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**Construction experiences from
underground works at Oskarshamn
Compilation report**

Anders Carlsson, Vattenfall Power Consultant AB

Rolf Christiansson, Svensk Kärnbränslehantering AB

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Svensk Kärnbränslehantering AB

Swedish Nuclear Fuel
and Waste Management Co
Box 250, SE-101 24 Stockholm
Tel +46 8 459 84 00



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Preface

The main objective with this report is to compile experiences from the underground works carried out at Oskarshamn, primarily construction experiences from the tunnelling of the cooling water tunnels of the Oskarshamn nuclear power units 1,2 and 3, from the underground excavations of Clab 1 and 2 (Central Interim Storage Facility for Spent Nuclear Fuel), and Äspö Hard Rock Laboratory. In addition, an account is given of the operational experience of Clab 1 and 2 and of the Äspö HRL on primarily scaling and rock support solutions.

This report, as being a compilation report, is in its substance based on earlier published material as presented in the list of references.

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1 Introduction

The Swedish Nuclear Fuel and Waste Management Co (SKB AB) is currently undertaking site characterisation at two different locations, the Forsmark and Simpevarp/Laxemar areas, with the objective of siting a geological repository for spent nuclear fuel. This report summarises the underground construction experiences from the Simpevarp project area and Äspö.

Within the Simpevarp area in the north-eastern part of the municipality of Oskarshamn, on the south-coast of the Baltic Sea, there are three major existing projects: the Oskarshamn Nuclear Power Station (O1, O2 and O3 units) including an underground storage site for medium and low grade radioactive waste (BFA), the Central Interim Storage Facility for Spent Nuclear Fuel (Clab 1 and 2) and the Äspö Hard Rock Laboratory (HRL).

Approximately 8,000 m of tunnels including three major rock caverns with a total volume of about 550,000 m³ have been excavated. In addition, there have been approximately 900,000-m³ open-cut excavations. The excavation of the various tunnels, rock caverns and foundations was carried out during the period of 1966–2000. In addition, minor excavation works were carried out at the Äspö HRL in 2003. The project area is shown in Figure 1-1.



Figure 1-1. A plan view of the project area showing the major projects. The underground structures are shown in green colour /Curtis et al. 2003/.

1.1 Objective

The main objective with this report, the Construction Experience Compilation Report (CECR), is to compile experiences from the underground works carried out at the Simpevarp project area and Äspö, primarily construction experiences. Based on requirements of the Final Repository, construction experiences are summarised from excavation, sealing and support point of view. Maintenance records are used to conclude the operational experiences of primarily rock support solutions. The Report can be used as a reference and support to other empirical methods.

1.2 Working methodology

The focus on the efforts to compile information on experiences from the underground construction at the sites has been to present the prerequisites for each of the three major construction projects at the Simpevarp and Äspö sites. This includes the actual understanding of the site conditions, the requirements on each of the underground projects and the construction methods used in each of the projects.

There is a considerable variation in the amount and wealth of detail in the underground and rock engineering data concerning the three construction projects, varying from almost nil to unprecedented quantity of information. These circumstances are evident in this Compilation Report, primarily in the scope and degree of detail when describing each of the three projects.

The main reasons for this range of variation in data are partly construction at the time being, and partly the purpose of each individual project. Considering the start of construction works, 1966 (O1) and 1970 (O2), it is not particularly surprising that there is a limited amount or sometimes non-existent underground construction data for Oskarshamn nuclear power units 1 and 2 including BFA. Tunnel documentation such as engineering-geological follow up, rock support documentation, etc was by no means a common practice at that time, while today such documentation is routine for projects with high demands on functionality and safety. For the construction phase of Oskarshamn nuclear power unit 3 starting in 1976, the geological documentation and rock condition descriptions showed a great improvement in scope, but still limited in both quantity and quality. As for Clab 1 and 2, the situation is quite different with site investigation reports, detailed geological mapping, rock support documentation and various rock engineering monitoring data covering the entire construction and operational phases.

One of the fundamental reasons behind SKB's decision to construct the Äspö Hard Rock Laboratory was to create an underground laboratory for research, development and demonstration in a realistic and undisturbed rock environment down at a realistic repository depth. Geoscientific research has been and is a natural part of the activities at Äspö HRL and is conducted in the fields of geology, hydrogeology, geochemistry and rock mechanics. Consequently, exceptional comprehensive geoscientific detailed site descriptions have been produced from the commencement of the site investigations in 1986, during the whole construction phase 1990–1994, and not at least from the commencement of operation (1995) up to present.

The authors have in principle focused on published material, mainly on SKB's reports, but also on other Swedish and internationally published papers. The main reason for using already published material is that the content has been checked and reviewed before publication, but even equally important, that the sources of information are traceable.

2 A learning process

Each of the major construction projects started with the specific experiences and requirements at the time being. It is also of importance to point out that two project owners have been acting in the area, the OKG Power Company and SKB. The first design and construction works at the site for the Oskarshamn 1 and 2 tunnels were to a high degree based on the experiences that the main owner of OKG at that time, Sydkraft AB, had earned from decades of hydropower projects in Sweden. SKB has had two independent construction projects in the area, the Central Storage Facility for Spent Fuel (Clab) and the Äspö Hard Rock Laboratory (HRL). The latter project started with the R&D experiences of site characterization methods from the Stripa Mine projects. In addition, based on both experiences from the SFR facility at Forsmark and by cooperation with the Atomic Energy of Canada Limited (AECL) at their Underground Research Laboratory (URL) in Manitoba in Canada, additional experiences from contractual arrangements for construction of a research facility were incorporated. Gradually through the works, the site-specific experiences were also guiding the works. The site-specific experiences were used in the following investigation and design works. This is illustrated in Figure 2-1.

The formal periodic review of the status of the Clab and Äspö HRL facilities has also been used as a reference for the design of a final repository at the Laxemar site from the lessons learned from maintenance of the underground facilities during 20 years.

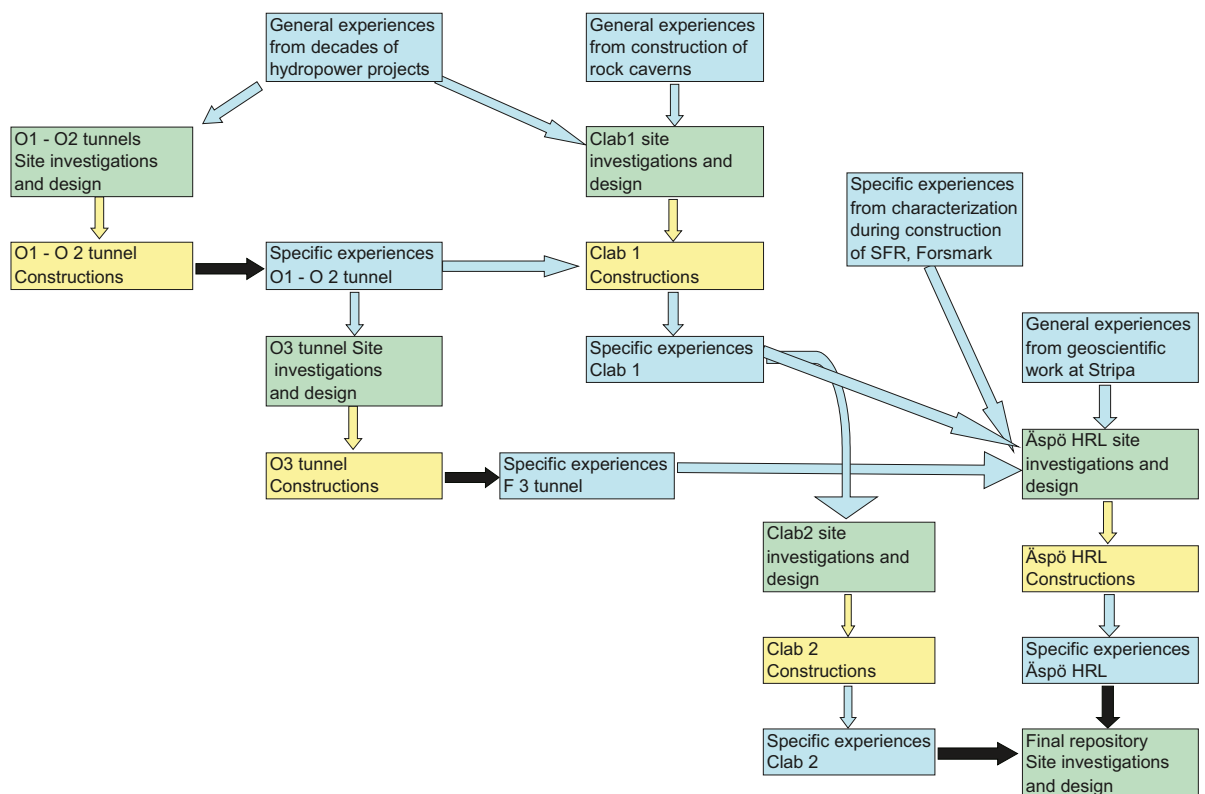


Figure 2-1. The sequential process of feedback from one construction project to another.

3 Geological and structural overview

3.1 Bedrock geology

The geological development in the Oskarshamn region, including the formation of existing rocks, as well as structural and tectonic overprinting, is complex and spans a period of c 1,900 Ma. The geological conditions have been studied in great detail during the last years within the ongoing site investigations for a final deep repository /SKB 2006b/. The results of the ongoing site investigations are used in this context to give a description of the geoscientific conditions in a regional scale compared to the conditions at Simpevarp and the Äspö HRL.

The following text gives a brief summary and for further information of the geological evolution and processes that might have affected the bedrock in the Oskarshamn region and the rest of the southern part of the Fennoscandian Shield, the Reader is referred to e.g. /Larson and Tullborg 1993, Milnes et al. 1998, Berglund et al. 2003/ and /SKB 2006a/. A map of the regional geology is shown in Figure 3-1.

The oldest rocks in the Oskarshamn region, though subordinate, comprise more or less strongly deformed and metamorphosed supracrustal rocks of predominantly sedimentary but also of volcanic origin. The formation of the metasedimentary rocks is constrained to the time interval c 1,870–1,860 Ma /Sultan et al. 2004/, and the rocks have their main expression north of the Äspö area /Bergman et al. 1998, 1999, 2000/. In the area immediately north of Oskarshamn and westwards, metagranitoids constitute an important lithological component.

The majority of the rocks at the present day erosional level in south-eastern Sweden were formed during a period of intense igneous activity c 1,810–1,760 Ma ago /e.g. Wikman and Kornfält 1995, Kornfält et al. 1997/, during the waning stages of the Svecokarelian orogeny. The dominant rocks comprise granites, syenitoids, dioritoids and gabbroids, as well as spatially and compositionally related volcanic rocks. The granites and syenitoids, as well as some of the dioritoids are by tradition collectively referred to as “Småland granites”. Equigranular, unequigranular and porphyritic varieties occur, and the compositional variation is displayed in Figure 3-2. Hence, the “Småland granites” comprise a variety of rock types regarding texture, mineralogical and chemical composition. The locally occurring Ävrö granite has been shown to have a large compositional variation, and thereby also a large variation in thermal properties /SKB 2006a/.

This generation of igneous rocks belongs to the so-called Transscandinavian Igneous Belt (TIB), which has a NNW extension into southern Norway. It is characterised by repeated alkalicalcic-dominant magmatism. Magma mingling and mixing processes, exemplified by the occurrence of enclaves, hybridization and diffuse transitions etc between different TIB rocks indicate a close time-wise and genetic relationship between the different rock types.

Locally, fine- to medium-grained granite dykes and minor massifs, and also pegmatite occur frequently. Though volumetrically subordinate, these rocks constitute essential lithological inhomogeneities in parts of the bedrock in the Oskarshamn region, e.g. in the Simpevarp area. They are roughly coeval with the TIB host rock /Wikman and Kornfält 1995, Kornfält et al. 1997/, but have been intruded at a late stage in the magmatic evolution. Furthermore, TIB-related mafic and composite intrusions occur locally.

The next rock-forming period in the Oskarshamn region took place c 1,450 Ma ago. It was characterised by the local emplacement of granitic magmas in a cratonized crust. However, this granitic magmatism was presumably a far-field effect of ongoing orogenic processes farther to the southwest of present Scandinavia. In the Oskarshamn region, the c 1,450 Ma magmatism is exemplified by the occurrence of the Götemar, Uthammar and Jungfrun granites, cf Figure 3-1 /Kresten and Chyssler 1976, Åhäll 2001/.

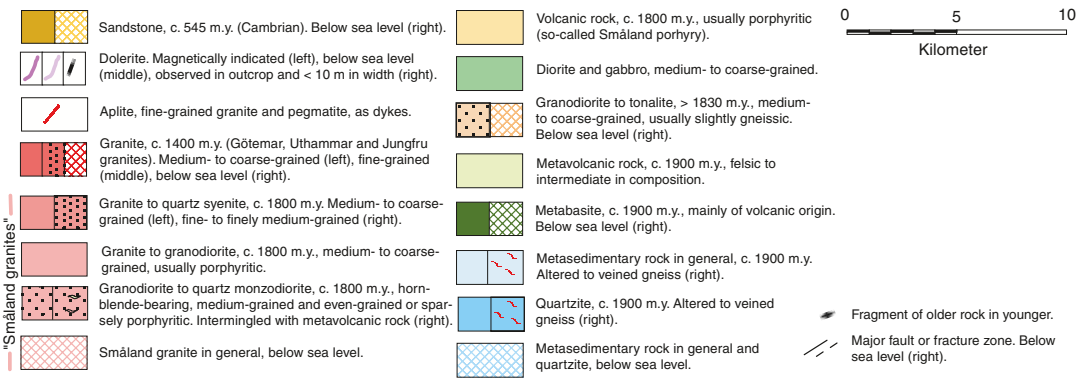
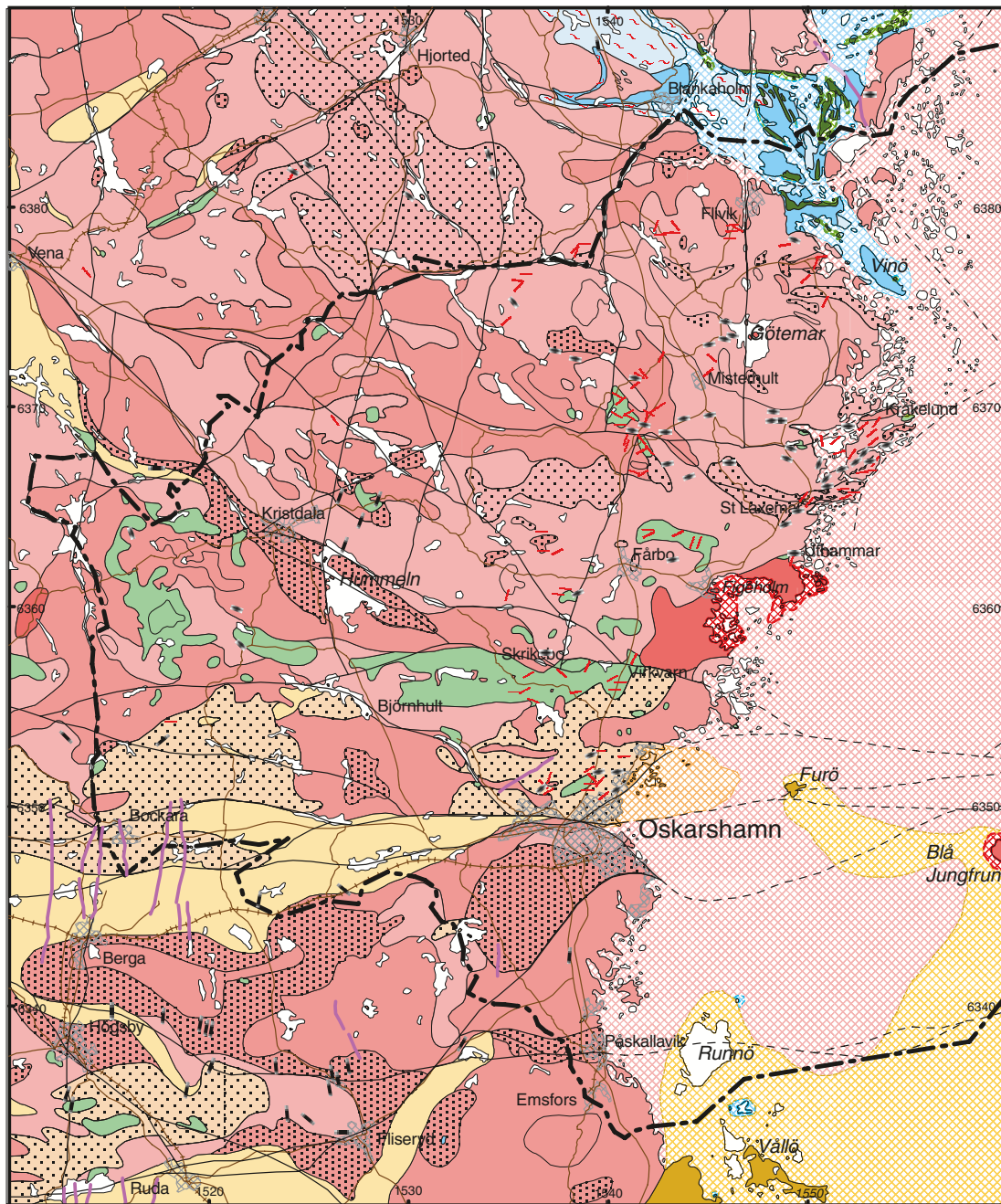


Figure 3-1. Bedrock map of the Oskarshamn municipality and the surrounding area /SKB 2006a/.

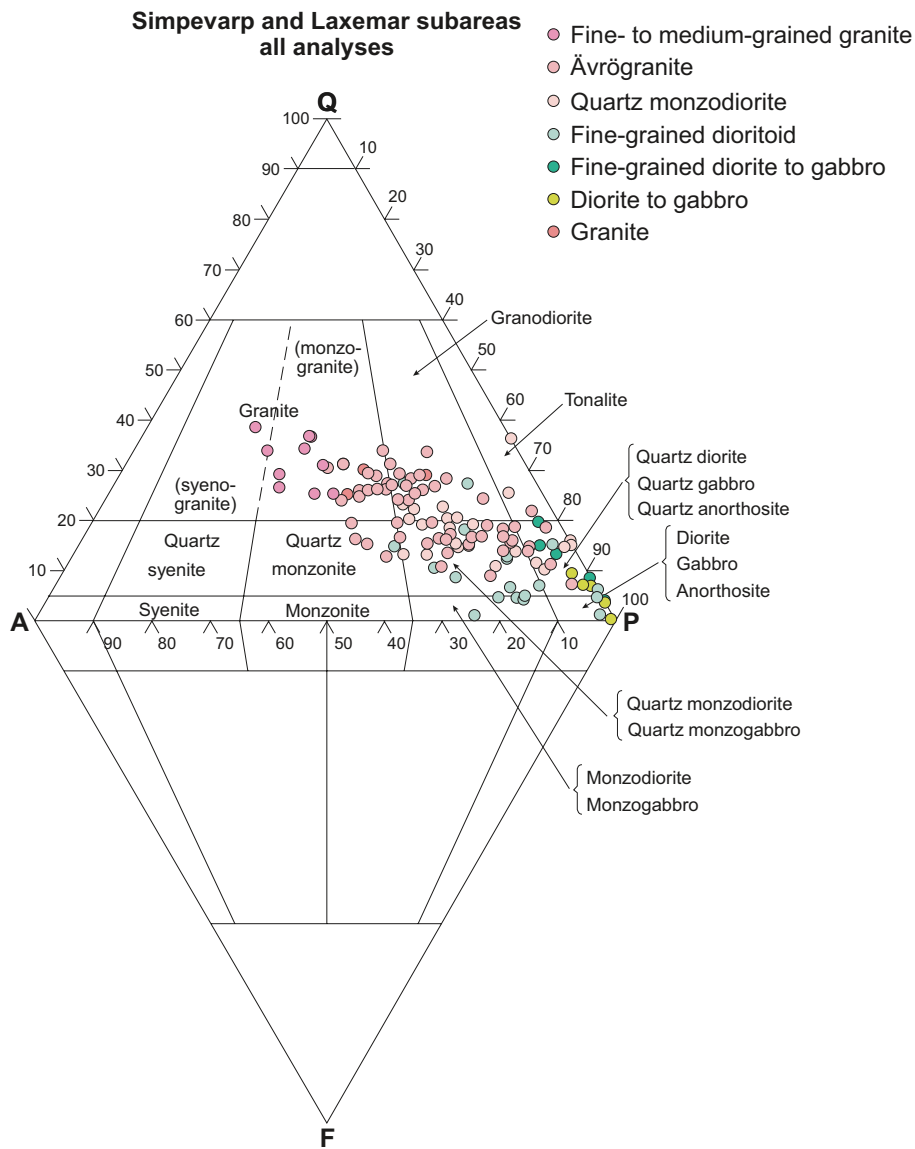


Figure 3-2. QAPF-diagram displaying the compositional variation of magmatic rocks, exemplified with modal analyses of rocks from the Simpevarp and Laxemar subareas /SKB 2006a/.

The youngest magmatic rocks in the region are scattered dolerite dykes that presumably are related to the regional system of N-S trending dolerites /Johansson and Johansson 1990, Söderlund et al. 2004/. Due to the generally high content of magnetite, they usually constitute linear, positive magnetic anomalies, and their occurrence and extension may, thus, be identified on the magnetic anomaly maps. Time-wise, they are related to the c 1,100–900 Ma Sveconorwegian orogeny that is responsible for the more or less strong reworking and present structural geometry in the bedrock of southwestern Sweden.

The bedrock at Äspö consists primarily of magmatic rocks belonging to the TIB as described above.

Earlier investigations at the Äspö site had a simplified description of the lithology. In the detailed scale (boreholes, tunnels) the different rock types are commonly “mingled” into each other (cf Section 3.4). It is difficult to visually tell the difference between the variants of the granite. During the construction of the HRL density logs in boreholes were used to estimate the distribution of the Ävrö granite and the Äspö diorite. Sections with a density larger than 2,700 kg/m³ were defined as diorite.

3.2 Structural development

3.2.1 Ductile deformations

The bedrock of southeastern Sweden has gone through a long and complex structural development, including both ductile and brittle deformation, since the formation of the oldest supracrustal rocks. The oldest deformation, which was developed under medium- to high-grade metamorphic conditions, was heterogeneous in character. The TIB rocks are characterised by a system of ductile deformation zones that seem to be developed during low-grade metamorphic conditions. Presumably, also the NE-SW trending Äspö shear zone /Gustafson et al. 1989, Bergman et al. 2000/, which is characterised by a sinistral strike-slip component, belongs to this system of ductile deformation zones.

Independent of the syn-deformational metamorphic grade, the dextral and sinistral strike-slip component in the WNW-ESE to NW-SE and NE-SW trending ductile deformation zones, respectively, indicate that a regional, c N-S to NNW-SSE compression prevailed during their formation and subsequent ductile reactivation. Consequently, this regional stress field is inferred to have prevailed for a considerable period, at least from the time of the intrusion of the 1,850 Ma TIB generations, or possibly earlier, until c 1,750 Ma ago. Most of the lithological contacts in the region, and also in the whole of southeastern Sweden, are more or less concordant with the orientation of the ductile deformation zones, which indicates that the emplacement of the TIB magmas was facilitated by ongoing shear zone activity. Together with the subsequent deformation of the TIB rocks, this testifies to the influence of the deformation zones in the present structural and lithological framework in the bedrock of southeastern Sweden.

3.2.2 Brittle deformations

Since no ductile deformation has been observed in the c 1,450 Ma granites /e.g. Talbot and Ramberg 1990/ or younger rocks, it is evident that only deformations under brittle conditions have affected the bedrock in the Oskarshamn region during at least the last c 1,450 Ma. However, the transition from ductile to brittle deformation presumably took place during the time interval c. 1,750–1,700 Ma, i.e. during uplift and stabilization of the crust after the Svecofennian orogeny.

During the subsequent geological evolution, faults and older ductile deformation zones have been reactivated repeatedly, due to the increasingly brittle behaviour of the bedrock. Brittle reactivation of ductile deformation zones is a general phenomenon. For example, the Äspö shear zones display clear evidence of being reactivated in the brittle regime /see also e.g. Munier 1995/. An inversion of the strike-slip component in the Äspö shear zone from sinistral during the older ductile deformation, to dextral during the younger brittle reactivation has been proposed by /Talbot and Munier 1989/ and /Munier 1989/.

The occurrence of c 1,000–900 Ma dolerite dykes in southeastern Sweden testifies to an Sveconorwegian tectonic influence, as the intrusion of the parent magmas was tectonically controlled.

According to /Milnes and Gee 1992/ and /Munier 1995/, the Ordovician cover rocks along the north-western coast of Öland are tectonically undisturbed, except for displacements at the centimetre scale. This indicates that these brittle deformation zones of regional character were not active in post-Cambrian time, but is related to the Precambrian tectonic evolution. However, post-Cambrian fracture zones/faults do occur in the Oskarshamn region. For example, the occurrence of joints filled with sandstone only east of the N-S trending fault in the western part of the Götö granite, i.e. the eastern block has been down-faulted in relation to the western block /Kresten and Chyssler 1976, Bergman et al. 1998/.

Most of the bedrock in the Simpevarp-Äspö area is only weakly deformed and fairly homogeneous at outcrop scale. The coastal areas around Laxemar, Simpevarp, Ävrö, Hälö and Äspö are,

however, located in a large-scale network of shear zones of ductile to semi-ductile character /SKB 2006a/. They are of regional, local major and local minor scale. Exposed individual shear zones in the network rarely exceeds a width of 1 m, but e.g. the Äspö shear zone (EW1) is approximately 100 m wide in the central part of the Äspö island. The shear zone is the western boundary of this coastal shear belt and has been traced as a magnetic lineament from the Uthammar granite in the south to at least 1–2 km northeast of Äspö. In addition to the exposures at Äspö it is observed in outcrops at three localities along the lineament south of Äspö /Bergman et al. 2000/. The background geophysical and topographical data from the ongoing site investigations /SKB 2006b/, together with available lithological information, show an area with multiple contacts between several rock types and intense magnetic anomalies which do not provide a conclusive interpretation of the Äspö shear zone.

