400 m of the tunnel was full-face bored with a TBM.

The drill-and-blast tunnel profile is typically square with a cross-sectional area of 25 m^2, except in areas where the tunnel bends in the spiral section. Here the area is in the range of 45 m^2. A typical drill-and-blast round involved drilling of 4.5 m long blast holes distributed at the tunnel face using a three-boom drilling machine, see Figure 2-2. The holes were then charged with explosives according to the drill-and-blast plan developed during the first 500 m of tunnelling and subsequently slightly modified. Blasting itself was carried out in a series of explosions (interval blasting), starting in the central holes and ending up in the periphery approximately five seconds after the first explosion. The rock debris was then mucked out and hauled by truck out of the tunnel. In this manner, typically 4.2 m of new tunnel was excavated during each round. After each round the newly exposed tunnel walls were characterised by the characterisation team, but before they were allowed to enter the tunnel section it had to be secured and cleaned (scaling of loose rock blocks).

Prior to every fourth round, longer probe holes were drilled in order to allow a rough inspection of the rock quality of the following 20 m. The 20 m long holes were of interest for the investigation programme and had to be kept intact after tunnelling. They were drilled at a slight angle (approximately 20°) to the tunnel extension line, one on each tunnel side, see further description in Section 5.2. In tunnel sections where the rock quality was expected to be bad or where the tunnel unexpectedly passed through bad rock, the normal probe drilling programme was changed to suit the situation, i.e. drilling for pre-grouting in case of water inflow.

The tunnel walls locally had to be supported due to poor rock quality by rock bolts or by a layer of shotcrete for safety reasons.

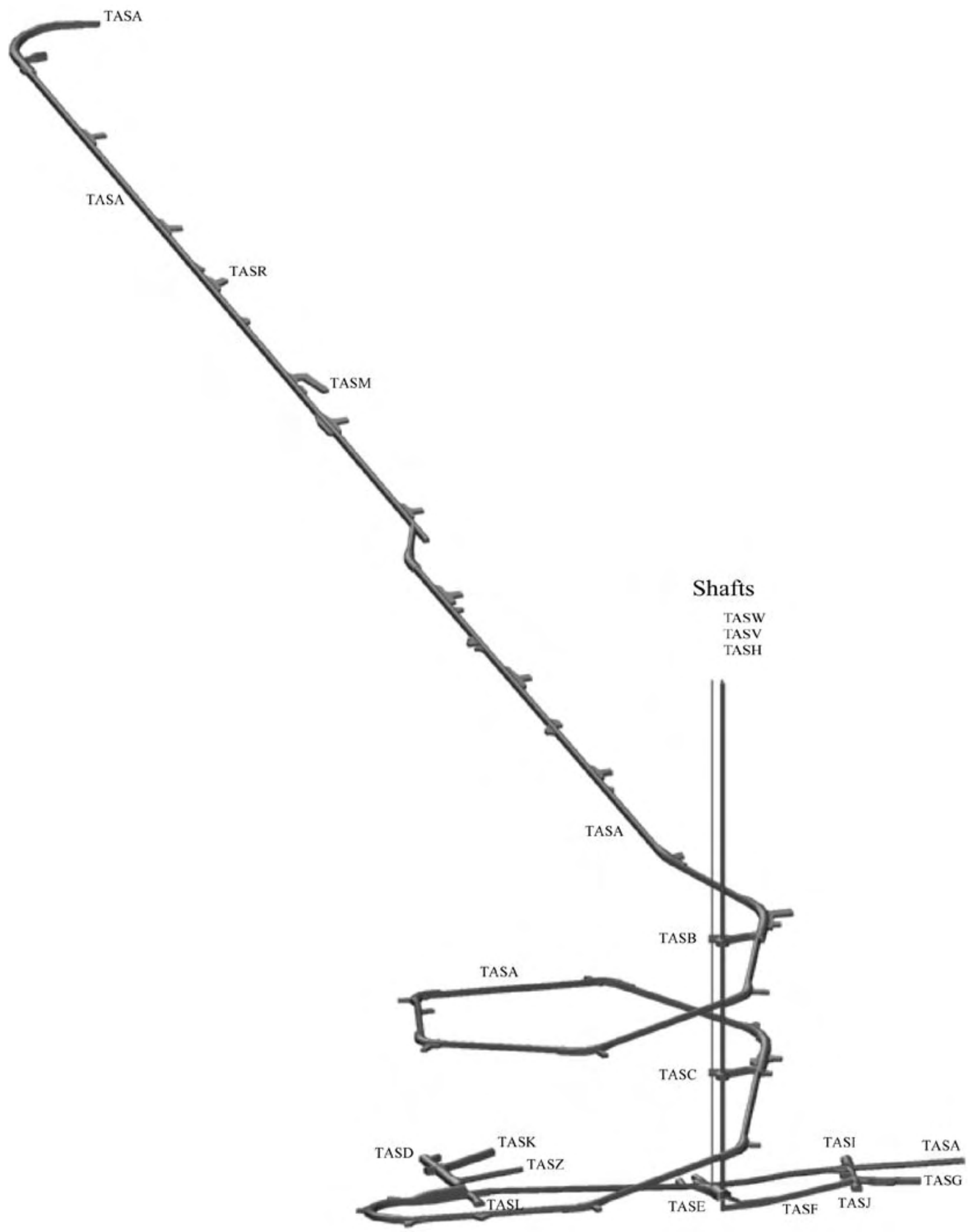


Figure 2-1. *Overview of the Äspö tunnel layout, autumn 1998. (TASA is the ID code for the main tunnel A.) The tunnel is viewed from a point located NNE of Äspö Island.*

The TBM tunnelling resulted in a circular tunnel of 5 m diameter, see Figure 2-3. Probe hole drilling every 20 m was performed in about half of the TBM tunnel. The main reason for these holes was for the investigation programme and to test the feasibility of drilling this kind of slightly angled holes from the front of a TBM. A 200 m long cored probe hole was initially drilled along the first part of the TBM tunnel, within the circumference of the tunnel.

Figure 2-2. The drill rig used for drilling of blast holes and probe holes.

Figure 2-3. The TBM machine in the assembly hall before TBM tunnelling start.

In addition to excavation of the main tunnel, a number of tunnel niches and short side tunnels were excavated for various reasons /Stanfors et al. 1997a/, /Markström and Erlström, 1996/ and /Markström, 1997/. Examples of nisches and side tunnels are (see also Figures 2-1 and 2-4):

- A side tunnel for visitor information (at c 100 m tunnel length).
- A niche for redox experiments in boreholes intersecting a fracture zone (at c 500 m tunnel length).
- A side tunnel for testing of methods for investigation and pre-grouting in water-bearing fracture zones (at c 700 m tunnel length).
- Short access tunnels to the shafts (at c 1,650, 2,600 and 3,400 m tunnel lengths).
- A large widening of the tunnel for assembling the TBM (at c 3,150 m tunnel length).
- A 50 m drill and blast tunnel parallel to the TBM tunnel, for excavation disturbance investigations (ZEDEX) (at c 3,200 m tunnel length).
- A side tunnel going down to the bottom of the Äspö underground facility, with the bottom pump sump and shaft bottom at the 460 m depth (at c 3,500 m tunnel length, beyond the bottom of the main tunnel). This side tunnel also act as a water reservoir in case of pump failure.

The most important underground constructions beside the tunnel are the shafts, which connect the tunnel system with the Äspö Research Village at the surface. The shafts are connected to the main tunnel through short side tunnels at three levels: 220 m, 330 m and 450 m. There are three shafts, one for the elevator and two for ventilation. Each one being raise bored in three steps: 220 m to the surface, 330 m to 220 m and 450 m to 330 m, in connection to that the main tunnel reached these levels, see Figure 2-5. The main shaft for the elevator has a diameter of 3.8 m, while the ventilation shafts are 1.5 m in diameter.

Special construction work was performed as to be able to collect, measure and pump in-flowing water. In selected tunnel sections, along the straight access tunnel and at the end of each tunnel leg in the spiral, water dams were constructed to collect all water inflow along the tunnel interval from the previous dam, see Figure 2-6. As water flows not only on the tunnel floor but also in the blast-damaged zone just below the tunnel floor, the dam had to be carefully constructed. The floor had to be carefully cleaned from rock debris so that the grouting could efficiently seal against the undisturbed floor. Holes were also drilled in the floor for installation of anchor rods and for injection of grout. A drain pipe was installed to collect all water and to lead it to a Thomson weir for flow measurement, se further description in Section 11.4. Special pump sumps for the drainage water were excavated at four levels in the tunnel (Figure 11-6).

Additional work carried out in the tunnel included the installation of electrical equipment (including lighting), water supply pipes to the tunnel and to the Äspö Research Village, drainage pipes and brackets for utility lines (electrical cables, signal cables, hydraulic lines, etc), see Figure 2-7.

The general aim was, as far as possible, to use “standard” construction techniques and routines, and to integrate the baseline geoscientific characterisation as smoothly as possible. Occasionally, however, more research-related construction activities were also conducted. One example was during the passage of fracture zone NE-1, where the huge inflow of brackish water, in combination with high pressures, caused severe problem for the pre-grouting work, which consumed a lot of time before it was solved /Bäckblom and Svemar, 1994/ and /Rhén and Stanfors, 1993/.

More detailed descriptions of the tunnel construction methodology are given in /Hamberger, 1993/.

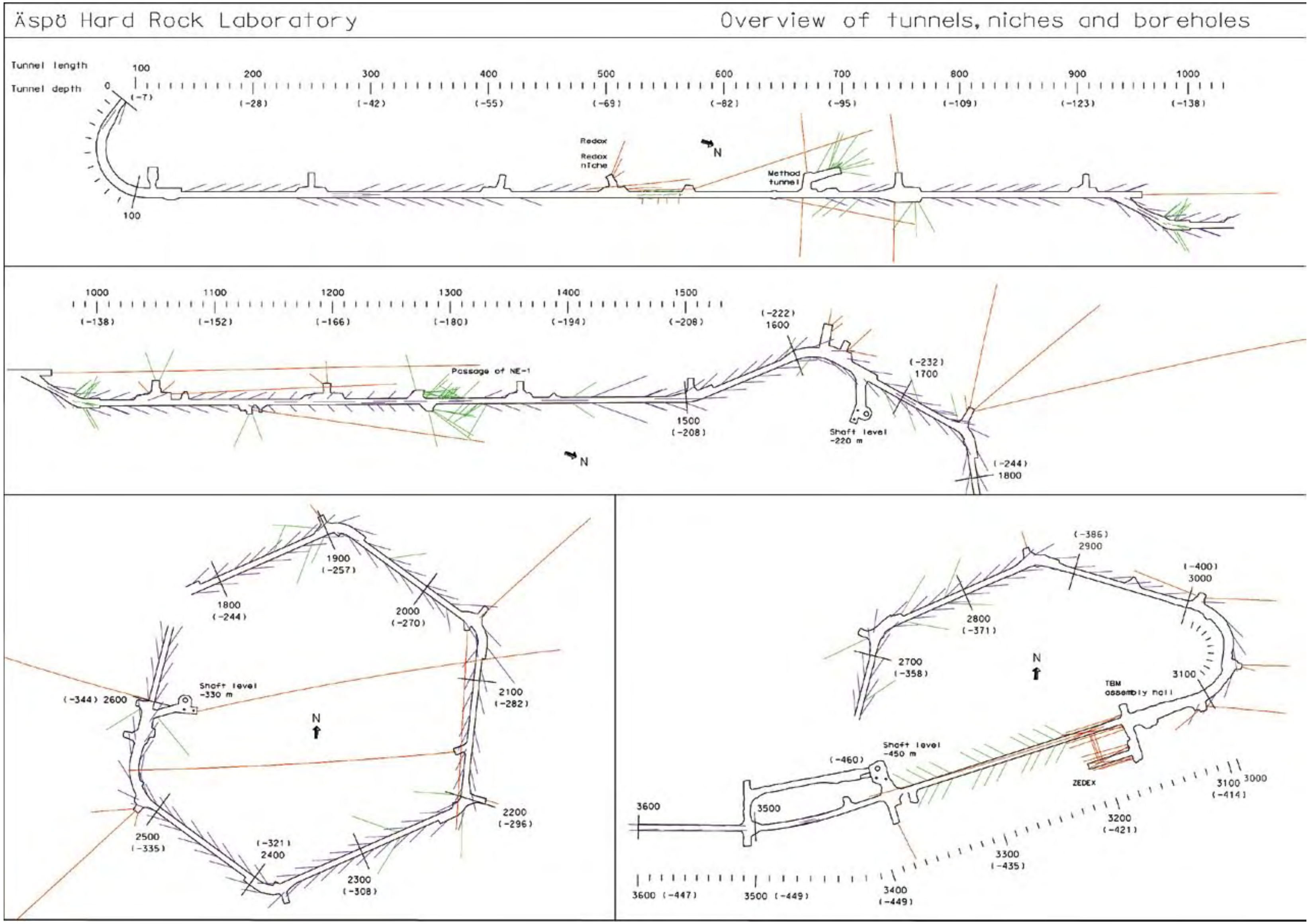


Figure 2-4. *Tunnels, niches and boreholes in the Äspö HRL tunnel.*

Figure 2-5. *Raise boring of shaft. Drilling machine at the surface and the raise boring head at the –220 m level before drilling of the first shaft section.*

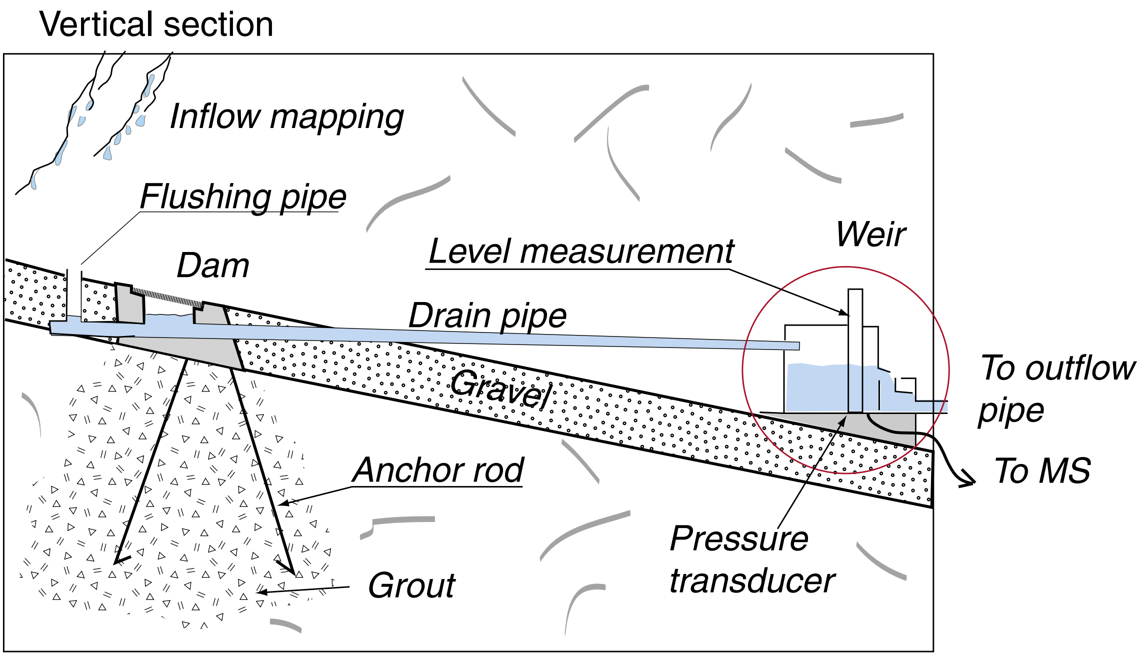


Figure 2-6. *Construction of water dams for collecting water inflow along tunnel intervals.*

Figure 2-7. *Pipes, cables etc installed on brackets on the tunnel walls.*

2.2 Overview of the geoscientific investigation programme

2.2.1 Outline

The investigations during the construction phase was predominantly related to the first two stage goals; to verify pre-investigation methodology and to finalise detailed characterisation methodology, see Section 1.2.2. These two goals are related in that the results of the detailed underground investigations are used to evaluate the models and predictions based on the pre-investigations as well as to evaluate the investigation method itself, as was discussed in Section 1.3, see Figure 1-4.

The main objective of the investigations during the construction phase was to document geological, hydrogeological, hydrochemical and rock mechanical conditions along the tunnel and to monitor groundwater behaviour with regard to pressure, flow and chemistry for subsequent comparison with the previously made predictions. In Figure 1-5 these investigations are called baseline characterisation and monitoring.

Another objective was to perform a detailed and supplementary characterisation of the rock volume outside the tunnel for the purposes of updating the geoscientific models and changing the tunnel layout. These specific investigations (Figure 1-5) were normally performed in longer cored holes and were initiated and carried out according to individually determined programmes.

All investigation results from the construction phase were used to update and modify the geoscientific models of the Äspö rock volume and the surrounding regional volume /Rhén et al. 1997b/. Evaluations of the pre-investigation models and predictions are reported in /Stanfors et al. 1997b/ and /Rhén et al. 1997c/.

2.2.2 Baseline characterisation

Baseline characterisation involved two main tasks of characterisation work: documentation of the tunnel wall and investigations of standard probe holes (Figure 1-5). Typical for the baseline characterisation was that all documentations and measurements were carried out in co-ordination with the construction work. A critical prerequisite for successful co-ordination was development of and adherence to strict working routines. The logistic framework for construction/investigation co-ordination for the blasted tunnel was as follows:

- Construction work;
  - drilling of blast holes,
  - loading of explosives,
  - blasting,
  - mucking out of rock debris,
  - scaling.
- Investigation work (during a period of one hour);
  - photo documentation,
  - tunnel wall characterisation.

Every fourth round (approximately) also included:

- Construction work;
  - drilling of two 20 m long probe holes.
- Investigation work (one additional hour available);
  - packer installation in probe holes,
  - water sampling from probe holes,
  - pressure build-up tests in probe holes.

The general routines for the tunnel characterisation and most of the methods applied are described and presented in Chapter 4.

The general routines for probe hole drilling are discussed in Chapters 4 and 5. Methods used in or related to probe holes are described and discussed in almost all chapters, except 7 and 10.

Strict standardised routines were also followed concerning data management and data presentation. Graphical presentations of all mapping, measurements, sampling etc were made along a 150 m interval of the tunnel. For every 150 m interval three sheets were produced, one for geological information, one for hydrogeological and groundwater chemical information and finally one sheet for rock quality, reinforcements and pre-grouting, see Figures 2-8, 2-9 and 2-10.

2.2.3 Monitoring programme

The monitoring programme included besides monitoring in the tunnel an extensive monitoring of surface boreholes. The surface monitoring included measurements of the groundwater head, pressure level, in-situ groundwater flow and chemistry. Hydrogeological monitoring in the tunnel included observations of pressures in selected holes and monitoring of water inflow to the tunnel, see Chapters 8, 9 and 11.

2.2.4 Special investigations

Special investigations were occasionally performed as a complement to the baseline characterisation or for other reasons. These investigations normally involved drilling of longer, cored investigation holes in which a variety of measurements and tests were performed. The type of measurements performed were related to the objectives of the special investigations.

Examples of special investigations (and experiments run during the construction phase) are (see Figure 1-5 /Stanfors et al. 1997a/):

- Supplementary investigations of fracture zones in the Äspö tunnel /Rhén and Stanfors, 1995/.
- Passage through water-bearing fracture zones /Rhén and Stanfors, 1993/.
- Investigation programme aiming at locating experimental sites and developing the layout of the lower part of the tunnel /Olsson et al. 1994/.
- The Redox experiment was run in short tunnel niche holes drilled into a vertical fracture zone at a depth of approximately 70 m, aiming at monitoring whether dissolved oxygen from the surface was transported due to the draw-down /Banward, 1995/.
- The ZEDEX experiment was carried out at the beginning of the TBM tunnel and in a parallel tunnel excavated by drill-and-blast. The objective was to investigate the geometry and character of the disturbed zone and compare between the construction methods /Olsson et al. 1996/ and /Emsley et al. 1997/. An earlier study of the blast-damaged zone and its dependence on variations in the drill-and-blast programme was carried out between 526 and 565 m in the tunnel /Christiansson and Hamberger, 1991/.
- The SELECT programme, aiming at locating experimental sites and preparing for coming tracer tests and chemical experiments /Winberg et al. 1996/.

2.2.5 Reporting of data and results

All data were compiled for standardised presentation of (1) geological data, (2) hydrogeological and groundwater chemistry data and (3) rock quality, reinforcements and pre-grouting data. All these data sheets were finally compiled in a report which also included core logs from 25 cored holes drilled from the tunnel /Markström and Erlström, 1996/ and /Markström, 1997/.

Collected data (mainly from baseline characterisation and monitoring, but also from special investigations), data evaluations and comparisons of results with predictions were presented in evaluation reports representing different sections of the tunnel.

- One report present data from the 0–700 m section, /Stanfors et al. 1992b/.
- Four reports (geology and rock mechanics, hydrogeology, groundwater chemistry and transport of solutes) covered the 700–1,475 m section, /Stanfors et al. 1993a; Rhén et al. 1993a; Rhén et al. 1993b; Wikberg and Gustafsson, 1993/.
- Four reports covered the 1,475–2,265 m section, /Stanfors et al. 1993b; Rhén et al. 1993c; Rhén et al. 1993d; Wikberg et al. 1993/.
- Four reports covered the 2,265–2,874 m section, /Stanfors et al. 1994; Rhén et al. 1994a; Rhén et al. 1994b; Wikberg et al. 1994/.
- One report present data from the tunnel section 2,874–3,600 m and shaft section 0–450 m, /Rhén, 1995a/.

The results of the investigations during the construction phase were extensively evaluated and compared with the pre-investigation model and the predictions based on pre-investigations. The extensive geoscientific information obtained was also used to update the geoscientific models of Äspö and the surrounding region. The technical reports summarising the results of the construction phase investigations are /Rhén et al. 1997a/, /Rhén et al. 1997b/, /Rhén et al. 1997c/, /Stanfors et al. 1997a/ and /Stanfors et al. 1997b/.

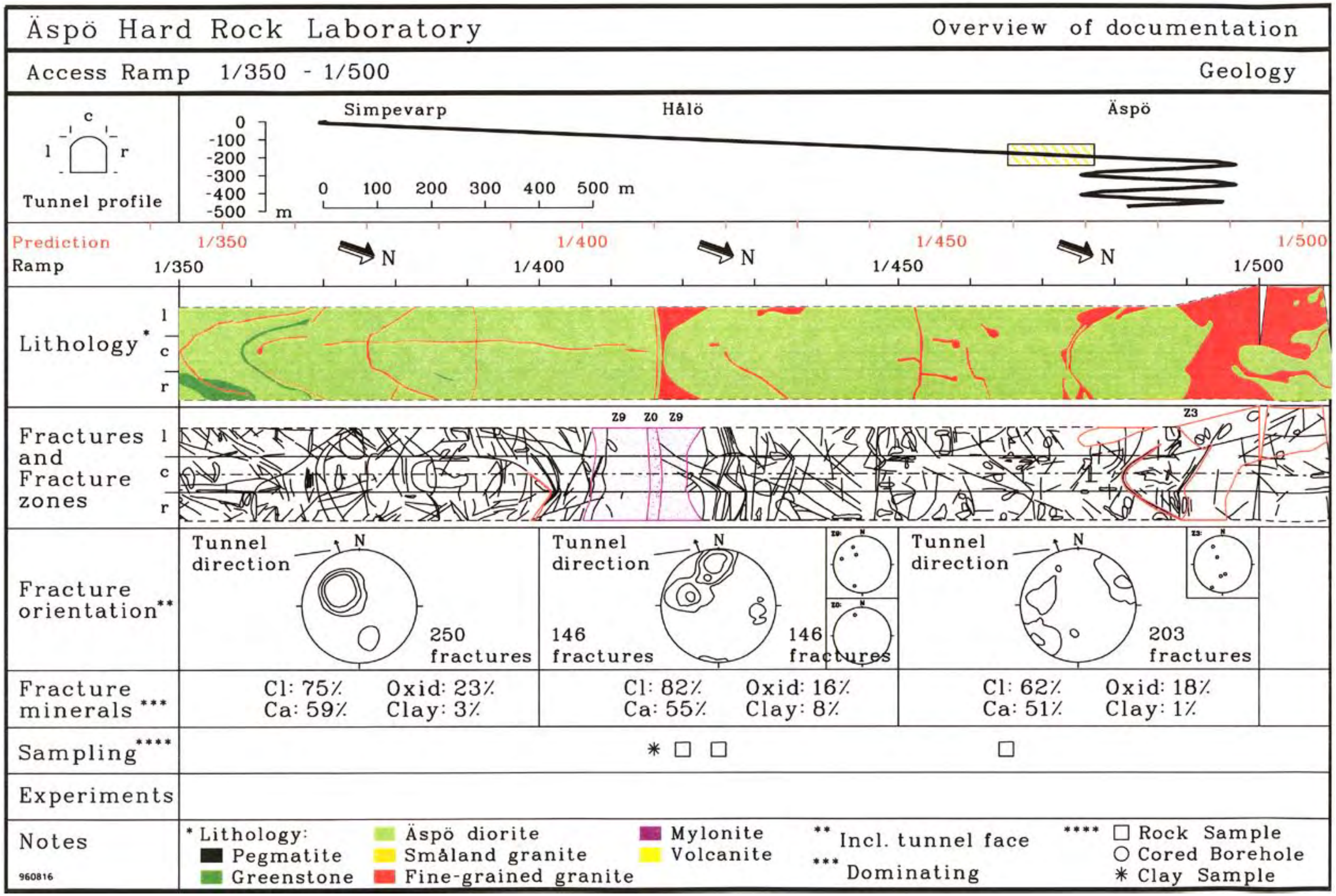


Figure 2-8. *Geological data from tunnel mapping presented in an overview 150-m sheet. Example from tunnel interval 1,350–1,500 m.*

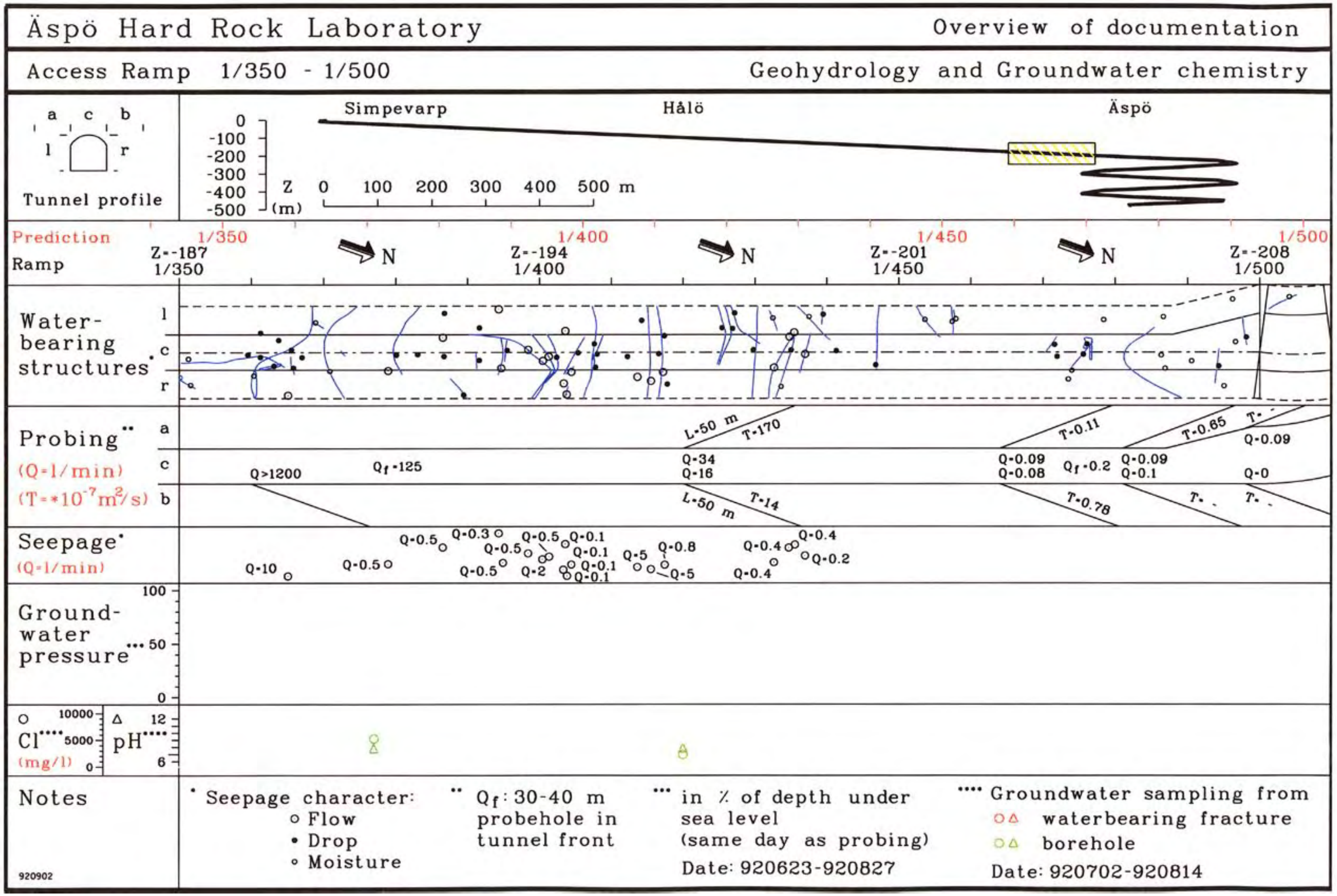


Figure 2-9. *Hydrogeological and groundwater chemistry data from tunnel mapping presented in an overview 150-m sheet. Example from tunnel interval 1,350–1,500 m.*

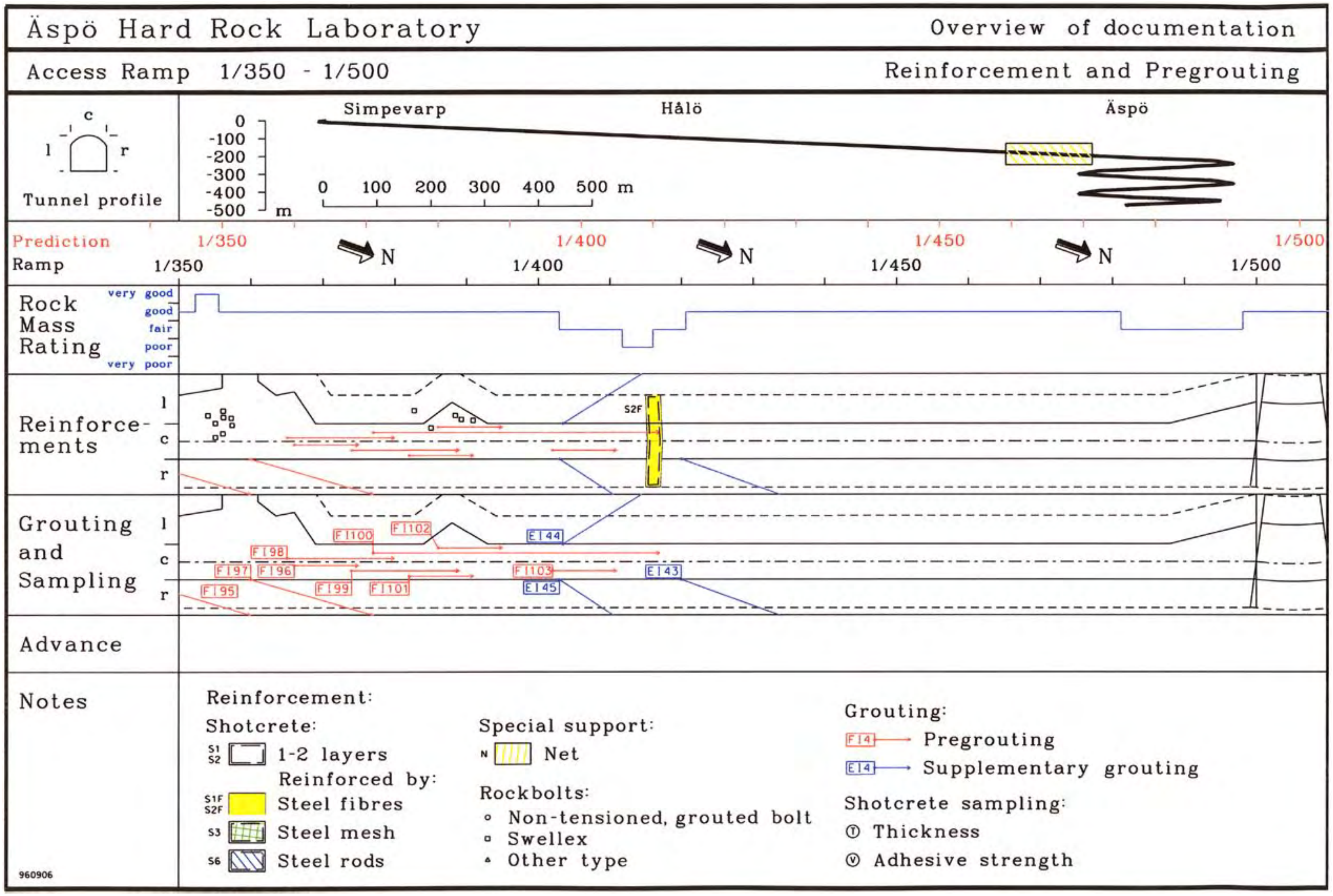


Figure 2-10. *Rock reinforcement data from tunnel mapping presented in an overview 150-m sheet. Example from tunnel inverval 1,350–1,500 m.*

3 Positional information

3.1 General

Geoscientific characterisation (modelling) of a rock volume is primarily based on measurements of parameters at selected locations in the rock mass and subsequent interpretations/evaluations of results. A precise positioning of the measurement locations is essential for an accurate evaluation of the results. Positioning of measurements in boreholes and tunnels are difficult to perform and a small discrepancy in the beginning can result in an unacceptable error at the end of the borehole or tunnel.

The coordinate system and the methods and accuracy of positioning the tunnels and objects in the tunnel and in boreholes are described in Section 3.2. The methods for positioning of the tunnel, reference points and chainage markers in the tunnel are described in Section 3.3. The positioning of mapped objects on the tunnel wall is based on the chainage markers and is described in Section 4.2. Positioning of boreholes is described in Section 3.4. Finally, the method for naming objects is described in Section 3.5.

3.2 Coordinate system

Boreholes and other features in the tunnel and at the ground surface were positioned in relation to the Äspö coordinate system, see Figure 3-1. This system relates to a local coordinate system used by the OKG Nuclear Power Plant. In relation to the Swedish national grid (RAK 2.5 gon V, RAK38), the Äspö system is 11.819° west of RAK north. The Z coordinate is the same as in the Swedish national height system RH00. The origin of the Äspö system 0/0 is located at 6360251.890/1550827.928 in the RAK system.

The orientations of geological structures are defined in relation to the magnetic north. The differences in angle between the two systems (as well as geographic north) are illustrated in Figure 3-2.

In the central data base SICADA (see Chapter 12), all coordinates are presented in both the Äspö system and the RAK system. Conversion between the two systems is done in SICADA.

The accuracy for the given coordinates (X,Y,Z) is ± 0.01 m and the accuracy for the given angles is ± 0.2°.

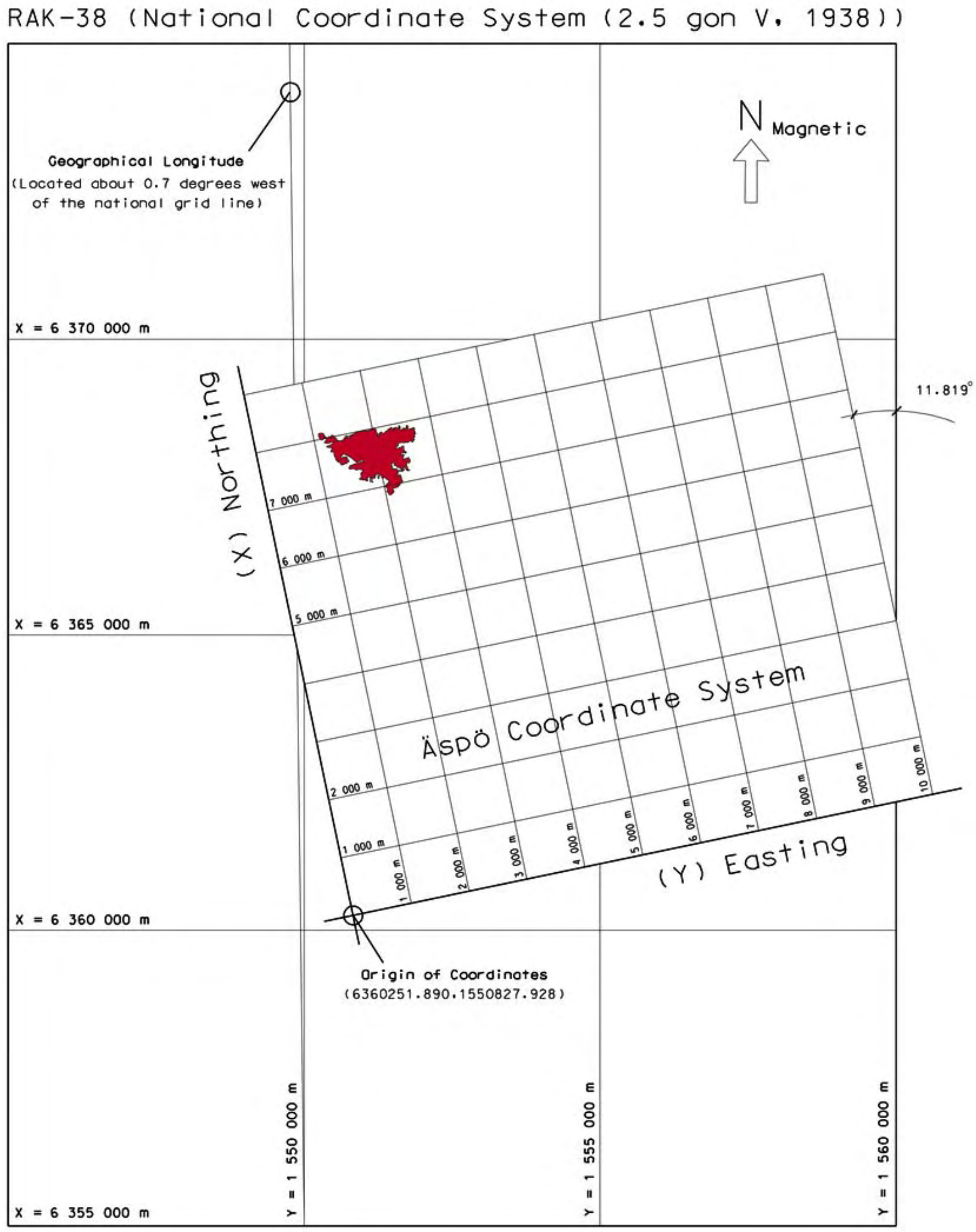


Figure 3-1. *Location of Äspö Coordinate System in relation to Swedish National Coordinate System denoted RAK-38.*

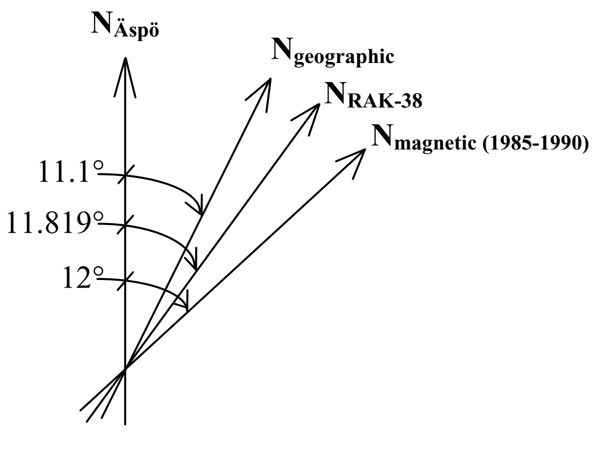


Figure 3-2. *Angle differences between the different systems.*

3.3 Location of tunnels

3.3.1 Method and instrument

The location of the tunnel was monitored by the contractor using a precision total station WILD TC1600 (SIAB as contractor) or WILD TC1000 (SKANSKA as contractor), see Figure 3-3. The survey was based on, reference points consisting of steel bolts drilled into the tunnel wall and laser reflectors placed on these bolts. Altogether, 53 reference points were positioned in the tunnel at regular intervals. The points were measured and related to the local Äspö system with X, Y and Z coordinates. The chainage measured from the tunnel entrance was given. During construction a survey line with chainage markers every ten metres was mounted along the walls one metre above the floor, see Figure 3-4.

By the guidance of a laser projector mounted on the wall or close to the roof the tunnel profile could be kept straight. The laser projector was positioned in such a manner that the laser beam followed the theoretical tunnel line. The position of the drilling rig was then adjusted in reference to the laser beam. A Beaver control unit on the drilling rig was then used to calculate the actual directions of the drill holes drilled for the blast scheme. The laser was only used along the straight lines of the tunnel.

A laser projector was also used for the TBM excavated part of the Äspö HRL. The laser projector was positioned so that the laser beam followed the theoretical tunnel line. Two plexiglas plates were placed on the TBM machine. One plate was positioned close to the drillhead and another plate was positioned a few metres behind the other one. Coordinate lines were drawn on the plexiglas plates. A small hole was made in the middle of the rear plate. The laser projector was adjusted so that the laser beam went through the small hole. In a straight tunnel the laser beam hit the front plate in the middle. In curves the laser beam on the front plate was shifted to either side depending on the direction of the curve. Small prisms were used in curves to adjust the laser beam. The estimated amount of shift of the laser beam to the side was calculated according to the curve radius and the shift of the beam on the front plexiglas plate in reference to the rear plate.

Figure 3-3. *Total station mounted on a reference point in the TBM tunnel.*

Figure 3-4. Survey line and chainage mark on tunnel wall in drill-and-blast tunnel.

3.3.2 Resolution and accuracy

The coordinates of the reference points were presented with a resolution of ± 0.001 m.

The accuracy of determined coordinates and angles depends on the location of the precision total station and the reflection mirror operated by the field assistant.

The accuracy in determination of coordinates is estimated to be within ± 0.01 m. The accuracy of determination of angles is estimated to be ± 0.2°.

The locations of the chainage markers are estimated to be within ± 0.1 m.

3.4 Location, direction and deviation of boreholes

3.4.1 Method and instrument

Cored investigation boreholes

The location of the cored boreholes in the tunnel was determined with the precision total station as described in Section 3.3.1. The location of the borehole collar was presented in the local Äspö system with X, Y and Z coordinates.

The deviations of the cored boreholes were determined using the Maxibor method. The initial direction of a borehole is of great importance for the Maxibor method. This was determined with the precision total station by surveying the position of the first Maxibor rod installed in the borehole. The Maxibor measures then the borehole deviation by using a CCD based image sensor which optically records the offset of reflector rings at pre-set distances inside a system of Maxibor rods. The steel rods are centralised in the borehole with help of centralisers positioned outside the rods, which means that they follow every bend of the borehole, see Figure 3-5. The Maxibor rods were lowered into the borehole by means of the drilling rig.

Measurements were made with a three metres interval along the whole borehole. Based on start direction and measured bends, inclination and declination are calculated every 3rd metre along the borehole. The inclination and declination values are presented in the database together with the calculated coordinates (X, Y, Z).

Percussion-drilled boreholes

The locations of the short probe holes and the percussion-drilled investigation holes were manually determined according to the chainage markers. The directions of the holes were determined manually with a compass. The probe holes were normally drilled parallel to the tunnel floor. The deviation along the percussion-drilled holes was not determined.

3.4.2 Resolution and accuracy

The coordinates of cored boreholes were presented with a resolution of ± 0.001 m. The accuracy is estimated to be within ± 0.01 m, see also Table 3-1.

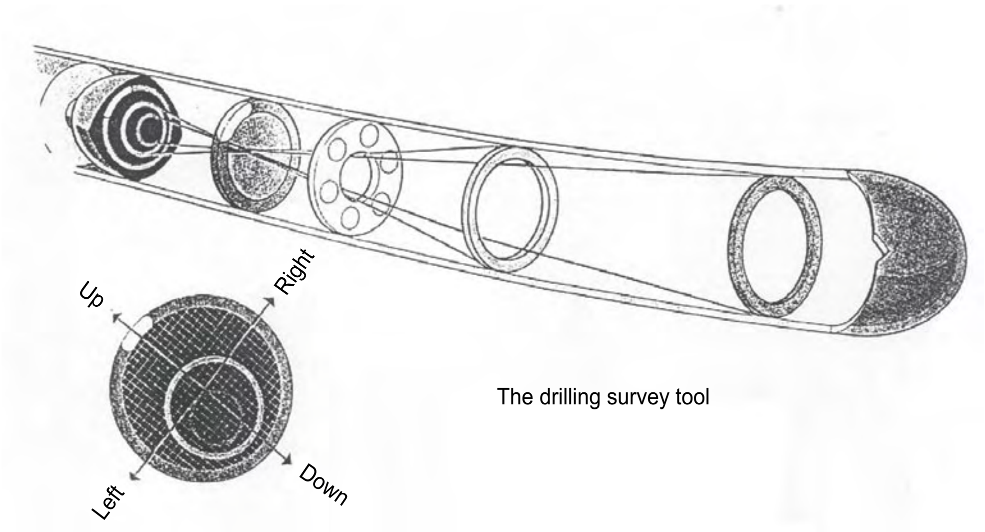


Figure 3-5. *Principle of borehole deviation measurements with the Maxibor system.*

Table 3-1. Resolutions and accuracies for geometric data for boreholes.

Resolution/accuracy	Cored, investigation borehole	Percussion-drilled borehole
Resolution of location	± 0.001 m	± 1 m
Accuracy of location	± 0.01 m	± 1 m
Resolution of direction	± 0.01°	± 1°
Accuracy of direction	± 0.2°	± 5°
Resolution of deviation (radial distance)	± 0.1 m	—
Accuracy of deviation (radial distance)	± 0.4 m per 100 m	± 1–2 m per 20 m

The accuracy of the deviation measurement obtained from the Maxibor survey depends on the care and precision of the measurement of the initial direction, the accuracy of the centralisation of the probe and the accuracy of the survey itself. The accuracy of the determination of the start direction is estimated to be ± 0.2°,which will result in a maximum error of 0.35 m per 100 m. A centralisation error of 1 mm (gap between outer diameter of probe and inner diameter of borehole) will result in a maximum error of 0.03 m per 100 m. The accuracy of the tool itself is better than 0.01 m per 100 m. This gives a maximum deviation error is less than 0.4 m per 100 m borehole.

The error in length determination is negligible when the Maxibor survey is conducted with drill rods. When the surveys are carried out using a wireline the length error should be calculated and adjusted for.

The accuracy of the location of the percussion-drilled probe holes was ± 1 m. The accuracy of the determination of the direction of the probe holes was ± 5°. The deviation of the percussion-drilled holes is dependent on structures in the rock and drilling performance and has not been measured in the project. However, the deviation for a 20 m probe hole is estimated to be within 1–2 m.

3.5 Naming of tunnels, boreholes and measurement/ sampling objects

Strict routines for naming objects are essential for the efficient performance of construction and investigation work as well as out of a quality point of view.

All geological information on the direction of structures is given relative to magnetic north, which for practical reasons is equal to geographic north and RAK north, see Figure 3-2.

For all objects (boreholes, sampling points, etc) in the Äspö tunnel a simple name convention (ID code) is used, which is equally essential for the organisation of data in the SICADA database, see also Chapter 12. The naming of objects in the tunnel is based on a seven characters code string, for example:

- KA2511A
- HC0003B
- SA0954F
- YA1654B

where:

- The first capital stands for type of object;
  K: cored investigation borehole,
  H: percussion-drilled investigation borehole,
  S: probe hole,
  Y: surface sample points.
- The second capital is the last letter in the code for the tunnel where the object is located, see Figure 2-1.
- The following four digits define the position of the object along the tunnel in metres.
- The last capital is a code for the position in the tunnel wall as described in Figure 3-6.

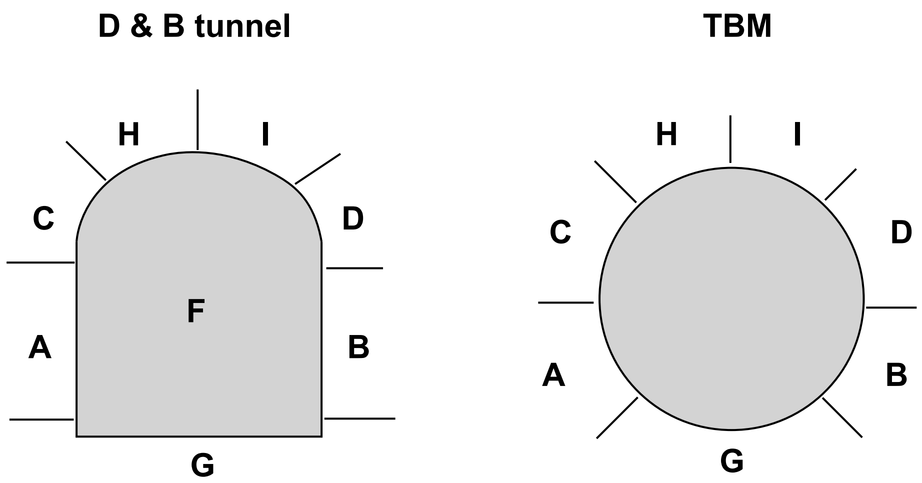


Figure 3-6. Code for position of objects in the Äspö tunnels (drill-and-blast tunnel and TBM tunnel).

In SICADA the ID code for tunnels is based on four characters, for example TASA. Decoded TASA means tunnel (T) at Äspö (AS) and (A) stands for tunnel A (or the main tunnel), see Figure 2-1.

3.6 Comments and recommendations

The coordinate system

The use of two coordinate systems, the RAK system and the Äspö system, has sometimes been confusing. Sometimes it has been unclear which coordinate system a measurement data package is related to. This has resulted in irritation among personnel, re-calculation of data, etc, and was costly and time consuming. In fact this is a QA issue, i.e. watertight routines in this area must be established before the start of investigation and they must be followed. The best option would probably be to use the national RAK system exclusively.

The geometrical data are presented in the Äspö system, but geological observations (i.e. strike of structures, fractures and fracture zones) are presented with reference to magnetic north. This was performed for practical reasons for the geologists working with a compass. In the modelling work the orientation of structures had to be recalculated into the Äspö system.

Location of tunnel and tunnel objects

During construction a survey line with chainage markers every ten metres was strung along the walls one metre above the floor. This sufficiently helped in localising geological objects.

Boreholes

Probe holes were localised with use of the chainage markers. Core drilled boreholes were positioned with help of the total station. Percussion drilled boreholes deviates from a straight line to a larger extent compared with core drilled boreholes.

Naming of objects

The convention for naming of objects is adequate. The ID code includes information on type of object and location, which is practical. Including the position of the borehole along the tunnel in the naming of the borehole was useful.

4 Characterisation of the tunnel

4.1 General

With reference to Section 2.2, the main investigation activity conducted continuously during the entire tunnelling of the Äspö HRL was geoscientific baseline characterisation, see Figure 1-5. The general purpose of this was:

- To map and document the geological, hydrogeological, hydrochemical and rock mechanical features along the tunnel, which made it possible to evaluate the models and predictions set up on the basis of pre-investigation data.
- To update the geoscientific models.

Predictions were made on different scales (see Table 13-1). Coherent mapping along the whole tunnel produced baseline data for comparison with predictions on the site scale. Specific characterisation of six 50 m blocks was used specially for comparison with block-scale and detailed-scale predictions /Stanfors et al. 1997b/.

The tunnel wall baseline characterisation is described in this chapter, while the description of the borehole investigations is integrated in following chapters.

The baseline characterisation was as mentioned earlier integrated with the tunnelling work and normally performed during one hour between each drill-and-blast round. Before the characterisation team was allowed to enter the tunnel, the ceiling and walls were scaled, cleaned and secured by the construction team. If the tunnel section was unstable and had to be supported by a shotcrete layer, the characterisation team performed a remote tunnel wall mapping extrapolated from the previously excavated section.

Characterisation of the tunnel walls was one major part of this characterisation, conducted according to standard routines developed during the initial few hundred metres of tunnel and thereafter only slightly modified. However, along the last 409 m of the tunnel, when the TBM technique was used, the documentation routines had to be adjusted, but still with the aim of producing the same type of data. The documentation routines also had to be modified for mapping in the shafts.

The tunnel wall characterisation, also presented in Figure 4-1, included:

- Photographic documentation.
- Geological mapping.
- Rock mechanical documentation.
- Hydrogeological mapping.
- Hydrochemical sampling.

A geologist and a hydrogeologist performed the characterisation. Two teams worked in shifts, mainly during the daytime but periodically also during nights, depending on the contractor's work schedule. Scan line mapping in 50 m blocks was primarily performed during weekends.

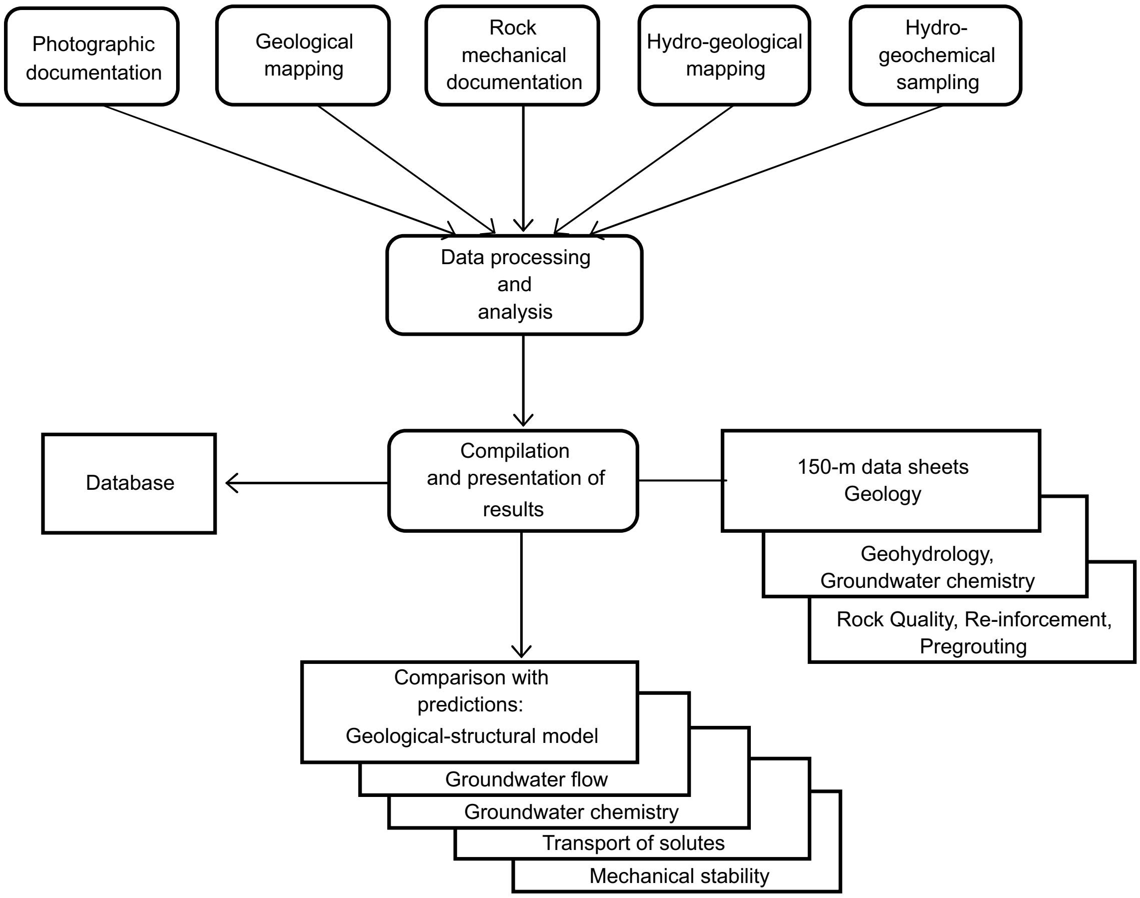


Figure 4-1. *Flow chart of tunnel mapping activities and procedures for use in evaluation of predictions, arranged according to key issues – Geological-structural model, groundwater flow, groundwater chemistry, transport of solutes and mechanical stability.*

An important tool for data management and presentation of the information was the Tunnel Mapping System (TMS) in which the data is compiled, organised and presented in a strict and condensed manner in order to facilitate comparison with predictions, see further Sections 4.8 and 4.9.

4.2 Base maps and surveying

4.2.1 Methodology

Surveys of the tunnel were conducted by the contractor, who provided a marked line along the left and right walls, see Section 3.3 and Figure 3-4. Short lines perpendicular to the long lines were made every 10 m (chainage marks). The chainage, which refers to the actual length of the tunnel in metres from the portal to the working face, was specified on the tunnel drawing.

The characterisation team prepared base maps of the tunnel geometry on the basis of the tunnel drawings supplied by the contractor. For each tunnel leg, coordinates and tunnel geometry were imported to the TMS at the site office, see Section 4.8. The base maps covered approximately 10 m (of which 5 m covered the last blasting round) on a scale of 1:100, see Figure 4-2. The tunnel was drawn with the longitudinal axis of the centre line

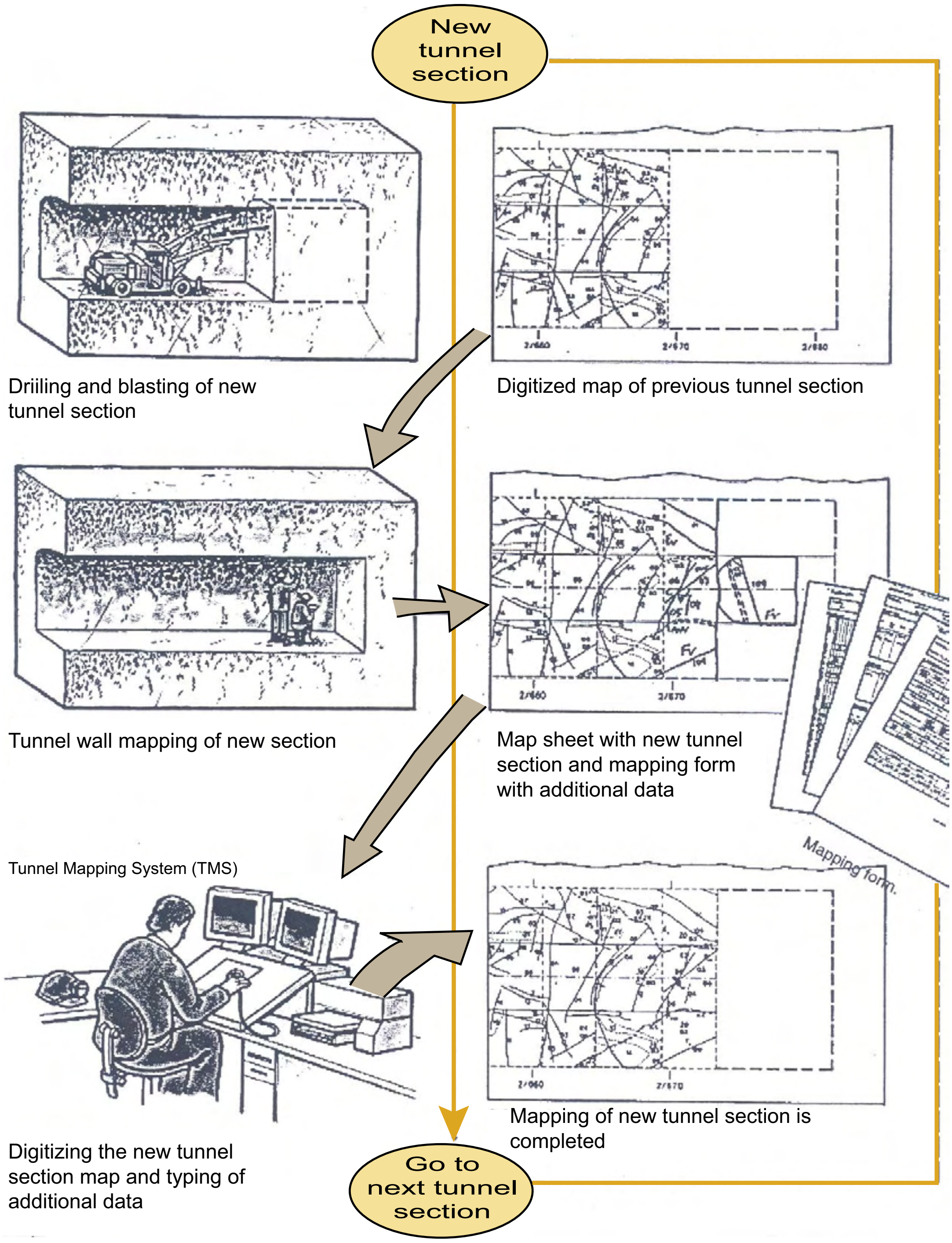


Figure 4-2. Principle of preparing basic map sheets in the TMS (from contractor's tunnel drawing), use in tunnel mapping and digitising for storage in the TMS database.

of the roof in the centre and the walls folded out. The periphery was divided into segments covering the left wall, left roof, right roof and right wall. The left wall was always placed in the uppermost part on the documentation sheet, the roof in the middle and the right wall at the bottom, i.e. tunnelling proceeds towards the right. The front was also folded out on the same map. However, mapping of the tunnel front was not digitised in the TMS.

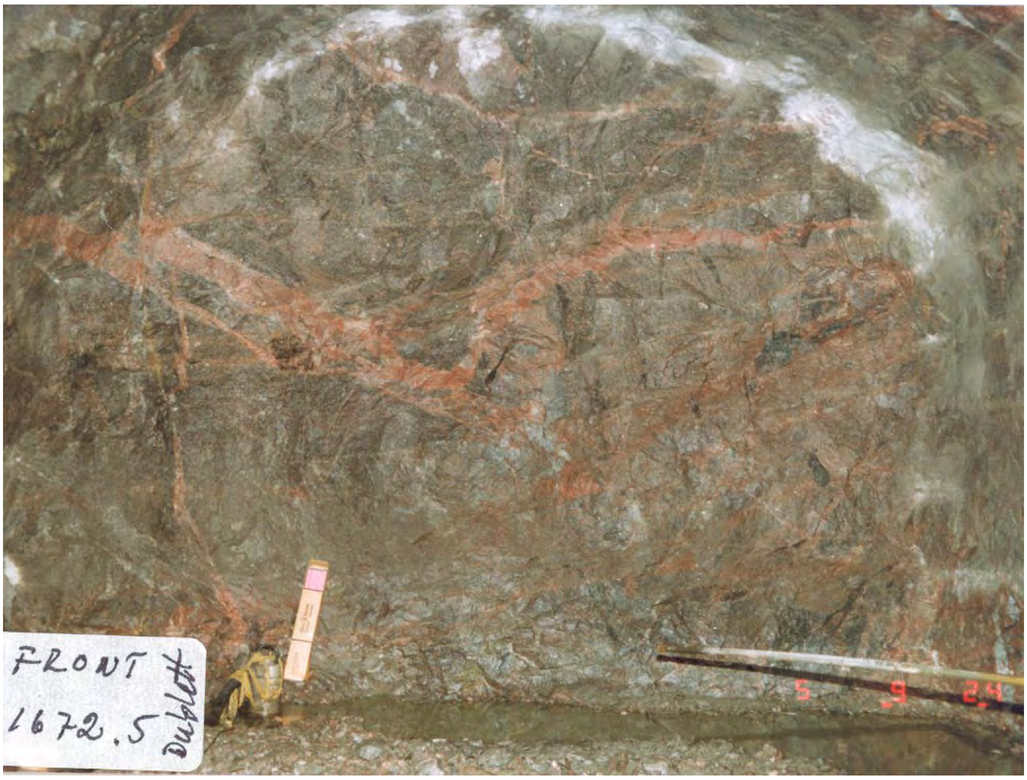


Figure 4-3. *Example from photo documentation of the tunnel front.*

4.3 Photographic documentation

The photographic documentation work in the drill and blast tunnel included beside routine documentation of roof and walls after each blasting round also documentation of objects of special interest, such as special rock types, structures, large inflows of water, etc.

The routine photos included the tunnel front, tunnel roof, left and right walls, see Figure 4-3. The photos were stored in the binder for mapping of each front. Negatives were stored in separate binders.

No routine photo documentation was performed during the TBM excavation or for the shaft.

4.4 Geological mapping

A baseline continuous geological mapping was performed along the whole tunnel. This was used to compare outcome with the predictions see Figure 1-5 and Figure 6-1. In the 50 m blocks, continuous mapping was supplemented by scan-line mapping. Core samples from representative parts of the principal rock types were taken for mineralogical studies and density and porosity determinations, see Section 6.4.

Regarding data presentation and reporting, see Section 4.9.

4.4.1 Continuous geological mapping

Continuous geological mapping was an important part of the baseline characterisation during excavation. Continuous mapping provided the main geological data set for evaluation of the predictions. The geological mapping was carried out after every round during drill-and-blast excavation and included the mapping of /Christiansson and Stenberg, 1991/:

- Rock type (colour, structure, grain size, texture, extent, alteration).
- Rock contacts (strike, dip, type).
- Fractures, trace length > 1 m (strike, dip, length, type, form, termination, displacement).
- Fracture zones (strike, dip, width, type, number of and orientation of fracture sets).
- Fracture filling materials (minerals).
- Surface properties (roughness, striation).

Methodology for the drill-and-blast excavation

The work started with determination of the correct chainage of the tunnel interval, using the chainage mark previously surveyed by the contractor in combination with a measuring tape.

Observed geological features on the newly excavated tunnel walls, tunnel roof and tunnel front were documented on the base map sheets produced by the TMS. The base map sheet included geological data from the previous mapped 5 m, see Figures 4-2 and 4-4. Fractures longer than one metre, rock contacts and fracture zones were initially drawn by hand on the base map sheet. Rock type descriptions, properties of fractures and fracture zones, fracture fill materials and different fracture set orientations were then added on a mapping form.

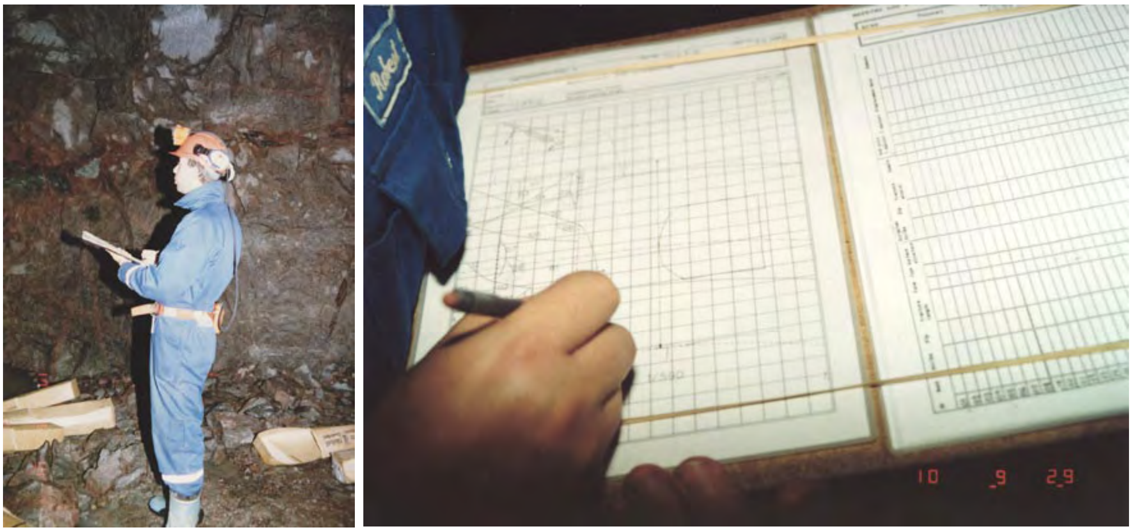


Figure 4-4. *Geological mapping in the tunnel.*

Samples of rock types and fracture fill materials were taken for closer examination when an unknown rock type or fracture filling material (clay) was encountered. Rock samples were in addition taken from representative parts of the four main rock types (Greenstone, Fine-grained granite, Småland granite and Äspö diorite) for evaluation of the detailed predictions made on the 5 m scale. The samples were taken from the rock wall as well as by drilling of short (a few dm) cored holes in the tunnel wall using a Pixi core drill machine ancored to the rock surface. All samples were given ID-numbers (see Section 3.5) before they were sent to a laboratory for (see Section 6.4):

- Density and porosity measurements.
- Microscopic modal analyses.

Most of the rock samples were taken during special sampling campaigns after the original tunnel wall mapping procedure was completed.

When geological mapping after each round was completed in the tunnel, the basic map sheet and mapping forms were taken to the site office where the information was transferred to the TMS, see further in Section 4.8.

Methodology in the TBM tunnel

Geological mapping of the TBM tunnel was performed after the TBM tunnel section was completed. Before mapping the tunnel was cleaned by washing with water. The objective of geological mapping of the TBM tunnel and the methodology used were the same as for the drill-and-blast tunnel /Stenberg, 1994/.

Methodology for shaft mapping

Shaft mapping was performed from an elevator platform after excavation and cleaning of the shaft /Rhén, 1995a/. The same parameters as for baseline mapping were determined. The mapping sheet was folded out with magnetic north as a reference line in the middle of the sheet. A 50 m long measuring tape was fastened with bolts along the reference line.

Since the cages were magnetic, traditional geological mapping with a compass was impossible. It was therefore essential to attach a meter-scale as a reference line, directed towards the north, on the shaft wall. Geological structures were oriented in relation to this reference line. Fracture planes, thin dykes and lithological contacts are approximately planar on the scale of the shafts. A special diagram was devised where mapped structures appear as sinoid curves. Approximate orientations of fracture planes and other structures were estimated by visual inspection. It was by this possible to calculate the orientation, strikes and dips of the mapped structures. Additional information, regarded as significant to this study, such as fracture fillings and estimates of water flow, was also recorded.

4.4.2 Geological scan-line mapping in 50 m blocks

The continuous geological mapping was complemented by scan-line mapping in the so-called 50 m blocks. The aim of the scan-line mapping was to provide detailed information for evaluating the more detailed predictions for the 50 m blocks with regard to:

- Fracture frequency data (fracture trace length > 0.2 m) along the scan-line, from which, for instance, RQD and other data could be calculated for the total number of fractures.

- Differences in fracture frequency distribution in different types of rock.
- Detailed description of a fracture zone and its immediate surroundings.

Methodology

The scan-line mapping was performed along a measuring tape placed along the right wall about 1 m above the tunnel floor. Scan-line mapping comprised mapping of the same parameters as were mapped in continuous geological mapping, see Section 4.4.1. However, now with fracture trace length > 0.2 m, i.e. smaller fractures were now included.