3.3 Engineering-geological overview Simpevarp project area – local scale

This brief, simplified overview of engineering-geological features is based on investigations and interpretations conducted during the course of the planning and construction stages of the Clab facility as well as on observations during construction. The main references used are /Moberg 1978, 1979, Eriksson 1982, Stanfors et al. 1997c, Stanfors and Larsson 1998, Larsson and Leijon 1999, Curtis et al. 2003/.

3.3.1 Lithology

The rock types in the area of the Simpevarp peninsula are generally homogeneous over large areas, but locally around Clab greater variation has been found with numerous dykes. Red-greyish to red, fine-grained granite occurs throughout the area, occasionally as small equidimensional bodies but more commonly as irregular dykes and veins in the older rock mass. The dykes and veins often run parallel with the foliation in a general E-W to NE direction. The degree of schistosity is very variable and there have occurred numerous different types and generations of alteration /Curtis et al. 2003/.

The main rock type within the site area of Clab (also within O1, O2 and BFA) is metavolcanites which consist of altered grey to greyish black fine-grained rocks of volcanic origin that can be generally termed dacites and andesites. The metavolcanite has been intruded to such an extent by Småland granite that it has recrystallized and has itself developed a more granitic structure. For the most part of the siting area of Clab, the metavolcanite may be described as a mixed rock type of granite and metavolcanite /Stanfors and Larsson 1998, Curtis et al. 2003/.

A simplified bedrock map of the Simpevarp peninsula including the major projects and the underground structures is presented in Figure 3-3.

3.3.2 Fractures and fracture zones

The fracture frequency can in general terms be correlated with rock type. The granite, consisting of granite to monzonite, has a typical block size of 1–3 m³. Whereas the dyke rocks, principally the red granite dykes, aplite, pegmatite and fine-grained vulcanite are more fractured having RQD values of about 50% and a block size of approximately 1 dm³.

Fracture mapping on surface rock outcrops was carried out to supply base data for the design of Clab /Moberg 1979/. The result showed a clearly dominant fracture orientation of ENE with dips generally steep to moderately steep. A secondary orientation of NS to NW, steeply dipping, was also identified. In addition the later drilling work identified the frequent presence of gently dipping fractures in the area.

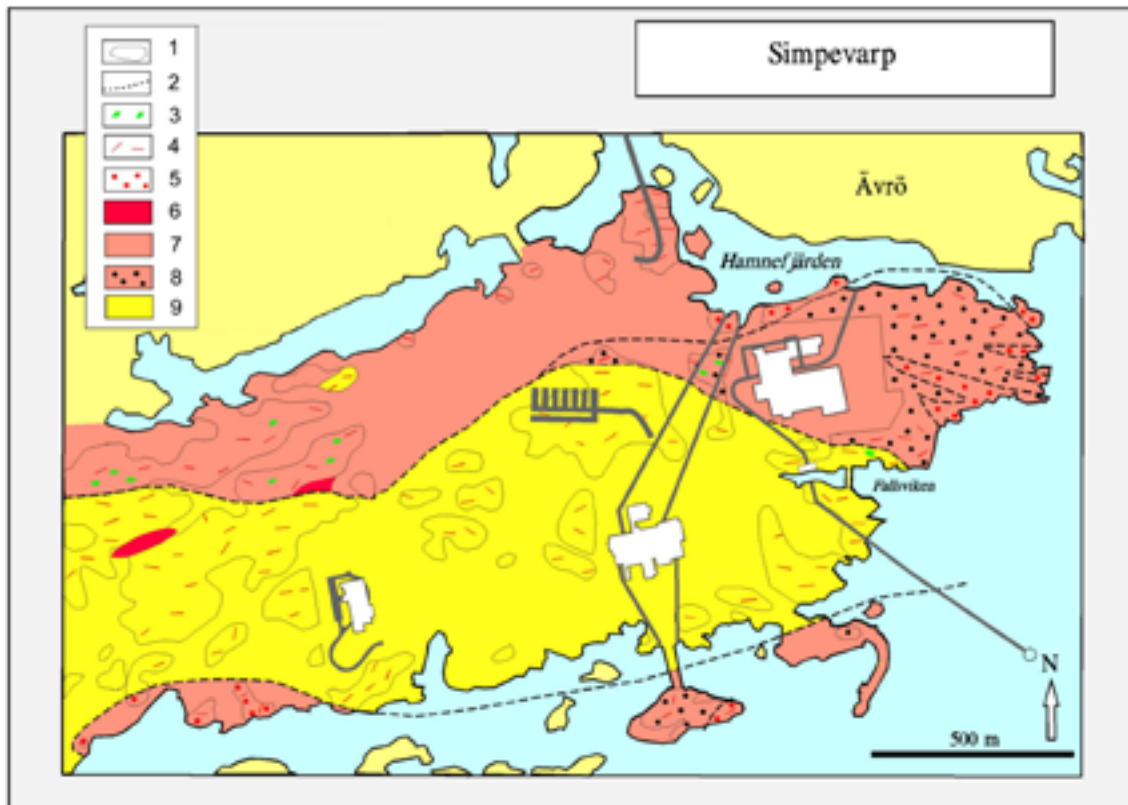


Figure 3-3. Simplified bedrock map of the Simpevarp peninsula including the major infrastructures and the underground structures. 1. Rock outcrop boundary, 2. Rock type boundary, 3. Inclusions of greenstone, 4. Veins of fine-grained granite, 5. Porphyritic structure, 6. Fine-grained granite, 7. Red to grey-red granite, 8. Quartz-diorite, 9. Dactylic to andesites vulcanite. After /Kornfält and Wikman 1987, Stanfors and Larsson 1998, Curtis et al. 2003/.

Fracture data from the core holes carried out for Clab 1 are compiled in Table 3-1. The table shows average fracture frequency (number of fractures per metre) subdivided into four different classes along with the percentage of each class represented as in each borehole. As seen, the fracture frequency is relatively high with an average frequency of 5–6 fractures per metre. The result from the geological mapping of Clab 1 cavern and access tunnel during construction is shown in Figure 3-4, and for Clab 2 in Figure 3-5.

During the construction phase, the mapping of the Clab 1 transport tunnel and the rock cavern identified the dominating fracture orientations to be as follows (Table 3-2).

The most common fracture fillings are chlorite and calcite, but clay also occurs as filling material. X-ray diffraction analysis was performed showing the main clay minerals to be illite, chlorite, montmorillonite and mixed layers with smectite components. The result of the diffraction analyses showed no or insignificant swelling tendency.

In Clab 2, the fracture mineralogy is dominated by chlorite followed by calcite. Quartz and epidote are of rare occurrence. Clay occurs in thinner horizons (about 1–20 mm) in two deformation zones, but otherwise only in limited extent. The RQD values are normally > 60, and often around 80–90 in the granitoid. There are 3–5 joint sets but only three sets occurred in each round.

In general, the rock mass of Clab 2 may be characterized as good or relatively good rock. According to definition poor rock in the Q-system occurs locally in connection with the shear zones, which are intersected by the rock cavern, canal transept and transport tunnels /Berglund 2001/.

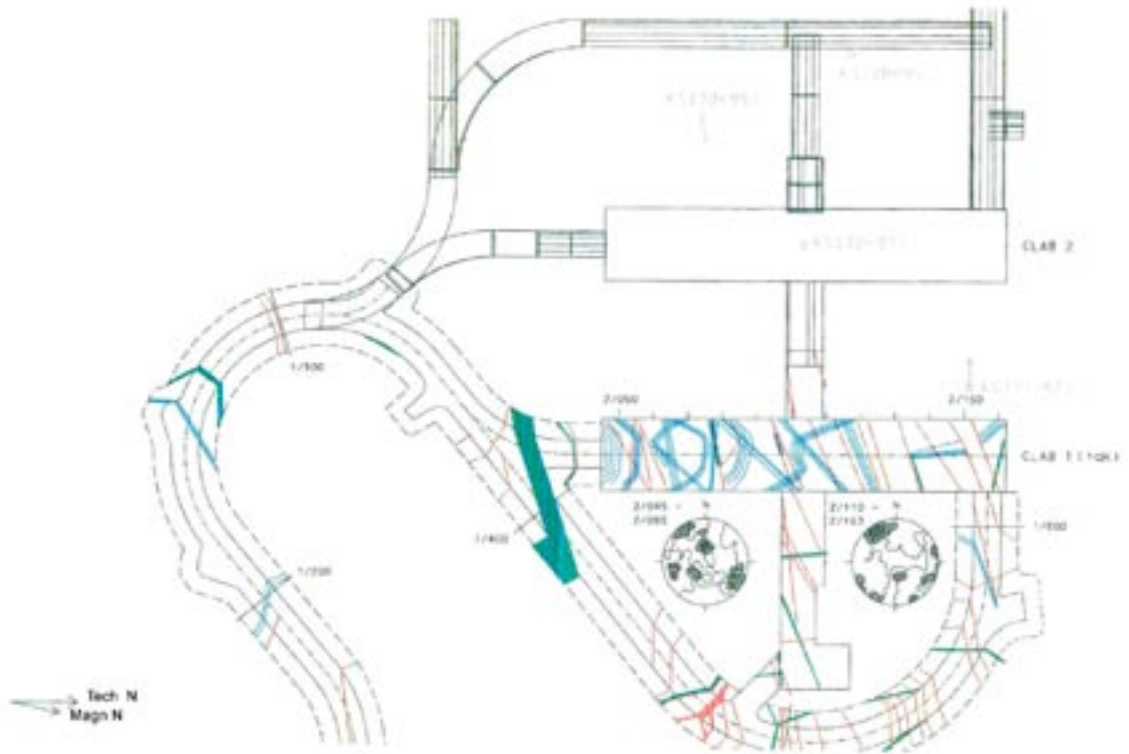


Figure 3-4. An overview of the geological mapping of cavern and tunnels, Clab 1. Modified after /Larsson and Leijon 1999/.

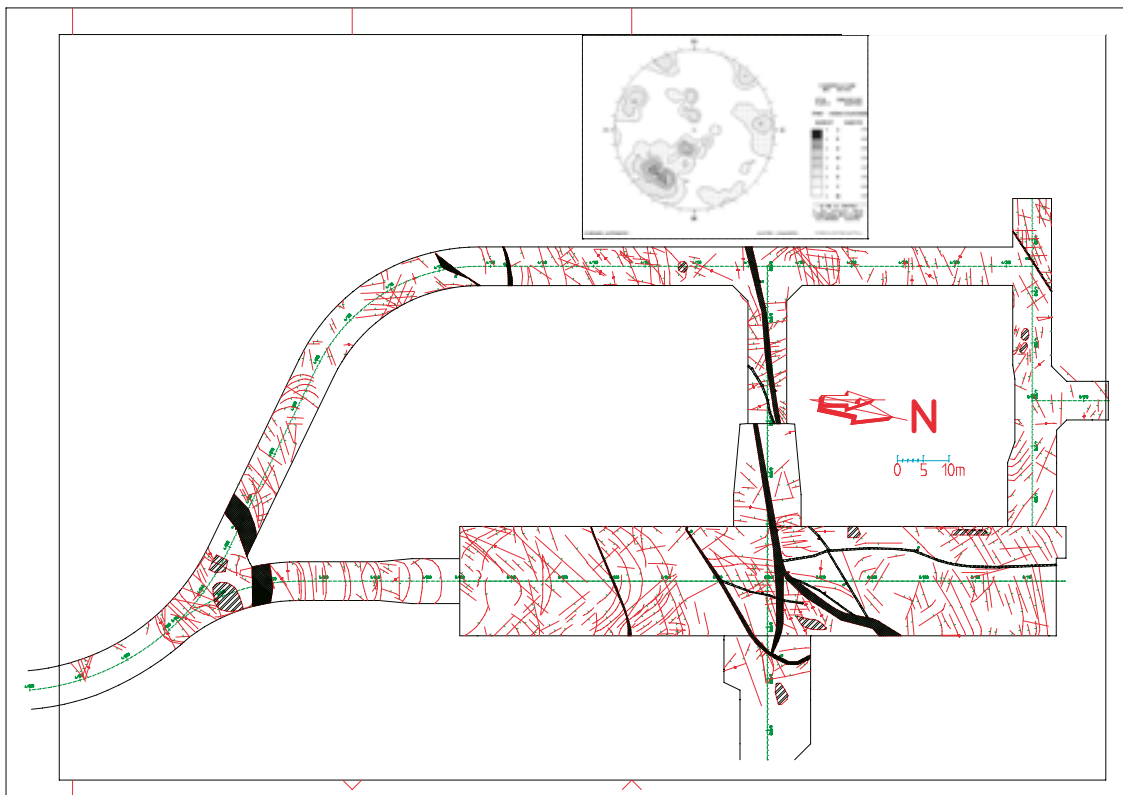


Figure 3-5. An overview of the geological mapping of cavern and tunnels, Clab 2. Modified after /Berglund 2001/.

Table 3-1. Data from core holes D1 to D11, carried out in connection with the construction of Clab 1 /Moberg 1979, Stanfors and Larsson 1998/.

Borehole, length (m)	Fracture class								
	1–3 frac/m		3.1–6 frac/m		6–9 frac/m		> 9.1 frac/m		
	Frac/m	%	Frac/m	%	Frac/m	%	Frac/m	%	
D1/40N	95	2.8	10	5.7	24	6.6–7.2	23	14–19	43
D2/45W	84	0–3	9	3.1–6	45	6.3–8.3	27	12–16	19
D3/45S	87	0–3	9	4.5	28	6.8–7.5	18	11–15.5	45
D4/75S	69	0–3	15	5.6	36	7.2	27	16.5	22
D5/90	62	0–3	21	3.9	45	6.6	21	13.0	13
D6/45S	86	0–3	21	4.8–5.2	36	7.1–8.7	24	9–10.7	19
D7/60N	72	0–3	17	3.0–3.5	25	7.7–8.2	26	10.2–11	32
D8/30W	73	0–3	19	3.1–6	55	6.1–9	19	> 9.1	7
D9/90	16	1.8	6.3	4.9	31	6.1–9	6	> 9.1	0
D10/90	50	0–3	8	4.5–5	38	6.1–9	22	9.3–9.7	32
D11/90	57	0–3	35	3.5–5.1	30	7.1	26	> 9.1	9

Table 3-2. Dominating fracture orientations in the transport tunnel and rock cavern, Clab 1 after /Eriksson 1982/.

Dominant fracture orientations, Clab 1			
Transport tunnel		Rock cavern	
Strike	Dip	Strike	Dip
N50E	90	N60E	80S
N65W	80S	N30E	80W
N25W	90	N20W	80E-90
N50E	30S		
N45W	20S		

Within the Clab site area, on the western part of the peninsula, there are three topographically as well as geophysical well-pronounced fracture zones. The zone immediately to the south of the rock cavern, and intersected by the transport tunnel has an orientation of ENE dipping almost vertical (cf Figure 3-4). The width of the zone is about 5 m. The zone, close to the north of Clab, has also an orientation of ENE with a dip angle 60–70° to the south. This zone is verified with core drilling and the width is estimated to 10–20 m. The third zone, about 200 m to the west of Clab has a N-S orientation, and according to borehole investigations exhibited as a heavily water conducting zone. Other zones, which were observed during the underground excavation works, may be characterized as having a limited width, from some 0.1 m up to 1 m.

3.3.3 Hydrogeological conditions

The excavation of the cooling water tunnels for O1, O2 and O3, as well as for the rock excavation of BFA encountered no significant inflows of water. The superficial rock mass at the foundations of the units was also very tight, although excavation depths reached down to 15 m below sea level. Virtually no water inflow to the foundation excavations was reported /Larsson and Leijon 1999, Curtis et al. 2003/.

The groundwater inflow in the transport tunnel and rock cavern for Clab 1 was measured as average inflow per week. Dripping or flowing groundwater occurred in some 20 locations in the transport tunnel. The variation of water inflow per week varied between 30–50 l/min indicating

a tight rock mass. These conditions were also supported by the water injection tests performed in 10 m deep boreholes drilled from the floor of the rock cavern, which showed only very small water loss. A major part of the peaks in inflow coincide with an increase of the precipitation.

Thus, the water inflow into the underground complex is small. At the start of operation in 1985, the total inflow to the transport tunnel and the rock cavern amounted to 60 l/min, and after 12 years of operation of the facility, the inflow had decreased to 40 l/min. This is equivalent to 5–6 l/min and 100 m tunnel.

Two distinct, foreseen, water-bearing shear zones were encountered in Clab 2, which involved an increased amount of temporary and permanent rock support as well as pre- and post-grouting. An extra effort in rock bolting was needed in the northern walls of the transept due to steeply dipping fractures with chlorite fillings. In addition to the two water-bearing shear zones, there are a number of thin deformation zones, which did not show any or only minor water inflows.

The purpose of the grouting, which were carried out was to limit the water inflow to Clab 2 to 30 l/min. In fact, the demand on maximum allowed water inflow was well contained. The total water inflow is less than 10 l/min /Bodén 2002/. The ground water analyses on samples from the rock caverns, pumping pits, drill holes and wells do not indicate any noteworthy contents of measured substances.

3.3.4 Rock stresses

In situ rock stress measurements by over-coring were carried out in three boreholes, and later in an additional two boreholes for Clab 2. The measurement device used for the measurements was developed by Vattenfall and is based on Leeman's three-dimensional theories /Hiltscher et al. 1979/. The result of the in situ stress measurements is summarized in Table 3-3 (including the measurements for Clab 2, 1997:1 and 1997:2).

According to the performed tests of cores, the average uniaxial strength was 200 MPa for un-fractured cores with a diameter of 42 mm and a relation length/diameter equal to 2.5. The E-modulus was determined to 80–94 GPa and the Poisson's ratio to an average of 0.27 with a variation of 0.23–0.33.

3.4 Engineering-geological overview Äspö HRL – local scale

This brief, simplified overview of engineering-geological features is based on investigations and interpretations conducted during the course of the planning and construction stages of the Äspö HRL facility as well as subsequently executed research work and monitoring during construction up to date. The main references used are /Berglund et al. 2003, Rhén et al. 1997b, Andersson and Söderhäll 2001/.

Table 3-3. In situ rock stress measurements, Clab*.

Hole	Measuring depth (m)	σ_H MPa	σ_h MPa	σ_v MPa	Orientation σ_H (°)
1997:1	34	6.6	4.7	4.5	9
1997:2	60	4.3	2.5	7.7	23
1979:1	23	7.6	4.0	-0.1	47
1979:2	41	7.1	2.0	2.4	42
1979:3	56	6.3	2.5	0.7	80

* The result is given as mean values for each measuring level and three measurements at each measuring level.

3.4.1 Lithology

The bedrock at Äspö consists primarily of magmatic rocks belonging to the TIB as described in Section 3.1. The main rock types that predominate in this generation are:

- 1) A medium-grained, equigranular granite to granodiorite, including subordinate quartz monzonite and monzodiorite (old name: Ävrö granite).
- 2) A medium-grained, sparsely to strongly porphyritic intrusive rock that varies in composition between granite and quartz diorite, including tonalitic, granodioritic, quartz monzonitic and quartz monzodioritic varieties (old name: Äspö diorite).
- 3) A grey, fine-grained, at places slightly porphyritic, intermediate rock.
- 4) Furthermore, dykes of fine-grained granite and pegmatite are frequently occurring.
- 5) Mafic rocks. These are undifferentiated amphibolites, but most of them are considered to be genetically related to the granitoids and dioritoids of TIB.

A simplified bedrock map of Äspö including the underground structures is presented in Figure 3-6.



Figure 3-6. Simplified bedrock map of Äspö including the underground structures /Andersson and Söderhäll 2001/.

3.4.2 Fractures and fracture zones

The distribution and orientation of fractures in the HRL area are presented in Figure 3-7. These fracture arrays are also present in a larger area, although the strong predominance of steep NW striking fractures found at southern part of the Äspö Island seems to be local phenomena. At other localities in the region, the northeast direction of fractures seems to be equally or even more important /SKB 2006a/.

Much of the input to the conceptual understanding of the HRL has successively been developed at Äspö in projects like “True Block Scale” /Andersson et al. 2002/, “Fracture Classification and Characterisation” /Bossart et al. 2001/ and Tracer Retention Understanding Experiment” /Winberg et al. 2000/. In the detailed scale, it is often found that ductile deformation zones are continuous over at least tens of metres along their strike. They are typically reactivated, possibly under ductile conditions, but generally at brittle conditions. The ductile deformation zones generally also have a higher frequency of fractures related to them, as compared to the average rock mass. This is the case both within and in the close vicinity to the deformation zone. Individual fractures and brittle deformation zones are often discontinuous or irregular over longer distances along their strike.

The current understanding of the geometry of deformation zones within the Äspö HRL has been compiled into a 3D RVS model, Figure 3-8. The block size is a cube with the side lengths of 1,000 m. Only the structure called NE-1 (Figure 3-9), equal to the Äspö shear zone had significance for the construction works, see /Rhén et al. 1997b/.

A minor deformation zone NE-2 crosscuts the tunnel system of the HRL, Figure 3-10. The deformation zone has significant geophysical indications at the surface but undulates or splays significantly towards depth. It has not been possible to distinguish between undulations or splay geometry. The undulating character shown in the model is chosen because it is the simplest solution. The width of the zone varies between ca 0.5 and 7 m. If the interpreted intersections are correct the zone becomes narrower towards depth and north. The intersection in the tunnel at chainage 3,337 (450 m depth) is a ca 1 dm wide mylonite mapped as fracture and in tunnel section 2,861 it is a narrow ductile zone, used as a possible location. In the nearby Q-tunnel is a number of ductile shear bands, some 1–3 dm wide and dipping 40–50° towards NW observed. These minor structures may belong to a set of spays located at the termination area of the NE-2 structure.

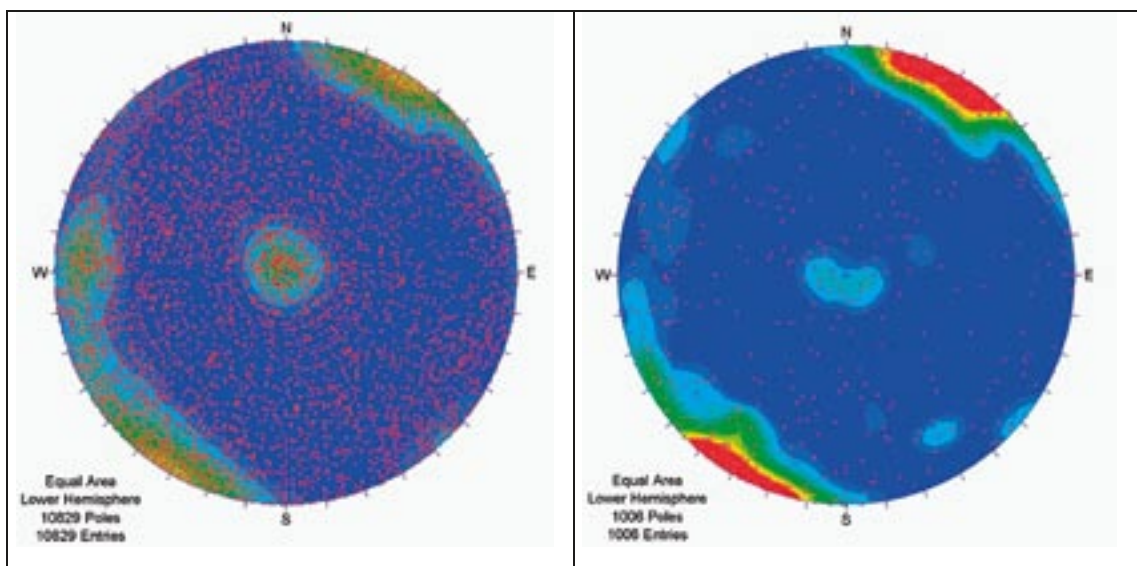


Figure 3-7. Fractures from the HRL tunnels, except for those mapped in fracture zones. All fractures to the left (maximum ca 5% per 1% area) and water bearing fractures to the right (maximum ca 9%). Contour intervals at 1% /Berglund et al. 2003/.

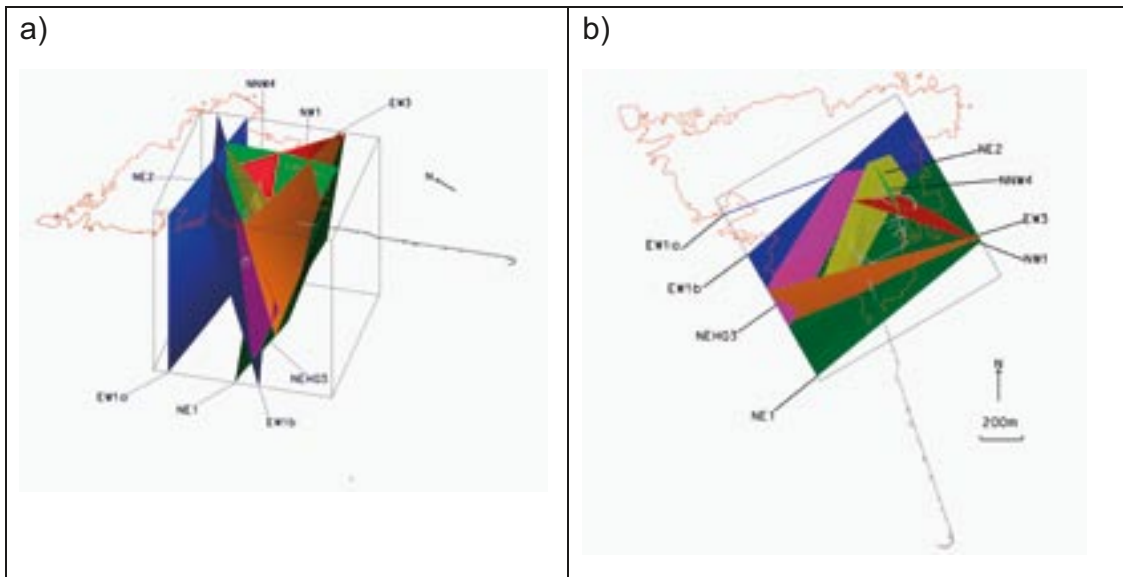


Figure 3-8. The 3D RVS model in a) isometric view and b) top view /Berglund et al. 2003/.

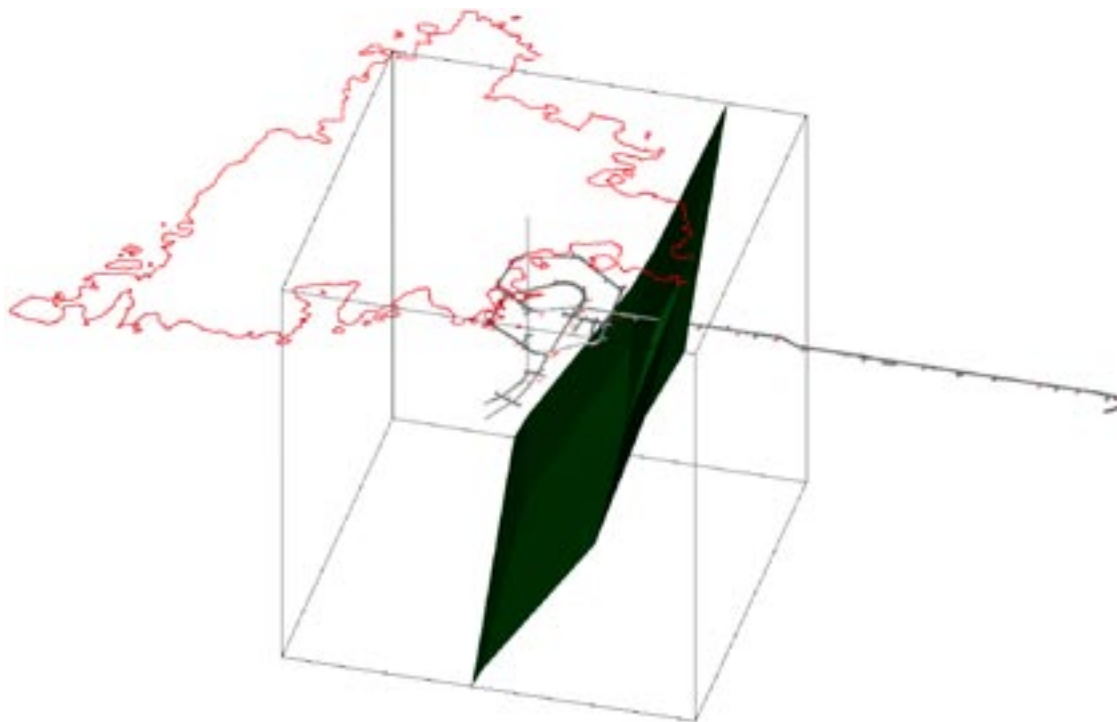


Figure 3-9. Location of the deformation zone NE-1 /Berglund et al. 2003/.

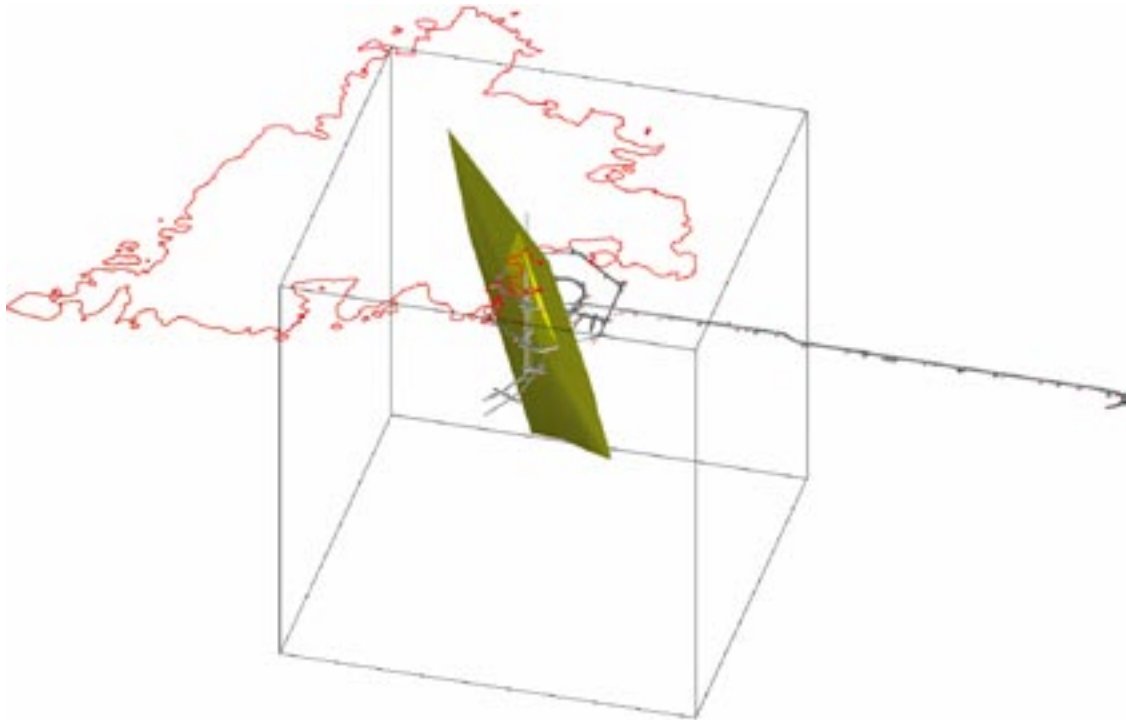


Figure 3-10. Isometric views of NE-2. Tunnel symmetry is shown together with the model domain boundary /Berglund et al. 2003/.

Based on the geometry of deformation zones presented in Figure 3-8, the 3D model of the HRL rock mass could be divided into 8 blocks, each block limited by deformation zones and the sides of the 1,000 m cube covering the model volume. Attempts have been made to identify differences in fracture statistics between the blocks; some examples are given in Figure 3-11. Possible differences in fracture statistics between the different blocks in the site model could depend on the directional orientation bias in the observed tunnel data. An alternative hypothesis may be the possibility to have true differences in fracture statistics in the different block. One reason could be the possible splays of fractures on one side of any deformation zone.

3.4.3 Mechanical properties

In connection with the work to develop a strategy for compiling a rock mechanics site descriptive model /Andersson et al. 2002/ application studies have been done with data from HRL. The result from this is a characterisation of the rock's deformation and structural strength properties and the mean value as function of the depth below ground surface as well as absolute values in different locations close to the major deformation zones, which exist outside the HRL. In a "Test Case", concluded by /Hudson 2002/ a target volume of rock, 600×180×120 m within 380 to 500 m depth was defined. The target volume was divided into 30×30×30 m large blocks. The best estimate of the large-scale deformation modulus of the rock mass was 42 GPa (range 35–45 GPa) outside the minor deformation zones. The results of modelling deformation modulus of the minor deformation zones ranged between 13 GPa to 45 GPa, depending on how the geological information on fracture frequency and occurrence of alteration products within the deformation zones were interpreted by the different modelling teams /Hudson 2002/.

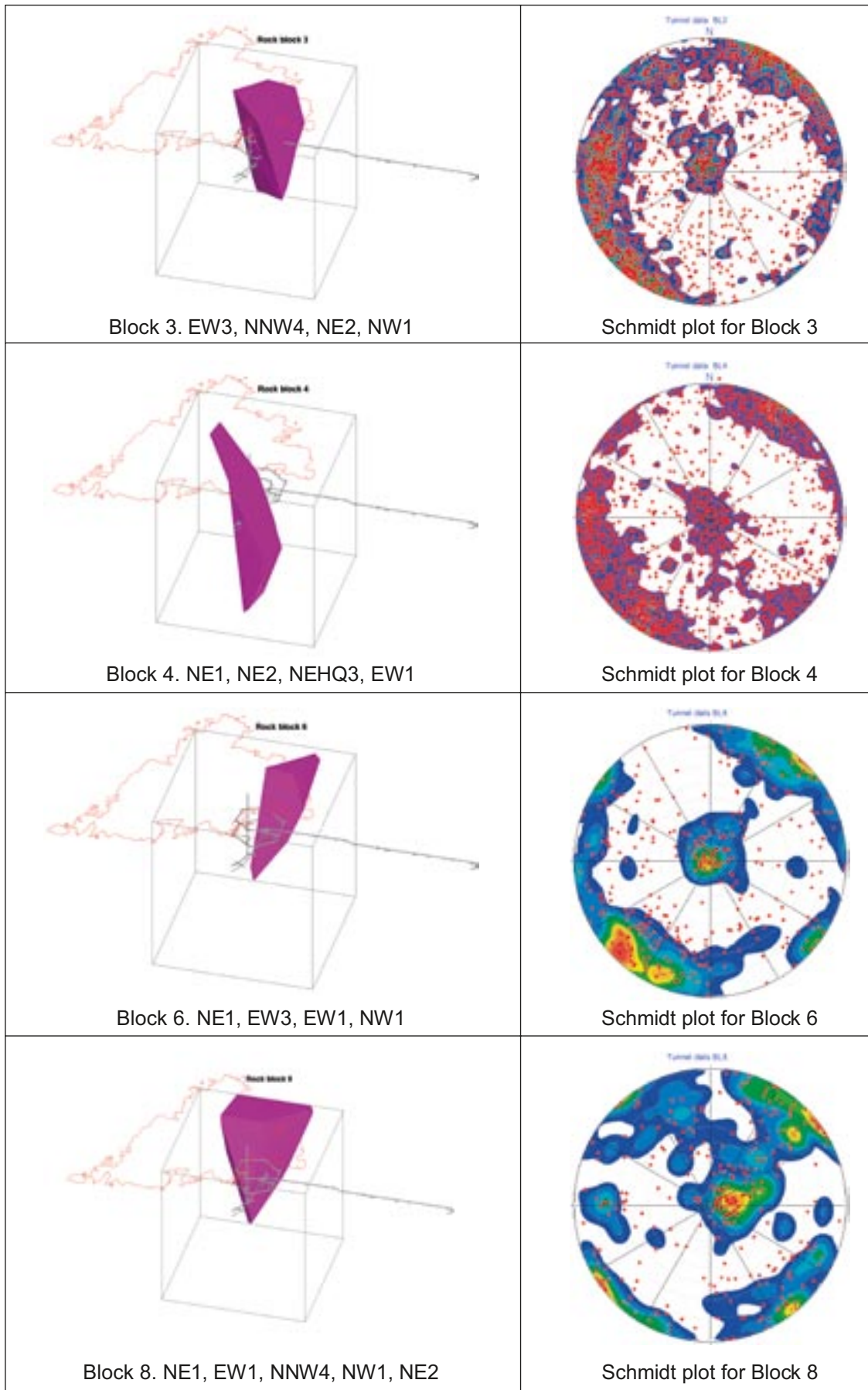


Figure 3-11. Geological blocks at the HRL divided by minor deformation zones and the related fracture orientation from each of the blocks /Berglund et al. 2003/.

The uniaxial strength of the “Äspö diorite” has been reported to be in the range of 180–210 MPa. However, local variation in the range of 140 to 300 MPa has been observed. Similar large scatter in uniaxial strength has been noticed within the ongoing site investigations. Detailed studies have identified a correlation between the mineralogical composition (see Figure 3-2) and uniaxial strength. The strength in the samples is decreasing with decreasing quartz content in the granite into the quartzmonzodiorite. But as the rock gets more mafic, the uniaxial strength increases again.

3.4.4 State of stress

Extensive stress measurements have been carried out at the HRL with both overcoring and hydraulic methods. The most complete review and evaluation of all data is found in /Ask 2004/. The most detailed measurement campaign was a test of three different stress measuring methods in two orthogonal boreholes at the 450 m level /Christiansson and Jansson 2003/.

It is found an excess of horizontal stresses already at shallow depth, as normal in the Fennoscandian shield /Martin et al. 2001/. The maximum horizontal stress is roughly equal to the maximum principal stress. At the 420 m level the maximum horizontal stress is in the range of 20–24 MPa and the minimum horizontal stress just above the vertical stress, which is equal to the weight of the overburden (11 MPa). At the 450 m level the maximum horizontal stress is in the range of 27–30 MPa and the minimum horizontal stress close to the vertical stress (12–13 MPa). The stress orientation through the site is rather consistent. All data collected show that the trend of the major principal stress lies in the range of 106–160° from the magnetic North. The orientation of the major principal stress is parallel to the most water bearing fracture set (cf Figure 3-7).

The difference in major horizontal stress from the 420 to the 450 m level is primarily believed to depend on stress relaxation in the block divided by deformation zone NE-2 (see block #8 in Figure 3-11).

3.4.5 Hydraulic conditions

The major water-bearing structure interfered with the tunnelling was the deformation zone NE-1 (Figure 3-9). The structure consist of an 8 m wide core zone of highly fractured and tectonized granite and mylonite. Within this core zone there is a 1 m wide section of clay gouge. A 15 m wide transition zone, consisting of fractured fine-grained granite and diorite, is situated on either side of the 8 m wide core zone. The dip is roughly 70° towards NW. Eight non-water-bearing fracture sets and three water-bearing sets were observed /Rhén et al. 1997b/. The transmissivity measured in 8 percussion boreholes from the tunnel through the NE-1 structure was in the range of 4×10^{-4} m²/s.

No other deformation zone in the HRL area has displayed similar hydraulic conditions. There is however observed individual fractures trending NW-SE; steeply dipping that may show extremely high transmissivity. These “high-permeable features” /Rhén and Forsmark 2000/ were observed in boreholes. They defined these features having an inflow rate in probe holes higher than 100 l/min or alternatively having transmissivities T of $> 1E-5$ m²/s.

/Munier and Hermansson 1994/ defined a special type of fracture zones, “fracture swarms”. These were zones with a relatively high frequency of parallel fractures. A number of water-conducting features can be correlated to the modelled fracture swarm intersections with the spiral in Äspö structural model. An example of distribution of water conductive features in the lower part of the spiral ramp is shown in Figure 3-12. As indicated in the figure many of the most significant water-bearing structures are oriented NW-SE.

The Äspö 97 geoscientific model /Rhén et al. 1997c/ showed a hydraulic anisotropy, based on results from all probe-hole drilling in the spiral ramp. The highest transmissivity was found in the NW-SE direction. Another conclusion by /Rhén et al. 1997c/ was that it was not possible to find any decrease in transmissivity with depth at the HRL site.

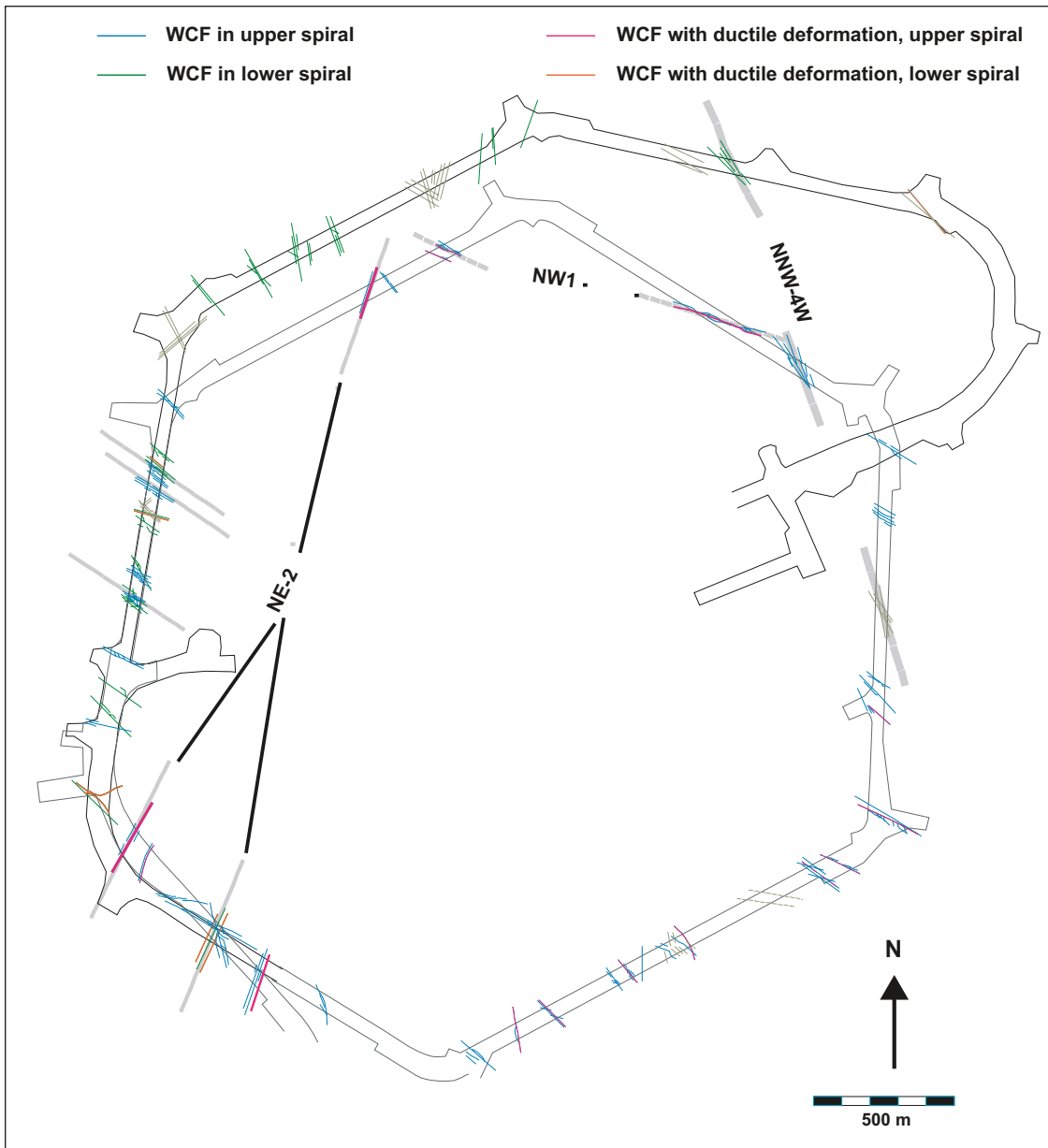


Figure 3-12. Top view of Äspö HRL showing water-conducting features (WCF) in the tunnel section 1,500 to 2,450. Fault planes with ductile deformation in wall rock adjacent to the WCF are indicated with greyish lines. Grey hatched lines denote structures tentatively possible to connect /Berglund et al. 2003/.

4 Oskarshamn power station – underground works

The Oskarshamn power station consists of three nuclear units (O1, O2 and O3), and an underground storage site for medium and low-grade radioactive waste (BFA). The facilities are owned by Oskarshamns Kraftgrupp AB (OKG), which was founded in 1965. Construction for O1 and O2 started in 1966 and 1970 respectively, and O1 was commissioned in 1972, and O2 in 1974. Construction for O3 began in 1976, and was commissioned in 1985. The extended construction period for O3 was a consequence of the general referendum in 1976, the Swedish Act for Nuclear Facilities (1977), and the referendum in 1980.

The major projects on the Simpevarp Peninsula are the power units (O1, O2 and O3 including the cooling water tunnels, BFA, Clab 1 and 2, and the access tunnel for the Äspö HRL (cf Figure 1-1).

4.1 Site investigations

Site investigations for O1, O2 and O3 for the main part consisted of seismic surveys and drilling to primarily investigate the rock conditions along the intake tunnels. As stated in /Curtis et al. 2003/, the construction phase of O1 and O2 was preceded by investigations including outcrop mapping, seismic refraction surveying and drilling. No comprehensive site investigation report was ever produced. Documentation during construction regarding the cooling water tunnels for O1 and O2 is rather insignificant when compared with current standards of required documentation for such important structures /Larsson and Leijon 1999, Curtis et al. 2003/.

The documentation from the investigations or construction of the BFA is even thinner compared to the documentation for the units O1 and O2. There exist no geological documentation from the BFA /Larsson and Leijon 1999, Curtis et al. 2003/.

Construction of O3 was preceded by an investigation phase including, outcrop mapping, geophysical surveying, and hammer drilling with permeability testing and borehole camera surveys. The reactor foundation area was further investigated by core drilling. Figure 4-1 gives an overview of the site investigations for O3.

No comprehensive site investigation report was ever produced. For the construction phase, geological documentation and rock condition descriptions showed a great improvement in scope when compared with those of the previous facilities. Geological mapping was carried out of all the tunnels and major foundations /Larsson and Leijon 1999, Curtis et al. 2003/.

4.2 Units O1 and O2

The rock construction works for O1 and O2 comprised open-cut excavations for foundations of various buildings, culverts, switchyards, roads etc. The underground works consisted of excavation for the cooling water tunnels and the caverns for the intermediate storage of low-grade radioactive waste (BFA). The total excavated rock volume for O1 and O2 was approximately 320,000 m³ of which about 50,000 m³ is from the cooling water tunnels and 80,000 m³ from the BFA.

4.2.1 BFA

BFA (cf Figure 1-1) consists of a transport tunnel from the ground surface to the storage level, a loading tunnel and set at right angles, seven short caverns for the storage of low grade radioactive waste and also, parallel with the loading tunnel a long cavern for medium radioactive waste.

Geological and rock condition documentation from the construction phase of BFA was not maintained, but obviously no problems were encountered with the rock excavation. The storage caverns were supported with one or two layers of shotcrete, although rock conditions were good. About 15 litres of water per minute are pumped from the underground facility. According to an unconfirmed statement, occasional grouting may have been performed during the construction phase /Larsson and Leijon 1999/. General data on BFA is given in Table 4-1.

4.2.2 Cooling water tunnels, O1 and O2

Each unit has its own headrace tunnel and discharge tunnel for cooling water. The cooling water flow is ca 20 m³/s and ca 27 m³/s for O1 and O2 respectively. The intakes for O1 and O2 are located on the southern shore of the Simpevarp Peninsula, and the outlets discharge in the Hamnefjärden on the north shore of the peninsula (cf Figure 1-1).

The portalling for the headrace tunnels were made on the seaside behind cofferdams. According to /Larsson and Leijon 1999/, the tunnelling work was carried out without any significant problems. Rock bolting was used in connection with tunnelling through some minor fracture zones and as spot bolting of local rock blocks. Shotcreting was applied in some minor zones of crushed rock of which the largest zone was found in the discharge tunnel. However, the zone did not cause any driving problems. As far as is known, grouting was not required in any of the two tunnels /Larsson and Leijon 1999/. General data on the cooling water tunnels for units O1 and O2 is given in Table 4-2.

Table 4-1. Tunnels and rock caverns, BFA.

	Length, m	Width, m	Height, m
Transport tunnel	180	7	7
Loading tunnel	160	9	9
Cavern for low grade radioactive waste; Nos. 7	7×50	12	9
Cavern for medium radioactive waste	160	12	12
Connection tunnels, Nos. 2	2×15	8	7

Table 4-2. Cooling water tunnels – Units O1 and O2.

	Unit O1	Unit O2
Headrace, length, m	210	210
Discharge, length, m	610	540
Tunnel area m ²	22	30
Rock excavation volume including cuttings, m ³	27,000	32,000

4.3 Unit O3

Rock excavation works for O3 consisted of the cooling water tunnels (two headrace tunnels and one discharge tunnel) and various foundations for buildings and other purposes. The rock excavation volume was approximately 730,000 m³, with about 80,000 m³ coming from the tunnels.

4.3.1 Cooling water tunnels, O3

The cooling water conveying system for unit O3 (cf Figure 1-1) comprises an intake, a headrace tunnel (two parts), a surge basin, a discharge tunnel, and an emergency coldwater tunnel. The tunnelling and rock construction experiences are briefly described in /Stanfors and Larsson 1998, Larsson and Leijon 1999, Curtis et al. 2003/.

The intake is sited about 500 m off shore where the water depth is approximately 18 m. The advantages in the form of colder water and less chance of ice accretion were judged to be worth the extra cost for the increased tunnel length and necessary intake structure.

The headrace tunnel consists of two parts: headrace tunnel 1 between the intake and Fallsviken and headrace tunnel 2 between Fallsviken and unit O3. The inner part of Fallsviken is isolated from the sea by a dam and acts as a surge basin (cf Figure 1-1).

The outlet is located in Hamnefjärden about 300 m to the east of the outlet from O2. An emergency cooling water tunnel links the unit with the headrace and the discharge tunnels.

The portalling for the two headrace tunnels were made behind the dam, and the portalling for the discharge tunnel was made behind a cofferdam at Hamnefjärden. The portalling for headrace tunnel 2 was combined with a rack cleaner building. The headrace tunnel 2 and the discharge tunnel are equipped with shut off arrangements for emptying.

General data on the cooling water tunnels for unit O3 is given in Table 4-3, and an overview of the site investigations carried out for O3 is shown in Figure 4-1.

4.3.2 Headrace tunnel 1

A vertical section of the intake for cooling water for unit O3 including number and location of core holes are shown in Figure 4-2. All blasting and rock support of the tunnel and preparation work for the deepwater intake was carried out in dry conditions from within the tunnel. The portalling was made behind a cofferdam at Fallsviken.

As seen in Figure 4-1 seismic investigations were carried out along side the sea portion of the headrace tunnel and locally at the location of the deepwater intake. In addition, probing with percussion was made at the intake to estimate the thickness of bottom sediments and rock quality, and thereby optimize the location of the intake.

Table 4-3. Cooling water tunnels – Unit O3.