The scan-line data was documented on a separate field form, from which the data were typed in a dBase file. A print-out was made and all values were quality controlled against the values in the field form. The forms were stored in a binder at the Äspö HRL.

4.4.3 Accuracy

The geologists measured the positions of objects by using a measuring tape or estimated distances by eye. Using these methods, it was estimated that the accuracy to which positions could be determined was within ± 0.5 m. The accuracy for the positioning of objects in the TBM tunnel and the shaft were in the range of ± 0.2 m.

The orientation of structures was measured using a compass with a resolution of 1° and an estimated accuracy of ± 5°. Compass measurements could be distorted by the drill rig within a distance of approximately five metres from the rig. Depending on the dip direction, the strike could be wrongly read by 180°. The strike was measured in the front of the compass if the dip was to the right and in the rear of the compass if the dip was to the left. This mistake happened occasionally in the beginning but after that the team got more trained this type of error was minimised. The strike could also be wrongly typed in the database, i.e. strike greater than 360° /Sirat, 1997/. However, quality control of data typed in has reduced this typing error from 1,700 m on.

The accuracy of geological parameters is more difficult to quantify. This is more a matter of correct definitions, geological skill and experience of the mapping personnel, and communication within the characterisation team. Accuracy is estimated to be higher for scan-line mapping in 50 m blocks compared with continuous mapping, mainly due to more time being available. A comparison of the difference in sampling by two teams has been discussed by /Munier, 1995/ and by /Stenberg, 1993/.

4.5 Rock mechanical characterisation

Continuous rock mechanical mapping was performed along the whole tunnel as a general basis for comparison with predictions, see Figure 1-5. Rock stress measurements were also performed in the pre-located 50-m blocks, see Chapter 10. Laboratory investigations on core samples were performed in the 5 m blocks, see Section 6.4.

Documentation of tunnel stabilisation work, i.e. pre-grouting and reinforcement of the tunnel wall, was done by the construction team. These data are included in the presentation and reporting of the investigations, see Section 4.9.

Rock mechanical characterisation was not performed in the shaft.

4.5.1 Continuous rock mechanical mapping

Continuous mapping of conditions and parameters of importance for the rock mechanical characterisation and evaluation of bedrock stability was carried out after each new excavation round in conjunction with, and by the same characterisation team as, the geological mapping. The purpose was to compile a set of baseline rock mechanical data for comparison with the rock mechanical predictions made for the whole tunnel, based on the pre-investigation data.

Continuous observations of rock burst indications, like cracking and tendency of spalling, was carried out in addition to the normal mapping procedure. After each round outfall of blocks and general instability was documented.

Methodology for drill-and-blast excavation

The rock mass was characterised by means of the Bieniawski Rock Mass Rating (RMR) classification /Bieniawski, 1989/. The RMR system is based on the following six parameters used to classify the rock mass, see also Table 4-1:

- Uniaxial compressive strength of rock material.
- RQD.
- Spacing of discontinuities.
- Condition of discontinuities.
- Groundwater conditions.
- Orientation of discontinuities.

For uniaxial compressive strength, a typical value for the rock type in question was used. Values for the other five parameters were estimated by the characterisation team: This was performed by inspection of the tunnel front wall along an imaginary line perpendicular to the main discontinuity set, according to the RMR method. The parameter values were summarised in an RMR value. The RMR value can range from < 20 to 100 and is subdivided into 5 classes with the following ratings:

RMR value	Description
100–81	Very good
80–61	Good
60–41	Fair
40–20	Poor
< 20	Very poor

Rock mechanical mapping in the tunnel was documented on a special field form. Further data processing with TMS is described in Section 4.8.

Methodology in the TBM tunnel

Rock mechanical documentation of the TBM tunnel was performed after excavation and cleaning of the TBM tunnel. The objective of rock mechanical documentation of the TBM tunnel and the methodology used were the same as for the drill-and-blast tunnel /Stenberg, 1994/.

Table 4-1. Classification parameters and their rating in the RMR system used at Äspö HRL.

Parameter	Assessment of values and rating						
Intact rock UCS (MPa)	> 250	100–250	50–100	25–50	1–25		
Point-load strength index (MPa)	> 10	4–10	2–4	1–2	0.04–1		
Rating (see note 1.)	15	12	7	4	1		
RQD (%)	> 90	75–90	50–75	25–50	< 25		
Rating	20	17	13	8	3		
Spacing of discontinuities (m)	> 2	0.6–2	0.2–0.6	0.06–0.2	< 0.06		
Rating	20	15	10	8	5		
Condition of discontinuities	Very rough. Not continuous. No separation. Unweathered.	Slightly rough. Separation < 1 mm. Slightly weathered.	Slightly rough. Separation < 1 mm. Highly weathered.	Slickensided or gouge < 5 mm. Separation 1–5 mm. Continuos.	Soft gouge > 5 mm or separation > 5 mm.		
Rating	30	25	20	10	0		
Groundwater state	Dry	Damp	Wet	Dripping	Flowing		
Rating	15	10	7	4	0		
Fracture orientation (°)	Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Dip 0–20 any strike
	Drive with dip		Drive against dip				
Dip (°)	45–90	20–45	45–90	20–45	45–90	20–45	
Rating	0	–2	–5	–10	–12	–5	–10

note 1. The rating was based on Intact rock UCS (Uniaxial compressive strength)

4.5.2 Rock mechanical characterisation in 50 m blocks

As a complement to continuous rock mechanical mapping, the following rock mechanical investigations were performed in the 50 m blocks:

- Rock stress measurements (for block-scale predictions).
  The measurements were carried out by means of overcoring technique, as further described in Section 10.2.
- Laboratory investigations on core samples (for detailed scale predictions).
  Core samples of Greenstone, Fine-grained granite, Småland granite and Äspö diorite (the four principal rock types) were collected for laboratory testing of rock mechanical characteristics (uniaxial compressive tests) and fracture surface properties (shear tests), see further description in Section 6.4.

4.6 Hydrogeological mapping

As for geological mapping and rock mechanical characterisation, hydrogeological mapping of tunnel walls was one of the major component of baseline hydrogeological characterisation, see Figure 1-5 and Figure 8-1. The second major component of baseline characterisation was hydraulic testing of probe holes, conducted approximately every 16 m along the whole tunnel, see Sections 5.2 and 8.2 and Figure 8-1.

Regarding data presentation and reporting, see Section 4.9.

4.6.1 Continuous hydrogeological mapping

Hydrogeological mapping was carried out continuously along the whole tunnel in conjunction with, and by the same characterisation team as, continuous geological and rock mechanical mapping. Water leakage was documented in tunnel roof, front and walls. No special hydrogeological mapping was done in the 50 m blocks.

Methodology for drill-and-blast excavation

Water leakage was mapped in direct conjunction with geological mapping /Christiansson and Stenberg, 1991/. The observed flow objects were documented as related to flow from rock, flow from contacts, flow from fracture and flow from fracture zone.

The nature and amount of leakage was characterised on a three-point scale (also noted on the mapping sheet):

v	patch of moisture, sporadic drops
vv	drops
vvv	flow

The quantity of leakage was estimated or measured using a graduated vessel and a stopwatch, or the number of drops per 15 sec were counted. A more detailed description of the characterisation of water leakage into the tunnel is given in /Rhén et al. 1994a/. Methods for more detailed inflow measurements, performed for checking purposes are also described in the same report.

The type of leakage was defined as Diffuse, Point, Node, Extensive or leakage from Bolt holes /Rhén et al. 1997b/.

The length of the wetted surface (wet fracture length) or, if it was diffuse, the form and area of the individual leakage was estimated. All leakage was marked, even from fractures shorter than 1.0 m.

Hydrogeological mapping in the tunnel was documented on the base mapping sheet and on a special field protocol, as in the case of geological mapping. A further description of data processing with the TMS is provided in Section 4.8.

Methodology in the TBM tunnel

Hydrogeological mapping of the TBM tunnel was performed after excavation and cleaning of the TBM tunnel. The methodology for hydrogeological mapping in the TBM tunnel was the same as in the drill-and-blast tunnel /Stenberg, 1994/. Figure 4-5 shows a relatively large water inflow in the TBM tunnel.

Figure 4-5. Photo from the TBM tunnel showing a point inflow of water. These kinds of distinct inflows are possible only in a TBM tunnel, where the flow paths close to the tunnel are not disturbed as in a blasted tunnel.

Methodology in the shaft

In the shaft, only water-bearing fractures were noted in the field form. No classification of moisture as drops or flowing water was done.

4.6.2 Accuracy

If the inflow was diffuse, or only moisture, it was difficult to estimate from which fracture the leakage came from. If the inflow was very little the amount of the inflow was difficult to estimate. Inflows as low as one drop per 15 seconds were noted, which corresponds to 0.00025 l/min, if one drop corresponds to 0.25 ml /Rhén et al. 1994a/. However, in a test performed the range was from 4 drops per ml and up to 12 drops per ml, with a median value of 8 drops per ml (1 drop is 0.12 ml).

4.7 Groundwater chemical sampling

The baseline hydrochemical characterisation included sampling of water during tunnel wall mapping (described below) and sampling of water during probe hole drilling (described in Chapter 9), both of which were included in the documentation sampling programme see Figure 1-5 and Figure 9-1. However most of the samples were taken from the probe holes and only a limited number from water leakage on the tunnel wall.

Regarding data presentation and reporting, see Section 4.9.

No baseline hydrochemical sampling was performed in the TBM section and in the shaft.

4.7.1 Groundwater sampling from tunnel wall leakage

Baseline hydrochemical sampling from tunnel wall leakage was carried out along the whole tunnel in conjunction with, and by the same characterisation team, as continuous geological mapping. No special hydrochemical sampling was done in the 50 m blocks.

Methodology for drill-and-blast excavation

Sampling was limited to leakage points with relatively large flows, i.e. all points with flows $\geq$ 1 l/min. Simple equipment, such as funnels, graduated vessels and stopwatches, were used. The water sampling procedure was as follows /Christiansson and Stenberg, 1991/:

- A volume of 1 l was collected.
- The date and time when the sample was taken were noted on the bottle. The sampling point was determined by chainage and ID for the leakage point, according to description in Section 3.5.
- The sample was then taken to the on-site mobile chemistry laboratory for further analysis.

The water samples were processed, analysed and stored as documentation samples (chemistry class 2), entailing determination of pH, Cl, HCO_3 and electrical conductivity, see further description in Chapter 9.

4.7.2 Accuracy

It is not relevant to discuss accuracy with regard to the sampling of leakage water from the tunnel wall. Regarding the accuracy of the chemical analysis, see Section 9.2.

4.8 Tunnel Mapping System (TMS)

4.8.1 General

The purpose of the Tunnel Mapping System (TMS) is to facilitate collection, management, storage and presentation of tunnel wall mapping data in an integrated and efficient manner. The TMS is an user friendly application which was developed using the powerful features of MicroStation from Intergraph. MicroStation was used together with the database dBase IV from Ashton and Tate. The TMS application, Microstation PC and dBase were run on an IBM compatible PC. Two monitors were used for the PC, see Figure 4-2.

4.8.2 Data management

Application

TMS was developed using the UCM (User CoMmands) language in MicroStation version 4.0. The application is launched by typing TMS at the system prompt. When the application was running the operator performed different tasks by activating commands taken from drawn areas on a paper menu that is fixed to a digitising table.

The different commands are located at the left on the menu. The TMS form used for mapping (base map sheet) is placed on the right and digitised, and information from the mapping form is typed in the database. When the different geological structures have been digitised, links between the digitised object and the dBase database are created.

Methodology

As described in Section 4.2 a base map sheet and mapping form were produced by the TMS for each round, se also Figure 4-2. In the tunnel, mapping was carried out and documented manually on the base map sheet and form, according to Sections 4.3 to 4.7. After each round the standard procedure was to transfer all information to the TMS database, with the exception of chemical analyses of water samples. The manually drawn lines, contours, data points, etc, were digitised and the parameter information was typed in.

Data presentation and reporting are described in Section 4.9.

Backup of data

The files where initially backed up every working day on diskettes which were stored in a fire resistant safe. Later on the TMS was connected to the network. Then a directory on the server was used as an on-line archive. A complete security backup of the server was performed once a month. Incremental backups were carried out every night.

4.9 Data presentation and reporting

4.9.1 150 m overview sheets

Compilation and presentation of data from the baseline characterisation, evaluation and comparison with predictions, and reporting were carried out according to standard routines. After each 150 m of excavation, all data from tunnel documentation and the probe holes were compiled and presented in a set of three condensed 150 m overview sheets concerning:

- Geology.
- Geohydrology and Groundwater Chemistry.
- Reinforcement and Pre-grouting.

Only data from the continuous characterisation along the whole tunnel is presented in these sheets. Data from the 50 m block characterisation is presented in the project reports, see Section 4.9.2, while data from the characterisation in the shaft is presented in /Rhén, 1995a/. All overview documentation sheets were also compiled in two reports after the entire tunnel was finished /Markström and Erlström, 1996/ and /Markström, 1997/.

Geology

The geological sheet provides a summary of lithology and fracture data from continuous mapping, see Figure 2-8. The lithology and fracture map is taken directly from the TMS database. However, the individual fractures in areas evaluated as fracture zones are excluded on the fracture map. Statistical analyses of fractures with regard to orientation and minerals are presented, normally in 50 m sections. Moreover, sampling points and tunnel sections within 50 m blocks or where special experiments were conducted are indicated.

Geohydrology and groundwater chemistry

The geohydrology and groundwater chemistry data from continuous tunnel characterisation are presented on the same data sheet, see Figure 2-9, which also includes the data from the probe hole investigations, see also Chapters 5, 8 and 9. Presentation of water-bearing structures and points of water leakage are taken from the TMS data base. Location of probe holes and data on water outflow, calculated transmissivity values as well as the first period of groundwater pressure are presented. Groundwater chemistry data include indication of sampling points (probe holes or tunnel leakage) and Cl^- and pH levels.

Reinforcement and pre-grouting

The reinforcement and pre-grouting sheet also provides data from continuous rock mechanical characterisation, see Figure 2-10. The RMR (Rock Mass Rating) is taken from the TMS database. Documentation on reinforcement (location and type of shotcrete, bolts and in a few cases net), pre-grouting and weekly advance was obtained from the construction team.

4.9.2 Data evaluation and reporting

Data from baseline characterisation were analysed and compared with the predictions. This evaluation includes the entire baseline characterisation, i.e. continuous characterisation along the whole tunnel, the probe holes, characterisation of the shafts and characterisation of the 50 m blocks, as well as the results of monitoring and special investigations described in other chapters of this report. The evaluation work is structured according to subject areas and reports are written for tunnel intervals of approximately 700 m, i.e:

- Geological-structural and rock mechanical evaluation (example of report from tunnel interval 2,265–2,874 m is /Stanfors et al. 1994/).
- Hydrogeological evaluation (example of report from tunnel interval 2,265–2,874 m is /Rhén et al. 1994a/).
- Evaluation of groundwater chemistry and transport of solutes (example of report from tunnel interval 2,265–2,874 m is /Wikberg et al. 1994/).

Example of further data evaluation from the baseline characterisation is presented in Figures 4-6 and 4-7, see also Section 2.2.5.

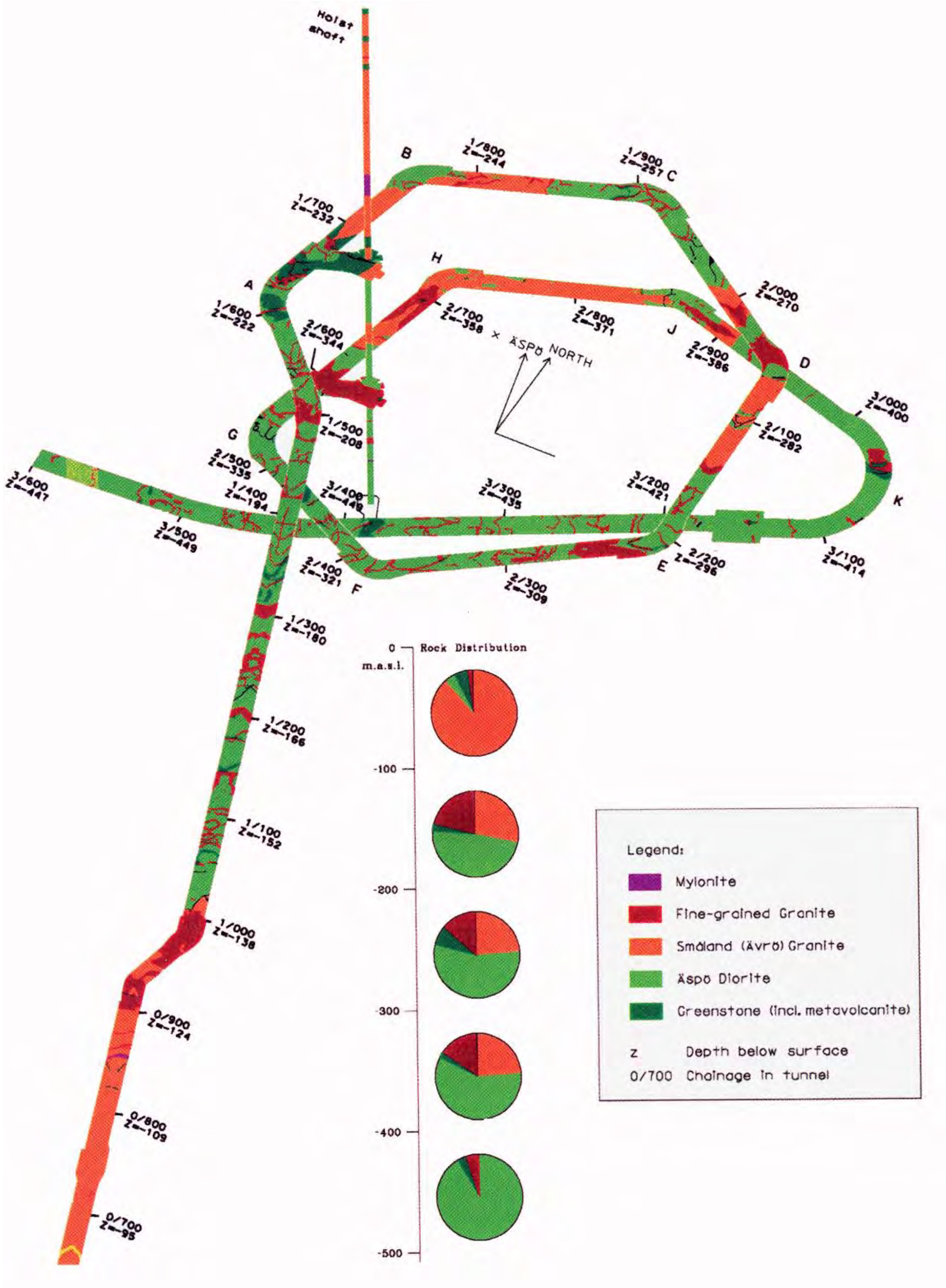


Figure 4-6. *Lithology of the Äspö tunnel.*

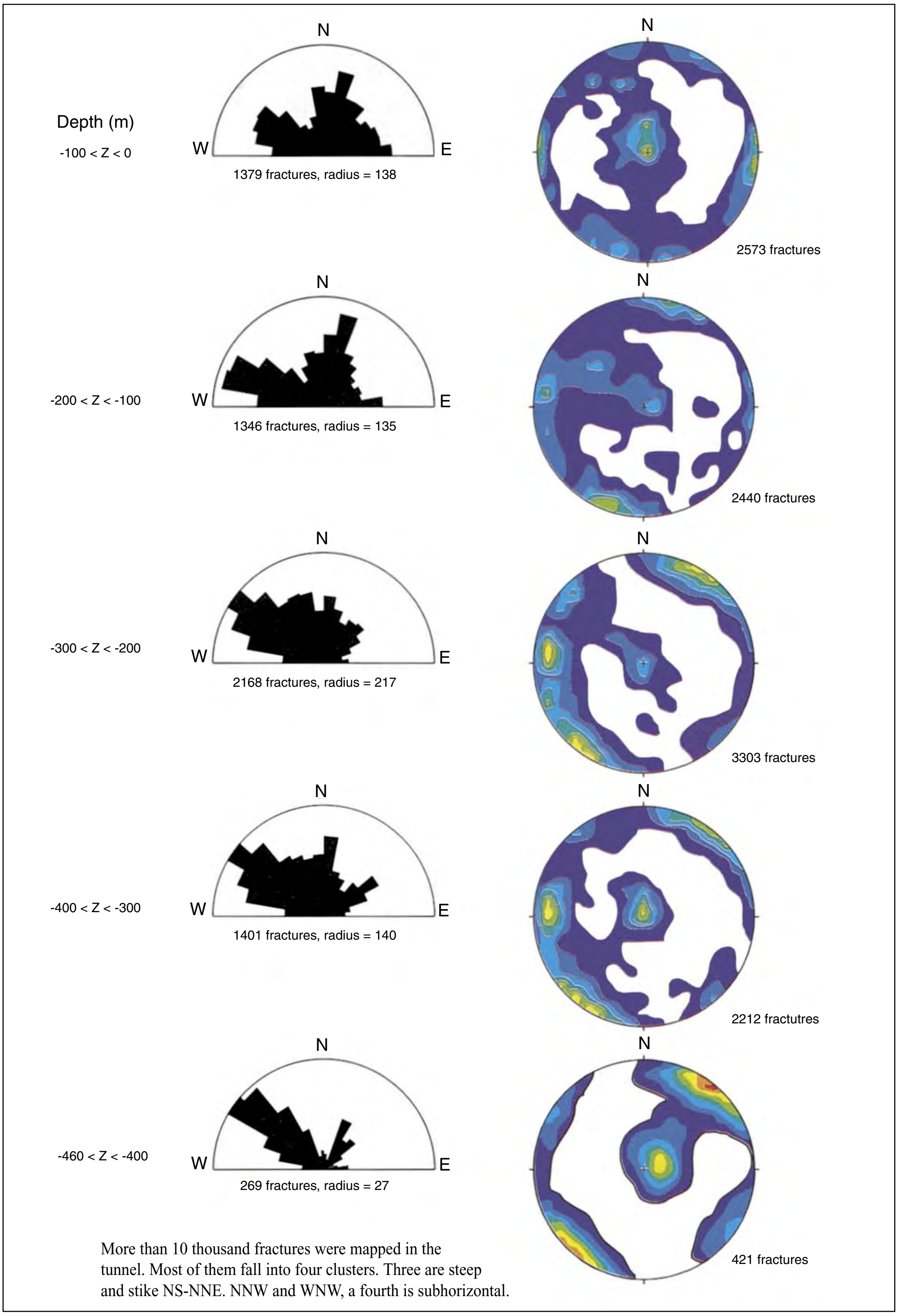


Figure 4-7. *Orientation of main fracture sets in the Äspö tunnel.*

4.10 Comments and recommendations on tunnel mapping

Characterisation was integrated with tunnelling and normally performed during one hour between each drill-and-blast cycle. This means that if fracturing was extensive, as in fracture zones, the limited time available sometimes resulted in a simplified characterisation of the fracture zone, i.e. all details could not be visualised. In some fracture zones the exposed rock was not stable and therefore had to be reinforced. In those cases the geologists were not allowed to enter the newly excavated tunnel part, and mapping had to be performed from the previous excavated round where the tunnel was safe, i.e. from a distance of approximately five metres. Mapping was then performed only as overview mapping, no details could be characterised. Mapping was supplemented by photographic documentation.

A complex fracture or fracture zone could be digitised in the TMS as a single fracture. A notation was then made that the fracture contained one or more parallel fractures with a specified spacing between the fractures. The fractures were generalised during drawing of the fractures. All parallel fractures and splay cracks were not visualised.

All drawings of fractures were done in 2D with the walls folded out. Drawings of oriented fractures in 3D, as well as true coordinates for all fractures, were provided later in the project. This could be done by computer calculations based on the mapping information and the tunnel geometry. Each individual fracture has to be individually measured by the total station if the accuracy of fracture location and orientation is to improve.

The one hour available for the baseline characterisation (two hours when probe holes were drilled) was often inadequate, especially as the allotted time was determined by the construction team and the characterisation team had to be ready to rush into the tunnel in order to use the allotted time most efficiently. Sometimes there were waiting periods if the construction work was delayed, but it was never possible to prolong the characterisation period. From the viewpoint of characterisation, these strict routines were not good. It would be better to have a more flexible time for characterisation. It might be slightly more expensive, but the quality of the characterisation would be better, although not more detailed.

The standard methodology for water leakage characterisation is difficult. The special measurements mentioned in Section 4.6 /Rhén et al. 1994a/ can probably be used more regularly.

When characterisation procedures are compared between the drill-and-blast tunnel and the TBM tunnel, the following conclusions can be drawn:

- Mapping of the TBM tunnel could not be performed at the tunnel front, as it can in the case of the blasted tunnel. On Äspö, the whole TBM tunnel (409 m in all) was, consequently mapped after it was completed. It would be possible to perform tunnel wall mapping from the TBM backup rig, some 20–30 m behind the tunnel front. But the actual tunnel face could never be mapped.
- One advantage of mapping directly after excavation is that the tunnel is clean. After a few rounds the tunnel wall is very dirty and has to be cleaned. This is of most importance for the blasted tunnel, however.
- Small fractures are more difficult (or impossible) to see in the TBM tunnel. On the other hand, some of the observed small fractures in the blasted tunnel may have been caused by the excavation procedure. Which of the fracture representations is most relevant is a matter of debate, see Chapter 13.

- The geometry of visible rock structures (fractures, dikes etc) was somewhat easier to document in the TBM tunnel. But the orientation, character and coating of individual fracture surfaces were almost impossible to measure and document, since they are not exposed as in the blasted tunnel.

The standard procedure for manual mapping in the tunnel and digitisation of map and data at the office was in general adequate. The intention was to digitise the previous round before mapping the next one. Occasionally this was not done, however, mainly due to the fact that the persons involved in the characterisation team was too few. It might be advisable to introduce a more computerised mapping procedure in the tunnel. Limited tests of bar code techniques and pen computers were carried out. The use of pen computers would probably be efficient, especially after the hardware is improved and is practical for use in harsh environments.

The 150 m-documentation sheets were very useful for condensed presentation of results. Production of the first sheets was relatively time consuming, but the procedures were eventually rationalised and the sheets were normally ready in about three weeks after completion of excavation of the tunnel interval in question.

Groundwater sampling from tunnel walls was phased out after some 500 m of tunnelling, after which samples were mostly taken from probe holes (with a few exceptions). The reason was that the quality of samples was better when taken from boreholes.

5 Drilling and related activities

5.1 General

Besides characterisation of tunnel walls, drilling and measurements in boreholes were the most important source of information for the geoscientific characterisation of the underground rock during tunnel construction. Drilling and borehole investigations are associated with baseline characterisation, monitoring and special investigations, see Section 2.2 and Figure 1-5.

Two categories of boreholes were used and will be discussed in this chapter:

1. Probe boreholes:
   Short (normally 20 m) percussion-drilled boreholes, regularly drilled and investigated after every fourth round within the baseline characterisations.
2. Investigation boreholes:
   Boreholes (cored or percussion-drilled) intended for certain purposes, drilled and investigated according to programmes specially designed for each borehole. Some of these holes were drilled as long probe holes through fracture zones etc.

An overview of the boreholes drilled in the tunnel during the construction phase is given in /Stanfors et al. 1997a/, see also Figure 2-4.

Drilling itself and measurements carried out during drilling are described in this chapter, while the different types performed are discussed in the following chapters.

In addition to these types of boreholes, data were also gathered and evaluated from grout injection boreholes. Drilling of these holes was similar to drilling of the probe holes. Data collected in these holes were observations of water flow, performed injection tests and grouting itself. These data were collected by the construction team, but have also undergone further evaluation /Stille et al. 1993/. These grouting activities will not be further dealt with in this report.

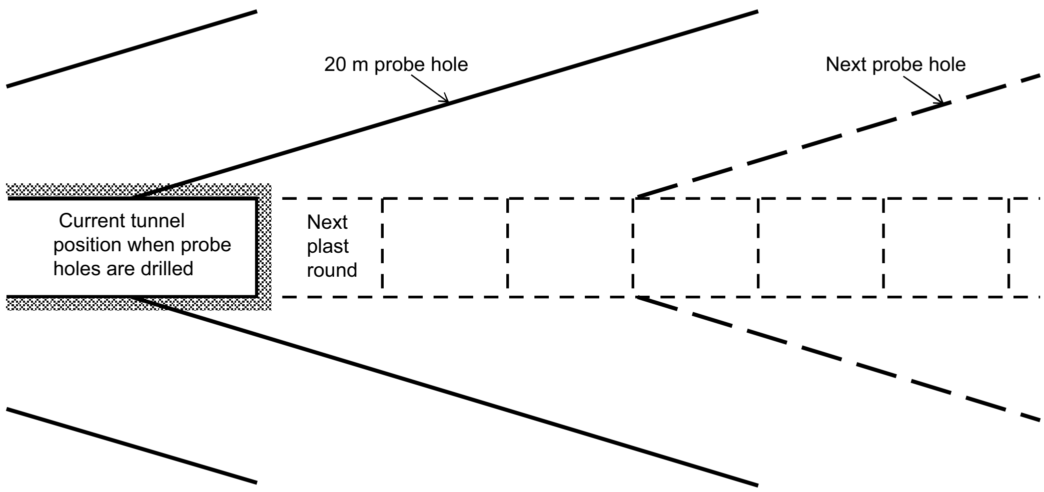


Figure 5-1. The probe holes were drilled to a length of about 20 m and were positioned about 4 m (one round length) from the tunnel face.

5.2 Probe boreholes

5.2.1 Purpose

Probe boreholes with associated borehole investigations comprised the second leg of baseline characterisation, see Figure 1-5. The general purpose of the probe holes was:

- To provide geoscientific information on the rock in the near field around the tunnel, to supplement the tunnel wall mapping discussed in Chapter 4. Focus was put on gathering of hydrogeological information.

The objectives of probe hole drilling and measurements during drilling were as follows:

- To provide a hole for the pressure build-up test.
- To provide additional data for evaluation of the pressure build-up test (lithology, position of water inflow and rate of drill penetration).
- To provide observation points for monitoring groundwater pressure.
- To provide an opportunity for sampling formation water.
- To enable the contractor to predict rock quality and the water situation ahead of the tunnel face.

The information obtained from drilling was used together with tunnel wall mapping for comparison with predictions and detailed geological characterisation.

Altogether, 302 of these 20 m probe holes were drilled within the baseline characterisation programme along the blasted tunnel and 14 along the TBM tunnel section. In addition, approximately 139 percussion-drilled investigation holes were drilled for other reasons, such as passage of more complex fracture zones, see Section 5.3.3.

5.2.2 Probe hole drilling in blasted tunnel

Methodology

Two 57-mm 20 m long probe boreholes were drilled every 16 m on either side of the tunnel face (approximately every 4th round) /Christiansson and Stenberg, 1991/. Probe hole drilling was performed in conjunction with the tunnel wall mapping, as indicated in Section 2.2.2, and was succeeded by hydraulic testing. The probe holes were directed approximately 20° out from the tunnel wall. They were aligned parallel to the bottom of the tunnel, i.e. inclining slightly downwards (approximately 14% or 8°). The probe holes were drilled to a length of about 20 m and were positioned about 4 m (one round length) from the tunnel face, see Figure 5-1. The probe boreholes were percussion-drilled with the production drilling rig, which was a 3-boom rig with 16 ft feeders, see Figure 2-2. The drill bit, 57 mm in diameter, was cooled by water flushing.

Instructions on collaring and the direction of the probe holes in relation to the tunnel direction were given to the contractor, who performed drilling in the same standard way for all holes.

Accuracy

The accuracy of the location of the percussion-drilled probe holes was ± 1 m, see also Chapter 3. The accuracy of the determination of the direction of the probe holes was ± 5°. The deviation of the boreholes was not measured, but was estimated to be less than 2 m for a 20 m probe hole. The accuracy of borehole length was 0.5 m. The accuracy of borehole diameter is dependent on the wear of the drill bit and is estimated to be ± 2 mm.

5.2.3 Probe hole drilling during TBM excavation

The 409 m long TBM tunnel was excavated by means of a totally different tunnelling technique compared with the blasted tunnel. The tunnelling itself did not call for regular interruptions for mucking out, etc, and the TBM machine occupied approximately 70 m of tunnel behind the tunnel front. The probe hole drilling programme therefore had to be modified.

The first four objectives of probe hole drilling in the blasted tunnel are in principle the same for probe hole drilling during TBM tunnelling, see Section 5.2.1. An additional objective for probe holes in the TBM tunnel was to evaluate the feasibility of integrating probe hole drilling with TBM tunnelling.

Due to practical problems, systematic probe hole drilling was only conducted between 3,436 m and 3,584 m of the TBM tunnel, see Figure 2-4. In the first part of the TBM tunnel a 200 m long cored probe hole was drilled and investigated, see Section 5.3. Some of the tunnel was excavated without any probe hole drilling at all. Altogether, only 14 probe holes were drilled during the TBM tunnelling.

The probe boreholes were drilled with two drilling rigs which were built into the machinery part of the TBM machine, see Figure 5-2. The probe holes were normally drilled through an opening in the TBM head to penetrate the tunnel front, and were lost as TBM tunnelling progressed. Only three probe holes were directed 7° out from the tunnel line, through the tunnel wall behind the TBM head. The lengths of these probe holes were between 16 and 24 m.

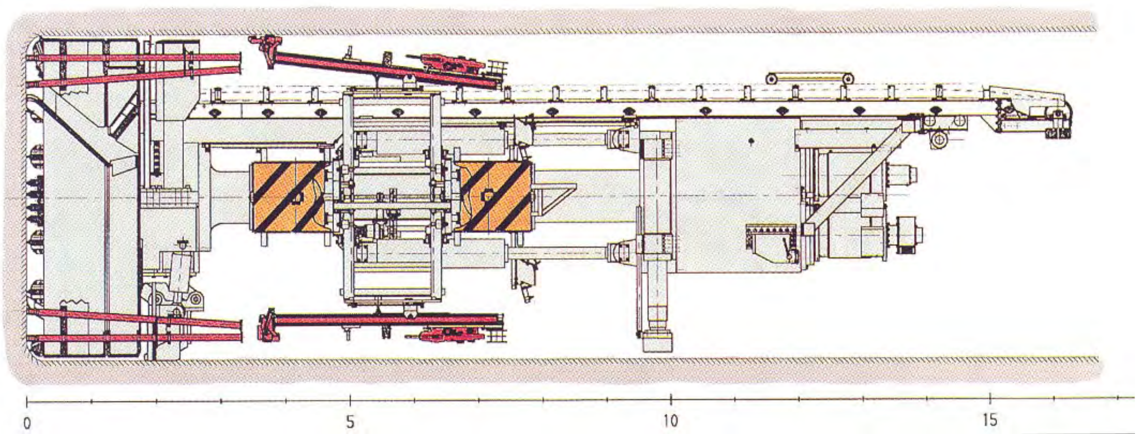


***Figure 5-2.** The two drill rigs (coloured red) on the TBM machine. The figure also shows the possible drilling directions through the TBM head (guide tubes shown in red) and the maximum drilling angle for inclined drilling through the tunnel wall.*

Accuracy

The accuracy of the location, direction and deviation of the TBM probe holes will not be discussed here, since only a few holes were drilled, no actual measurements were performed and most of the holes were lost. In some respect it might be relevant to refer to the accuracy of probe hole drilling in the blasted tunnel.

5.2.4 Measurements during probe hole drilling in blasted tunnel

The following parameters were recorded or observed during drilling of the probe holes in the blasted tunnel see Figure 5-3:

- Rate of penetration (ROP).
- Water inflow rate.
- Colour of drill cuttings in the flushing water.

Methodology

To determine the rate of penetration, the time required for the drill rod to penetrate 0.5 m was measured manually. As drill rods were added, the water inflow rate was also measured and noted in field forms. Observation was also made of the colour of the flushing water (or drill cuttings in the water) in order to determine changes in rock type, mineral alteration, etc. All observations, as well as the start and stop of each drill rod, were recorded in real time (in order to permit analysis of any hydraulic interference with other observation points).

The information was used in the geological and hydrogeological characterisation, see further description in Chapter 13 and Figure 13-1 and Figure 13-2.

Figure 5-3. *Penetration rate, colour of drill cuttings and water inflow rate were recorded during drilling.*

Drilling parameters were automatically recorded by a computerised drilling control device, Bever control, during the second half of the tunnelling. The drilling parameters measured were rate of penetration (ROP), penetration pressure, drive pressure and rotational pressure. This information complemented the manual observations made for the purpose of evaluating the usefulness of automatic recording. There was no automatic recording of the flush water flow rate and return water flow rate.

Accuracy

The accuracy of depth determination (of rock boundaries, fracture zones and water inflows) is estimated to ± 0.5 m. The determination of rock types, which was based on the colour of the flushing water, was influenced by the water flow rate and the length of the borehole, and it is not relevant to quantify its accuracy.

5.2.5 Measurements during probe hole drilling in TBM tunnel

The ambition was to observe and measure the same parameters during probe hole drilling in the TBM tunnel as in the blasted tunnel, i.e. ROP, water inflow etc (see Section 5.2.4, Methodology). However, the conditions for monitoring and documenting drilling from the TBM machine were different; the working space was limited and the characterisation team had no real access to the probe holes. Some observations of the water outflow rate and the colour of flushing water were however made by visual inspection through a hatch in the TBM head.

5.3 Investigation boreholes

5.3.1 Purpose

The main purpose of investigation drilling was:

- To characterise the far-field rock volumes 20–400 m from the tunnel.

A large number of investigation holes were cored, providing valuable samples along the entire borehole. An even larger number of percussion-drilled holes were drilled. The boreholes were used for various geoscientific investigations and monitoring, which are discussed later on in this report. Most investigation boreholes were associated with special investigation activities or sub-programmes, see Section 2.2.4 and Figure 1-5 and Figure 2-4.

The cored borehole was normally placed in a special drilling niche by the tunnel wall so that the boreholes could be drilled and investigated during ongoing tunnelling activities. The boreholes were drilled 20–400 m in the desired direction.

In cases where large water outflows and high water pressures were expected, drilling was done through a casing and a valve system, see further in Section 5.3.4.

5.3.2 Core drilling

Methodology

Standard diameter core drilling

Altogether, 51 cored boreholes were drilled from the main tunnel during the Äspö HRL construction phase, and an additional 50 or so normally short, cored holes from side tunnels, most of them within the ZEDEX experiment. In most cases 56 mm boreholes were drilled, but for special purposes 76 mm diameter boreholes were drilled. Drilling was normally carried out with standard T-56 or T-76 drill bits and core barrels, resulting in core diameters of 42 and 62 mm, respectively, see Figure 5-4.

A double tube core barrel with a length of 3 m was used. The cores were placed in a wooden core boxes. The depth of the core uptake was calculated from the length of the drill string in the hole and was noted on the core box. During core mapping the depth was adjusted depending on the length of the core (see Section 6.3).

During core drilling, flushing water was used to cool the drill bit and to clean the borehole from drill cuttings. In order to monitor groundwater contamination, a coloured tracer (normally uranine) was added to the drilling water in a certain concentration. This coloured tagging of the water was intended for all water consumed in the tunnel, and was done in an uranine station connected to the water supply line at the beginning of the tunnel (see Section 9.8).

Large-diameter coring

For a special study related to fracture aperture distribution, samples of fractures were taken by drilling large-diameter (200 mm) cored boreholes along a fracture plane from the tunnel wall. Drilling was performed with a 200 mm drill bit and a single core barrel, which resulted in core diameters of 192 mm. A core barrel with a length of 0.5 m length was used. These holes were normally only 1 to 2 m long. /Hakami, 1994/.

***Figure 5-4.** Drilling of short core boreholes in the TBM-tunnel.*

Accuracy

The accuracy of borehole location, direction and deviation was presented in Section 3.4.2. The accuracy of the drilling depth/length determination is estimated to be ± 0.01 m.

However, the depth of the core pieces retrieved may deviate from this drilling depth if the core was not broken at the absolute bottom of the current hole. This deviation cannot be determined exactly but can normally be adjusted for at the next uptake or a few uptakes later. Every uptake is independent on the preceding uptakes. A high core recovery reduces this problem, see also Chapter 6. Depending on the length of the core still standing in the bottom of the hole the uncertainty in borehole depth is estimated to be about ± 0.1 m.

5.3.3 Percussion drilling

Methodology

The methodology for drilling percussion investigation holes was similar to the drilling of the short probe holes (Section 5.2.2), i.e. the same drilling machine was used and the same types of data were recorded. Altogether, 139 percussion-drilled investigation holes were drilled.

Percussion drilling after TBM excavation

21 of the percussion investigation boreholes were drilled after TBM tunnelling was completed. The purpose of these holes was to monitor the water pressure around the TBM tunnel in detail, as a function of time and distance from the tunnel. The boreholes were drilled in the first 200 m of the TBM tunnel, to a length of 8 m and with an angle of 45° in relation to the tunnel axis, see Figure 2-4. Mechanically expanded triple packers were installed in the boreholes, see discussion in Section 13.3.7.

The drilling methodology and measurements during drilling, as well as accuracy estimates, were the same as for probe holes (Section 5.2).

5.3.4 Arrangements for high water outflows

For special cases, when the borehole was expected to penetrate highly water-conducting features in the rock, i.e. when there was a risk of large water outflows (often at high water pressures), the drilling technique and borehole completion was improved /Rhén and Stanfors, 1993/. The problem with these water outflows was encountered both during drilling and during the subsequent measurements in the boreholes. Flow rates greater than 1,000 l/min and groundwater pressures up to 4.5 MPa had to be managed.

Figure 5-5 illustrates the principle of how drilling was performed through the water conductive fracture zone NE-1. The work procedure was as follows:

- Excavate the drill niche.
- Drill a large-diameter hole for the casing.
- Excavate the pump niche (only used twice; KA1061A and KA1131B).
- Install the casing in the borehole, grout behind the casing and anchor the casing with bolts (from the pump niche).
- Install the valve and guide rail (out to the drill niche).
- Start core drilling from the drill niche through the casing and valve in the pump niche.

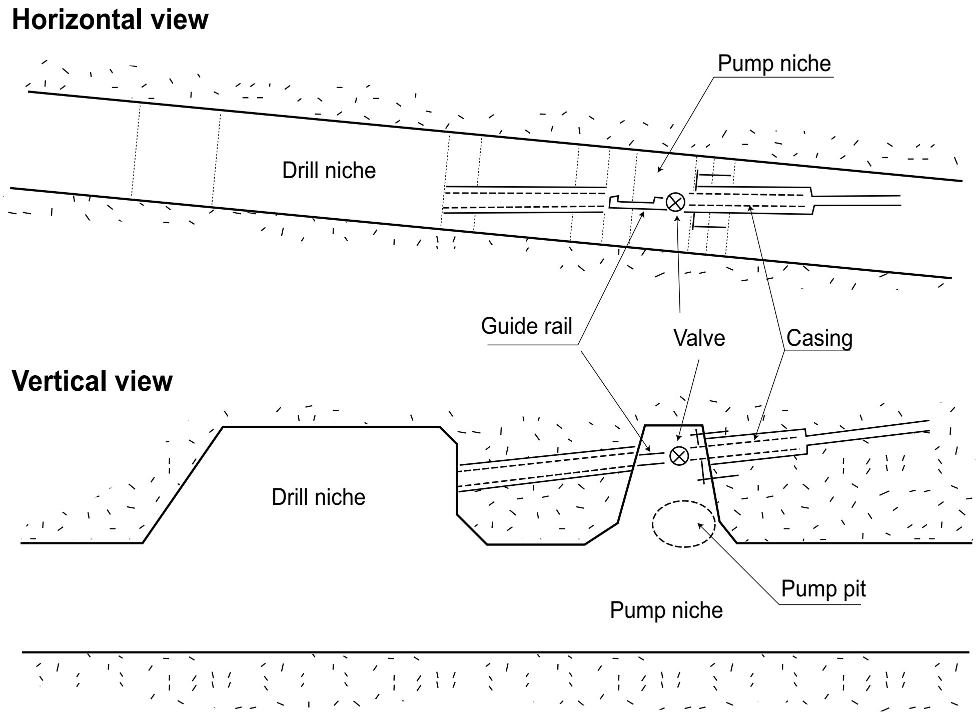


Figure 5-5. *Arrangement for drilling from the tunnel when high water pressures and large inflow rates are expected in the boreholes.*

Almost the same technique was used for percussion boreholes. The difference was that the casing was installed directly from the drill niche, and during drilling of the borehole to the final depth a protection device was attached to the casing in place of the valve. The valve was attached after the borehole was drilled to the final depth.

Figure 5-6 illustrates the technique of grouting and anchoring the casing and the valves at the end of the casing, see also Section 5.4. The photo shows the fountain of water from one of the boreholes penetrating through the highly water conducting fracture zone NE-1 when the valve was opened.

The arrangement also made it possible to stop the outflow of water by closing the valve when the drill string was pulled out for core recovery and when drilling was completed, see also Section 5.3.5.

5.3.5 Cement grouting during core drilling

Sometimes, due to mechanical instability of the borehole wall, heavy water outflows from the hole, etc, cement grouting had to be performed in order to be able to run geophysical probes, to set packers in the hole or even to be able to continue drilling. The intention was to grout only the problematic sections of the hole in order to minimise the hydraulic and/or groundwater chemical impact. Normally, hydraulic pressure build-up tests (or flow meter logging) were carried out before grouting. Radar logging is an example of a method, which should also preferably be performed after grouting.

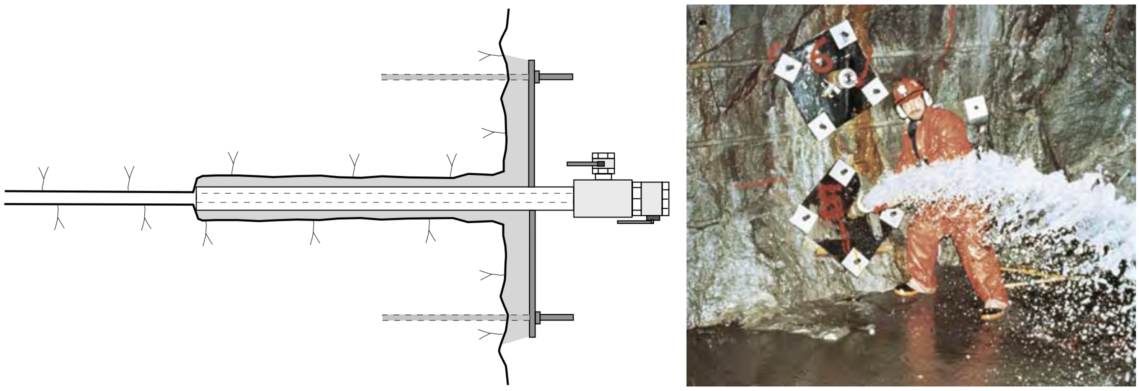


Figure 5-6. *Casing arrangement for sealing of boreholes. Left: casing, sealed and anchored with grout and bolts, and valves. Right: water fountain when the valve is opened in a borehole penetrating fracture zone NE-1.*

The most common reason for grouting was excessively high water outflows from the borehole or mechanical instability in the borehole (often a combination of both). Grouting was performed in different ways, mainly depending on the depth to the water inflow into the borehole. As the length and number of grouted sections should be minimised, grouting was normally performed during drilling with the aid of a specially designed packer system. Deciding when drilling should be interrupted for grouting, and what interval should be grouted, was tricky. As a guideline, an acceptable outflow rate at which borehole probes, i.e. radar probes, could be fed into a hole was 50 l /min.

A single packer, see Figure 5-7, was installed in the borehole alongside the drill string and inflated with pressurised water through a separate water pipe. Cement was then injected through the drill string and the hole was grouted from the packer down to the bottom of the borehole. After hardening the grouted interval was re-drilled and drilling of the borehole was continued. The expected result was a stable borehole with acceptable water flow rate, partially grouted and accessible for borehole investigations, although for a limited number of methods. If the inflow point was less than, say, 50 m from the casing, grouting was normally carried out from the borehole casing to the end of the borehole.

As an option, the grout injection packer system could be used for grouting between two packers. This permits retroactive grouting of borehole sections when drilling has proceeded far beyond the relevant section so that single packer grouting is insufficient. This double packer grouting technique was developed late in the project and was used successfully only one time.

5.3.6 Measurements during core drilling

Observations of water outflow during drilling were made in order to be able to correlate abrupt changes in outflow with hydraulic pressure responses in surrounding boreholes connected to the hydro monitoring system, see Chapter 11. Such changes could be treated as interference tests, if satisfactorily documented. Outflow measurements were also used to judge whether grouting was necessary, as discussed in the previous section.

Figure 5-7. Packer equipment used for grout injection during core drilling. The uppermost for 76 mm boreholes (single packer injection) and the lower for 56 mm boreholes (single packer and double packer injection).

Methodology

Drill start and drill stop for every rod were measured in real time. Whenever water inflow was observed it was noted in real time and the amount of water was estimated or measured with a funnel or a graduated vessel and a stopwatch.

Automatic recording of drilling parameters using a GEOPRINTER was only done while drilling one cored borehole. As the equipment did not work properly (due to malfunction and/or error in the maintenance of the equipment) and plotting was very time-consuming, neither this equipment nor any other system for automatic recording of drilling parameters was used.

5.4 Borehole completion

Casing was never installed along an entire borehole. For most boreholes (percussion-drilled and cored) no casing was used at all. However, when high water outflow was expected (as was discussed in Section 5.3.4) a casing was installed in the outermost part of the borehole. This short casing (3.3 m) was grouted and anchored with bolts. The casing was equipped with a valve to contain the borehole water and to keep the formation water pressure as natural and stable as possible. The same arrangement was also used for most investigation boreholes (percussion-drilled and cored) at lower levels of the tunnel, from approximately 1,000 m tunnel length, and those for which borehole measurements (in particular hydraulic measurements) were planned to be done later on.

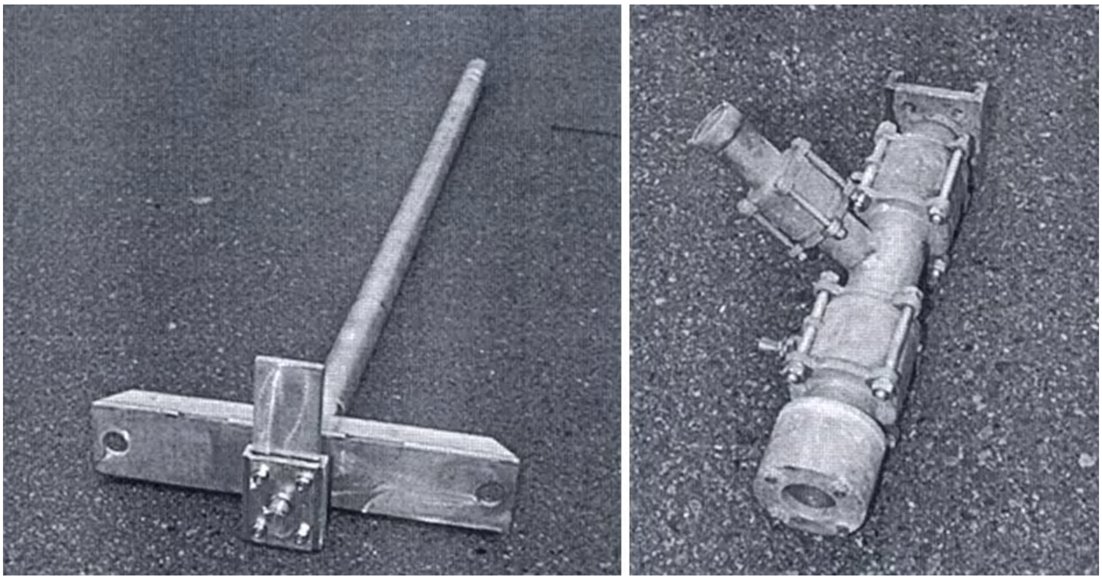


Figure 5-8. Improved borehole sealing arrangement. Casing with docking device for installation in a borehole (as in Figure 5-6), mounted with a pressure containment lock. Photo also shows a sealing device with ball valve used during drilling.

The casing and valve arrangement was progressively improved during the course of the project. The improved version includes a kind of docking arrangement, which allows for smooth interchange between different pressure containment devices, such as pressure locks, ball valves, borehole sealing for use during hydraulic tests in the borehole, etc, see Figure 5-8.

All other boreholes were sealed by means of at least one packer close to the mouth of the borehole. The packers were set immediately after the hole was drilled, see further in Chapter 8.

5.5 Comments and recommendations

Observations during drilling of percussion boreholes provide good opportunities for estimating rock quality and water inflow, at least for construction purposes. Automatic recording of drilling parameters can provide useful information if the data is systematically correlated with tunnel mapping data.

Protection of installed packers in the probe holes did not always work sufficiently. Sometimes the protruding part of the packer was destroyed or damaged either by blasting or more frequently by the mucking out machine. It worked out better when the position of the packer was marked with a coloured mark on the rock surface above the packer and by communication with the contractor people.

Technology and procedures for probe hole drilling during drill-and-blast tunnelling turned out to work well. However, the opposite was true of probe hole drilling during TBM tunnelling. Neither procedures nor technology were suitable for practical use, so probe holes were only drilled along a part of the TBM tunnel. The integration of the probe hole drilling machine with the TBM and the working conditions for the characterisation team must be improved, and better opportunities must be provided for following and documenting the drilling procedure. It should be possible to drill probe holes at a larger angle from the tunnel line, and the direction of drilling must be more flexible in general, see further the hydrogeological comments in Chapter 8.

Probe hole drilling may also be done with long probe holes, such as the 200 m cored hole along the first part of the TBM tunnel. Which procedure is the best depends on the conditions and what kind of information is expected from the probing. Installation of casing when large inflow rates and high water pressures were expected has worked very well.

Borehole instability and high water inflow during and after drilling is always a problem underground. Grouting may be used but is always a compromise between the different users of the borehole. Some measurements can be performed before grouting, while other methods have to be excluded.

Single packer grouting during drilling worked quite well, but selective grouting between packers was only successfully performed once, and cannot yet be regarded as being a proven technique.

6 Geological borehole investigations

6.1 General

An extensive geological-structural characterisation programme was carried out in boreholes during construction of the Äspö HRL. The general purpose of the geological investigations was:

- to provide data for comparison with predictions and to submit additional geological-structural information for updating of models.

An overview of the subjects forming the basis for the comparison is shown in Table 13-1.

While the geological mapping of the tunnel was described in Chapter 4, the geological investigations performed in the different boreholes will be described and discussed in this chapter, see Figure 6-1. With reference to Section 2.2 and Figure 1-5, this chapter will describe and discuss all geological borehole investigation methods, most of which pertain to special investigations, while some pertain to baseline characterisations in probe holes. The methods are:

- Investigations of drill cuttings.
- Core logging.
- Borehole TV logging.
- Laboratory analyses of rock samples.

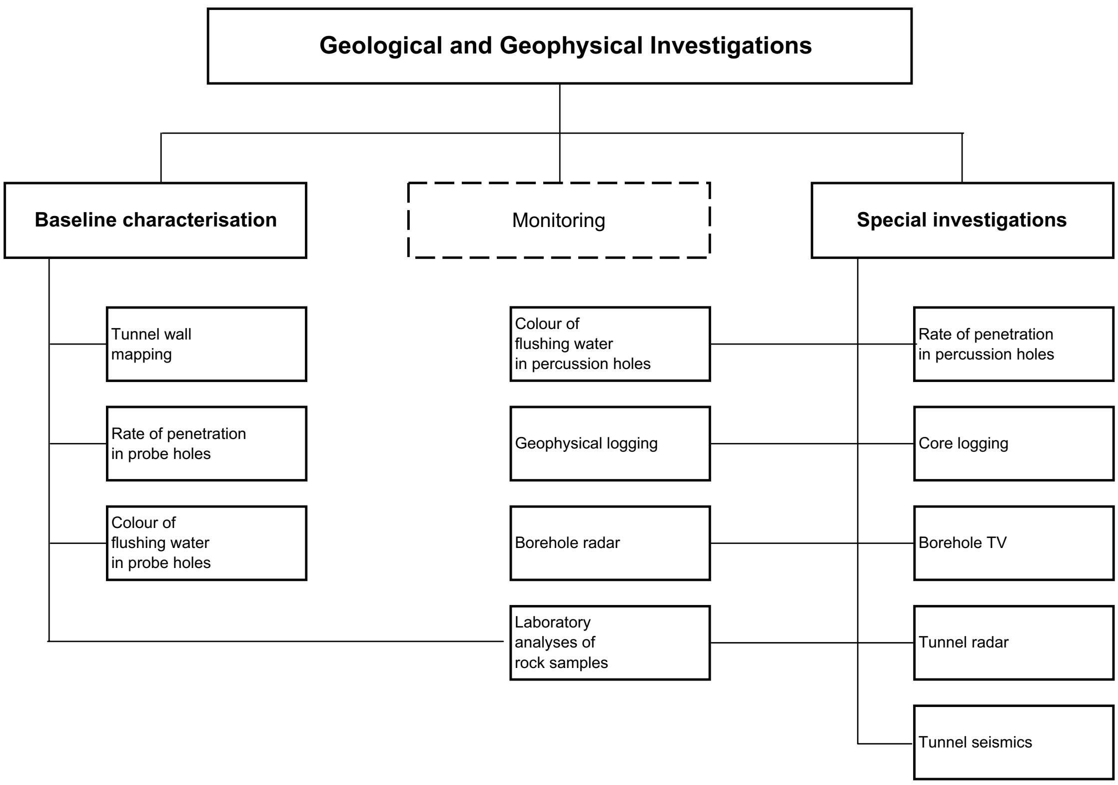


Figure 6-1. Geological and geophysical investigations performed during the Äspö HRL construction phase.

The drill cutting investigations were mentioned in Section 5.2.4, while the other methods will be described in this chapter.

The described methods are used for the characterisation of lithology (rock type, mineral composition, etc), rock structures (fractures, crush zones, foliation, etc) and mechanical properties.

6.2 Core logging

6.2.1 Purpose

The purpose of core logging was:

- To determine rock type distribution, content and character of fractures, crush zones and other geological-structural properties of a continuous core sample representing the rock formation along and around the borehole.

Besides for comparing with predictions, core logging data was used for detailed planning of the construction work and for updating the geological model.

During drilling, "preliminary core logging" was performed to get quick information about fracture density, fracture zones and lithology, in order to make decisions for sampling, special tests or to stop the drilling.

The core can be examined and analysed in a number of ways, with more or less use of additional data from geophysical logging, drilling parameter recording, etc. Core logging is the descriptive documentation of the continuous rock sample along the borehole. For engineering purposes, core logging is mainly used for documentation and to determine rock quality. For the geoscientific investigation programmes, core logging and geological characterisation also provide the framework for the other geoscientific models.

Approximately 100 investigation boreholes were core drilled during the Äspö HRL construction phase (including the ZEDEX experiment), and most of the cores were logged using methods presented in this chapter /Stanfors et al. 1997a/.

6.2.2 Instruments

Core logging was performed using the computer-based core logging system PetroCore. This system was developed and introduced for the pre-investigation phase of the Äspö HRL /Almén and Zellman, 1991/. It has been used for logging of almost all cores throughout the construction phase and during the operation phase of the Äspö HRL.

The PetroCore System uses a length measuring unit with a pointing device connected to a code wheel that produces the length values along the core. This device is placed on top of the ordinary core box and connected to the computer, see Figure 6-2. Geological characterisation data are typed in via the keyboard. The PetroCore software includes a database and programmes for recording, processing and printing/plotting of results.

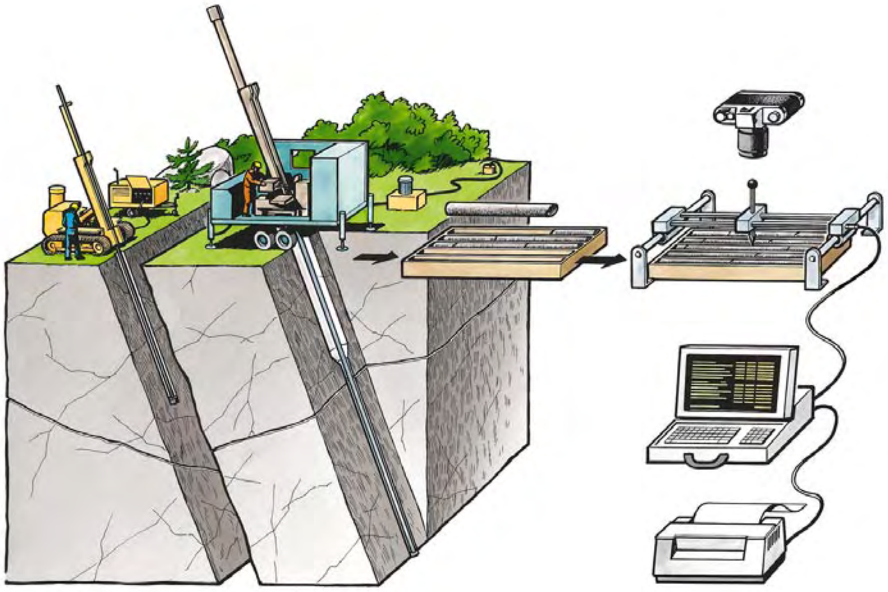


Figure 6-2. *Illustration showing the principles of the PetroCore system.*

6.2.3 Methodology

Careful handling of the core at the drill site and measurement of drill length for each core uptake are important prerequisites in this process. For every core uptake, the drillers measure the actual drilling depth and note it on the core box as well as in the "Drillers' record". These uptake depths constitute bench marks along the borehole.

Right next to the drilling machine, a site geologist first made an initial crude geological analysis, "preliminary core logging", concerning mainly rock type, fracture zones and sometimes intensity of fracturing (RQD or fractures/m).

Complete core logging was then performed in the core logging laboratory, which was housed in a standard 18 foot container. Before logging of the core, a length check was performed. In practice, a section of the core is reconstructed and adjusted to the uptake depths, actually stretched or compressed a little. Adjustment for stubs and core losses were made to make the core fit the bench marks. Some bench marks did not match the overall scheme, due to either inaccurate measurements or interpretations of stubs or core losses. In those cases the marks were excluded and the logging had to rely on the marks that fit the scheme. After adjustment, the uptake depths (bench marks) were marked on the core.

The length measuring device from the PetroCore System was then placed on top of the core box. The first step of the core logging process comprised recording of the length data (position along borehole) for all objects such as rock boundaries, rock alteration, veins, fractures, crush zones, etc. Step 2 comprised characterisation of the aforementioned objects, such as:

- Rock type (colour, structure, grain size).
- Alteration in the rock (type, intensity).
- Structural feature in the rock (type, intensity).

- Natural fracture (filling, surface, roughness, alteration, width, aperture and orientation to core axis).
- Sealed fracture (filling, width and orientation to core axis).
- Vein (filling, width and orientation to core axis).
- Crush zone (character as for fracture and piece length).
- Core loss.

These data were typed into the PetroCore database. Additional tools for the characterisation were angle measuring device, knife, magnet and hydrochloric acid. The core was described, from top to bottom according to instructions. If necessary, samples were taken and sent for further analysis, see Section 6.4. Complot images were produced progressively during core logging, see Figure 6-3.

A colour photograph was taken on the core, both in dry and wet condition. Wetting the core makes the lithology more clearly visible, while the dry core shows the fractures better. Finally, before storage, a raw data plot and a backup disc were produced. The principle data flow in core logging is shown in Figure 6-4.

Further evaluation of the core logging data normally took place at the office. Special investigations with customised plots and the final standard complots, including RQD and fractures/m, were produced. The data files were then transferred to the SICADA database, see Chapter 12.

The PetroCore System also handled orientation of data. If the core was logged with relative orientation, orientation data were related to a reference line. In the event TV-logging had been used for true orientation of certain structures, these true oriented structures could be used for performing true orientation of all structures within reconstructed core intervals. Orientation data were plotted in stereo diagrams. However, even though the PetroCore System was used extensively orientation of cores was only done on a selected number of boreholes.

6.2.4 Accuracy

The major sources of error in core logging are:

- For length measurements:
  - Inaccuracy in core uptake depths.
  - Core losses and difficulties in re-assembling the core string.
- For fracture orientation:
  - Non-planar surface.
  - Orientation of the core.
- For determination of lithological and structural parameters:
  - Rock type.
  - Type and character of fracture.
  - Type of fracture filling.

Length accuracy is very dependent on the skill of the drillers. Careful handling and accurate measurement are essential for getting good results.

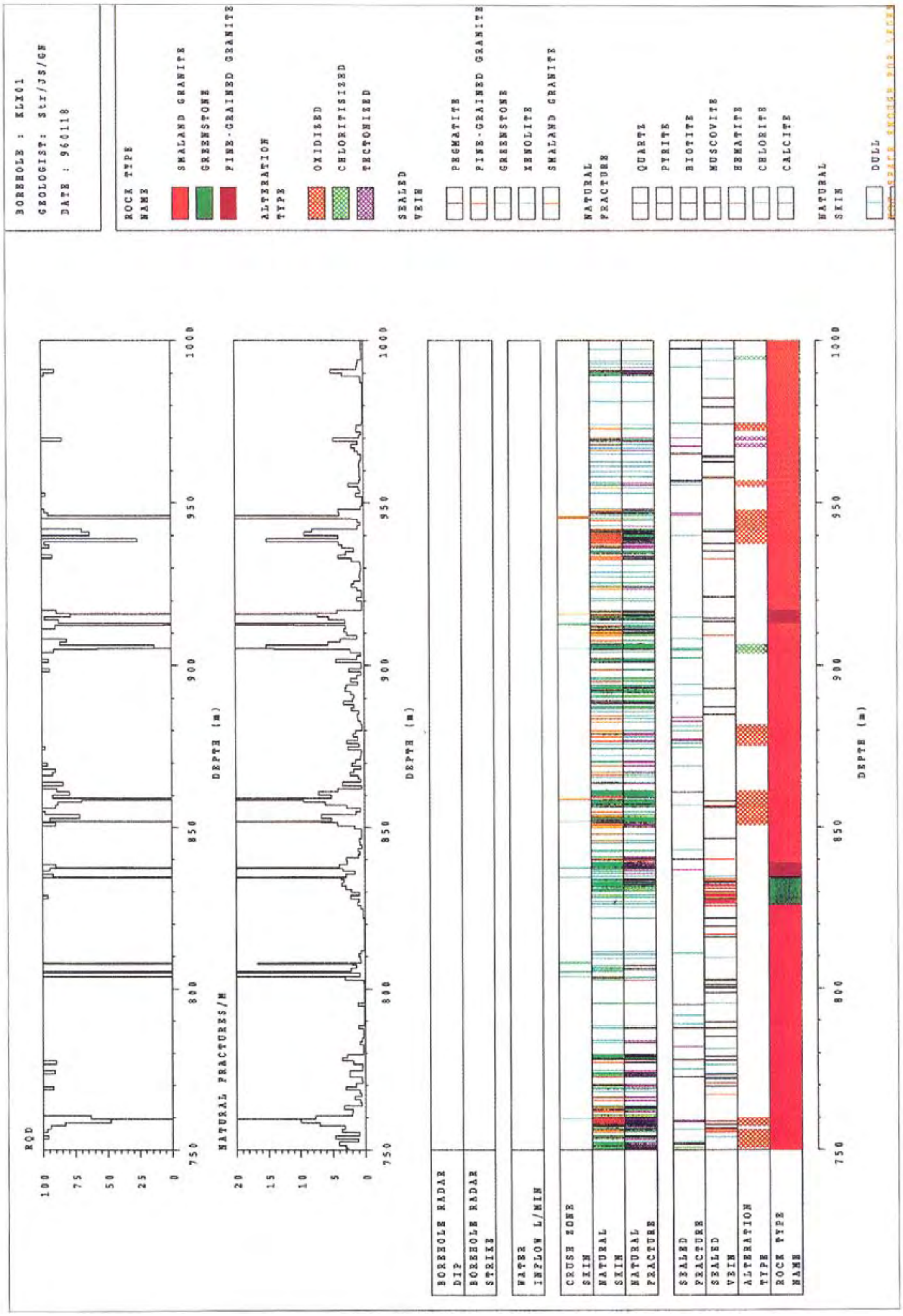


Figure 6-3. *Core log from borehole.*

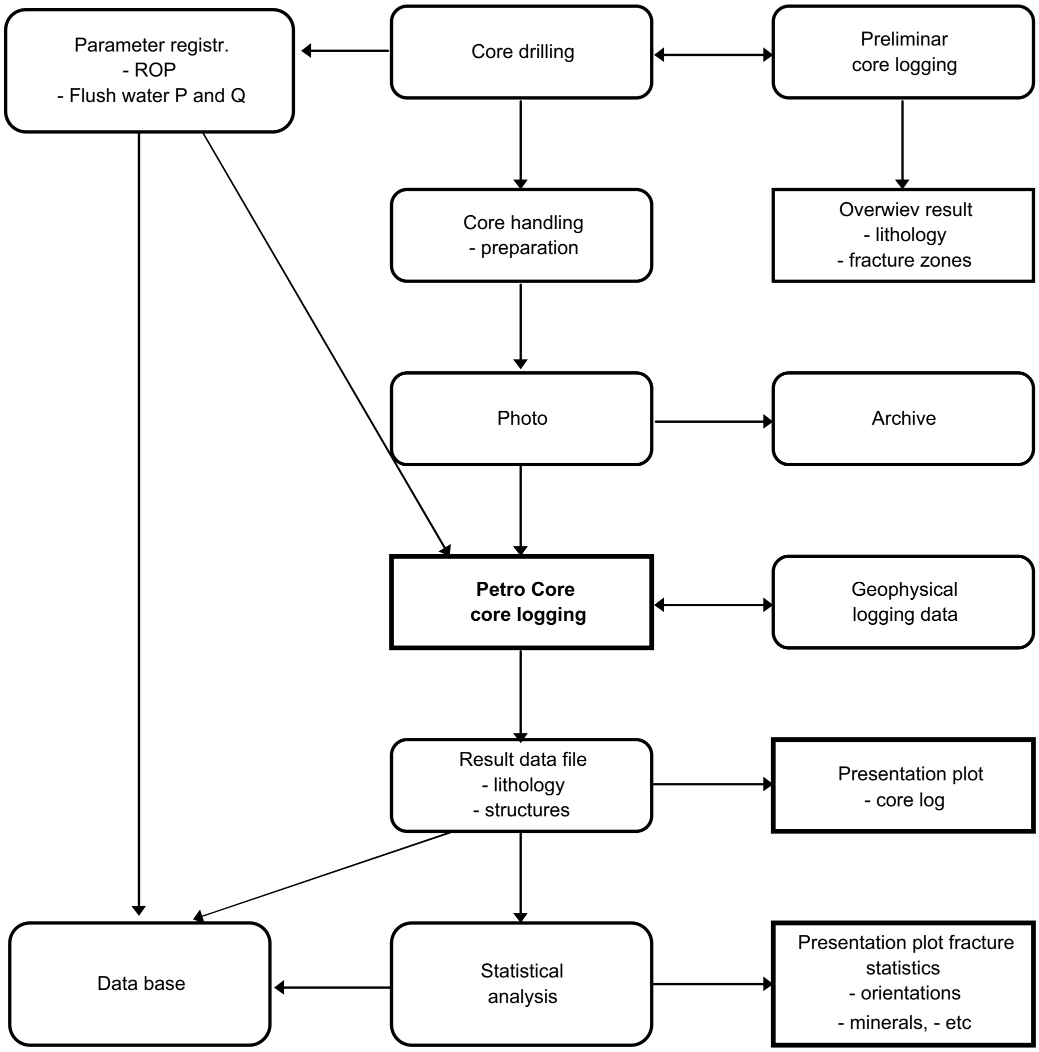


Figure 6-4. *The principal data flow for core logging.*

The drilling depth (length) data are given by the drillers for each core interval (at core uptake) and are theoretically very accurate (cm), as long as the correct procedures are followed. These data are therefore used as references for the length measurements along the boreholes. From time to time core stumps are left in the bottom of the borehole. These stumps can cause problems if not included in the length calculation. If the core is densely fractured and have core losses or crush zones in certain intervals, problems may arise in reassembling core pieces and hence in accurately measuring lengths between the bench marks.