	Length, m	Area, m ²	Volume, m ³
Headrace tunnel 1	670	50	35,000
Headrace tunnel 2	370	50	20,000
Discharge tunnel	220	50	13,000
Emergency cooling water tunnel	235	30	7,000
Total	1,495		75,000

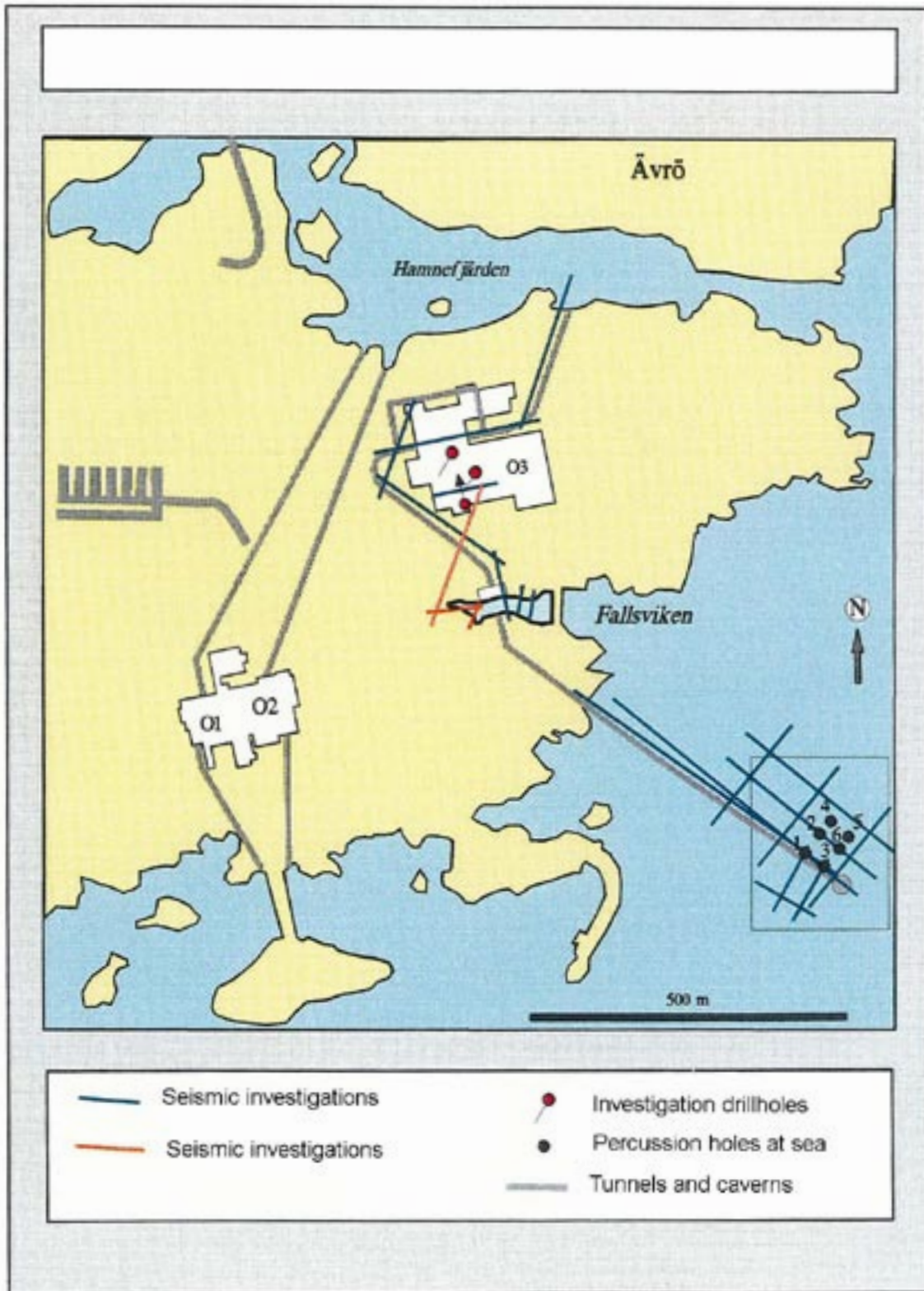


Figure 4-1. Overview of site investigations for unit O3. Modified after /Stanfors and Larsson 1998/.

Out from the coastline probing ahead of the tunnel face was continuously carried out with two 30-m long probe holes directed ahead and above the tunnel face. This continuous probing was done in order to identify potential zones of weakness, allowing grouting ahead of the face if necessary and confirming minimum rock cover available to the tunnel crown.

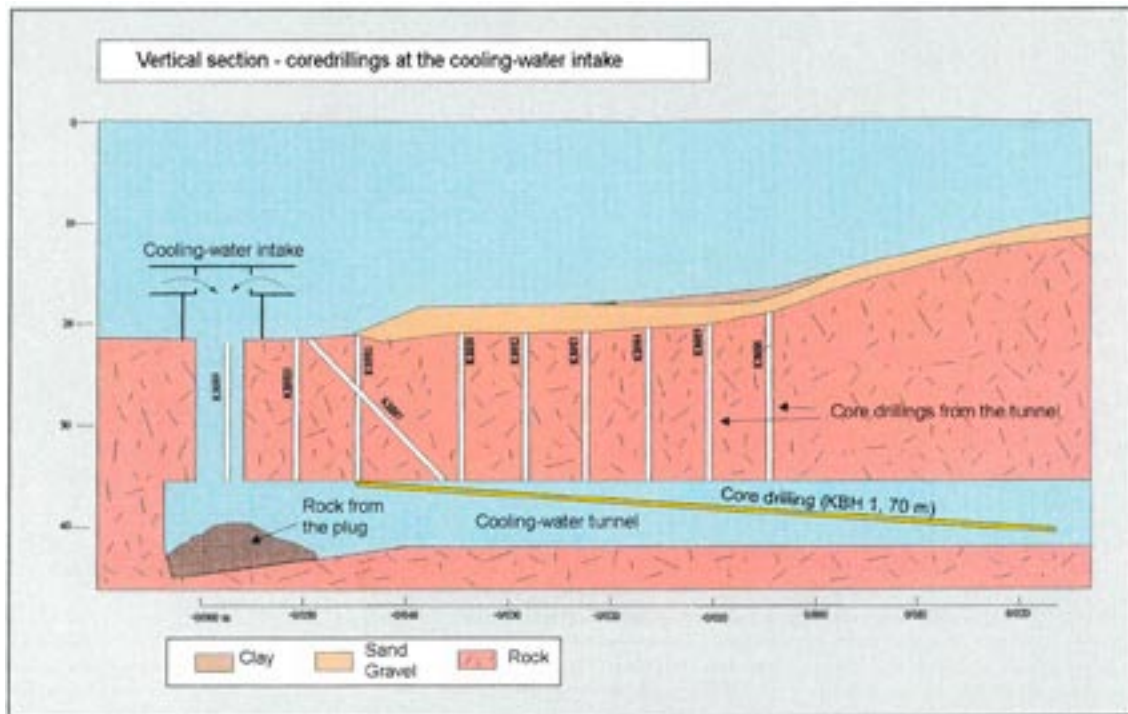


Figure 4-2. Vertical section of the cooling water intake for O3 showing location of core holes drilled from the tunnel. Modified after /Stanfors and Larsson 1988/.

Pre-grouting was carried out in three sections:

- Curtain grouting in a fracture zone about 100 m from the coast.
- Curtain grouting in an area with gouge filled fractures about 250 m from the coast.
- A series of seven grout curtains from the tunnel's last 100 m, towards the intake, as well as further two after excavation.

The aim of the third series was to ensure a tight tunnel crown due to concerns about the limited rock cover in this section (about 15 m). The probing indicated a zone of weakness occurred about 400 m off the shoreline. A 70 m long core hole was drilled from the tunnel face towards the intake. The fracture frequency was high to very high for the first 60 m with numerous, occasionally weathered, brecciated zones. However, the rock was generally fresh between such zones, and in the last 10 m the rock quality was better.

Tunnel excavation through the zones was achieved without any special difficulty due to the pre-grouting and the timely installation of temporary support in the form of shotcrete after each round. At a later stage about 20 m length of tunnel was given additional support of 150–250 mm thick layer of fibre-reinforced shotcrete. In the rest of the tunnel, including the section on land, one or two layers of shotcrete were applied to about 50% of the roof and 25% of the wall surfaces. The need for rock bolting was minimal, with a general frequency of 1 bolt per metre of tunnel.

Towards the intake location, where there was less rock cover, additional probing was carried out with some twenty of holes. Additionally, ten vertical core holes were drilled from the tunnel roof up to the rock surface in order to locate the best position for the shaft (cf Figure 4-2). The investigation led to the shaft position being moved some 20 m further from the coast. An extensive grouting and rock support programme was carried out to ensure the stability of the shaft. This included over 200 grouting holes and 50 tons of cement compared with a total figure of 85 tons for the entire tunnel grouting works. It should be noted that the majority of the grout injected around the shaft location was probably lost through superficial fractures to the overlying sediment.

4.3.3 Headrace tunnel 2

The zone of weakness at Fallsviken influenced the rock mass at the portalling of the tunnel. Therefore, a cast-concrete arch was applied at the mouth of the tunnel and the tunnel was supported with reinforced shotcrete along an approximately 20-m section from the portal and into the tunnel. In addition, the last part of the tunnel connecting to the turbine building was supported with reinforced shotcrete along a section of about 25 m, primarily due to minor rock cover and the close vicinity to buildings.

In total, around 80% of the roof and wall surfaces were supported with shotcrete. Rock bolting was only used at the portals. The water inflow to the tunnel was practically equal to nil; only one single inflow of less than 2 l/min was observed.

4.3.4 Emergency cooling water tunnel

The tunnel was excavated in good rock. Around 60% of the roof and 30% of the walls were supported with one layer of shotcrete, and in total, about 30 rock bolts were installed; i.e. 0.15 bolts/m tunnel. The tunnel was completely dry during the excavation.

4.3.5 Discharge tunnel

A steeply dipping chlorite-clay gouge filled fracture zone, with a width of up to 500 mm, runs along the tunnel roof from the turbine building right up to a change in the tunnel alignment. The zone has an east-west strike and cuts both the foundation of the turbine building and the intake of the headrace tunnel 2. Consequently, it has a length of more than 300 m but was obviously too narrow to be identified by the seismic survey work. Along this section the tunnel roof was supported with fibre-reinforced shotcrete partly due to the presence of the fracture zone and partly due to the reduced rock cover over the tunnel crown and proximity to the buildings.

Around 70% of the roof and 50% of the walls were reinforced with shotcrete, mainly at the portals and in connection to unit O3. No documentation on rock bolting, nor water inflow occurrences has been found. Instalments of several drains in the shotcrete indicate that some water inflows may have occurred at the outlet of the discharge tunnel.

4.4 Summary of underground works

The underground rock excavation works involve about 215,000 m³, and a total tunnel length of approximately 4,000 m. The undersea tunnelling represents a rock excavation volume of ca 35,000 m³ and a tunnel length of 670 m.

There are no documentations available, which reports any particular difficulties due to rock conditions encountered during the tunnelling work. Rock support consisted of the installation of rock bolts when certain minor fracture zones were encountered and also occasional spot bolting for isolated blocks. Minor closely jointed or brecciated sections had their surfaces stabilized by shotcrete. The tunnel excavation encountered no significant inflows of water and only minor grouting work was carried out for either O1 or O2.

As for BFA, available documentation reports no rock engineering problems with the rock excavation. Although rock conditions were evidently good, one or two layers of shotcrete supported the storage caverns. About 15 litres of water per minute are pumped from the underground facility, which indicate very dry rock conditions.

The most demanding undertaking, from a rock-engineering point of view, was perhaps the excavation work for the cooling water intake of O3. The work comprised rock excavation underneath a 5 to 6 m thick rock plug and a water head of 20 m. However, the excavation work was achieved without any special difficulty due to careful planning of remedial measures and actual performance of the work.

5 Clab – underground works

5.1 Introduction

The Clab facility, the Central Interim Storage Facility for Spent Nuclear Fuel, is located about 800 m to the west of the Oskarshamn nuclear power units O1 and O2. The facility is owned by SKB AB whilst the operation was contracted to OKG AB up to 2006 after which SKB took over the operation. The facility for intermediate storage of spent nuclear fuel is a central storage serving all nuclear power plants in Sweden. The fuel elements are stored in water filled concrete basins, located in rock caverns with a rock cover of approximately 30 m.

The construction works for Clab 1 started in autumn 1980 and was completed in February 1982. The facility was taken into operation in 1985.

In 1996, the Board of SKB decided to commence a planning and design study for an extension of the Clab 1 facility with an additional rock cavern, Clab 2, similar to the present one, in order to increase the storage capacity.

In March 1998, SKI approved, according to the Swedish Act for Nuclear Facilities, SKB's application to extend the storage capacity of Clab and to build, own and operate a new rock cavern, Clab 2 enabling the storage capacity to be increased from the present 5,000 tons to 8,000 tons of spent nuclear fuel. SKI's approval was followed by a Government approval as well as Water Right Court approval of the said extension of storage capacity. Construction of Clab 2 began in January 1999 and the underground excavations were completed in late 2000. Clab 2 was taken into operation in 2005.

5.1.1 Clab 1

Clab 1 comprises a surface facility for reception of spent nuclear fuel and an underground part with storage basins (Figure 5-1). The surface facility contains equipment for ventilation, water-cleaning and cooling etc, and office space for administration staff and operational personnel. In total about 80,000 m³ of rock was excavated for the foundation of the surface building with a foundation depth of approximately 20 m.

The storage basins are constructed by reinforced, casted concrete with a lining of stainless steel, and the basins are placed in a rock cavern. The rock cavern is connected to the ground surface by a transport tunnel, a combined elevator and ventilation shaft plus a shaft for the spent nuclear fuel.

The length of the transport tunnel is about 500 m and has an area of 35 m². The elevator and ventilation shaft is square shaped with an area of 120 m² and a net height in rock of about 15 m. The total height between bottom and original rock surface is approximately 44 m. The elevator for spent fuel is circular with a gross diameter of 3.5 m and an inner diameter of 2.0 m.

In addition to the transport routes and the rock cavern containing the storage basins there is a transept, with the main purpose to connect the elevator shafts with the rock cavern. In the design and construction of Clab 1, preparations were made for an additional rock cavern by excavating a 20 m blind prolongation of the transept.

The rock cavern has a length of 117 m, a width of 21 m and a height of about 27 m. The length of the transept is 58 m, the width varies between 11 and 13 m and a height varying between 12 and 14 m.

The rock excavation volume of the rock cavern and the transept is approximately 80,000 m³, and in total, the underground excavation volume is about 100,000 m³.



Figure 5-1. Layout of the Clab 1 and 2 showing the surface facilities and underground structures.

5.1.2 Clab 2

The additional rock cavern was constructed parallel to the existing one, at a distance of 40 m. The dimension of the new cavern is approximately the same as for the Clab 1 cavern; i.e. a length of 117 m, a width of 21 m and a height of about 27 m. Due to the sensitivity of the facility in operation, the excavation works were surrounded by rigorous restrictions on groundwater, deformations, blasting, grouting, rock support etc. In total, about 87,500 m³ of rock was excavated for Clab 2.

Careful calculations were carried out, using various modelling tools, in order to show that necessary stability could be maintained for the existing cavern and reached for the new one. The rock parameters used for the calculations were determined based on the result from geological pre-investigations, as well as experiences from the existing cavern /Stille and Olsson 1996, Stille et al. 1997, Stanfors et al. 1997c/. The governmental regulators examined the result of the studies before permission to start the construction was granted. In addition, an extensive monitoring programme for deformations was developed, comprising the use of extensometers and optical convergence measurements /Fredriksson et al. 2001/.

5.2 Site investigations

5.2.1 Clab 1

Preliminary site investigations for Clab 1 were carried out in 1978 and 1979 with the siting exercise for Clab /Moberg 1978, 1979/. The investigations comprised seismic refraction, outcrop mapping, eleven core-drilled boreholes including mapping and water injection tests, in situ rock stress measurements and soil and rock probing. The investigations confirmed the area's suitability and formed a basis for the preliminary design work.

During the construction phase the following site investigation activities were carried out /Larsson and Leijon 1999/.

- Drilling investigations ahead of the advancing face.
- Engineering-geological follow up.
- Hydrogeological measurements and assessments from both the excavations and surrounding area.
- Deformation measurements during construction.
- Drilling investigations in the excavation invert to assess foundation conditions for the storage basins. A total of 12 core-holes, 10 m deep, were drilled from the bottom of the rock cavern of which 11 were vertical holes.
- Documentation of rock support and grouting works.

The majority of the above listed documentation from the construction phase has been compiled by /Eriksson 1982/.

An example of the geological follow up of the transport tunnel, Clab 1, is shown in Figure 5-2, and Figure 5-3 demonstrates the documentation of rock support in the transport tunnel and pre-grouting works in the transept and rock cavern, Clab 1.

5.2.2 Clab 2

The main site investigation for the extension of the Clab facility was carried out during 1995 with additional investigations in 1997. The results of the complete investigations are described in detail in /Stanfors et al. 1997c/. The purpose of the 1995 investigation campaign was to

- Confirm that the available rock plinth to the west of the existing rock cavern had a sufficient volume and sufficient rock quality for the siting of the new rock cavern.
- Characterise orientation and quality of deformation zones surrounding the new rock cavern in order to optimise the use of the rock plinth.
- Obtain basis for estimating rock support and grouting in tunnels and cavern.
- Obtain basis for estimating the local groundwater influence.

Figure 5-4 illustrates the initial position at the start of the 1995 site investigations.

The investigations carried out in 1995, together with the experiences gained during the construction of Clab 1 clearly demonstrated that the rock mass conditions were feasible for an extension of the facility. Some additional information was judged to be of importance for the confirmation and detailing of the geological model elaborated in 1995, and furthermore to provide basis for the detail design of the underground structures for Clab 2. Therefore, supplementary investigations were undertaken in 1997 /Stanfors et al. 1997c/.

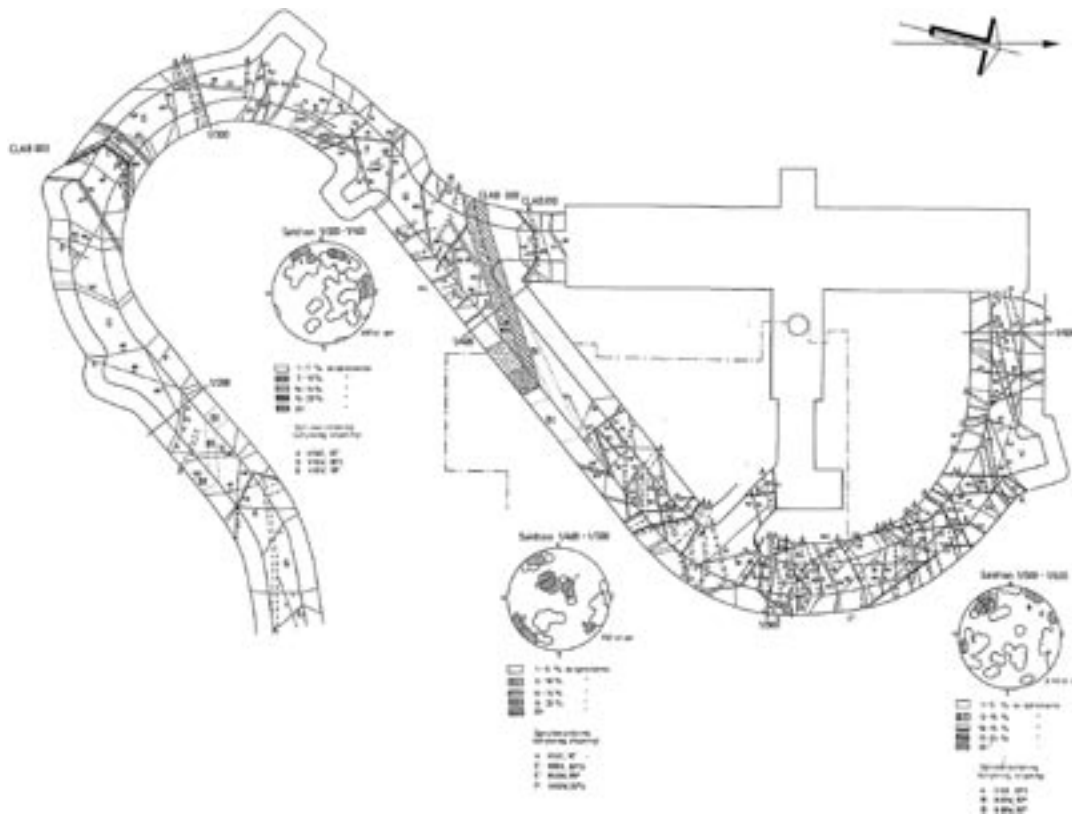


Figure 5-2. Geological follow up of the transport tunnel, Clab 1 /Eriksson 1982/.

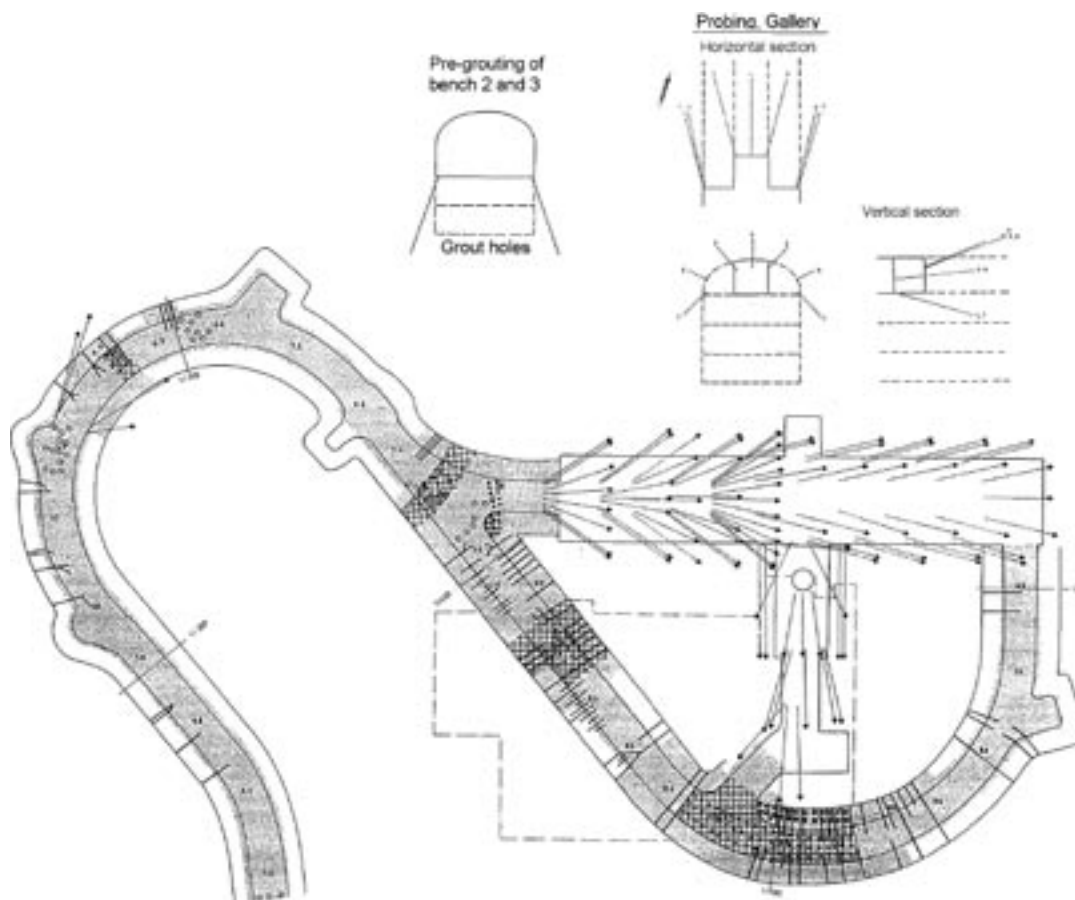


Figure 5-3. Documentation of rock support in the transport tunnel and pre-grouting works in the transept and rock cavern, Clab 1. Modified after /Eriksson 1982/.

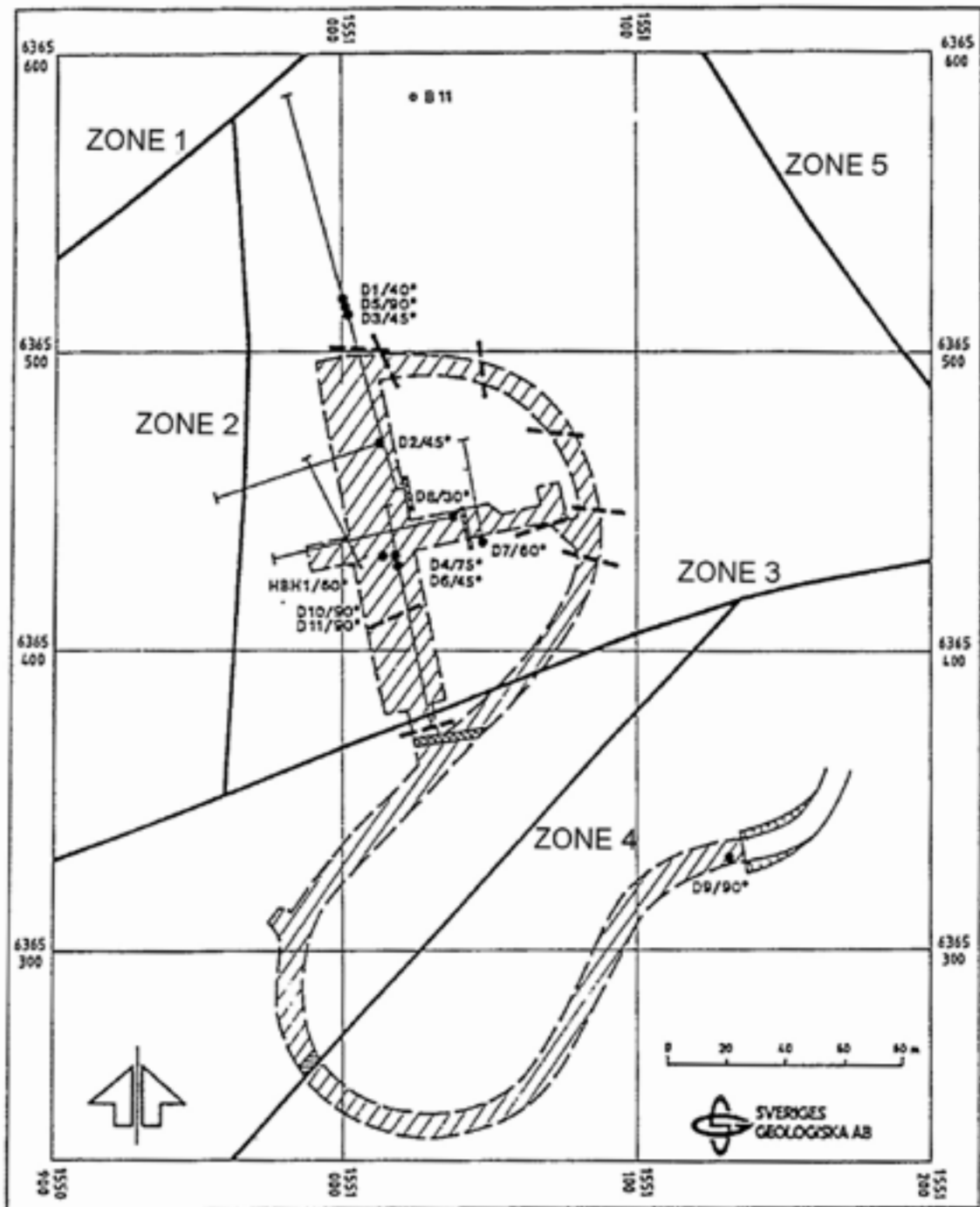


Figure 5-4. Initial position at the start of the 1995 site investigations /Stanfors et al. 1997c/.

The 1995 investigation campaign comprised geological reconnaissance of an approximately 1 km² large area connecting to the Clab facility. The reconnaissance included mapping of rock types, fractures and characterisation of other geological structures with focus on constructability. The investigations contained seven refraction seismic profiles (in total about 2,000 m), about 310 m core drilling (three boreholes) and 266 m percussion drilling (five boreholes). Water injection tests with double packers were performed in the core holes and test pumping in the percussion holes (the entire hole length).

The 1997 investigations constituted one 300 m long refraction seismic profile, percussion drilling to determine the bedrock level above the planned cavern and 144 m core drilling (two boreholes) in which core logging and water injection tests were carried out. In addition, in situ

rock stress measurements by overcoring were made in one of the core holes (see Table 3-3 in Section 3.5.4). A layout of the drillings made 1995 and 1997 is shown in Figure 5-5.

In situ rock stress measurements were also carried out for the new rock cavern (cf Table 3-3) in a vertical borehole in the centre of the prospective cavern at the crossing between cavern and transept. The measurement indicated somewhat lower stresses in comparison with Clab 1 cavern. The rock cover, 20 m, was judged to be sufficient for the new cavern.

In summary, the result of the site investigations demonstrated that the rock mass involving Clab 2 rock cavern was suitable for siting of the cavern as well as for a potential third rock cavern. The investigations indicated that there would be some local increase in rock support and grouting in the access tunnel when intersecting the southern fracture zone and possibly when the tunnel reach contact with the northern fracture zone. Otherwise, it was judged that the rock support and grouting works would be similar to the works in the Clab 1 cavern.

Based on the result of the site investigations and performed studies, SKB made the following standpoint:

- The rock mass within the chosen area was of acceptable quality for constructing a stable rock cavern of the same size as for Clab 1.
- A distance of 40 m between the parallel running rock caverns was adequate for avoiding negative rock mechanic influence from the new cavern on the existing installations.
- Conventional blasting could be used for the excavation work.

The underground excavations was continuously followed up by an engineering geological mapping /Berglund 2001/. An overview of the mapping of cavern and tunnels for Clab 2 is shown in Figure 3-5.

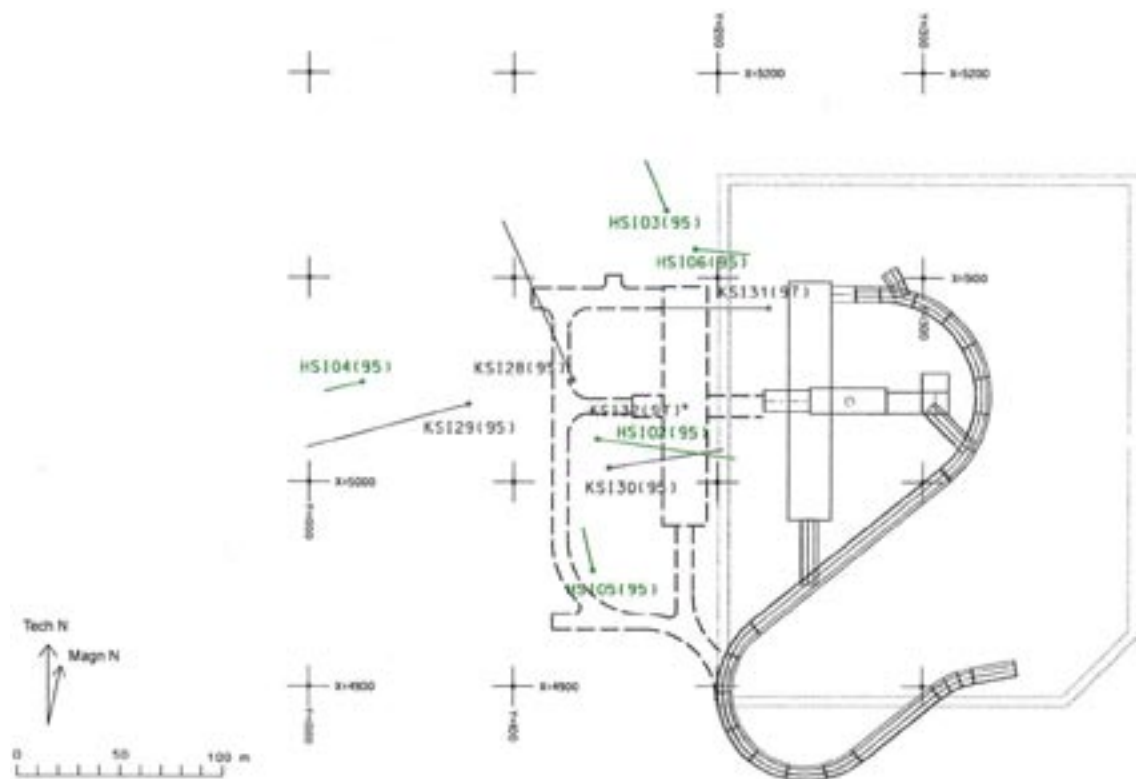


Figure 5-5. Layout of the drillings performed during the site investigation campaign in 1995 and 1997. Green colour represents percussion drilling and blue colour denotes core drillings /Stanfors et al. 1997c/.

After the completion of the underground works, the project organization concluded that the standard and scope of the site investigations were adequate and that further investigations such as additional boreholes would not have led to an improved basis for modelling, planning and construction /Bodén 2002/.

5.2.3 Rock mechanical calculations and deformation measurements, Clab 1

Rock mechanical calculations were carried out for the rock cavern of Clab 1. The program package BEFEM was used for the numerical analysis. The rock mass was assumed un-fractured and isotropic and behave linear-elastic. The E-modulus was assumed to be 30 GPa (1/3 of the rock material). The horizontal initial stress was varied between 4 and 8 MPa.

Calculated stresses and deformations around the rock cavern were almost proportional to the initial stress. At 8 MPa and completely excavated cavern, the maximum compressive stress became 25 MPa in the crown, and the maximum deformation towards the cavern was 8 mm. The calculations gave an up-lift of the roof of 3 mm.

Deformation measurements in the rock cavern were carried out in two sections. Two different instruments were employed for the measurements: Sliding Micrometer and Distometer. The Sliding Micrometer was used for measurements at each metre in 22–25 m long boreholes of which eight holes were placed in the walls, and two vertical holes were drilled from the ground surface towards the roof (cf Figures 5-6 and 5-7). In connection with the start of the construction of Clab 2, the measurements were made by extensometers. Using the Distometer, the deformations were measured across the rock cavern (convergence) between grouted dowel rods.

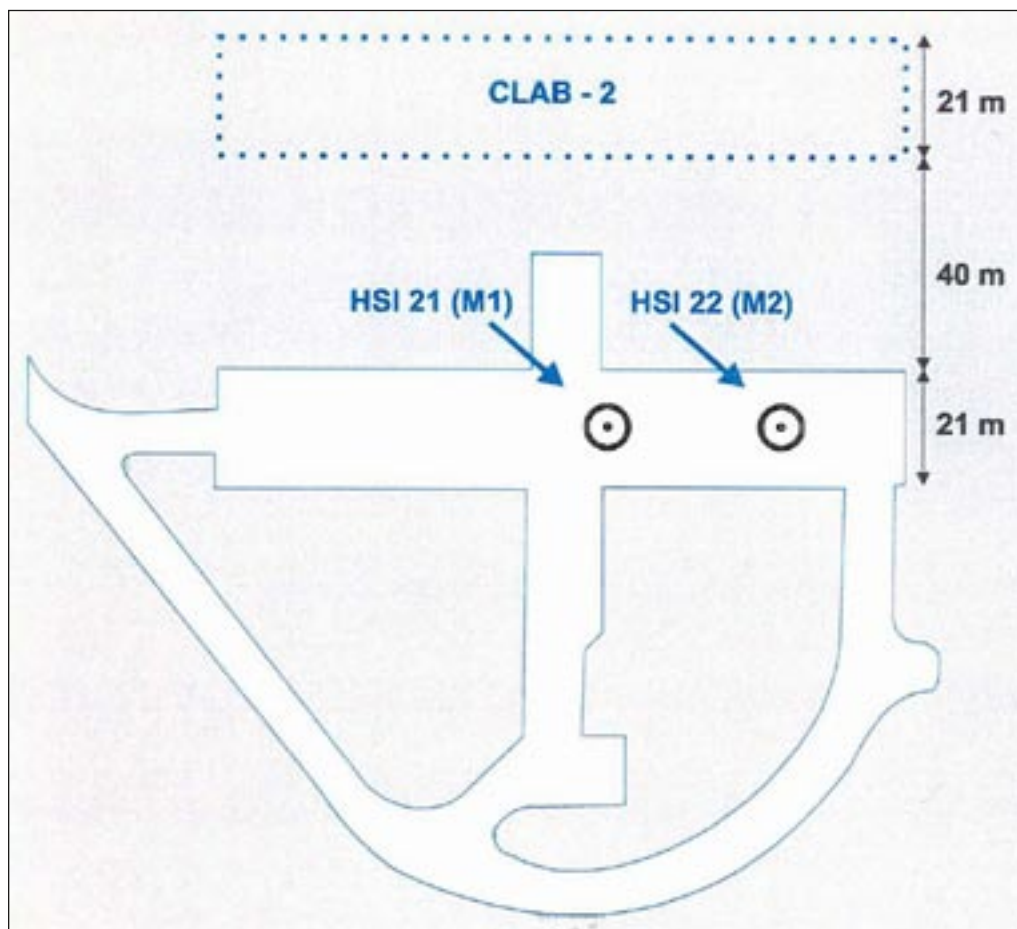


Figure 5-6. Horizontal section of the Clab 1 underground facility showing the location of the measuring holes drilled from the ground surface /Bodén 2002/.

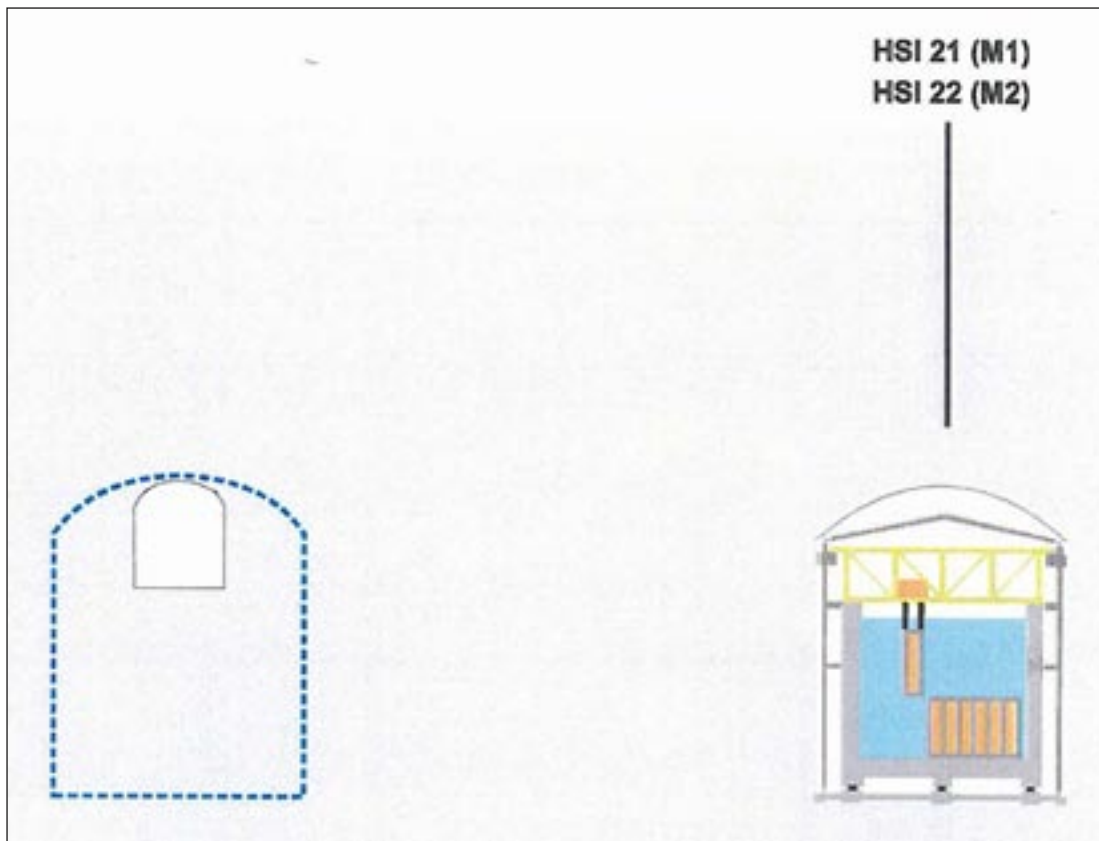


Figure 5-7. Vertical section of Clab 1 rock cavern showing the two measuring holes placed in the rock volume above the rock cavern /Bodén 2002/.

Triangulation with the Distometer showed that the roof had an up-lift of 2 mm during the excavation, whilst the Sliding Micrometer demonstrated that the measuring holes were shortened with only 0.2 mm. An inward movement of 1.5–3.5 mm in the walls was measured with the Sliding Micrometer and the Distometer gave a convergence between 2 and 7 mm for the different measuring sections. Considering that a part of the deformation in the walls occurred before the initial measuring occasion, it is fair to say that the correspondence between calculated and measured deformations was good. The deformations stopped immediately when the excavation was finalized at the turn of the year 1981–1982, which is a confident indication that the rock mass is stable.

The measurements of the deformations continued after the completion of the rock cavern. The measuring intervals were decreased in number after 1986 to a two-year measuring interval. Gradually, inwards movements were observed, which initially was believed to indicate instability. However, a rough estimate demonstrated that the movements were due to a slow heating caused by the operation and consequently expansion of the rock mass surrounding the cavern. This conclusion was also confirmed by numerical analysis. The measurements as well as the analysis demonstrated that the movement was in the range of 0.5–1.0 mm up to 1990, and, from a theoretical calculation point of view, 1.0–2.0 mm up to year 2030 /Larsson and Leijon 1999/.

The result from the measurements in the two boreholes HSI 21/M1 and HSI 22/M2 within the time period 1981–2000 is shown in Figure 5-8.

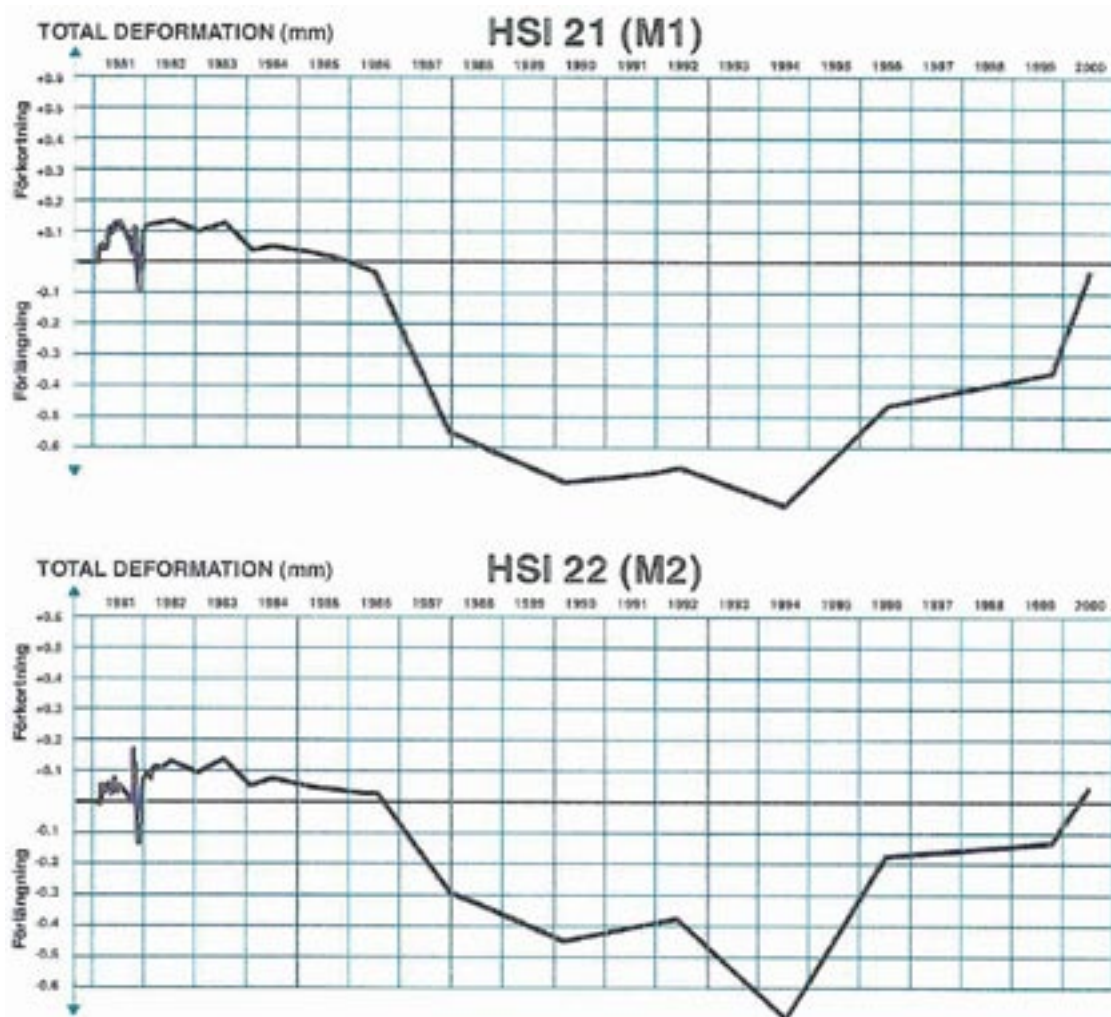


Figure 5-8. Compilation of the measuring result in boreholes HSI 21/M1 and HSI 22/M2 from 1981 up to June 2000 /Bodén 2002/.

5.2.4 Rock mechanical calculations, deformation measurements and vibration monitoring, Clab 2

The information and knowledge of the geology obtained from the pre-investigations and during the construction of Clab 1 were used for the planning of the extension of Clab 2 together with additional site investigations. By using this detailed information, all significant discontinuities in the rock mass could be identified well in advance before the construction, and no unforeseen rock engineering difficulties or problems occurred during the underground works.

A rock mechanic analysis of stability conditions and potential influence on the existing rock cavern was carried out /Stille and Fredriksson 1996/, which concluded a pillar width of 40 m, and indicating that a further increase of the width would not tangibly decrease the influence on the existing rock cavern at the construction of Clab 2 rock cavern. Furthermore, the analyses showed that there would be no risk for large-scale instability.

The influence of blast induced dynamic loads on the existing rock cavern was analyzed /Stille et al. 1997/. The result of the study showed that the blasting work for Clab 2 could be executed by applying today's blasting technology and normal restrictions without jeopardize the stability and function of the existing cavern. It was proposed to carry out the rock excavation in steps with top heading and three horizontal benches, and an interacted quantity of explosives of

maximum 3–4 kg cart ridged blasting agent. In addition, the study recommended that blasting schemes and excavation steps should be adjusted successively to experiences gained during the excavation.

A measuring programme on limit values for vibrations was established /Johansson 1998/. The programme was based on a separate analysis of blast induced dynamic loads on the existing rock cavern /Stille et al. 1997/. The limit values were stipulated for the existing storage cavern, the concrete structures in the cavern, auxiliary system, transformer building and equipment and machinery sensitive to vibrations. The permissible vibration limits were chosen with a higher factor of safety than normally applied for equivalent plants and buildings due to the strict safety demands on the operation of Clab 1, and thus to minimize disturbances on the running operation of the existing plant. The results of the vibration measurements were accessible immediately after blasting allowing the contractor to promptly adjust the drilling and distribution of charge. A number of exceeding vibration limits resulted in changes in excavation sequences and specifications on cooperating charge during the progress of the work /Bodén 2002/.

An extensive monitoring programme for deformations was developed /Stille and Fredriksson 1998/ and the monitoring in and around the two caverns were carried out continuously by applying the following methods:

- Sliding micrometer.
- Precision levelling of the ground surface.
- Extensometer measurements.
- Convergence measurements (initially by the use of Distometer and later by optical convergence measurements).

Three-dimensional models were used for the verification of the predicted influence of the new cavern on the old one. During the whole construction phase the agreement between predicted and observed deformations were considered satisfactory for the verification of the design /Fredriksson et al. 2001/.

5.3 Summary of underground works, Clab 1

The access tunnel connects to the rock cavern at three locations at three levels; roof level at the southern face, intermediate level at the elevator shaft (the transept) and roof level at the northern face (cf Figure 5-1).

The rock cavern was excavated in stages: top heading followed by two side-slashes to full width and three horizontal benching. The excavation of the top heading of the rock cavern and the transept started at the southern face of the cavern. The upper bench was excavated via the transept and the two lower benches via the northern tunnel connection. The height of the third bench at the northern face was only about 3 m wherefore just a short ramp was needed to reach the level of the second bench. Horizontal benching has the advantage of being cautious to the remaining rock, which was of significance with respect to the foundation of the storage basins.

Dripping and running water was encountered at about twenty locations along the access tunnel. However, pre-grouting was carried out at one location only, approximately 100 m to the south of the cavern's southern face and post-grouting was carried out in the zone immediately to the south of the same face.

Continuous pre-grouting was carried out in the cavern and transept walls and roof while the floor was kept un-grouted in order to avoid pressure build-up and to gain a certain water pressure release. In spite of the pre-grouting campaigns, about 50 drains were installed behind the shotcrete, and when the excavation of the cavern was completed, about 300 funnels were placed

in the roof to collect water inflows. The total water inflow to the finished tunnel and rock cavern is still relatively low. At commissioning, 1985, the total inflow was measured as 60 l/min, which after 12 years operation, had reduced to barely 40 l/min. This inflow corresponds to 5–6 l/min and 100 m tunnel.

The access tunnel's roof and 40% of the walls were supported with 50 mm un-reinforced shotcrete, partly due to safety reasons and partly to reduce the need for future maintenance scaling works. Pattern bolting was carried out in the roof at deformation zones and also occasional spot bolting. In total, 200 bolts were installed in a 500 m length of tunnel.

A high factor of safety was set for the cavern rock support. The cavern roof is supported by 100 mm reinforced shotcrete and systematic bolting consisting of 26 mm threaded bolts with end plates and four bolts per square metre. The walls are supported with 50 mm un-reinforced shotcrete. Reinforced shotcrete arches were constructed in each corner at the junction between the cavern and the transept. In addition, reinforced shotcrete and pattern bolting were used in the roof between the arches.

The support philosophy was principally to apply spot bolting in the walls and to use pattern bolting only for larger weakness zones. The predominant fracture orientation intersected at right angles to the cavern, but there was also a persistent secondary fracture orientation perpendicular to the predominant fracture orientation. These two dominant fracture sets ran almost parallel with the cavern walls, which led to that the spot bolting became as extensive as the pattern bolting. Due to the height of the walls (up to 23 m), 6 m long bolts were used. Where unfavourable fracture orientations were encountered 8 m bolts were installed. In total about 2,500 bolts were installed in the cavern and transept.

5.4 Summary of underground works, Clab 2

5.4.1 Rock engineering design

The conceptual rock engineering design of Clab 2 was based on analyses and reports, mainly as presented in /Stille and Fredriksson 1996, Stille et al. 1997, Stanfors et al. 1997c/. These reports comprised rock mechanical analyses on rock mass stability and influence on existing rock cavern, blast induced dynamic load on existing cavern and the result and interpretation of the site investigations for Clab 2.

The design prerequisites involved requirements of standards such as technical standard, environmental standards, geotechnical class etc, supervision programme, bearing strength, stability and durability in the form of loading, classification of the rock mass, material and calculations.

Specifications for the typical rock support design were given on material of support elements and proposed typical rock support of the various underground structures. This constituted the basis for the typical support drawings. The rock support design was based on the Q-system.