The accuracy of angle measurements is in the order of ± 5–10°, but in most cases object heterogeneity reduces accuracy. Fractures as a whole tend to be more or less imperfect. The accuracy of core orientation is even worse, yielding errors in the range of ± 10–20°.

The accuracy of lithological and structural parameters is very much dependent on individual skill and experience. Therefore, only a few persons were involved in the core logging operations.

6.2.5 Comments and recommendations

In order to eliminate, or at least minimise, subjective estimates of rock types and other logging parameters, “calibration” of the different core loggers on special test core samples is recommended.

Core logging suffers from some problems with regard to length accuracy. It is not very reliable in sections with fracture zones, crush zones and core losses. At the same time, these sections are often the most interesting.

Another drawback is the tendency to overdramatise sections with small inter-fracture distances or crossing fracture sets. These sections are often represented by a pile of gravel in the core box, due to mechanical breaking by the drilling process.

Core logging is unsurpassed as a tool for sampling and characterising the rock. No other method can give as good samples, in fact it is the only way to get a real rock sample from inside a rock volume. However, for orientation of rock discontinuities (mainly fractures) core logging is in practice not very feasible, due to the problem of orienting the core itself, as was mentioned in Section 6.2.4. For that purpose an efficient borehole TVsystem (such as the BIP system, see Section 6.3.2) is recommended.

Some of the fractures in the core have been induced or re-opened by the drilling. Drilling-induced fractures can be recognised by their surface character, but re-opened fractures are more difficult to distinguish from natural open fractures. Fractures with minerals and/or surface alteration are almost impossible to categorise as sealed or natural fractures. When compared with tunnel mapping, the number of natural fractures from core logging seems to be overestimated. The fracture frequency from the BIP System is normally lower than from core logging, and probably agrees better with the actual situation in the rock /Labbas, 1997/.

TV-logging with the BIP system, for detection and geometrical characterisation of fractures and other discontinuities, and core logging, for characterisation of lithology and fracture surface character and minerals, seem to be an unbeatable combination for geological borehole documentation.

SKB has therefore further improved the characterisation methodology by developing the BOREMAP software, which can be regarded as a system for mapping rock core and borehole simultaneously. Mapping is performed by analysis of the BIP image on the screen and adding characteristics from the core. For efficient presentation of results a complot application is used, based on the WellCAD system.

6.3 Borehole TV logging

6.3.1 Purpose

The purpose of borehole TV logging was:

- To orient fractures and other structures in the borehole.
- To provide a geological overview in boreholes where cores were not taken.
- To inspect the results of pre-grouting in fracture zones.

TV-logging was also used to provide orientation data for core orientation. This was performed in three boreholes. For this purpose the so-called “SKB camera” was used, see Section 6.3.2.

Four different types have been used:

1. The SKB TV camera. A black and white camera with fibre-optic cable used for inspection of up to 1,000 m long boreholes. Limited use for orientating fractures.
2. The BIP system used for high resolution imaging of borehole walls. Gives orientation of structures in up to 1,500 m long boreholes.
3. The Olympus Videoscope colour CCD camera which was primarily used to investigate spreading of grout in fractures in up to 22 m long boreholes.
4. The Pearpoint system consists of a colour CCD camera. The system was primarily used for inspection of grouting in up to 150 m long boreholes. It was also used for general characterisation of fractures and rock types.

The Olympus Videoscope and the Pearpoint TV camera were used for special investigations, such as checking of grout spreading and documentation of fracturing in small scale investigations. These investigations were normally performed in comparatively short boreholes.

During the last stage of the construction phase, a new digital 360° TV logging system, the BIP system, was introduced and frequently used. This system orients the structures and makes high resolution, photo-like images.

6.3.2 Instruments and methodology

An ordinary TV logging system comprises a small TV camera, small enough to be lowered down into a borehole. The camera is connected to a video recorder via a borehole cable. From this standard configuration, improvements can be made to boost efficiency and quality.

The SKB TV camera

The SKB TV camera is a "black and white" system with forward-facing camera conforming to the PAL standard (in /Almén and Zellman, 1991/ it was called ABEM Borehole TV). The surface unit communicates with the camera via a 1,000 m fibre-optic cable, which is the same as for the BIP system.

The recorded image shows a view like looking down into a hole, and is therefore good for inspecting the borehole wall to check the possible risk of equipment getting stuck, or as an aid in "fishing" operations, when equipment is stuck.

The system was also used for orientation of fractures, but for that purpose it is far from ideal. The alpha and beta angles, i.e. the fracture angle (alpha) relative to the borehole (the core axis) and the rotation angle (beta) in the borehole, were then first determined. The alpha angle was estimated by means of distance markers mounted on a rod in the front of the camera. The longer the distance between the inner and outer extremes of a fracture, the smaller the alpha angle. Knowing this distance and the borehole diameter, it is possible to calculate the alpha angle. The beta angle was determined on the TV screen using a 360° measuring device. It is measured in reference to north with a small compass or to the plumb line with a gravitational ball or bubble. Based on the borehole orientation and the alpha and beta angles, the true fracture orientations were calculated using software from the PetroCore system, which was described in Section 6.2.

The BIP System

The BIP (Borehole Image Processing) System, developed by RaaX of Japan, is a digital system that generates a continuous image of a folded-out borehole wall. On the X axis the image covers 360° with one pixel per degree. The Y-axis is the borehole length and is as long as the borehole file. There are three logging-speed-dependent resolutions (1-0.5-0.25 mm/pixel).

SKB's version of the system, BIPS-1500, is converted to work with the RAMAC (the borehole radar) battery and fibre cable system, resulting in improved image quality for recording in deep (down to 1,500 m) boreholes, see Figure 6-5. The BIP system was not put into use until the very end of the ordinary construction phase, but has since then been used frequently.

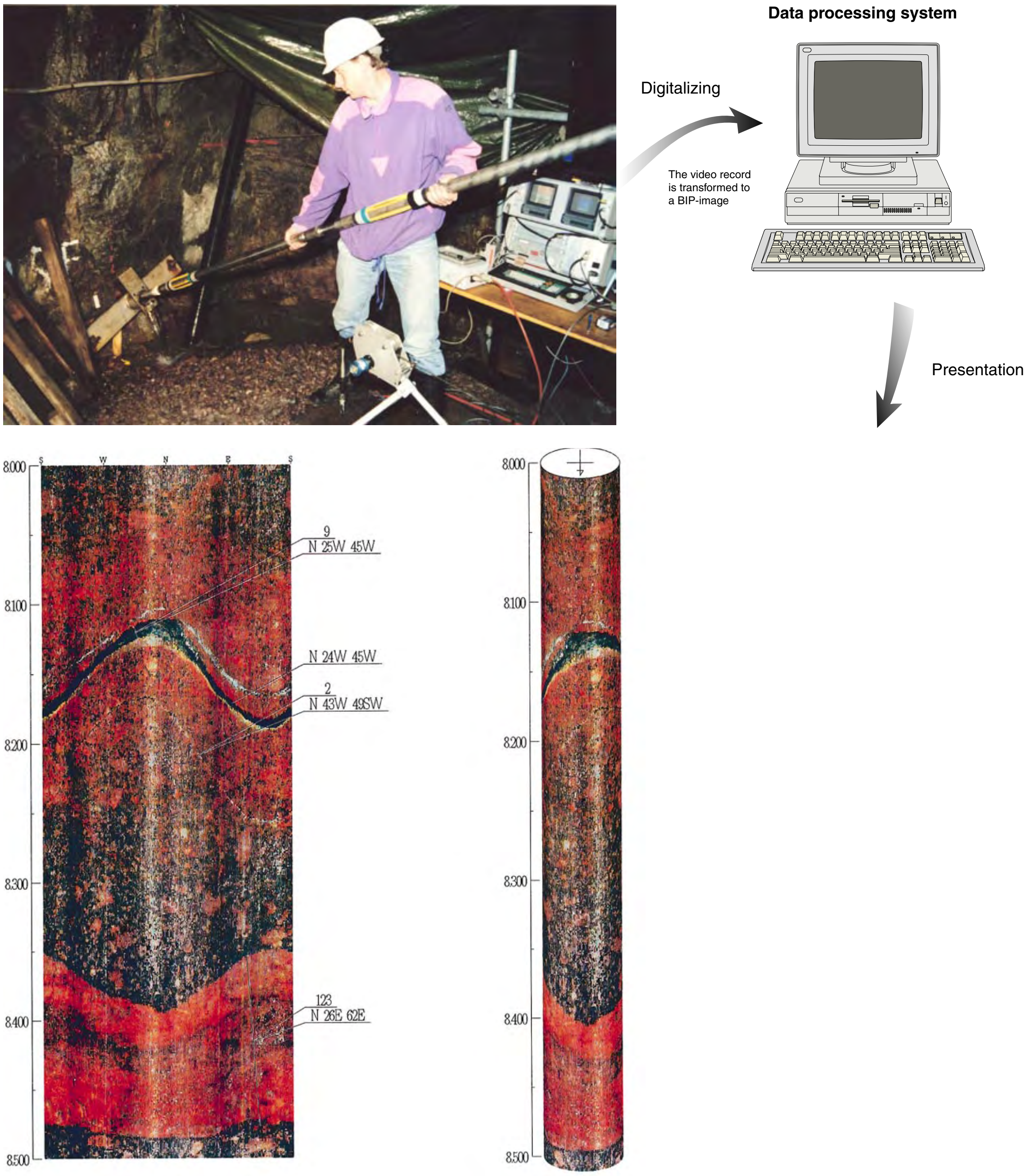


***Figure 6-5.** Illustration showing main principles of the BIP system and resulting images including a folded-out 360° version.*

The system uses a small video camera, which looks downhole into a conical mirror. The mirror reflects the borehole wall into the camera, creating a ring-shaped video image of the borehole wall. Within the ring-shaped image, a ring of 360 pixels indicates which data will be digitally recorded by the system. As the camera moves down along the borehole, everything on the borehole wall passes the pixel ring, which is captured (or scanned) every 1/50th of a second. The pixel ring is later cut open and laid out as a line, so that “north” or “up” is always in the centre. Information on what is “north” and “up” is obtained from a compass or a manually operated gravity ball orientation device (for inclined holes). As TV logging proceeds, the thin pixel lines are placed under each other to form the new “folded-out image”.

The capture of the pixel ring data is trigged by a pulse code wheel on the logging cable. A steady logging speed is of importance for good image quality. The length scale is displayed with the image on the video screen. The grabbing of images can be performed in three different resolutions, 1, 0.5 and 0.25 mm. One of the critical limitations is the frequency of the TV system. Due to this limitation the logging speed is also limited to: 1.5, 0.75 and 0.38 m/min for the three resolutions.

In the centre of the ring-shaped video image is an ordinary compass, used to supervise the direction of the borehole. An automatic electronic compass is used for data collection in vertical boreholes. For inclined boreholes there is a guided manually operated gravitational device for checking borehole probe rotation in reference to the plumb line.

The processed image is recorded on Magneto Optical Disks (MO disks), which can hold 128 MB or about 100 m of the lowest resolution each.

In analysis of fracture geometries with the BIP processing software, small markers are placed in the image on the trace of a fracture. The system then calculates the orientation in reference to the borehole direction or, if the borehole is vertical, in reference to north, as well as the true orientation (strike/dip or dip direction/dip), which is presented with the borehole colour image. The apparent width can also be measured in the image and presented on the image. A user-definable characterisation chart is used for characterisation of the objects. From four columns of characterisation categories (Sort, Form, Condition and Remark), with up to twelve choices in each, one category is chosen from each column. All objects can be processed by filtering and presented in stereo diagrams, giving a good illustration of their true orientation. A distribution diagram that shows orientation in three dip intervals (0–30°, 30–60° and 60–90°) provides a good overview of the object orientations along the borehole.

Olympus Videoscope

The Olympus Videoscope consists of a colour CCD (charged coupled device) camera together with optics and fibre-optic illumination system. The CCD produces a true-colour, real-time image with high resolution. The camera optics includes tip adapters with a field of view for 120 or 150° and a side view adapter. The diameter of the camera is 16.5 mm with centring devices of 30, 50 or 70 mm. Surface components of the system are a TV monitor, a Video recorder, a control unit and a light source. The system can be used for a borehole length of up to 22 m. The Videoscope was used mainly to investigate the spreading of grout in fractures, see example of a grout-filled fracture in Figure 6-6. The adapter used for this purpose was the front-looking adapter. The investigation with the Videoscope was performed in percussion-drilled boreholes. The cable was marked every metre. The depth was checked by a short pause every half metre and an audio recording of the position, was made using a microphone.

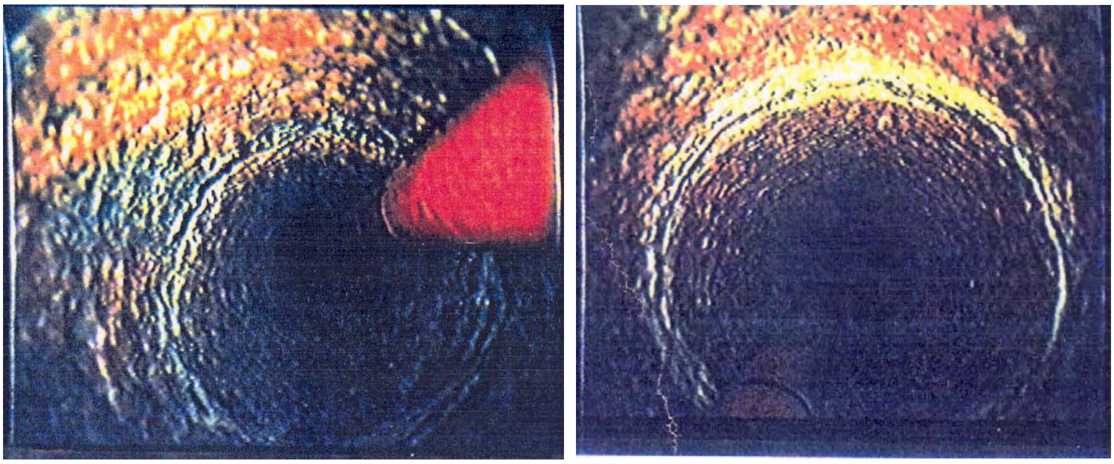


***Figure 6-6.** Fracture filled with grout inspected by Olympus Videoscope.*

Pearpoint system

The Pearpoint colour flexiprobe inspection system is available in the PAL standard and consists of a colour CCD camera. The camera incorporates remote focus and auto iris. The system includes a control unit with a TV monitor, a Video recorder, a rod counter, a light head and a 150 m cable rod. The diameter of the camera is 44 mm. The working length of the system is 150 m.

The Pearpoint system was used mainly to investigate the spreading of grout in fractures, but also for inspection of fractures and rock types using the front-looking camera. The investigation with the Pearpoint system was performed in percussion-drilled boreholes and cored boreholes up to a length of 150 m. The length was checked by a metric rod counter providing a resolution of 0.1 m. The length was displayed directly on the TV monitor. Gathering of data during inspection could also be performed by audio recording and a microphone.

6.3.3 Accuracy

As far as length accuracy is concerned, TV logging suffers from the same problems as other geophysical logging methods, i.e. cable tension. The cable for the BIP and the SKB TV gives a length error of approximately 0.5 m/100 m, for vertical and water-filled holes. However, all boreholes made during the construction phase were horizontal or sub-horizontal, and the probe was hoisted by means of rods. This greatly improved the length accuracy.

For orientation of fractures with the BIP system and the SKB TV system (whereby core logging was also performed), significant fractures or other objects seen both in the core and in the TV image were used for length correction. This was performed regularly with the BIP system, giving this system a theoretical length accuracy of 0.01 m, however, in good rock it is estimated to be 0.1 m.

The black and white “SKB camera” uses metre-marked cable to measure length. The operator speaks into a microphone and the length is recorded on the sound track of the videotape. This is not very accurate, but it serves its purpose.

For the Olympus Videoscope and the Pearpoint system, the accuracy of the length measurement is dependent on the marks on the stiff cable, and is estimated to be less than 1%.

Regarding fracture orientation, it is very difficult to quantify the accuracy of the SKB TV, so this will not be done. Quantification is somewhat easier for the BIP system, but it is still quite dependent on the skill of the operator. Accuracy mainly depends on accuracy in the determination of alpha and beta angles, but may also be affected by errors in borehole orientation and deviation data. The errors in strike and dip are estimated to be within 20%. The error in the determination of aperture width cannot be determined, although the width of a fracture varies as much as the fracture's own maximum width.

6.3.4 Comments and recommendations

The use of the front-looking SKB TV camera is not recommended where the water is dirty or contaminated with colloids. The fact that it is a black and white system renders mineral identification impossible. However, after the introduction of the BIP system it is regarded as a complement, to be used for special purposes, such as inspecting the openness and wall stability of a borehole, as it is superior in perceiving relief and giving a 3D impression.

The BIP System is less sensitive to dirty water due to a relatively small water gap between the probe and the borehole wall.

BIP is a unique complement to ordinary core mapping when it comes to geometry (length, width and orientation), fracture zone description and the ability to visualise the borehole, as also was discussed in Section 6.2.5.

The advantages of the BIP system are first of all the excellent images and the orientation possibilities. Another advantage is the ability to study individual fractures within a fracture zone, which normally end up as a pile of gravel in the core box. Since only the sinus-shaped trace of a fracture is seen, its disadvantage is a limited ability to characterise the fractures by mineral, form, roughness and alteration. However, the aim is not to take the place of but to complement core logging.

Borehole TV is a good tool for grout inspection. Coloured grout was easy to detect by Pearpoint investigations.

6.4 Laboratory analyses of rock samples

6.4.1 Purpose

The main purpose of the laboratory analyses of rock samples from the tunnel was to determine:

- The main rock types and their mineralogical composition.
- The fracture fillings which were not possible to identify by ocular inspection.
- The mechanical properties of the main rock types.

6.4.2 Methodology

Geological analysis of rock samples

Rock samples from the tunnel wall or small drill cores from just inside the tunnel wall were taken according to descriptions in Section 4.4. The samples were analysed regarding density and porosity and subsequently investigated by means of microscopic modal analyses in order to determine the main rock types and their composition.

Density

Density is closely related to the mineral composition of the rock sample. The influence of porosity in crystalline rock is less than 1%. The amount of mafic minerals and SiO_2 content determines the density of the samples. The three different rock types which this investigation was focussed on, i.e. fine-grained granite, Småland granite and Äspö diorite, have densities in the range 2,650–2,750 kg/m^3. The density of the rock samples was calculated as the ratio of the water-saturated weight of the rock sample to the total volume of the rock material and the pores. The bulk volume was determined according to the Archimedes principle as the difference between the saturated surface-dry weight and the weight of the sample submerged in a water tank, divided by the density of water. The weights are determined within an accuracy of ± 0.1 g, making the determined density of the rock accurate to within ± 8 kg/m^3.

Porosity

Porosity (water-accessible porosity) determined by laboratory measurements is the sum of the kinematic porosity and the diffusion porosity. Porosity is determined as the ratio of the volume of interconnected pores and the total bulk volume of the sample. The pore volume is determined by the difference between the water-saturated surface-dry weight and the oven-dry weight, divided by the bulk volume. The oven-dry weight of a sample is determined after it is dried in an oven at a temperature of 105°C for a period of 48 hours. The water-saturated weight is determined after it is placed in a vacuum chamber at 25 mm Hg for at least 3 hours. It is then soaked in water for 48 hours. The measured porosity is defined as a percentage of the total volume and can be determined to within an accuracy of about 0.005% under optimal conditions.

Modal analyses

The composition of a rock expressed in terms of the relative amounts of minerals actually present is called mode. We refer to a procedure, which yields such a statement, and usually to the statement itself, as a modal analysis. The analyses are then used to classify the rocks.

The classification of rocks using modal analyses is based on a system devised by /Streckeisen, 1967/ and subsequently improved by /IUGS, 1973, 1980/. The principle states that the classification should be based on the modal mineralogy of the rocks. According to a long-established practice among petrologists, feldspars, quartz and feldspathoids are the principal minerals used for the classification, which is displayed on a double three-component triangular diagram, see Figure 6-7. Normally only the upper triangular diagram, with quartz and feldspar, is used because most rocks contain no feldspathoids. The diagrams are divided into fields where each field represents a certain rock. When the volume percentages of the principal minerals are known they are recalculated to 100% and then plotted on the diagram.

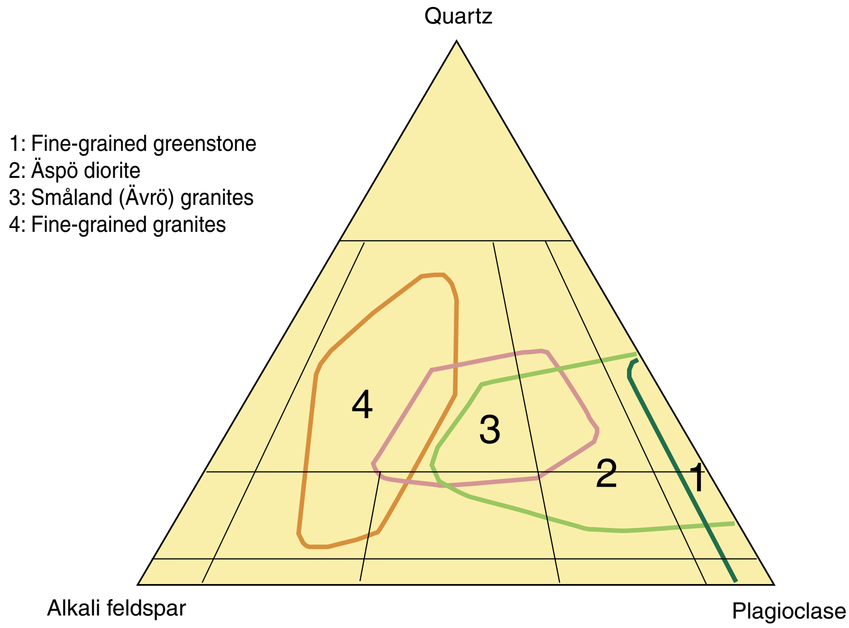


Figure 6-7. *Modal classification according to /IUGS, 1973, 1980/, of four rock groups from the Äspö area.*

To obtain a quantitative determination of the minerals in a rock, a thin section is analysed under the microscope using a point-counter device. The point-counter enables the thin section to be moved incrementally in two directions perpendicular to each other. This means that an area of the thin section is covered by a grid with evenly distributed points. At each point the specific mineral is determined and registered by an automatic counter. The counter can also display the volume percentages of the different minerals. To ensure statistical significance of the analysis, a sufficient number of points must be counted. The method is a simple, quick and cheap procedure to determine the composition of many rocks.

Mineralogical analysis of fracture fillings

A number of fracture fillings were analysed for the purpose of determining their mineral composition and whether they were of swelling character.

X-ray diffraction techniques

The investigation was performed by means of standard X-ray diffraction (XRD) techniques. Approximately 50 g of the bulk sample was dispersed in distilled water by means of ultrasonic baths and agitation. The fraction smaller than 10 µm was separated and filtered by means of a vacuum filtering technique. The applied vacuum forced the suspension through a membrane filter with a pore size of 0.47 µm. The clay particles became oriented with their basal surfaces parallel to the filter surface. The clay cake thus produced was mounted on a glass slide and air dried. The prepared samples were investigated over the 3–40° 2θ range with an angle speed of 1°/min on a Philips PW 1710 diffractometer and CuKα-radiation. Ethylene-glycol-treated mounts were analysed to obtain supplementary information on swelling vs. non-swelling clay minerals. A relative estimation of the quantities was performed by comparison with computer-generated X-ray diffractograms of known mixtures and compositions.

Coarse-grained and crystalline samples were gently ground and the powder packed into a sample holder and then investigated by XRD of the 3–60° 2θ range.

Testing of rock mechanical properties

In order to evaluate the rock mechanical predictions established for the Äspö HRL tunnel, field and laboratory tests have been performed. The tests included field studies for making quantitative descriptions of discontinuities in the tunnel as well as laboratory testing of fracture properties and elastic properties. In the field studies, the joint surface roughness was described by linear profiling, compass and disc-inclinometer. Compass and disc-inclinometer were used to determine geometric variations. Four discs of different sizes were used. Linear profiling in two directions (horizontal and perpendicular to that) gave waviness. Estimates of roughness profile and joint roughness coefficient (JRC) were made by comparing graphical references in /Brown, 1981/.

In the laboratory, shear tests were performed on existing joint surfaces to determine their fracture properties. The elastic parameters (uniaxial compressive strength, Poisson's ratio, elastic modulus and brittleness) were determined by uniaxial compressive tests. With reference to Section 4.5, the laboratory tests were carried out on core samples taken from short (approximately 1 m deep) boreholes drilled from the tunnel wall. Joint surface strength was determined by Schmidt hammer tests on rock samples. All testing and preparation of specimens have been carried out in accordance with ISRM's recommendations /Brown, 1981/.

Shear tests

Shear tests were performed by shearing of existing joints in core pieces, in accordance with ISRM's recommendations. Each core half enclosing a joint was grouted into a steel cylinder. Expanding grout was used to keep the specimens fixed in the cylinders. All the test core pieces were still in the steel ring after the test procedure was finished.

The test procedure for shear testing was:

- The steel rings were placed in the shear machine.
- The normal load on the specimen was raised and reduced in 10–20 steps, to a maximum load of 15 MPa. The normal displacements were measured during the test.
- The specimen was sheared 3 mm four times at different normal stresses from approximately 1.5 MPa to 6.0 MPa, starting from the lowest and increased for each test. The shear forces and the shear and normal displacements were measured.

Uniaxial compressive tests

Uniaxial compressive tests were carried out with an MTS press (Mechanical Testing System) of very high stiffness. The high stiffness is necessary for registration of deformation at failure, which determines the brittleness ratio for the sample. Uniaxial compressive strength, Young's modulus, Poisson's ratio and brittleness type were determined for each sample. The tests were carried out as follows:

- The samples were prepared according to ISRM's recommendations regarding specimen dimensions and tolerances.
- Strains were measured with foil strain gauges and were sampled and saved, together with applied forces, in computer files. Load deformation curves were plotted during the test to determine the brittleness ratio.
- Young's modulus was determined as secant values for 0.2% strain and Poisson's ratio for 0.1% strain.

6.4.3 Accuracy

The weights are determined within an accuracy of ± 0.1 g, which means that the accuracy of the determined density of the rock is within 8 kg/m^3. The measured porosity is defined as a percentage of the total volume and can be determined to within an accuracy of about 0.005% under optimal conditions. The results of the modal analyses of rocks are estimated to be accurate to within ± 5% for the major minerals.

The accuracy of the X-ray diffraction analyses are semiquantitative and in the range ± 5–10%. Qualitatively, the analyses have a high accuracy for identification of minerals that are present in amounts exceeding ± 1–5%.

The uniaxial compressive test results are estimated to be accurate to within ± 3–5 %.

6.4.4 Comments and recommendations

The results deviate significantly from the predictions based on pre-investigation data, especially as regards the JRC and the JCS. In fact, it was the use of /Brown, 1981/ which caused the problem. There are now appropriate methods for applying the JRC and JCR tests. /Bandis et al. 1981/ showed how the long-suspected scale effects could be handled.

7 Geophysical borehole investigations

7.1 General

Geophysical investigations are indirect characterisation methods for geological and hydrogeological properties. Some methods measure different geophysical parameters while other methods measure the response to an induced geophysical disturbance. Most "passive" methods are single-hole methods while "active" methods are both single-hole and crosshole methods.

The geophysical methods used during the tunnel construction phase is described and discussed in this chapter. The general purpose of geophysical investigations was:

- To submit physical parameter data of the rock as input for the geological and hydrogeological evaluation and modelling.

With reference to Section 2.2 and Figure 1-5, all geophysical methods were related to the special investigation block and will in this chapter be presented as follows, see also Figure 6-1:

- Geophysical borehole logging.
- Radar investigations.
- Seismic investigations.

Sometimes borehole TV is regarded as a geophysical method, but in this report the borehole TV method is discussed among geological methods (see Chapter 6).

7.2 Geophysical borehole logging

Geophysical logging include a group of single-hole methods measuring physical properties of the borehole wall and of the rock volume close to the borehole, see Figure 7-1. There are a great number of geophysical logging methods available, some of which are sensitive to electrical or magnetic properties of the rock and/or the pore water in the rock formation. Other rock properties are measured by means of radiometric or acoustic methods.

Use of geophysical logging in the underground investigations performed at the Äspö HRL was very limited, e.g. only five long cored boreholes were measured /Rhén et al. 1995/ and /Nilsson P, 1995/. In addition a set of short cored holes and percussion-drilled holes were logged during blasting damage investigations in 1991 /Olsson, 1991/.

7.2.1 Purpose

The purpose of geophysical logging is to provide information on geophysical parameters of the rock. The results are used as a complement to geological and hydrogeological data for the characterisation of rock types and fracture elements (fractures and fracture zones). The aims of and methods used in the construction-phase underground investigations at Äspö were as follows:

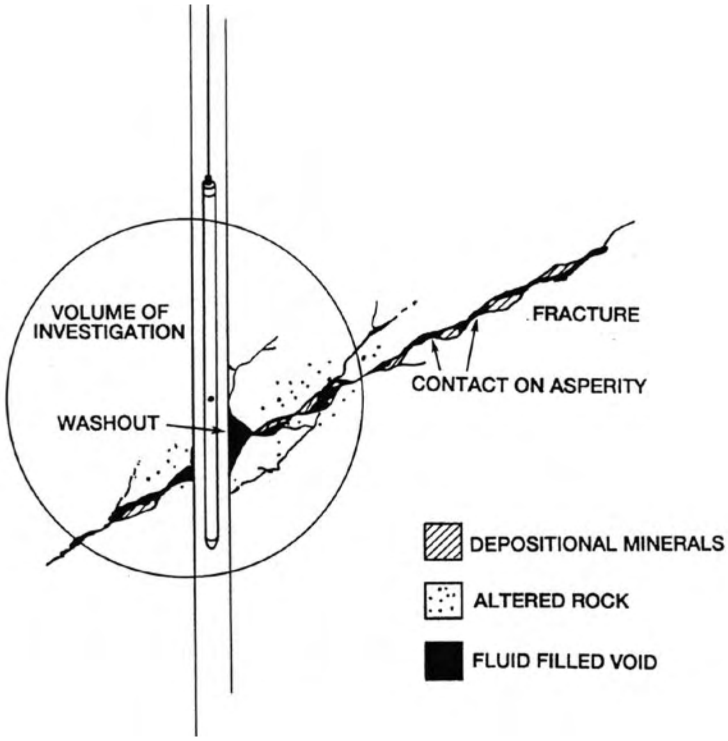


Figure 7-1. *Schematic example of natural fractures intersecting a borehole and the geophysical volume of investigation /National Research Council, 1996/.*

Information mainly on lithology (rock types, etc):

- gamma (natural gamma) method,
- density (gamma-gamma) method,
- susceptibility method.

Information on fractures and fracture zones:

- resistivity methods,
- velocity (sonic) method.

The presentation in this chapter will focus on these methods. Methods that were commonly used during the investigations are Caliper and porosity (neutron) methods.

7.2.2 Equipment for geophysical logging

The versatile, lightweight WELLMAC Logging system, manufactured by MALÅ GeoScience, was used for the underground logging. The system includes a surface unit with software, cable, winch and measuring wheel, a controller probe and a measuring probe suite, see Figure 7-2.

Surface unit, including software

The surface unit contains a PC for probe control and data acquisition. The software ensures convenient and efficient calibration, data acquisition, processing and presentation.

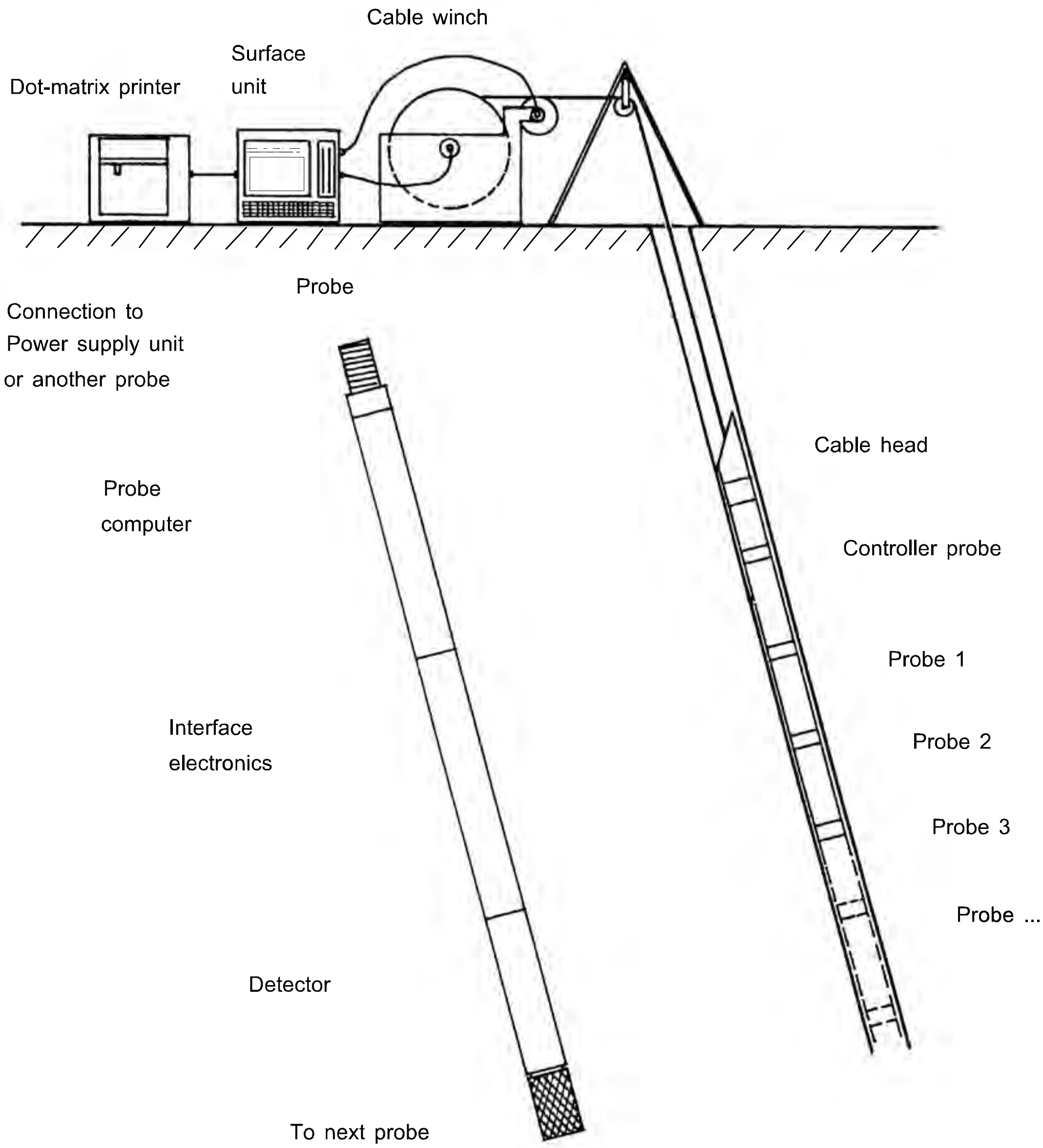


Figure 7-2. *The WELLMAC logging system.*

ASCII files (text tables) are used throughout the program for both collection and processing. All data can be accessed via a text editor for editing. As measurement proceeds, the data can be displayed on the screen as numerical values or as continuously updated log curves. The log curves can also be plotted in real time.

Data processing includes correction of depths to compensate for cable stretch (if known) and measurement errors introduced by the measuring wheel. Corrections are also made for casing height and the distances between the points on the probes at which measurement takes place, as well as for the point to which cable length is referred.

Data processing also includes calibration against calibration tables and adjustment for drift, if necessary and known (temperature drift in the electronics for example). Statistical noise filtering is also included whenever necessary.

Processing is executed by combining a number of files column by column for composite plotting. Plotting can be carried out linearly or logarithmically, to any desired scale and using any desired grid and pen thickness/line type. Generally speaking, plots can be formatted in accordance with the API standards. The header used for the plot comprises a filled-in form that can be supplemented as desired before plotting starts.

Winch, cable and measurement wheel

Depending on borehole depth, location and access, different cables and winches can be used. The cables contain 4 conductors for probe operation. Steel-reinforced cables or lightweight urethane-jacket cables are available. Manually or electrically powered winches are used, the latter with length capacities of up to 1,500 m. When logging in horizontal boreholes, the probes have to be pushed into the borehole. Glass fibre or aluminium rods are used for this purpose.

The measuring wheel is equipped with a mechanical odometer with a digital readout that is mounted adjacent to the wheel. An optical encoder with electrical pulse output is mounted on the opposite side. The resolution is 0.1 m for the mechanical readout and 0.01 m for the optical encoder. The measuring wheel is mounted on either a tripod or the borehole casing.

Geophysical probes

The borehole probe assembly used for geophysical logging is composed of a controller probe and a measuring probe suite (one or several measuring probes).

The controller probe is a common top probe for each WELLMAC probe suit. The controller probe converts the 200 V DC received from the surface power supply into the voltages needed by the different measuring probes. It also handles two-way modem communication between surface unit and probes.

Since each measuring probe in the WELLMAC system is a stand-alone unit, probes can be combined in virtually any combination. Each probe has its own onboard microprocessor that stores raw data and sends it to the surface. Measurements with the different geophysical probes will be described in the following sections.

The stainless steel probes used in the WELLMAC Logging System are joined by flush threads. The maximum cable length for operating the probes is 1,500 m and the maximum water pressure is 150 bars. A built-in alarm in each probe indicates any water leakage that may occur.

7.2.3 Geophysical logging methods

Resistivity methods

The resistivity probe can be combined with different sensor packages depending on configuration and operation. With the resistivity sensor, five resistivity parameters are measured sequentially: Short Normal 0.4 m (16"), Long Normal 1.6 m (64"), Lateral 1.6–0.1 m (64"–4"), Single Point Resistance and Self Potential (SP is not used at Äspö), see Figure 7-3 (Temperature is optional for this sensor.). Other sensors (not used at Äspö) are fluid resisitvity/temperature sensor and induced polarisation time domain sensor (Fluid temperature and resistivity were measured with the UCM flow meter probe, see Section 8.7).

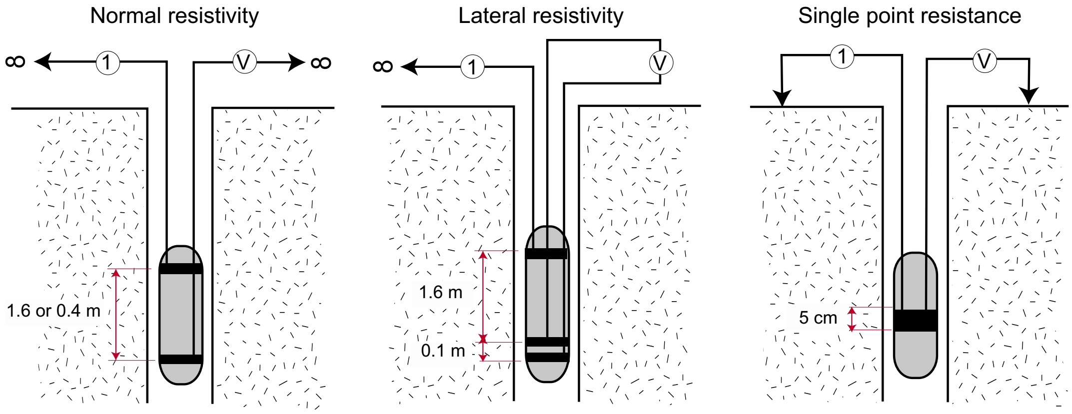


Figure 7-3. *Electrode configuration for resistivity logging.*

The electronic section of the resistivity probe is the same for all three types of sensors. The resistivity probe must be connected as the lowest probe in the probe suite.

When used for single point resistance, the probe measures the contact resistance between the probe and the borehole wall, thus permitting conducting minerals and water-bearing fractures to be detected.

When used for normal/lateral resistivity, the probe measures the apparent resistivity in the bedrock. The normal resistivity log will indicate zones with anomalous resistivity, such as fractures containing water and conducting minerals. Here, it is possible to make quantitative estimates of rock quality in general and also the width and resistivity of fractures. In many types of rock it is possible to find a direct relationship between resistivity and porosity. Lateral resistivity is used together with normal resistivity to indicate the presence and character of fracture zones. As a rule, lateral resistivity provides detailed information about the boundary between conducting and resistive bedrock.

The resistivity probe is calibrated by connecting a variable resistor to the system, thus calibrating the response to a known resistance.

The measurement range for apparent resistivity is from 1 to 100,000 Ohm, for self potential –4,000 mV to +4,000 mV, for temperature 0 to 70°C and for fluid resistivity 1–500 Ohm.

Gamma (natural gamma) method

The gamma probe measures the natural gamma radiation in the borehole. A 1" diameter x 1.5" long NaI crystal is used as the detector. The total natural gamma radiation from potassium, uranium and thorium is measured. Variations in the concentrations of these elements normally correspond to lithological changes in the rock. Hence, the natural gamma method is used to detect boundaries between different types of rocks.

Calibration is done in a calibration jig, using a small radioactive source to simulate the radioactive equivalent to a known deflection.

The measurement range is from 1 to 100,000 cps. After calibration and correction for borehole size the results can be presented in microRoentgen/hour (µR/h).

Density (gamma-gamma) method

This method is used to determine the density of rock by emitting gamma rays from a radioactive source in the probe and then measuring the radiation energy arriving at the detector in the same probe. The gamma ray absorption is different for different minerals, high-density minerals generally absorb more than low-density minerals. Changes in lithology and the presence of large fracture zones can therefore be indicated by the method. The radioactive source in the density probe is Caesium-137 and the detector is a 1" diameter x 1.5" long NaI crystal (the same as in a natural gamma probe). Under certain conditions the density method can also be used to determine variations in rock porosity.

The probe is calibrated in two boreholes (56 and 76 mm diameters) with known density (determined from density measurements on the core from these holes). When logging is done in boreholes with larger diameters, a correction algorithm has to be integrated in the calibration program.

The measurement range is from 1 to 10 g/cm^3.

Susceptibility method

This method is used to determine the magnetic susceptibility (magnetisability) of rock. Since each type of rock has a characteristic content of ferromagnetic minerals (mostly magnetite), a susceptibility log provides information about lithological changes in the rock. The method can also be used for the location of fracture zones, due to oxidation of magnetite to haematite in these zones.

The measuring principle of the susceptibility probe is that a solenoid creates a magnetic field, which also will be influenced by the magnetic minerals in the rock. These variations in the magnetic field are measured as changes in the current in the solenoid.

Measuring of different pads with known magnetic susceptibility performs the calibration of the instrument. As the instrument indication is strongly influenced by the diameter of the borehole, calibration has to include this factor.

The measurement range is from 1×10 E-5 to 2 SI-units.

Sonic Velocity method

The Sonic Velocity Log records the time compression sound wave takes to travel one foot through the formation. This time, called Delta T (DT), is dependent on the elastic properties of the formation and is used as an indication of fracturing. When the lithology is known, DT measurement can be used to estimate porosity. When run together with other porosity-sensitive tools, it can provide useful crossplot information to determine unknown lithology and porosity.

DT measurement is also used to compute integrated transit time (ITT), useful for interpretation of seismic measurements.

The WELLMAC surface unit and software did however, not operate the Sonic Velocity probe used at Äspö. The probe uses an acoustic transmitter of magnetostrictive type to generate the compressional sound wave. One portion of this sound wave travels along the borehole wall to a pair of receivers spaced 1 foot from each other. The distance from the transmitter to the near receiver is 3 feet. The time difference (DT) for the sound wave to travel to the two receivers is computed and expressed in μs/ft. The probe contains acoustic insulation material between the source and receiver, which attenuates the direct sound wave through the probe.

No field calibration is needed for DT measurement.

7.2.4 Performance

Depending on number and type of geophysical parameters to be measured the, measurement campaign is conducted as one or more borehole loggings. A logging tool is assembled, consisting of the controller probe and a measuring probe suite. The surface unit and the winch with cable are placed near the borehole mouth. The measuring wheel is mounted on the borehole top casing or on a tripod.

For logging horizontal or sub-horizontal boreholes, the tools must be pushed into the borehole. Connecting a pushing rod to the logging tool does this. The rod is then pushed into the borehole and readings are taken continuously every 0.1 m, triggered by the measuring wheel. After 2 m a new rod is connected to the first one. The measurements continue and new pushing rods are connected until logging along the entire borehole is completed. If more parameters are to be measured, a new logging tool is assembled containing other measuring probes, and the borehole logging procedure is repeated.

Heavy water outflow from the hole makes it difficult to push the rods into the borehole. The degree of difficulty is dependent on which tool is used, but as a rule of thumb, outflows greater than 200 litres/min are difficult to handle.

Depth (or length) measurements along the borehole are made with the measuring wheel. In addition, the logging cable is marked every 50 m, which is used for adjustment of the measurement wheel.

7.2.5 Data processing and presentation

The measurements with all the geophysical logging tools are performed from the surface data and control unit with the aide of the measurement software AQUIRE. Raw data are recorded as volts, amperes, counts, etc.

Raw data are converted into real units such as ohmm, μR/h, etc via calibration constants. For some of the methods the data processing also includes corrections for temperature, depth adjustment, borehole diameter, fluid resistivity, etc.

The refined datafiles from the geophysical loggings are plotted in separate graphs presented side by side in so called composite logs, see Figure 7-4. These datafiles are also stored in the SKB database SICADA. The data flow chart is shown in Figure 7-5.

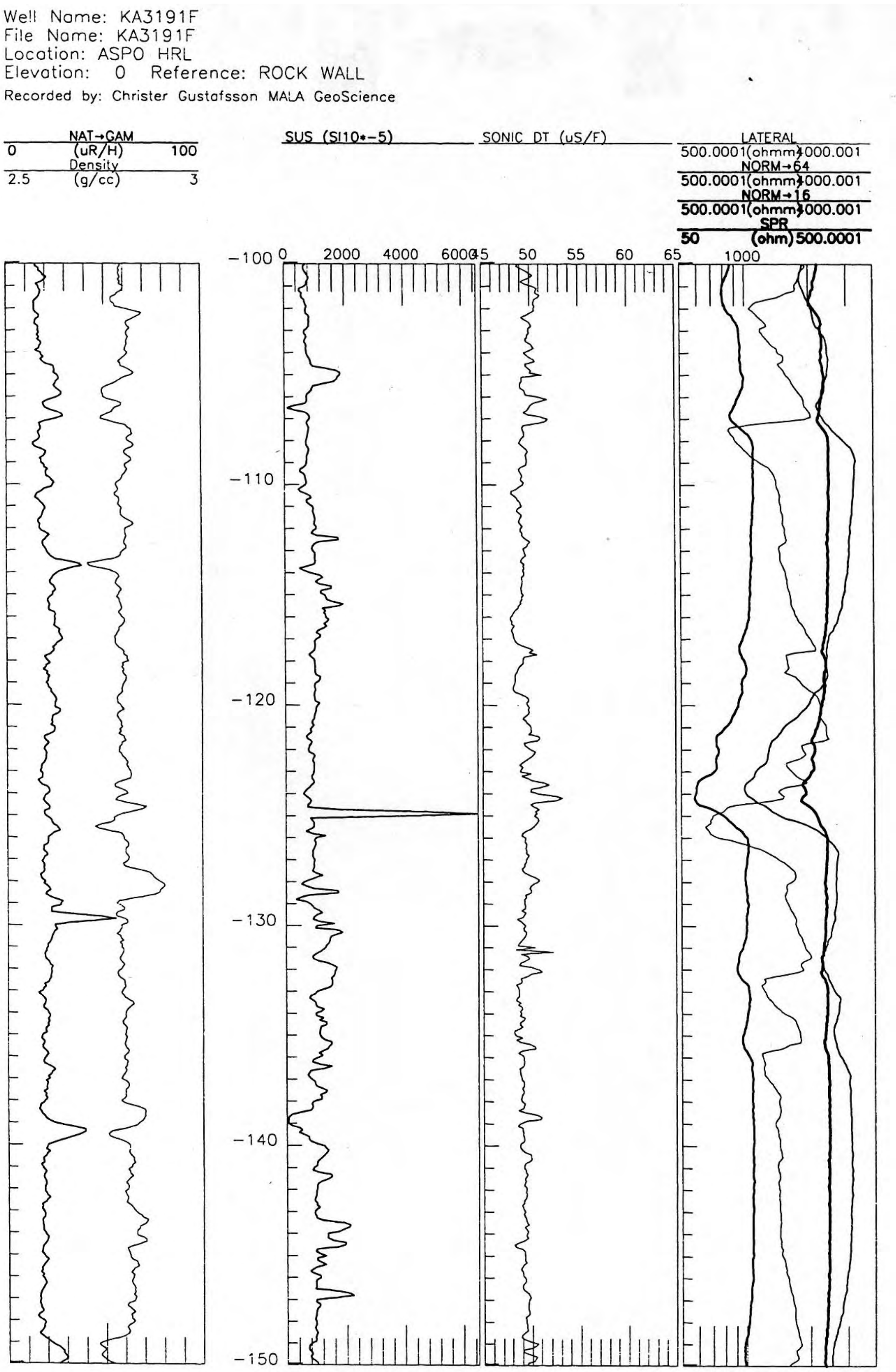


Figure 7-4. Composite log for the geophysical loggings performed in borehole KA3191F.

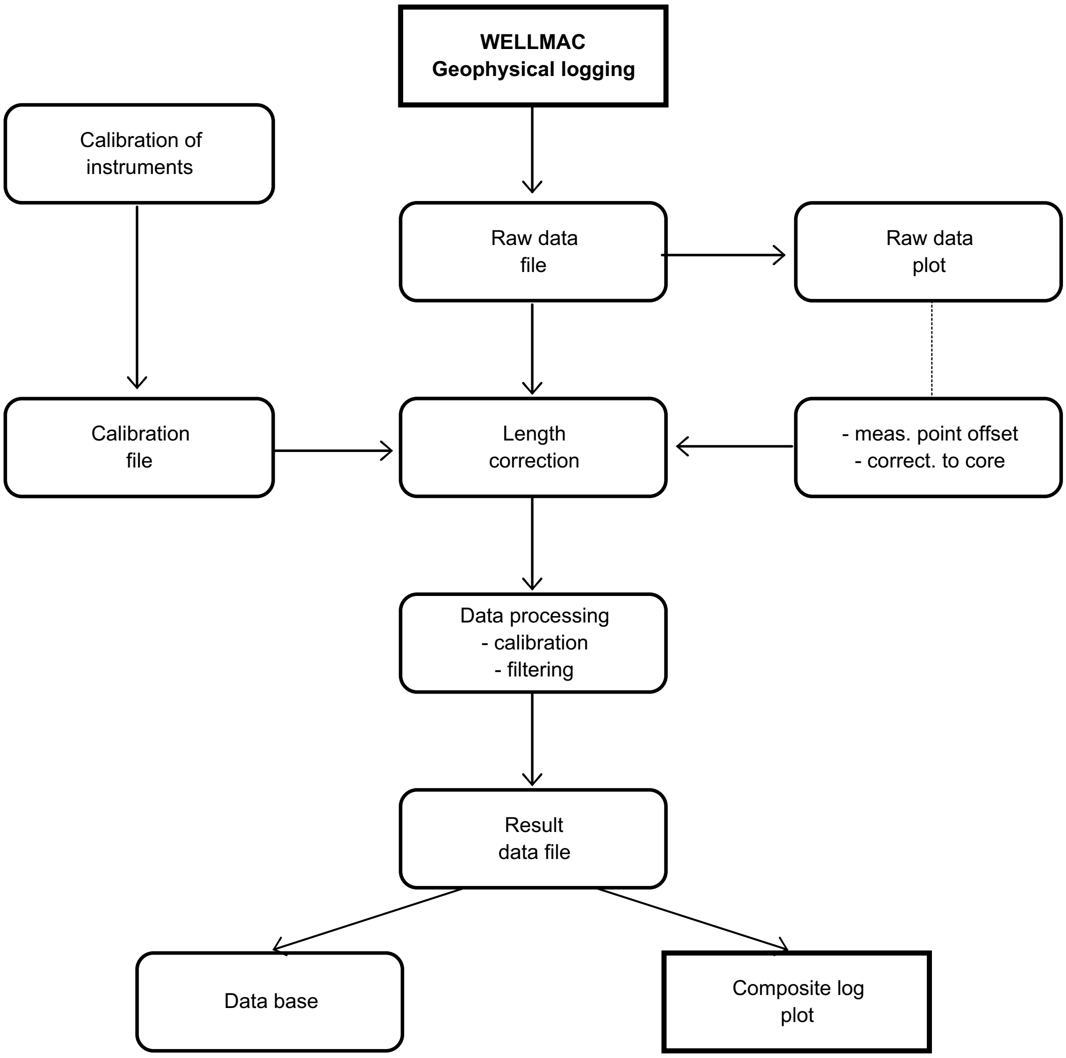


***Figure** 7-5. Data flow chart for geophysical logging.*

7.2.6 Accuracy

The resolutions and accuracy for the different geophysical logging methods are shown in Table 7-1.

Table 7-1. Measuring range, resolution and accuracy of the geophysical logging methods used.

Method	Unit	Range	Resolution	Accuracy
Resistivity				
- normal	Ohmm	1–100,000	3% of reading	± 5% of reading
- lateral	Ohmm	1–100,000	3% of reading	± 5% of reading
- fluid	Ohmm	1–500	3% of reading	± 5% of reading
Single point resistance	Ohm	1–100,000	3% of reading	± 5% of reading
Susceptibility	SI	$1\text{x}10^{-5}$–20	1% of reading	± 4% of reading
Sonic	μs/ft	10–300	1% of reading	± 4% of reading
Gamma	cps	1–100,000	1% of reading	No calibration
Density	g/cm^3	1–10	2% of reading	± 4% of reading

The accuracy of the depth measurements is dependent on the tension of the logging cable. The tension of the steel cable is less then the tension of the polyurethane cable. After depth correction, accuracy can be estimated to be better than ± 1%. First the depth is adjusted in reference to length marks on the cable. If geological information from a core is available, i.e. rock boundaries, the length is adjusted in reference to these known marks.

7.2.7 Comments on geophysical logging

Cross plots of density versus susceptibility gave useful information permitting discrimination between Småland Granite and Äspö Diorite. Different types of fine-grained granite could be distinguished on the basis of natural gamma radiation.

The single point resistance tool and the sonic tool identified single fractures.

In horizontal boreholes the probes were pushed into the borehole by rods. If there was water inflow, this could make it difficult to push the rods into the borehole. The upper limit of inflow was about 200 l/min.

For practical reasons, logging could not be performed close to the drift front during periods of excavation. Logging was performed in boreholes located in niches.

7.3 Radar investigations

The following radar methods were used underground during the construction phase of the Äspö HRL tunnel:

- Radar measurements in boreholes;
  - with RAMAC system.
- Radar measurements from tunnel;
  - with RAMAC system and tunnel/surface antennas,
  - with RAMAC/GPR system.

7.3.1 General description and purpose

The main purpose of the borehole radar was:

- To locate and orient structures in the rock mass, such as fracture zones.

Most radar measurements were performed in cored boreholes drilled as investigation holes for detailed characterisation of rock volumes far away from the tunnel. A total of 18 holes were radar-investigated, all measurements using the directional antenna.

Two campaigns of tunnel radar investigations were conducted. The aims of the first and the second tunnel radar measurements were:

- To evaluate the applicability of radar in a tunnel for locating fracture zones ahead of the tunnel face /Olsson, 1992/.
- To locate and orient minor water-bearing fractures and other discontinuities and rock structures in the vicinity of the TBM and D&B tunnels /Stenberg and Forslund, 1996/. The study was conducted within the ZEDEX experiment. The second measurement was performed using the RAMAC/GPR (Ground Penetrating Radar).

The radar technique is based on propagation of radar waves through the rock. Frequencies used for geological applications normally fall in the range 10 to 1,000 MHz. Radar wave propagation is sensitive to the electrical properties of the rock, mainly its dielectric permittivity and electrical conductivity. The variation of these properties is related to other physical properties of the rock, which are of more direct interest to groundwater flow paths, such as total porosity and fracturing.

7.3.2 The borehole radar system RAMAC

Instruments

In principle a radar system consists of a transmitter, a receiver, a signal control unit, a data collection system, and a display unit. For a short pulse system such as the RAMAC system, the received signal may be analyzed with respect to the propagation time and amplitude of the first arrival (crosshole transmission mode) or the propagation time of later events normally caused by reflection from inhomogeneities in the rock (reflection mode). A detailed description of the RAMAC system and radar theory can be found in /Olsson et al. 1987/, /Falk et al. 1989/ and /Sandberg et al. 1991/. The RAMAC system is shown in Figure 7-6.

The RAMAC system works in principle as follows: The dipole-type transmitter antenna generates a radar pulse that propagates through the rock. The pulse is made as short as possible to obtain high resolution. The reflected and attenuated pulse is received by the receiver antenna and amplified and registered as a function of time. From the full wave record of the signal the distance (travel time) to a reflector, the strength of the reflection, and the attenuation and delay of the direct wave between transmitter and receiver can be deduced. Measurements are made, for example every 0.5 m along a borehole, and the result is displayed in the form of a radar diagram, see Figure 7-7.

In directional radar measurements the receiving antenna consists of an array of four loop antennas. Using the information from the four individual antenna loops together with orientation information on the positions of the individual antennas, the field of an arbitrarily

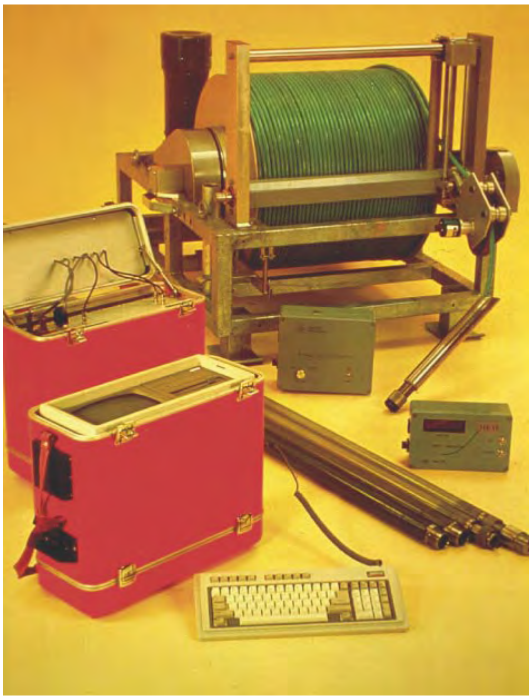


Figure 7-6. *The borehole radar system RAMAC, consisting of computer unit, control unit, transmitter and reciever probes with batteries, and cable winch with a measuring wheel.*

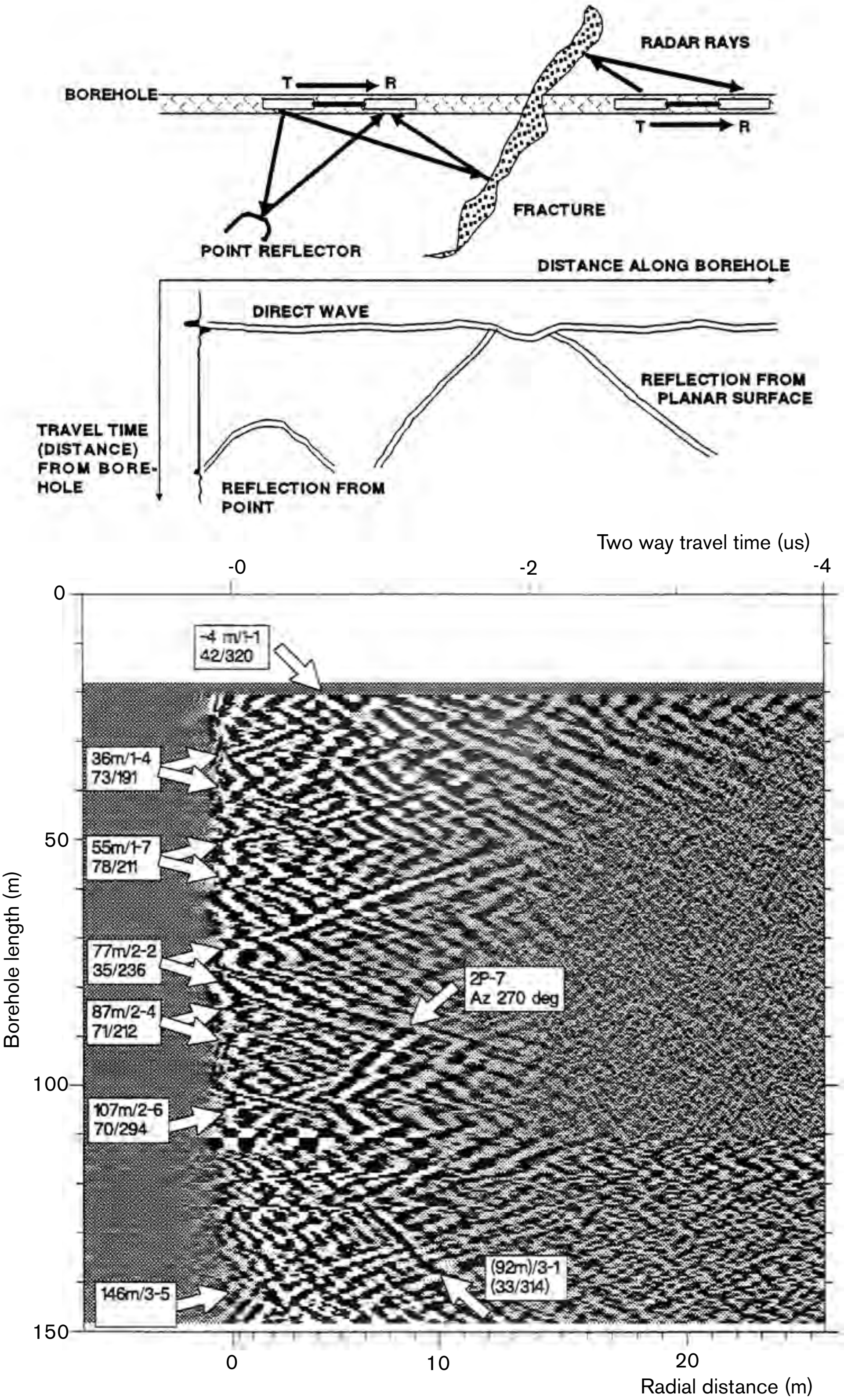


Figure 7-7. *Above: The principle of the borehole reflection radar and the characteristic patterns generated by plane and point reflectors. Below: An example of a radar diagram from borehole KA1061A from Äspö HRL. The borehole is oriented in semi-horizontal direction and drilled towards fracture zone NE-1 /Stanfors et al. 1992a/. The borehole length is presented in vertical direction in the figure. The two way travel time or the radial distance to the reflection objects is presented in horizontal direction /Stanfors et al. 1992a,b/.*

rotated ideal directional antenna can be synthesised. The measured data is displayed as a series of radar diagrams, or azimuth maps (for instance one for each 10° sector), in the interval 0–170°. If rotation is continued (180–350°), the same diagrams will reappear but with the phase inverted, i.e. a reflecting plane will have two maxima and two minima in a full 360° rotation.

The directional receiver has two or three sensors for orientation of the probe: a vertical sensor used for inclined boreholes, a three-component fluxgate magnetometer used in vertical boreholes, and a plunge sensor (optional).

Optical fibres are used for transmission of the trigger signals from the computer to the borehole probes and for transmission of data from the receiver to the control unit. The major advantage of optical fibers is that they have no electrical conductivity and will not support radar waves propagating along the borehole.

As there is no direct connection between the transmitter and the receiver, they can be put into the same or separate holes. In other words, the radar can be used for both single-hole and crosshole measurements. The system also provides absolute timing of the transmitted pulses and calibrated gain in the receiver, which makes it possible to measure the travel time and the amplitude of the radar pulses in a crosshole measurement and hence provide data for a tomographic analysis. The absolute time depends on length of the optical fibres and is hence a quantity, which has to be obtained by calibration for a given set of optical fibres. The technical specifications of the system are given in Table 7-2.

Table 7-2. Technical specifications of the RAMAC® borehole radar system.

General	
Frequency range	20–80 MHz
Performance factor	150 dB
Sampling time accuracy	1 ns
Maximum optical fiber length	2,000 m
Maximum operating pressure	250 Bar
Outer diameter of probes	48 mm
Minimum borehole diameter	56 mm
Transmitter (60 MHz)	
Peak power	500 W
Operating time	8 h
Length (with battery)	5.3 m
Weight (with battery)	16 kg
Receiver (Directional 60 MHz)	
Bandwidth	10–200 MHz
A/D converter	16 bit
Least significant bit at antenna terminals	1 µV
Data transmission rate	1.2 MBaud
Operating time (with long battery)	8 h
Length (with long battery)	5.7 m
Weight (with long battery)	18 kg
Control unit	
Microprocessor	RCA 1806
Clock frequency	5 MHz
Sampling frequency	30–4,000 MHz
No of samples	256–4,096
No of stacks	1–32,767
Time window	0–11 µs

Measurement performance

The principle of a single-hole reflection measurement is depicted in Figure 7-7.

The radar probe with transmitter and receiver are lowered or, in horizontal holes, pushed into the hole while the distance between them is kept constant by glassfibre rods. The depth is measured by means of a measuring wheel and recorded by the control unit. At measurement points (normally every 0.5 m) the winch automatically stops and a measurement is performed (takes about 40 seconds for directional measurements and about 15 seconds for dipole measurements). A typical logging time for directional radar measurement of a 500 m borehole is about 12 hours.

Interpretation procedure

A schematic data flow for borehole radar measurement is shown in Figure 7-8. The radar measurement is interpreted with the aid of the computer program RADINTER.

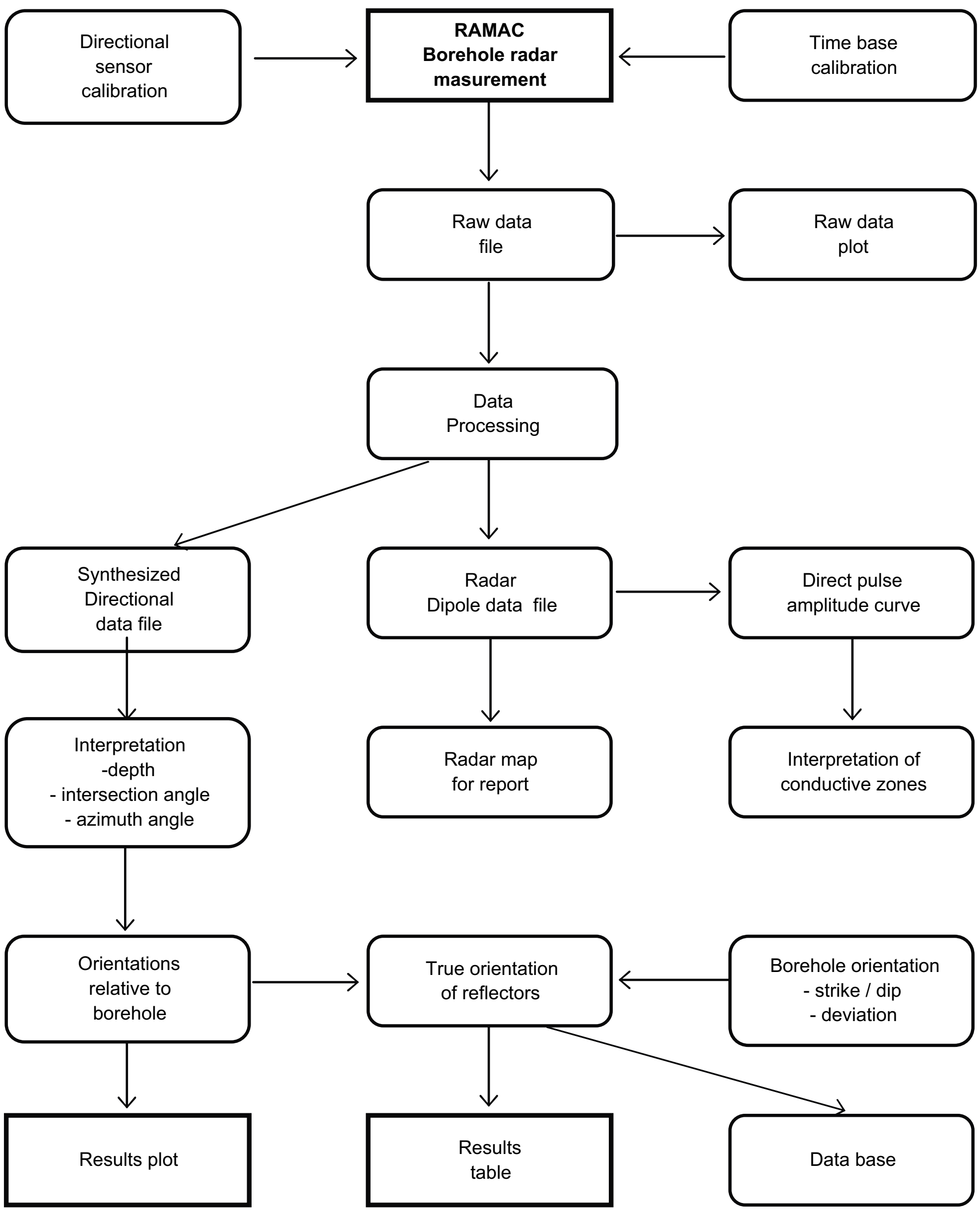


Figure 7-8. *Data flow chart for borehole radar measurements.*

The velocity is determined by a Vertical Radar Profile (VRP) measurement at the site. Measuring the difference in arrival time between the direct and the reflected pulse determine the distance to a reflecting object. The basic assumption is that the speed of propagation is the same everywhere. The two basic patterns are point reflectors and plane reflectors as shown in Figure 7-7.