The contract documents with adherent instructions on rock support and grouting works were based on the rock engineer designer's knowledge at the time of the release of the tender documents. This means that the knowledge was mainly based on site investigation results and the feedback of experience from Clab 1. To the extent that actual conditions during the excavation works for Clab 2 would differ from the assumed conditions an adjustment of the rock support and grouting was applied during the course of the work; i.e. the observational method was practiced during the excavation and support work /Bodén 2002/. Examples of such adjustments were:

- Rock support of the identified shear zone located in the centre of the rock cavern. The change in orientation and splay of the shear zone were not predicted wherefore adjustment of rock support was made.

- The performed testing of the shotcrete showed low adhesion in areas of the metavolcanite probably due to numerous fine fractures in the rock mass causing rupture in the rock itself; i.e. behind the bond between rock and shotcrete. As a consequence, all the bolts installed in the metavolcanite were equipped with top plates.
- The grouting methodology was significantly modified since the rock mass turned out to be considerably tighter than predicted.

5.4.2 Rock excavation

In total, about 87,500 m³ of rock was excavated. The underground works were carried out with high and strict demands on cautious and careful blasting, collaring, hole deviation, and angular deviation and damage zone. Stipulated restrictions on allowable influence on existing facility and its surrounding entailed among other things reduced advance. A number of trial blasting were carried out in existing core holes and in the vicinity of Clab 2 in order to increase the knowledge of the rock mass properties from a blasting point of view /Bodén 2002/.

Drilling pattern, distribution of charge and ignition pattern were established. Inspections of vibration levels were made after each round, and the result of the inspections constituted in possible adjusting of next round, if needed.

Using a drilling rig Atlas Rocket Boomer 353 S equipped with a computerized and recording directional system made the holes for the rounds and rock dowels. The diameter of the drill holes was ϕ 48–52 mm.

The charging was normally carried out from the working platform of the drilling rig, and only cart ridged blasting agent was used in order to be able to charge the exact prescribed quantity of explosives. The type of blasting agents for the cut and slashing was Dynamex ϕ 25–32 mm and for the contour Detonex 80 g/m. Nonel 0–60 ms was used for firing the rounds.

The transport tunnel was driven with full face (34,5 m²). The specific drilling and charging for the transport tunnel was 3.1 m/m³ and 1.6 kg/m³ respectively.

The gallery of the rock cavern was excavated with one pilot drift and two side-slashes. Specific drilling and charging for the pilot drift was 2.5 m/m³ and 1.36 kg/m³ respectively and for the side-slashes 1.2 m/m³ and 0.7 kg/m³ respectively. The benches of the rock cavern were excavated with horizontal benching. It was originally planned to use three benches and half width of the cavern but due to restrictions in vibration levels, the excavation was changed to four benches and half width of the cavern. Specific drilling and charging for the benches was 0.87 m/m³ and 0.5 kg/m³ respectively /Bodén 2002/.

The transept between the two storage caverns had been excavated to its half-length at the construction of Clab 1; i.e. 20 m. The method of excavating the remaining 20 metres from the new storage cavern was based on the principle of blasting bench by bench and leaving a remaining 4 metre thick pillar to protect the existing storage cavern including installations until the entire excavation of the new storage cavern was completed. The rock pillar was supported with rock bolts and shotcrete and blast holes were pre-drilled successively during the benching of the channel tunnel. The final excavation of the pillar was made in rows from underneath with Detonex 80 g/m and deformation measurements by extensometers were performed /Bodén 2002/.

Loading and rock haulage were carried out in two operations and the loading started after 20–30 minutes airing of blasting fumes. The rock was first transported to an intermediate surface deposit close to the project site after which the rock material was transported to a final deposit area.

5.4.3 Rock mass deformation modulus

Extensometers were installed on three levels in the Clab 2 cavern (cf Figure 5-9). The measuring values were recorded continuously enabling comparison with established check limits. Measured deformations in the north part of the cavern were larger in comparison to the south part, and correspond better with the calculations with an E-modulus equal to 10–20 GPa. As for the north part, a good comparison is obtained with $E = 40$ GPa. The results are shown in Figure 5-10 /Fredriksson et al. 2001/.

5.4.4 Rock support

The rock support for the storage cavern and the transept comprised pattern bolting and shotcreting. Steel fibre reinforced shotcrete with a thickness of 100 and 50 mm was used in the roof and walls respectively. After shotcreting, drilling and installation of the bolts were carried out and finally a 20 mm un-reinforced shotcrete layer was applied over the entire roof and wall surfaces. The crossing between the storage cavern 1 and the transept was supported with an intersecting vault of shotcrete arches.

The type of rock bolts used was Ks 500St ϕ 25 mm and the length varied between 4.0 and 6.4 m. The bolts were installed at 1.5 and 2.0 m centres depending on local variations in the layout such as crossings. In total, 2,500 rock bolts were installed. The boreholes were drilled with the tunnel rig, and the instalment was made manually from a working platform. The bolts were equipped with top plates in areas where adhesion between rock and shotcrete in the roof and walls was judged to be poor (cf Section 5.4.1). The cement mortar used had a cement water ration of 0.3 and was placed according to the SN-method /Bodén 2002/.

The rock support works in the transport tunnel comprised mainly spot bolting with a bolt length of 4 m and 50 mm un-reinforced shotcrete in the roof and 30 mm in the walls down to one metre above the invert. Pattern bolting and 50 mm fibre reinforced shotcrete and also 30 mm un-reinforced shotcrete were used in cross tunnel areas. Temporary support was carried out when deemed necessary from a working safety point of view.

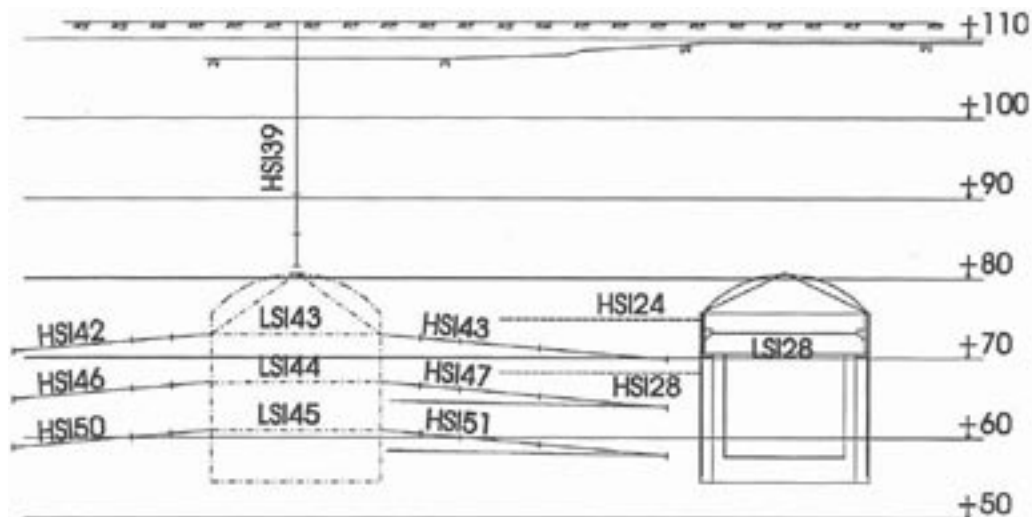


Figure 5-9. Positioning of extensometers /Fredriksson et al. 2001/.

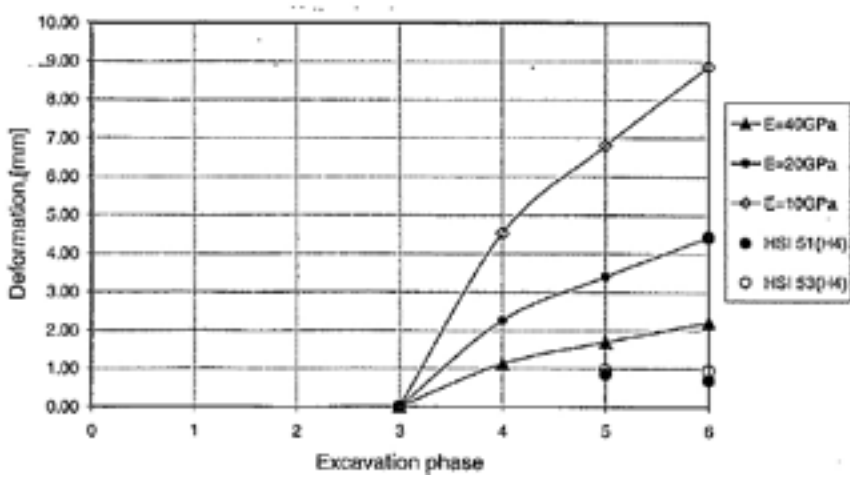
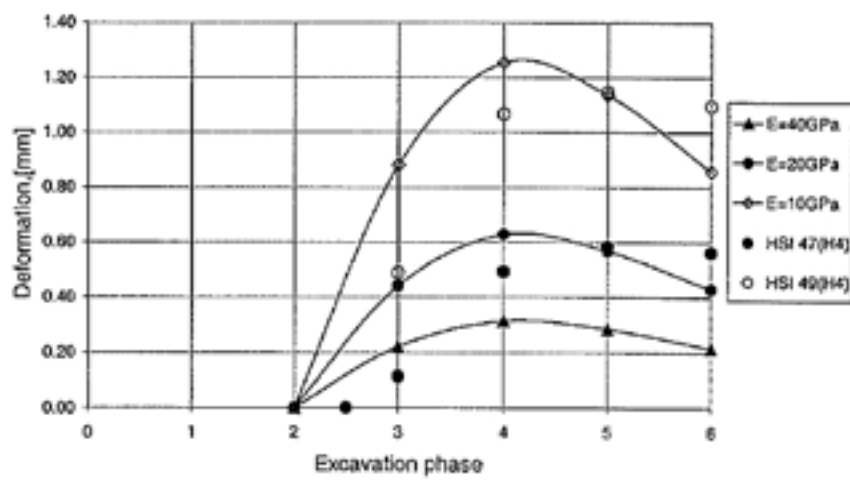
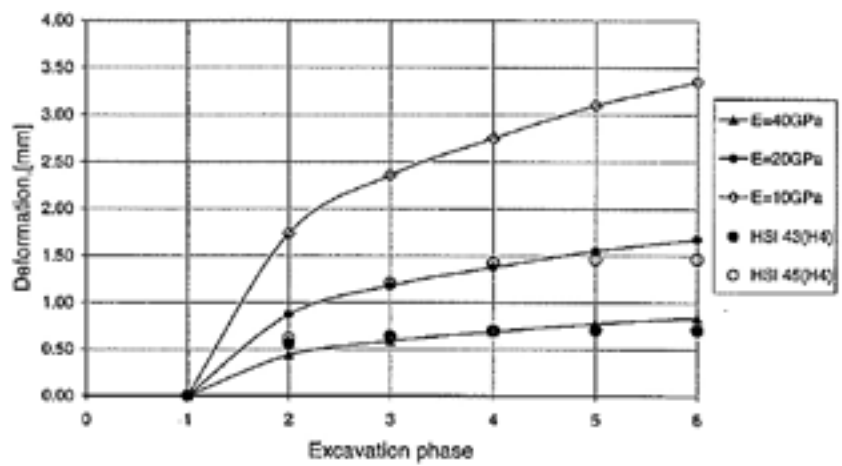


Figure 5-10. Comparison between calculated and measured deformations in Clab 2 /Fredriksson et al. 2001/.

About 10% of the installed rock bolts were tested using the Boltometer method. The Boltometer is an electronic instrument for non-destructive in situ testing and control of the quality of grouted rock bolts /Thurner 1979, Bergman et al. 1983/. For calibration purposes three bolts were installed in the crossing from the transport tunnel towards the storage caverns 1 and 2 respectively. Three of the tested production bolts showed defective grouting, and three bolts indicated reduced performance /Bodén 2002/. A number of easily accessible reference rock bolts (18) were installed to enable long-term testing and quality control.

In general, fibre reinforced shotcrete was used with a concrete cover of un-reinforced shotcrete.

The performed testing of the shotcrete showed low adhesion in areas of the metavolcanite probably due to numerous fine fractures in the rock mass causing rupture in the rock itself; i.e. behind the bond between rock and shotcrete (see above and Section 5.4.1). As a consequence, all the bolts installed in the metavolcanite were equipped with top plates independent of estimated Q-value /Bodén 2002/.

5.4.5 Grouting

The grouting works in the storage cavern and in the transept was carried out as continuous pre-grouting while the grouting in the transport tunnel was based on the result of the pilot drilling. Using mobile computerized equipment for simultaneous borehole grouting carried out the grouting.

The continuous pre-grouting operations in the storage cavern and the transept were performed in the top heading (roof and spring line) with horizontal 20-m long boreholes angled 10 degrees towards the tunnel. After grouting, 3–4 rounds were blasted and another grouting was executed. The grouting of the walls was carried out with vertical holes from the benches. Thus, the pre-grouting was performed from three levels: (1) top heading and walls, (2) walls from the invert of the top heading, and (3) walls from bench level +62.50. In total, about 19,600 litres of grout was used with a water-cement ration of 0.7–0.8.

5.5 Inspections and rock engineering experiences from the operation of Clab 1 and 2

5.5.1 Overview of rock inspections

After Clab 1 was taken into operation, the underground facility has been subjected to recurrent inspections of rock quality and support, and measurements of rock deformation. In connection with the construction of Clab 2, the rock inspection programme was revised and supplemented with respect to rock deformations /Stille and Fredriksson 1998/, alarm system /Chang 1998/ and limit values for vibrations /Johansson 1998/. The observation and measurements of groundwater follows /Rhén and Ejdeling 1998/. SKB introduced a measurement and surveillance programme when the underground construction works were completed with special emphasis on the installation works

An inspection programme was elaborated for the complete underground facility at the commencement of the operation phase of Clab 2 /Lundin 2003/. In principle, this programme follows the original programme for measurements, inspections and maintenance /Larsson 1997/.

The underground structures of Clab are, like any other tunnels in Sweden in which personnel is working, subject to the rules of the Swedish Work Environment Authority (Arbetsmiljöverket). The following rules applicable for Clab are AFS 2001:1 and AFS 2000:12.

The recommended time interval between the rock inspections of the entire underground facility has been set to a minimum of four years. In addition, there are a number of various monitoring and inspections, which are carried out with a frequency from once per month (inspections of the opening gaps between the ceiling and the rock surface in the caverns and in the transport tunnel) up to once every second year (such as measurements of rock deformation and temperature, basin movements, movements in expansion joints between the transept and the basins, and groundwater levels around the facility). Pre-stressed anchors in the Clab 1 cavern are checked once a quarter as well as opening gaps between concrete and rock walls in the caverns, lower auxiliary building, the transept and the surface plant. The quantity of pumped water from the underground facility is also measured once a quarter. The time intervals between the various inspections were shorter during the installation phase of Clab 2.

SKB appoints an inspection team for the so-called Large Inspection (every fourth year), and the team members shall have good knowledge of the Clab facility and be experienced in concrete and rock engineering issues.

5.5.2 Rock engineering experiences from the operation phase

The first production round for Clab 2 was blasted on 18th of January 1999 and the last one was blasted on the 26th of September 2000.

A rock inspection and compilation of measurement results from the period 1998 to 2004 were carried out in 2004 /Lundin 2004/. In conclusion, the measurements and the inspections carried out during the extension of Clab show that the Clab facility was not negatively affected by the extension with the new cavern.

Monitoring

Table 5-1 summarises the results from the extensometer and convergence measurements carried out 1998–2004 /Lundin 2004/. Table 5-2 shows the check limits and measuring accuracy for the measurements.

Levelling in 36 positions has been carried out in the storage basins in Clab1 distributed along the base of the basin. The measurement is performed once every fourth year (Figure 5-11). The result shows that the differential settlement is within the check limit.

Table 5-1. Summary of results from extensometer and convergence measurements carried out 1998–2004.

Object	Measurements	Measuring period	Result
Roof – cavern 1	Mini-extensometer	1999–2001	Very small movements; stable conditions during the Clab 2 extension
Roof – cavern 1	Manual optical convergence	1999–2002	Max movements in one section from 1999 to Autumn 2000: 1.7 mm; other sections: 1.0 mm; after completion of excavation: 0.1 mm
West wall – cavern 1	Extensometer	1999–2004	Movements: < 1 mm
East wall – cavern 1	Extensometer	1998–2004	Stable conditions
Walls below the concrete roof – cavern 1	Manual optical convergence	1999–2004	Deformation within check limits. Max difference in length: 2 mm
Rock volume above cavern 1 – from ground surface	Extensometer	1998–2004	Stable conditions

Table 5-2. Check limits and measuring accuracy.

	Check limit (mm)	Measuring accuracy (mm)
Extensometers	± 3.0	± 0.05
Convergence	± 4.0	± 0.5
Precision levelling	± 5.0	± 0.5
Mini extensometer	–	± 0.1

Thus, the extensometer, convergence and levelling measurements carried out during 1998–2004 demonstrate stable conditions.

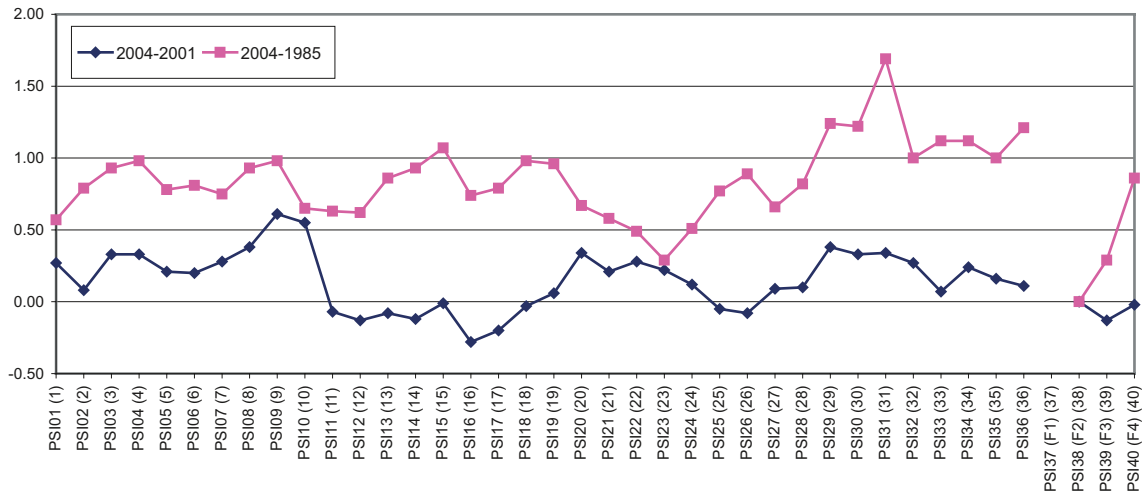


Figure 5-11. Levelling of storage basins, Clab 1, 2004-06–2001-11 and 2004–1985 /Lundin 2004/.

Measurements of basin movements (and temperatures) are carried out as a function control of the plain bearings of the basins. The longitudinal movements are within the check limits, ± 3.0 mm (cf Figure 5-12). The extension of the basins in a North-South direction, which occurred between June and August 2001, is probably due to that the basin temperature was approximately 4°C higher than normally during a period in July 2001. In 2002 and 2003, the extension of the basins was influenced by large temperature changes in the basins. The transverse movements (East-West) are very small /Lundin 2004/.

During the construction stage, a check limit of water inflow to the underground structures was set to 80 l/min, and during the operation phase 60 l/min. The recording of water inflow to cavern 1 and 2 from 1999 to 2004 is illustrated in Figure 5-13. It is to be noted that the quantity of water inflow is below the allowable quantity both during the construction works and the

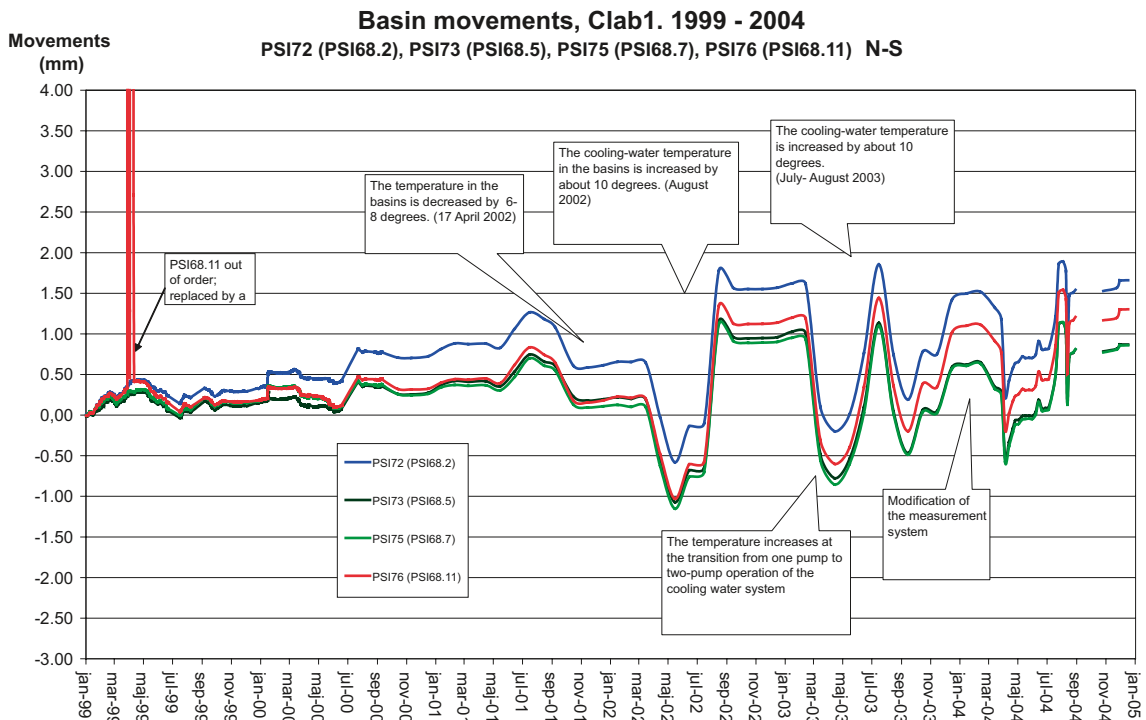


Figure 5-12. Clab 1, basin movements N-S /Lundin 2004/.

operation phase. The increase of water inflow during the summer 2003 is explained to be a result of the heavy rainfall occurring in the Oskarshamn region in the first week of July 2003 /Lundin 2004/. The diagram in Figure 5-13 is, however, influenced by a longer time period since the diagram represents mean values of inflow during an 8-week period. Furthermore, the higher inflow may be affected by incorrectness in the pump control, which occasionally occurred. Since the water inflow is calculated from operation time of the pumps, and at times when the pumps were in operation without pumping water, this may partly explain the higher water inflow /Lundin 2004/.

Rock inspection

The latest rock inspection of the underground Clab facility was carried out in 2004 comprising inspection of the opening gaps between concrete and rock walls in the caverns and the auxiliary building, transport tunnel, and the opening gap above the ceiling in the caverns.

The objective with the inspection was to observe and substantiate any moisture, damages or other conditions, if any, of importance for the maintenance and safety of the plant. It should be added that the environment in the opening gaps between concrete and rock surfaces in Clab 1 is significantly moister in comparison with Clab 2 due to differences in ventilation.

For each of the inspected elements in Clab 1 and 2, the inspection report /Lundin 2004/ concludes concordantly that: *“No observations of damages or other conditions of importance for the safety of the facility have been recorded.”*

In conclusion, the measurements performed during the operation phase and the 2004 rock inspection of the Clab 1 and 2 underground facilities do not indicate any damages, deterioration or failure of installed rock support, nor any stability problems or unexpected deformation. No problems related to groundwater have been identified.

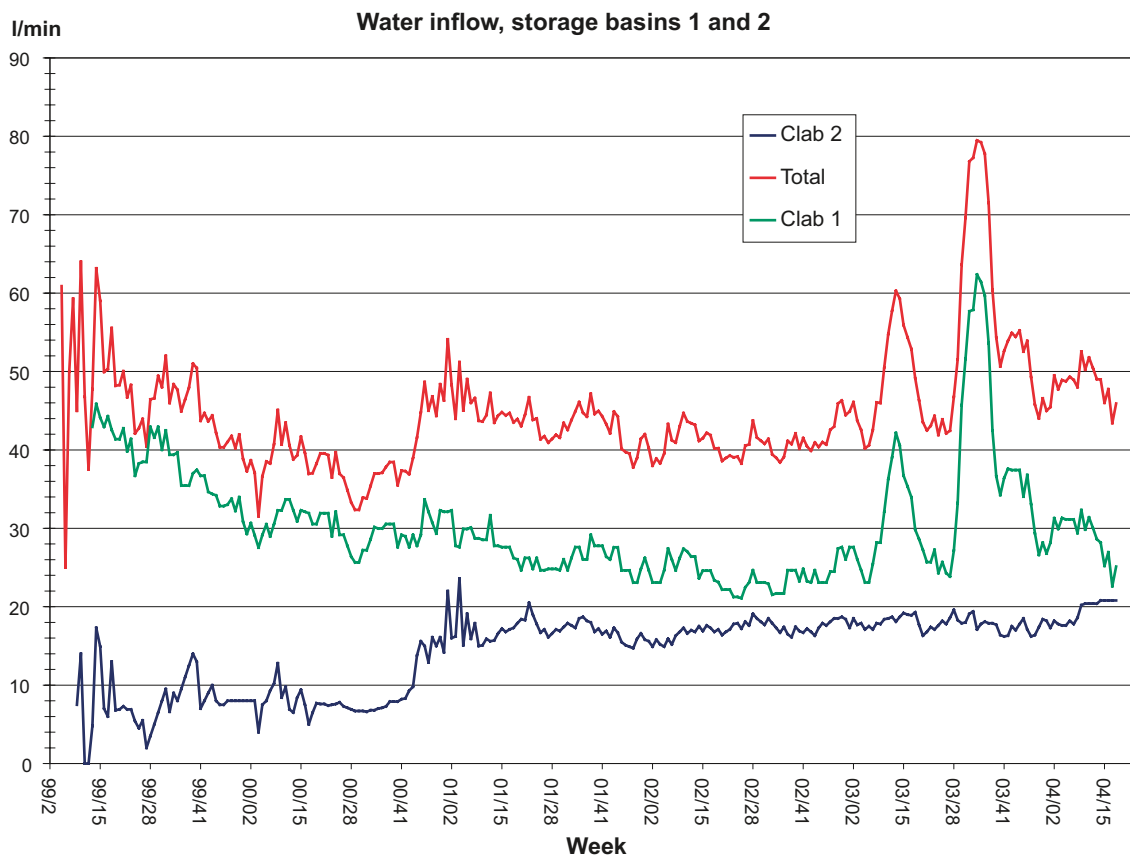


Figure 5-13. Water inflow in Clab 1 and 2 rock caverns /Lundin 2004/.