From the radar reflection measurement it is possible to determine the angle of intersection between the hole and a reflecting fracture plane as well as the point of intersection in the borehole /Olsson et al. 1987/. The synthesised data set obtained from directional antenna measurements, as described in Section 7.3.2, is used to determine the direction to the reflecting fracture plane. The azimuth of minimum strength of a reflector representing a fracture plane is then determined. One of the two azimuth minima is chosen after comparison of the directional signal with the dipole signal (which is also extracted from the directional dataset). The selected minimum is used together with the intersection angle of the reflector with the borehole axis, the intersection depth of the reflector with the borehole and the actual borehole orientation (from deviation measurements), in order to calculate the 3-D orientation of the reflector.

The two azimuth minima result in different 3-D orientations and it is therefore essential for choosing the correct azimuth value. In cases where this selection of minimum is uncertain or impossible, the two possible orientations can be presented. Two possible orientations can also be presented in cases where the reflectors can be seen in the two azimuth maps but not in the dipole component map.

In many cases, reflections from fracture zones and other inhomogeneities in the rock mass are not readily observed in the original radar data. In order to enhance reflections, the radar data are digitally filtered. A detailed description of the different processing steps can be found in /Olsson et al. 1987/.

Presentation of results

Reflectors identified from directional antenna reflection maps are listed in tables. Examples can be found in /Carlsten et al. 1995/. The table includes a reflector identification number, the interpreted depth of intersection in the borehole, and the angle of intersection between the reflector plane and the borehole axis. Furthermore, the table contains the gravity azimuth of minimum strength, an estimate of the intensity of the reflector, the determined orientation of the reflector presented by the dip and the strike of the reflector. Finally, the position of and a comment on the geological character of corresponding structures close to or at the intersection point of the radar reflector are presented in the table.

Accuracy

The following sources of error contribute to the accuracy of the interpretation of radar reflectors:

- The borehole length to the intersection of the reflector.
- The angle to the borehole axis.
- The azimuth value.
- The borehole deviation data.

The measurement depth has been defined by the 2 m long glassfibre rods used to push the probe into the borehole. The accuracy of the intersection length of a reflector in the borehole is about ± 1 m when a 60 MHz antenna is used. The accuracy may be less for reflectors with a very small angle to the borehole axis.

The accuracy of the angle between the borehole axis and a reflector ranges from ± 1° for angles in the interval 0–30° to ± 2° in the interval 31–60° and ± 5° in the interval 61–90°.

The accuracy of the azimuth values of a reflector in a 360° system around the borehole is ± 10°. In order to get the true orientation of the reflector, the azimuth value and the angle to the borehole axis must be related to the inclination and declination of the borehole. The quality of the deviation measurement thereby contributes to the accuracy of the true orientation of reflectors. This means that in vertical boreholes the accuracy is less for sub-horizontal structures, while in horizontal boreholes the accuracy is less for sub-vertical structures.

The radar measurements performed at the Äspö HRL have been influenced by the high salinity of the groundwater, which is about 10‰ compared to 0.1‰ for fresh water. The high salinity results in shorter penetration range of the radar waves and generally weaker reflectors compared with the circumstances for radar measurements performed in a fresh groundwater environment. The radar range at Äspö HRL is represented by a cylinder with a radius of 20–25 m around the borehole. These conditions at the Äspö HRL have made the interpretation of orientation more difficult to perform. As a consequence the routines for interpretation of orientation were changed. In cases where the selection of azimuth minimum is uncertain, for the reasons given above, both alternatives for orientation are presented. This was not the case for radar measurements performed earlier at the Äspö HRL.

Comments and recommendations

The presence of saline groundwater at Äspö HRL has shown that radar measurement can be performed in such an environment, even if it results in shorter penetration range and weaker reflectors. This experience altered the routines for the interpretation of radar reflectors, so that both alternatives for orientation are presented when the reflector is weak or the definition of azimuth value is uncertain.

In horizontal boreholes the radar probes were pushed into the borehole by rods. If there was water inflow, this could make it difficult to push the rods into the borehole. The upper limit of inflow was about 50 l/min to be able to push the radar probes in the borehole.

For practical reasons, radar logging could not be performed close to the drift front during periods of excavation. Radar measurements were performed in boreholes located in niches.

7.3.3 Tunnel (and borehole to tunnel) radar

Instruments

Tunnel antennas for the RAMAC system were developed during 1989 and 1990 /Falk, 1991/. The tunnel antennas replace the borehole antennas for use in tunnels or on the ground surface. Optical fibres to the control unit connect the tunnel antennas in the same way as the borehole antennas. Compatibility allows radar measurements to be performed with one tunnel antenna and one borehole antenna. A 60 MHz antenna was used for the tests in the Äspö HRL.

The RAMAC/GPR (Ground Penetrating Radar) equipment has been developed from the borehole radar equipment for the specific purpose of being used for ground surface radar surveys. The antenna frequency used in this study was 100 MHz.

Methodology

Fixing the tunnel antennas to the tunnel wall performs tunnel radar reflection measurements. The dipole antenna can be oriented either vertically or horizontally and parallel to the tunnel in order to check the effect of using different polarizations. Point measurements are taken at regular intervals, for example 0.25 or 0.5 m.

For Vertical Radar Profiling (VRP) measurements, performed along the tunnel for the purpose of investigating the rock ahead of the tunnel face, the RAMAC borehole transmitter was placed in a short borehole while the tunnel receiver was moved along the tunnel.

Detailed description of both measurement modes is found in /Olsson, 1992/. GPR measurements are performed in a manner similar to tunnel radar reflection measurements. Measurement points are much denser, in this case 0.05 m, /Stenberg and Forslund, 1996/.

Accuracy

The accuracy of reflection measurement in tunnels is similar to that of reflection measurement in boreholes, i.e. the intersection length of the reflector and the angle to profile. It should be observed that the dip angle between the reflector and the profile is the apparent dip.

The angle to the tunnel axis ranges from ± 1° for angles in the interval 0–30° to ± 2° in the interval 31–60° and ± 5° in the interval 61–90°. The accuracy of intersection depth in the tunnel of a reflector is about ± 1 m using a 50 MHz antenna and ± 0.5 m using a 120 MHz antenna. Accuracy may be less for reflectors with very small angles to the tunnel axis.

Comments and recommendations

The radar method is sensitive to metallic obstacles and electrical installations in tunnel, and it has a shorter penetration range than the seismic method. The penetration of radar waves is sensitive to saline water and to an overall high fracture frequency. Reflections may be received from other sides or from the tunnel roof. Measurements on the tunnel floor, which consists of backfilled excavation muck saturated with high saline water, may reduce penetration more or less totally.

7.4 Seismic investigations

7.4.1 General description and purpose

In the early stages of tunnel construction, seismic investigations were carried out along tunnels and in boreholes drilled from the tunnels. Reflection surveys, with prediction distances of several hundred metres, were performed to determine the positions and orientations of the fracture zones.

At later stages, smaller-scale seismic investigations were carried out between boreholes. Reflection and tomographic analyses were performed to confirm and refine the fracture zone model inferred from earlier surveys and to determine physical properties of the rock mass, e.g. elastic moduli.

In the case of reflection surveys, the sources and the receivers are placed on the same side of the rock features to be imaged. This allows large volumes of rock to be probed with a relatively small instrumental deployment. A drawback of reflection imaging is that the knowledge gained by this method consists mainly of the geometry of the boundaries, with little insight offered on the properties of the rock between the boundaries. In the case of tomography, the investigation is limited to the area between coplanar arrays of sources and receivers. Tomography has reduced capabilities for resolving the geometry of the rock features, but provides information on the intrinsic physical properties of the rock.

Three seismic campaigns were carried out during the construction phase and will be reviewed in this section:

- Prediction of fracture zones ahead of the tunnel, in 1991 /Olsson, 1992/.
- VSP surveys in three boreholes drilled from Turn 2 of the tunnel spiral, in 1994 /Olsson et al. 1994/.
- Investigations before, during and after tunnel excavation works, as part of the ZEDEX experiment, 1995–1996 /Olsson et al. 1996/ and /Emsley et al. 1997/.

7.4.2 Instruments and methodology

Prediction of fracture zones ahead of the tunnel

The potential of the seismic methods for predicting fracture zones ahead of the tunnel was investigated in a study performed in 1991 /Olsson, 1992/. Three-component geophones were bolted on the tunnel wall, at 1 m intervals, forming a 96-station profile. Sledgehammer blows were made in the tunnel, along the receiver array. Additionally, explosive caps were detonated in holes drilled laterally from the tunnel. Seismic reflection imaging was used to detect fracture zones ahead of and around the tunnel and to estimate their position and orientation.

Complicated patterns of source-generated noise appear in the data, when the sources and receivers are both located on the tunnel wall. The noise has been attributed to the interference and scattering of several modes of interface waves travelling along and around the tunnel. The reflection events are generally drowned in noise and intensive data processing is needed to enhance the reflected wave-field. Accidental busts of coherent noise may also be enhanced by processing and appear as artefacts. This 'tunnel noise' decreases rapidly when either the sources or the receivers are moved away from the tunnel surface by placing them in boreholes.

Another drawback of line shooting along the tunnel, as opposed to the inverse VSP layout with sources in lateral boreholes, is that the orientation of the reflectors cannot be completely resolved. Due to the axially symmetric geometry, only the radial distance from the tunnel and the angle with the tunnel axis can be derived, while the azimuth relative to the tunnel remains undetermined. The use of three-component receivers does not help, as the polarisation of the waves is strongly distorted by the presence of the tunnel.

In the inverse VSP layout, with the seismic sources placed in lateral boreholes, the disturbing tunnel waves are reduced, but the observation above referring to the non-usefulness of the polarisation still stands. However, with this set up the offsets of the sources provide more geometrical constraints and allow an almost complete three-dimensional determination of the reflector orientations. Two equally possible orientations remain, i.e. symmetrical with respect to the plane defined by the source, and a linear receiver array.

VSP surveys from Turn 2 of the tunnel spiral

VSP surveys were performed in three downward-inclined boreholes drilled from Turn 2 of the tunnel spiral in 1994 /Olsson, 1994/. Explosive charges were detonated in short holes distributed along the tunnel. On average, 10 shot points were used for each borehole. Three-component geophones, shown in Figure 7-9, were placed at 1.25 m intervals from 10 to 120 m depth and at 2.5 m intervals between 120 m and the bottom of each hole, at depths of 260 –300 m.

Since the days of the Stripa Project almost two decades ago, multi-offset three-component VSP surveys have been used more and more extensively in connection with site characterization in crystalline rock for determining the 3D positions and orientations of fracture zones. VSP combines a large investigation depth with a relatively high sensitivity to rock discontinuities. Compared with tunnel seismic surveys, more accurate directional information can be obtained, especially when the structures interpreted from VSP surveys in several boreholes are combined in a single, comprehensive site model. However, standard VSP processing and interpretation techniques are not sufficient for mapping fracture zones in crystalline rock. New procedures mostly based on the Image Point transform /Cosma et al. 1994/, were developed to enhance weak reflections and to determine their 3D orientation.

The VSP surveys from Turn 2 of the tunnel spiral were performed in conjunction with radar measurements, interpretation being done in parallel and, towards the end of the process, jointly for the two methods. As VSP covers a significantly larger investigation range than radar, the reflectors interpreted by seismic methods in a small volume are fewer, but generally depict extensive site features.

Figure 7-9. *The R8-XYZ geophones, equipped with side-arms for clamping, used for the VSP surveys from turn 2. DC motors activate the arms and the clamping control is independent for each unit.*

Investigations performed as part of the ZEDEX experiment

Crosshole P- and S- wave velocity and attenuation tomography, as well as crosshole reflection imaging, were performed in 1994, 1995 and 1996 before and after excavating the TBM (Tunnel Boring Machine) and the D&B (Drill and Blast) tunnels located at a depth of 420 m below the surface /Cosma et al. 1994/ and /Emsley et al. 1997/.

The PS8R ultra-seismic tool used with ZEDEX has been designed for use in 56 mm or larger boreholes. The source consists of module housing, the piezoelectric transducer and a down-hole high-voltage power module. The piezoelectric transducer is positioned transversely to the ho1e axis and two pistons are opposed to the tip of the transducer. The source controller includes a sequence generator producing trains of pulses of pseudo-random length at pseudo-random time intervals. The receiver contains eight transverse piezoelectric transducers spaced at 0.15 m. The hydraulic clamping system is similar to the one used for the source. The usable frequency band is from 10 kHz to 70 kHz. The tool can be used in both single-hole and crosshole configurations.

The crosshole set up allows both the dip and the azimuth of the reflectors to be estimated. However, as with inverse VSP (prediction of fracture zones ahead of the tunnel), if the data are collected in a plane, e.g. in a pair of coplanar boreholes, two orientations symmetrical with respect to the plane are equally possible. This ambiguity disappears for reflectors perpendicular to the crosshole section; i.e. semi-vertical fractures can be imaged unambiguously in horizontal crosshole sections.

Figure 7-10 presents examples of both transmission and reflection tomographic results from boreholes B2 and B4, which form a horizontal section between the TBM and the D&B drifts. NW-SE trends can be observed both in the tomographic and in the reflection images. These trends are also consistent with the positions and orientations of the fracture zones interpreted from the VSP surveys in Turn 2.

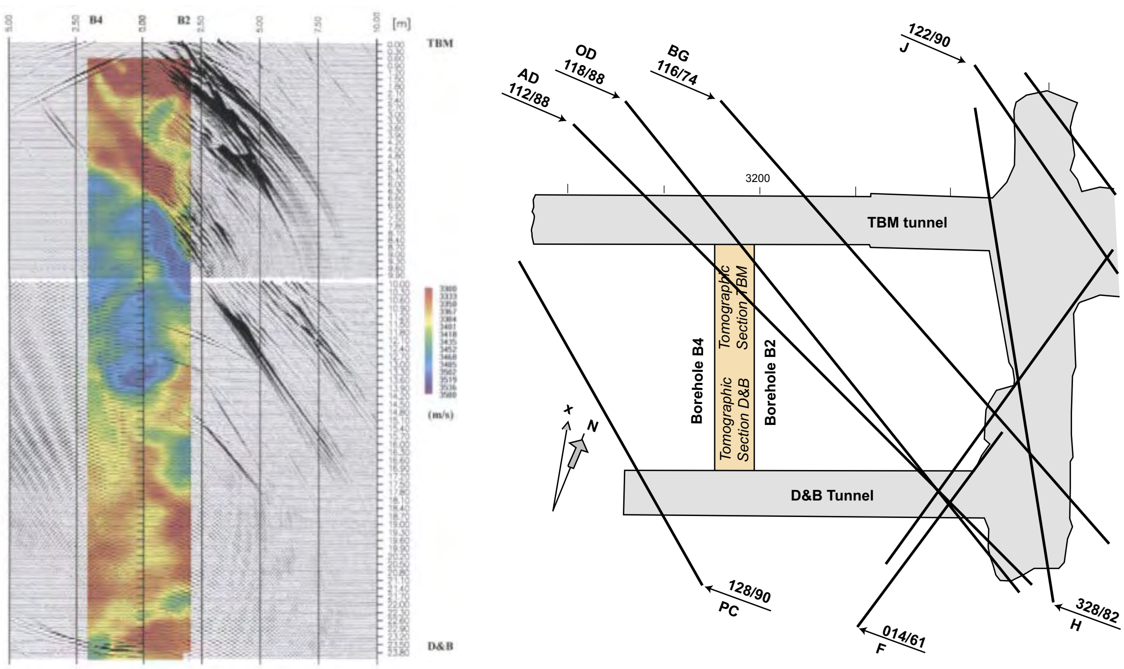


Figure 7-10. *Reflection imaging and transmission tomography between the ZEDEX boreholes B4 and B2.*

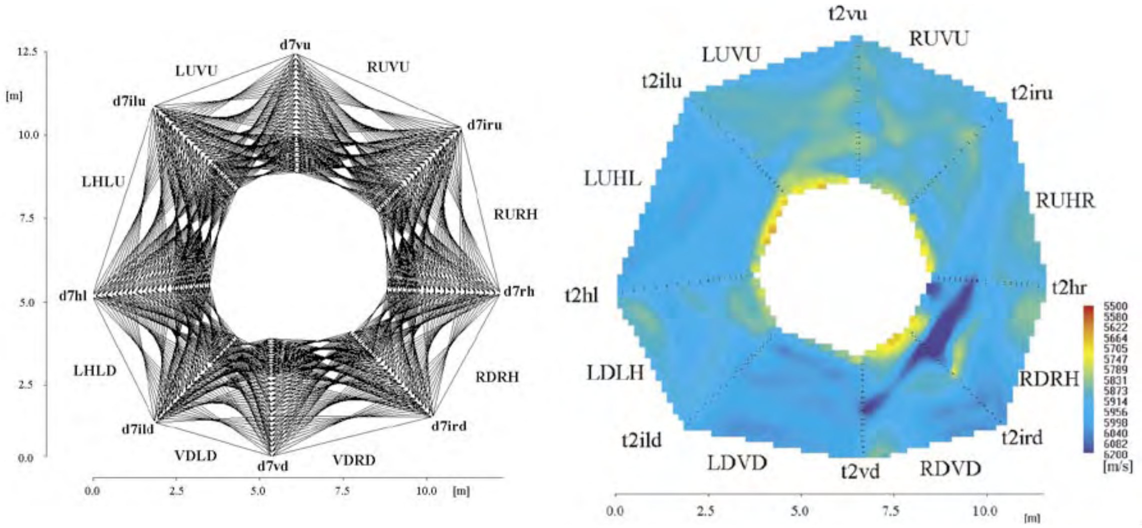


Figure 7-11. Ray diagram and P-wave tomographic velocity distribution in the eight crosshole sections measured around the TBM tunnel. Every tenth ray is shown.

For investigation of the near field, detailed seismic tomography surveys were performed in 1996, in two fans of radial boreholes. The measurements were carried out using the same borehole source and multi-receiver probe as in the previous ZEDEX experiments. As shown in Figure 7-11, eight crosshole layouts between pairs of adjacent boreholes were measured. The measurements were performed to a radial distance of 3.6 m from the drift perimeter. A variable increment of the source and receiver positions produced a more densely covered zone close to the drift.

7.4.3 Accuracy

The resolution of reflection surveys

A straightforward definition of the resolution of the reflection surveys can be given in terms of how accurately the reflected waves can be picked in the time-depth shot gathers. Accuracy depends both on the signal-to-noise ratio and on the frequency content and, for reasonably good quality data, it can conservatively be taken as 1/10 of the average period of the signal. To extend this definition to the accuracy of the spatial location of fracture zones, two additional ingredients are needed: a reliable velocity model, allowing the conversion of times into distances, and a method for determining the orientation in space of the reflectors. The velocity model is normally obtained by running transmission surveys, e.g. tomography, in parallel with reflection surveys. A precision better than ± 1% can be reached in converting times to distances by using the velocity field produced by tomography.

For the VSP prediction ahead of the tunnel and the surveys from Turn 2 described in Section 7.4.2 the average frequency was 300 Hz. With a velocity of 6,000 m/s, the average wavelength is approximately 20 m. The 1/10 limit gives a resolution of ± 2 m. When the limited precision in estimating the velocity field is also taken into account, the accuracy is better than ± 3 m at the maximum target range of 300 m.

Precision in determining the orientation of reflectors depends on, among other factors, the area of the reflector on which detectable reflections actually occur. This area is very small for reflectors perpendicular to the receiver array, while reflectors parallel to the array are probed along larger areas. Orientation can be more precisely determined in the latter case.

The azimuth of the reflectors, briefly discussed in Section 7.4.2, is initially estimated by polarization analysis. This estimate is improved by combining the interpretation of several profiles, shot in a muti-offset multi-azimuth configuration. By polarization analysis alone, orientation can be determined with a precision of 10–200 with two possible solutions at 1,800.

The resolution of tomographic surveys

Tomography offers a set of techniques for the inversion of travel time and amplitude loss to velocity and attenuation fields. Tomography performs two tasks simultaneously: it determines magnitudes of physical parameters and assigns the values determined to regions of the rock mass. The resolution of tomographic methods therefore has two aspects: the measurable variation of a given parameter, and the size and shape of the spatial element in which the parameter can be determined independently.

Physical resolution, related to the magnitude of the parameter to be imaged, depends primarily on the accuracy of travel times and coordinates. With ZEDEX, the precision of the P-wave times has been estimated at ± 2 µs, with travel times ranging from 0.3 to 0.6 ms, i.e. less than ± 1%. Regarding the coordinates, errors of ± 2 to 3% are quite common. Reducing the positioning errors to the same level as the timing errors is sometimes hindered by the similar appearance of positioning artefacts and genuine anisotropy.

Spatial resolution depends on the density and continuity of the arrays and on view angle diversity. Propagation effects, e.g. ray bending, must also be considered.

7.4.4 Comments and recommendations

VSP generally succeeds in determining the geometry of the fracture zones, even with very diverse orientations, but shadow zones of complicated shapes may exist. Crosshole reflection imaging does not produce complicated shadow zones, but field implementation is more complex than with VSP.

Besides inferring a geometrical model of detectable rock features, seismic surveys produce information regarding the character of the features detected. For the experiments where both transmission and reflection methods were used, the rock model was gradually built up by combined interpretation of reflection and transmission data. This model included distributions of the elastic properties of the rock and estimates of anisotropy.

At Äspö it has been shown that it is possible to map and seismically characterise rock features, at least in the range of hundreds of metres, and most of the geometrical predictions made in one stage were verified in the next. On the metre scale of the ZEDEX experiment, seismic results could be directly compared with rock mechanical and hydraulic data.

Reflection techniques focus more on the geometry of the structures and less on the intrinsic properties of the rock mass. Therefore, reflection imaging mainly provides information on the location and orientation of reflectors. Tomography has reduced capabilities for resolving the geometry of the rock features, but provides information on the intrinsic physical properties of the rock. Increasing efforts are being made to combine these two investigation approaches as parts of a ‘waveform tomography’ approach. At present, waveform tomography is roughly 1,000 times more intensive computationally than either tomography or reflection imaging and practical results cannot be obtained with desktop PCs. However, there are clear signs that the future of seismic site characterisation lies in that direction.

8 Hydrogeological borehole investigations

8.1 General

An extensive hydrogeological testing programme was carried out during the construction of the Äspö HRL tunnel. The general purpose was:

- To provide data for comparison with predictions and to submit additional hydrogeological information for detailed groundwater characterisation and updating of models.

An overview of the subjects forming the basis for the comparison is shown in Table 13-1.

A large number of pressure build-up tests were performed in order to determine hydraulic parameters such as transmissivity, specific capacity etc. The testmethod varied, depending on whether the tests were performed in probe holes near the tunnel face or in so-called investigation holes at various locations along the tunnel. Interference tests were used for characterisation of large rock volumes and for identification of the geometry and extent of hydraulic conductors.

With reference to Section 2.2 and Figure 1-5, this chapter describes and discusses all hydrogeological borehole methods used during the Äspö HRL construction phase. The methods are as follows, see also Figure 8-1:

- Pressure build-up tests in probe holes.
- Pressure build-up tests in investigation holes.
- Groundwater pressure monitoring during drilling.

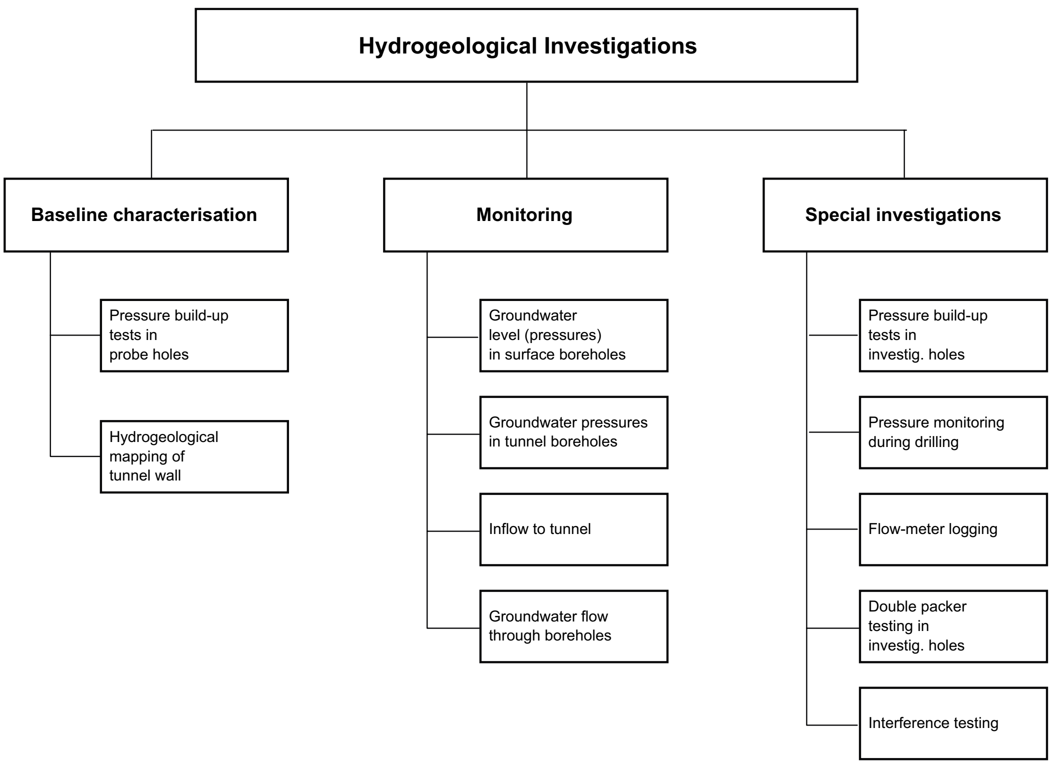


Figure 8-1. *Hydrogeological investigations during the Äspö HRL construction phase.*

- Interference tests.
- Tests with double packer system in investigation holes.
- Flow meter logging.
- Groundwater flow measurements.

8.2 Pressure build-up tests in probe holes

8.2.1 Purposes

As presented in Section 5.2, probe hole drilling and investigations were a part of the baseline characterisation, see also Figure 1-5. Probe hole drilling itself, during drill-and-blast excavation and during TBM tunnelling, was described in Sections 5.2.2 and 5.2.3 respectively.

Immediately after completion of the probe holes, 295 pressure build-up tests pressure build-up tests were performed. The purposes of the pressure build-up tests were:

- To determine hydraulic properties of the bedrock close to the borehole
- To identify hydraulic responses between pairs of probe holes at opposite sides of the tunnel.

The pressure build-up tests in the drill-and-blast excavated tunnel were carried out by the characterisation team in direct connection with every fourth tunnel front mapping. The extra time needed for the pressure build-up tests, including drilling was in the range of 2–2.5 hours.

In the TBM-bored tunnel, the tests in the probe holes were executed only in a limited part of the tunnel. They were always performed at night, when the TBM machine was not in operation.

8.2.2 Pressure build-up tests in probe boreholes in drill-and-blast tunnels

Instruments

The following equipment was used for the pressure build-up tests Figure 8-2:

- Single packer assembly.
- Valve arrangement.
- Pressure transducers.
- Data logger.

Mechanically operated packers were applied to shut off the test section. The sealing rubber length of the packers was 0.3 m and the rubber diameter 53 mm. Rotating a nut on the innermost of two packer pipes sealed the packer, resulting in an axial compression of the packer so that the rubber was squeezed out to the borehole wall. The packer system (including the packer pipes) had a standard length of 6.0 m and an inner diameter of 13 mm. In boreholes yielding more than approximately 5 l/min, packers with an inner diameter of 27 mm were used. These packers were often shorter: 4.0 m or 1.8 m, see Figure 8-3. The shorter alternatives were used if the holes were too curved to permit installation of the 6 m long packer system.

Figure 8-2. *Pressure build-up tests just behind the tunnel front during drill and blast tunnelling.*

Figure 8-3. *Equipment for pressure build-up tests in probe holes. Left: Packers for pressue build-up tests in probe holes. The long "standard" packer to the right, long packer for high flow in the middle and a short packer for severe installations in curved holes. Right: Borre MDL data logger and pressure transducer mounted to the BAT rubber disc connection of the valve arrangement.*

A valve arrangement, including a pressure gauge for manual reading and a sealing BAT rubber disc mounted in a nozzle, was connected to the inner packer pipe. For very low-conductive boreholes, a ca 30 cm tecalan tube (ID 4 mm) was mounted on the valve arrangement for measuring the flow.

The pressure transducers used were Druck PTX 510-00 or Druck PTX 610-0I. The pressure range was 25 bars, 35 bars or 50 bars depending on the depth of the tested boreholes in the tunnel. A hypodermic needle was mounted on the transducer housing. When the transducer was connected to the valve arrangement on the packer pipe, the needle penetrated the rubber disc, providing hydraulic communication between the measurement section and the transducer.

The data logger BORRE MDL version 2.2 was utilised for the data recordings. Basically, this is the same logger as was used for monitoring groundwater head in the surface boreholes on Äspö /Almén and Zellman, 1991/. However, the tunnel logger has been modified to cope with the severe conditions prevailing in the tunnel. All components of the data logger – including multiplexer, power supply (battery and mains transformer), microprocessor, A/D converter and memory – are enclosed in a watertight box. The box is provided with legs and a handle grip for transport. The data logger is equipped with 13 input channels, but only the five first channels are equipped with waterproof connectors at the bottom of the box. The data logger is also equipped with connectors for data communication and external power supply. This permits measurements to be made without opening the box.

The measurement software is very flexible with regard to sampling intervals etc. A measurement programme can be started (or changed) either by a temporarily connected laptop PC, or by using the key set at the front of the data logger. The key set permits three measurement options. The keys "SLOW" and "FAST" result in measurement at one-hour and five-minute intervals, respectively. For hydraulic testing, the "SEQ" option was usually used. This option has gradually increasing intervals and started with 2–6 seconds, depending on the number of channels used. After 30 minutes, the measurement interval was three minutes. These "SLOW", "FAST" and "SEQ" options can easily be re-programmed from the PC.

Methodology

Before testing, the pressure transducers were calibrated using the reference system of the Hydro Monitoring System, see Section 11.3.2 and Figure 11-3.

The standardised procedure for pressure build-up tests in the two probe holes drilled every fourth round was as follows:

- Immediately after completion of the first probe hole, the packer was installed and the measurement section sealed off.
  - The packer was in most cases manually positioned in its location. Only when extreme flow rates were encountered the drilling rig was needed for packer installation.
  - The valve arrangement was mounted on the packer pipe and the valve was left open.
  - If no water flow was observed, the borehole and/or the packer pipe was filled with water in order to evacuate the air.
- As soon as the probe hole on the opposite tunnel wall was completed, the packer installation procedure was repeated in this borehole.
- The valves were held open for at least 45 minutes in each borehole so that water flowed through the measurement sections. The water flow rate was measured manually with graduated cylinders and a stopwatch several times during the flow period. If the flow rate was very low, a 0.3 m long tecalan tube (ID 4 mm) was mounted on the valve and the flow rate was determined by measuring the rising water level in the tube. Flow rates down to about 10^{-4} l/min could be measured.
- The less permeable of the two boreholes was closed about 15 minutes before the other.
  - A few minutes before closing of the valve, the pressure transducer (connected to the data logger) was mounted on the valve arrangement.

  – About 30 seconds before valve closing (and start of pressure build-up), data logger recording was initiated. The “SEQ”option was selected.

- Monitoring of the pressure build-up continued for 30 minutes or more in each borehole.
  – The pressure gauge was also manually read during the entire measurement sequence. These readings – together with flow data, borehole geometry data, packer position, start/stop times, etc – were noted in a field record.
- Stopping the data logger and disconnecting the pressure transducer from the valve arrangement terminated the pressure build-up test. The packers were left (sealed) in the holes, and the valve was left closed for long-term, low-frequency, manual observations of the pressure gauge.

Data processing and evaluation

After completion of the test, the raw data was retrieved from the data logger to a desktop computer at the site office, either directly or via a laptop computer. The raw data file, consisting of A/D levels in hexadecimal codes, was converted to an ASCII file with the aid of a conversion program and a file containing the calibration constants from the calibration mentioned above, see Figure 8-4. Further conversion created new files, permitting plotting of the pressure build-up on linear, logarithmic and semilogarithmic scales, see Figure 8-5.

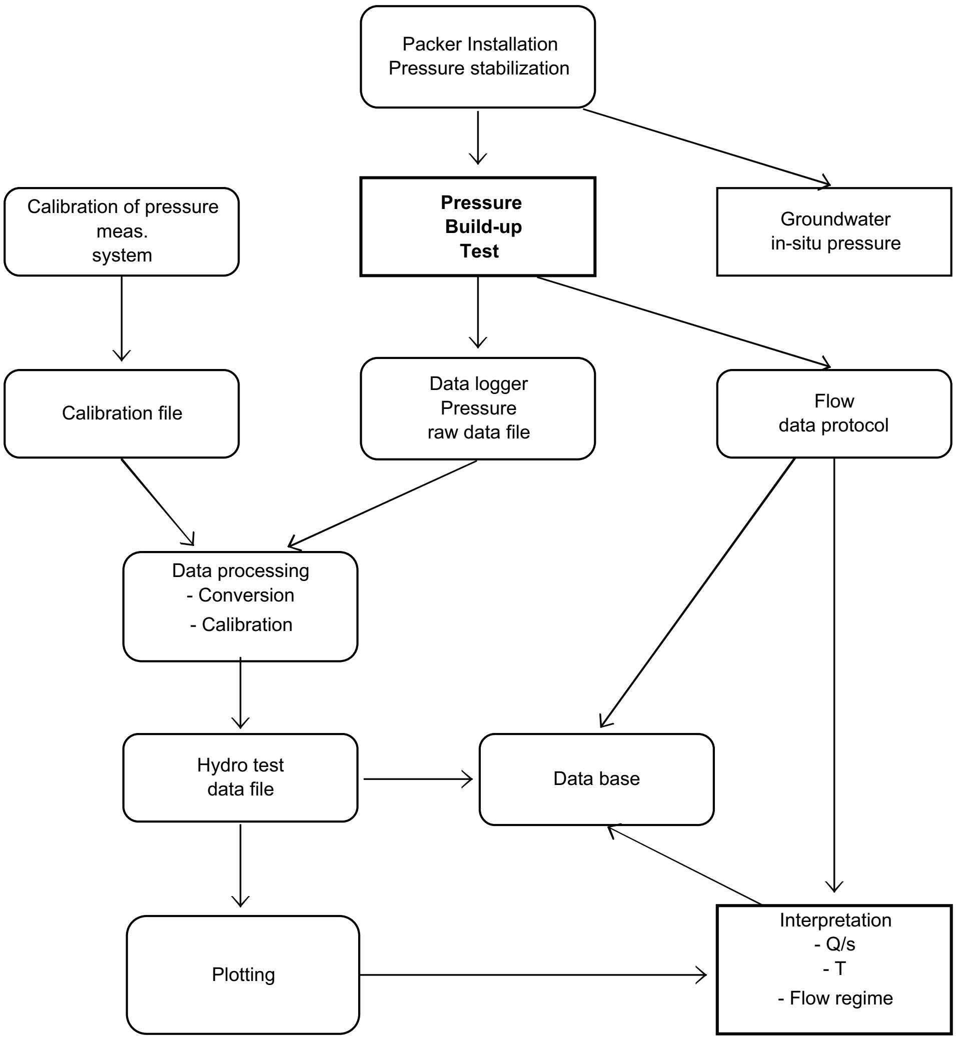


***Figure 8-4.** Data flow chart of pressure build-up tests.*

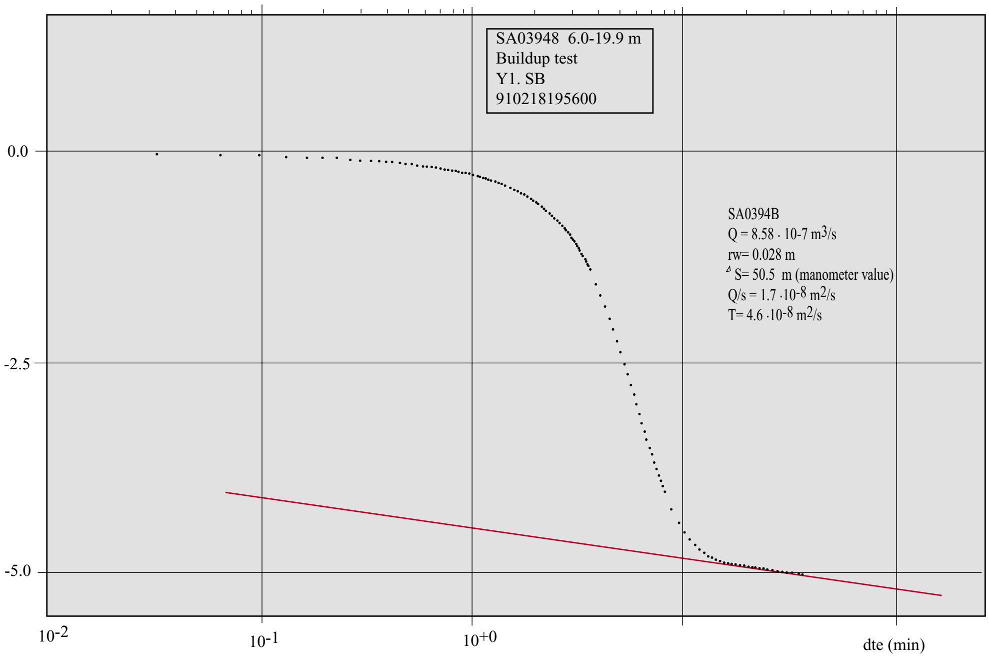


Figure 8-5. *Semi-logarithmic diagram showing pressure build-up data from probe hole SA0394B.*

Graphs and data from the field record of each test were compiled in a report, which was used for evaluation of the test. The primary evaluated parameters were:

- Specific capacity, Q/s (m^2/s).
- Transmissivity, T (m^2/s).

The specific capacity is, as mentioned above, equal to Q/s, where Q is the calculated average water flow rate before closure of the valve and s is the maximum change pressure head, expressed in metres (of water column), during the test /Rhén et al. 1991/.

The flow regime can be determined by analysing the logarithmic plot. Usually the flow can be considered to be radial to the borehole during the latter part of the pressure build-up phase. Transmissivity is then calculated with Jacob's semilogarithmic approximation of the Theis well function /Theis, 1935/:

T = 0.183 Q/Ds

where: Q = the average flow rate before closing the valve (m^3/s)

Ds = the pressure head change in metres during a decade along the straight line in the semilogarithmic diagram.

When the diagrams indicate a more complex flow regime, alternative evaluation methods should be considered, such as for example those presented in /Earlougher, 1977/ and /Gustafson, 1986/.

Data files from measurements and evaluated results are stored in the SICADA database, see Figure 8-4.

After every 150 m of excavation, the site office produced an overview of documentation including geological mapping, data on geohydrology and groundwater chemistry etc, see Section 2.2.2 and Figure 2-9. The following results were included from the pressure build-up tests in the probe holes /Rhén et al. 1993/ and /Markström and Erlström, 1996/:

- Inflow to the probe holes, Q (l/min).
- Transmissivity, T (m^2/s).
- Hydrostatic pressure in the probe.

8.2.3 Pressure build-up tests in probe holes during TBM excavation

The methodology of the tests in the TBM tunnel was similar to the methodology used in the drill-and-blast tunnel. The test equipment was, however, modified in order to suit the conditions prevailing in the vicinity of the TBM head.

Instruments

An inflatable packer sealed off the measurement section. The sealing length was 0.4 m and the packer was expanded against the borehole wall by means of pressurised water from a manually operated pump. The water from the measurement section was discharged through a stainless steel pipe string, with an OD/ID = 33/21 mm. The groundwater pressure in the test section was transferred to the tunnel through a second pressure line. At the end of this line, a valve arrangement similar to the one described in Section 8.2.2 was mounted, including a pressure gauge and a "BAT connection" for the pressure transducer.

Methodology

The pressure build-up tests during TBM tunnelling were carried out and reported in almost the same way as the tests in the drill-and-blast tunnel, see Section 8.2.2. The main difference lay in the practical arrangements and working procedures in the almost "non-existent" working space in the machinery part of the TBM machine, see Figure 8-6. The pipe string and the packer were installed and fixed in position with the aid of the drill rig. The hydraulically expanded packers were another difference mentioned above. There was also a difference in the overall routines for the probe holes, i.e. probe hole drilling was not conducted in the same standardised manner as for the blasted tunnel, as was mentioned in Section 5.2.3.

Figure 8-6. *Pressure build-up tests during TBM excavation – an unpleasant activity.*

The TBM was equipped with two drilling rigs that could be rotated almost ± 180° relative to the tunnel line, see also Sections 5.2, 5.5 and Figure 5-2. Drilling could be done through two holes in the TBM head, making it impossible to deviate from the tunnel line. Another possibility was to drill boreholes in the tunnel wall behind the TBM head. The maximum angle out from the tunnel line for that alternative was 7°. It turned out to be very difficult to drill in the wall of the TBM tunnel, so only three such holes was drilled.

The outcome of the tests was not altogether satisfactory. It turned out to be very difficult to evacuate all air from the boreholes, as almost all the holes were directed horizontally or slightly upwards. This was due to the limitations on drilling in the specified directions. In particular when TBM tunnelling was directed more than 2% upwards, which was the case for almost all the holes as they were drilled during the last 100 m of the tunnel, it was impossible to maintain the slightly downward direction of the probe holes. Those holes were drilled through the TBM head. It was also impossible to evacuate all air from the measurement section within the time available and with the equipment used.

Data processing and evaluation

Data processing and evaluation of data were performed in the same way as for the pressure build-up tests in the probe holes during drill-and-blast excavation, see Section 8.2.2.

8.2.4 Accuracy

The uncertainty of the evaluated hydraulic properties stems from:

- Manual water flow rate measurements.
- Groundwater pressure measurements (the measurement system, insufficient evacuation of air, etc).
- Evaluation of tests.

The error in determining the flow rate has been estimated to be ± 5% in the interval 0.01–100 l/min. If larger or smaller flow rates were measured, the error was estimated at ± 10–20%.

The pressure transducers have a combined error due to non-linearity, hysteresis and repeatability of 0.3%, full scale. The typical value is 0.15%. This error was reduced by regular in-situ calibration. The errors in the calibration constants are related to the status of the pressure reference system, see Section 11.3.2. Based on the instrument data and the calibrations, the total error in absolute pressure was estimated to be ± 2–5 kPa. The error in small pressure changes is smaller.

The errors in the evaluation technique are more difficult to estimate. Of most importance is perhaps defining the correct groundwater model (flow regime) for the test. In very low-permeable boreholes the duration of the test was too short, resulting mainly in WBS (Well Bore Storage) effects, which caused difficulties in interpretation of data.

8.2.5 Comments and recommendations

Pressure build-up tests in the probe holes during drill-and-blast excavation could be made at almost any time, requiring good equipment preparations. Four to six packers of different length and width were brought down to the tunnel front to suit the different flow conditions prevailing in the boreholes. In general, the installation of the packers required two persons. In very high-flow holes even this was not enough; help from the drillers was also required.

As tunnel excavation progressed, routines for pressure build-up tests in probe holes were developed and worked well from packer installation to completion of the field report. The biggest problem was evacuating the air in low-permeable boreholes. A thin hose connected to a funnel was used to fill up the borehole with water from the inside. This procedure was time-consuming and in many cases not sufficient to get rid of all the air, which made the evaluation more difficult.

The first blasting round after the pressure build-up tests was critical for the equipment sticking out of the borehole. The packer pipe end and valve arrangement, were sometimes destroyed during mucking out, sometimes due to misunderstanding in the construction team.

In high-flow boreholes, difficulties were sometimes encountered in fixing the packers in position. After the rubber was expanded as much as possible, the packer could still creep if the rubber was not expanded for a few more minutes after the first time.

In the deeper part of the tunnel, the groundwater pressure in the boreholes was so high that expanding the packer was not sufficient to fix the packer. It was necessary to anchor the packers with chains and rock bolts.

In extremely high-flow holes, when the packer with large inner pipe (and thin rubber) was used, the rubber element sometimes wriggled out of the packer frame. Using a stiffer rubber material and vulcanising the rubber end to the packer frame solved this.

The pressure build-up tests during TBM drilling were performed from the space between the TBM head and the TBM grippers. The space there was very cramped, wet, dirty, slippery and warm, making it difficult to carry out satisfactory tests, Figure 8-6.

As discussed in Section 8.2.3, the options for drilling holes were limited to drilling through the TBM head and drilling low-angle holes in the side wall. These low-angle holes in the side wall were also considered to be a risk from the rock-mechanical point of view, since the boreholes could transmit high water pressures close to the wall and cause scaling.

The packer was installed in the holes using the drill rig on the TBM. The biggest problem was purging the air from the boreholes, since they were directed upwards in the last part of the tunnel, see Section 8.2.3.

If hydraulic testing in probe holes drilled during TBM tunnelling is a part of the investigation programme, the drilling equipment must be designed to permit greater flexibility in drilling and better conditions for hydraulic testing of the borehole.

8.3 Pressure build-up tests in investigation boreholes

Purposes

Several investigation boreholes (percussion holes and cored holes) were drilled for different purposes and from different locations in the tunnel during the construction of the Äspö HRL tunnel, see Section 5.3. Investigation of fracture zones and characterisation of rock volumes for detailed tunnel layout and for location of experimental sites were common reasons.

As a rule, pressure build-up tests were carried out in the investigation boreholes. A total number of 135 pressure build-up tests were performed in the investigation boreholes. The purposes of the tests were:

- to determine the hydraulic properties of the bedrock around the borehole,
- to identify and characterise conductive features penetrated by the respective boreholes.

Instruments

The equipment used in the percussion boreholes was identical to that used in the probe holes, i.e. the same type of mechanically operated packers and the same measuring instruments, see Section 8.2.2.

In the cored boreholes, the measurement sections were sealed-off by inflatable single packers similar to the one used for pressure build-up tests in the TBM tunnel. The sealing length of the rubber was 1.0 m and the packer was expanded against the borehole wall by water pressure. Pressurising the water with nitrogen gas in a pressure tank created the inflation pressure. The packer pressure was normally set to 15 bars above the expected hydrostatic pressure in the borehole (or the maximum groundwater pressure).

The water from the measurement section was discharged through an aluminium pipe string, with an OD/ID = 33/21 mm. At the end of the pipe string a valve system was mounted, similar to the one described in Section 8.2.2, including a pressure gauge and a "BAT connection" for the pressure transducer. As an alternative, the transducer could also be connected to a separate pressure line from the measurement section to the measurement system in the tunnel.

Pressure variation during the tests in the cored holes was monitored with the same equipment as was used in the pressure build-up tests in probe holes, i.e. data logger BORRE MDL version 2.2 and Druck PTX pressure transducers of various pressure ranges, see Section 8.2.2.

After approximately 1,000 m tunnel length, a casing was fixed in almost all investigation boreholes, see Section 5.3. A valve was mounted on the casing, enabling the borehole to be closed during interruptions and after the drilling. The casing had an extra outlet on which a valve arrangement similar to the one used for probe holes was mounted, see Figure 5-5.

Methodology

As for pressure build-up tests in probe holes the pressure transducers were calibrated before testing, using the reference system of the Hydro Monitoring System, see Section 11.3.2.

The test procedure in the investigation boreholes varied somewhat depending on the borehole type and on when in relation to drilling, the test was carried out.

When the percussion boreholes were tested directly after drilling, the test procedure was identical to that described in Section 8.2.2. However, the boreholes were often closed after drilling and the tests postponed to a later occasion. In these cases as well, the test sequence was almost identical as for the probe hole tests. The difference was that the flow period started when the valve was opened and that the pressure measurements also included pressure drawdown during the flow period.

The tests in the cored investigation boreholes were often performed during interruptions in the drilling. The borehole interval drilled since the previous break was then tested. The length of the test interval, delimited by an inflatable packer and the borehole bottom, varied from 10 m to 100 m.

As described in Sections 5.3.4 and 5.3.5, the drilling of investigation holes was sometimes interrupted due to borehole instability or high-permeability zones (excessive water outflow), and part of the hole was grouted before drilling was continued. In these situations, a pressure build-up test was conducted in the section to be grouted. As the grouted section should be short, a packer was generally placed as little as 10 m from the borehole bottom, in order to grout just the bottom of the borehole, see Figure 5-6.

The test sequence for the measurement sections in the cored boreholes was similar to the one described in Section 8.2.2. After installation in the borehole, the packer was inflated and the measurement section was discharged for 30–60 minutes. The pressure build-up period lasted between 30 and 120 minutes. The groundwater pressure was measured as described in Section 8.2.2.

In some investigation boreholes, a packer did not seal off the measurement section. Instead the arrangement on the casing was used, see also Section 5.3.4. With this arrangement the test section length was equal to the whole borehole length, excluding the casing length.

Data processing and evaluation

Calibration, data processing and evaluation of data were performed in the same way as for the pressure build-up tests in the probe holes, see Section 8.2.2.

Results from pressure build-up tests in investigation boreholes are presented in a number of reports referred to in /Stanfors et al. 1994/. One example of these reports is /Rhén et al. 1994a/. The evaluation theory is described by /Gustafson, 1986/ and /Rhén et al. 1997a,b/.

Accuracy

The sources of error associated with pressure build-up tests in investigation boreholes are similar to those for tests in probe holes, see Section 8.2.4. The error in the pressure measurements is the same, since identical equipment is used.

The borehole intervals tested in the investigation boreholes were long or/and very high-flow. The problem of air evacuation was therefore less compared with the tests in the probe holes at the tunnel front. The accuracy of the flow measurements was also better. For flow rates < 100 l/min, the error was estimated to be ± 5%, and in the flow interval > 100 l/min the error was estimated to ± 10%.

Comments and recommendations

The investigation boreholes were normally drilled from niches. This made it relatively convenient to carry out the tests without disturbance from other tunnel activities. The tests were also easy to plan since the size of the flow rate and the groundwater pressure was as a rule known prior to the start of testing. The relatively high flow rates lead to less or no problems with air in the test section. Except for the installation of the pipe string and a hydraulically expanded packer, which required two persons, one member of the characterisation group could carry out pressure build-up tests in the investigation holes. When packers were used it was very important to anchor the packer pipes or the pipe string to the tunnel wall.

When long investigation boreholes are drilled from underground, high flow rates may occur that require some action (grouting etc) before drilling can continue. In order to obtain hydraulic data on the conductive feature, one has to be prepared to conduct a hydraulic test at any time during the drilling. Guidelines on when and how grouting is to be done should be given, reflecting the "hydrogeological" aim of minimising the grouted section, in case detailed hydraulic tests will be performed later. Correct documentation on the grouting is very important, to permit interpretation of hydraulic tests performed later in the borehole.

It was difficult to get reliable results from low-conductivity sections, mostly due to unstable pressure conditions in the rock mass before a pressure build-up test. The boreholes were generally open during the period when the packers were moved to the test position, which creates a drawdown response followed by a pressure build-up after inflation of the packers, making the evaluation becomes more uncertain. Later during the construction phase, double packer tests and a new borehole sealing device were used in cored boreholes, see Section 8.6.

8.4 Groundwater pressure monitoring during drilling and tunnelling

Purpose

During tunnel excavation and drilling of the investigation boreholes, pressure observations were made in existing boreholes, from the surface as well as from the tunnel. The groundwater head was monitored by the Hydro Monitoring System (HMS), either with a standard sampling rate or with an increased sampling rate for certain boreholes (HMS, see Chapter 11). Portable data loggers and pressure transducers were also used for boreholes not fitted with HMS equipment. The purpose of the pressure monitoring was:

- To observe pressure responses caused by tunnelling or drilling through water-bearing structures in order to identify the geometry and if possible also determine the hydraulic properties of those structures.

Such organised observations were conducted on a large number of occasions during the drilling of investigation holes during the construction phase of the Äspö HRL. All of them were from the drill-and-blast tunnel, and the most spectacular event was the passage of fracture zone NE1.

Instruments

Packers were usually installed in the observation boreholes at the start of the monitoring. Mechanical packers (in probe holes and percussion-drilled investigation holes) or hydraulically operated packers (in cored investigation holes) were used, see description in Sections 8.2, 8.3 and 11.2.2. A few holes without packers but with casing and valve arrangements were also used, also mentioned in Section 8.3.

The groundwater pressure head was monitored either by the HMS (see Chapter 11) or by means of the data logger BORRE MDL and Druck PTX pressure transducers, the same as for pressure build-up tests, see Section 8.2.2.

For boreholes not already connected to the HMS, the valve arrangement was mounted on the pipe string or on the casing for connection to the pressure transducer.

Methodology

Before measurement, the pressure transducers were calibrated with the HMS system, see Section 11.3.2. The measuring instruments were installed and connected to the selected observation boreholes. These holes were chosen since they penetrate water-bearing features expected to be in hydraulic communication with the new borehole or otherwise relevant for updating the hydrogeological model.

Observations of water outflow during drilling were made to be able to relate abrupt changes in hydraulic pressure responses in surrounding boreholes to drilling operations.

The HMS and/or the data loggers monitored groundwater head in the observation boreholes during the whole drilling period. The sampling interval was five minutes during measurement periods, compared with the standard sampling interval of one hour.

Data processing and evaluation

After the drilling, data from the drill crew documentation were transferred to a data file and the drilling rate was plotted with an appropriate time scale. Pressure data from the data loggers were processed as described in Section 8.2.2. The pressure variation in the observation boreholes (including selected boreholes monitored by the HMS system) was then plotted in diagrams with the same time scale as the drilling rate diagrams.

The plots were used to evaluate a possible correlation between pressure responses in the observation holes and features in the new borehole. An instant pressure loss in an observation borehole indicated that the new borehole penetrated a water-bearing fracture in contact with the observation hole, and was hence a tool for determining the geometry of the permeable feature, see Figure 8-7. The position of the fracture in the drilled borehole can be estimated by comparing the time of the pressure loss with the time on the drilling rate plot. Notes by the drill crew and by the site geologist are also taken into account when analysing the data, see Figure 8-8.

In a similar way, water conductors penetrated by the advance of the tunnel could be identified by comparing the groundwater head curves with the time entries in the tunnel excavation log. Examples of reports dealing with results from groundwater monitoring during drilling and tunnelling are /Olsson et al. 1994/, /Rhén and Stanfors, 1993/ and /Stanfors et al. 1992a/.

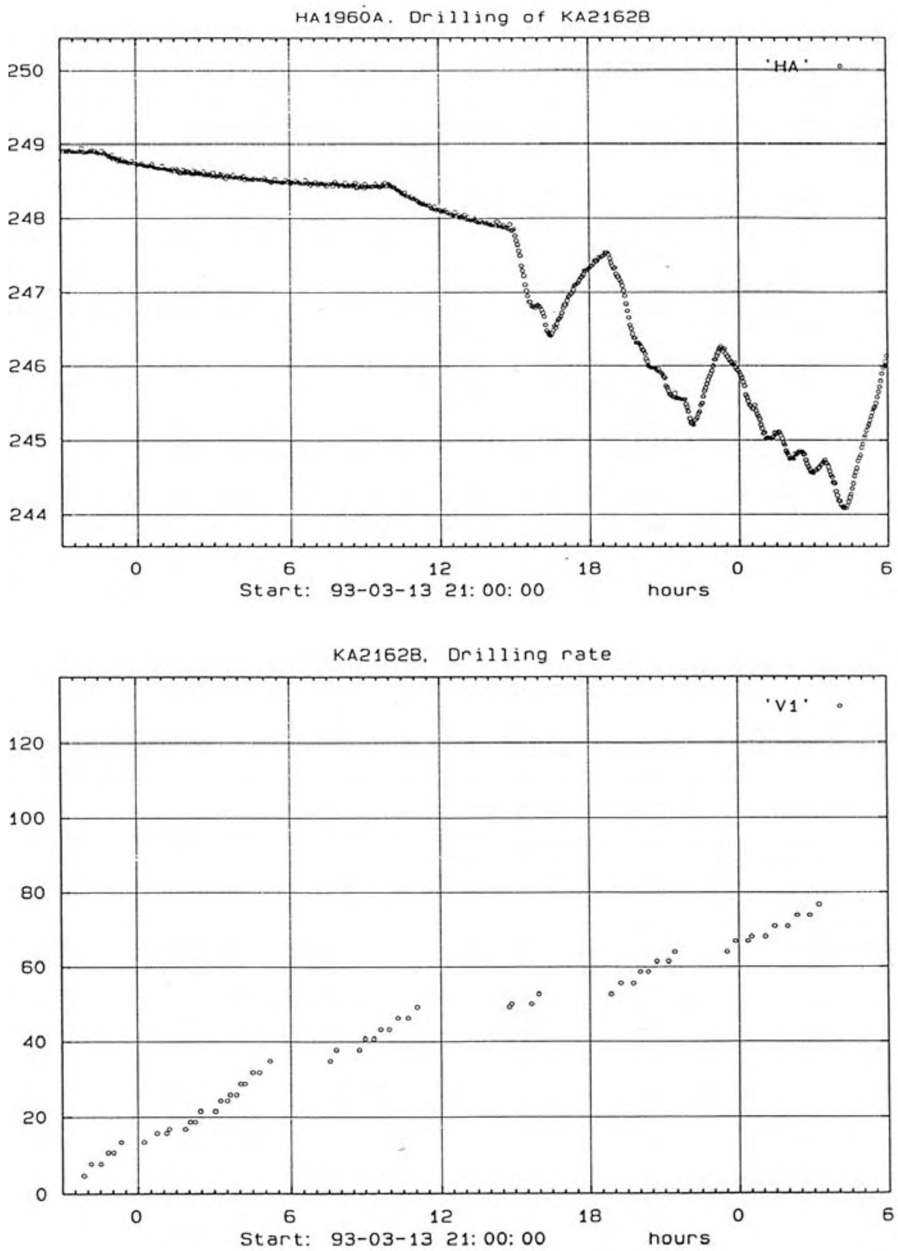


***Figure 8-7.** Pressure responses in borehole HA1960A during drilling of cored borehole KA2162B.*

Accuracy

The total error in the pressure measurements by data loggers and transducers is estimated to be ± 2–5 kPa, see Section 8.2.4. The error in small pressure changes is smaller. The accuracy of the HMS measurements is described in Section 11.3.2.

Comments and recommendations

When the portable data loggers were used to record pressure, the loggers could be left at a tunnel borehole for weeks. The battery had to be changed and data had to be collected regularly. However there was not always time to maintain the loggers, due to the higher priority of other activities such as tunnel mapping and probe hole testing. Often it was difficult to access the loggers due to blasting, ventilation, mucking and other tunnel construction activities.

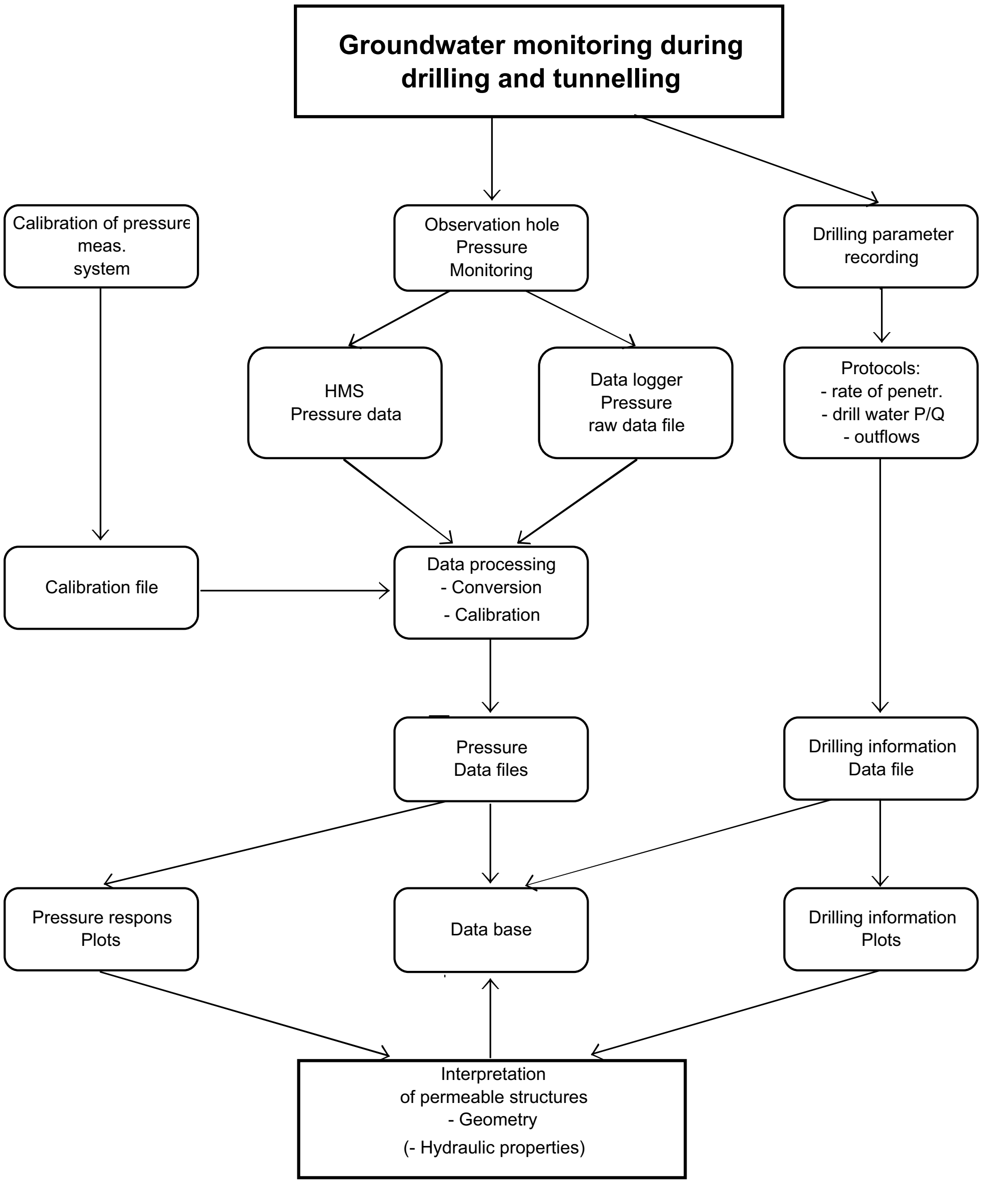


Figure 8-8. *Data flow chart of groundwater monitoring during drilling and tunnelling.*

The loggers were exposed to very severe conditions such as dripping saline water and moisture. During the first stages of the investigations the long-term pressure measurements failed on several occasions due to entry of moisture into the logger box. Later the cable connectors were replaced with ones of higher quality and the leakage stopped. If the loggers were located in a wet, dripping environment they were covered with plastic sheets, tarpaulins or some other kind of protection.

8.5 Interference tests

Purposes

During construction of the Äspö HRL, a large number of interference tests were performed in the tunnel boreholes. Observation boreholes included not only tunnel boreholes, but also surface boreholes on Äspö and surrounding areas. In this way large volumes of the bedrock could be hydraulically characterised.

The purposes of the tests were:

- To provide data permitting evaluation of the hydraulic properties of major water-bearing fracture zones.
- To determine the extent and geometry of major water-bearing features, and to allow geological and geophysical interpretation of major fracture zones.
- To produce drawdown and recovery data usable for the calibration of numerical groundwater models.

Instruments

The borehole to be tested, as well as the observation boreholes, was usually fitted with packers or with a casing valve before the test start. Mechanical packers (in probe holes and percussion-drilled investigation holes) or hydraulically operated packers (cored investigation holes) were used.

The groundwater head in all the surface boreholes and the groundwater pressure in permanently installed observation holes in the tunnel were monitored by the HMS, see Chapter 11. The groundwater pressure changes in the test hole and in the remaining observation boreholes were recorded by the data logger BORRE MDL version 2.2 and pressure transducers Druck PTX, the same as for other pressure build-up tests, see Section 8.2.2.

The valve arrangement was mounted on the packer, on the pipe string or in the casing for connection of the pressure transducer.

Methodology

Before testing, the pressure transducers were calibrated with the aid of the HMS system, see Chapter 11.3.2. The measuring instruments were installed and connected to the test borehole and to the observation boreholes, which were not connected on-line to the HMS. The observation holes were selected in advance, since they penetrated water-bearing features expected to be in hydraulic communication with the “pumpwell”, or were expected to provide relevant information on the rock volume monitored.

- The flow period started when the valve on the test hole was opened. Prior to opening the data loggers were started. The SEQ option (gradually increasing intervals) was chosen. The sampling intervals in the HMS-connected observation boreholes were generally lower than the standard sampling rate, at least at the beginning of the flow phase.
- The length of the flow period varied, depending on the aim of the test and on the size of the rock volume that was to be characterised, from 30 minutes up to 48 hours. The flow rate was measured manually by means of a stopwatch and graduated vessels. For the most permeable test sections a calibrated 1 m^3 tank was used.
- Shortly before the end of the flow period, all data loggers were again set to the SEQ option. If necessary the scanning rate of the HMS-connected boreholes was adjusted. The recovery phase started when the valve was shut. Pressure build-up was monitored for between 1 and 48 hours, depending on the length of the flow period.

Data processing and evaluation

A principal data flow chart for interference testing is shown in Figure 8-9.

Pressure data from the data loggers were processed as described in Chapter 8.2.2. Subsequently the pressure variation in the test borehole and the observation boreholes (including selected boreholes monitored by the HMS system) was plotted. Both the drawdown and the recovery phases were plotted on linear, logarithmic and semilogarithmic scales.

The logarithmic plots were used for evaluation by the type curve method, where drawdown or recovery data are matched against a type curve of the hydraulic response from the well and the formation and the hydraulic parameters are evaluated from the matching points of the diagrams.

In the semilogarithmic plots, a straight line is fitted to the data curve according to the semilog approximation of the Theis well function. The hydraulic properties of the formation are evaluated from the slope of the line and its position.

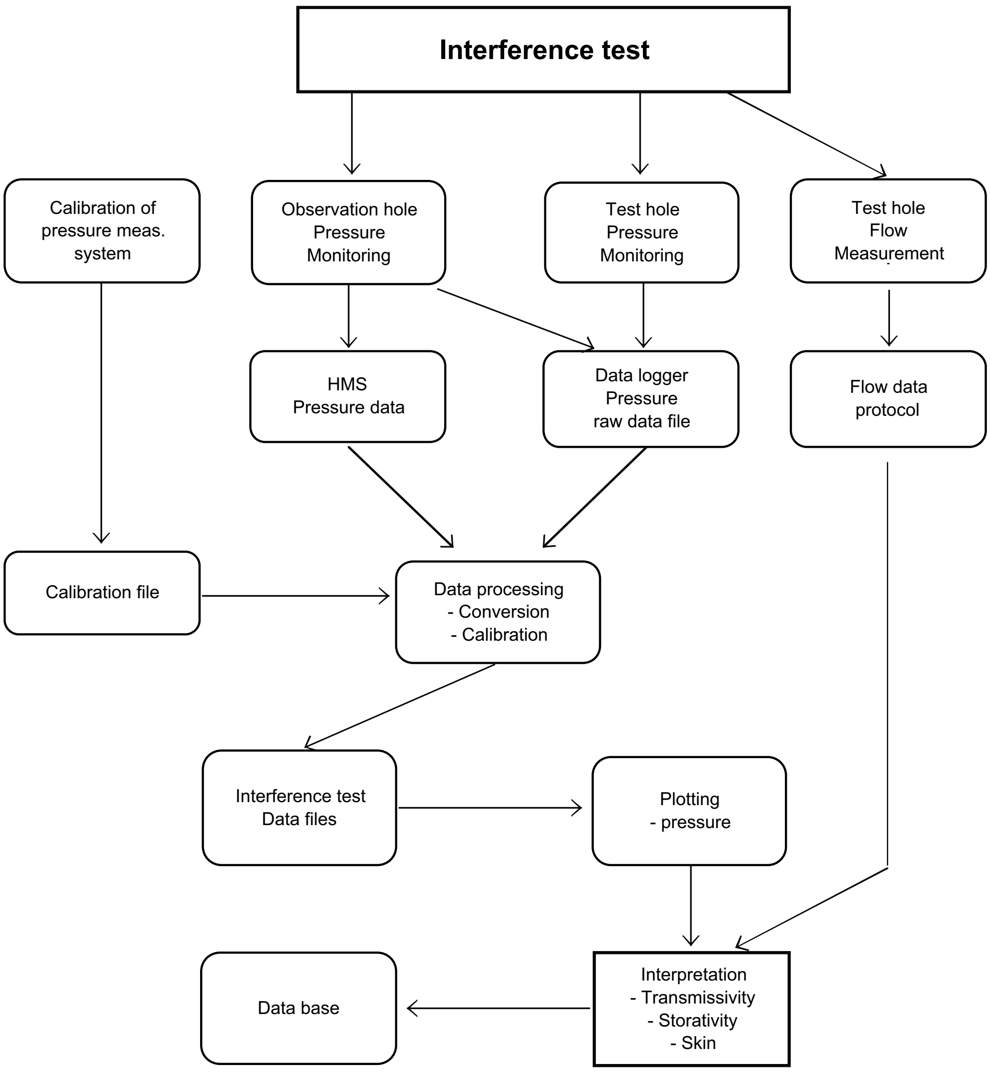


Figure 8-9. *Data flow chart of interference testing.*

The main parameters that can be estimated from type curves or by evaluating the straight line in semilogarithmic plots are:

- Transmissivity.
- Storativity.
- Skin.

Examples of results from the interference test are presented in /Forsmark and Stenberg, 1993/, /Olsson et al. 1994/ and /Olsson, 1994/. The evaluation theory is described by /Gustafson, 1986/, /Rhén et al. 1997c/, and /Kruseman and de Ridder, 1990/.

Accuracy

Errors associated with interference tests are similar to those associated with pressure build-up tests. The following sources of error can be identified:

- Manual water flow rate measurements.
- Groundwater pressure measurements.
- Evaluation of tests.

The error in determining flow rate has been estimated to be ± 5% in the interval 0.01–100 l/min. If larger flow rates were measured, the error could be estimated to be ± 10%.

The total error of the pressure measurements by data loggers and transducers is estimated to be ± 2–5 kPa, see Section 8.2.4. The error in small pressure changes is smaller. The accuracy of the HMS pressure measurements is described in Section 11.3.2.

The errors in the evaluation technique are more difficult to estimate. Of most importance is perhaps defining the correct groundwater model (flow regime) for the test.

Comments and recommendations

Interference tests presume that no other activity in the vicinity influences the pressures (piezometric levels), complicating the evaluation. During testing in the large fracture zones in the tunnel, e.g. NE-1 and EW-3, activities in the surface holes at Äspö, Ävrö and Laxemar could also cause unwanted responses.

The very high flow rates in some of the boreholes were measured with a graduated 1 m^3 water tank, which was not originally made for these measurements. A vessel, which was more adapted to the volume measurements, would have increased the accuracy of the high flow rate determinations.

The interference tests were often carried out on weekends when activity in the tunnel was low. Except for the high flow measurements, described above, the whole test could be managed and surveyed by one person. One person could handle several data loggers, thanks to the fact that the measurement intervals could be set in advance.

For interference tests, individual sampling rates were chosen for those borehole sections that were judged to show responses to the test. Measurements in the pumped or flowing (in the tunnel) borehole were generally made with an interval of one or a few seconds for the first minutes and then with a progressively increasing timeincrement. The initial time increment for observation sections was generally 5 minutes or less, depending on the distance to the pumped or flowing borehole. If about 20 measurements were performed for each log-cycle, this was judged to be adequate for evaluation of the test.

8.6 Tests with double packer system in investigation boreholes

Purpose

During the early stages of the construction phase of the Äspö HRL tunnel, the pressure build-up tests in investigation boreholes were performed as single packer tests. The packer and the end of the borehole delimited the measurement sections, but sometimes also by the casing with shut-off valve (see Section 8.3) and the end of the borehole.

Later on, hydraulic testing equipment for double packer sections was assembled. The use of a double packer system is a major advantage, particularly for long boreholes. This equipment included a downhole test valve and a sealing device at the casing, as described below. The double packer tests were normally carried out as pressure-controlled flow tests with subsequent pressure build-up. They were performed in borehole KA2511A, in the TBM pilot borehole and in boreholes drilled for the ZEDEX and SELECT programmes. In all, 225 tests were performed.

The purposes of the double packer tests were:

- To determine the hydraulic properties of the rock around the borehole, systematically along the borehole.
- To characterise certain conductive features penetrated by the respective boreholes, for detailed characterisation.

Instruments

The downhole equipment used consists of two inflatable packers, separated by perforated pipes, a mechanically operated valve, a pipe string and two pressure lines, see Figure 8-10. A bypass line equalises the groundwater pressure on both sides of the measurement section.

Figure 8-10. *Testing equipment used for double packer testing.*

The downhole valve is opened/closed by pushing/pulling the pipe string a distance of 70 mm. The two pressure lines are used for packer inflation and for pressure measurements in the test section, respectively.

The borehole pressure is trapped using a sealing device at the casing top. A spliced rubber cone with openings for the pipe string and for the two pressure lines seals off the borehole Figur 8-11. The device permitted the test tool to be moved in the borehole without depressurizing the entire borehole. A pressure transducer connected to the casing measured the groundwater pressure outside the test section. Depending on its magnitude, the flow was measured by one of three flow meters. The pressure in the test section was kept constant manually by means of regulation valves and a display showing the section pressure.

The main technical features of the system are:

- Double packers for 56 mm borehole:
  - rubber sealing length 1.0 m,
  - inflated by water,
  - operated by pressurised nitrogen gas over water in a pressure tank.
- Pipe string:
  - OD/ID = 33/21 mm, 3 m length, aluminium,
  - double-sealed, threaded pipe joints of stainless steel.
- Test valve:
  - mechanically operated by the pipe string,
  - opened/closed by pulling/pushing the pipe (piston) 70 mm,
  - OD = 44.5 mm,
  - the friction losses are 1.5 kPa at 5 l/min and 8.0 kPa at 20 l/min.
- Flow meters:
  - Q1: Micromotion model D6 (mass flow meter)
    flow range: 0–1 l/min,
  - Q2: Fisher and Porter, model COPAX DN 4
    flow range 0–8 l/min,
  - Q3: Fisher and Porter, model COPAX DN 15
    flow range 0–100 l/min.
- Pressure transducers:
  Druck PTX 510 abs
  - pressure range 0–5 MPa,
  - combined effect of linearity, hysteresis and repeatability is better than
    ± 0.3% of full scale (typical value is ± 0.15% of full scale)

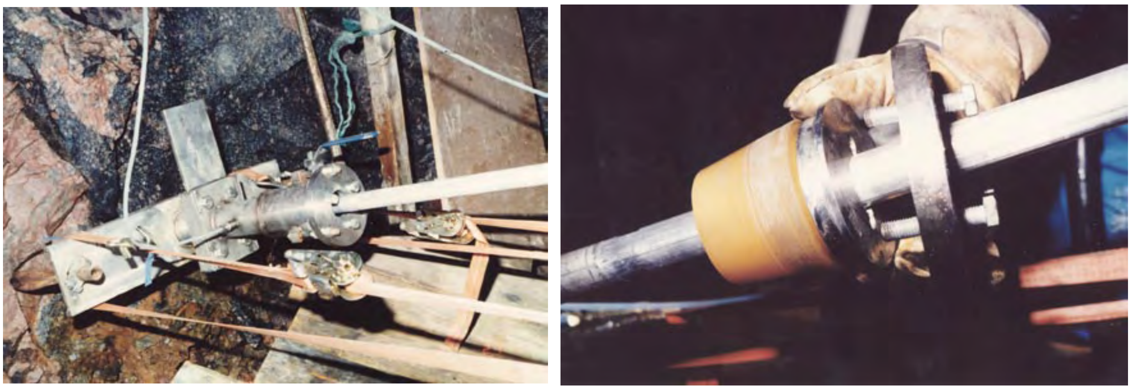


Figure 8-11. *The borehole sealing device for hydraulic tests mounted to the casing (showed in Figure 5-8). A rubber cone seals around the pipe and tubes.*

- Data loggers:
  Borre MDL version 2.2, see Chapter 8.2.2
  – two data loggers were used, one for pressure monitoring and one for flow rate monitoring.
- Pressure tubes:
  – tecalan OD/ID = 6/4 mm.

Methodology

Before testing, the borehole casing valve or borehole sealing arrangement described in Section 5.3.4 and 5.4 was replaced with the sealing unit for hydraulic tests and the testing tool was assembled and put into the hole. The sealing rubber cone (in two halves) was put in place to seal around the pipe string and the pressure lines. While the test tool was being moved in the borehole, the sealing cone had to be lightly squeezed (to allow a small water leakage for friction reduction), but at the test positions the seal was squeezed harder.

The pressure transducers were calibrated using the reference system of the Hydro Monitoring System, see Section 8.2.2 and Figure 11-3. The flow meters were calibrated by comparing flow meter values with flow rates measured with graduated vessels and a stopwatch.

The actual hydraulic test procedure varied to some extent, but a standard test was carried out as follows:

- The packers were inflated by water pressure to approximately 1.7 MPa above the expected hydrostatic pressure in the borehole. The pressure was regulated by nitrogen gas. The packer expansion phase lasted 20 minutes in order to minimize creep effects.
- The flow phase was started by opening the downhole test valve (i.e. by pulling the pipe string 70 mm). The test section pressure was kept constant manually at about 200 kPa below the initial pressure, using the regulation valves on the flow meter unit. Shortly before the start of the flow period, logarithmic scanning of the two data loggers was selected (the SEQ option, see Section 8.2.2). The flow period usually lasted 45 minutes.
- The flow period was ended by closing the downhole test valve and the recovery phase started. Monitoring of the flow rate was stopped, while pressure monitoring continued for another 30 minutes (SEQ scanning).
- After completion of the test, the packers were deflated and the test tool was moved to the next test section.

During the entire test, manual readings of the pressure gauges and manually measured flow rates were noted in the field record together with borehole geometry data, packer positions etc.

Data processing and evaluation

A schematic data flow chart is shown in Figure 8-12.

Pressure data was retrieved, converted and plotted in the same way as described in Section 8.2.2. The flow rate was plotted in linear graphs and in graphs with the inverse of the flow rate on the y-axis and logarithmic time on the x-axis. The primary evaluated parameters were specific capacity and transmissivity. The evaluation method was the same as that described in Section 8.2.2. But transmissivity was also evaluated by an alternative method using the inverse flow plot, see /Olsson, 1994/ and /Kruseman and de Ridder, 1990/.

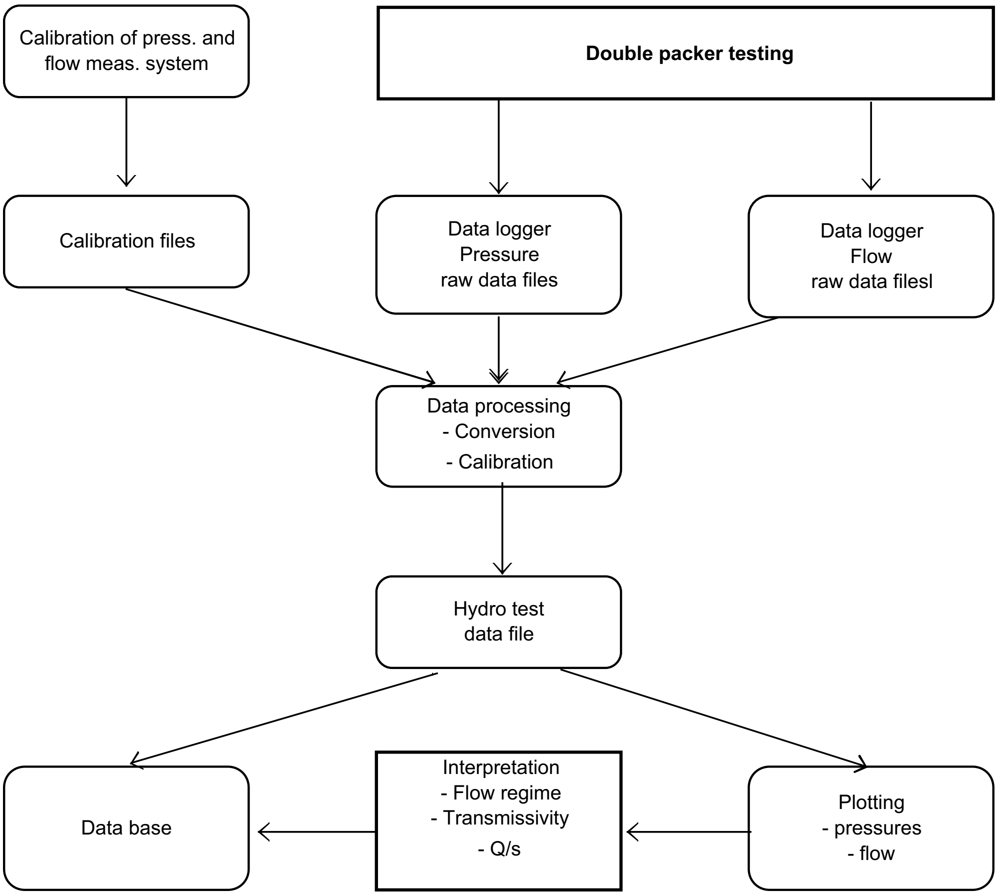


Figure 8-12. *Data flow chart of double packer testing.*

The results of double packer testing in investigation boreholes are described by /Forsmark and Stenberg, 1994/ and /Olsson, 1994/.

Accuracy

The sources of error associated with double packer tests are associated with pressure build-up tests in the probe holes (Section 8.2.2):

- Groundwater pressure measurements.
- Flow rate measurements.
- Evaluation of tests.

The total error in absolute pressure measurements has been estimated to be ± 2–5 kPa. The error in small pressure changes is smaller.

The accuracy of the flow rate depends on the flow meter used and the flow interval:

- Flow rates measured by flow meter Q1 with an accuracy ± 0.4% of actual flow for rates ± 1×10^{-4} l/min.
- Flow rates measured by flow meter Q2 with an accuracy ± 1% of actual flow for rates ≥ 0.8 l/min and an accuracy ± 0.1% of full scale for rates ≤ 0.8 l/min.

- Flow rates measured by flow meter Q3 with an
  accuracy ± 1% of actual flow for rates ≥ 10 l/min and an
  accuracy ± 0.1% of full scale for rates ≤ 10 l/min.

The errors involved in the evaluation technique are more difficult to estimate. Of great influence is likely identifying the correct flow regime for the test. The background water pressure of the rock formation also has an impact on the evaluation. In the low-permeable sections, the pressure stabilisation period was too short to measure that pressure.

Comments and recommendations

The boreholes tested with the double packer system were often several metres above the tunnel floor. A platform had to be built. It was almost impossible to determine all the equipment needed for the test in a systematic way. Because of this it was difficult for the operators to control all the instruments. The high air humidity and dripping water made it necessary to encapsulate all the instruments very carefully. In spite of this, measurement failures occurred due to moisture.

Due to moisture problems, the development of a new Underground Hydraulic Test system (UHT-1) was initiated. The system was built and tested during 1997. In UHT-1, all the vital instruments are assembled in a container and fixed on the container walls or arranged on shelves. This facilitates to keep sensitive equipment dry.

During some of the tests, a leak was observed at the rubber cone. This caused a pressure drop in the borehole. The time interval between packer inflation and opening of the test valve was in general too short to evaluate the initial (undisturbed) pressure in the vicinity of the borehole. The duration of this period is recommended to be prolonged and specifically determined for each test section, depending mainly on the transmissivity.

8.7 Flow meter logging method

General

Flow meter logging is a borehole logging method, which was originally used to determine the vertical flow along a borehole in order to detect in-flowing and out-flowing sections. The method presupposes a significant difference between the groundwater heads in the aquifers penetrated by the borehole. During the pre-investigation phase of the Äspö HRL, this method was used in combination with a pumping test in the borehole, which resulted in a flow log showing major water-conducting features along the borehole /Almén and Zellman, 1991/. A significant differential pressure for flow meter logging is also reached in underground boreholes drilled from a location below the formation's hydraulic pressure head, e.g. the groundwater flows out from the open borehole.

Purposes

The purpose of the flow meter logging were:

- To indicate groundwater-conducting fractures and fracture zones.
- To estimate the transmissivities of these features.

Flow meter logging was used quite frequently in investigation boreholes. The logging was carried out when the drilling was completed or at breaks during the drilling, sometimes before cement grouting for borehole stabilization, see Section 5.3.4.

Instruments

The equipment set-up for flow meter logging is in principle as follows:

- Flow meter probe.
- Surface control unit.
- Cable, winch and measuring wheel.
- Push rods, when needed (aluminium or glass-fibre).

Two different flow meters have been used in the Äspö HRL project, the MLS flow meter probe and the UCM flow meter probe, see Figure 8-13.

In the MLS flow meter, the rotational speed of a propeller is measured by means of a pulse counter. The rotational speed is then converted into flow velocity in the borehole. The flow rate for the actual borehole diameter can then be calculated. The flow specification for the MLS flow meter is presented in Table 8-1. The probe diameter is 44 mm.

In the UCM flow meter, the direction and magnitude of the water flow affects the travel times for acoustic waves (the Doppler effect). The probe diameter is 54 mm. The flow specifications for the UCM flow meter are presented in Table 8-1.

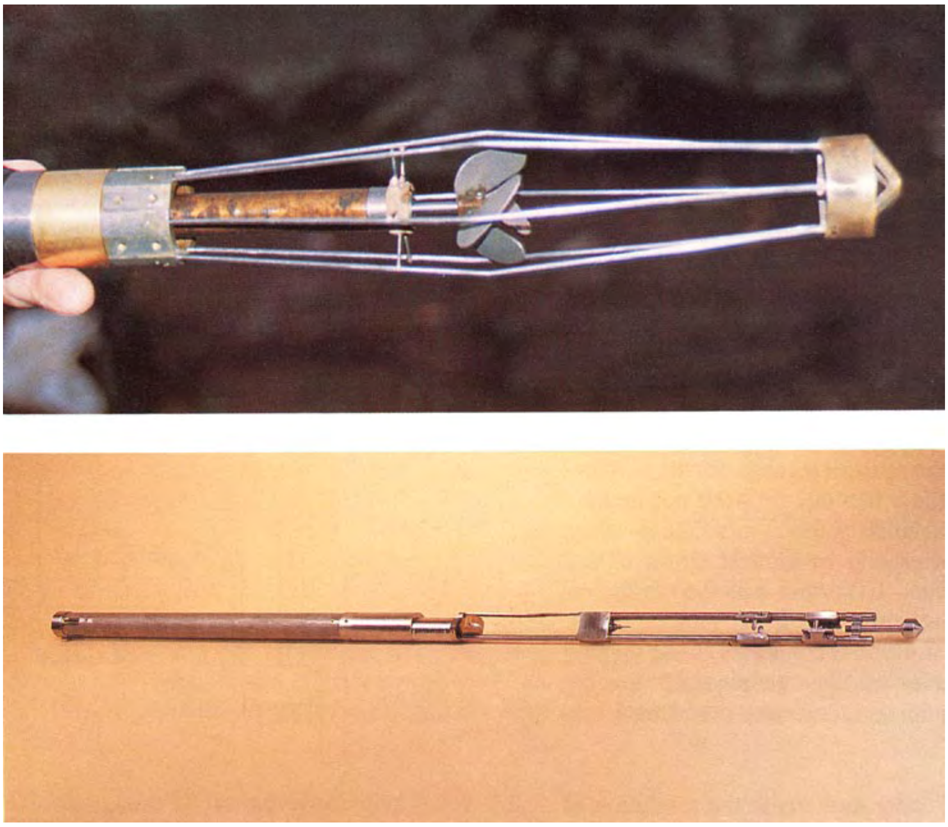


Figure 8-13. *The MLS flow meter probe (above) and the UCM flow meter probe (below).*

Table 8-1. Flow measurement specifications for the MLS and UCM flow meters.

	Velocity (m/s)	Flow in 56 mm (l/min)	Flow in 76 mm (l/min)
MLS probe			
Range	0.015–5	5–900	10–1,600
Resolution	0.015	2% of reading	2% of reading
Accuracy		10% of reading	10% of reading
UCM Probe			
Range	0.001–3	0.15–450	0.3–900
Resolution	0.001	2% of reading	4% of reading
Accuracy		5% of reading	10% of reading

The UCM flow meter also measures the resistivity and temperature of the borehole fluid. The measuring ranges for fluid resistivity are 0.13–15 ohmm (when induction conductivity cell is used) and 5–500 Ohmm (when resistivity sensor is used). The measuring range for temperature is –5°C to +45°C.

For measurements in boreholes, from the surface, the MLS and the UCM flow meters was used down to 1,500 m depths. The maximum logging depth (length) in horizontal boreholes depends mostly on type of push rod, whether a rig is available, and the amount of out-flowing water from the borehole, see next section.

Methodology

The surface unit and the logging tools are placed near the borehole mouth. For logging in underground (normally (sub)horizontal) boreholes, the tools must be pushed into the borehole. Connecting a pushing rod onto the logging tool does this. The rod is then pushed into the borehole and readings are taken intermittently every metre. More closely spaced measurements are recommended over major inflow sections. Every 2nd metres a new rod is connected to the rod string, until the measurement of the entire borehole is completed.

In heavily flowing boreholes, problems may be encountered pushing the tool into the borehole. The magnitude of the problems depends on which tool is used (easier with the smaller MLS) and of course on the borehole length. In general, outflows greater than 200 litres/min created problems.

The distance measurements along the borehole are controlled with the pushing rods.

Basic calibration of the flow meters is performed in a calibration device at the workshop. The probe is put in a pipe of the same diameter as the boreholes (56 mm and/or 76 mm) and water is pumped through the pipe at different flow rates and directions. For calibration of small flow rates, it is important that water with the same temperature as the surrounding air is pumped through the pipe in a closed system, to avoid thermally induced interference.

A field check is always performed after logging. The maximum flow rate is checked by installing the probe at one metre below the end of the casing. This gives the maximum flow in the borehole. The 0 level is checked when the probe is at the bottom of the borehole.

Data processing and evaluation

A schematic data flow chart is shown in Figure 8-14.

Raw data recorded as counts/s or mV (for MLS and UCM flow meters, respectively) are converted after calibration into litres/min. For the UCM flow meter, mV values for fluid resistivity and temperature were also converted to Ohmm and °C, respectively. The converted and calibrated data files from the logging are plotted as "geophysical logging graphs", see Figure 8-15.

While the primary purpose of flow meter logging was to indicate groundwater-conducting features, a secondary purpose was to get a rough estimate of the transmissivity of some of these conductors. The T-value was in principle determined by using the flow difference over the conductor and the evaluated transmissivity and outflow rate from the pressure drawdown for the entire borehole as input parameters to:

$T_i = Q_i \times T_{tot}/Q_{tot}$

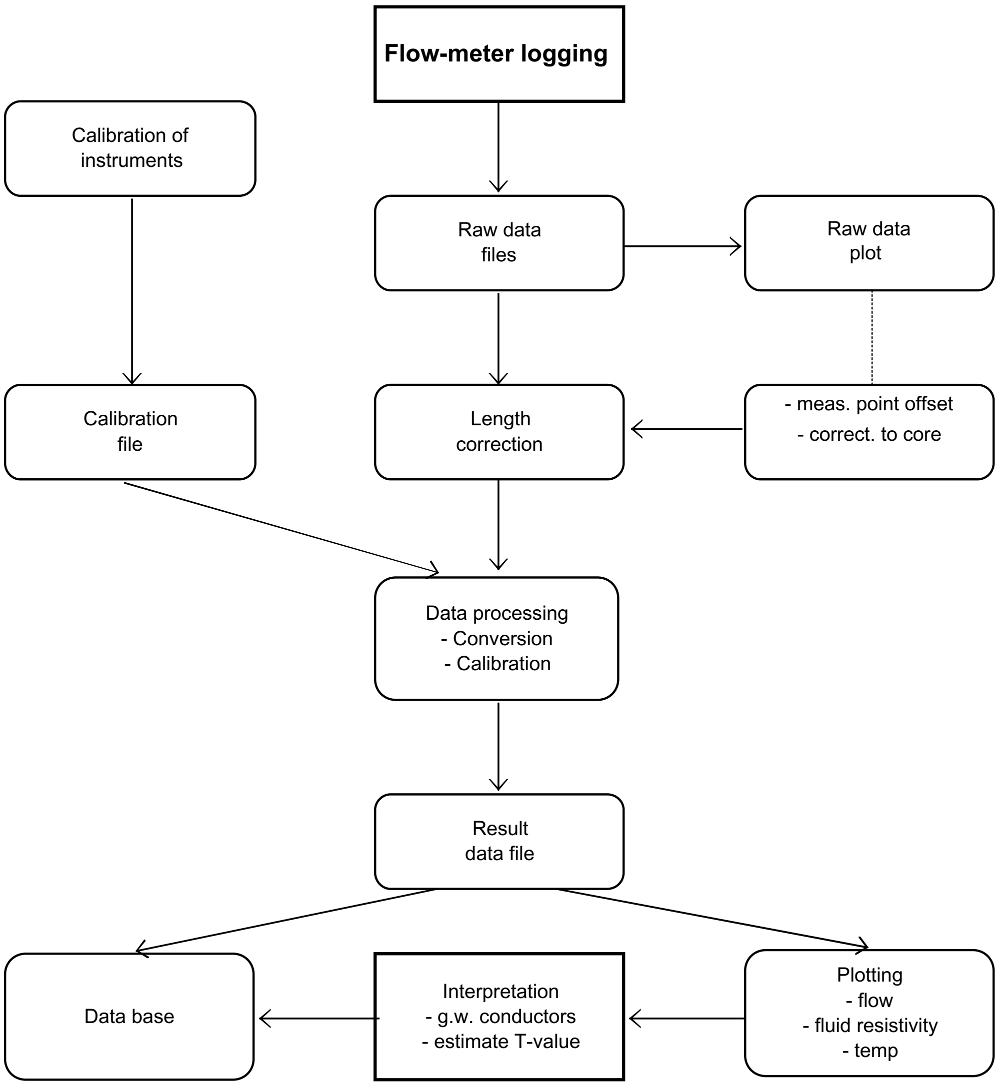


Figure 8-14. *Data flow chart of flow meter logging.*

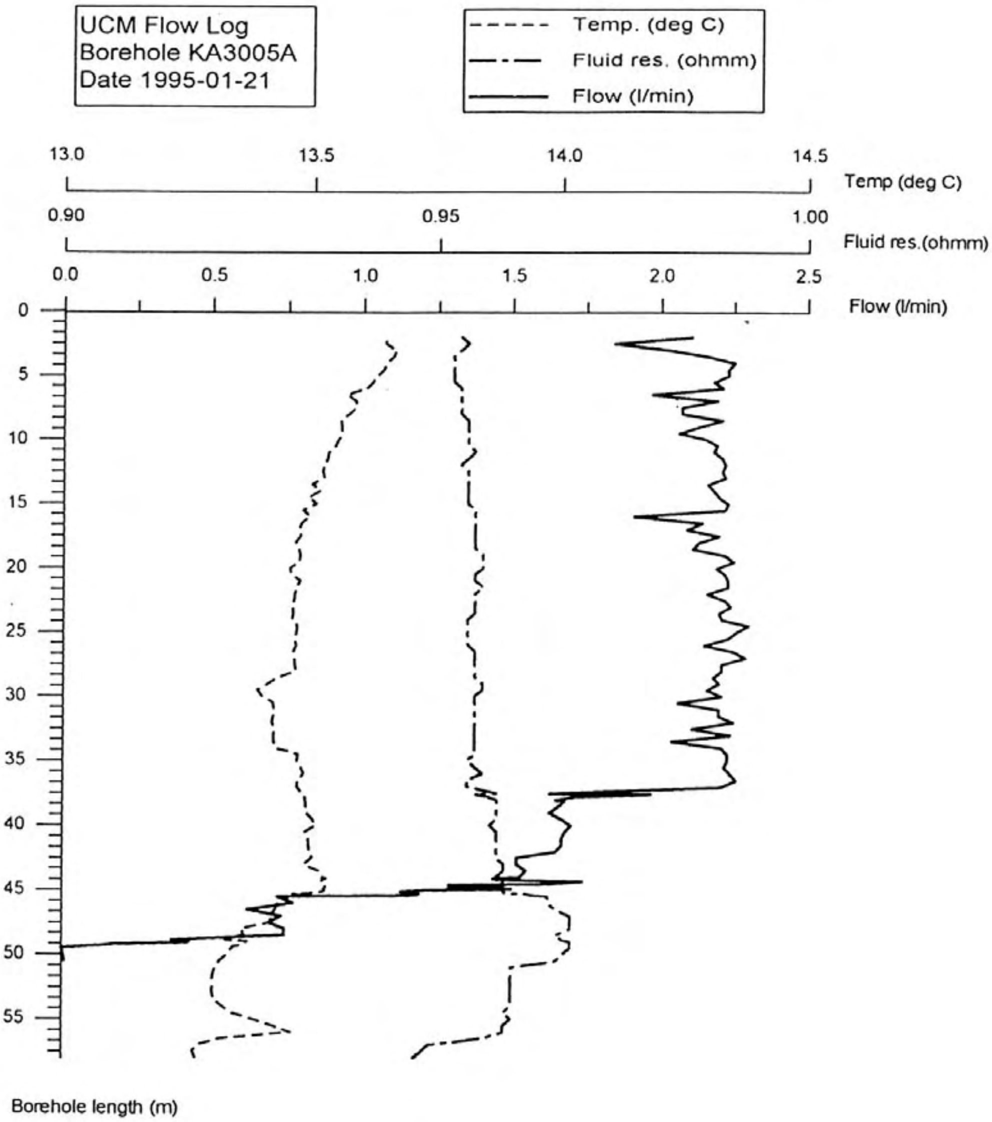


__Figure 8-15.__ Flow meter survey with UCM in borehole KA3005A.

Accuracy

The sources of errors for flow meter logging are related to:

- Errors in length measurements.
- Errors in flow measurements.
- Errors in data evaluation.

The accuracy of the depth determinations is estimated to be better than ± 1%. The flow accuracy for the MLS and UCM probes are given in Table 8-1.

A varying diameter of the borehole is one of the parameters affecting the flow determination for a conductor. However, in cored holes in hard rock, diameter changes of importance only occur locally in fractured sections, and these local flow artefacts can be neglected, as the flows are determined from measurements on both sides of such zones. Another source of error is related to inaccurate centring of the probe. Particularly in 76 mm boreholes, centring devices must be used. The determination of transmissivity may be affected by an uneven pressure drawdown along the borehole. At high flows in particular, the friction losses in the borehole and over the probe will change as the flow decreases below the highly conducting features. These types of errors have not been quantified, and the calculated values have only been used as estimates.

Comments and recommendations

The flow meter logging method gives good indications of water-bearing features that intersect the borehole. Flow meter logging is a comparatively fast method, however, when more detailed characterisation and more accurate transmissivity values are needed, double packer tests have to be performed.

The flow meter method tends to mainly monitor fractures with a higher inflow while fractures with a lower inflow often get overprinted. During high inflow it was often difficult to push the probe into the open hole with the rods. Therefore, the recommended maximum flow rates for the UCM flow meter are 100 l/min and 200 l/min for 56 mm and 76 mm boreholes, respectively. For larger flow rates, the MLS flow meter should be used.

In long holes, especially from the surface, with increasing salinity towards depth, the dense saline water may just rise some distance up along the borehole during drawdown (pumping), thus causing a no-flow section of the borehole, which may not reflect true conditions.

8.8 Groundwater flow measurements

General

The dilution method permits local measurement of groundwater flow through particular borehole sections under natural (undisturbed) conditions during a pumping test or during the excavation of tunnels and shafts. This method is used to estimate groundwater flow in-situ, in the formation, in the fractures and in the fracture zones.

If flow rates measured during a pumping test are compared with natural gradient conditions, the data set may provide indications of how features are connected. Increased flow rate during pumping verifies the existence of flow paths connecting the measured section with the pumping borehole and provides additional information on the flow distribution in the hydraulic conductors that are in contact with the pumping borehole. Such measurements are useful for calibration of numerical groundwater flow models.

Drainage during the excavation of the underground facility will cause changes in the groundwater flow system /Ittner and Gustafsson, 1995/. These changes were also predicted for in a number of borehole sections /Gustafson et al. 1991/.

Purpose

The purpose of dilution measurements was:

- To measure the groundwater flow through borehole sections in order to compare with predictions.

A total of 64 groundwater flow measurements were performed using the dilution method during the pre-investigation phase, of which 22 were combined with tracer injection in the LPT-2 combined pumping and tracer test. During the construction phase, 23 dilution measurements combined with water sampling were carried out in order to monitor changes in water chemistry and flow pattern due to the drainage by the tunnels.

Instruments

The dilution measurements were carried out in the surface boreholes that were used for groundwater monitoring. The multipacker system allows one or two of the packed-off borehole sections to be equipped for dilution measurements and sampling of groundwater for chemical analyses, see Figure 11-2 and further description in Section 11.2.2. Out of 72 borehole sections on Äspö, 22 were equipped for dilution measurements. The dilution measurements were carried out in test sections with lengths between 7 and 145 m, and at depths ranging from 50 to 850 m.

Methodology

The basic principle of the method is that the groundwater in the borehole test section, straddled by packers, is labelled with a tracer, which will be diluted by the formation groundwater flowing through the test section. The dilution rate of the tracer is proportional to the groundwater flow rate through the borehole section /Gustafsson and Andersson, 1991/.

The measurement in surface boreholes begins by removing the small packer and pressure transducer used for groundwater level monitoring. A package containing a circulation pump (Grundfos MP1 or Pleuger Mini-Unterwasserpumpe), a packer and a filter is installed in the same standpipe. On some measurement occasions a pressure transducer was also installed, connected to the borehole section via a thin tube through the small standpipe packer, see Figure 11-2. A tracer test unit is connected to the pump outlet tube and the tube emerging from the bottom of the test section, see Figure 8-16. The tracer test unit consists of a flow meter and shut-off valves for control and manual readings of the circulation flow, a pressure gauge to check the pressure and connections for the tracer injection device and a water sampler /Ittner, 1994/.

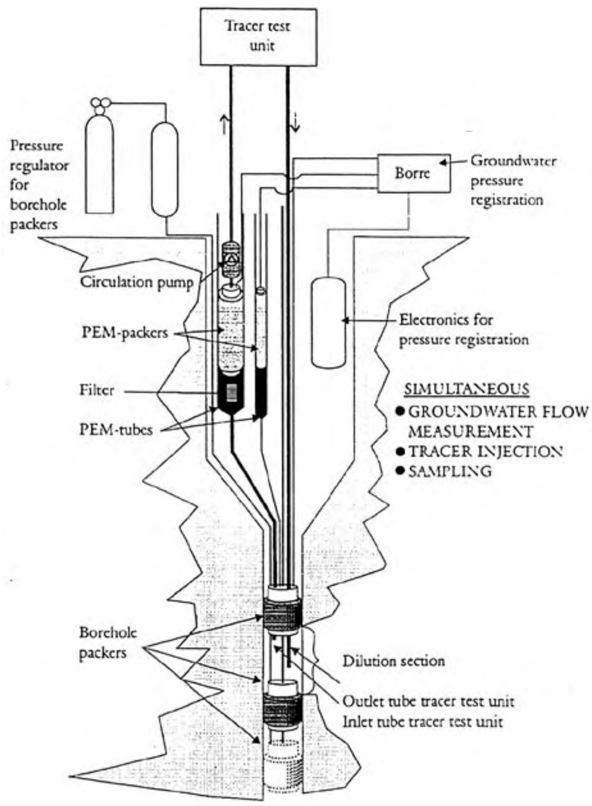


Figure 8-16. *Principal outline of borehole equipment for dilution measurements.*

The next step is to start the circulation pump and inject a small amount of concentrated tracer solution into the circulating water. In order to maintain complete mixing of tracer in the section, the circulation pump is run continuously during the entire test. The sampler is programmed to take about 40 small-volume samples (a few millilitres) during 4–5 days of dilution measurement. The total measurement time depends on the flow rate, where a high flow rate results in a fast dilution and hence shorter measurements time.

The samples are brought into the laboratory and analysed for tracer concentration. The fluorescent dye tracer uranine was used for all dilution measurements during the construction phase.

Data processing and evaluation

A schematic data flow chart is shown in Figure 8-17.

The primary data from a dilution measurement consist of tracer concentration, sampling time and water volume in the test section, see Figure 8-16. The groundwater flow rate through the borehole section is calculated from the equation of continuity for the dilution of a homogeneously distributed tracer solution in a constant volume at steady-state groundwater flow /Halevy et al. 1967, Gustafsson and Andersson, 1991/.

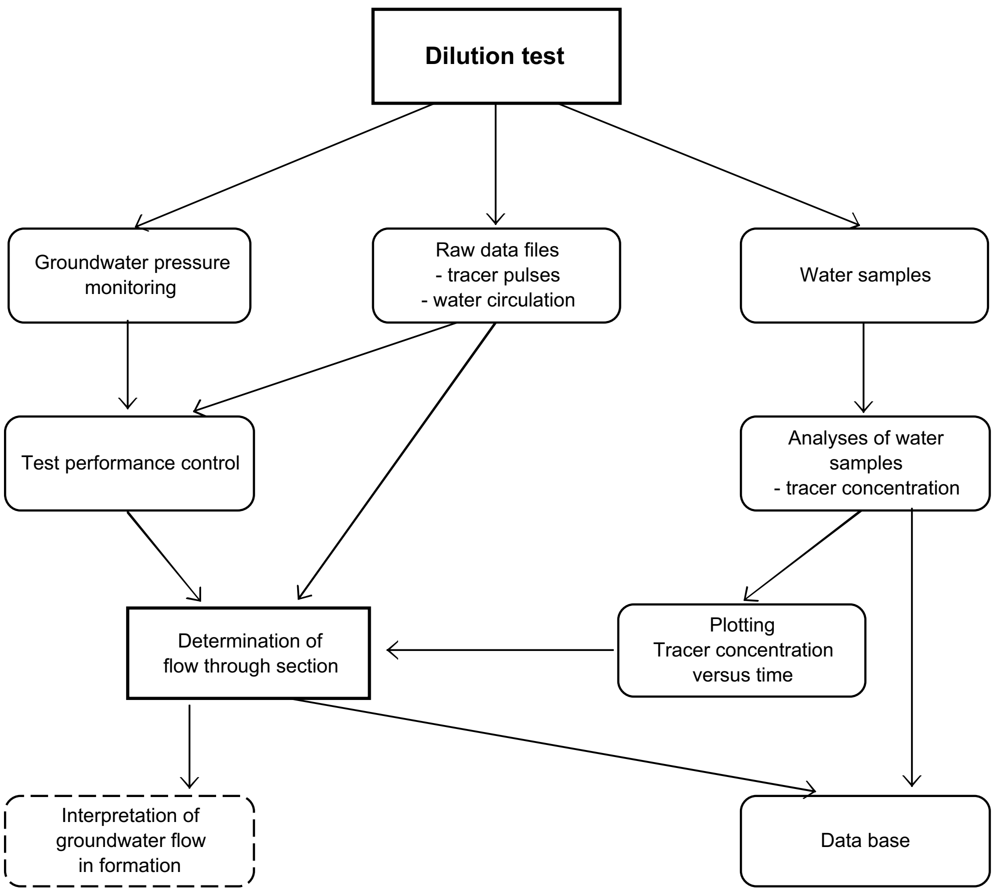


Figure 8-17. *Data flow chart of dilution tests.*

The groundwater flow in the rock formation is determined from the calculated groundwater flow rate through the borehole test section and an assumption concerning the flow field around the test section. The transformation of the flow through the borehole to a flow in the rock mass is dependent on borehole skin, borehole diameter, test section length and hydraulic gradient direction in relation to the borehole /Gustafsson, 1986; Rhén, 1995b; Rhén et al. 1997a,b/. The transformation can also further be complicated if more than one hydraulic structure intersects the test section.

Measurement range

The lowest limit of detection is set by the ratio between the dilution caused by molecular diffusion of the tracer into the fractured/porous aquifer and the dilution caused by by advective groundwater flow through the test section /Institut für Radiohydrometrie, 1969/ and /Gustafsson, 1986/. In practice, the lowest measurable flow is limited by the time available. Within the Äspö monitoring programme, a flow lower than 0.3 ml/min has only been observed once.

The highest measurable flow is limited by the ability to keep the tracer completely mixed in the borehole test section. In general, the groundwater flow rate should not exceed 1/10 of the mixing flow rate, possibly up to 1/2. This ratio is determined by several factors, such as the length and volume of the test section, how the fractures are distributed within the section and how circulation tube inlets and outlets are located. With the Äspö Multipacker System, the highest measurable groundwater flow rate has been about 300 ml/min. Out of 87 measurements carried out from August 1989 to May 1994, the measurement limit was exceeded on three occasions /Ittner, 1994/.

Accuracy

The following sources of error are associated with the dilution measurements:

- Calculation of test section volume.
- Determination of tracer concentration.
- Sample volumes.
- Incomplete mixing (at high groundwater flow rates).

The test section volume can be calculated with an accuracy of about ± 2%. The accuracy of tracer concentration is estimated to be within ± 5%, and is dependent on the accuracy of the tracer analyses and the degree of tracer sorption on borehole walls and/or on suspended particles in the groundwater. The samples taken to determine the decrease of tracer concentration will also contribute to the total flow through the test section. Therefore, the volume of water and tracer mass removed by the samples is measured in each dilution measurement and, if necessary, taken into account when the flow is calculated. The error introduced by neglecting the sample volumes is in most cases less than 1%. The error due to incomplete mixing has hitherto not been analysed in detail. Laboratory tests in a plastic tube borehole (GEOSIGMA, unpublished material) indicate that this error is very small, less than ± 1% at a groundwater flow rate that is 1/6 of the mixing flow rate.

Altogether, the error in determining the flow rate through the borehole test section is estimated to be below ± 10%.

The groundwater flow in the rock formation can probably be determined with an accuracy of approximately ± 75%, considering the disturbance of the flow field around the borehole test section. In borehole sections of considerable length, with more than one hydraulic feature intersecting the test section, vertical (along borehole) currents may be encountered. These will yield a more uncertain transformation of the flow rate through the borehole section to the groundwater flow in the rock formation /Gustafsson, 1986; Rhén, 1995b; Rhén et al. 1997a,b/. This error cannot be quantified in general terms; it must be considered and determined in each individual case.

Comments and recommendations

During the pre-investigation phase, when the Multipacker Systems were newly installed and the groundwater table was at moderate depth, the dilution measurements were easy to perform and worked well. Only three measurements failed due to leakage in downhole tube fittings. At the end of the construction phase, with the Multipacker System being installed for more than five years, and with drawdowns of more than 50 m in some boreholes, dilution measurements became problematic. The PEMtube standpipes were flattened, due to low water levels inside the pipes, which made it difficult or impossible to lift out the pressure transducer device from the PEMtube standpipe and install the circulation pump package. Dilution measurements were also rendered impossible by damaged or clogged downhole circulation tubes and by large drawdowns exceeding the circulation pump suction head. During the construction phase, four measurements failed and six measurements could not be performed due to the limitations of the downhole equipment and circulation pumps used.

It is recommended that the technical devices is improved, especially the PEM tube standpipe, to make it possible to install the pump package in the PEM tube and conduct dilution measurements even at large drawdowns.

The advantage of the dilution method is that at present it is the only method that directly measures the groundwater flow rate through boreholes with permanently installed packers, and in borehole test sections exceeding 2 m in length. The measured flow rate through the test section is also relatively exact (an error of ± 10%).

The limitation of the Multipacker System is that large groundwater flows through the borehole section (more than 300 ml/min) are not possible to measure with the pumps, tubes and test section length used so far.

In fractured media, hydraulic gradient disturbances rapidly alter the groundwater flow rate even at large distances (e.g. Äspö HRL). Potential hydraulic disturbances must therefore be kept under control, and dilution measurements should not be conducted on sites with uncontrolled hydraulic disturbances.

The transformation of the groundwater flow rate through the test section to a flow rate in the rock formation should be further investigated, considering different skin factors and hydraulic gradient directions in relation to the borehole /Rhén et al. 1997a,b/.

9 Hydrochemical borehole investigations

9.1 General

An extensive geohydrochemical investigation programme was carried out during the construction phase of the Äspö HRL. The general purpose of the groundwater sampling and chemical characterisation was (see also Table 13-1):

- To provide data for comparison with predictions and to obtain additional chemistry information for detailed modelling and a deeper understanding of groundwater origin and evolution.

With reference to Section 2.2 and Figure 1-5, this chapter will describe and discuss all methods used for water sampling and chemical analysis within the baseline characterisation programme (excluding sampling from tunnel walls), the monitoring programme, and special investigations, see also Figure 9-1. Due to the similarity of the procedure used for sampling of "first strike water" for chemical analysis from probe holes and from investigation holes, these are presented in one section and follow-up sampling or monitoring sampling in a second section, as follows:

- Documentation sampling programme
  - probe holes,
  - investigation holes,
  - tunnel wall leakage points (see Section 4.7).
- Monitoring sampling programme
  - probe holes,
  - investigation holes,
  - inflowing water (measuring dams).
- Special sampling programme

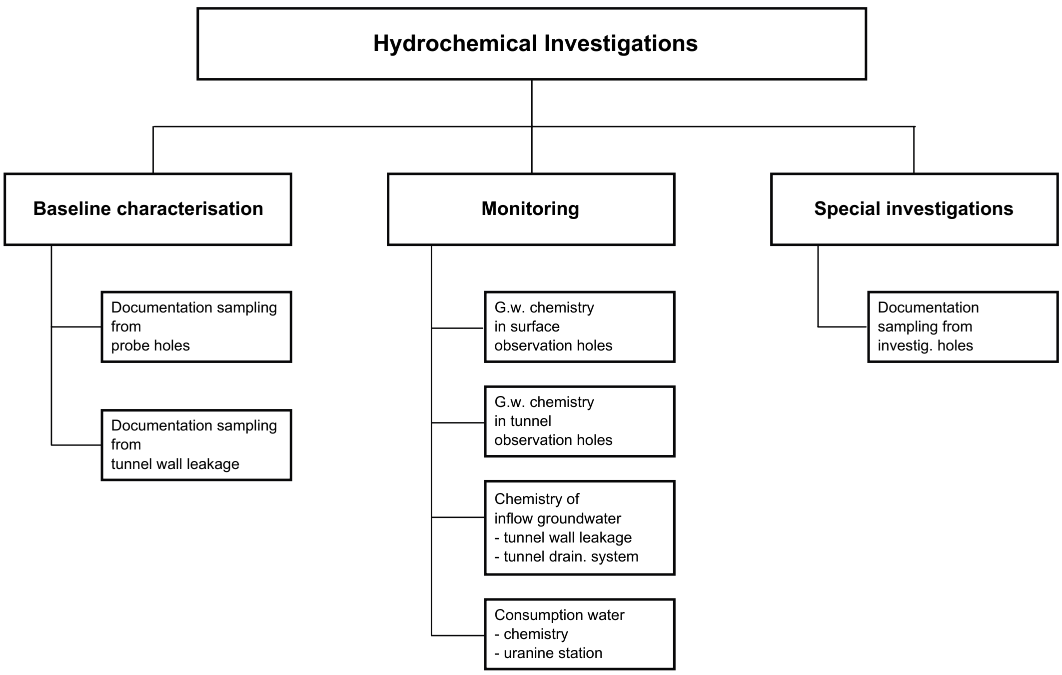


Figure 9-1. Hydrochemical investigations performed during the Äspö HRL construction phase.

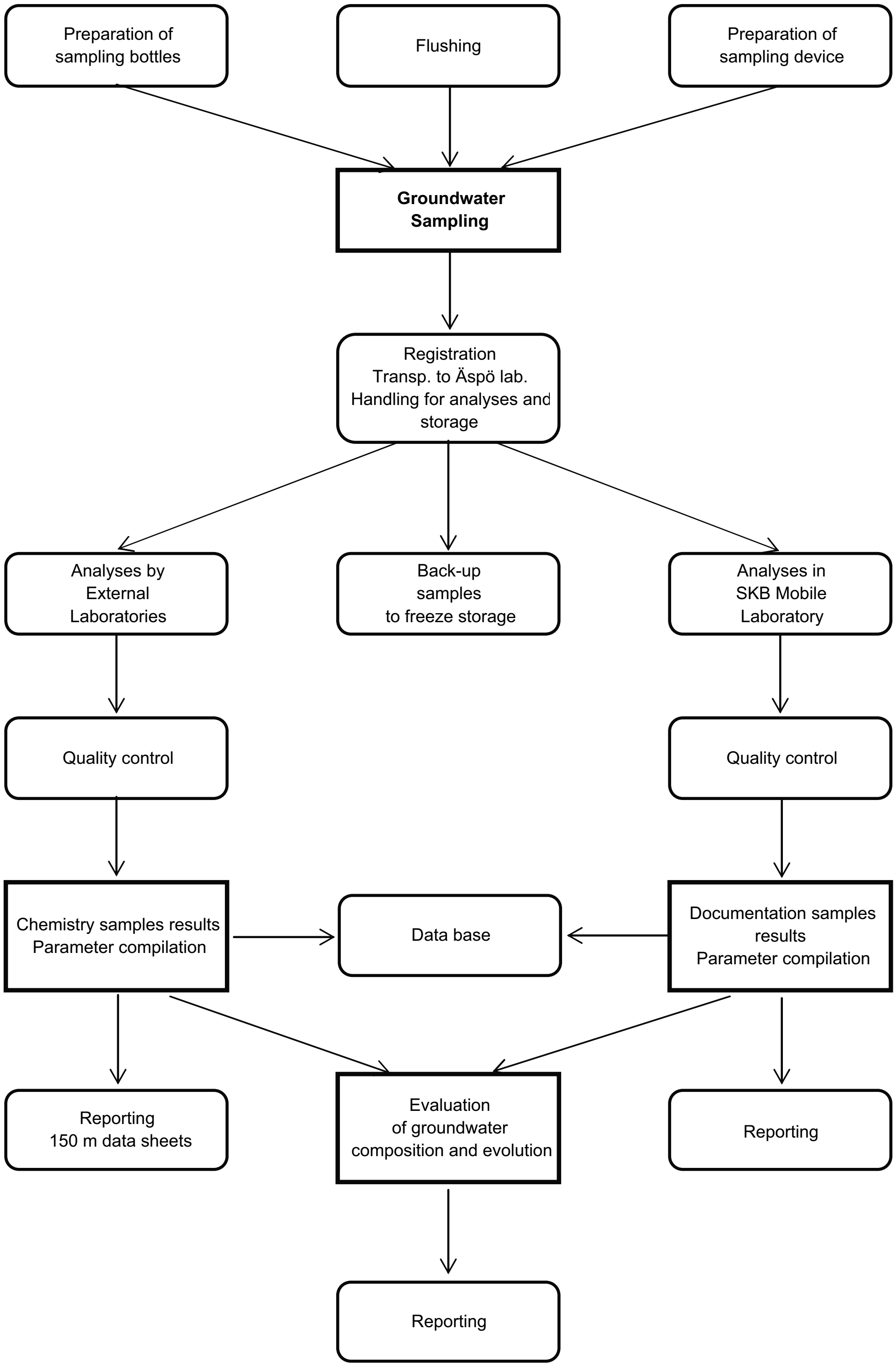


Figure 9-2. *Principle flow chart for groundwater sampling and analysis.*

The methods and routines used for sampling and analysis were specially designed for each programme, according to its specific purpose. Two main methods were used in the documentation programme, yielding documentation samples, and in the monitoring sampling programme, yielding chemistry samples. These are described in greater detail in Sections 9.2 and 9.3.

The sampling programmes for the different special projects differ from the documentation sampling and the groundwater chemistry monitoring, as well as from each other, depending on the purpose/aim/object of each project. Generally these sampling programmes were more extensive and involved more so-called special analyses and special sampling (see Table 9-3). These sampling programmes will be only briefly mentioned in Section 9.4.

Sampling of groundwater was performed according to strict routines involving preparation of sample bottles, sampling itself, handling of samples and back-up samples, and storage according to type of sampling programme, see Figure 9-2. The analysed parameters and the procedures for the chemical analyses of the water samples were also specified for the two sample types.

During the Äspö project, SKB introduced an even more standardized structure for water sampling and chemical analysis. Five different chemistry classes or levels (where a higher number indicates more extensive sampling and analysis) were defined, of which Class 2 corresponds to documentation samples and Class 4 to chemistry samples, see Section 9.5.

In order to trace contamination of the samples from drilling, hydraulic testing etc, all water consumed in the tunnel was tagged with the fluorescent colour tracer Uranine. A brief description of the Uranine Station used for tagging the water is presented in Section 9.8.

9.2 Sampling for the documentation programme

9.2.1 Purpose

Most of the documentation samples were taken from the regularly drilled probe boreholes, just after they were drilled and in conjunction with tunnel wall mapping and pressure build-up tests in these boreholes. Documentation samples were also taken during or just after drilling of investigation boreholes.

The purpose of the documentation programme was:

- To collect "first strike water" to provide a first set of chemical parameter data to be used as a reference.

A limited number of parameters were determined: pH, electrical conductivity, chloride and alkalinity, see methods in Section 9.7.3 and Table 9-3.

9.2.2 Equipment

Sampling of new boreholes did not require any borehole specific equipment. Due to the higher pressure of the groundwater in the borehole in relation to the tunnel, the water flowed out of the hole (except where watertight conditions existed), hence no pumping equipment was needed. In probe holes (and short investigation holes), the samples were normally taken from packed-off borehole sections (or from "casing with valve" installations) and in conjunction with pressure build-up tests (after packer installation and before pressure build-up), see Chapter 8. In longer investigation boreholes, documentation samples were most often taken during drilling, using drill pipe and sometimes packer equipment. Only a few samples were taken from open boreholes.

The documentation sampling did not require any special bottles or preparations. However, special boxes were made to facilitate handling of samples. They were prepared with empty bottles and brought to the sampling site. A graduated cylinder and a stopwatch were used for flow rate measurement.

9.2.3 Methodology

Sampling from probe holes

In connection with every fourth blasting round, two probe holes were routinely drilled, one on each tunnel side, see Section 5.2. Drilling was carried out in conjunction with tunnel wall mapping by the characterisation team, and the probe holes were fitted with packers and pressure build-up tested by the same personnel, see Section 8.2.

The sampling procedure was simple, see Figure 9-3. The valve on the packer pipe string was opened and the sample was taken by filling the bottles in the prepared sampling box (see Section 9.6). Samples from probe holes were collected if the water flow rate exceeded 5 litres per minute. A similar procedure was followed in “casing with valve” arrangements.

A sample would consist of several bottles for analysis, uranine checking and frozen storage as a back-up sample, see Table 9-2. All bottles were marked with the borehole ID code (section, if needed), date, time and flow rate, which was determined while filling the bottles.

Sampling from investigation boreholes

Due to the different purposes of investigation drilling, and different types and lengths of boreholes, the sampling conditions varied. Samples were taken during breaks in the drilling or after drilling. The sample was taken from the open hole, from the drill pipe string, from the test pipe string or from “casing with valve” arrangements. A sample was taken during the drilling of the investigation borehole when the outflow suddenly increased and before grouting operations started, see Chapter 5.

However, sampling itself was performed using a procedure similar to that used for the probe holes.

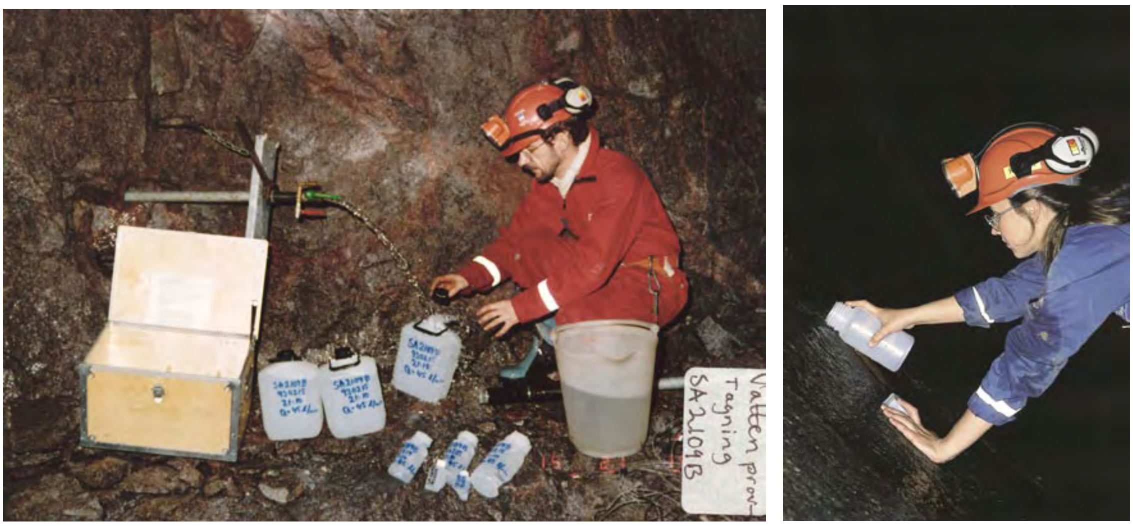


Figure 9-3. *Groundwater sampling for the documentation programme.*

9.3 Sampling for the monitoring programme

9.3.1 Purpose

On the basis of the results from the first sampling campaign (see Section 9.2) a limited number of holes were selected for repeated sampling. These samples were called "chemistry samples", and were subjected to a larger analytical programme for complete chemical characterisation, i.e. Chemistry Class 4 or in a few cases Chemistry Class 5, see Section 9.7.2. The same routines were used for monitoring groundwater chemistry changes in Äspö surface boreholes.

The purpose of the monitoring sampling programme was:

- To conduct complete chemical characterisation of the Äspö groundwater.
- To monitor the Äspö groundwater chemistry on a regular basis.

9.3.2 Equipment

All monitoring samples were taken from installed packers or casings. As in the case of documentation sampling, no pumping equipment was needed in underground boreholes. Monitoring sampling from surface boreholes was performed in multipacker-equipped boreholes, see Section 11.2.2 /Almén and Zellman, 1991/. This sampling was performed by the use of a small electrical downhole pump, a water level probe and needle valves for regulating the flow rate. Two pump alternatives were used, a built-in Pleuger Mini Unterwasserpumpe or a Grundfos MP 1 pump.

Basic equipment for flushing and sampling consists of various tecalan tubes and tube fittings as well as several graduated cylinders of different volumes and a stopwatch for flow measurements. A handy sampling device was used for collection of the samples. It consists of a ball valve, a 0.45 μm on-line filter, a manometer and a security valve to protect the filter.

Most water was filtered through the on-line filter into acid-washed bottles. The samples used for iron determination and other trace element ICP-AES analysis were protected from contamination by even more careful handling, using a disposable on-line filter and acid-washed bottles prepared with hydrochloric acid.

In-situ recording of electrical conductivity was also performed in most of the multipacker-equipped surface boreholes, although normally in only two of the sections. These measurements were part of the groundwater monitoring programme and were performed with the HMS (hydro monitoring system), see Section 11.2.4.

9.3.3 Methodology

Sampling from probe holes and investigation boreholes

Before sampling, the section was flushed with a volume at least five times the borehole section volume.

The sampling device and a tecalan tube were then connected to the section outflow (pipe string or tube from the borehole section) and the flow was adjusted to a reasonable rate. The filter was connected to the tecalan tube, and the bottles were filled.

A set of samples would consist of several bottles for the different analyses in the mobile laboratory, for the external analyses by specialized laboratories, and for frozen storage as back-up sample (see Section 9.6 and Table 9-2). The sample bottles were immediately marked with ID code, section, date and the acid added (if any).

Sampling of inflow water in measuring dams

This type of sampling was not performed on the same regular basis as the rest of the monitoring sampling. The sampling was performed as Chemistry Class 1, Class 2 or Class 3 (see Table 9-1).

One bottle was filled with water (from the v-notch overflow, see Figure 11-5) and brought to the laboratory where, in the simplest case (Chemistry Class 1) only electric conductivity and pH were measured. The sample portions used for the analyses of hydrogen carbonate, chloride and major cations by ICP-AES (Chemistry Class 2 and Class 3) were filtered in the mobile laboratory using disposable filters (0.45 µm) and a syringe. The ICP sample portion was filtered into an acidified 100 ml bottle.

Monitoring of inflowing water to the tunnel also included continuous recording of electrical conductivity in the pump sumps and in a few measuring dams, as a part of the HMS (Hydro Monitoring System), se further in Section 11.4.5 and Figure 11-6.

9.4 Special sampling programmes

Besides the frequently performed documentation sampling and monitoring sampling activities, other hydrochemical characterisation work was also performed, referred to here as special sampling programmes. The purposes and methods of these programmes varied. Two examples are mentioned below.

The large-scale REDOX experiment

The purpose of this project was to study the effects of surface water inflow and expected changes in oxidizing conditions on the deep environment. Four investigation holes were drilled, of which three were drilled from the surface. Additional sampling points were leakage in the tunnel and from the Baltic Sea.

The samples were subject to special chemical analyses, closely corresponding to Chemistry Class 4. Water from one borehole was conducted slowly through a measuring cell containing Eh, pH and electrical conductivity sensors, which were connected on line to the HMS system via a BORRE logger, see Section 11.5. The project and its results are reported by /Banward, 1995/ and /Nilsson A-C, 1995/.

Groundwater chemistry in in low-conductive rock

A pilot study was performed aiming at finding methods for sampling and analysing trace elements in stagnant saline groundwater. The other sampling programmes carried out at Äspö (during the pre-investigation and construction phases) sampled water from sections with hydraulic conductivities higher than ca 10^{-8} m/s and focussed on major components.

The sampling in this project focussed on low conductivity sections, i.e. less than ca 10^{-9} m/s, and trace element analysis. However, water samples were also subject to basic analysis according to chemistry sample routines.

Special packers and sampling equipment were developed, with all metal parts covered by teflon and tubes made of PEEK and built to minimise the dead volume in the system /Nilsson A-C, 1995/.

9.5 Chemistry classes

9.5.1 Background

During the Äspö project, SKB introduced a standardized structure for water sampling and chemical analysis. Five different Chemistry Classes or levels were defined, where a higher number indicates more extensive sampling and analysis, as mentioned in Section 9.1. The aim of the classification was to establish a few, strict routines for all groundwater chemical sampling and analyses needed.

9.5.2 Description

The five Chemistry Classes differ with respect to the number of parameters determined and, as a consequence, the complexity of the sampling procedure.

Samples of classes 1–3 involve simple classification procedures, used to get an idea of the type of water in the sample. The documentation samples (see Section 9.2) correspond to Chemistry Class no. 2.

Class no. 4 samples correspond to the chemistry samples (see Section 9.3). It is the most frequently used procedure to characterise the water. The sampling involves several bottles, on-line filtering, preservation and bottles of special types. Class no. 5 includes basically the same parameters as Class no. 4, plus a number of special analyses – isotopes, trace elements etc.

The different Chemistry Classes are described in Table 9-1.

9.6 Sample handling

Sample handling includes marking of sample bottles, sample registration, special treatment such as acid addition, transport to laboratory and storing of back-up samples.

Before sampling, sets of bottles were assembled in sampling boxes. Number of bottles and preparations for the two main sample types – Documentation sample (from the documentation sampling programme) and Chemistry sample (from the monitoring sampling programme) – are shown in Table 9-2.

Table 9-1. Definitions of Chemistry Classes (Documentation samples correspond to Class 2 and Chemistry samples to Class 4).

Chemistry class	Description	Standard parameters	Optional parameters	Back-up sample
Class 1	Sample handling, Basic check of water type.	El. cond, pH Uranine[I]	None	None
Class 2	Sample handling, Check of water type.	El. cond, pH Cl, HCO_3, Uranine[I]	^{3}H, ^{2}H, ^{18}O	Optional[II] 3 × 5 (l) + 1 (l)(acid add.)
Class 3	Sample handling, Determination of non-redox-sensitive major components.	El. cond, pH Cl, HCO_3, Br, SO_4, SO_4_S[III], Major cations[IV] (except Fe, Mn), Uranine[I]	^{3}H, ^{2}H, ^{18}O	Optional
Class 4	Extensive sampling, Complete chemical characterisation.	El. cond, pH Cl, HCO_3, Br, SO_4, SO_4_S[III], Fe (total, ferrous), DOC, Major cations[IV], ^{3}H, ^{2}H, ^{18}O, Uranine[I]	HS-, NH_4	2 × 250 (ml) 250 (+ 50) (ml) (acid add.)
Class 5	Extensive sampling, Complete chemical characterisation, including special analyses.	El. cond, pH Cl, HCO_3, Br, SO_4, SO_4_S[III], Fe (total, ferrous), DOC, Major cations[IV], ^{3}H, ^{2}H, ^{18}O, Uranine[I]	HS-, NH_4, NO_2 (or NO_2+NO_3), PO_4, F, I, TOC, ^{13}C, ^{14}C U and Th isotopes, Ra and Rn isotopes, Trace metals	2 × 250 (ml) 250 (+ 50) (ml) (acid add.)

Notes:

I Only measured when uranine was used in the drilling process and as long as no extra uranine was added to the borehole e.g. in tracer tests.

II In the documentation sampling programme, back-up samples were collected when water flow rate > 10 l /min.

III Sulphate sulphur determined by ICP-AES.

IV Major cations are Na, Ca, K, Mg, Si, Fe, MN, Li and Sr.

Special sampling:

It is recommended that the very special and seldom performed sampling listed below be combined with a Class 5 sampling procedure.

- Colloids/particles.
- Bacterial activity.
- Fulvic and humic acids.
- S and Sr isotopes.
- Gas.

Table 9-2. Sample use, volumes and preparations for the two main sample types.

Sampling programme – sample portion use	Volume (ml)	Filtration on-line	Acid-washed bottle	Acid addition
Documentation sample (Chemistry class 2				
– El. conductivity, pH, Cl, HCO_3, Uranine	2×500	No	No	No
– ^2H, ^3H and ^{18}O	2×100	No	No	No
– Freeze stored back-up sample	3×5,000[I]	No	No	No
	1×1,000[I]	No	No	Yes (1% conc. Suprapur HCl)
Chemistry samples (Chemistry Class 4)				
– pH, El. conductivity, HCO_3, Cl, SO_4, Br, Uranine	2 × 250	Yes (C)	No	No
– Fe(+II), Fe(tot), SO_4_S, Major cations	500[II]	Yes (N)	Yes	Yes (1% conc. Suprapur HCl)
– DOC	250	Yes (C)	No	No
– ^2H, ^3H and ^{18}O	100–500	No	No (dried bottle)	No
– Sulphide (optional)	110–130 [III]	Yes	No	No (preservation)[IV]
– Ammonium (optional)	2 × 25	No	No (reagent washed	No
– Frozen stored back-up sample	2 × 250	Yes (C)	No	No
– Frozen stored back-up sample	250 (+ 50)[II]	Yes (N)	Yes	Yes (1% conc. Suprapur HCl)

Notes:
I If water flow rate > 10 l per minute.
II Divided into portions for Fe(+II), Fe(total), ICP and back-up.
III Winkler bottles, somewhat varying volumes.
IV Preservation with 0.5 ml 1 M Zn-acetate and 0.5 ml 1 M NaOH.
(C) High capacity on-line filter from Colly Company AB, 0.4 µm.
(N) Nucleopore filter 0.4 µm (Single use).

The sample portions for hydrogen sulphide determinations were collected in so-called Winkler bottles and immediately preserved with 0.5 ml 1 M Zn acetate and 0.5 ml 1 M NaOH.

Immediately after sampling, the bottles were carefully marked with section ID code, date and acidification type (if any). After completed sampling and transportation to SKB's mobile chemistry laboratory (see Section 9.7.1) at Äspö, the sample was registered. Each chemistry sample was given a serial sample number printed on a label attached to each bottle. The back-up bottles were transferred to the freezer along with the bottle/bottles for DOC determination. The 250 ml DOC bottle was divided into two portions, frozen and then sent to two laboratories.

Some of the analyses – mainly pH, major anions and redox-sensitive components – were performed in the mobile field laboratory by the Äspö HRL chemist, see Section 9.7.2. For other analyses, sample portions were sent to external laboratories. Sample portions of each sample, with as well as without acid preservation, were frozen and stored as back-up samples.

9.7 Analysis of water samples

9.7.1 SKB’s mobile chemistry laboratory

As mentioned in Section 9.6, SKB’s mobile chemistry laboratory was used for handling of water samples as well as for conducting some of the chemical analyses. The laboratory itself is equipped with titrimetric, spectrophotometric and ion chromatographic instruments, see Figure 9-4 and the description in /Almén and Zellman, 1991/.

9.7.2 Analyses performed

As mentioned above, two types of water samples were collected: documentation samples from the documentation sampling programme and chemical samples from the monitoring sampling programme.

The chemical parameters determined from the two sample types are presented in Table 9-1, where Chemistry Class 2 corresponds to documentation samples and Class 4 corresponds to chemical samples.

9.7.3 Analysis methods

The analytical methods, laboratories and detection limits are given in Table 9-3a,b. To ensure data quality, several components were determined by more than one method, and several control samples sent to more than one laboratory. Generally, the concentration determined by different laboratories agreed within ± 10%. If the difference exceeded this limit, the analyses were repeated.

The analytical methods, laboratories and detection limits have been thoroughly described by /Nilsson A-C, 1995/. Quality control and data management of the chemical data are also described in that report.

Examples of sampling information and the most important parameters for groundwater chemical characterisation in major fracture zones are given in Table 9-4.

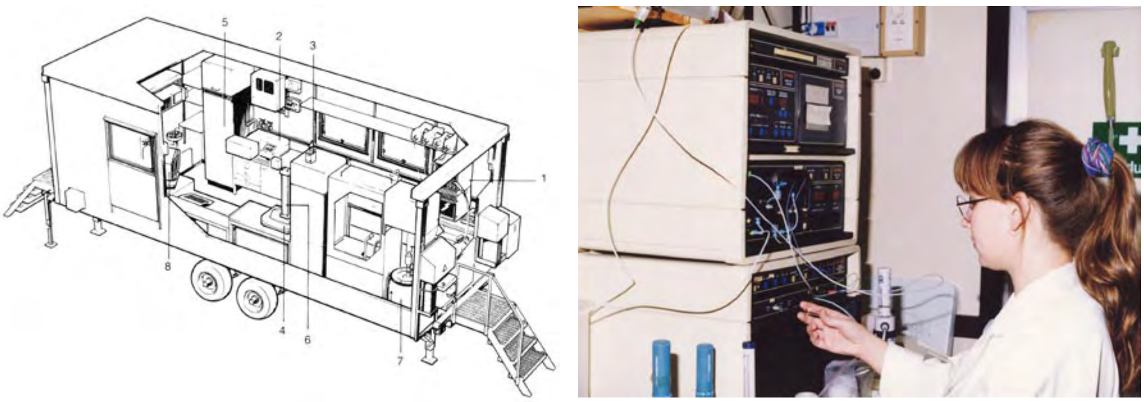


Figure 9-4. The chemical analysis unit of the mobile chemistry laboratory.

Table 9-3a. Methods, laboratories and detection limits for chemical analyses of elements and compounds in groundwater samples.

Components/ parameters	Method	Laboratory	Detection limit (mg/l)
Na	ICP-AES	KTH, SGAB[1]	0.04[4]
Ca	ICP-AES	KTH, SGAB[1]	0.006[4]
K	ICP-AES	KTH, SGAB[1]	0.04[4]
Mg	ICP-AES	KTH, SGAB[1]	0.005[4]
Si	ICP-AES	KTH, SGAB[1]	0.004[4]
Mn	ICP-AES	KTH, SGAB[1]	0.0002[4]
Fe(tot)	spect. (Ferrozine)	MFL	0.005
	ICP-AES	KTH, SGAB[1]	0.002[4]
Fe(+II)	spect. (Ferrozine)	MFL	0.005
Sr	ICP-AES	KTH, SGAB[1]	0.001[4]
Li	ICP-AES	KTH, SGAB[1]	0.001[4]
Cl	Titr. (SIS 028120) IC[2]	MFL	10
Br	IC	MFL	0.1
	Neutron activation	Studsvik AB[3]	
HCO_3	Titr. (SIS 028139)	MFL	0.5
HS^-	Spect. (SIS 028115)	MFL[3]	0.01
SO_4	IC	MFL	0.05
SO_4_S	ICP-AES	KTH, SGAB[1]	0.02[4]
pH	SS 02 81 22	MFL	
el. cond.	SS-EN 27 888	MFL	
drilling water %	Fluorimetric	MFL	0.2%
DOC	ASTRO M. 2001	IVL	0.5 (1)
	High temp. catalytic combustion	Marin kemi[3]	0.2
	Shimadzu TOC – 5000		
	Shimadzu TOC – 5000	Univ of Helsinki[3]	0.2

Notes:
1 Control analyses, one sample per each fifth or tenth collected sample.
2 Method used for concentrations < 10 mg/l.
3 A few determinations.
4 Detection limit (2), ICP-AES method, report limit 5*DL.

Table 9-3b. Methods and laboratories for chemical analyses of elements and compounds in groundwater samples.