6 Äspö Hard Rock Laboratory – underground works

6.1 Introduction

The Äspö Hard Rock Laboratory (HRL), in the Simpevarp area in the municipality of Oskarshamn, constitutes an important part of SKB's work to design and construct a deep geological repository for final disposal of spent nuclear fuel and to develop and test methods for characterization of a suitable site.

One of the fundamental reasons behind SKB's decision to construct an underground laboratory was to create an opportunity for research, development and demonstration in a realistic and undisturbed rock environment down to repository depth. The underground part of the laboratory consists of a tunnel from the Simpevarp peninsula to the southern part of Äspö where the tunnel continues in a spiral down to a depth of 450 m. The rock volume and the available underground excavations had to be divided between all the experiments performed at the Äspö HRL.

Figure 6-1 shows an overview of the plant layout and in Figure 6-2 the allocation of a selection of the experimental sites in Äspö HRL is shown.



Figure 6-1. Overview of the layout of the Äspö Hard Rock Laboratory.

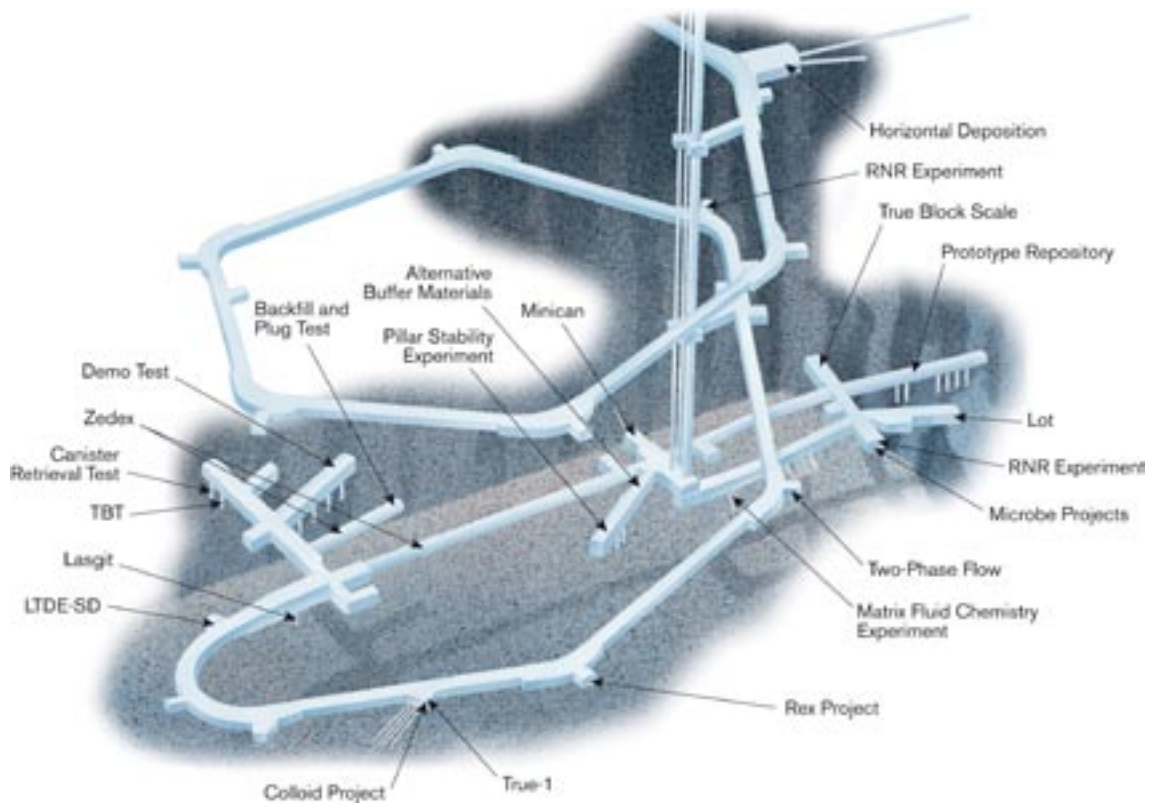


Figure 6-2. The allocation of a selection of the experimental sites in the Äspö Hard Rock Laboratory.

The Äspö HRL and the associated research, development and demonstration tasks, managed by the Repository Technology Department within SKB, have so far attracted considerably international interest. During 2006, nine organisations from eight countries participated in the cooperation or in Äspö HRL-related activities.

Design work was initiated in the spring of 1989 and SKB's application for a license to construct Äspö HRL was approved in 1990 and the work began in the autumn 1990. The construction work was completed in the summer of 1995, and the actual operation of the facility started the same year.

The design and construction of the HRL has comprised several parts and stages. Shaft as well as a ramp was discussed as alternatives for access to facility level. An access ramp was preferred since a ramp would give a better operational flexibility and a larger rock volume for investigations in comparison with a shaft.

The underground part of the laboratory consists of a tunnel from the Simpevarp peninsula to the southern part of Äspö where the tunnel continues in a spiral down to a depth of 450 m (Figure 6-1). The total length of the tunnel is 3,600 m where the last 400 m have been excavated by a tunnel boring machine (TBM) with a diameter of 5 m. The first part of the tunnel has been excavated by conventional drill and blast techniques. The underground tunnel is connected to the ground surface through a hoist shaft and two ventilation shafts. Äspö Research Village is located at the surface on the Äspö Island and it comprises office facilities, storage facilities, and machinery for hoist and ventilation.

The tunnel ramp was excavated from the Simpevarp peninsula about 1.5 km out under the island Äspö. The descent to the tunnel is situated in the vicinity of the Oskarshamn nuclear power plant. The tunnel reaches the island Äspö at a depth of 200 m and then continues in a hexagonal spiral down to a depth of 340 m below the sea level (contractual requirement). The tunnelling

of this first construction part was entirely done by means of drill and blast methods. For the second part of the spiral, from –340 to –450 m level, full-face boring with a TBM was tested from 420 m. Also the first part of the second spiral follows the hexagonal shape and was carried out by drill and blast. The tunnel then goes down to the –450 m level close to the shafts and continues horizontally westward to an experimental rock volume.

Three shafts have been built for communication and supplies to the experimental levels. Two shafts with a diameter of 1.5 m are for ventilation, and one hoist shaft with a diameter of 3.8 m. The shafts were excavated by raise-boring technique.

The total length of the tunnel is 3,600 m, depth of the communication shafts is 450 m and the total excavated rock volume is approximately 150,000 m³. The first part of the tunnelling work included all rock excavation and installations in the tunnel down to the –340 m level as well as raise boring of shafts from –220 m to the ground level. The second part of the tunnel work included the TBM tunnelling and the remaining rock excavations, raise boring and installations down to full depth.

A comprehensive compilation of the findings from site exploration, design and construction stages are given in the following reports:

- Stanfors R, Erlström M, Markström I. Äspö HRL – Geoscientific evaluation 1997/1. Overview of site characterization 1986–1995. SKB TR 97-02.
- Rhén I (ed.), Bäckblom G, Gustafson G, Stanfors R, Wikberg P. Äspö HRL – Geoscientific evaluation 1997/2. Results from pre-investigations and detailed site characterization. Summary Report. SKB TR 97-03.
- Stanfors R, Olsson P, Stille H. Äspö HRL – Geoscientific evaluation 1997/3. Results from pre-investigations and detailed site characterization. Comparison of predictions and observations. Geology and Mechanical stability. SKB TR 97-04.
- Rhén I, Gustafson G, Wikberg P. Äspö HRL – Geoscientific evaluation 1997/2. Results from pre-investigations and detailed site characterization. . Comparison of predictions and observations. Hydrogeology, Groundwater chemistry and Transport of solutes. SKB TR 97-05.
- Rhén I (ed.), Gustafson G, Stanfors R, Wikberg P. Äspö HRL – Geoscientific evaluation 1997/2. Models based on site characterization 1986–1995. SKB TR 97-06.
- Almén K-E (ed.), Olsson P, Rhén I, Stanfors R, Wikberg P. Äspö Hard Rock Laboratory. Feasibility and usefulness of site investigation methods. Experience from the pre-investigation phase. SKB TR 94-24.
- Almén K-E, Stenberg L. Äspö Hard Rock Laboratory. Characterisation methods and instruments. Experiences from the construction phase. SKB TR-05-11.

The selected information about design and underground works at the Äspö HRL as presented in this report is mainly derived from /Hamberger 1993, Bodén et al. 1996, Hedman 1999/ and /Larsson and Leijon 1999/. In particular, /Bodén et al. 1996/ and /Hedman 1999/ analyse general design and layout aspects in full detail. Descriptions of and experiences from the grouting campaigns carried out in the access tunnel are first and foremost derived from /Stille et al. 1993/.

6.2 Site investigations

Exceptional extensive investigations were carried out for the Äspö HRL, during the planning stage as well as during the construction phase. The scope of the investigations was a direct consequence of the purpose and character of the facility.

The site investigations for the Äspö HRL started in late 1986. The site investigation phase involved extensive field measurements, from ground level as well as from boreholes, and

analyses aimed at characterizing the rock formation with regard to geology, geohydrology and hydrochemistry and rock mechanics. The results of the investigations are presented in, inter alia, /Gustafson et al. 1989, Wikberg et al. 1991, Stanfors et al. 1991, Almén and Zellman 1991/.

Prior to excavation of the HRL, predictions for the excavation phase were made. These predictions were based on data collected during the site investigations conducted between 1986 and 1990. The predictions concerned five key issues: (1) geological structures, (2) groundwater flow, (3) groundwater chemistry, (4) transport of solutes and (5) mechanical stability. These predictions were made in three scales: site scale (100–1,000 m), block scale (10–100 m) and detailed scale (0–10 m) /Gustafson et al. 1991, Rhén et al. 1995/.

A large amount of data was collected during the excavation, for example piezometric levels in about 150 borehole sections in boreholes around Äspö HRL, inflows to the tunnel, mapping of lithology and fractures and leakage characteristics. Groundwater sampling and measurements of hydraulic properties of the rock mass along the tunnel were also made /Rhén et al. 1995/.

The exceedingly comprehensive reporting of the works described above is not for self-evident reasons related in detail in this compilation report. Analyses of the results in full details are presented in, inter alia, /Bäckblom et al. 1990, 1994, Rhén et al. 1995, 1997abc, Stille et al. 1997, Stanfors et al. 1997b, Almén et al. 1997/. Several other in-depth reports are also given in the list of references (Chapter 10).

6.2.1 Site characterization

Below a brief outline is given of the geoscientific evaluation, mainly conclusions, concluding remarks, from pre-investigations and site characterization. This outline is in its entirety derived from /Rhén et al. 1997a/. The goals of pre-construction and construction phase characterization 1986–1995 is given in Table 6-1.

The site investigations for the Äspö Hard Rock Laboratory prior to construction of the facility were planned to meet two requirements:

- First to conduct the investigations necessary to design the underground facility so that it could be constructed with presently available technology without major problems down to a depth of about 500 m. The construction of the Äspö HRL was successfully made down to a depth of 450 m with only minor layout changes.
- Secondly to obtain a thorough understanding of the rock conditions based on investigations of the surface and investigations in and between boreholes drilled from the surface.

Site characterization in conjunction with construction work at Äspö HRL basically confirmed the pre-construction models. However, the models were more detailed after the construction period.

Table 6-1. The goals of pre-construction and construction phase characterization 1986–1995.

Stage	Description
Goal 1	Demonstrate that investigations at the ground surface and in boreholes provide sufficient data on essential safety-related properties of the rock at repository level.
Goal 2	Refine and verify the methods and the technology needed for characterization of the rock on the detailed site investigations.
Goal 3	Refine and test on a large scale at repository depth methods and models for describing groundwater flow and transport of solutes in rock.
Goal 4	Provide access to rock where methods and technology can be refined and tested so that high quality can be guaranteed in the design, construction and operation of the final repository.

The work at Äspö showed that such pre-construction models could be obtained for the studied key issues through the application of standard methodology of good quality for measurements, data analyses, modelling and evaluation. Standard geological and geophysical methods in combination with hydraulic tests showed lithological domains and the geometrical framework.

Hydraulic tests in and between boreholes showed the existence of the major hydraulic conductors and their geometry. In spite of scaling problems, reasonable estimates of hydraulic conductivity were achieved.

Sampling of groundwater was done and subsequent chemical analyses were put the Äspö groundwater in a regional context as well as created an understanding of the past evolution of the groundwater at the site.

The technology for rock stress measurements and interpretation was available, but it was judged that additional studies were needed to explain differences in results from different methods.

Tracer tests were used to examine the connectivity of the hydraulic framework and to find crude parameter estimates for non-sorbing transport.

Considering the stage goals the characterization should demonstrate that investigation at the ground surface and in boreholes provide sufficient data on essential safety-related properties of the rock mass at repository level (Stage goal 1). The work at Äspö demonstrated that relevant safety-related data could be obtained.

The results and experience from Äspö are partly general in nature and partly site-specific and they should be relevant for planned site characterization in the Swedish bedrock. If these findings should be transferred to other types of bedrock and target depths appropriate modifications of the characterization and modelling could be required.

In site characterization for a deep repository, methods should only be used that have been shown to be useful and feasible in practice. Should new methods be considered in the site investigations for a deep repository, the Äspö facility will provide excellent conditions for testing prior to application at a real site.

The methods and technology needed for characterization of the rock in the detailed site investigations were tested and developed (Stage goal 2). Valuable experiences of construction-testing integration were also obtained.

The site characterization at Äspö was used to refine and test on a large scale at repository depth methods and models for describing groundwater flow and the transport of solutes in rock (Stage goal 3). The work demonstrated many different approaches to modelling that seemed useful.

Design and construction of the Äspö Hard Rock Laboratory, incorporating both drill- and blast excavation and mechanical excavation using a tunnel boring machine provided access to rock where methods and technology can be refined and tested so that high quality can be guaranteed in the design, construction and operation of the final repository (Stage goal 4) /Rhén et al. 1997a/.

6.2.2 Rock mass classification

A classification of the rock mass which was to be intersected by the decline and ramp was made by /Stille and Olsson 1990/ in which the rock mass was divided into five different rock classes, classes A–E. The rock mass classification was based on the pre-investigations performed in the area and the conceptual model that was established.

The purpose of the classification was to divide the rock mass into representative groups in which the rock mechanical characteristics were different. For this classification, the Geomechanics Classification System proposed by Bieniawski was applied. This system operates

with five parameters describing the rock mass and it is simple to use when the classification is based on pre-investigation data. If any parameter is missing, it is possible to estimate the value of the missing parameter.

In the Geomechanics Classification System the parameters describing the rock mass are allocated points according to a proposed scale. The rock mass is described by parameters for strength of rock material, RQD-value, spacing of discontinuities, conditions of continuities and the inflow of water to the underground development. The sum of all points allocated to the different parameters describes the rock mass in the form of the RMR-value (Rock Mass Rating). Finally the RMR-value is adjusted for the fracture orientation. The RMR-value varies between 0 and 100.

The following prognosis of the rock mechanical conditions to be expected to occur in the tunnel is presented in Table 6-2 /Stille and Olsson 1990/. The rock classes were related to the Geomechanics Classification System, RMR /Bieniawski 1976, 1989/.

The predicted RMR-values from the site investigations as presented in Table 6-2 and the RMR-values based on tunnel mapping during construction /Markström and Erlström 1996/ are shown in Table 6-3. As seen, the predicted and the observed RMR-values are in reasonable good agreement.

Table 6-2. Rock mass classification along the Äspö HRL tunnel based on site investigations /Stille and Olsson 1990/.

Rock class	RMR-value	Description
A	> 72	Competent rock of Smålands granite or diorite, sparsely fractured, RQD 90–100, distance between fractures 1–3 m. Existing fractures are closed. The rock mass is dry or minor inflow of water may occur, < 25 l/min and 10 m. Fracture orientation is favourable for the tunnel orientation.
B	60–72	Granite or diorite with a higher fracture frequency than Rock class A, RQD 50–90, distance between fractures 0.3–1.0 m. Competent greenstone/metavolcanics or fine-grained granite with RQD > 75. Existing fractures are closed and rough. Minor inflow of water, < 25 l/min and 10 m. There is also some Rock class A but with unfavourable fracture orientation.
C	40–60	Granite with a high fracture frequency, RQD 50–75, distance between fractures 5–50 cm. Fracture surfaces are planar-undulating and occasionally altered. Inflow of water 25–125 l/min and 10 m. Highly fractured diorite, RQD < 25, distance between fractures 5–30 cm. Closed fractures with unaltered fracture surfaces. Minor inflow of water. Fine-grained granite, RQD 50–75, 0.3–1.0 spacing between fractures. Inflow of water under moderate pressure, 25–125 l/min and 10 m. Greenstone, RQD 60–95 with 0.3–1.0 spacing between fractures. Unaltered fracture surfaces, surface staining only. Minor inflow of water, occasional outwash of open fractures.
D	< 40	Minor fracture zones with RQD < 50, 5–30 cm spacing between fractures. Zones are < 5 m wide and are intersected by the tunnel at a sharp angle. Fracture surfaces are planar-undulating and altered. Fractures are occasionally filled with clay minerals. Inflow of water under moderate pressure, 25–125 l/min and 10 m.
E	< 40	Zones with highly fractured granite or mylonite. Zones are > 4 m wide and are intersected by the tunnel at a sharp angle. Fracture surfaces are altered. Inflow of water 25–125 l/min and 10 m. The tunnel intersects some sub-horizontal fracture zones with highly fractured rock, RQD < 25. The width of the intersected zones is 10–40 m. Fracture surfaces are altered. Inflow of water 25–125 l/min and 10 m. In /Sturk et al. 1996/, data originates from evaluation of the actual distribution in the lower ramp tunnel. In this case, two classes have been identified to describe the rock mass along the TBM tunnel, namely Class A+B (RMR > 60) and Class C (RMR 40–60) equal to good and fair rock respectively according to /Bieniawski 1976/.

Table 6-3. Comparison between predicted and observed RMR-values along the tunnel (section 700–2,874 m) /Markström and Erlström 1996, Stanfors et al. 1997b/.

Rock class	RMR-value	Predicted distribution	Observed outcome
A	> 72	23%	28%
B	60–72	50%	39%
C	40–60	19%	29%
D	< 40	3%	4%
E	< 40	5%	

The classification of the rock mass along the tunnel section (due to a change in the planned tunnel layout it was only possible to make a comparison between the prediction and outcome of tunnel section 700–2,874 m) showed that there was a relationship between RMR-values and rock type. As shown in Table 6-4, the fine-grained granite has a lower mean and larger range of RMR-values than the other rock types (greenstone, Äspö diorite and Småland granite).

In /Sturk et al. 1996/, data originates from evaluation of the actual distribution in the lower ramp tunnel. In this case, two classes were identified to describe the rock mass along the TBM tunnel, namely Class A+B (RMR > 60) and Class C (RMR 40–60) equal to good and fair rock respectively according to /Bieniawski 1989/. Tunnel orientation both parallel and perpendicular to fractures.

The occurrence and distribution of rock classes was modelled using the spire model (new data generating process) and the occurrence of discontinuities was modelled using a Poisson process. The input parameters were initially assessed subjectively based on engineering experience. The geological model assumed vertical or sub-vertical structures crossing the tunnel with a fairly large angle /Sturk et al. 1996/.

Table 6-4. Outcome of RMR-values in different rock types along the tunnel (section 700–2,874 m) /Stille and Olsson 1996/.

	Greenstone	Fine-grained granite	Äspö diorite	Småland granite
Mean	64	48	69	65
Interval	53–74	15–89	28–97	35–92
Std deviation	6	13	10	11
No of values	18	69	289	202

Table 6-5. Rock mass classification along the TBM tunnel /Sturk et al. 1996/.

Rock class	RMR-value	Description
A+B	> 60	Competent rock of Småland granite, Äspö diorite or greenstone, sparsely to moderately fractured with distance between fractures ranging from 0.5–3 m. Fractures are closed and mainly rough. RQD is 50–100. No or only minor inflow of water.
C	40–60	Småland granite and fine-grained granite with high fracture frequency (RQD 50–75) or highly fractured Äspö diorite (RQD < 25). Distance between fractures ranges from 0.05–0.5 m. Fracture walls are planar-undulating and occasionally altered. Minor or only limited inflow of water. This rock also includes greenstone (RQD 60–95) with fracture spacing ranging from 0.3–1 m. Unaltered fracture walls with surface staining only. Minor inflow of water, occasional outwash of open fractures.

An alternative for the calculation of rock class distribution was used by applying Bayes-Markov process for calculating the probability of entering certain states when moving in different directions /Rosén 1995/. This model was used for calculating the expected distribution of rock classes along the TBM tunnel. A comparison between the different models is presented in Table 6-6.

Both models underestimate the occurrence of rock class A+B, and according to /Sturk et al. 1996/ it is difficult to draw any conclusions about the applicability of the different geological models based on the limited testing carried out. It should also be noted that both models require some sort of initial engineering judgement on expected rock class distribution in the actual rock mass. In addition, it is obvious that the final results are sensitive to these initial assessments.

6.2.3 Comparison between prediction and outcome

Apart from the general purposes, to form the basis of siting, site adaptation and construction, the objectives of the investigations contained tasks such as:

- To verify investigation methods; i.e. to demonstrate that investigations from ground surface and in boreholes provide adequate data to safety related rock mass properties on deposition level. Comparisons between the predictions and observations were made during excavation to verify the reliability of the pre-investigations.
- To establish detail investigation methodology; i.e. to develop the methodology further in order to verify investigation methods and techniques to be used in characterizing the rock mass at a selected site.
- To test models suitable for description of barrier functions of the rock mass; i.e. to develop and test methods and models further to describe groundwater flow, radionuclide migration and chemical properties on deposition level.

One important element in verifying and testing methods and models involved prognosis of the rock mass conditions by means of investigations from the ground surface, and subsequently compare the prognosis with actual conditions during construction /Larsson and Leijon 1999/.

The geoscientific evaluation, based on comparisons between prediction and outcome is presented in detail in /Gustafson et al. 1991, Stanfors et al. 1997b, Rhén et al. 1997a/. However, in this section only some conclusions, derived from /Rhén et al. 1997a/ are reported.

Geological models

The following are a selection of conclusions concerning geological models /Rhén et al. 1997a/.

Predictions of lithology on the regional scale are mostly reliable as regards the distribution of major lithological domains such as granite diapirs and big massifs of basic rock, where aero-geophysical data was available and well-exposed bedrock.

Table 6-6. Expected rock class distribution along the TBM tunnel calculated using different geological models /Sturk et al. 1996/.

Rock class	Initial prognosis Sturk et al.	Updated prognosis Sturk et al.	Updated prognosis Rosén	Out- come
A+B (%)	80	81	71–73	92
C (%)	20	19	29–27	8

The main rock types were generally well predicted for most of the tunnel rock volume whereas the spatial distribution of minor rock types was uncertain. Borehole data provided only representative information for a small volume close to the borehole. In a complex rock mass such as at Äspö therefore dikes and veins of fine-grained granite and greenstone for example could not be predicted with respect to exact position at depth.

The position and general properties of major fracture zones were accurately obtained from the surface and borehole investigations in particular for sub-vertical structures. The main structural pattern was normally revealed at an early stage of the site investigations using air-borne geophysics and lineament interpretation. Accurate information on the dip and character of the structures was obtained from more detailed investigations in the form of ground-based geophysics and drilling (see Figure 6-3).

Refraction seismic, core drilling, geophysical logging and radar in boreholes resulted in generally good agreement between the prediction and outcome where major sub vertical fracture zone data are concerned, such as orientation and general character. An exception, however, was fracture zone NE-2, which was predicted to be major and dip to NW, outwards from the spiral but underground the zone demonstrated to be a minor fracture zone dipping to the SE. The reason for the poor prediction was that only one cored borehole crossed the zone, which turned out to be winding and irregular.

All horizontal structures found in the tunnel were narrower and less hydraulically important than had been predicted, based mainly on geological field observations. There was no agreement, except as regards the existence, between prediction and outcome.

The reliability of minor (local) discontinuities (less than 5 m wide) was good regarding their existence and hydraulic character but not so good as regards the extent and structural character at depth.

The characteristics of small-scale fracturing in the rock mass between the fracture zones such as orientation of the main fracture sets and fracture infillings were mainly concordant with the predictions (cf Figure 6-4). Spacing was almost within the predicted range. It was not possible to estimate fracture length from borehole investigations at depth.