Components/parameters	Method	Laboratory
U (µg/l)	ICP-MS	SGAB
U (Bq/l)/(µg/l)	Alfa Spectrometry	Studsvik AB
Th	ICP-MS	SGAB
	Alfa Spectrometry	Studsvik AB
trace elements	ICP-MS	SGAB
trace elements	INAA	Studsvik AB
^{226}Ra, ^{222}Rn	Gamma Spectrometry	Studsvik AB
^{234}U	Alfa Spectrometry	Radiofysik, Lund
^{235}U	Alfa Spectrometry	Radiofysik, Lund
^{238}U	Alfa Spectrometry	Radiofysik, Lund
^{232}Th	Alfa Spectrometry	Radiofysik, Lund
^{230}Th	Alfa Spectrometry	Radiofysik, Lund
^{228}Th	Alfa Spectrometry	Radiofysik, Lund
^{3}H	Natural Decay counting	Energiteknikk, Kjeller
^{2}H	Mass Spectrometry	Energiteknikk, Kjeller
^{18}O	Mass Spectrometry	Energiteknikk, Kjeller
^{13}C	Accelerator measurement	Svedberg Laboratory, Uppsala
PMC (^{14}C age)	Accelerator measurement	Svedberg Laboratory, Uppsala
Laboratories:		
MFL	SKB Mobile Field Laboratory	
KTH	Royal Inst. of Technology, Dept. of Chemistry, Inorganic Chemistry	
SGAB	Svensk GrundämnesAnalys AB, Luleå	
Studsvik AB	Studsvik AB, Nyköping	
IVL	Institutet för vatten och Luftvårdsforskning, Stockholm	
Marin kemi	Umeå Marina Forskningscentrum, Marin kemi, Norrbyn, Hörnefors	
Radiofysik, Lund	Radiation Physics Department, Lund University	
Energiteknikk, Kjeller	Institutt for Energiteknikk, Kjeller, Norway	
Methods:		
ICP-AES	Inductively-Coupled Plasma Atomic Emission Spectrometry	
ICP-MS	Inductively-Coupled Plasma Mass Spectrometry	
IC	Ion Chromatography	
Tetr.	Titrimetric Method	
Spect.	Spectrophotometric Method	
Pot.	Potentiometric Measurement	
Fluorometric	Spectrofluorimetric Method	
ASTRO M 2001 Carbon analyser (Trademark)		
INAA	Instrumental neutron activation	

Note:
PMC (Percent Modern Carbon) is calculated from the C-14 age by means of the formula:
$PMC = 100\times e^{((1{,}950-y-1.03t)/8{,}274)}$
where y = year of C-14 age measurement and t = C-14 age.

Table 9-4. Results from groundwater chemical characterisation of major fracture zones (excerpt from /Rhén et al. 1997c, A3.2).

Fracture zone	Reprens-ting day 0=90-10-14	ID-code	Penetra-ting zone	Tunel length, m	Date	Inflow rate m^3/s $\times 10E^{-3}$	SNO	Na mg/l	K mg/l	Ca mg/l	Mg mg/l	HCO_3 mg/l	Cl mg/l	SO_4 mg/l
NE–1a, 1b	750	HA1327B	X	1,327	92-10-15	5.0	2,023	1,610.0	9.4	648.0	128.0	252	3920.0	225.0
NE–1a, 1b	950	SA1229A		1,229	93-06-23	5.0	2,120	1,847.9	24.5	598.5	156.1	426	4210.9	101.0
NE–1a, 1b	1,150	HA1327B	X	1,327	93-12-15	5.0	2,208	1,760.0	13.7	684.0	157.0	259	4310.0	255.3
NE–1a, 1b	1,350	SA1229A		1,229	94-06-07	5.0	2,256	1,735.4	26.1	512.1	151.7	336	3982.2	241.5
EW–3a	750	SA1420A	X	1,420	92-10-15	0.8	2,024	1,540.0	10.2	715.0	123.0	170	3930.0	226.2
EW–3a	950	SA1420A	X	1,420	93-06-22	0.8	2,116	1,484.2	9.7	487.9	124.5	215	3419.9	307.0
EW–3a	1,150	SA1420A	X	1,420	93-09-29	0.8	2,183	1,600.0	13.7	480.0	139.0	214	3530.0	331.8
EW–3a	1,350	SA1420A	X	1,420	94-06-07	0.8	2,257	1,426.5	15.7	395.8	116.8	206	3052.5	290.3
NE–2a–1	750	SA1641B		1,614	92-11-19	0.003	2,035	1,570.0	8.3	1,250.0	80.2	37	5160.0	296.4
NE–2a–1	950	SA1614B		1,614	93-06-22	0.003	2,117	1,953.7	5.2	1,710.4	65.9	32	6207.3	424.0
NE–2a–1	1,150	SA1614B		16,14	93-09-28	0.003	2,184	1,880.0	6.7	1,390.0	90.8	81	5650.0	332.4

Fracture zone	Reprens-ting day 0=90-10-14	ID-code	^3H (TU)	^2H (SMOW)	^{18}O (SMOW)	Fe (tot) mg/l	Fe^{2+} mg/l	DOC mg/l	PH	Calcite (logIAP/KT)	Log pCO_2 (bar)	Indicator of sulphat reduction
NE–1a, 1b	750	HA1327B	17.0	–65.3	–7.4	2.160	2.150	–	7.4	0.58	–2.21	Bacteria
NE–1a, 1b	950	SA1229A	16.0	–60.0	–7.3	2.891	–	21.0	7.0	0.33	–1.62	GW
NE–1a, 1b	1,150	HA1327B	18.0	–50.6	–7.5	2.640	2.430	4.8	6.9	0.05	–1.76	
NE–1a, 1b	1,350	SA1229A	22.0	–52.8	–7.0	–	–	–	7.0	0.17	–1.72	
EW–3a	750	SA1420A	17.0	–72.0	–8.7	1.110	1.110	–	7.6	0.66	–2.57	
EW–3a	950	SA1420A	31.0	–59.0	–7.5	1.941	1.920	–	7.3	0.30	–2.18	
EW–3a	1,150	SA1420A	22.0	–52.5	–7.0	–	–	–	7.3	0.29	–2.18	
EW–3a	1,350	SA1420A	33.8	–57.0	–7.5	–	–	–	7.2	0.10	–2.10	
NE–2a–1	750	SA1641B	8.0	–103.1	–13.1	–	–	1.0	7.4	–0.01	–3.06	
NE–2a–1	950	SA1614B	4.2	–85.5	–11.5	0.309	0.298	1.0	7.6	0.22	–3.34	
NE–2a–1	1,150	SA1614B	4.2	–77.6	–10.4	–	–	1.0	7.4	0.35	–2.73	

9.8 The uranine station

Since 1984 SKB has used the fluorescent colour tracer Uranine for tagging water used for drilling and hydraulic testing in site investigations. The aim is to check the amount of water artificially introduced into the formation, i.e. the amount of contamination of the water samples.

To simplify the mixing of Uranine into the water used in the Äspö tunnel, a Uranine station was constructed. In the Uranine station, all water used for drilling, hydraulic testing, etc was tagged with uranine at a constant concentration. The station was put into operation in May 1991 and intially located outside the visitor nische at tunnel section 0/0130 m. The station was later moved to pumping station No 3 outside the shaft entrance at –220 m depth.

A dosage tank is filled manually with Uranine concentrate (110 g/400 l). From this tank, the concentrate is pumped into the tunnel water pipeline. A flow meter on the pipeline triggers the pump via a control unit, allowing the Uranine concentration of the tunnel water to be kept at 0.2 mg/l, see Figure 9-5.

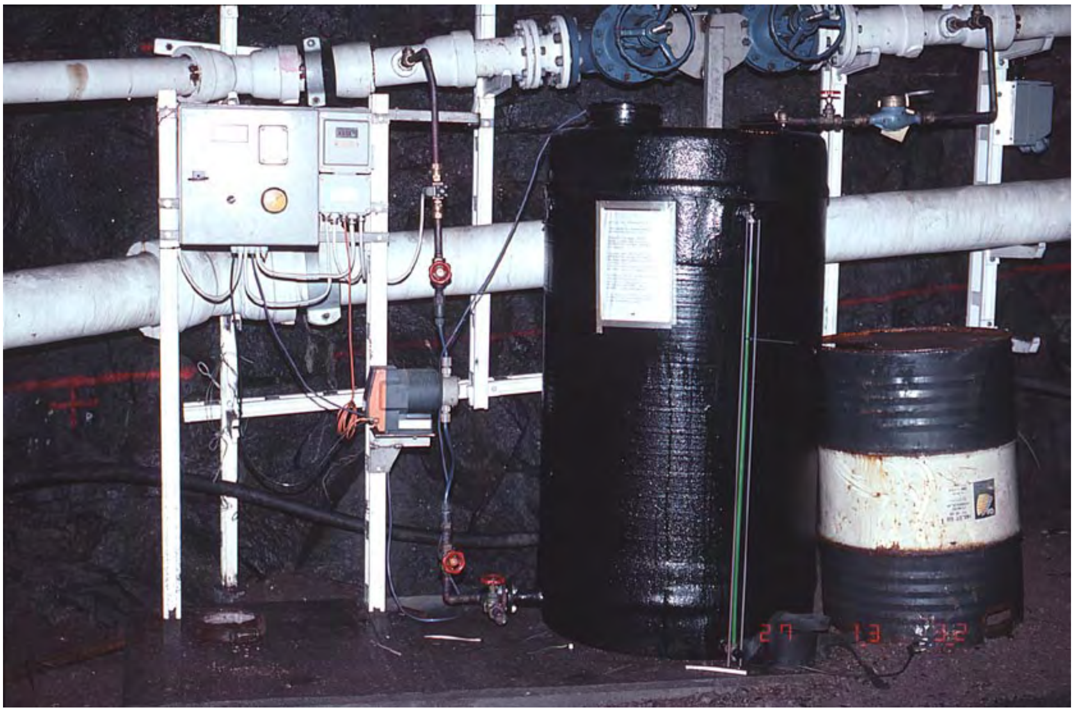


Figure 9-5. *The Uranine Station.*

9.9 Accuracies

9.9.1 Possible errors caused by sampling and sample handling

Sampling

An inappropriate sampling procedure can result in samples, which are not representative of the groundwater in the sampled borehole section. Sampling of the surface boreholes can be more critical in that respect than sampling in the tunnel boreholes, where no pumping is needed. Excessive pumping and/or insufficient flushing of the boreholes are the two reasons for such errors.

Sample handling

Errors of a second type are due to contamination or changes/reactions within the samples and are described in the list of sensitive components/parameters below.

- **pH:** Errors in the pH values due to the pressure change and its influence on the carbonate system can be expected. Further, pH measurement is carried out after collection and transport to the mobile laboratory. One or two hours often elapse between sampling and measurement.
- **Hydrogen carbonate (Alkalinity):** Changes in the HCO_3 concentration can occur if the time between sampling and analysis is too long. Generally the alkalinity titration is performed on the sampling day, which is satisfactory.
- **Ferrous and total iron:** Broken filters (too high pressure) can cause erroneous results if not discovered. Oxidation of ferrous iron will take place if the time between sampling and analysis is too long. If possible, spectrophotometric analysis of iron is performed on the sampling day, which is satisfactory. Oxidation and precipitation of all the iron present will take place if, by mistake, a sample is collected in a bottle without acid being added.
- **Ammonium:** The risk of contamination by the air in the tunnel or by the syringe used to take the sample portion is fairly high. It is not easy to collect accurate and reproducible volumes with a syringe.
- **Hydrogen sulphide:** It is important to collect the water in such a way that no air is entrapped in the bottle or dissolved in the sample. Hydrogen sulphide may be lost during sampling.
- **DOC (Dissolved Organic Carbon):** Bacterial growth in the samples will cause errors. A long storage time at room temperature or a long transport time may cause erroneous results. The samples are frozen as soon as possible after sampling and before transport to the analysing laboratory.
- **Tritium:** There is a risk of contamination by humidity in the air, especially when the tritium content in the sample is low. The water for tritium analysis is sometimes collected in 100 ml dried plastic bottles (early samples). These bottles are not completley gastight and the storage time before transport to the laboratory must therefore be as short as possible.
- **Radon:** If the water flow rate is low, some Radon may be lost during the time it takes to collect the water. The container (5 litres) must be filled all the way up. The sample must be sent immediately to the laboratory, and long transport times must be avoided.
- **Trace metals:** It is very difficult to avoid contamination by common trace metals such as Al, Cu, Cr, Co, Ni, Pb and Zn. Sampling for analysis of these elements requires special borehole instrumentation/equipment (see Section 9.4) and is generally not performed.

- **Uranine (drilling water residue):** Uneven mixing of uranine in the drilling water or inadequate supervision of the performance of the uranine station will lead to an erroneous initial value for calculation of the drilling water residue in the sample. Long storage of the sample under daylight conditions before performing the fluorometric measurement may cause some decomposition of the fluorescent compound.

9.9.2 Sources of analytical errors

Generally, the high concentration levels in many of the groundwater samples may give rise to problems or errors in some of the analytical methods. A list of other possible sources of error for a selection of components/parameters is given below.

- **pH:** pH needs to be measured as soon as possible and the temperature of the groundwater is often around 15° centigrade. If the temperature of the sample is not checked it is easy to get a temperature difference between sample and room and/or between standards for calibration and sample.
- **Hydrogen carbonate (Alkalinity):** In some cases, foaming samples cause slow mixing and make it difficult to see the titration end point. Samples from the boreholes HBH02 and HBH05 are often of this type.
- **Bromide:** Bromide is sometimes difficult to determine by means of ion chromatography if the chloride peak is very dominant. The samples are, if possible, diluted to a chloride concentration < 300 mg/l and the standards for the calibration are matched to the samples so that the chloride concentration of the standard is close to that of the sample.
- **Total and ferrous iron:** Samples with a colloidal fraction that passes through the filters will cause disagreement between the ICP determination and the spectrophotometric method. The colloids may be excluded from, or only partly included in, the results of the spectrophotometric method, but are included in the total iron determined by ICP.
- **Ammonium:** There is a high risk of contamination by air, by reagents, by the volumetric flasks or by deionized water. The volumetric flasks are washed with the reagents. Precipitation causes difficulties in samples with high calcium and magnesium concentrations.
- **Hydrogen sulphide:** Precipitation causes problems in some waters with high calcium concentrations.
- **Potassium:** The ICP determination has low sensitivity, and ionisation effects can cause errors.

9.9.3 Quantification of errors

Measurement uncertainties for a number of components/parameters are given in Table 9-4 below. The uncertainties are based on:

- Estimation from experienced repeatability and reproducibility (MFL).
- Relative standard deviation for repeated analyses of a control sample (certified standard) on different measurement occasions. (ICP by KTH and SGAB).
- Recoveries of internal standards (IFE).
- 3× instrumental spread (relative standard deviation) of repeated measurements of samples (C-14 age).

Table 9-4. Measurement uncertainties for different components, laboratories and analytical methods.

Components/ parameters	Laboratory	Method	Measurement uncertainties
pH, Conductivity,	MFL	Potentiometric	± 0.1 pH unit
Cl,		Titrimetric	± 5%
HCO_3		Titrimetric	± 5%
Na, Ca, K, Mg, S, MN, Fe, Si, Li, Sr	KTH, SGAB	ICP AES	± (4–5)%
SO_4, Br	MFL	Ion Chromatography	± 10%
Fe (tot), Fe(+II)	MFL	Spectrophotometric	± 10%
HS-			± 10%
NH_4	MFL	Spectrophotometric	± 10%
^{3}H	IFE	Natural decay counting	± 0.5 Bq/l (4.2 TU)
^{2}H		MS	± 1.0 *
^{18}O		MS	± 0.2 *
^{13}C	Tandem Lab, Uppsala	Accelerator measurement	–
PMC (^{14}C age)	Tandem Lab, Uppsala	Accelerator measurement	< ± 12%
Trace metals	SGAB	ICP MS	± (15–20)%

Note:
* The unit is "per mill deviation from SMOW (Standard Mean Oceanic Water)".

9.10 Comments and recommendations

Sampling and analysis of groundwater during tunnel construction was different from previous work done by SKB. Instead of sampling and analysis being performed in one and the same borehole section over a two- to four-week time period, several different boreholes were sampled almost daily and the number of samples was greatly increased. Under these circumstances, the available space in the mobile field laboratory was too restricted to allow efficient work.

The conditions for performing high-quality sampling in the tunnel were good. When the groundwater flows towards the tunnel, it can maintain natural hydraulic pressure. Flushing the section prevents contamination by drilling water – in the chemistry samples the uranine content is consistently lower than 1%.

Introduction of the Chemistry Classes facilitated sampling and sample handling as it created more standardised routines and far fewer combinations of components to be analysed. This made it easier to know what analyses had or had not been done on each sample.

The documentation sampling routines worked well with the handy boxes prepared with sample bottles that were filled and then delivered to the mobile field laboratory for analysis.

Chemistry sampling became more time consuming. Instead of the water being conducted straight into the laboratory, as was the case during the pre-investigations, the samples had to be collected and transported to the laboratory. To speed up sampling in the tunnel, fixed valve installations on the packers were prepared. These valve installations were practical but corroded very quickly and were therefore only used to a limited extent.

Pleuger pumps were used for sampling in the surface boreholes. The Grundfors MP 1 pumps were not used very often as they were found to interfere with the HMS radio communication system.

Transportation of the sample led to a delay between sampling and analysis. This is of little consequence as long as the pH, alkalinity, iron and ammonium determinations can be done on the sampling day, but this was not always possible due to lack of analysis capacity.

It is important that information reaches the chemists concerning how/in what connection a sample or a series of samples are collected (during drilling, before grouting etc) in order to avoid uncertainties regarding sampled borehole sections etc.

The analysis work proceeded reasonably well during the construction phase, except for the inconveniences caused by the very limited laboratory space.

It is also important to have smoothly functioning data storage routines from the start of new investigations. This was not the case at the beginning of the construction phase.

Generally, the quality of the analysis work was comparable to that in the pre-investigation phase.

Some comments and recommendations regarding certain parameters and components follow below.

Representative pH values can only be obtained by in-situ measurement in the borehole sections or by measurement in a pressurised through flow cell connected to the borehole outlet. The batch pH measurements at atmospheric pressure were only useful for obtaining approximate values and detecting irregularities such as the influence of grouting etc. It is possible to improve the batch measurement method by performing the measurement outside the borehole outlet with a portable pH meter combined with a temperature sensor. The usefulness of this is questionable, however.

The DOC results reported during the construction phase were uncertain. It is important that the consulted laboratory/laboratories have the equipment and the experience needed for analysing DOC in water of high salinity. The DOC result seemed to depend on which analysis method was used. Several laboratories were consulted, see Table 9-3b, and the analysis results differed considerably between them.

Sampling for tritium analysis could be improved by using vacuum containers similar to the ones used to collect blood samples. This would reduce the risk of contamination by humidity in the air.

Further comments on groundwater chemistry are given in /Laaksoharju and Skårman, 1995/.

10 Rock mechanical investigations

10.1 Introduction

The general purpose of the rock mechanical characterisation program was, see also Table 13-1:

- To provide data for comparison with predictions and to submit additional geomechanical information for detailed characterisation and updating of models.

Rock stress measurements will be described in this chapter, while rock mechanical documentation of tunnel walls and laboratory investigations of rock samples were described in Chapters 4 and 6, respectively. With reference to Section 2.2 and Figure 1-5, the stress measurements and most of the laboratory investigations are associated with special investigations, while the tunnel wall documentation is a part of the baseline characterisation.

10.2 Rock stress measurements

10.2.1 General description and purposes

Rock stress measurements during the Äspö HRL pre-investigation phase were performed using both a hydrofracturing technique and an overcoring technique /Almén and Zellman, 1991/. The results of these measurements formed the basis for the predictions mentioned above /Gustafson et al. 1991/.

Concurrent with the excavation of the access ramp, rock stress measurements were conducted in series of short, near-horizontal boreholes drilled from pre-determined locations along the ramp.

The purposes of the rock stress measurements during the construction phase were:

- To evaluate predictions made prior to development of the tunnel ramp.
- To provide background information for determining stress conditions on a local scale.

The overall state of stress also determines the boundary conditions for the various experiments to be conducted later at Äspö.

The rock stress measurements during the construction phase were performed by means of the overcoring technique. In all, 67 measurements were successfully performed in 14 boreholes representing 10 locations along the tunnel ramp. The vertical depths of these locations varies from 143 m to 416 m below the ground surface

10.2.2 The overcoring stress measurement method

The stress measurements at Äspö HRL construction phase were performed using an overcoring technique in short (< 20 m) boreholes drilled from the access ramp. The basic principle of the overcoring technique is to measure deformation (or strain) at the surface of a borehole in a piece of rock, which is stress-relieved by means of overcoring. Assuming linear elastic rock behaviour, the state of stress can be back-calculated on the basis of recorded deformations and known or estimated material properties. A couple of overcoring techniques exist and two of them, the CISRO Hollow Inclusion technique /Leeman, 1968/ (42 measurements) and the Vattenfall Hydropower Borre Probe (25 measurements) /Ljunggren and Klasson, 1996/, were employed in the Äspö HRL construction phase.

The field work includes borehole drilling, instrument installation, overcoring and testing of the rock cylinders obtained from the overcoring in a bi-axial load cell.

The CSIRO Hollow Inclusion overcoring technique

The CSIRO Hollow Inclusion technique is based on a concept introduced by Leeman in the early sixties, /Leeman, 1968/, where a soft-type gauge is used to record the strains at the surface of the overcored borehole. The CSIRO HI-cell is shown schematically in Figure 10-1. The method is a well established and commonly used worldwide.

The HI-cell incorporates a total of twelve strain gauges, each with a base length of 10 mm, orientated in nine different directions. The HI-cell forms a hollow epoxy inclusion, which is glued directly onto the borehole wall for a distance of approximately 100 mm. The strain gauges are cast inside the epoxy, and are thus protected from water. The core relaxation process can be monitored during overcoring by means of a readout cable extending from the cell, through the drill string, to the recording device.

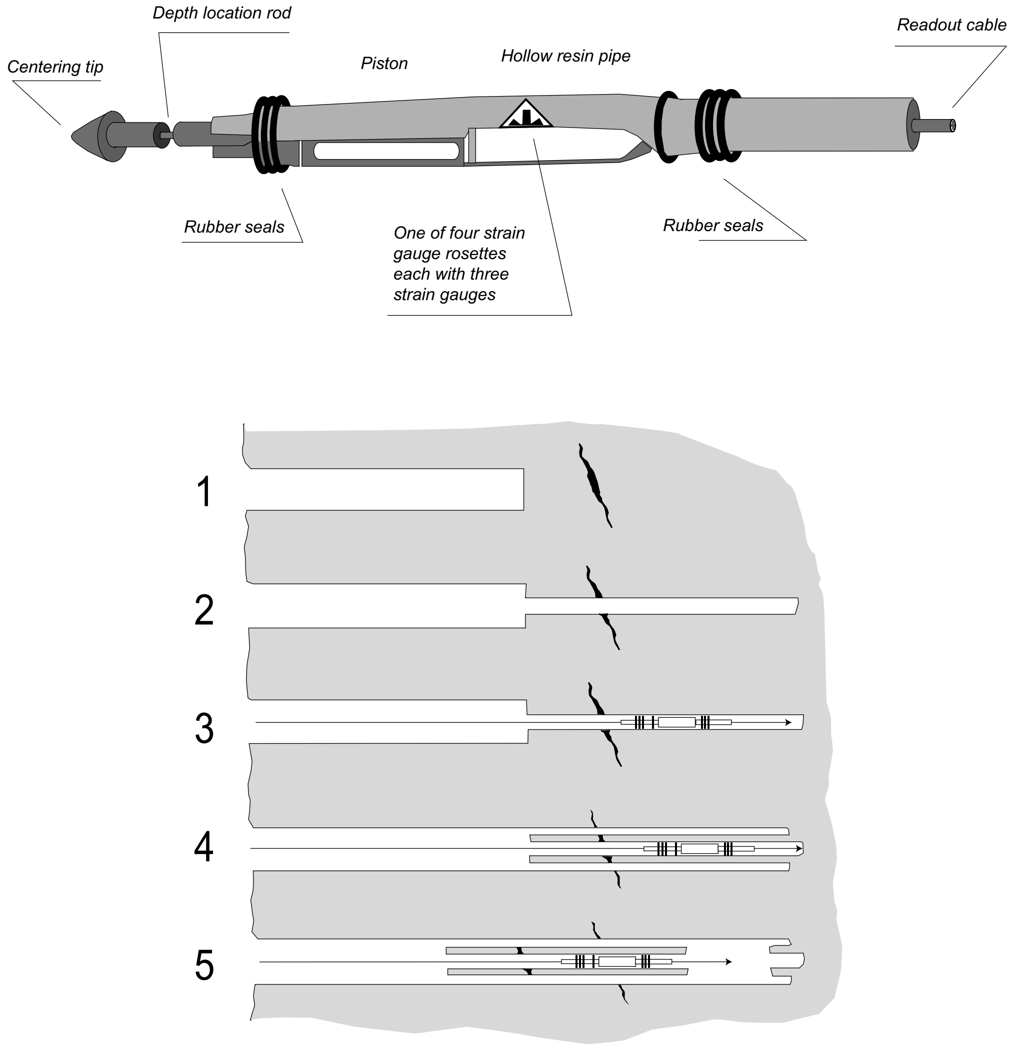
The Vattenfall Hydropower Borre Probe

The Vattenfall Hydropower three-dimensional Borre Probe is based on the same measurement principle as the CSIRO HI-cell. However, instead of using 12 gauges, the Borre Probe incorporates 9 active strain gauges, see Figure 10-2. Other differences are that the Borre Probe includes a data logger powered by battery, which allows the complete overcoring procedure to be recorded at pre-set time intervals without any cable connections to the borehole surface. After completion of overcoring and core retrieval from the borehole, all data in the logger are transferred to a laptop computer for preliminary evaluations on-site. The Borre Probe is designed for use in water-filled boreholes, which allows overcoring measurements to be conducted in declined and deep boreholes.

10.2.3 Measurement procedure

The rock stress measurements are conducted according to the following procedure, which in principle is the same for the both techniques used (see also Figure 10-1 and 10-2 for more detailed descriptions of the procedures).

- A hole (minimum diameter 76 mm for the Borre Probe and 86 mm for the CSIRO HI-cell) is drilled to the desired distance from the tunnel.
- At measurement depth, a smaller pilot hole is drilled.
- The measurement probe is installed into the pilot hole and strain gauges are fixed to the borehole wall.
- The measurement gauge installation is overcored and the strains accompanying core relaxation are recorded.
- The hollow rock cylinder, with the probe inside, is recovered and inspected.
- The cylinder is subjected to a bi-axial test. This involves applying an external pressure, while recording resulting strains. The test allows the elastic properties of the overcored volume to be determined.



1 A 86 mm diameter borehole is drilled to a pre-determined depth.

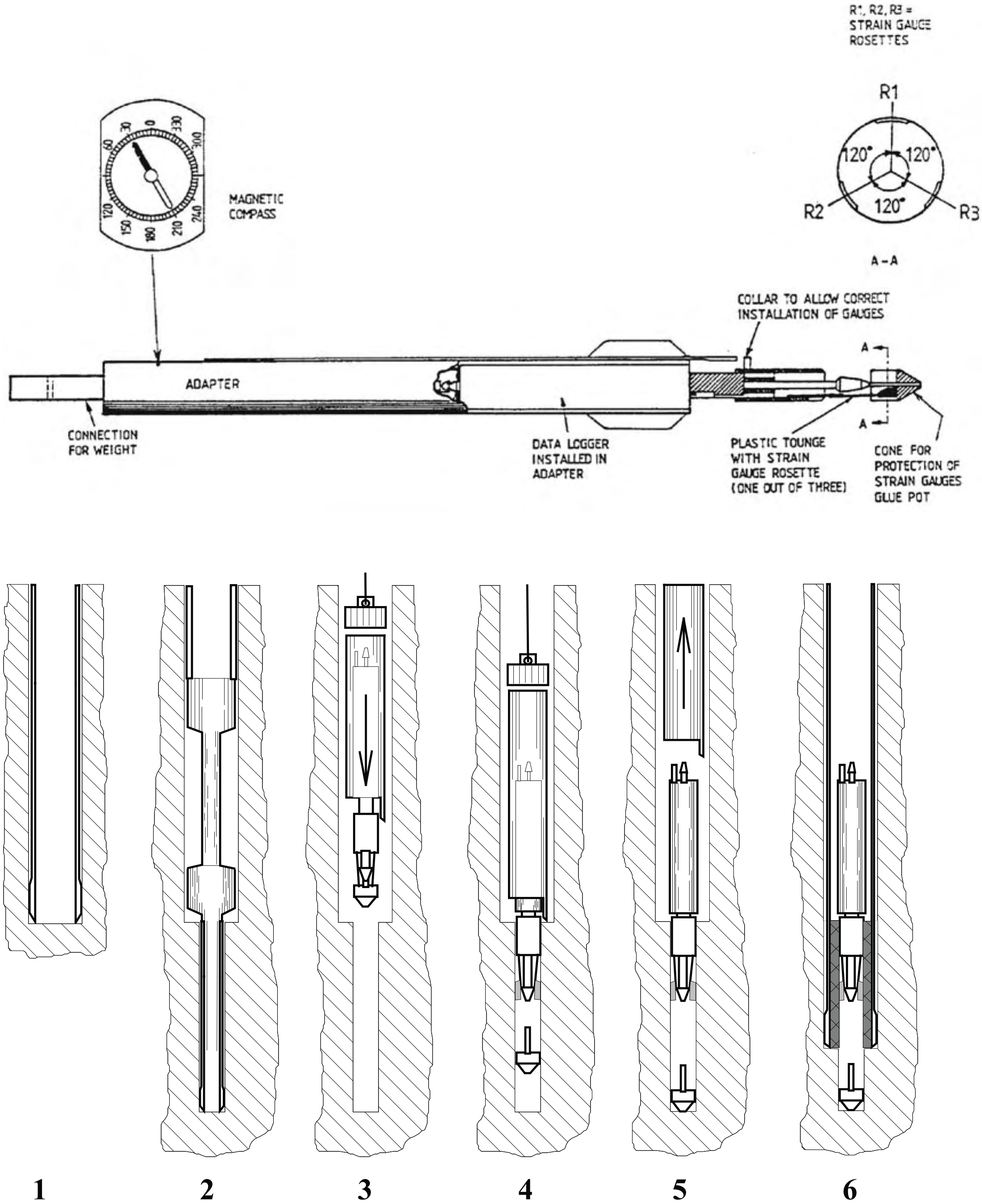
2 A smaller (36 or 38 mm) borehole is drilled at the bottom of the larger hole. This pilot hole is centered very precisely using drill string guide rods. The core from the pilot hole is inspected to determine a suitable position for stress measurement.

3 Glue is mixed and poured into the hollow gauge. The measurement probe is installed in the pilot borehole. The piston extrudes the glue so that the strain gauge rosettes inside the gauge are cemented to the pilot hole wall. The glue is allowed to set overnight.

4 Overcoring is conducted using a 86 mm diameter core bite. Stress relaxation is recorded through the strain gauges.

5 The core is recovered and placed in a bi-axial load cell. After the bi-axial testing the core is cut in two halves in order to allow for inspection of the glue bond between the gauge and the borehole wall.

Figure 10-1. *Schematic illustration of the CSIRO HI-cell and measurement procedure.*



1 Advance ϕ76 mm main borehole to measurement depth.

2 Drill ϕ36 mm pilot hole and recover core for appraisal.

3 Lower Borre Probe in installation tool down hole.

4 Probe releases from installation tool. Gauges bonded to pilot-hole wall under pressure from the nose cone.

5 Raise installation tool. Probe bonded in place.

6 Overcore the Borre Probe and recover to surface in core barrel.

Figure 10-2. *Schematic illustration of the Borre Probe and measurement procedure.*

The most important prerequisite for the successful performance of overcoring tests is that no fractures exist in the rock volume to be overcored. The active length/volume of the measurement point (i.e. of the strain gauge part of the probe) is on the centimetre scale. I addition, the properties within a rock volume of approximate 1 dm^3 around the measurement point influence the measurements. Hence, the selection of measurement point is of crucial importance and will be based on existing data, inspection of the investigation borehole core, the pilot hole core and the measurement engineer's experience from previous measurements, see also Sections 10.2.5 and 10.2.6.

Figure 10-3 graphically illustrates a typical overcore strain data response. Readings taken from borehole KA3579G at Äspö HRL.

The overcoring equipment should be calibrated at regular intervals to check measurement accuracy. A straightforward check of equipment function is done by performing strain measurement in a material with known properties which is subjected to pressure in a bi-axial cell. The material properties of the calibration core are calculated from the strain changes obtained in this test. The evaluated properties are then compared with the production specifications for the material.

10.2.4 Data processing and stress determination

The closed form solutions for evaluation of the complete stress tensor from strain data have been presented by /Leeman, 1968/. Usually, linear elastic and isotropic rock conditions are assumed. Several computer codes are available for this type of calculations. The two codes used for the overcoring measurements at Äspö HRL (one for the HI-cell and another one for the Borre Probe) are based on the same fundamental equations and differ only in the presentation of the calculated results. Both methods provide redundancy in the measurements, the HI-cell somewhat more, since the number of independent strain measurements exceeds the minimum number required to solve the stress tensor.

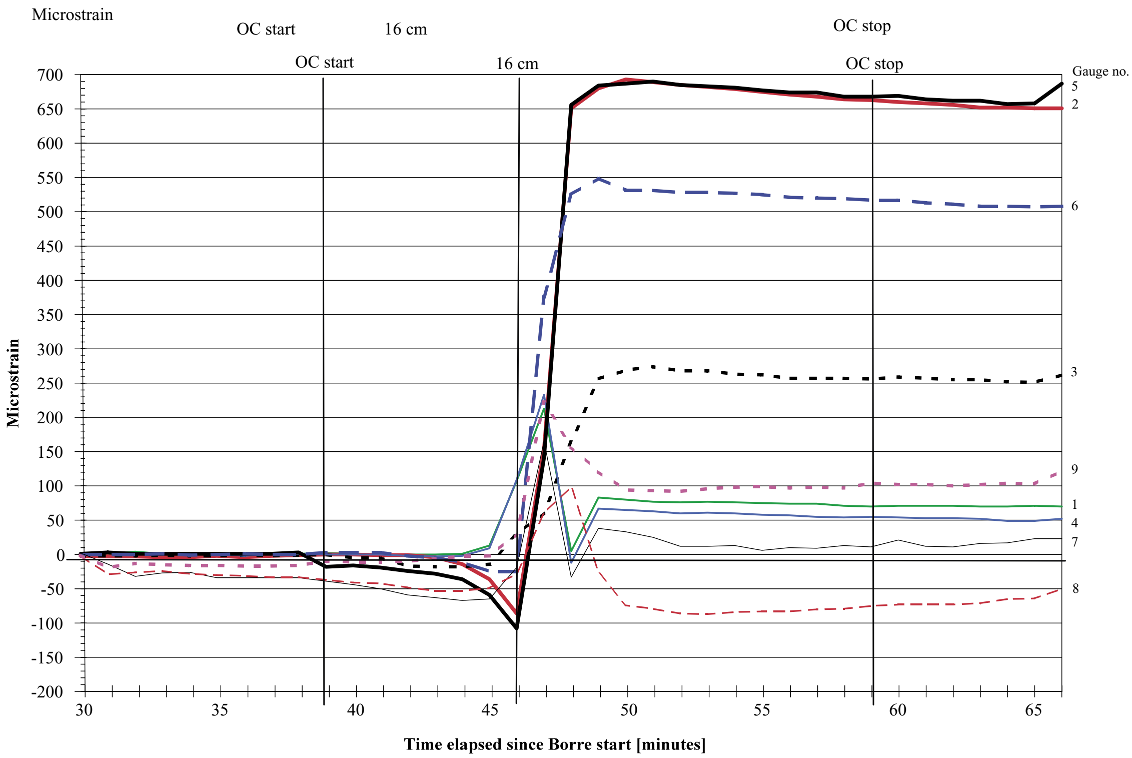


Figure 10-3. Typical overcore strain data, example from borehole KA3579G at Äspö HRL.

The programme therefore utilises a multiple least squares regression technique to obtain the best fit of the stress tensor to the strain data. Furthermore, it permits detection and rejection of single outlying strain values as well as statistical analysis.

Data from bi-axial load cell test on the overcored rock sample are used to determine the elastic properties of the rock, as well as to check the performance of the strain gauges. The rock properties are calculated assuming plane stress conditions, according to the formulas for a thick-walled cylinder subjected to a uniform, external pressure presented by /Obert and Duvall, 1967/. Bi-axial test results from borehole KA 3579G are illustrated in Figure 10-4. The elastic properties of the rock are required for the subsequent stress computations.

The complete stress tensor is determined from a successful three-dimensional overcoring stress measurement, i.e. a total of six independent components. Results can be presented in terms of e.g. principal stresses or components in any coordinate system.

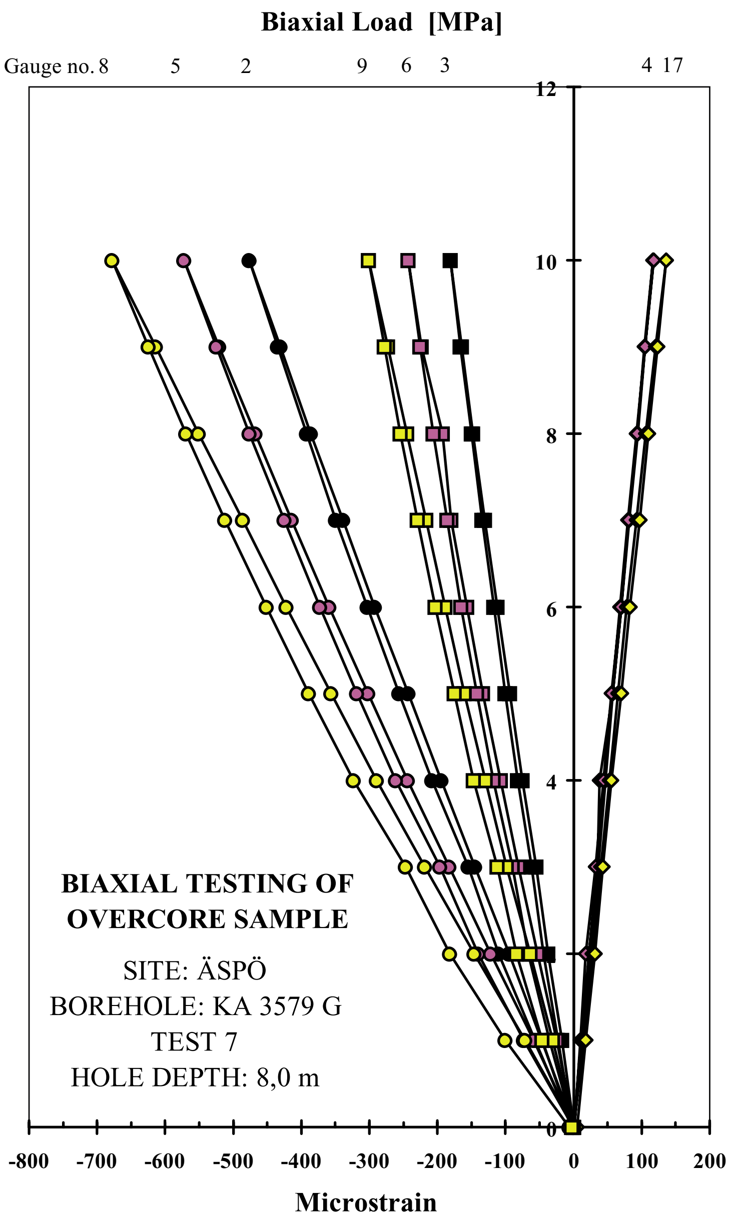


Figure 10-4. *Bi-axial test results from borehole KA 3579G.*

As each overcore measurement is a point measurement of rock stress involving a very small rock volume, at least three successful measurements at approximately the same location are recommended for calculating a representative average value of rock stress magnitude and orientation for that location.

The schematic data flow in overcoring stress measurements is shown in Figure 10-5.

10.2.5 Accuracy

The following sources of error associated with overcoring stress measurements can be identified:

- Rock properties.
- Measurement procedures.
- Analysis procedure.

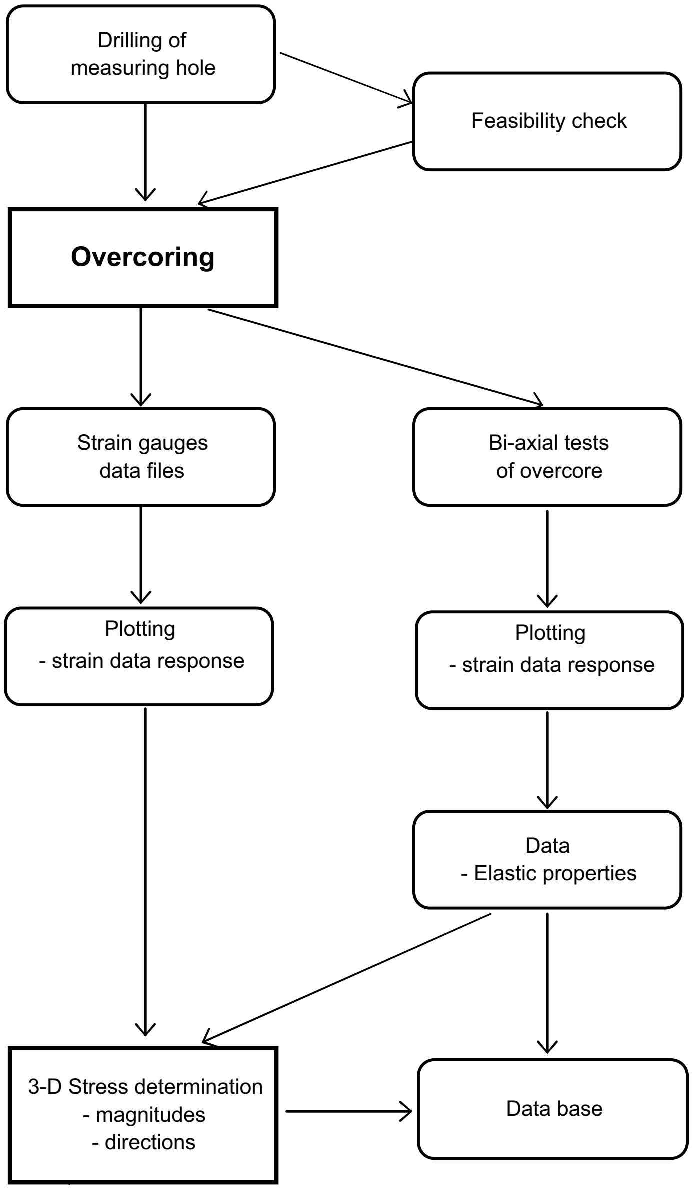


Figure 10-5. *Principal data flow for overcoring stress measurements.*

In discussing the accuracy of a measurement, it is necessary to distinguish between the degree of agreement between a measurement result and the real stress state in the point/location, and the representativity of the point measurement of the stress distribution in the rock mass. Since the overcoring technique involves only a very small rock volume in each measurement, it may well be that the point result from the measurement is in good agreement with the local stress state, but this local stress state may not be representative of the rock mass stress field.

The errors in an overcoring measurement can be divided into systematic and non-systematic errors. Experience and testing have shown that instrument errors (that is, the difference between the actual strain imposed on a discrete gauge and the corresponding readout) can be neglected under the given circumstances.

From measurements in rock that can be classified as almost ideal with respect to homogeneity and linear elastic behaviour etc, we know that the scatter in magnitude for a group of measurements at the same location is within a few MPa for magnitudes up to 25–30 MPa. Figure 10-6 presents results from measurements performed during the construction and operation phase.

The problem is to estimate the errors introduced by the fact that rock does not fulfil the assumptions of homogeneity and linear elastic behaviour. The material defects consist of heterogeneity, inelasticity and occasionally non-linearity. On the other hand, the redundancy existing in each measurement, and the calculation of mean values on a so-called measurement level, including several discrete measurements, effectively evens out the errors related to heterogeneity.

Poor stability prior to overcoring as well as drifting gauges after the drill bit has passed the location of the strain gauges may introduce errors. An error in the determination of the elastic properties due to poor bi-axial testing results will have an effect on the calculation of the stresses. As a general guideline, it can be stated that Young's modulus has a linear effect on the evaluated magnitudes, where a 10 GPa error in Young's modulus will introduce an error of approximately 1 MPa in the stress magnitudes. The effect of the Poisson's ratio on the stress magnitudes increases with a higher Poisson's ratio. For Poisson's ratios above 0.3 the effect becomes more dramatic and accelerating.

In conclusion it can be argued that overcoring rock stress measurements will give good, useful stress results, not without errors but with errors small enough not to invalidate the usefulness of the method for any application, under the following conditions:

- The equipment must be experimentally sound. This is checked by means of different types of testing such as bi-axial testing on materials with known properties. Any temperature effects can also be checked and compensated for in the in-situ measurement.
- The engineers must have long experience of overcoring measurements and geomechanics competence. Expertise in understanding rock behaviour is vital in the analysis of both overcoring- and bi-axial testing results.

It reveals that considerable scatter may be attributed to either measuring errors, or to local variations in the in-situ state of stress, or to a combination of both. It appears that the uncertainties associated with the overcoring technique are largely determined by local rock conditions. It is also known that the techniques have generally been found to be capable of producing highly reproducible results, provided that they are correctly applied in suitable rock formations, e.g. /Doe et al. 1982/.

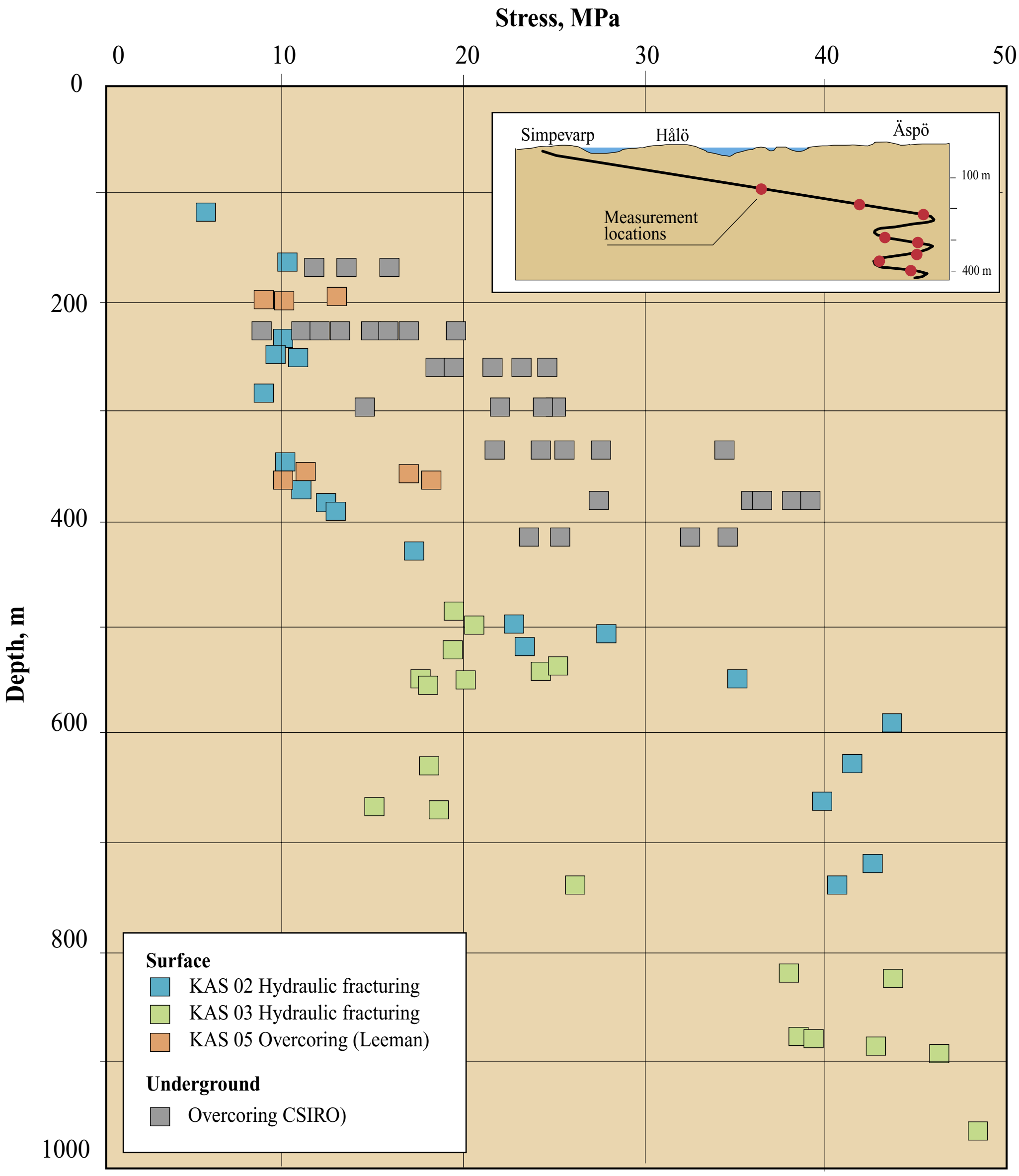


Figure 10-6. *Rock stress measurements in the Äspö HRL.*

10.2.6 Comments and recommendations

The field conditions for performing high quality measurements in separate niches along the ramp were good. At each site (borehole), a number of overcoring measurements were performed and analysed.

The response in each strain gauge to the unloading of the rock cylinder is measured in an overcoring measurement. The logging technique may differ between different types of overcoring equipment. Logging that allows a continuous recording of the gauge response from the start of overcoring until the end of coring and breakage of the core offers the best prospects for an evaluation. The process as such can be understood, and where fully or partially malfunctioning gauges can also be identified with greater accuracy.

Unfortunately, it is a common practise to quote a result for each measurement gauge, which is then compared with its neighbours. It must however, be understood that a single measurement result is insufficient to determine the sate of stress in the rock mass. The only way to analyse stress measurement results is to compare confident site measurements (compilation of point measurements). Even then there will be variations between locations in the investigated rock volume due to inhomogeneity in the rock mass. However, a general picture usually emerges.

Stress fields vary from point to point within a rock mass, more in some areas than in others. Often, detailed measurements only document this variability more accurately. The "average" stress field for a small volume may not change.

Virgin stress measurements should be viewed as a sampling problem in a very imperfect underground world of rock. Given the number of measurement locations and the number of individual stress measurements used to calculate each location result, the Äspö HRL stress field is, in the opinion of the authors, very well documented.

It must be recognised that overcoring involves only a very small rock volume in each discrete measurement, which may be regarded either as an advantage or a disadvantage depending on the purpose of the measurements. The advantage is that it allows the very local stress variation to be recognised and understood. In terms of rock mass stress field determinations, the involvement of the small volume is, however, a disadvantage. The latter must then be compensated for by conducting up to 4–5 measurements close to each other to identify stress variability and then afterwards calculating mean stress values to obtain the rock mass stress state.

Rock stress data have been compiled by /Leijon, 1995/. Examination of the data shows that:

- There are outliers, but overall data consistency is fair.
- The magnitude of the major principal stress is relatively high and increases rapidly with depth.
- The orientation of the major principal stress is sub-horizontal and NW-SE.
- The ratio between the major and intermediate principal stress is on average about 1.9, but ranges up to 3.0.

In other words, the state of stress seems to be highly anisotropic in the $\sigma_1 - \sigma_2$ and $\sigma_1 - \sigma_3$ planes. The level of stress anisotropy is generally much smaller in the $\sigma_2 - \sigma_3$ plane.

The fact that the ramp measurements were conducted at least 2 tunnel diameters away from the tunnel wall indicate that the influence of the tunnel opening on the measured stresses is small and less than the expected variation in the stress field between two neighbouring measurement points. In future measurements, however, attention must be devoted to determination of the elastic properties of the rock. A number of bi-axial tests yielded Poisson´s ratios above 0.3. Such values must be considered anomalously high and have a significant impact on calculated stress magnitudes /Myrvang, 1997/.

11 Groundwater monitoring

11.1 General description and purpose

With reference to Section 2.2 and Figure 1-5, monitoring of hydrologically related variables during the construction of the Äspö Hard Rock Laboratory tunnel is an essential part of the project's validation programme. Based on the pre-investigation results, groundwater models were developed /Wikberg et al. 1991/. Based on hydrogeology and hydrochemistry data, the changes in groundwater pressure and groundwater chemistry in surface boreholes during tunnelling were predicted, as were the changes in groundwater flow through the same boreholes and groundwater inflow to tunnel sections, see also Table 13-1 /Gustafson et al. 1991/.

The general purpose of groundwater monitoring was:

- To provide data for comparison with predictions and to provide additional information for detailed hydrogeological and hydrochemical characterisation and updating of models.

The variables monitored and presented in this chapter or elsewhere are as follows (see also Figures 8-1 and 9-1):

- Groundwater piezometry
  - piezometric levels in surface boreholes; see Section 11.2.3,
  - absolute pressure in tunnel boreholes; see Section 11.3.2.
- Groundwater inflow
  - water inflow to tunnel and shaft sections; see Section 11.4.2,
  - water flow in pipes; see Section 11.4.3,
  - water vapour transport in ventilation air; see Section 11.4.4.
- Electrical conductivity
  - of water in surface boreholes; see Section 11.2.4,
  - of water flowing into the tunnel; see Section 11.4.5.
- Groundwater flow
  - through sections in surface boreholes; see Section 8.8.
- Groundwater chemistry
  - monitoring sampling programme; see Chapter 9.

The groundwater monitoring programme started during the pre-investigation phase with piezometric levels and electrical conductivity in surface boreholes /Almén and Zellman, 1991/. This programme was gradually intensified with the inclusion of additional variables during the construction phase.

The entire system with borehole installations at the surface, tunnel installations, sensors and loggers connected to computer networks is called the Hydro Monitoring System (HMS), see /Almén and Johansson, 1992/. Generally speaking, it consists of a distributed computer and data logger network and specific components for the different measuring objects and types of variable to be monitored. The specific components are measuring gauges and other hardware such as packers, pipes etc, and will be described along with the different monitoring methods in Sections 11.2 to 11.4. The large number of monitoring points, more than 250, requires efficient data management. Most of the recording instruments are connected to the computer and data logger network in the HMS to allow on-line data acquisition, while additional recordings are made by stand-alone data loggers or manually, see Section 11.5. The management and presentation of data are described as well.

This chapter describes the groundwater monitoring programme and the HMS as they were at the end of the construction phase, see Figure 11-1.

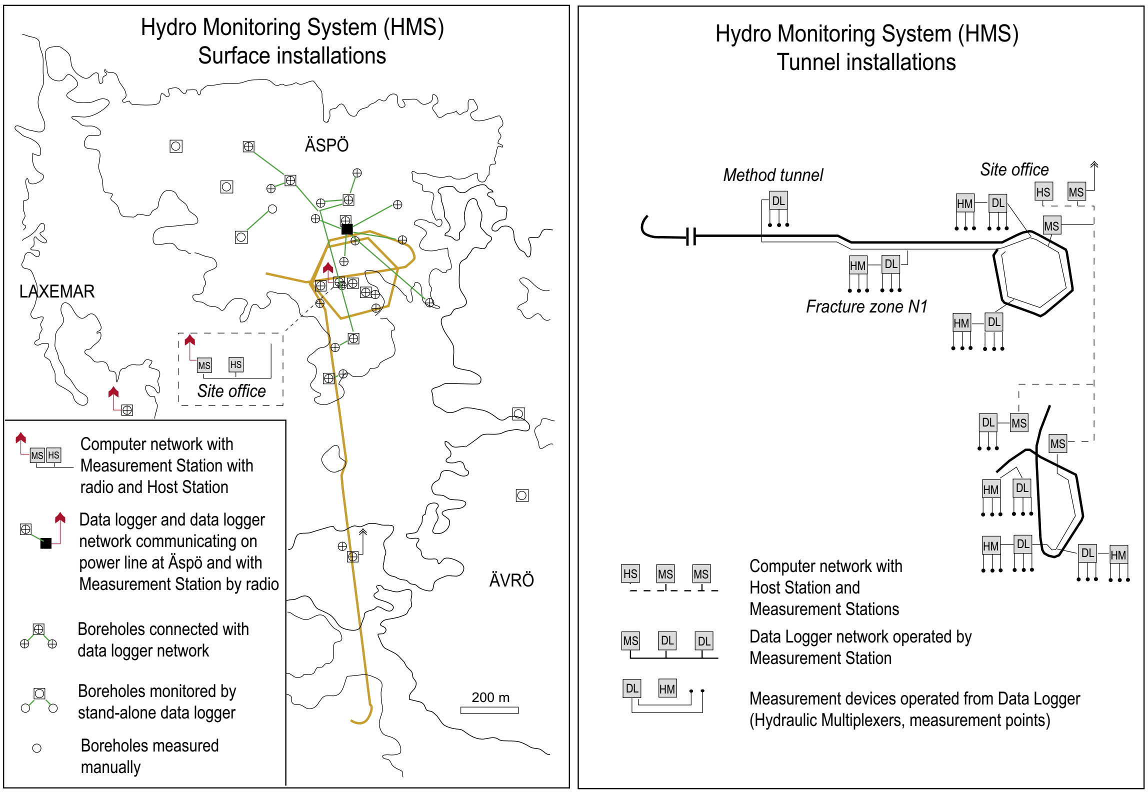


Figure 11-1. *Overview of the Äspö HRL groundwater monitoring system, 1997.*

11.2 Monitoring in surface boreholes

11.2.1 Introduction

The specific instruments used for monitoring in surface boreholes are described in this section. Among the variables monitored in surface boreholes, only piezometric levels and electrical conductivity are described. Groundwater flow is also a monitored parameter, as it was predicted for and therefore measured a couple of times during the construction phase as well as by the borehole instrumentation. However, as the groundwater flow method does not use the data acquisition system in the HMS and as the performance of the method is more related to the hydrogeological methods, it was described in Chapter 8. For a similar reason, monitoring of groundwater chemistry is described in Chapter 9.

11.2.2 Instrumentation

The borehole instruments and recording sensors are briefly described in this section. The data acquisition system including data loggers, communication devices, computers and software are described in Section 11.5.

For details on instrumentation in surface boreholes, see /Nyberg et al. 1996/ and /Almén and Zellman, 1991/. The instrument set-up in the cored boreholes is shown in Figure 11-2, while technical details are given in Figure 11-3. The instrumentation in the shorter percussion-drilled holes was less complex. In one borehole, monitoring was performed with another system, the Piezomac system.

Most cored boreholes are 56 mm in diameter and 200–1,000 m long. They are of the telescopic type, i.e. enlarged to approximately 155 mm in the uppermost 100 m, which allows for instrumentation of the type described below. The cored borehole equipment is mounted on and carried by a string of aluminium rods. The equipment is carried by a steel wire in the percussion boreholes.

Inflatable rubber packers divide most boreholes into different sections. Normally the instrumentation (number of packers) is more extensive in cored boreholes than in percussion boreholes.

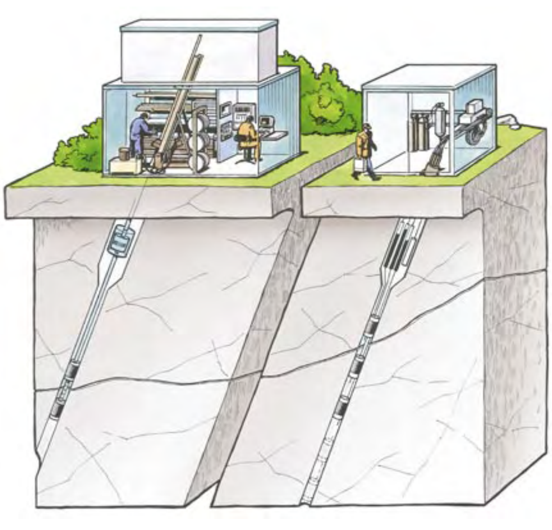


Figure 11-2. *Instrumentation for groundwater monitoring in surface boreholes. To the right, calibration by means of manual levelling.*

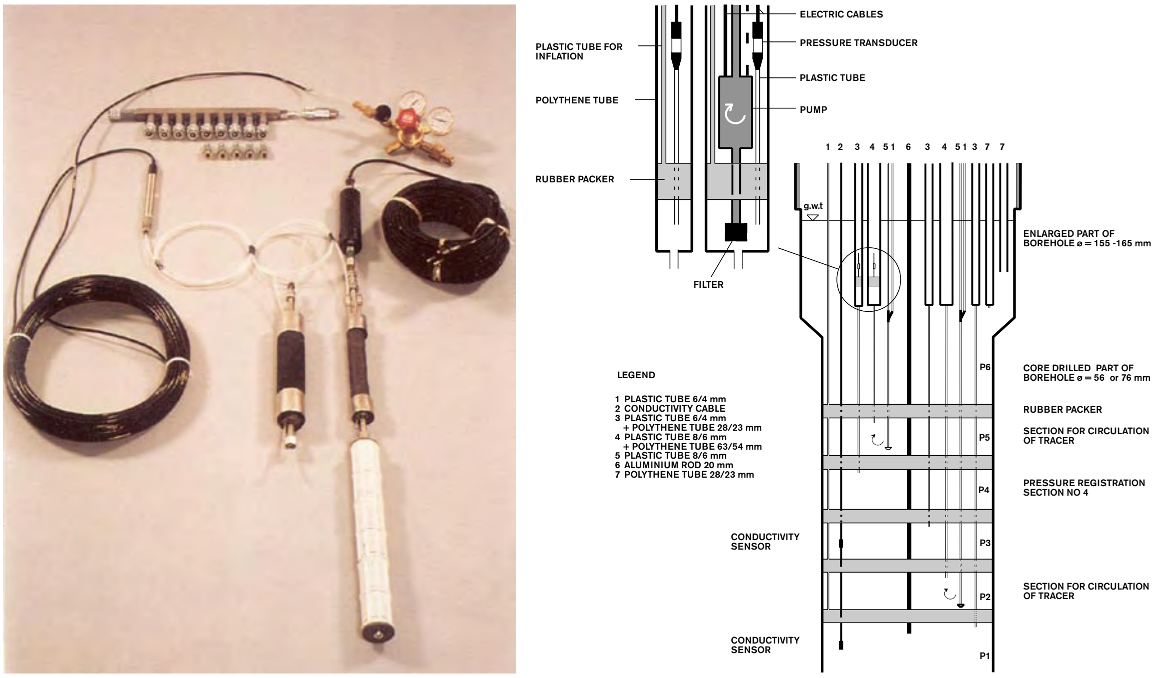


Figure 11-3. *Details of packer installation in a cored borehole.*

Each section has a hydraulic connection to a standpipe in the uppermost approximately 90 m of the hole. The hydraulic connection, with bypass through the packers, is a Tecalan tubing (OD/ID 6/4 mm). The standpipe (PEM tube, 28/23 mm or 63/54 mm) has a pressure transducer installed for water level measurement, normally a Druck PTX 160/D or PDCR 830. To achieve a rapid response to pressure changes in the actual borehole section a small packer is inflated below the water level in the standpipe. The pressure transducer is connected to the borehole section via a thin tube through the small packer. In the beginning of 1993, this small packer had to be removed in many sections to permit manual levelling. Percussion boreholes do not have this small packer installed.

One or two sections in most cored boreholes are equipped with a second tube between the section and the ground surface. This tube is of the same size all the way to the surface. In the upper enlarged part of the borehole the tube branches, and a third tube leads up to the surface for deaeration. The purpose of this special equipment in some sections is to circulate water during tracer injection tests or dilution tests. This circulation is maintained by means of a small electrical pump installed in the larger standpipe, see Section 8.8.

The packers in cored boreholes are inflated by water, pressurised by nitrogen gas, and in percussion boreholes by gas pressure only. The water-inflated packers are expanded with a pressure of approximately 15 bar, while the pressure in the gas-inflated packers varies depending on the surrounding hydrostatic pressure. To prevent freezing, denaturated alcohol is added to the water used to inflate the packers. The small packers can be inflated either way.

The electrical conductivity of the fluid is measured in two sections by conductivity sensors in most cored boreholes on Äspö. A signal cable from each sensor passes the borehole packers to the surface. The sensors are of a two-electrode type, made of gold and with a cell constant of 2.0. The measurements are not temperature-compensated.

11.2.3 Piezometric levels

Purposes

The purposes of monitoring groundwater piezometric levels are:

- To determine the hydraulic head in the rock mass under undisturbed conditions and to measure changes in the hydraulic head during construction of the tunnels in the Äspö HRL.
- To measure the pressure responses during all kind of hydraulic tests.

According to the general purpose of the monitoring programme, these data are used for comparison with predictions and model validation as well as for calibration of numerical groundwater models, for identifying major conductive structures and for evaluation of the hydraulic properties of major water-bearing zones. More specifically, monitoring of groundwater pressures in the rock mass was used for:

- Interpreting interference tests.
- Interpreting hydraulic responses during the excavation of the Äspö tunnel.
- Interpreting hydraulic responses during drilling from the tunnel.
- Measuring natural water pressures (undisturbed by the tunnel).
- Measuring drawdown during excavation.

Methodology

For details on monitoring methodology in surface holes, see /Nyberg et al. 1996/.

In most of the boreholes, the water level in the standpipes is measured with a gauge-type pressure transducer (relative to atmospheric pressure) connected to a data logger. In on-line-connected boreholes, the measurements are made on eight-minute intervals. The data was normally stored every second hour unless the change between the last stored value and the measured value exceeds 0.2 m water gauge. In boreholes with stand-alone data loggers, the measurement interval is two hours. To permit calibration of the monitored levels, manual levelling in the standpipes (with the small packers deflated) is carried out once every 1–2 months, see Figure 11-2. Monitored data are compared with the manual levellings and corrected to account for borehole deviation. If the two differ, calibration constants are changed and the procedure is repeated until an acceptable fit is achieved. The measurement interval for boreholes that are only manually recorded is also 1–2 months.

The instrumentation is checked in connection with manual levelling once every 1–2 months. Alarms on measurement computers in the HMS (see Section 11.5) are checked frequently (many times a week) to discover failures of transducers and other equipment.

The final results are presented in metres above sea level. This requires deviation measurements (x,y,z coordinates) along the borehole and at the height of the casing. Sometimes it is of interest to determine the absolute pressure at the top of a packed-off section. This value can be calculated if the vertical distance from the top of section to the water table in the tube connecting the section with the ground surface and the density of water in the tube are known, see /Rhén et al. 1994a/.

On 3–6 occasions every year, all data are scrutinised and obviously erroneous data are deleted, see Section 11.5.2.

Accuracy

Errors in groundwater level estimates may be a combination of errors in:

- Pressure gauge readings (for example hysteresis, non-linearity, movements of the pressure transducer and erroneous transducer).
- Levelling of the borehole casing.
- Levelling of the borehole groundwater surface (for calibration purposes).

When calculating the absolute pressure at the top of a packed-off section, errors due to uncertainty in the estimation of the density of water in the tube connecting the section with the ground surface must also be accounted for. Errors related to borehole deviation measurements may also contribute to the total uncertainty.

The total error under hydraulically undisturbed conditions has been estimated to be roughly ± 0.05 m for percussion boreholes and top sections in cored boreholes, and ± 0.15–1 m for packed-off sections in cored boreholes. The higher figure for packed-off sections stems from greater difficulties in levelling the water surface in the 28/23 mm PEM tubes; a special water level meter with lower accuracy has to be used. The lower limit in the interval applies to the uppermost section, where an extra tube for levelling purposes (with no transducer cable) is normally installed.

As a consequence of substantial drawdowns in many boreholes, the manual levellings were more difficult to carry out. The errors in these boreholes may therefore be larger than stated in the previous paragraph. Errors caused by failure of the mechanical or electronic equipment in boreholes are to some extent eliminated, but sometimes they are difficult to recognise and may therefore reduce the reliability of data for shorter periods.

When only relative level changes during shorter periods are considered, for example during some hydraulic tests, the errors are considerably smaller.

Comments and recommendations

Using standpipes offers some benefits compared with other systems. Replacing a failed pressure transducer is easy without necessitating lifting out all the instrumentation. The transducers can easily be calibrated by manual levelling. It is possible to use pumps inside the standpipes for water sampling, as well as for circulation of water in a section (for tracer tests). These pumps are mobile and can easily be replaced and repaired.

The gauge-type pressure transducer requires a thin plastic tube to the surface to provide atmospheric pressure as a reference. Water vapour transport into this tube sometimes causes condensation that interrupts the contact with atmospheric pressure and causes damage to the transducer after some time. To overcome this problem, two vent lines were sometimes used (to permit circulation of dry air), but nowadays failed transducers are replaced with absolute pressure transducers. This means that atmospheric pressure needs to be measured so it can be subtracted.

During the autumn of 1992, as a consequence of the tunnel excavation work, substantial drawdowns were observed in many borehole sections on Äspö. In some sections the drawdown exceeded 100 m. As the standpipe is about 100 m in length, transducer measurements as well as manual levelling were then impossible in such sections. But even with less drawdown, if the pressure outside the wide tube of the upper part of the standpipe is considerably higher (several tenths of meters) than inside, the tube may become compressed. This also prevents manual levelling and replacement of equipment inside the tube. To overcome this, the wide tubes must be stiffer. Another way is to use smaller transducers, permitting smaller tubes, but then it will be impossible to pump for sampling and circulation. Both alternatives have been studied.

When the distance to the groundwater table increases, manual levelling may be difficult due to friction between the cable and the tube.

When the vertical distance to the water table becomes too great, it is also impossible to deflate the hydraulically inflated packers due to the hydrostatic pressure from the water-filled tube connecting the packer to the surface (the inflation line). This is a problem both for the packers in the borehole and the small PEM packers in the standpipes. In the borehole packer system, this problem has been solved by use of a second inflation tube. With such a tube it is possible to remove the water from the system by replacing it with gas. The PEM packers expand at approximately 60 m of hydrostatic pressure and the borehole packers at approximately 45 m. The PEM packers in the tubes have therefore been exchanged for gas-inflated packers in some boreholes.

Another problem found is clogging in the small standpipes and the inflation lines. This has been found in a few boreholes and after long installation periods (several years). The clogging material seems to be chemical precipitates or bacteria, or a combination thereof, and may be dependent on the plastic material in the tubing. The problem has not yet been fully solved.

The multipurpose monitoring set-up was sufficient for the pre-investigation phase. However, during the construction phase, the groundwater pressure measurement system did not meet the needs for sections with large drawdown. This problem requires further study before a monitoring system can be selected for a new site investigation.

11.2.4 Electrical conductivity

Purpose

Where saline water is present at depth, the saline/fresh water interface may change during hydraulic tests or tunnel excavation. Such changes were quantified in the predictions for the Äspö HRL construction phase, so the purpose for monitoring electrical conductivity was:

- To record changes in the salinity (in relative values) for comparison with predictions.

By "relative values" is meant that the accuracy of the measuring system does not allow data to be used as absolute measures of salinity. This has to be determined by water sampling.

Methodology

For details see /Nyberg et al. 1996/.

Electrical conductivity was measured with sensors in most of the cored boreholes on Äspö, and in two of the monitored sections. The deeper sensor in each borehole is connected to the data acquisition system and a value is measured and stored every 4th hour, while the upper sensor is read manually once a month.

Calibration is carried out at the surface, with the cables connected, before installation in the borehole. A two-point linear method was used for the most part (at 667 and 5,864 mS/m). In KAS05 and KAS11 (from June 1992), a second-degree polynomial is fitted to a four-point calibration (at 141.7, 600, 1,290 and 2,482 mS/m).

Accuracy

The electrical conductivity sensors are strongly non linear and the conductivity at measurement depth is not known when calibration is done. Unfortunately, the two-point method gives a poor result, since the calibration range is too wide in relation to the non linearity of the sensors. The second-degree polynomial fitted to a four-point calibration gives a much better result.

Because of the poor calibration, problems with the electrical conductivity sensors (sudden jumps in the reading), and difficulties checking the sensors after instrumentation, one must be very careful when interpreting the results. The absolute values are very uncertain.

Despite this, it is possible to draw some conclusions concerning changes in electrical conductivity within single sections. It is not advisable to compare absolute values from different sections.

Comments and recommendations

The purpose of conductivity monitoring was to record changes, not to obtain absolute values. The recording fulfilled that purpose to some degree; there were changes that certainly did not correlate with the aquifer. During the course of the programme, a need also arose to use absolute values. However, accuracy was not sufficient for that purpose. Due to the non linearity of the sensors, more than two calibration points are necessary since the measuring range can be wide. Regular water sampling to get laboratory values of electrical conductivity from the actual sections would increase the reliability of measured values considerably. However, sampling from the sections may be difficult, mainly due to difficulties in obtaining a representative sample. This problem is less pronounced in a circulation section than in a single pipe section. The problem is accentuated in sections with salinity stratification.

11.3 Monitoring in tunnel boreholes

11.3.1 Introduction

Instruments and methods used in tunnel boreholes are described in this section. Only groundwater pressure is measured in these boreholes.

11.3.2 Groundwater pressure

Purposes

The purposes of groundwater pressure monitoring in tunnel boreholes are almost the same as for the groundwater piezometric levels:

- To measure the groundwater pressure changes in the rock mass during and after construction of the Äspö HRL.
- To measure the pressure responses during all kinds of hydraulic tests.

For further discussion of how data was used, see Section 11.2.3.

Instrumentation

Instrumentation in tunnel boreholes may be of different types. In investigation boreholes with more than one section, the packers dividing the borehole are always of the hydraulic type. One-section boreholes (investigation holes or probe holes selected for monitoring) may have either a mechanical packer or a valve mounted on the borehole casing, see also Chapters 5 and 8. The hydraulic packers are inflated by means of nitrogen gas pressure over water in a pressure vessel connected to the packer system. All borehole instrumentation is anchored to the tunnel wall, for safety reasons.

Usually, the pressure in a borehole section is transmitted via a plastic tube and a hydraulic multiplexer to a pressure transducer. Several sections are connected to the same pressure transducer via the multiplexer, see Figure 11-4.

The multiplexer has up to 16 magnetic valves that open towards the pressure transducer one after another for all sections connected. Two of the inlets to the hydraulic multiplexer are reserved for reference pressure to permit calibration of the pressure measuring system. A data logger, the same logger that collects data from the pressure transducer, operates the valves.

a)

b)

c)

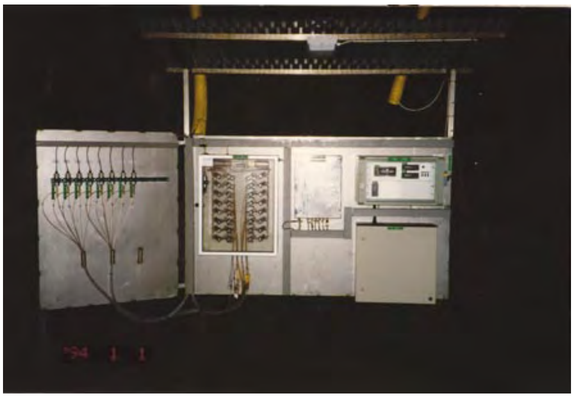


Figure 11-4. *Equipment installed in the tunnel for groundwater pressure measurements with a hydraulic multiplexer, connected to a Borre data logger and further to a measurement station. The photos show:*
a) the reference pressure tank at tunnel section 2,850 m,
b) the reference pressure station with BAT connectors for four reference pressures and
c) the monitoring station with hydraulic multiplexer and Borre data logger.

The reference pressure system consists of calibration vessels at four carefully levelled locations and tubes connected to the hydraulic multiplexers. These vessels are situated at tunnel lengths of 120 m (level –15 m), 650 m (–89 m), 1,191 m (–163 m) and 2,850 m (–377 m). The system is filled with de-ionised water to give well-defined pressures. An air tube connected to the top of the calibration vessels delivers barometric pressure from the surface.

One limitation of the hydraulic multiplexer system is that the measurement frequency is sometimes not high enough for interference tests. Prior to some specific tests, a number of borehole sections were temporarily connected to individual pressure transducers instead of being connected to a hydraulic multiplexer. In these cases a number of transducers were mounted on a panel where tubes from the reference pressure system were available to permit in-situ calibrations. In addition, a few sections were continuously connected directly to an individual pressure transducer.

Methodology

The pressure transducers in the hydraulic multiplexers and in the pressure transducer panels are all of the absolute type (i.e. they measure the total pressure, not relative to atmospheric pressure). Transducers in the Druck PTX 600 series are used. The pressure is normally measured every 4th minute but stored only every 2nd hour. If the change since the latest stored value exceeds a predefined amount (1–2 kPa), the measured value is stored. During interference tests, measurement/storage frequency may be higher and the conditional value for storage may be lower. However, the measurement frequency for the multiplexers is limited by the time needed to stabilise the section pressure to the pressure transducer after valve opening. A delay time of 10 seconds (adjustable 0–999) between valve opening and measurement is therefore used for each section connected to the multiplexer.

The HMS using reference pressures automatically calculates offset and gain for the pressure transducers. These values may, after examination, be used as calibration constants.

Results are presented as total pressure on the pressure transducer (in kPa), which includes atmospheric pressure and the pressure resulting from the tube between the section and the pressure transducer. If the densities of the water in the tube and in the aquifer, the atmospheric pressure and the levels of the section and the pressure transducer are known, hydraulic head can be calculated for a specific location in a section. In the HMS, a certain algorithm has been used to calculate head values for the middle of each section /Rhén et al. 1994a/.

Three times a year, graphs of offset and gain are examined and the calibration constants are, if necessary, adjusted retrospectively. At these times, pressure data for all sections are also scrutinised and obviously erroneous data are deleted, see Section 11.5.2.

Accuracy

The following sources of errors are identified for groundwater pressure monitoring in tunnel boreholes:

- Pressure measuring instruments and methodology.
- Inappropriate delay time.
- Calibration constants.
- Level of borehole section and pressure transducer.

The accuracy of the measuring instruments is mainly a function of the accuracy of the pressure transducer. According to the manufacturer, the combined effect of nonlinearity, hysteresis and repeatability results in an error of ± 0.08% F.S. (full scale reading). This means ± 4 kPa for a 0–50 bar pressure transducer. The pressure ranges for the transducers used at the hydraulic multiplexers are 0–40 or 0–50 bar.

The delay time error is eliminated if the time between valve opening and measurement in the hydraulic multiplexer is long enough. The choice of a 10-second delay time is a compromise between the desire to measure rapid pressure responses and the need for pressure equalisation between measurements in the hydraulic multiplexer.

The error in the calibration constants is associated with the status of the reference pressure system: the accuracy of the levels in the calibration vessels and the pressure transducers, the estimate of the density of the water in the tubes, and the possible presence of air in the system. The inaccuracy associated with the calibration constants is estimated to be approximately ± 7 kPa.

To calculate the absolute pressure at a certain location in a section, errors in estimates of the density of the water in the tube between the section and the pressure transducer must be taken into consideration. The accuracy of the the pressure transducer levels and of the location in the section for which pressure is to be calculated must also be taken into account.

Comments and recommendations

The use of a hydraulic multiplexer offers some benefits compared with individual pressure transducers. One single transducer is used for the entire multiplexer. This transducer measures 14 different borehole sections, resulting in a substantial cost reduction. When this single transducer is calibrated, the result is valid for all connected sections. Two of 16 channels on the multiplexer are reserved for reference pressure, permitting a semi-automated calibration procedure.

The limited measuring frequency for the multiplexer, dependent mainly on the delay time, was discussed in the methodology section above. Many factors affect the delay time:

- Hydraulic conductivity of the section.
- Length of the section.
- Length of the tube between the section and the hydraulic multiplexer.
- Magnitude of deviating pressure inside the hydraulic multiplexer.

Increased delay time decreases the risk of error in the pressure measurements at the same time as the maximum measurement frequency declines. Normally the 10-second delay time is long enough, but for some tests in short sections with low hydraulic conductivity it was not.

On a few occasions, leaking magnetic valves on the hydraulic multiplexers were replaced.

A mechanical packer with three short measurement sections was developed and tested in the TBM tunnel. As explained in Section 5.3.3, 8-m-long percussion-drilled boreholes were drilled at an angle of 45° from the tunnel line. The aim of the installation was to study the pressure profile in the rock near the tunnel wall. However, it turned out to be very difficult to install the packer system, as the packer system was stiff and the boreholes were not straight enough. There were also problems with leakage due to the borehole wall surfaces.

In several cases the borehole, changed direction slightly at the position were a new drill rod was added, resulting in a curved borehole. Cored boreholes would probably not cause these problems.

11.4 Monitoring of water inflow to the tunnel system

11.4.1 Introduction and purposes

Water inflow to the tunnels and shafts is of key interest in evaluating the hydrogeological modelling results.

The inflow of groundwater to the tunnel system was the subject of predictions based on pre-investigation hydrogeological modelling. The purposes of monitoring water flow into the tunnel and shafts were therefore:

- To obtain data on water inflow to different tunnel sections as a function of time as the excavation progressed for comparison with predictions.
- To study methods for water balance measurements.
- To provide information for numerical modelling.

The monitoring consists of the following methods described in Sections 11.4.2 to 11.4.5:

- Monitoring of groundwater flow from the rock into tunnel sections and shaft sections.
- Monitoring of water flow in pipes.
- Monitoring of water vapour transport.
- Monitoring of electrical conductivity of water flowing into the tunnel.

The water inflow to the tunnel and shaft was measured in delimited sections: 18 in the tunnel and 3 in the shaft.

11.4.2 Water inflow to tunnel and shafts

Instrumentation

The water inflow along the tunnel is collected at certain locations by dams across the tunnel and diverted to a gauging box equipped with a v-notch weir (Thomson weir). A pressure transducer calibrated against a ruler mounted on the box measures the water level in the box, see Figure 11-5. After passage through the gauging box the flow is diverted to a discharge pipe shared by a number of gauging boxes which finally leads into one of the pump sumps in the tunnel, see Figure 11-6. The pressure transducers in all the v-notch weirs are connected to the HMS.

Methodology

Water levels in the gauging boxes are used in the HMS to calculate flow rates by means of a discharge equation, expressing flow rate as a function of level. Normally the level is monitored every 10th second but stored only every 30th minute unless the change since the latest stored value exceeds a predefined value. The value is usually 1 mm, but due to oscillating levels in some gauging boxes this value had to be increased to avoid sampling too much data. Flow rates in the HMS are given in m^3/s.

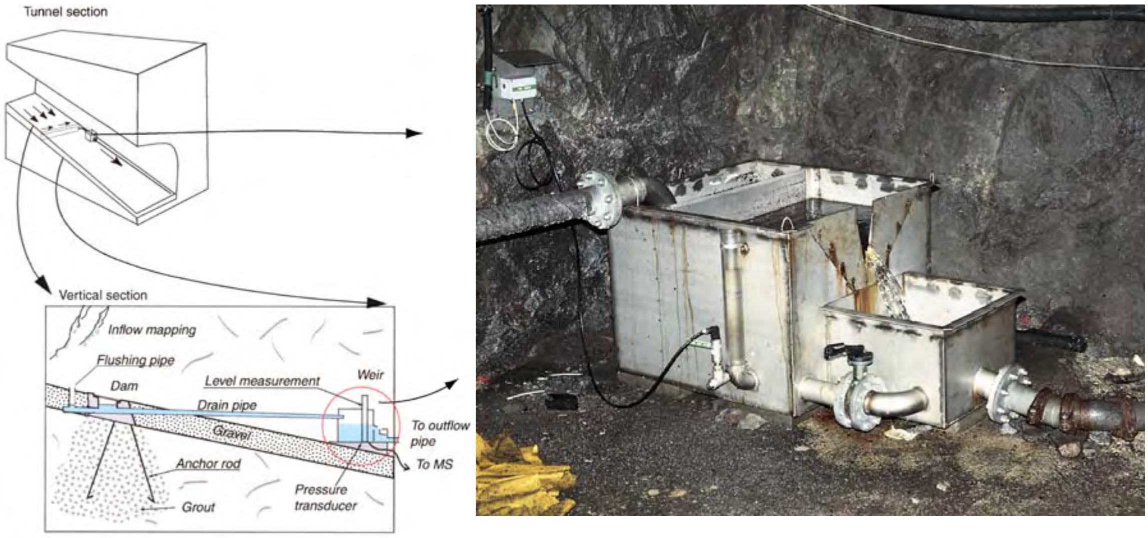


Figure 11-5. *The system for measuring water inflow to a section of the tunnel. Illustration of construction the ditch and dam across the tunnel floor and v-notch veir for flow measurement.*

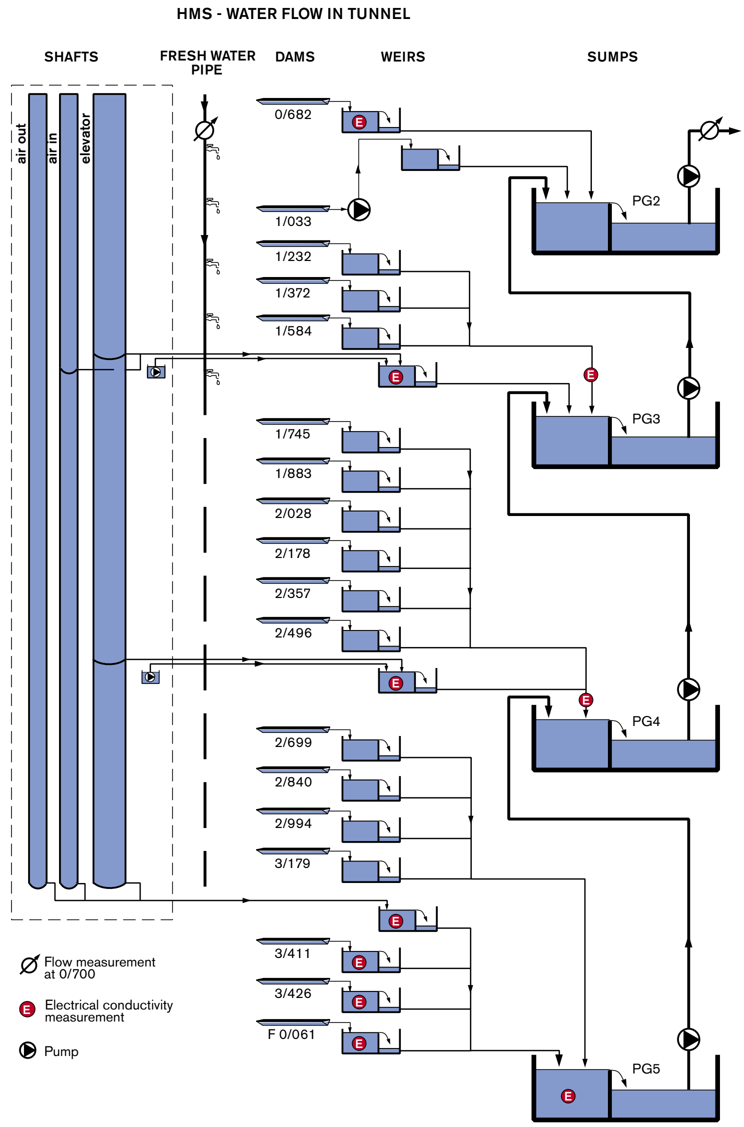


Figure 11-6. *Layout of the drainage system of the Äspö tunnel, with flow measurement stations, pump sumps, etc.*

Initially the discharge equation for a weir is determined. The flow rate is measured at four different levels on the ruler. The pressure transducer is then calibrated against the ruler by altering the level in the box. The reason for this two-step procedure is to avoid a new determination of the discharge equation every time a pressure transducer has to be replaced or if the calibration equation for the transducer is changed.

The levels in the gauging boxes are read manually once a month to permit adjustment of the calibration constants for the pressure transducers. Once a year the discharge equation is checked by a field measurement of the existing flow rate, and if necessary a new discharge equation is determined.

Accuracy

The following sources of errors are identified:

- Water level uncertainties.
- Discharge equation for the v-notch weir.
- The maintenance of the gauging box.
- Clogging in the connection between the gauging box and the pressure transducer.

The water level uncertainty is composed of errors in the transducer and errors in the annual readings of level in the gauging box. The error due to the transducer is insignificant compared to the error due to uncertainties in the manual readings of level.

The absolute error in flow rate caused by erroneous level reading is to a high degree a function of the magnitude of the flow, owing to the non-linear relationship between level and flow. In relative terms this means that the error at the mean flow rate existing in the tunnel is approximately ± 5%, while at low flow rates the error may be as high as ± 10%. However, obstacles on the weir and sediments in the dams may cause considerably higher errors.

When the flow rate does not deviate too much from the interval where the measurement points were selected to determine the discharge equation, the error due to the equation is within a few percent.

Maintenance of the v-notch weirs and dams is important. If there are obstacles or coatings on the weir, the relationship between level and flow rate is affected.

Relocating the transducer alleviated initial problems with mud clogging the pipe where the transducer is mounted. Still, regular maintenance is important, especially during excavation work. Taking regular readings of the ruler to correct for time drift and other changes in the calibration constants for the transducer is also necessary.

Comments and recommendations

The discharge equations for the v-notch weirs have been determined with a very good fit.

One problem at the Äspö HRL was the delay in the construction of the dams and other facilities for measuring flow rates from the dams. For practical reasons, it proved difficult to construct a dam closer than about 150 m from the tunnel face without it interfering too much with the excavation work. Therefore, a number of dams were constructed far beyond the tunnel face, which of course made the estimation of the flow (as a function of time) into the tunnel uncertain and cumbersome. Measurement of the flow into the tunnel can, and should, be carried out in a better way than was done at the Äspö HRL. It should also be remembered that more frequent measurements in space and time would also interfere with the contractor's work, and that making a dam of good quality is quite expensive.

The devices that measured the flow rates from the dams were Thomson weirs, allowing for manual measurements and continuous monitoring by the HMS. However, experience has shown that regular maintenance of the equipment and readings of the ruler are important in order to assure good quality of the flow rate data. During tunnel excavation, the trucks hauling out the shot rock give rise to dust, causing sedimentation in the dams and the weirs. Proper maintenance procedures must be instituted so that cleaning of the dams and weirs is done when needed. Otherwise the pressure readings from the pressure transducer mounted on a weir may contain errors due to incorrect pressure measurement or the fact that the conditions for the calibration curve for the weir are not fulfilled. If the dams fill up with sediment, the water just flows over the dam to the next dam.

11.4.3 Water flow in pipes

Instrumentation

Both incoming pure water and pumped-out drainage water flows in pipes. The flow is measured by acoustic "clamp-on" type flow meters, clamped on the pipes for incoming water (on top) and for pumped-out water (below), see Figure 2-7. The sensors are situated approximately 700 m from the tunnel entrance, close to (and before) the first water inflow station.

Methodology

The flow of incoming pure water varies from zero to several litres per minute. This flow meter is calibrated with a watch and a bucket.

The drainage water is pumped from one sump to another towards the surface, see Figure 11-6. From the top sump the water is pumped into the sea. The pump in each sump works at maximum capacity until the sump is emptied, then the pump shuts off until the sump fills up again. The outflow is measured after the uppermost sump at 700 m. This means that the flow is either zero or roughly 60 litres per second. This meter is calibrated by measuring the level difference in the sump for a period and by knowing the area of the sump, from which the mean flow rate can be calculated.

Both of these flow meters measure very frequently, every 5th and 10th second (for pumped-out and incoming water, respectively), but the value is only stored if a significant change has taken place. The threshold value for the incoming water is 0.05 l/s and for the pumped-out water 1 l/s.

Accuracy

Two sources of errors could be identified:

- The flow meters.
- The calibration procedure.

Using error estimates provided by the manufacturer of the flow meters is not satisfactory. Material constants must be known for different pipes, and the errors caused by using incorrect constants are unknown. The pipes consist of different material layers and may be coated on the inside. Calibration is therefore necessary.

Measurements of incoming consumption water achieve quite good accuracy, and calibrating the meter by means of the described method is quite easy. A rough estimate of the error is ± 5–10% of the measured flow rate.

The error in the pumped-out water flow rate depends mainly on the error in the sump area estimate used to calculate volumes when calibrating the flow meter. Since there is no documentation on the technique used to estimate the sump area, it is difficult to give a value for this error. Assuming that the area is a rectangle, and that the error in the length estimate is ± 0.01–0.03 m, the resulting error in the flow rate will be ± 2–5%. Including errors in the flow meter, a figure of ± 5–7% may be a fairly good guess.

Comments and recommendations

It is necessary to work in close co-operation with the contractor to facilitate good measurements of the incoming and pumped-out water. During the Äspö HRL construction phase, there were a few interruptions in the measurements of the pumped-out water due to problems with the drainage system. The solution to the problem was using a temporary pumping system without any flow measuring devices. Problem of this kind should be expected and the measurement system should therefore be flexible in order to minimise interruptions in the measurements.

11.4.4 Water vapour transport in the ventilation air

Instrumentation

Transport of water vapour has been estimated at a tunnel length of approximately 700 m, both with the incoming air in the ventilation tube and with the outgoing air in the tunnel. The vapour is calculated from measurements at single points of air velocity, air temperature and air humidity in the tunnel and ventilation tube, respectively.

The sensors are placed in the middle of the tube and the tunnel sensors are placed close to the wall of the tunnel, see Figure 11-7.

Methodology

Measurements of wind velocity have been used to calculate the flow of air in the tube and in the tunnel. The relationship between the air velocity in a point and the flow of air in the cross-sections of the tunnel and the tube is determined by calibration measurements. These measurements are performed in a grid across the cross-sections. Air temperature and humidity are used to determine the water vapour content of the air. Temperature and humidity were found to be almost equal in the entire cross-section. By means of these measurements and calculations, it is possible to determine the vapour transport across the entire cross-section. The principle is the same for both the tunnel cross-section and the tube cross-section.

Accuracy

The main sources of errors are:

- The measuring instruments.
- Insufficient reliability of measurements.

No systematic estimation of different errors has been performed.

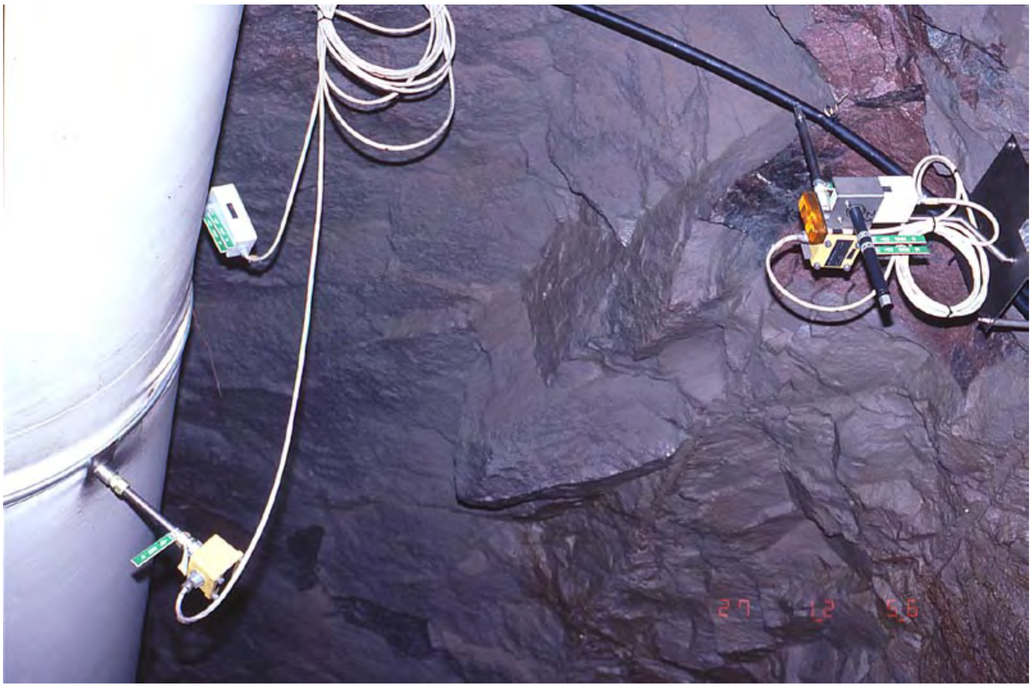


Figure 11-7. *Installation for measurements of vapour transport in ventilation air. Sensors are placed in the middle of the tube and tunnel sensors are placed on the wall.*

Measurements of air temperature and air humidity are quite accurate and the approximation of equality across the whole cross-section is quite good both in the tube and the tunnel.

Measuring the air velocity with tolerable accuracy is quite easy, but using a single point measurement to estimate the air flow in the whole tunnel cross-section may be questionable. In the tunnel, it is necessary to place the sensor close to the wall (due to haulage traffic in the tunnel), which results in poor representativity for the whole tunnel cross-section. The low air velocity in the tunnel also affects accuracy. Calibration of the air flow in the tunnel is necessary. For air flow measurements in the tube, the sensor is placed in the middle of the tube, providing good accuracy.

Comments and recommendations

The sensors for humidity and temperature do not work properly after a certain length of time, probably due to the diesel fumes. It has been necessary to replace them after a couple of years.

Dust, grass and other matter have obstructed the wind sensor in the tube. Frequent checks and rinsing a couple of times every year has been necessary.

After boring of the ventilation shafts and the hoist shaft, the air flow in the measured section at 700 m is not the only air flow out of the tunnel, and measuring the flow in these three shafts is very difficult and was never carried out. These measurements were therefore discontinued some years after the boring of the shafts. The decision not to install any recording instruments in the shafts also stemmed from the knowledge that the water transport in the ventilation air was a negligible part of the total water balance in the tunnel, see /Rhén et al. 1997b/.

11.4.5 Electrical conductivity

Purpose

As described in Section 11.2.4, changes in the salinity of the groundwater due to excavation were modelled and predicted. The purpose of measuring the electrical conductivity of the inflowing groundwater to different tunnel sections was:

- To contribute to the determination of salinity changes in the formation for comparison with predictions.

Knowledge of the salinity, or changes in the salinity, of the inflowing water is essential in determining the origin of the inflowing water and for numerical groundwater flow modelling.

Instrumentation

Electrical conductivity is measured with a 4-electrode conductivity meter, consisting of a housing box with an electronic unit and an integral sensor. The meter is mounted either on a gauging box for flow measurements or on the common discharge pipe that conducts water from the gauging boxes to the sump, see Figure 11-8.

Methodology

The electric conductivity meter is connected to a logger on the HMS. A value is measured and stored once every hour.

Once a year the meters are calibrated using four buffer fluids with well-defined electrical conductivities.

Figure 11-8. *Installation for electrical conductivity measurements in a Thomson weir, representing the conductivity of water flowing in from a tunnel interval.*

Accuracy

The following sources of errors are identified:

- The measuring instrument.
- Coatings on the sensor.
- The buffer fluids used to calibrate the instrument.

No careful calculations have been made of errors, but a rough estimate gives a figure of around ± 10 mS/m.

Comments and recommendations

Initially there were some problems due to an unusual output signal from the conductivity meter, but after this was solved the system seems to work well. The sensors must be cleaned regularly.

11.5 Data acquisition system

11.5.1 General

As mentioned in Section 11.1, the data acquisition functions allow for efficient management, quality control and presentation of measured variables. Most of the measurement points are monitored on-line by means of a computer and a data logger network, while others are measured by stand-alone data loggers or manually.

Development and initial installation of the Hydro Monitoring System (HMS) took place in 1991, when the tunnel front was at about 900 m. The design criteria and objectives of the HMS were:

- To rapidly identify, at the site office or remote workplaces, hydraulic responses caused by tunnel excavation.
- To serve as a monitoring system for various hydrogeological tests.
- To fulfil the legal requirements on monitoring of hydrological data.
- To permit easy functional checking of the entire system.
- To be a central QA tool.
- To provide efficient collection, management, storage and presentation of large data records.

Since the initial installation the HMS has been gradually expanded to keep up with the progress of the tunnel, see further in Section 11.5.2.

For details on the HMS, see /Nyberg et al. 1996/, /Almén and Johansson, 1992/ and /Almén and Zellman, 1991/.

11.5.2 Data management

Instrumentation

The system now consists of four measurement stations connected by a computer network. One station is a host station at which all data are collected once a week. In the initial HMS installation (1991), the host station and one measurement station were located in the temporary site office by the tunnel mouth, and one measurement station was located in the tunnel /Almén and Johansson, 1992/. The host station is connected to the Ethernet LAN in the Äspö HRL, which is in turn connected to SKB's corporate Ethernet in Stockholm, see Figure 11-9. Two kinds of data are managed:

- Manually read data.
- Data monitored by a sensor.

These data enter the system in two ways:

- On-line data loggers are frequently polled for new data.
- Manually read data and manually dumped data from data loggers are entered into the system as data files.

Data loggers connected on-line to a measurement station are connected via a radio, a power line modem or a data logger network (BorreNet).

The on-line system is designed to handle interruptions in communication. Data can be stored in data loggers and in measurement stations, in a data logger for at least five days and in a measurement station for at least four weeks. The host station has stored all data since on-line monitoring started in 1991. Backup of the stations to tape is done every week.

Most data loggers are of the BORRE type, a multichannel data logger with a 16-bit A/D converter. Most of the BORRE loggers communicate with a measurement station via radio or power line modem, but some are manually dumped into a portable PC. One type of BORRE logger can operate the magnetic valves on the hydraulic multiplexer, see 11.3.

Some data loggers are of the GRUND type, a single channel data logger with a 13-bit A/D converter. All of the GRUND loggers are manually dumped into a portable PC.

Two data loggers are of the PIEZOMAC type, a multichannel data logger with a 15-bit A/D converter. Both these data loggers are manually dumped into a portable PC.

Methodology

The measurement station includes a powerful software package for measurement and data processing. An overview of data flow and data management within the HMS is shown in Figure 11-10.

Data are measured at set intervals, but the value is not stored unless it differs by a certain amount from the latest stored value. A value is always stored at a specified interval regardless of whether it differs enough or not. For each measured channel, it is possible to use second-degree calibration constants with time drift in both offset and gain. The quadratic term is often used for electrical conductivity and the time drift in offset is frequently used for level data from the GRUND data logger in surface boreholes. Different sets of calibration constants are used during different periods, which means that many different sets of calibration constants exist for the same channel. Each set of constants may be changed at any time, even long after the measurements have been made.

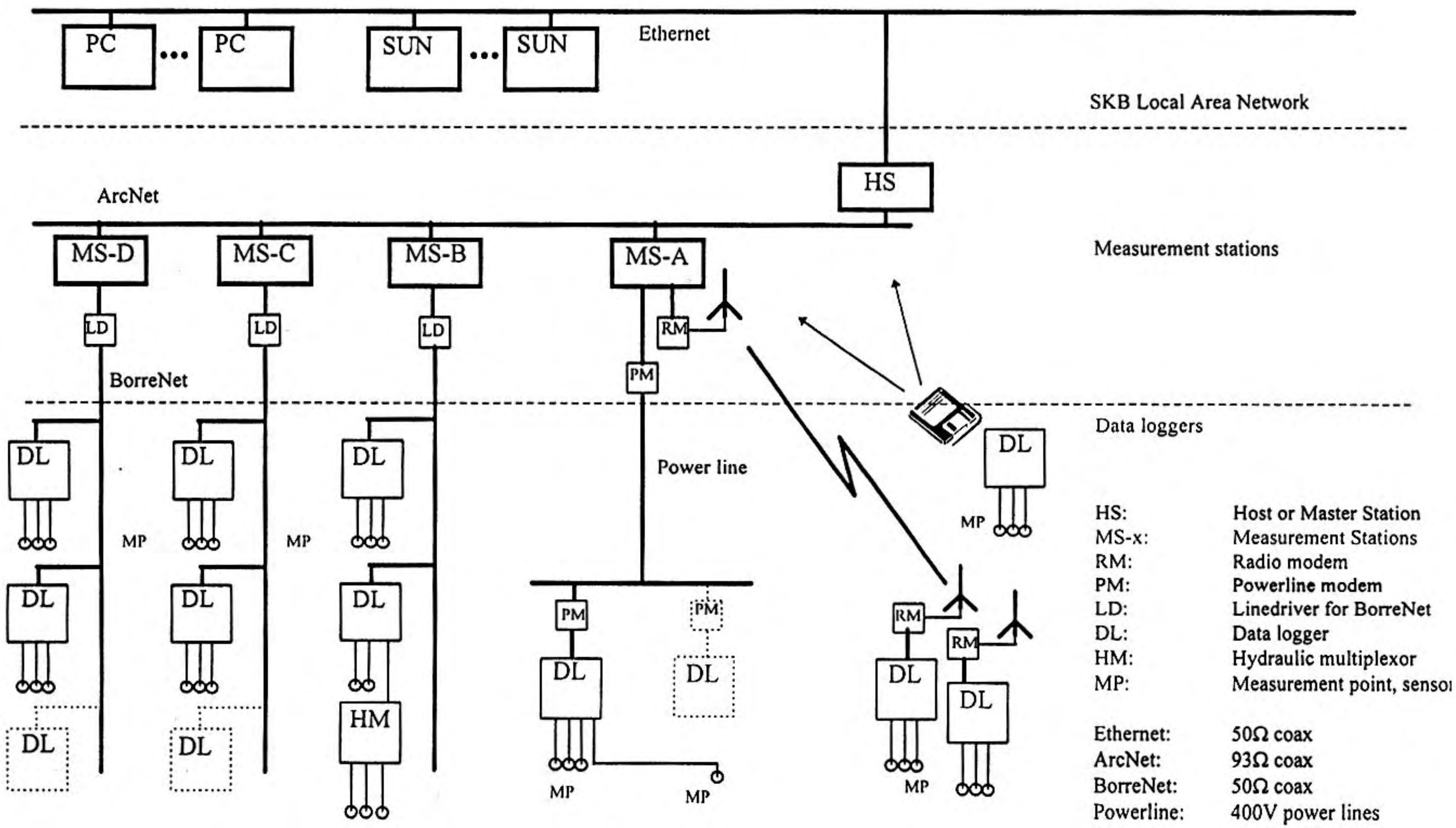


Figure 11-9. The structure of the Hydro Monitoring System system as of 1997.

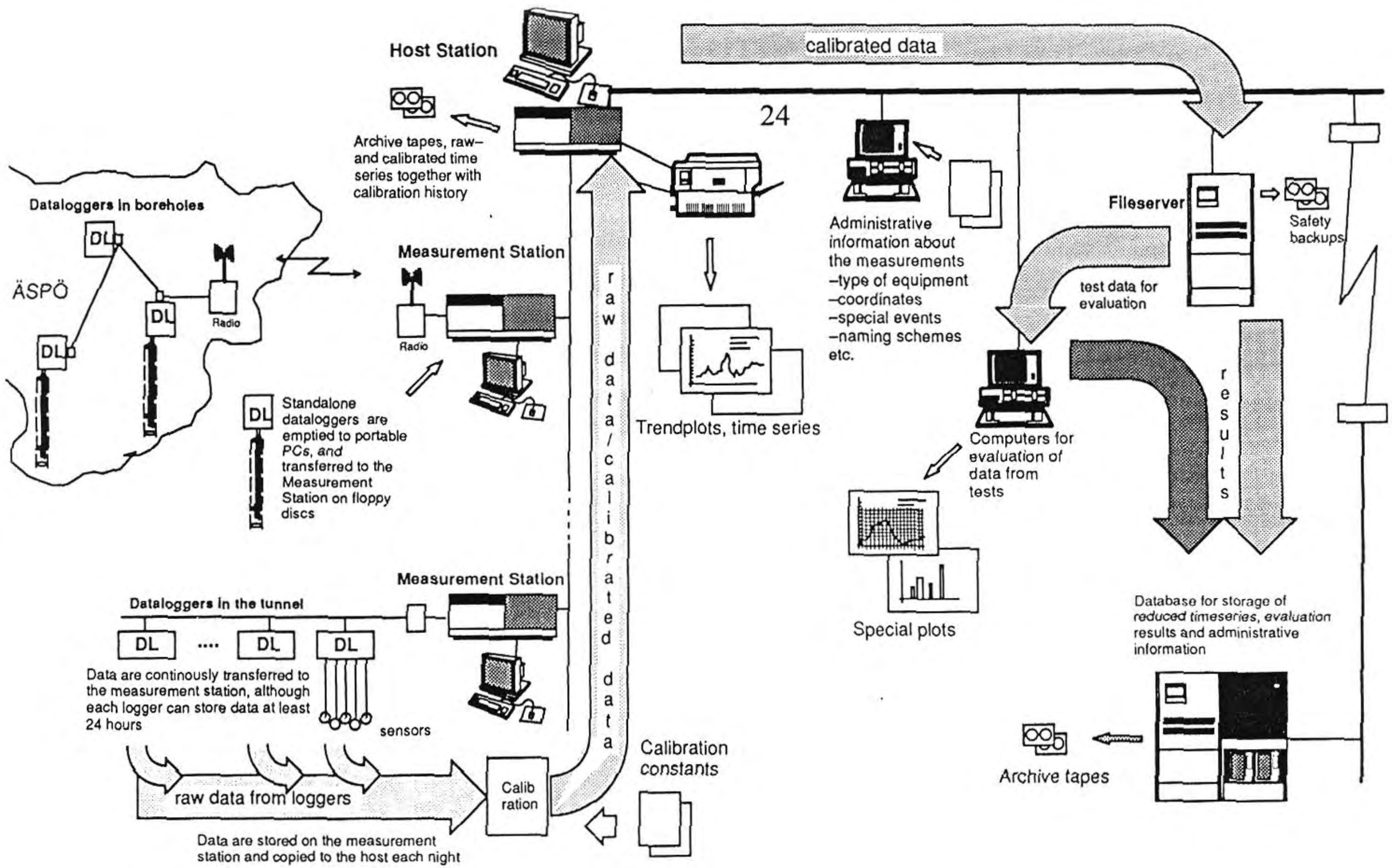


Figure 11-10. *Overview of data flow and data management in the HMS, from measurement to data presentation and storage (before moving host station to site office on Äspö island).*

So-called calculated channels are also used. Each such channel uses one predefined equation that uses up to four other channels as input and up to ten different constants. The input can be both measured channels and other calculated channels. These channels also use different sets of constants over time in the same way as the measured value channels. Calculated channels are used to calculate flow in a weir (from water level in the weir), water vapour transport (from air velocity, air temperature and air humidity), etc.

All data and calibration constants are checked about three times a year, and obviously inappropriate data are deleted. Between these occasions, data from all connected transducers are checked once a week to make sure that data from all locations is collected and plausible.

Data are copied to SKB's main database once a year. Only one data value per 24 hours per channel is stored in this database.

A schematic data flow chart for groundwater monitoring is shown in Figure 11-11.

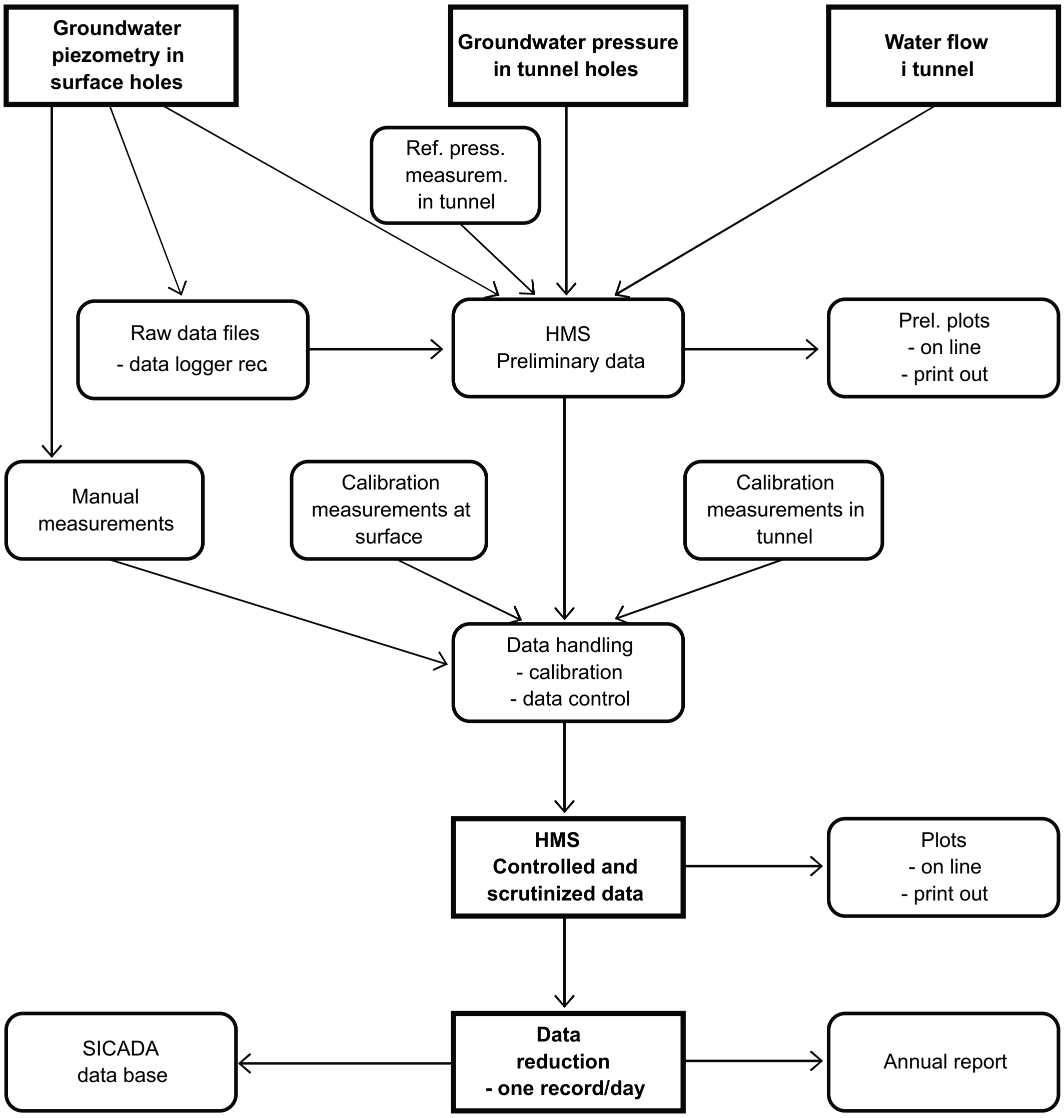


Figure 11-11. Data flow chart for groundwater monitoring.

Comments and recommendations

When the system started, it functioned appropriately but not perfectly. It was, therfore continuously redesigned ever since the beginning. This was possible by the support of technically qualified personnel. The source code of the software for both measurement and data processing has been modified.

11.5.3 Data presentation

The objective of the HMS with regard to data presentation is:

- to produce graphs with data versus time for all measured and calculated data.
- to produce report files with data for all measured and calculated data, to be used for further analyses and for storage in the SKB main database.

For details see /Nyberg et al. 1996/ and /Almén and Johansson, 1992/.

Instrumentation and Methodology

Graphs can be viewed on the local graphic terminals (all measurement stations) or on a PC (log-on from anywhere in the SKB net) with Tektronix emulation software. Graphs can be printed on a laser printer (HP LaserJet III or higher) connected to the host station or connected via the SKB net.

The graphs contain up to six channels of data versus time, see Figure 11-12. Defining a stand-in channel for each defined channel in the graph is possible. For example, manually levelled data may be used if monitoring has failed. The output can be all values or values at certain time intervals, from one second to 24 hours. An unlimited number of predefined graphs can be stored in the system.

Tabular reports in ASCII format may either be produced on-screen or saved in files. Each report can hold up to eight channels. As with the graphs, each channel can have a stand-in channel, output can be all values or values at certain time intervals, and an unlimited number of predefined reports can be stored in the system.

Graphs and reports can be grouped together in batches. By defining individual graphs as belonging to one or more batches, it is possible to plot a batch of graphs with a single command, for example all graphs containing groundwater levels in boreholes on Äspö.

Reports are used as input to other programs and to send data to the SKB database SICADA, and to external users of the data.

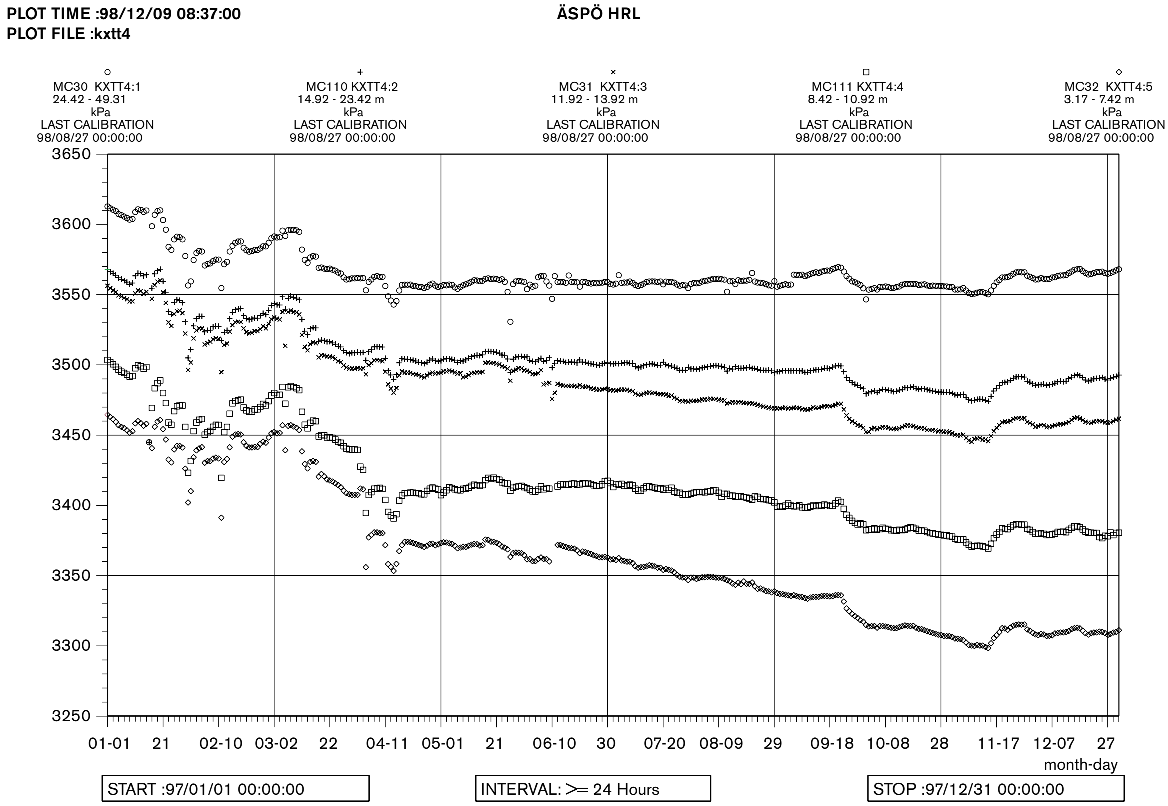


Figure 11-12. *Example of diagram generated at the measurement host station.*

11.6 Summary comments on groundwater monitoring

The monitoring installation in surface boreholes was in general sufficient for the pre-investigation phase. However, it turned out that the equipment for monitoring the pressure in the surface boreholes was not designed for the large drawdowns close to the tunnel spiral, see Section 11.2.3. At the end of the excavation period, several of the borehole sections close to the tunnel spiral stopped functioning. However, these two problems are not believed to have had a major detrimental effect on the evaluation of hydraulic properties or the testing of groundwater flow models.

Another problem was corrosion. Some equipment installed for long-term monitoring started to corrode fairly quickly, causing problems. The problem of corrosion needs to be considered for systems for long-term monitoring.

The accuracy of the measurements of electrical conductivity has not been a high priority. Therefore, calibration and maintenance of the equipment have been insufficient to permit any substantial conclusions to be drawn.

The use of hydraulic multiplexers to measure pressure in tunnel boreholes has worked well for long-term monitoring with a low measurement frequency. However, in tests with a high measurement frequency, accuracy has sometimes fallen to an unacceptable level (see Section 11.3.2).

The measurements of water vapour transport in the ventilation air showed that it was a negligible part of the total water balance in the tunnel (see Section 11.4.4).

As was mentioned in Section 11.1, the HMS used during the construction phase is based on the groundwater monitoring system installed during the pre-investigation phase. The HMS was further expanded during the operating phase, and some technical modifications were made. During 1997 the system was upgraded from the OS9 operating system to Windows NT. As a consequence, the computers at all measurement stations were replaced. This upgrade increased the capacity of the system and improved the user interface, although performance is virtually the same. The number of monitoring points increased, although some monitoring points were excluded after the construction phase. Table 11-1 shows the groundwater monitoring programme at the end of the construction phase.

Table 11-1. The groundwater monitor programme at the end of the construction phase.

Parameter	Total	On-line monitored	Stand-alone monitored	Manually measured
Surface boreholes	60	31	26	3
– groundwater pressure sections	156	93	38	25
– electrical conductivity sections	20	10	0	10
– circulation/sampling sections	20	–	–	–
Underground boreholes	33	33	0	0
– groundwater pressure sections	84	84	0	0
Underground; other objects	35	35	0	0
– water inflow; v-notch weirs	21	21	0	0
– electrical conductivity	10	10	0	0
– water vapour transport in ventilation air	2	2	0	0
– total water in pipes, in/out	2	2	0	0

12 Database system

12.1 General description and objectives

With reference to Section 1.2.2, one of the main goals of the Äspö HRL is to:

- Test the quality and appropriateness of different methods for characterising the bedrock with respect to conditions of importance for a final repository.

One of these methods is the management of the huge amount of data, which are collected and interpreted during site investigations and detailed characterisation. Hence, the database system underwent development, testing and improvement during the pre-investigation phase and the construction phase of the Äspö HRL.

The main objectives of the database system are:

- To manage and archive the huge amount of geoscientific data collected during earlier as well as coming site characterisations.
- To offer efficient and quality-assured routines and tools for storage and retrieval of data.

This chapter describes the new geoscientific investigation database SICADA, as well as related routines for data management. SICADA is and will be one of SKB's most important database systems. SICADA efficiently serve planned investigation activities at the future candidate sites as well as the experiments at the Äspö HRL.

12.2 SICADA (Site Characterisation Database)

12.2.1 Evolution

Development of the GEOTAB database was initiated in 1986 and the first data were stored in 1987. The aim in setting up this geoscientific database was to preserve all data from the Study Site investigations, performed by SKB during the period 1977–1986. It was also aimed to establish a tool for management of all new data from the planned pre-investigations at Äspö. GEOTAB was originally based on the relational database system from Mimer Information System, but in 1992 it was exchanged for the Ingres relational database system developed by Ingres Corporation.

During the construction phase of the Äspö Hard Rock Laboratory, new needs led to the development of the SADB (Site Activity Database). The main data table in the SADB was a complete event list describing all measurements and engineering activities performed at the site in sequence, like the contents in an ordinary diary. The first version of SADB was ready in October 1993. From that time GEOTAB and SADB were used concurrently.

Directly after the introduction of SADB, a discussion started concerning the possibility of combining the concepts of GEOTAB and SADB. This discussion resulted in a decision to develop a new database that combined these concepts. A project was defined and the work started in June 1994. This work resulted in SICADA (Site Characterisation Database).

The first version of the system was completed at the end of 1995. SICADA is based on the OpenIngres relational database system from Computer Associates.

All data in the former databases GEOTAB and SADB have been successfully transferred to the SICADA system.

12.2.2 Data model

The central data table in the system is the activity_history table. Each data row in this table has a unique activity identifier. This identifier uniquely associates measured data with only one activity in the activity_history table. The activity identifier is located in the first column of the table. Normally the activity identifier is hidden, but it is always present in the background and is handled automatically by the system.

Activity identifiers were introduced in order to make it possible to link an arbitrary number of investigation data tables to a certain activity. Hence, activity identifiers are present in all investigation data tables in the whole system.

Each data row in the activity_history table also has a time stamp and a user identification code to show when data were entered in the table and who entered them.

12.2.3 Data structure

A hierarchical data structure was implemented in the GEOTAB system in order to make it easy to find and retrieve any investigation data. This data structure is also used in the SICADA system. The hierarchy is composed of four levels:

- Science (Level 1)
- Subject (Level 2)
- Method (Level 3)
- Activity (Level 4)

The SICADA data structure contains the sciences engineering, geology, geophysics, groundwater chemistry, hydrology, meteorology and rock mechanics. The principal structure with an excerpt of the information content for each hierarchical level within the seven sciences is shown in Figure 12-1.

Every set of investigation data in SICADA has been collected from boreholes, tunnels or other objects. Simple name conventions have been established and used for objects. The name convention for objects in the Äspö tunnel was described in Section 3.5. Seven characters are used for objects in the Äspö tunnel, such as for the cored borehole KA2511A. The naming of surface boreholes is somewhat different, where only five characters are used. An example is the cored borehole KAS02 and the percussion borehole HAS05. The capital letters K and H are still used for cored and percussion-drilled holes, AS is the area code for Äspö, and finally 02 is a sequence number. For example, KAS02 was drilled before KAS03 and HAS05 was drilled before HAS06. The object codes (sometime called ID codes) and the hierarchical data structure are the key information when searching for data in the SICADA system.

Level 1 SCIENCE	Level 2 SUBJECT	Level 3 METHOD	Level 4 ACTIVITY
Engineering	Tunnel excavation etc.	Drill and blast etc.	D&B - Round drilling D&B - Charging D&B - Round D&B - Ventilation etc.
Geology	Tunnel mapping etc.	Tunnel mapping etc.	Tunnel mapping with TMS
Geophysics	Borehole logging etc.	Resistance etc.	Single point resistance logging
G.W. Chemistry	Analyses etc.	Water etc.	Water sampling, class 1 Water sampling, class 2 Water sampling, class 3 Water sampling, class 4 Water sampling, class 5 etc
Hydrology	Disturbance tests etc.	Pressure build up etc.	Pressure build up test
Meteorology	Temperature etc.	Temperature etc.	Temperature from SMHI
Rock Mechanics	In situ stress etc.	Overcoring etc.	Overcoring

Figure 12-1. *The hierarchical data structure of the SICADA system, with all sciences is shown, but only an excerpt of subjects, methods and activities. The arrows indicate the search order when retrieving data from the database. Note that in most cases there is a one-to-one association between a certain method and an activity, but in some cases several activities are associated with only one method (e.g. the set of activities associated with the method /Engineering/Tunnel excavating/Drill and blast).*

It is not possible to store all investigation data sets or parts of data sets in data tables in SICADA, but they are at least stored as file references. Some examples of this type of data set are borehole radar images and geophysical profiles.The file reference is an optional activity tag available during data registration. In actual fact there is an on-line file archive managed by the SICADA system. This on-line archive is called SICADA File Archive. A registered file reference is actually an on-line pointer to the file in the SICADA File Archive.

12.2.4 Applications

Three user applications/programs have been developed, namely:

- **SICADA/Diary**
  This application is used to enter or update data in the database.
- **SICADA/Finder**
  This application is used to retrieve data from the database.
- **SICADA/Retriever**
  This application is used to retrieve data from the database. (Looks like the former GEOTAB application)

The **SICADA/Diary** application is used to log activities and capture data from the work at a typical investigation site. This application mainly works on the activity log table in the SICADA system. Activities in the log can be added, modified or deleted. By selecting an activity in the log it is possible to show additional related information or retrieve the investigation data associated with the activity. The activity log table is an activity diary and all selected activities are always shown in chronological order by default. The user can define other specific search orders. The contents of the activity log window are dependent on the search criteria specified by the user in the window "SICADA/Diary Control Panel".

The **SICADA/Finder** application is used to retrieve data from the data tables available in the database. When a table has been selected, it is possible to get printouts, export data to file or only display data on the screen. Data cannot be displayed in graphic form, but the file export utility enables the user to use his own preferred presentation software. SICADA/Finder makes it possible to retrieve data from one table or combine two tables and then retrieve the result. Search conditions can be set on any column in selected table(s) without knowing anything about the tricky SQL language that is used by the application in the background.

The **SICADA/Retriever** application is a classic text terminal program that is useful for the long-distance user who is connected to the network via a serial modem. The user can only view and/or retrieve data with this application.

12.2.5 Database accessibility

The SICADA system is made up of several parts, of which the most important is the database itself. It is based on the UNIX version of the Relational DataBase Management System (RDBMS) OpenIngres. OpenIngres and the SICADA database are installed on a UNIX workstation located at the Äspö HRL as shown in Figure 12-2.

As shown in Figure 12-2, a second computer is also involved as an application server. In this way the overall performance of the database server has been optimized. The SICADA applications are also installed on the database server, but users are recommended to run the applications on the application server.

To have access to SICADA, a user with an account (login) in the domain skb.se needs to be registered as an authorized SICADA user. The SICADA Database Administrator manages the authorisation process.

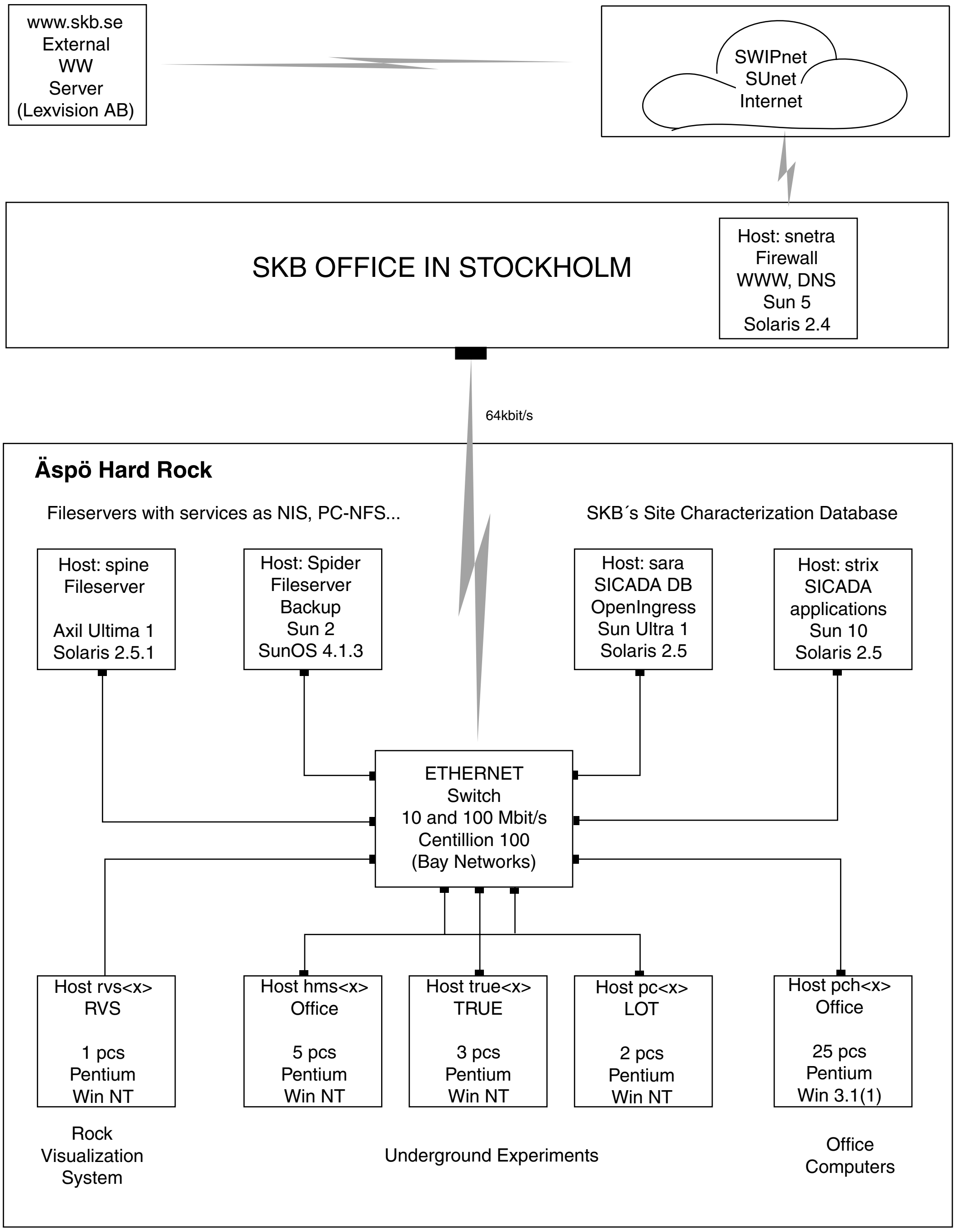


Figure 12-2. *The location of the SICADA database server in the Local Area Network at the Äspö HRL, as of 1997. The LAN at the Äspö HRL is a part of SKB's Wide Area Network (WAN).*

12.3 Database operation

12.3.1 Working methodology in practice

All tasks pertaining to compilation, quality control, evaluation and documentation of collected data are in general very time-consuming. When it comes to quality assurance of a typical investigation data set, it is strongly recommended that the work be carried out in the general sequence shown below.

1. Planning of the field work.
2. Execution of the field work according to the relevant method manual.
3. Data qualification (useful data or not).
4. Total or partial recollection of data if needed.
5. Database entry – Registration of performed field activities.
6. Database entry – Registration of measured data.
7. Database entry – Quality control of registered records (5b), i.e. entry of QC stamps on all records. (Registered data are compared with the original measured data).
8. **Retrieval of background data from the database (e.g. positional information for boreholes).**
9. Evaluation.
10. Database entry – Registration of the results of the evaluation.
11. Database entry – Quality control of registered records (8a), i.e. entry of QC stamps on all records. (Registered data are compared with the original results).
12. Database entry – Final quality check resulting in a Quality Signature for all activities entered.
13. Documentation (Field Report).

Item #8 has been listed and typed in bold for a special and important reason. Experience has shown that many technical authors are reusing published numerical values from old field reports instead of extracting quality-controlled data from the investigation database. As a result, errors in old field reports have been inherited by later reports. This is a serious quality problem that all technical authors should be aware of.

Note that all data entry activities should be done before any field report is written, otherwise the control of data will more or less be lost.

Regarding updating of registered data, this can be done by anyone, but in that case the valid quality stamp and the activity quality signature are removed. This means that a new quality stamp and a new quality signature need to be entered by the person who did the update.

Entry of new types of data sets in the database should be discussed with the Database Administrator before any field activities are performed. The Database Administrator will then prepare the database before the field crew delivers the first set of investigation data.

12.3.1 Organization and responsibility

The Data Systems group at the Äspö Hard Rock Laboratory is responsible for operation and development of the SICADA system. Any technical and administrative question concerning the database should be addressed to the Database Administrator.

All investigation data should as far as the field crews enter possible into the database. The characterisation manager has the responsibility to check that data is stored properly in the database. Regarding accessibility to the database, a user must be registered as an authorised SICADA user. An authorised user has the right to read, enter and update data in the database.

12.4 Comments and recommendations

The regulatory authorities are following the progress of SKB's siting work for the nuclear waste repository. Management of investigation data is a highly demanding and critical task in the licensing process. The site descriptions, repository design and safety assessment must be based on correct and relevant data. Hence, the data management routines need to be focused on the following aspects in a long term perspective:

- Traceability.
- Accessibility.
- Data security.
- Efficiency (system integration and user friendly applications).

A high quality baseline for the safety assessment will be established if the conditions specified above are met.

Designing, implementing and maintaining a data management system suitable for site investigations and underground experiments has turned out to be a highly demanding task. One of the main objectives with the Äspö HRL is to test and develop methods and technologies before they are applied at the candidate sites. Hence, the Äspö HRL can be said to be a "dress rehearsal" of the future realisation of the deep repository. In this context, efficient techniques are required to handle, interpret and archive the huge amount of data collected at a site. There are no short cuts in the process of data management during a running investigation program. The lessons learned so far include the following:

- Important software and database applications should be tested and implemented before the start of any investigation activities at a site. There is usually not enough time available to start programming advanced systems concurrently with the ongoing activities at the site.
- All activities pertaining to compilation, quality control, evaluation and documentation of collected data are in general very time-consuming. When it comes to quality assurance of a typical investigation data set, it is strongly recommended that the work be carried out in the general sequence discussed in Chapter 12.3.1
- A local coordinate system must be decided, established and documented before any investigations are performed at a site, see also Chapter 3. The coordinate system should also be suitable for the underground investigations and all planned or conceivable experiments. The system should be a Cartesian coordinate system with a positive orientation (right hand system). The following name convention for the axes is recommended:

(Local) East	is the signature of the X axis.
(Local) North	is the signature of the Y axis.
(Local) Elevation	is the signature of the Z axis.

  Transformations between the local system and the national grid system should also be established.
- Directional information such as azimuth/bearing and inclination should always refer to the orientation of the axes in the local coordinate system. The azimuth should be measured positive in a clockwise direction from the (Local) North axis and the inclination should be measured positive upwards from the East-North plane.

SICADA is and will continue to be one of SKB's most important database systems. The database should efficiently serve planned investigation activities at the future candidate sites as well as the experiments at the Äspö HRL. The first investigation database, GEOTAB, was set up by SKB during the 1980s. The aim of setting up this database was to preserve all data from the KBS-3 investigations and the pre-investigations at Äspö. In 1995 GEOTAB was replaced by SICADA, but all data were successfully moved to the SICADA system. The data model and the data structure in SICADA have proved to comprise a general, flexible and stable database concept.

The different parts of SKB's Data Management System will be further refined in conjunction with the ongoing and planned activities in SKB's siting work, as required by the regulatory authorities and SKB's internal organization. Regarding system integration, new or improved interfaces for data transfer are planned to be developed.

13 Usefulness of investigation methods

13.1 Introduction

In the previous chapters of this report the different investigation methods used during the Äspö HRL construction phase have been described, and discussed with regard to errors and uncertainty in determined, analysed and/or calculated parameter values or other kind of geoscientific information. Moreover, other comments of the different methods have been ventilated, like those related to the practical performance of the measurements or tests, which is a major task as most of the investigations were conducted in parallel with the construction work. The underground environs itself often called for special efforts even for the personnel not directly involved in the active tunnelling work affected in order to conduct the right thing in the right manner.

Referring to the two first stage goals of the Äspö HRL (see Chapter 1) and the approach of verification of pre-investigations (Section 1.3), the pre-investigation methods have been evaluated by comparing predictions based on pre-investigation models with data and results from the construction phase and there upon updated geoscientific models, see also Figure 1-4. The general evaluation of the pre-investigations has been reported in a package of other reports /Rhén et al. 1997a,c; Stanfors et al. 1997b/ see also Figure 1-6. In this chapter the investigation methods will be evaluated with regard to usefulness for characterising a rock volume from underground, with regard to geological, geohydrological, hydrochemical and rock mechanical properties.

The evaluation of usefulness of underground investigation methods will be structured according to the key issues used for the pre-investigation modelling and predictions, ie Geological-structural model, Groundwater flow (hydrogeology), Groundwater chemistry (hydrochemistry), Transport of solutes and Mechanical stability models (or rock mechanics). The subjects for which predictions were set up before the construction, see Table 13-1, have been measured or in another way been determined during the construction phase with use of the investigation methods presented in the previous chapters. Some of the subjects have been slightly modified or adjusted, see Table 13-1, while other (not predicted) subjects or parameters also have been measured/determined during the construction phase. The latter are more directly related to detailed underground investigations (the second stage goal).

Figures 13-1, 13-2, 13-3 and 13-4, one figure for each key issue (transport of solutes is however included in hydrochemistry), give a condensed scheme of how the different investigation methods were used and how parameter information were interacted to produce data for each subject. This structure will be the basis of the usefulness evaluation in the following sections.

Table 13-1. Presentation of key questions and subjects (adjusted) for which predictions based on pre-investigation models were made.

Key question	Subject	Site scale	Block scale	Detailed scale
Geological-structural	– Lithological unit	x		
Model	– Rock boundaries	x	x	
	– Rock composition	x	x	
	– Rock type characteristics	x	x	x
	– Small scale fracturing	x	x	x
	– Major fracture zone	x		
	– Minor fracture zone	x	x	
	– Single open fractures		x	
Groundwater flow	– Hydraulic conductivity	x		x
	– Water bearing zone	x		
	– Conductive structure		x	
	– Flow distribution		x	
	– Head at boundary	x		
	– Pressure in boreholes	x		
	– Flow into tunnel + total	x		
	+ zone	x		
	+ tunnel parts (legs)	x		
	– Groundwater flux	x		
	– Salinity (Boreholes/Legs)	x	x	
	– Point leakage			x
	– Disturbed zone + axial flow and pressure near conductive structure		x	
	+ pressure (conductivity, axial flow)			x
Groundwater chemistry	– Composition of groundwater in major fracture zones of different rock types	x		x
	– Quality changes		x	
	– Redox conditions			x
Transport of solutes	– Flow paths	x		
	– Arrival time	x		
	– Saline interface	x		
	– Natural tracers	x		
Mechanical stability	– Rock quality	x	x	
Model	– Rock stress	x	x	x
	– Stability		x	x
	– Mechanical characteristics			x
	– Fracture surface properties			x

13.2 Investigation methods for geology

13.2.1 General

The evaluation of the underground investigation methods, used for geological documentation and characterisation during the construction phase, is based on the subjects for which predictions were set up at the end of the pre-investigations /Gustafson et al. 1991/. During and after tunnel excavation the predicted data and geological, structural model, have been validated against observations and measurements in the tunnel, shaft and underground boreholes /Rhén et al. 1997a/, see Figure 13-1. Methods used are discussed with respect to precision and usefulness, see also Table 13-2 and Figure 13-1.

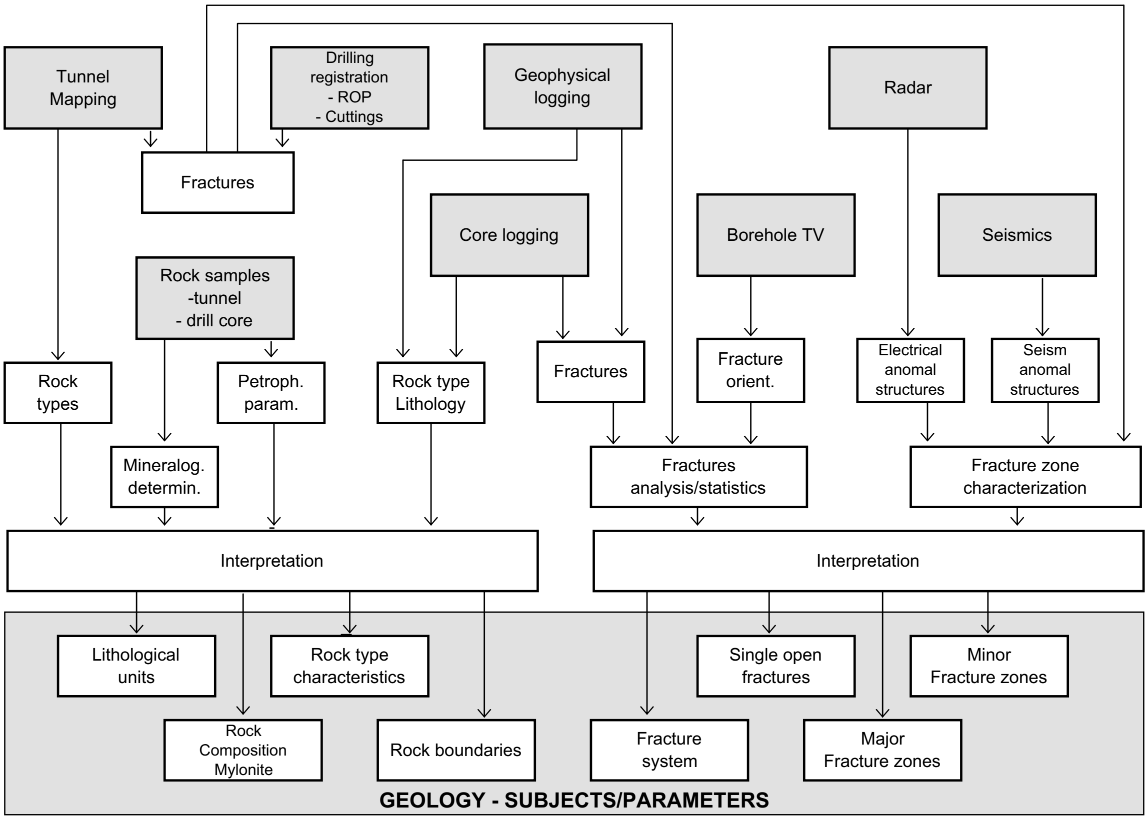


Figure 13-1. *Summary flow chart describing the different geological investigations and their subjects/parameters.*

Table 13-2. Judgement of usefulness of the geological methods used in the construction phase.

Subject	Methods	Usefulness Site scale	Block scale	Detailed scale	Notes
Lithological units	Geological tunnel mapping	3	3	3	
Rock composition	Probe boreholes and percussion holes	–	1	1	
Rock boundaries	TV-logging	–	2	2	
	Core logging	3	3	3	
	Geophysical borehole logging	1	1	1	
Rock type characteristics	Geological tunnel mapping	3	3	3	
	Core logging	3	3	3	
	Geological analysis of rock samples	–	–	2	
	Mineralogical analysis of fracture fillings	–	–	3	
Small scale fracturing	Geological tunnel mapping	3	3	3	
	Core logging	2	2	2	
	TV-logging	3	3	3	
	Geophysical borehole logging	2	2	2	
Major fracture zones and minor fracture zones	Geological tunnel mapping	3	3	3	
	Core logging	3	3	3	
	Percussion boreholes	–	2	2	Minor zones
	TV-logging	–	–	–	
	Geophysical borehole logging	2	2	2	
	Radar methods	2	2	2	Minor zones
	Sesmic methods	2	2	2	
Single open fractures	Geological tunnel mapping	–	3	3	
	Core logging	–	3	3	
	TV-logging	–	2	2	
	Radar methods	–	2	2	

Very useful = 3 Useful = 2 Less useful = 1 Not applicable or used = –

13.2.2 Lithological units – rock composition – rock boundaries

General

The subjects "Lithological units" and "Rock composition" refer to an overall description and distribution of the main rock types on a regional and site scale. Mylonite is included in "Rock composition". "Rock boundaries" refer to the boundaries between the different rock types.

A "Lithological unit" (or "Rock unit") is here defined as a big volume of one of the most frequent rocks in the Äspö tunnel area such as Småland (Ävrö) granite or Äspö diorite.

Inclusions or interesting dikes of minor rock types may be included in a "Lithological unit". "Rock composition" of a lithological unit comprises the relative distribution and description of the different rock types in the unit. A "Rock boundary" can be defined as the contact between two different rocks. The gradual transition boundaries between the closely related rock types Småland (Ävrö) granite and Äspö diorite are normally tight and not mechanically and hydraulically important. Contacts between granite rocks and greenstone are more often fractured and mineralogically altered.

For the lithological modelling of the Äspö rock volume, the subject "Lithological unit" was mainly used on the site scale. "Rock composition" and "Rock boundaries" were used on the site scale and block scale. Predictions were set up before excavation on the same scales which were used for the modelling work.

During the construction phase the same subjects were used as for the modelling and predicition work /Stanfors et al. 1997a,b/.

Judgement of methods

With reference to Table 13-2 the judgement of methods is as follows.

Geological mapping

Continuous geological mapping in the tunnel, according to description in Section 4.4, gives the best information on rock composition and rock boundaries for the characterisation of a rock mass /Stanfors et al. 1993a,b/ and /Stanfors et al. 1994/.

A big advantage with mapping between every round is to have the use of the front and clean tunnel walls. It also makes a continuos follow-up possible. A disadvantage is the normally very restricted time available between rounds.

Probe boreholes and percussion boreholes (measurements while drilling, MWD)

Probe boreholes, regularly drilled and investigated every fourth round, and other percussion boreholes provided some additional information on lithology and rock boundaries outside the tunnel walls by observing rate of penetration, colour of flushing water and investigation of drill cuttings. The procedures were described in Sections 5.2 and 5.4.

A disadvantage with observations of colour of flushing and drill cuttings, is the low precision. Observations of drilling rate can be very much improved by using an automatic recording device, which was also used during the second half of the tunnelling.

TV-logging

Different TV logging methods were used, as described in Section 6.3. TV-logging with Pearpoint Flexiprobe System or BIPS in percussion boreholes can provide additional information on lithology and rock boundaries outside the tunnel walls.

An advantage of the Pearpoint System is that the device is rather easy and fast to handle. A disadvantage is lack of orientation possibilities, and the restricted range (150 m). The advantage of the BIP System is first of all the possibility to make orientation of structures but also the very good pictures which make is possible to identify different rock types with good accuracy. A big disadvantage of all logging methods is that water inflow under high pressure strongly limits the use at depth in the tunnel.

Core boreholes – core logging

Core drilling and logging methods were described in Sections 5.3 and 6.2, respectively. Core boreholes were normally not drilled for lithological characterisation but core mapping data will of course, provide very good information on the extent of rock units and rock boundaries outside the tunnel.

Geophysical borehole logging

Of the different probes used for lithological investigations (Section 7.2) the gamma probe has been found to be very useful especially for detecting dikes of fine-grained granite which normally have a high concentration of potassium. This rock type normally also has a low density, which is indicated by the density probe, and a rather low magnetic susceptibility which indicated by use of the susceptibility log. Greenstone is mostly well indicated by high susceptibility and increased density. The resistivity method is useful for indicating mineralogically altered rock. The density log was also useful in order to distinguish the acid granite from the more basic granitic varieties (Äspö diorite).

13.2.3 Rock type characteristics

General

"Rock type characteristics" refer to mineralogical composition and petrophysics of the main rock types.

Ocular observations during tunnel mapping and core mapping complemented with numerous microscopical, model analyses and measurement of petrophysical parameters such as density and porosity provided information for the characterisation of the main rock types.

The subject "Rock type characteristics" was used on all scales in the modelling work. Predictions on a detailed scale were made as typical examples for the four most frequent rock types in the area: Småland granite, Äspö diorite, Fine-grained granite and Greenstone. The subject was used in the same way during the construction phase.

Judgement of methods

With reference to Table 13-2 the judgement of methods is as follows.

Geological mapping

Continuous geological mapping complemented with more detailed scan-line mapping (Section 4.4) provided very good information for characterisation and classification of the different rock types in the tunnel. It is normally only possible to distinguish a few main rock types by ocular inspection /Munier and Hermansson, 1992–1993/.

Core boreholes – core logging

The drill cores were mapped with the highest precision using the Petro Core System, see Section 6.2. Considerable attention was devoted to rock type and alteration variation along the cores. Basic data on fracture spacing and surface characteristics of the fracture surfaces, including mineral filling and coatings, were also obtained by logging of drill cores. The core mapping methods provides the best information on all lithological parameters but the method is rather time-consuming.

Examination of the colour, grain-size and structure of the cores, together with the results of the chemical and thin-section analyses and geophysical logging data, has provided more detailed information for characterisation of the main rock types Äspö diorite, Småland (Ävrö) granite, fine-grained granite and greenstone.

Geological analysis of rock samples

Using methods as described in Section 6.4, data from laboratory measurements of physical properties (density, magnetic susceptibility and porosity) of a large number of representative rock samples (short core boreholes in tunnel walls) contribute very well to the classification of the main rock types. A disadvantage of the petrophysical laboratory measurement is the need for a great number of representative samples to get accurate mean values. Sampling by core drilling is rather expensive.

Mineralogical analysis of fracture fillings

Samples from short core boreholes in the tunnel walls were microscopically investigated (thin section analysis), see Section 6.4.2. Modal analysis was the best method for classification of the different rock types.

Quantitative mineral identification on fracture fillings was an efficient analytical tool. However, the method did only give semi-quantitative estimations on the relative amount of multi mineral samples.

13.2.4 Small scale fracturing

General

The main aim of the fracture investigation in the tunnel was to characterise the rock mass and collect data in order to compare predictions and outcome concerning small scale fracturing.

The fracture investigation comprised orientation of the main fracture sets, determination of dominating fracture minerals as well as fracture spacing, length and roughness of fracture surfaces. Water leakage was also documented.

The subject "Small scale fracturing" was used on all scales in the modelling work but only on the block and detailed scales (here designated "Fracture system") in the predictions. During the construction phase the subject "Small scale fracturing" has been used on all three scales.

Judgement of methods

With reference to Table 13-2 the judgement of methods is as follows.

Geological mapping

The continuous geological mapping in the tunnel comprised fractures longer than 1 m. The fractures were mapped with the aspect of length, orientation, properties and fracture fill materials. The geological scan-line mapping comprised all fractures longer than 0.3 m.

Tunnel mapping is the best method to investigate the fracture pattern and the properties of fractures. The accuracy of data measured by compass and tape is normally good enough for characterisation of a rock mass. Using the same methods accuracy is normally somewhat higher in TBM tunnels and shafts with the exception of some small structures.

Core boreholes – core logging

Underground core drilling provided additional information on fracturing outside the tunnel. The drill cores were mapped using the Petro Core System (Section 6.2). Most attention was devoted to fracture data such as spacing and fracture surface data including mineral fillings and coatings. Normally cores from 76 mm boreholes give better opportunity for fracture surface observations than cores from smaller boreholes.

It is important to notice that the amount of "natural" fractures in a drill core is normally overestimated due to the fact that many sealed fractures are broken during drilling and handling of the core. These fractures are sealed and tight in the tunnel. The number of "open" fractures does not match between boreholes and tunnel.

TV-logging

TV-logging with the BIP System is an excellent method for detecting and orientation of most fractures in both core and percussion boreholes (Section 6.3). A restriction of the method is high water inflow in the boreholes.

Geophysical borehole logging

Sonic, resistivity and caliper logs are very good as complement to TV-logging with the BIP System for fracture characterisation (Section 7.2).

13.2.5 Major fracture zones – minor fracture zones – single open fractures

General

One of the main tasks in the characterisation of a rock mass is to investigate which of the geological structures will have the most important rock-mechanical and hydraulic significance.

Of a great many structures mapped on Äspö the term "Major fracture zone" is used for a feature with a width of more than about 5 m and an extent of several hundred metres where the frequency of natural fractures is at least two times higher than the mean fracture frequency in the surrounding rock.

"Minor fracture zone" is used for a feature with a width of less than about 5 m. On southern Äspö steeply dipping structures of this kind have been found to have an en-echelon character and an extent of less than about 100 m. They are often good hydraulic conductors.

The term "Single open fracture" is mainly used for persistent open fractures – up to 1 dm wide – which have been found to be important hydraulic structures on southern Äspö. They seem to occur in an en-echelon pattern – like the minor fracture zones – across the Äspö-Hålö area.

The main aim of the underground investigations was to characterise the fracture zone with the aspects of orientation, width and properties in order to compare predictions and underground observations.

For the geological-structural modelling of the Äspö rock volume, on three geometrical scales, the subjects "major fracture zone" and "minor fracture zone/single open fractures" were used. Predictions were set up before excavation on the same scales which were used for the modelling work.

During the construction phase, characterization of the fracture zones was normally restricted to approximately 50 m outside the tunnel walls (block scale) by use of boreholes and radar. Detailed characterization of the fracture zones was performed on detailed scale by tunnel mapping.

Judgement of methods

With reference to Table 13-2 the judgement of methods is as follows.

Geological mapping

Continuous geological mapping, according to Section 4.4.1 is the best method to characterise a fracture zone but very often, especially the major fracture zones, could only be inspected at distance for safety reasons before reinforcements. Shotcrete supported parts of the tunnel are impossible to map in more detail.

Core boreholes – core logging

Underground core drilling is the best method especially for localisation of a fracture zone but it is generally very difficult to penetrate sections of crushed and clay-altered rock using small drill bits without grouting (Section 5.3).

The drill cores were mapped using the Petro Core System (Section 6.2). Most attention was devoted to rock quality data such as fracture spacing and surface characteristics of the fracture surfaces, including mineral fillings and coatings. Rock type and alteration were also mapped.

Percussion boreholes (drilling and measurements while drilling)

Percussion drilling provided valuable information on position and some characteristics (rock quality and water contents) of the fracture zones (Section 5.2). An disadvantage is the restricted length (~40 m) and a less good precision concerning the direction of the holes.

TV-logging

TV-logging with the BIP System can provide valuable information on fracture orientation in fracture zones but the method is not possible to use in boreholes with bad rock quality (Section 6.3).

Investigation in the boreholes by use of the Pearpoint Flexiprobe System shows fracture, water inflow and in some cases also grouted fractures. It is not possible, however, to get an overview – only very detailed information of a small part of the borehole. Orientation of fractures can only be made very approximately. Coreholes normally give better results than percussion holes with the Pearpoint Flexiprobe System.

Geophysical borehole logging

Among geophysical methods described in Section 7.2, the density and the susceptibility logs are well adapted to indicate increased fracturing in the fracture zones. Single open fractures can be detected by use of the gamma log (high natural gamma) and the resistivity log (anomalous resistivity for fractures containing water). Measurements of the Lateral resistivity in combination with the normal resistivity are especially useful methods to indicate the presence and character of fracture zones. The caliper log is also useful in detecting fracture zones. A general disadvantage concerning all geophysical logging in boreholes is that too bad rock quality and high water inflow make it almost impossible to use the method.

Radar methods

Borehole radar and tunnel radar measurements (Section 7.3) are very useful methods for detecting and orientation especially minor fracture zones and single open (water-bearing) fractures. The range of the radar measurements is estimated to be 10–60 m. A disadvantage is that the penetration of radar waves is sensitive to saline water and to an overall high fracture frequency. It is difficult to make a clear evaluation of the correlation between radar reflections and geophysical structures.

The radar range obtained in reflection and VRP mode was approximately 20 m. Somewhat better resolution compared to seismics. Measurements with tunnel antennas can be performed efficiently with a high production rate. Significantly better results is obtained when the radar transmitter is placed in a borehole during VRP surveys compared to using a tunnel antenna as source. An advantage with RAMAC/GPR is that measurements can be performed efficiently with a high production rate and it is a non-destructive investigation method.

Seismic metods

The VSP (vertical seismic profiling) and HSP (horisontal seismic profiling) methods (Section 7.4) were found to be important as a complement to the borehole- and tunnel radar data, especially after three-dimensional processing using a technique with Image Space filtering, which has been developed for seismic studies in crystalline rock. The method is useful for determining orientation of fracture zones. An advantage is that the seismic methods are not sensitive to saline water as the radar method.

13.3 Investigation methods for hydrogeology

13.3.1 General

The evaluation of the underground investigation methods for hydrogeological characterisation during the construction phase is based on the subjects for which the predictions were set up at the end of the pre-investigations, see Table 13-1 /Gustafson et al. 1991/. However, “salinity borehole/legs” is presented in Section 13.5. During and after tunnel excavation the predicted data and the hydrogeological model have been validated against observations and measurements in the tunnels, shafts and boreholes. Methods used are discussed with respect to precision and usefulness, see also Table 13-3 and Figure 13-2.

Besides the methods mentioned in Table 13-3 the geological model is an important base for the hydrogeological interpretations. The identification of the positions of zones in the tunnel was mainly based on the geological identification (see previous section).

Table 13-3. Judgement of usefulness of the hydrogeological methods used in the construction phase.

Subject	Methods	Usefulness Site scale	 Block scale	 Detailed scale
Hydraulic conductivity, water bearing zone,	Mesurements during probe hole drilling	2	2	1
Conductive structure	Pressure build-up tests in probe holes	3	3	2
	Measurements during drilling of investigation holes	2	2	1
	Pressure build-up tests in investigation holes (single packer and double packer system)	3	3	3
	Groundwater monitoring during drilling and tunnelling	2	2	–
	Flow-meter logging	3	2	1
	Interference tests	3	3	2
Boundary conditions, Pressure in the rock volume	Monitoring in surface boreholes	3	3	2
	Monitoring in tunnel boreholes	3	3	2
Flow into tunnel	Water flow into tunnels and shafts (Dams (collecting the water flow along the tunnel) and weirs (measurement of the flow rate))	3	2	–
	Water flow in pipes	3	–	–
	Vapour transport by the ventilation air (temperature, humidity, air velocity)	3	–	–
	Electrical conductivity	2	2	
Groundwater flux	Groundwater flow mesurements (Dilution method)	3	–	–
Point leakage	Hydrogeological mapping	–	–	2
Disturbed zone	Monitoring in probe holes of water pressure	–	2	2
	Water flow into tunnels	–	2	1
	Pressure build-up tests in probe holes	–	2	2

Very useful = 3 Useful = 2 Less useful = 1 Not applicable or used = –

13.3.2 Hydraulic conductivity, water-bearing zone, conductive structure

General

The subject “Water-bearing zone” refers to the deterministically defined water-bearing zones (or hydraulic conductor domains, as they are called in /Rhén et al. 1997c/) for which the transmissivities are estimated. The subjects “Hydraulic conductivity” and “Conductive structure” refer to the properties in the entire rock volume or the rock mass between the water-bearing zones (called hydraulic rock mass domains in /Rhén et al. 1997c/). “Hydraulic conductivity” is presented as a statistical distribution for a defined rock volume. “Conductive structure” is presented as a statistical distribution of the distances between features with a transmissivity larger than a specified value or within a specified interval and primarily provides information of features smaller than “Water-bearing zones”. The subjects were a part of the evaluation of the predictions and are the base for the property description of the hydrogeological model. The characterisation during the excavation details the hydrogeological model which makes it possible to check the models set up before start of excavation.

The methods used were:

- Measurements during probe hole drilling.
- Pressure build-up tests in probe holes.
- Measurements during drilling of investigation holes.
- Pressure build-up tests in investigation holes (single packer and double packer system).
- Groundwater monitoring during drilling and tunnelling.
- Flow-meter logging.
- Interference tests.

Figure 13-2 shows schematically the methods used for estimation of the hydraulic properties related to the subjects.

Judgement of methods

With reference to Table 13-3 the judgement of methods is as follows.

Mesurements during probe hole drilling

During drilling, the inflow of water (flow rate and position in the borehole) and the rock composition were documented (see Section 5.2). Small flow rate changes cannot be seen, the resolution of the flow rate changes to position in the borehole is one drill rod and the interpretation of the rock composition from the out flowing water may be difficult. Still the observation made during drilling is considered useful for the evaluation and the modelling of features near the tunnel.

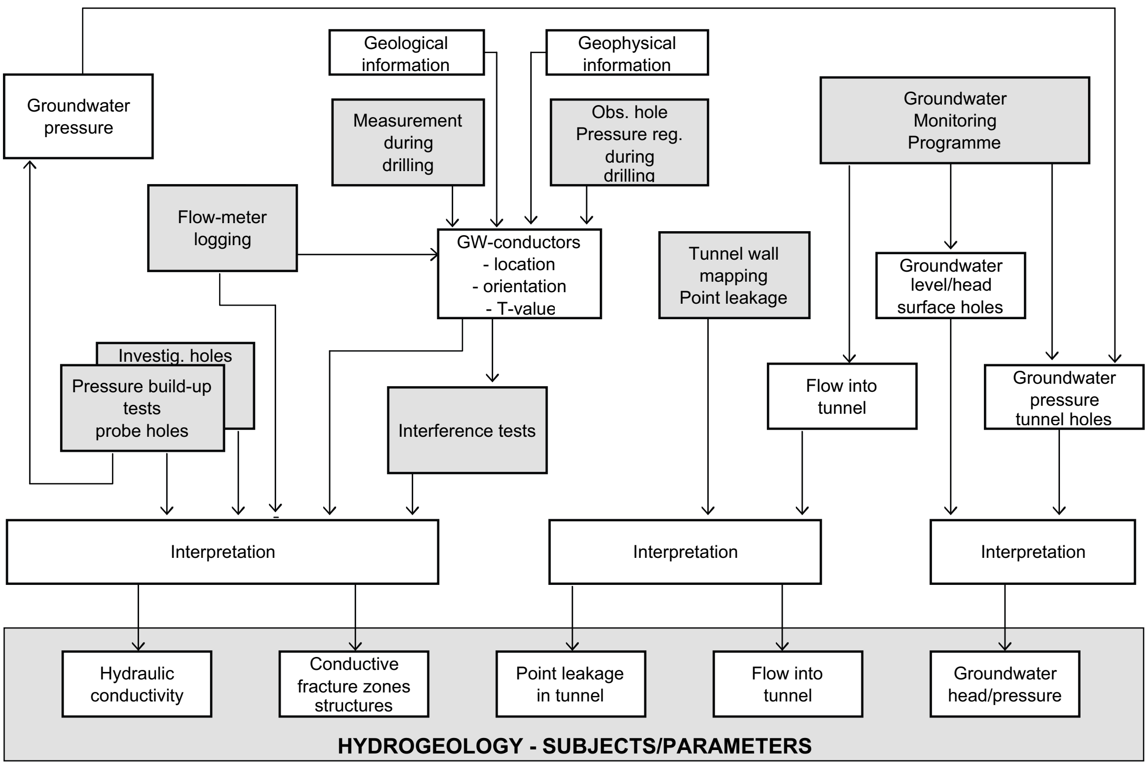


Figure 13-2. *Summary flow chart describing the hydrogeological investigations and their subjects/parameters.*

Pressure build-up tests in probe holes

Pressure build-up tests in probe holes are described in Section 8.2. The method of using probe holes close to the tunnel face and pressure build-up tests was chosen for the following reasons:

- To estimate the hydraulic properties of the undisturbed rock. The properties are more undisturbed behind the tunnel face than in front of it, at least if short boreholes are tested.
- The pressure build-up test is a simple and reliable method.
- The probe holes drilled ahead of the tunnel face provide the builder and contractor with information about the rock properties and of water problems.

Transient testing methods are preferred because they provide an opportunity to evaluate the flow regime and give some rationale for the choice of evaluation method. The hydraulic resistance around the borehole ('skin') that is always more or less present may also be separated from the properties of surrounding rock, which cannot be done using stationary evaluation methods.

A number of the tests in the probe holes, with test length about 15 m, with low conductive sections gave a typical well-bore storage response for the entire test time. In the re-evaluation of the data the specific capacity was used to estimate the transmissivity of these low conductive test sections, see /Rhén et al. 1997c/. It is judged that approximate transmissivities can be obtained in this way for the low conductive sections.

The experience is that the method provided data as expected in most cases. In a few cases long probe holes (30–40 m) were used for pre-grouting causing disturbance of the natural conditions, and the evaluated hydraulic properties from other probe holes drilled in this grouted section were possibly lower than the undisturbed values. However, the disturbed tests were too few and estimated to not affect the statistics of the hydraulic properties.

An alternative to these short boreholes could be long core holes drilled ahead of the tunnel face and hydraulically tested before continuing the excavation. This will however cause a fairly long delay before the excavation can continue and the tests are less robust, see "Pressure build-up tests in investigation holes". One advantage could be that the drilling is performed within the planned tunnel circumference leaving no boreholes to seal and testing of undisturbed hydraulic properties may be more reliable if continuos pre-grouting is planned. Another advantage is that more undisturbed water samples could probably be obtained, at least if water is sampled in a proper manner during drilling. A disadvantage is that pressure observation around the tunnel, during and after the excavation, can not be made.

Measurements during drilling of investigation holes

During drilling of boreholes from the tunnel the water flow into the borehole was measured or estimated as a function of borehole depth, see Section 5.3. The flow rates and the positions for the increases of the flow rates are approximate and it is impossible to observe small increases of the flow rates beyond a large inflow point in a borehole. Never the less, the method is judged as very good to achieve early flow estimates along the borehole to a relatively low cost. The results are useful as a base for a first assessment of hydraulic conductor domains, estimation of hydraulic conductor frequency and discussion of packer positions in the borehole. However, experiences during 1998 with wireline drilling indicates that it is very difficult to make flow rates observation during drilling.

High inflow rates creates problems with the drilling and planed testing, see "Pressure build-up tests in investigation holes" but it also creates problems for the contractor if the large amounts of water flows to the tunnel face. On a few occasions the excavation of the tunnel was quite problematic, because of large water inflow.

Pressure build-up tests in investigation holes (single packer and double packer system)

Pressure build-up tests in investigation holes are described in Sections 8.3 and 8.6. Pressure build-up tests were performed for the entire length of most cored holes. A few times the double packers were used to be able to test smaller sections of the cored borehole. In a few cases the drilling was interrupted and single-packers were used to test the last part of the borehole drilled. Draw-down and recovery periods were generally about 30 + 30 minutes.

The pressure build-up tests (in investigation holes as well as probe holes) are useful as it is the only way to get quantitative estimates of the hydraulic properties. The type of tests to be performed must be based on the purpose and time limits. Approximate values of transmissivity distribution can be obtained with test of the entire borehole in combination with flow logging. If more certain estimates of the hydraulic properties of a certain section in the borehole or along the entire borehole is needed, hydraulic tests with a double packer system must be used to perform tests along the drilled borehole or drilling has to be interrupted for specified drill-depths and single-packer must be used to test the last part of the borehole drilled.

Drilling of investigation holes was useful as a better knowledge of the geometry and properties of the hydraulic conductor domains were obtained. However, it turned out to be difficult to do the characterisation as planned in some cases. The reason was high inflow rates that limited the possible investigation methods or the grouting of the borehole was needed before the drilling could continue (see Sections 5.3 to 5.5). Due to this some long core holes were not systematically investigated along the entire borehole with a certain method. The implication of this is that a property evaluated from a frequency distribution becomes less reliable if the base for the evaluation is not a sample based on a regular sampling interval along the borehole and/or the same test method. From a strictly statistical point of view this is negative, but in most cases it is judged that satisfactory hydraulic data were obtained for developing the model around the tunnel.

In order to increase the possibilities to sample data from boreholes with high flow rates a casing was developed, see Section 5.3.4. In boreholes with high flow rates and /or problems with the stability with the borehole a methodology with hydraulic testing followed by grouting was also developed in order to get hydraulic data for sections that had to be grouted, see Sections 5.3.5 and 5.5. A problem was to define suitable criteria for when grouting was to be performed.

Another problem is that it is difficult to get reliable results from low conductivity sections of a borehole because of the elasticity of the equipment and because of unstable pressure conditions in the rock mass before a test. The test methodology can be improved but the tests will be more time consuming.

Groundwater monitoring during drilling and tunnelling

During drilling of boreholes and during the excavation of the tunnel, the goundwater pressure was in several cases monitored in a number of tunnel boreholes and always in the surface boreholes (Sections 8.4, 11.2 and 11.3). Based on the drill record (the borehole depth as a function of time and water inflow as a function of borehole depth) or the tunnel face position and the measured pressure (as a function of time) in the observation boreholes and, also the draw-down pattern, a conclusion was made on the connectivity between water-bearing zones.

The method was found useful. However, careful planning is needed for selecting boreholes and equip these with data loggers, unless they are already connected to a measurement system such as HMS. The selection should be based on expected influence of a hydraulic pressure response and boreholes within that volume that possibly should respond or not respond to a sudden inflow to the drilled borehole or the tunnel. Evaluation may also be quite extensive if there is a large number of observation points.

Flow-meter logging

After drilling or during periods when the drilling was interrupted flow-meter logging was generally performed in cored holes using a MLS spinner probe or UCM acoustic probe, see Section 8.7.

It is a fast method that gives good estimates of the flow rate and the positions of the increases of the flow rates. The flow rate distribution along the borehole and the transmissivity evaluated for the entire or part of the borehole has been a base for estimation of an approximate transmissivity distribution along the borehole. (In some cases the flow rate distribution from the drilling was used if no flow-meter logging was available). The precision of the positions of flow increases is mainly dependent of the measurement interval, which can be adjusted to shorter sections around main inflow points. Smaller increases of flow rates are in several cases not seen. The reason is that small increases of the flow rates beyond a large inflow point in a borehole are masked by the large inflow rate. This means that the frequency of smaller conductive feature will generally be underestimated. If a detailed resolution and more reliable quantitative estimates of the actual hydraulic properties (low as well as high conductive parts) is needed, hydraulic tests with a double packer system must be used to perform tests along the borehole.

Interference tests

A limited number of interference tests were made by allowing selected tunnel boreholes (the entire or section of it) to flow and at the same time monitor the pressures in surrounding boreholes (Section 8.5). Draw-down and recovery periods were generally from one up to about 40 hours.

These tests were very useful for estimating the hydraulic properties of the hydraulic conductor intersected by the flowing borehole and also for evaluation of the connectivity between the hydraulic conductor domains and sometimes the conductive features near the flowing borehole.

Interference tests can be rather time-consuming in planning, execution, processing of data and evaluation of data. It is very important to plan interference tests and other activities, which may cause pressure responses (for example drilling) so that they do not interfere with each other. If other tests or activities caused pressure responses, they may ruin the interference test.

13.3.3 Boundary conditions, Pressure in the rock volume

General

"Boundary conditions, Pressure in the rock volume" are estimated based on the water level or water pressure measurements in borehole sections and are important for interpreting the hydraulic responses in the rock mass due to tunnel excavation as well as hydraulic tests and to give essential data for the groundwater flow modelling (calibration and testing of models). The subjects were a part of the evaluation of the predictions and it gives possibilities to check the results from groundwater flow modelling made before start of excavation.

The methods used were:

- Monitoring in surface boreholes.
- Monitoring in tunnel boreholes.

The piezometric levels of the groundwater in the Äspö, Ävrö and Laxemar areas were measured in a large number of boreholes drilled from ground level /Rhén et al. 1997b/. The percussion boreholes, generally 100–200 m deep, contained 1–3 measurement sections. The cored holes from surface, which are up to 1,000 m deep, had up to 6 measurement sections. Some of the boreholes drilled from the tunnel were also equipped with packers and connected to the monitoring system in the tunnel. The short probe holes were manually measured about twice a year. See Sections 11.2 and 11.3 for more details.

The short probe holes (Section 5.2 and 8.2) were used for measurement of the pressures close to the tunnel. Initial pressures, behind the tunnel face, were reported in the overview figures for hydrogeology (Figure 2-9, Section 4.9.1) and the pressures measured there after were used to interpret the properties if the "Disturbed zone" described in Section 13.3.7.

Judgement of methods

With reference to Table 13-3 the judgement of methods is as follows.

Monitoring in surface boreholes and in tunnel boreholes

The monitoring is described in Sections 11.2 and 11.3. The measurement intensity of the monitoring of the water pressures in space and time is judged to be mainly sufficient for the evaluation of the responses during the excavation of the tunnel and during hydraulic tests as well as for numerical groundwater flow modelling. However, the problems with the large drawdown, which made it impossible to measure some surface borehole sections, was negative for the possibilities to more thoroughly evaluate and model the pressure responses due to last part of the excavation of the tunnel.

Effects of earth-tide, precipitation, barometric pressure and sea level changes on the water pressures affects the water pressures to some extent and the measurement intensity should be chosen in such a way that the effects can be quantified and used for the evaluation of the pressure responses measured.

13.3.4 Flow into tunnel

General

"Flow into tunnel" represents all flow measurements in the tunnel aimed at getting data for interpreting the hydraulic responses in the rock mass during tunnel excavation and to give essential data for the groundwater flow modelling (calibration and testing of groundwater flow models). For the modelling purposes it is very important that the data quality is good in terms of accurate flow rates, time resolution of the measurements and a reasonable number of measurement sections along the tunnel. The subject was part of the evaluation of the predictions but is difficult to predict, because the flow rate is very dependent of the amount of grouting in the tunnel and not of the bedrock properties.

The methods used were:

- Weir flow into tunnel and shafts.
  - Dams that collect the water flowing along the tunnel.
  - Weirs that measure the flow rate.
- Water flow in pipes.
- Vapour transport by the ventilation air (temperature, humidity, air velocity).
- Electrical conductivity (of the water flowing into tunnels and shafts).

At tunnel chainage 700 m the total inflow and outflow of water were measured, see Section 11.4. The air-velocity, humidity and temperature were also measured for the air flowing in and out at tunnel chainage 710 m. The flows of vapour in and out of the tunnel were estimated from these values. Approximately every 150 m along the tunnel a concrete dam was built in the tunnel floor, and the dam was connected to a weir downstream. These dams divided the tunnel into a number of sections which separately could be measured more or less continuously.

In most cases there were several conductive fracture zones between two dams. In such cases the inflow from each zone could only be estimated very approximately. The distribution along the tunnel of the mapped flow into the tunnel (Section 4.6) formed the basis for distributing the measured flow rate at the weirs onto fracture zones and defined tunnel sections between two dams.

Judgement of methods

With reference to Table 13-3 the judgement of methods is as follows.

Water flow into tunnels and shafts (Dams (collecting the water flow along the tunnel) and Weirs (measurement of the flow rate))

The construction of the dams was slightly modified early during the construction. After this modification the dams became easier to clean from sedimentation of finer fractions in the ditch. It is also judged that dams are tight and due to this the flow rates measured for a tunnel section are considered accurate.

From the groundwater flow modelling view point it is important to get reliable measurements in time and space of the flow rates, if more detailed simulations are to be made to test or calibrate the hydrogeological model. Dams should be constructed upstream and downstream of a hydraulic conductor's domain, where high inflow rates are expected, and otherwise on a regular interval.

There were problems with the construction (delayed construction of the dams and late installation of the weirs) and maintenance (sedimentation in the dams and weirs, see Section 11.4.2) of the dams and weirs which had important negative consequences of the data quality. For some periods during the excavation data was lacking or was of low quality which made the flow rate estimates very uncertain.

Water flow in pipes

Measuring the incoming pure water, used for drilling etc, and the water that is pumped out from the tunnel system is very important. These measurements are described in Section 11.4.3. The data are important as they, together with vapor transport, show the total flow of water into the entire tunnel system.

The maintenance of the pipe systems is generally the responsibility of the contractor. It is important to assure that the measurements can be more or less continuous and are not interrupted for longer periods because of construction works related to the maintenance of the drainage system in the tunnel.

For a fairly long period during the excavation of the Äspö HRL tunnel, data was lacking because no flow measurements were made due to the constructors work with the drainage system, which made the flow rate estimates very uncertain for that period.

It may be difficult to calibrate flow meters measuring the water that is pumped out, as the rates may be high (Section 11.4.3). It is important to construct the sumps in such a way that make the calibration easy and that the sumps are not leaking. The constructed sumps at Äspö HRL had a minor leakage and could have been better designed for calibration purposes.

Vapour transport by the ventilation air (temperature, humidity, air velocity)

The measurements are described in Section 11.4.4. The amount of water transported as vapour turned out to be small at Äspö and was therefore of minor importance for estimating the total inflow to the tunnel. However, this may not be the case at another site and it is therefor important to measure the water flow transported by the ventilation air in order to see to what extent the vapour flow affects the estimates of the flow rate into the tunnel.

Measuring the air velocity can be problematic and choosing measurement sections is important. This is outlined in Section 11.4.4.

Electrical conductivity

The measurements are described in Section 11.4.5. The measurements are important for the groundwater flow modelling as it is possible to estimate the mean salinity from the electrical conductivity of the water flowing into the tunnel between two dams (or other defined tunnel section which corresponds to the water flowing where the measuring takes place). The values can be used for calibration or tests of groundwater flow models.

13.3.5 Groundwater flux

General

The subject "Groundwater flux" represents the estimated groundwater flow rate in the rock mass and is based on dilution measurements performed in borehole sections. The subject was a part of the evaluation of the predictions. The measurements are useful for interpreting the hydraulic responses in the rock mass during natural (undisturbed) conditions and during tunnel excavation.

The method used was:

- Groundwater flow measurements (Dilution method).

Dilution measurements were performed in a number of borehole sections a few times during the construction phase. The experience from these measurements is reported in /Rhén et al. 1997a,b/.

Judgement of methods

With reference to Table 13-3 the judgement of methods is as follows.

Groundwater flow measurements (Dilution method)

Groundwater flow measurements are described in Section 8.8. Even though it is evident for a number of reasons that there are difficulties in estimating the proper fluxes in the rock mass from dilution measurements /Rhén et al. 1997b/, the dilution measurements are useful and a feasible way of finding out whether or not there are hydraulic communication interims of flows and not just pressure responses.

More reliable predictions and measurements can most probably be achieved if shorter test sections for dilution measurements are used. The test section should also preferably just straddle the hydraulic conductor domain or just be in what is considered to be a hydraulic rock mass domain. This may stand in conflict with the way in which the entire borehole is instrumented as only a limited number of test sections can be installed.

13.3.6 Point leakage

General

"Point leakage" represent a number of mapped entities in the tunnel:

- "Wet tunnel area" (wet tunnel area by rock type),
- "In flow characteristics" (a quantitative measure of the flow rate from each conducting feature) and
- "Inflow types" (a description of the in inflow to the tunnel for each conducting feature).

The subject was a part of the evaluation of the predictions and is useful for the detailed description of the mapping of the tunnel during excavation.

The method used was:

- Hydrogeological mapping.

Judgement of methods

With reference to Table 13-3 the judgement of methods is as follows.

Hydrogeological mapping

Hydrogeological mapping is described in Section 4.6. The quantification and characterisation of the leakage into the tunnel when mapping the walls and roof is difficult but it seems to be possible to obtain a rough estimate of the quantity and distribution along the tunnel. However, it is difficult to make quantitative estimates of the water flowing in through the tunnel walls and, frequently, also identifying leaking fractures and locating leaks along fractures.

If the flow into the tunnel is quantified by just mapping flowing features, neglecting dripping features and moisture on the rock surface, this seems to give around 80% of the total flow from the walls and roof. The mapping and quantifying of flowing features only in the tunnel can be done rather quickly and gives approximately the right flow rate through walls and roof.

It is judged that the mapping of the "character", "type" and the flow rate is feasible but only the "character" and the flow rate are useful data for the modelling. However, "type" gives some possibilities to observe differences between lithological units, as for example wet area compared to dry area, that can give some insight in the flow properties. The observation is however made on a rock surface covering the EDZ (Excavation Disturbed Zone) and these may not be representative for the properties of the undisturbed rock.

13.3.7 Disturbed zone

General

The subject "Disturbed zone" represents the Excavation Disturbed Zone (EDZ) and the rock mass closest to the tunnel. Transmissivity and pressure measurements in the probe holes, grouting documentation and the measurements of the flow into the tunnel has been the base for the evaluation of the hydraulic properties of the "Disturbed zone". The subject was a part of the evaluation of the predictions.

The methods used were:

- Monitoring in probe holes of water pressure.
- Water flow into tunnels.
- Pressure build-up tests in probe holes.

Calculated conductivity changes perpendicular to the tunnel axis ("Skin" for the tunnel) were based on the pressure measurements in the probe holes, the evaluated hydraulic conductivities and the measured flow into the tunnel. The documentation of grouted sections was also important for the evaluation of the Skin. Details concerning the experiences from these measurements and evaluation methods is presented in /Rhén et al. 1997b/. Estimation of the Skin factor involves several difficulties related to pressure measurements outside the tunnel, the flow into the tunnel and the undisturbed hydraulic properties.

Judgement of methods

With reference to Table 13-3 the judgement of methods is as follows.

Monitoring in probe holes of water pressure

The pressure distribution around the tunnel was estimated from pressures in probe holes. Measurement section was approximately 15 m. The measured pressures along the tunnel were then assigned to the mapped rock types in order to provide an estimate of the pressure distribution around the tunnel for each rock type.

The pressure measurements was useful in that sense it provided a good picture of the very large variability of the water pressure close to the tunnel wall, indicating a low hydraulic connectivity between the fractures. It clearly showed that a large number of measurements are needed to get a reasonable accurate description of the pressure distribution. However, as it was only one measurement section used, normally situated 2–10 m outside the tunnel wall, the pressure decrease could not be analysed as a function of distance from the tunnel wall. Several measurement sections radial out from the tunnel could have been useful for the evaluation of the properties around the tunnel but would have demanded much more resources, at least if all probe holes would have been measured.

A few tests with a mechanical packer system with 3 measurement sections was tested (Section 11.3). The design was probably good but the measurements was not successful due to curved holes (percussion drilled holes) and a stiff packer system causing installation problems and leakage.

The measurements, made manually a few times during the construction for each borehole, were also good in that sense that the pressure measurements were obtained at a low cost as the probe hole were mainly made for estimating the undisturbed hydraulic properties and to provide the builder and contractor with information about the rock properties and of water problems.

Water flow into tunnels

The positions of the dams was not well suited for estimation of the inflow from individual water-bearing zones as it was longer tunnel sections that were measured. Estimates of the Skin for these zones became uncertain due to this. See Section 13.3.4 for further comments.

Pressure build-up tests in probe holes

See Section 13.3.3 for comments.

13.4 Investigation methods for groundwater chemistry

13.4.1 General

The evaluation of the underground investigation methods for hydrochemical characterisation during the construction phase is based on the subjects for which the predictions were set up at the end of the pre-investigations, see Table 13-1 /Gustafson et al. 1991/. However, due to the conditions in the tunnel it was impossible to investigate the properties predicted in detailed scale. Therefore a separate task was planned for the sampling and analyses of groundwater from low-conductive rock.

During and after tunnel excavation the predicted data and the hydrochemical models have been validated against observations and measurements in the tunnels, shafts and boreholes. Methods used are discussed with respect to precision and usefulness, see also Table 13-4 and Figure 13-3.

Table 13-4. Judgement of usefulness of the hydrochemical methods used in the construction phase.

Subject	Methods	Usefulness Site scale	 Block scale	 Detailed scale	Notes
Groundwater chemistry in major fracture zones	**Sampling within the documentation programme**				
	Tunnel wall leakage	1	–	–	
	First strike sample from probe holes	2	1	–	
	Sampling within the monitoring programme				
	Repeated sampling in selected probe holes	3	2	1	
Quality changes	REDOX experiment	2	3	–	
Groundwater chemistry in low conductive rock	Pilot test with prototype equipment	–	–	1	

Very useful = 3   Useful = 2   Less useful = 1   Not applicable or used = –

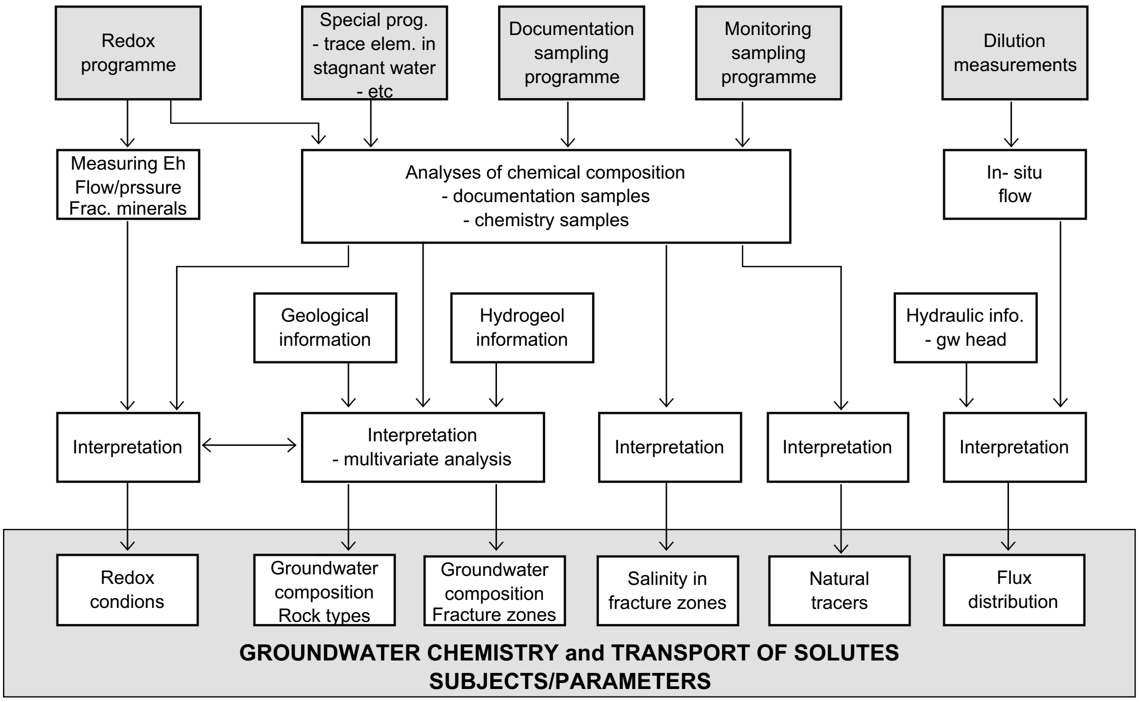


Figure 13-3. *Summary flow chart describing the methods investigating the groundwater and their subjects/parameters.*

13.4.2 Groundwater chemistry in major fracture zones

General

The hydrochemistry of major fracture zones was the main issue of the groundwater chemistry predictions. It also included the only spatial variability in the hydrochemical model of Äspö. Predictions were made for the situation at the point where the tunnel breached the fracture zone and before the inflow to the tunnel had significantly changed the hydrochemistry. The investigated properties of the groundwater are the major constituents sodium, potassium, calcium, magnesium, chloride, bicarbonate, sulphate, trace element concentrations, ferrous and ferric iron, sulphide, manganese, isotopic constituents, carbon-14, tritium, deuterium, oxygen-18, pH and Eh values. Analyses of isotopic ratios of sulphur, strontium, carbon and uranium, gas content, organic matter and microbes are frequently added.

The results of modelling efforts has been summarised by /Smellie and Laaksoharju, 1992/, /Smellie et al. 1995/ and /Rhén et al. 1997c/.

Judgement of methods

With reference to Table 13-4 the judgement of methods is as follows.

Sampling within the documentation programme

Sampling from tunnel wall leakage, according to methods described in Section 4.7, was made at the beginning of the construction phase. It was soon considered to be impractical to conduct. Also the data of the analyses was of poor quality.

Sampling from newly drilled probe holes and investigation holes was made regularly when the inflow exceeded 5 l/min (methods described in Section 9.2). These samples were collected in conjunction to the pressure build up tests and provided the most useful data for comparison to predictions. Only the pH, electrical conductivity, bicarbonate and chloride contents were analysed. If major cations and anions had been included, the results would have been more useful.

Sampling within the monitoring programme

Repeated sampling was done in a few selected probing holes, surface boreholes and from inflow water in measuring dams, according to methods described in Section 9.3. The repeated sampling was more carefully done and with the aim to capture the changes due to drawdown and inflow caused by the tunnel and to up-date the general hydrochemical model of Äspö.

Due to the careful sampling technique and the comprehensive analysis programme, these data were useful for identifying also the microbial processes which were affecting the groundwater chemistry.

13.4.3 Quality changes

General

Due to the draw-down and inflow to the tunnel an increase of the flow rate of recharging surface water is expected. Oxidising groundwater could be expected to penetrate deep into the rock and possibly reach the repository depth. The retardation of such a redox front was calculated and investigated within the REDOX experiment. For this experiment the Eh values and the other redox sensitive constituents were focussed, (ferrous and ferric iron, sulphide, manganese) /Banwart, 1995/.

Judgement of the REDOX experiment

With reference to Table 13-4 the judgement of method is as follows.

The REDOX experiment was established at a pre-selected fracture zone which was broken by the tunnel at 515 m tunnel length and the depth of 70 m. Three boreholes were drilled into the fracture zone and used for on-line pH and Eh-monitoring and for groundwater sampling, as described in Section 9.4. The instrumentation was simple, robust and useful for the purpose. The flexibility was good and needed as the focus of investigation was shifted during the duration of the experiment. A tracer test conducted from the surface provided valuable information on the heterogeneity in the flow field and the size of the active flow domain.

13.4.4 Groundwater chemistry in low conductive rock

General

The groundwater chemistry (hydrochemistry) of low conductive rock was approached by a pilot study. A special packer system was developed and put together in duplicate. It was manufactured in materials which would allow for tracer element analysis. The samples were also used for a laboratory intercalibration. All possible trace elements were included. The results, as expected showed that many of them were below the detection limit.

Judgement of a pilot test with prototype equipment

No specific methods were used except for the packer system, see Section 9.4. The experience of that is such that the equipment will not be used for the further assessment of hydrochemistry in low conductive rock mass. However, it will be kept installed in the present boreholes.

13.5 Investigation methods for transport of solutes

13.5.1 General

The evaluation of the underground investigation methods for transport of solutes characterisation during the construction phase is based on the subjects for which the predictions were set up at the end of the pre-investigations, see Table 13-1. During and after tunnel excavation the predicted data and the transport of solutes model have been validated against observations and measurements in the tunnels, shafts and boreholes. Methods used are discussed with respect to precision and usefulness, see also Table 13-5 and Figure 13-3.

Table 13-5. Judgement of usefulness of the methods for transport of solutes used in the construction phase.

Subject	Methods	Usefulness Site scale	 Block scale	 Detailed scale	Notes
Flow paths Arrival time	Large scale tracer tests	3	–	–	
Saline interface	Sampling within the documentation programme	3	3	–	
	Sampling within the monitoring programme	3	3	–	
	Monitoring in surface boreholes – electrical conductivity				
	Flow into tunnel – electrical conductivity	2	2	–	
Natural tracers	Sampling within the documentation programme	3	3	–	
	Sampling within the monitoring programme	3	3	–	
	Special sampling programme	3	3	–	

Very useful = 3 Useful = 2 Less useful = 1 Not applicable or used = –

13.5.2 Flow paths and arrival time

General

The subject "Flow paths and arrival time" represents estimates of the flow paths and the transport times along these flow paths in the rock mass under certain conditions. It is the spatial distribution of hydraulic properties (hydraulic conductivity and transmissivity and flow porosity), hydrochemical conditions along the flow path and the hydraulic boundary conditions that controls the flow paths and transport times. The subject was a part of the evaluation of the predictions.

The method used was:

- Large scale tracer tests.

A few attempts were made during the pre-investigation and construction phases to estimate the flow porosity of the rock mass. For example, prior to construction a combined long-term pumping and tracer test (LPT-2) was conducted to test the hydraulic connectivity of hydraulic conductors and to derive estimates on flow porosity. During the construction period some efforts were directed to the use of other types of natural tracers as well as to derive transport parameters for non-sorbing transport. A large scale tracer test was performed in fracture zone NE-1 before the tunnel was excavated through the zone in order to estimate the flow porosity and with the purpose to get information useful for the design of the grouting operations.

Judgement of methods

Large scale tracer tests

The test methodology is described in /Almén et al. 1994/. With reference to Table 13-5 the judgement of the methods is as follows.

Large-scale tracer tests are to some extent difficult to perform and interpret but are useful to obtain information on large-scale connectivity. The tests on larger scales may also demand a fairly long test time, involve a large number of observation points for pressure and points for tracer injection. Because of this the large-scale tests also become quite expensive to perform. The test performed in the fracture zone NE-1 was however fairly limited and was not expensive. An important reason for this was that the monitoring system in the surface boreholes was equipped with circulation sections that easily could be used for tracer injection, see Section 11.2. The results were useful but the test conditions were difficult resulting in uncertain parameter estimates. (Flow into the tunnel, and not just the flow out of the flowing borehole where that tracer was collected, probably affected the result and the test time was rather short.)

Test methods and methodology for evaluation have to be better developed to obtain relevant transport parameters. Work of this kind has already started at the Äspö HRL.

13.5.3 Saline interface

General

The subject "Saline interface" represents the spatial distributions of salinity in space and time. The subject is important as the spatial distribution of the density affects the groundwater flow pattern and magnitude. The density of water depends to a large extent of the salinity. The subject was a part of the evaluation of the predictions.

The methods used were:

- Sampling within the documentation programme.
- Sampling within the monitoring programme.
- Monitoring in surface boreholes – electrical conductivity.
- Flow into tunnel- electrical conductivity.

Judgement of methods

With reference to Table 13-5 the judgement of methods is as follows.

Sampling within the documentation programme and the monitoring programme

The sampling programmes are described in Sections 9.2 and 9.3.

Water samples were taken in core holes drilled from ground surface for chemical characterisation in sections equipped for sampling by pumping (generally two sections per core hole drilled from ground surface where also dilution measurements were made) before starting excavation of the tunnel and were repeated for some of the borehole sections as the excavation proceeded. Water samples were taken for chemical characterisation in some core hole sections in the tunnel also but mainly from short probe holes drilled along the tunnel.

Water samples taken for chemical characterisation in core hole sections are considered to be reliable and useful for the groundwater flow modelling. The strategy for taking samples can probably be refined, see Section 13.5.4.

Flow into tunnel and monitoring in surface boreholes – electrical conductivity

The monitoring of electrical conductivity are described in Sections 11.2 and 11.4.5.

The monitoring of the electrical conductivity in the surface boreholes is considered to be less useful as the measurements are uncertain.

At the installation of the packers the test sections are pumped for some time and at the end the electrical conductivity is measured. Low conductive sections are difficult to pump as it takes long time to get the formation water up to surface. The measured electrical conductivity is considered useful but less reliable compared to the water sampling made during the programmes mentioned in Table 13-5.

At the end of the construction phase water samples were also taken at the weirs and generally Cl content, pH and electrical conductivity were measured. The values represent a mean value for a tunnel section that is coupled to the measured flow rate for the same tunnel section. The results from weirs are considered to be reliable and useful for the ground-water flow modelling.

13.5.4 Natural tracers

General

The subject "Natural tracers" represents the spatial distributions of a number of chemical components in space and time. These were ^{18}O, D, T, HCO_3, SO_4, K, Ca, Mg, Na, Sr in the predictions. During the construction period the origin and evolution of the groundwater was described and the effect from mixing and reactions was examined separately, using a new method named Multivariate Mixing and Mass balance calculations (abbreviated to M3) based on the components above. A standard multivariate technique, called Principal Component Analysis (PCA) is the base in these calculations. Extreme waters in the M3 calculations are called end-members and reference waters. A reference water is a well-sampled groundwater which resembles an assumed or modelled end-member e.g. Glacial meltwater. These reference waters are the base for the mixing calculations, see /Rhén et al. 1997a,b/ for more details. The subject is important as the spatial distribution and change with time may give important information of the groundwater flow pattern and magnitude, which are useful for the calibration and testing of groundwater and transport models. The subject was a part of the evaluation of the predictions.

The methods used were:

- Sampling within the documentation programme.
- Sampling within the monitoring programme.
- Special sampling programme.

Judgement of methods

With reference to Table 13-5 the judgement of methods is as follows.

Sampling within the documentation programme, the monitoring programme and special sampling programme

The sampling programmes are described in Sections 9.2, 9.3 and 9.4.

The proportions of different water types, (reference waters, such as glacial, deep saline, Baltic Sea and meteoric) were evaluated from samples collected in the main hydraulic conductors on several occasions during construction. The results can be used to assess the hydraulic connectivity and flow direction in the zones as the proportions of the reference waters change with time in the zones /Laaksoharju and Skårman, 1995/.

Water samples were taken in core holes drilled from ground surface for chemical characterisation in sections equipped for sampling by pumping (generally two sections per core hole where also dilution measurements were made) before starting excavation of the tunnel and were repeated for some of the borehole sections as the excavation proceeded. The results are reported in /Nilsson A-C, 1995/. Water samples for chemical characterisation were taken mainly from short probe holes drilled along the tunnel but also in some core hole sections in the tunnel.

The sampling programmes are considered useful but improvements can be made. It is important to have a sampling strategy that gives a reasonable number of points in space where time series are established for natural conditions as well as for the construction phase. This forms the basis for evaluation and simulation of flow paths and flow times on a large scale. It is also important to measure the flow rates into the tunnel sections during construction and the chemical composition of the water flowing into the tunnel sections.

In future site investigations, more emphasis should be given to the natural tracers as a means of understanding the hydraulic connectivity. The technique developed for evaluating the groundwater types and proportions can be utilised.

13.6 Rock mechanical investigation methods

13.6.1 General

The evaluation of underground investigation methods, used for rock mechanical documentation and characterisation during the construction phase, is based on the subjects for which predictions on three scales were set up at the end of the pre-investigations. During and after tunnel excavation the predicted data and rock mechanical model have been validated against observations and measurements in the tunnel and underground boreholes. Methods used are presented and discussed with respect to precision and usefulness, see also Table 13-6 and Figure 13-4.

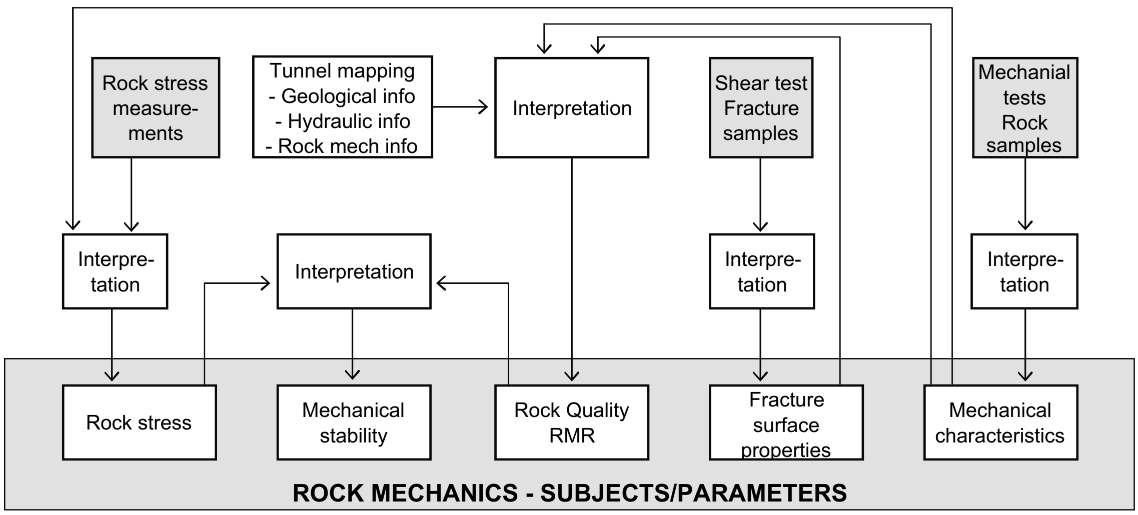


Figure 13-4. *Summary flow chart describing the methods investigating the rock mechanical properites and their subjects/parameters.*

Table 13-6. Judgement of usefulness of the rock mechanical methods used in the construction phase.

Subject	Methods	Usefulness Site scale	Block scale	Detailed scale	Notes
Rock quality	Tunnel mapping	3	3	3	
Rock stress	Rock stress measurements, overcoring method	–	2	1	
Stability	Stability observation in tunnel;	3	3	3	
	– rock burst indication				
	– rock outfall, etc.				
Mechanical characteristics and fracture surface properties:					
– Compressive strength	Unconfined compressive test	–	–	3	
	Empirical references	–	–	2	
– Elastic moduli	Unconfined compressive test	–	–	3	
	Empirical references	–	–	2	
– Poisson's ratio	Unconfined compressive test	–	–	3	
	Empirical references	–	–	2	
– Brittleness ratio	Unconfined compressive test	–	–	1	
	Empirical references	–	–	1	
Fracture surface properties	Shear testing	–	–	2	
	Graphical references	–	–	2	
	Empirical characterisation	–	–	2	

Very useful = 3 Useful = 2 Less useful = 1 Not applicable or used = –

13.6.2 Rock quality

The subject "Rock quality" comprises quality classification of the rock mass. Rock quality is dependent on a number of geological parameters such as rock type, fracture frequency and fracture properties. To quantify the rock quality the Rock Mass Rating system, RMR, has been used. The RMR-system is described by parameters for: 1. Strength of rock material, 2. RQD-value, 3. Spacing of discontinuities, 4. Condition of discontinuities, 5. Water inflow and 6. Joint orientation in relation to the tunnel geometry.

On the site scale, rock quality was predicted based on rock types present and presence of fracture zones in the geological structural model. On the block scale, rock quality was determined based on the properties of the various rock types and their distribution. During the construction phase the subject "Rock quality" was used on all three scales.

Judgement of methods

For the pre-investigation models and prediction, methods such as bedrock mapping, seismic refraction and core borehole investigations were used. The cores were logged to give further information on the different RMR parameters.

During the construction phase the only method used was tunnel mapping. With reference to Table 13-6 the judgement of the method is as follows.

Tunnel mapping of RMR-parameter

With reference to Section 4.5, mapping (estimation) of the RMR-parameters was performed on detailed scale in conjunction with the ordinary geological baseline mapping after every round. The six parameters were measured or estimated at the front along a thought line, perpendicular to the main discontinuity set and the values were summarised to a RMR-value.

The mean value of laboratory measurements of uniaxial compressive strength for the actual rock type was used for the parameter "Strength of rock material". "RQD" and "fracture spacing" were measured and "condition of discontinuities" was observed during mapping. "Water inflow" and "joint orientation" were estimated.

Mapping of fracture zones and hydraulic tests in probe boreholes were used for evaluation of rock quality in the block and site scales.

13.6.3 Rock stress and stability

The subject "Rock stress" refer to measurements of rock stress in boreholes. The subject "Stability" comprises the mechanical stability conditions in tunnel roof and walls.

The Rock stress condition is an important factor for the mechanical stability of underground openings. The main objective of determining rock stress conditions is to verify that the present stress levels are within the normal range in relation to experience from other Swedish underground constructions.

On the site scale and block scale the rock stresses were estimated as the average condition to be anticipated within a rock volume of site or block scale. Variations in rock stresses on the detailed scale will provide information on local variations in different stability aspects.

During the construction phase rock stress measurements were performed on the block scale and detailed scale by use of the overcoring method.

Judgement of methods

With reference to Table 13-6 the judgement of methods is as follows.

Rock stress measurements

The pre-investigation models and prediction were based on rock stress measurements in three deep surface boreholes. The surface borehole measurements employed both hydraulic fracturing and overcoring techniques.

Concurrent with the excavation of the tunnel, overcoring measurements were made in a series of 12 to 18 m long near horizontal boreholes drilled from locations along the tunnel, with the main objective to evaluate predictions made prior to excavation, see further in Chapter 10.

The overcoring technique is believed to provide a high accuracy, especially in determining stress orientation.

Stability observation

Observations, concurrent with tunnel mapping, of rock burst problems like cracking and tendency of spalling is one of the best methods to indicate stability problems due to high rock stresses.

Observations of overbreak, outfall of blocks and general instability in tunnel roof and walls connected with cleaning and inspections is the best method to control the stability conditions in an underground opening.

13.6.4 Mechanical characteristics and fracture surface properties

The subject "Mechanical characteristics" comprises laboratory testing of core samples. The subject "Fracture surface properties" refer to fracture surface investigations of core samples.

During the modelling and prediction stages the mechanical characteristics investigated on the detailed scale comprised rock strength, elastic moduli, Poisson's ratio, brittleness ratio and JRC-values. The same subjects, except the JRC-tests, were used during the construction phase in order to evaluate the predictions on detailed scale.

Judgement of methods

With reference to Table 13-6 the judgement of methods is as follows.

Unconfined compressive tests

During the construction phase, core samples were taken from short holes drilled from the tunnel. A total of ten tests were generally made for each parameter and rock type. The cores for each rock type were selected from 2–8 different boreholes, mainly located in the first 1,500 m of the tunnel. These measurements provided valuable information on local variations of the mechanical characteristics.

13.7 Summary comments of the evaluation of underground investigation methods

In Sections 13.1 to 13.6 methods for more or less direct observation of features coupled to the conceptual model were discussed. A number of methods as positional information (Chapter 3), data evaluation and reporting (for example Section 4.9), sample handling and analysis of water samples (Sections 9.6 and 9.7), data acquisition system (Section 11.5) and data base system (Chapter 12) have not or just briefly been discussed. However, it is very important for the evaluation work (in terms of quality and time) that these parts work well. To some extent the building of models could be separated from the excavation work, but the integrated work approach and frequent updating of models was of great value for the detailed design of the tunnel and for the planning of investigation holes. Of most importance for the model evaluation was also the integration of the key issues "Geological structural model", "Groundwater flow", "Groundwater chemistry", "Transport of solutes" and "Mechanical stability".

13.7.1 Geological structural model

The most useful method for updating the geological model is continuous geological mapping complemented with scan-line mapping of minor fractures. This mapping between every round gives very good information on rock composition, rock boundaries, structures and fracture zones on different scales. Underground core drilling is the best method for investigation the extension and character of fracture zones outside the tunnel. Percussion boreholes also provide some additional information outside the tunnel combined with TV (BIPS) logging and geophysical borehole logging.

Borehole radar and tunnel radar measurements are useful methods for detecting and orientating of especially minor fracture zones and single open fractures.

The VSP (Vertical Seismic Profiling) and HSP (Horisontal Seismic Profiling) methods were found to be useful as a complement to the tunnel radar data for determination orientation of fracture zones.

13.7.2 Groundwater flow

Most of the methods used are considered useful for the updating and refining of the hydrogeological model. There are however a number of methods/methodologies that should be improved as for example the measurements of water flow into the tunnel, flow logging and measurements of large drawdown. The delay of construction of dams and installation of weirs for the flow measurements was not acceptable for part of the construction period. The monitoring system will probably be in operation for a long time and it is important that the system is designed for that. Possibly the monitoring system shown in this report has to be improved to some extent due to this. There are also alternatives of how to sample hydrogeological (and groundwater chemical) information that should be analysed. The suggested alternative (long investigation holes ahead of tunnel face) has positive as well as negative sides compared to the main strategy (short probe holes) used during the excavation. The performance of hydraulic tests can probably also be improved by means of constructing a new test rig that is easy to move and to establish at the boreholes.

Preferably, a few standardised investigation methods should be performed in a consistent way in an entire borehole or along a tunnel. Specially designed tests may then be conducted in, for example, parts of the borehole where a hydraulic conductor domain is assumed to intersect the borehole. This increases the possibility of later re-interpretations and modifications of the models. Another reason is that the variability of the evaluated transmissivities or hydraulic conductivities is very large, which implies that the number of tests should be rather large in order to get reliable frequency distributions as the base for defining the hydraulic properties. Several standard test lengths should be used systematically in all boreholes and a very short test length should be used systematically in selected boreholes. This should provide a good basis for the hydrogeological model.

The standardised investigations should also be comparable to the pre-investigation methods in terms of test section lengths and test times. The evaluation and updating of models will be easier as the scaling of the hydraulic conductivity /Rhén et al. 1997a/ will not be an issue in some analysis made. If anisotropic conditions are present, or believed to be present, the investigation strategy must take that into account.

Grouting will most probably be made along parts of the tunnel system. It is important that the investigation strategy takes that into account as most hydraulic tests must be performed in undisturbed (un-grouted) rock to give useful data for the hydrogeological model. On the other hand, mapping of grouted fractures gives very useful information of the active hydraulic system.

High flow rates from boreholes can probably be found at several sites and the investigation strategy must take that into account. Drilling and tests should be performed in such a way that the problems for the contractor is minimised and the testing methods (most of them at least) must also be feasible in boreholes with high flow rates. In some cases grouting is probably needed if drilling is to be continued and/or some methods are going to be used. In such a case a minimum test program must be performed before grouting. The grouting technique used over the borehole sections with high flow rates and/or low stability can also probably be improved.

From the hydrogeological point of view the hydraulic tests made during the excavation with the TBM was more or less a failure. If TBM is to be used for the excavation it must be much better designed for drilling investigation holes and hydraulic testing. Drilling long cored boreholes ahead of the TBM may be a solution but still the testing possibilities from the TBM should be improved considerably.

The measurement intensity of the monitoring of the water pressures in space and time is judged to be mainly sufficient. However, it would have been preferable with somewhat more reliable measurements of the natural conditions. To some extent the natural conditions were disturbed by performance of the investigations, mainly the hydraulic tests. It is also likely that the investigations within a regional area have to be somewhat more extensive than what were made for the Äspö HRL to get a better confidence of the boundary and initial conditions, mainly hydraulic but also groundwater chemical and on the properties of the rock mass in a regional context. These last points above however concern rather the pre-investigation phase than the construction phase.

13.7.3 Groundwater chemistry

Groundwater samples were collected from probe holes and drips in the tunnel roof. None of these methods turned out to give valuable information, since the hydraulic disturbance by the tunnel had already caused an impact on the undisturbed conditions. Knowing this there are possibilities to improve the results of groundwater sampling in underground excavations. The potentially useful methods are:

- Groundwater sampling during drilling of long pilot holes.
- Arrangements for sampling of probe holes in the tunnel.

The sampling in the pilot holes is used to give the unperturbed conditions. The arrangements in the probe holes are made to give a view of changes caused by the tunnel.

13.7.4 Transport of solutes

The groundwater flow and chemistry have been carefully followed during the construction of the HRL tunnel. The experiences from this work indicate that there have been major changes of the conditions due to the tunnel excavation and inflow to the tunnel. It is therefore not possible to observe the undisturbed condition and the changes thereafter in the short probe holes. Long probing holes for each tunnel leg may be useful to investigate the dynamic groundwater conditions during an excavation. Short probe holes can be used for sampling when the long time changes around the constructed tunnel is to be monitored.

It is important for modelling purpose to have a sampling strategy of the important chemical constituents that gives a reasonable number of points in space where time series are established for natural conditions as well as for the construction phase.

A few deep boreholes for sampling of groundwater and hydraulic tests are needed to support the modelling of transport of solutes, but also groundwater flow and groundwater chemistry. It is not sufficient to just take samples in boreholes close to surface and in boreholes from the tunnel.

13.7.5 Mechanical stability

The most useful method for updating the Rock quality subject of the model is mapping of the RMR-parameters in conjunction with the geological tunnel mapping after each round. Mapping of fracture zones was used for updating of rock quality on different scales.

Rock stress measurements, by use of the overcoring method, were valuable to provide information on the variation of rock stresses in the rock mass.

Observations of rock burst problems like cracking and tendency of spalling is the best method to indicate stability problems due to high rock stresses.

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