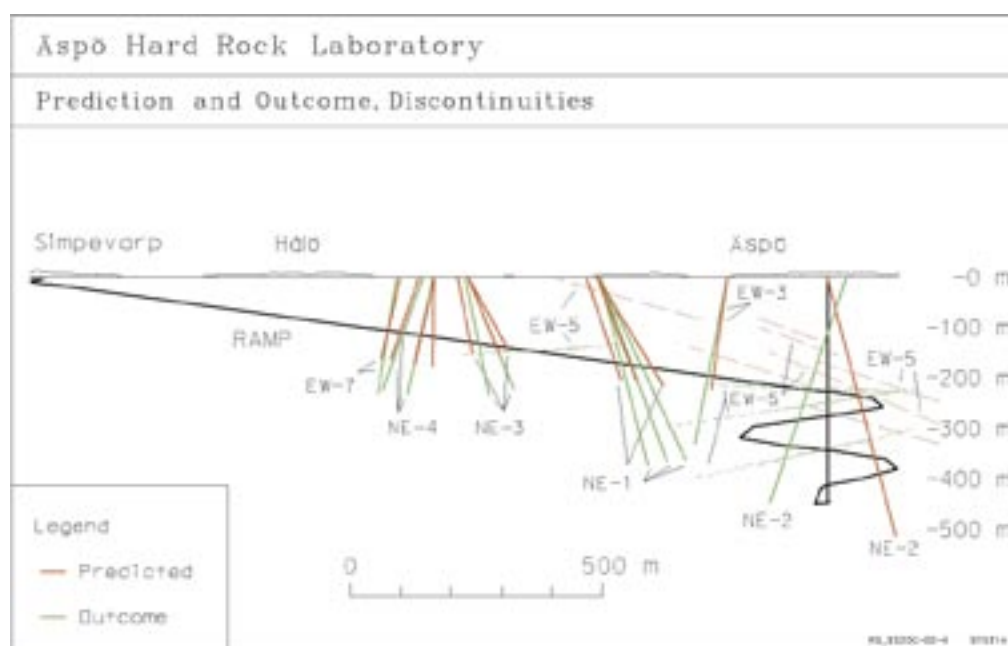


Figure 6-3. Prediction and outcome of major fracture zones /Rhen et al. 1997a/.

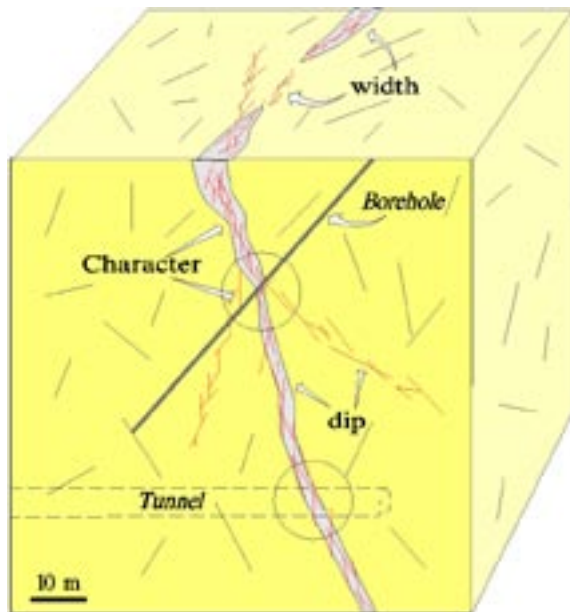


Figure 6-4. The figure illustrates that modelling fracture zones as planes of constant width and orientation is a gross over-simplification /Rhén et al. 1997a/.

Mechanical stability

The following are a selection of important conclusions concerning mechanical stability /Rhén et al. 1997a/.

At Äspö experts judged there would only occur some possible stress related problems in greenstones. No indications of stress induced spalling were collected and thus the expert judgement of the mechanical stability at Äspö proved to be correct. Models for excavation stability were not thoroughly tested due to the limited stress levels at Äspö.

The difference between the results from laboratory testing of rock type parameters in the site investigation phase and the excavation phase was significant. The outcome for several parameters was wider than the predicted range, thus not fully accounting for the natural variations in the rock. Further, the number of samples was small, thereby providing a low level of confidence for mean and variance values. The coupling between rock types, mechanical properties and thermal properties has been explored and evaluated in connection with the site description – version 1.2 and 2.1 /SKB 2006ab/.

The prediction of rock stress orientation corresponded well to the outcome. The relation between the maximum horizontal stress and the theoretical vertical stress, K_0 , was predicted to be in the range of 1.7 while the outcome was 2.9. Average values for individual boreholes varied between 1.7 and 4.0.

The difference between the predicted rock stress levels and the outcome can possibly be explained by geometric factors and geological variations. It is also probable that a large portion of the differences was due to the different methods used to make the measurements. This has been further explored by /Ask 2004/. Hydraulic fracturing resulted in lower maximum horizontal stress levels than the results of measurements made using the overcoring method.

The prediction of rock quality for the tunnel, using the RMR system, showed acceptable correspondence to the observations made in the tunnel (cf Section 6.2.2). The rock quality is very dependent on the rock type. Fine-grained granite exhibits both larger variations and significantly lower mean RMR values than greenstone, Småland (Ävrö) granite and Äspö diorite /Andersson and Söderhäll 2001/.

Groundwater flow models

The following are a selection of conclusions concerning models for groundwater flow and overall evaluation concerning hydro-geological concepts derived from /Rhén et al. 1997a/. Hydrogeological and grouting results are given in Section 6.5.

The prediction of the total flow into the tunnel was successful but the flow rate into the tunnel was difficult to predict as the amount and effect of grouting was not known beforehand. The flow rate was more or less governed by the effect of the grouting as grouting was only performed when the tunnel intersected conductive parts of the rock mass.

The hydraulic conductor domains controlled the drawdown to a large extent in the modelling approach used at the Äspö HRL. The groundwater flow simulations are described in /Gustafson et al. 1991, Rhén et al. 1991, 1997c, Svensson 1995, Wikberg et al. 1991/.

The hydraulic test methods used can in general be said to have been sufficient for the models made. The different investigations methods applied for the site investigation phase of the Äspö Hard Rock Laboratory comprised airlift tests, clean out and pumping test of boreholes, spinner or flow-metre measurement of the borehole, injection tests (3 m packer interval and 30 packer interval), interference tests, groundwater monitoring, borehole deviation measurements and dilution test.

To construct a reliable model it is important to perform tests on different scales systematically in the boreholes, both for scale relationships but also to gain flexibility in the interpretation of how to divide the rock mass into hydraulic conductor domains and hydraulic rock mass domains. It is also important to perform large-scale interference tests for modelling purposes.

It is probable that there is some spatial correlation within the hydraulic rock mass domains due to some large and more transmissive features not accounted for in the concept used.

Groundwater chemistry

The following are a selection of conclusions concerning overall evaluation of models for groundwater chemistry derived from /Rhén et al. 1997a/.

The processes considered to have the largest impact on the groundwater chemistry were mixing, calcite dissolution and precipitation, redox reactions and biological processes. In addition to these fast processes, the long-term groundwater/rock interaction has largely affected the groundwater chemistry and produced brine with a total salinity of nearly 100g/l.

Mixing of water from different sources was considered to be the main reason for the observed hydro chemical situation. Methods and models were developed for identifying the different sources of groundwater at the Äspö site and the way in which groundwater at specific locations in the rock mass could be described as a mixture of water from these sources.

It was clearly demonstrated that the groundwater at depths greater than 100 m are reducing, and that the dissolved oxygen in the infiltrating surface water is consumed by bacteria. In addition to the inorganic reactions between oxygen and minerals, the effective oxygen consumption by bacteria strengthened the general opinion that anoxic (oxygen free) conditions could always be expected in the deep groundwater.

The observed biological processes were (1) oxygen consumption by oxidation of organic matter, (2) reduction of iron minerals through oxidation of organic matter and (3) reduction of sulphate by oxidation of organic matter.

The experiences and knowledge made /Rhén et al. 1997a/ to define four practical and relevant levels of chemical information:

- i major dissolved components.
- ii I + trace elements and stable isotopes.
- iii ii + pH sensitive elements, tritium and carbon-14.
- iv iii + Eh and redox sensitive components.

6.3 Rock engineering design

The design work started in 1989 and the design and construction comprised several parts and stages (cf Section 6.1). The design approach was flexible allowing adjustments to be made as knowledge increases about the rock mass conditions once the excavation work has started; i.e. an active design approach was chosen. In this particular project, a close cooperation with research staff and scientists for the planning of experiments also implied flexibility in the design and construction work.

The design that was chosen had the portalling at the Simpevarp peninsula, two spiral turns and a final depth of –450 m. The decision on tunnel dimensions was decided on the profiles of various vehicles at a maximum of 3.5 m in height and 3.0 m in width. The section chosen in construction stage 1 was 25 m² and 5 m wide. The width was however reduced to 4.8 m during construction. In curves and in passing places, the tunnel has been widening to a total of 8 m, having a cross sectional area of 42 m². The TBM tunnel (410 m) has a diameter of 5.0 m. At the facility level, there are a number of drifts and spaces for experimental activities and service systems. The total length of the drifts is about 470 m /Bodén et al. 1996, Hedman 1999, Larsson and Leijon 1999/.

Curve radius in the conventionally blasted tunnel is normally 20 m, except in the longer curves, at the entrance and just ahead of the TBM assembly shop, where the radius is 40 m.

The tunnel is used for heavyweight transports. The lift shaft (diameter 3.8 m) is used for light-weight transportation and for transporting personnel. The lift shaft and the two ventilation shafts (diameter 1.5 m) are connected to landing levels at –220 m, –330 m and –447 m. The inclination in the tunnel was chosen to be 140 ‰, levelling out to 100 ‰ in curves and passing places. The choice of 100 ‰ enables a vehicle to start with less trouble than in steeper rise. The inclination in the spiral's second turn was chosen to be 140‰ also in the curves which proved to work well /Bodén et al. 1996, Hedman 1999/.

Turning niches have been provided for at the beginning of each curve. Originally they were planned for use when excavating the tunnel, but also for investigation borings. Initially the turning niches were located at the end of each curve, but they were moved to the beginning of the curves on request from the contractor. Passing places in the access tunnel are located 150 m apart from each other. In the spiral tunnel, passing places are placed at the exact end of each curve. The reason for the location of areas at the end of the curve was that vehicles driving down the tunnel should not have to reverse when encountering a vehicle on a straight section.

Various designs of the communication shafts were evaluated during the planning period. The raise-boring alternative of the three shafts was not initially considered to be the best alternative due to the high costs. However raise boring was chosen due to the advantages of low risk of personnel injuries involved in such boring compared to drilling and blasting /Bodén et al. 1996, Hedman 1999/.

The ventilation and the passenger lift systems were purchased as functional contracts.

The allowed declination in the TBM assembly cavern was changed during the design stage from the maximum declination of 40‰ to the 20‰ that were finally decided on. The launch hole for the TBM boring at –420 m was blasted flatter than it was designed for, wherefore, on the advice of the contractor, the declination of the TBM tunnel was increased to 145‰ down to the –450 m level. The walls of the TBM launch tunnel must have counter stays for the pressure plates of the TBM. The contractor designed the counter stays, and normally they would be cast in concrete. However, in this case, the rock mass quality was sufficiently good for the contractor to use cautious blasting in order to create the counter stays and consequently no costly concrete casting was needed.

The transformers were built-in containers and then put in position in niches. Experience from the operation of the facility has shown that it would have been better if the niches had been made wider and higher to facilitate accessibility and maintenance.

Initially it was planned to cast the fireproof wall located between the spiral tunnel and the landing areas but the contractor wanted to avoid any structures involving casting, wherefore different solutions were studied. The chosen solution was to cast a concrete frame, secure it to the rock, and brick up the rest of the wall with lightweight clinker blocks, a solution that proved to be successful /Bodén et al. 1996, Hedman 1999/.

6.4 Rock excavation and construction equipment

6.4.1 Introduction

The underground construction works of the Äspö HRL are described in /Hamberger 1993, Larsson and Leijon 1999/ and in /Hedman 1999/ from which some selected data and experiences are summarized below. As described in Section 6.1, the total length of the tunnel is 3,600 m of which about 3,200 m refer to conventional drill and blast and about 400 m was excavated by TBM. In addition, three 450 m deep shafts were built using raise-boring technique.

6.4.2 Excavation by drill and blast technique

The cross-sectional area of the 3,200 m long drill and blast tunnel is 25 m². Two drilling rigs were used: one computerized drilling rig fitted with 18-ft feed (Tamrock Data Maxi) and secondly, one conventional type of rig equipped with 3 booms fitted with 16-ft feed (also a Tamrock rig). This manually operated rig was equipped with a computerizing directing device, the so-called Bever Control enabling recordings of drilling parameters such as pressure, drill rotation pressure, penetration pressure etc. This type of manual rig proved, according to /Hedman 1999/, to be more suitable for probe drilling and other types of drilling.

A limitation of the damage zone in the remaining rock of only 0.3 m in the contour and 0.6 m in the floor when blasting full face was tested at the beginning. The limitation of the damage zone in the walls did not create any problem. However, keeping the floor damage as low as 0.6 m proved to be difficult at a reasonable cost using available explosives. After a blasting damage investigation had been made /Hamberger 1993/, the contractor and the client agreed on a blasting configuration reasonable for both parties. It caused a theoretical damage zone of less than 0.5 m in the contour and less than 1.5 m in the floor when using Nitro Nobel Dynamex and Gurit. Only explosives in cartridges were allowed due to electric current in the ground /Hedman 1999/.

Scaling was carried out manually.

An electric powered Load Haul Dump (LHD), a Toro 500E, was used to muck out rock. The LHD had a bucket volume of 7 m³, equivalent to about 18,000 kg of blasted rock. SCANIA 143 6x4 trucks were used for rock haulage. The trucks were equipped with particle traps and catalyser.

The tunnel support was carried out with conventional methods; i.e. spot bolting, pattern bolting, un-reinforced and fibre reinforced shotcrete and grouting. In the first straight part of the tunnel (about 1,600 m), up to the spiral, about 5% of the length was supported with un-reinforced shotcrete, 15% with fibre-reinforced shotcrete and 2.5% with mesh-reinforced shotcrete, mostly shotcrete arches. Almost a third of the shotcrete was applied in the deformation zone NE-1. The rock support, stability, support philosophy and grouting in the Äspö HRL tunnel is outlined in Section 6.5 and 6.6.

6.4.3 Excavation with TBM

The TBM tunnel has a length of 409 m having a diameter of 5 m. The first layout for the second phase of construction was based on a tunnel spiral with a radius of 150 m, making full turn from the position of the shafts on level -340 m down to the -450 level. It was also realised that if the TBM failed to maintain the curve radius it could come in contact with the deformation zone NE-1. By experience from the earlier passage of NE-1 it was clear that another contact should be avoided due to the extent of grouting that was needed for a safe passage of NE-1 /Larsson and Leijon 1999, Hedman 1999/.

Alternative layouts were studied in order to explore possible ways to minimize risk. It was decided to start the TBM tunnel half way down at a point in the western region of the spiral and go with a moderate horizontal radius (250 m) down to the final level. In this way the NE-1 could be avoided and the direction of the TBM tunnel became more or less perpendicular to some minor but water-bearing fractures /Hedman 1999/. The TBM tunnelling was carried out during the summer of 1994.

The TBM used was a TBM type Jarva Mk15 manufactured by Robbins Europe equipped with a 5 m diameter drilling head. The weight of the drilling head was about 50 tons, and the total weight of the TBM was about 200 tons. To adapt the machine to the requirements and conditions of the borings at Äspö it was equipped with some special functions as /Hedman 1999, Larsson and Leijon 1999/:

- Two drilling machines, COP 1238, for drilling of holes for probes and grouting.
- Passings in the boring-head for drilling of holes in the tunnel face.
- Electrical frequency charger for step less variable speed.
- Logging programs for TBM and probe drilling.
- Short back-up rig with only six decks.
- A spare diesel generator with capacity for ground water drainage pumps.
- Operators cabin designed as a rescue chamber.

The TBM arrived dismantled to the site, and was assembled in the erection chamber at -420 m. A 200 m long core hole was drilled in the centre line of the planned tunnel before the start of the TBM. The result of that core holes was used for the planning of grouting during boring. The option to drill probe holes through the boring head and the logging program was requested by SKB. The boreholes for grouting and hydro-tests were performed successfully from the TBM by the two drilling rigs. Totally, 56% of the TBM tunnel was grouted. When probe drilling, the inflow in a single hole was up to 120 l/min. During the TBM tunnelling no reinforcement of the rock was needed. The first 200 m of the tunnel were bored downwards with a declination of 14.5%, The mucking out was done with a Toro 500E, which carried the muck from the end of the conveyor belt at the back up rig to the erection chamber. From this point rock haulage to the ground surface was done with the SCANIA 143 trucks /Larsson and Leijon 1999, Hedman 1999/. Some data from the TBM boring is shown Table 6-7.

Table 6-7. TBM boring data, Äspö HRL /Hedman 1999/.

Average capacity	1.36 m/h effective boring time
Highest capacity	2.5 m/h effective boring time
Average weekly production	32 m
Best day	14.7 m
Best week	48.7 m
Utilization degree	Approximately 30%

6.4.4 Excavation of shafts with raise boring

Three shafts were constructed for communication and supplies to the experimental levels. Two shafts with a diameter of 1.5 m are for ventilation, and one shaft with a diameter of 3.8 m is for the lift. The shafts were excavated by raise-boring technique. According to /Hedman 1999/ and /Larsson and Leijon 1999/, the reason for choosing raise-boring was to minimize the hazards during the construction period in comparison of using conventional raise drilling and blasting.

The first stage of the raise boring was carried out from the ground surface down to the –220 m level. As regards the larger diameter shaft (lift shaft), a pilot hole of 360 mm was drilled in order to fulfil the high requirements on limitation of deviations. The deviation, however, turned out to be slighter larger than acceptable and reaming of the lower part, from the –220 m level up to the –170 m level was done with a 4.1 m reamer head. For the other two raise boring stages (100 m long each), the pilot holes were within the deviation limits and the 3.8 m reamer was used all the way. The demands for the ventilation shafts (diameter 1.5 m) were not so high and a smaller machine and pilot hole were used /Hedman 1999, Larsson and Leijon 1999/.

In the first part, down to the –220 m level, pre-grouting was performed in the pilot holes. Limitations similar to those for grouting in the tunnel were imposed also in this work. The result were unsatisfactory and an extensive grouting work was needed after reaming of the shafts. The pre-grouting from –220 m down to –450 m was therefore performed in drilled holes peripherally around the positions for the shafts. This method gave a satisfying result and no grouting work was needed after the reaming. For the upper part above the –220 m level, installations had been made in order to collect the groundwater inflow and leading it to the drainage system /Hedman 1999, Larsson and Leijon 1999/.

6.4.5 Excavation experiences

The experiences from the underground construction works at the Äspö Hard Rock Laboratory are presented in /Hedman 1999, Larsson and Leijon 1999, Hamberger 1993/. The last mentioned author deals with the first part of the tunnelling work, while the other authors cover the planning and design phases as well as the second part of the tunnelling work including the TBM drilled part. An outline of the experiences given in the mentioned reports is given below. In addition, engineering issues encountered during tunnelling through the deformation zone NE-1 evaluated and concluded by /Chang et al. 2005/ are reported below.

The general experience from blasting was that it is most important for the achievement of smooth and little damaged tunnel periphery to have straight and carefully directed blasting holes. The experience was that computerized drilling rigs were not fully developed at that time. A skilled driller achieved the best drilling results when drilling manually, aided by a computerized directing device /Hamberger 1993/.

An electric loader considerably improved the working environment for the workers and the researches. The trucks for rock haulage were also part in ensuring of a good working environment. The trucks were equipped with a particle trap and a catalytic converter.

The experience from the TBM tunnelling was that the vertical curve from the erection chamber to the declined part of the tunnel created problems at the beginning with some damages to the first and last deck of the back up rig. The first part of the horizontal curve presented also a problem due to the fact that in this part the tunnel described both a horizontal and a vertical curve. After this part, a horizontal curve of 230 m was performed. Problems with cutters followed in the first 200–300 m of the tunnel and totally 90 cutters were exchanged /Hedman 1999/.

Due to the decline of the tunnel it was difficult to pump water in front of the TBM head until start up of the boring. This caused problems with transport of the muck when starting after shorter or longer breaks. The inclination of the conveyor belt caused by the declination of the tunnel made the saturated muck goes in the wrong direction.

The very saline water created a lot of problems for the construction equipment as well as the permanent installations. This implies high demands on daily maintenance on e.g. drilling rigs. The choice of materials and coatings for the installations had to be made carefully to withstand the corrosive environment /Hedman 1999/.

The stability conditions during the underground excavation works and during the present operational stage have been favourably. The rock support has mainly comprised spot bolting to secure rock blocks in the roof. Rock support could be omitted in large parts of the tunnel, and more significant rock support measures were only used in connection with larger fracture zones, deformation zones /Larsson and Leijon 1999/.

Rock spalling was not observed during the tunnelling. Occasionally, click noise was heard after blasting, and minor tendencies of extraction of intact rock were noted /Larsson and Leijon 1999/.

The tunnel down to the spiral intersected a number of fracture zones, and the passages of the deformation zones NE-3 and NE-1 were in particular time consuming due to high water inflows resulting in comprehensive investigations and grouting works. At NE-3 the excavation was in contact with a site investigation borehole running parallel with the tunnel roof. This together with unfavourably fractures zones motivated a minor displacement laterally, relatively to planned tunnel alignment /Larsson and Leijon 1999/.

/Chang et al. 2005/ have made an eminent summary of the engineering issues encountered during tunnelling through NE-1, based exclusively on findings in a number of SKB's reports. The summary presented in /Chang et al. 2005/ is in its entirety given below.

The major difficulty experienced during the passage of NE-1 was related to the grouting and drilling operations due to the fractured nature of the rock mass and the relatively high water pressure. Initially grouting was carried out with restrictions on the maximum volume of grout that was to be pumped into each hole and the injection pressure in order to limit the extent of the grouted zone. In addition, the initial concept was based on relatively short grout holes that did not penetrate the entire deformation zone as well as using a grout mix that had a relatively low initial strength. This concept was planned with the aim of minimizing impact on hydrological conditions.

Due to the unsatisfactory sealing results of the grouting, changes were made to the grouting strategy. Tests were conducted to determine the optimum grouting fan, injection pressure, grout medium and initial grout strength. It was found that long grouting holes penetrating the whole fracture zone and grouts with higher initial strengths gave more effective grouting results. Grouting holes with high water-cement ratios did not have desired sealing effects. Theoretical and practical evidence also indicated that the grouting pressure needed to exceed twice the water pressure to provide a sufficient dispersion of the grout cement. A new grouting concept based on these findings was then employed and the water inflow was reduced to an accepted level. The tunnel excavation proceeded at a relatively rapid pace of about 2 m per day after implementation of the new grouting strategy.

Problems were encountered during the drilling due to water pressure acting on the drill rods and pushing them out of the hole at high speeds. This resulted in dangerous working conditions for

the drill crews and caused partial flooding of the drill site. Introducing a special drilling arrangement that included a combination of drill niches, pump niches, borehole casing and a special water valve solved the problem. When the drill rod was extracted, the valve was closed in order to limit the inflow of water from the borehole, which meant that the drill site remained relatively safe and dry. The system also improved the possibility to measure flow rates from the borehole.

No indications of tunnel instability have been noted in the reports with regard to the passage of deformation zone NE-1. However, concerns relating to high water pressure destabilizing the tunnel face, particularly during the drilling and grouting have been noted. In order to guard against this, the distance between the tunnel face and deformation zone NE-1 was kept to about 20–30 m to ensure face stability during the drilling and grouting operations. The boreholes drilled from the face were cased to prevent water from being introduced to the fractures in the vicinity of the face. Rock support consisted of fibre-reinforced shotcrete, rock bolts and steel mesh installed in tunnel walls and roof, no details have been found which suggest the use of face support. No details of deformation measurements were found in the SKB's reports.

6.5 Rock support

6.5.1 General

The rock support philosophy for the underground structures was partially based on the requirements set by the research and scientific staff in their request to avoid or significantly minimize disturbances on the experiments; i.e. to use as little shotcrete as possible, or preferably use wire mesh instead of shotcrete, and to completely avoid grouting.

In reality, conventional rock support measures were applied, not least considering safety aspects. However, there is no doubt that the contractor during the excavation adapted his rock support work, to the utmost possible extent, to the above mentioned requirements without waiving the safety aspects or lower his standards of quality. It should be emphasised that the contractor's operational support works in the tunnel was carried out after due consultations with SKB.

The rock bolts used at the Äspö HRL were tensioned and un-tensioned reinforcement bars, mainly Örsta bolts, Titan bolts and Swellex. In Sweden, at that time, Swellex was not accepted as permanent rock reinforcement. Before using Swellex it was tested by the Swedish Corrosion Institute to ensure that it would withstand the corrosive environment in the Äspö HRL. The tests showed that a coated Swellex would suffice. SKB therefore approved the Coated Super Swellex type as components in the permanent rock reinforcements /Hamberger 1993/. In total, more than 500 expandable roof bolts of Swellex were used in the Äspö Hard Rock Laboratory. The exact amount and their locations are accounted for in /Markström and Erlström 1996/.

The Örsta bolt is a galvanized and epoxy coated bolt with an expanding tip. It immediately secures loose rock. When the bolt is grouted it gets an extra cover to protect it from corrosion, which makes it acceptable as permanent rock reinforcement. The Titan bolt was used at the Äspö HRL, although they are mostly used in soft rock conditions. The bolt comprises a drill rod with a drill bit. The hole is drilled and the rod is then left and grouted in the hole, functioning as a bolt. Titan bolts were used in the Äspö HRL when passing the deformation zone, NE-1. They were used for spiling and were placed in a row in the roof, pointing forward and slightly upward, forming a secured roof over the next round. This was made in most rounds, each row of bolt overlapping the previous when passing through NE-1 /Hamberger 1993/.

Un-reinforced as well as fibre reinforced shotcrete was used in the Äspö Hard Rock Laboratory.

Placing a cast concrete ring around the shaft did a reinforcement of the lift shaft's connection to the floor of the landing area. The ring was given a circular outline, although a cheaper quadrangular outlining could have been used without making the construction inferior. A quadrangular outline would also have eased the installation of the safety crate surrounding the lift shaft /Hedman 1999/.

6.5.2 Stability and support philosophy

A stability philosophy for the underground structures of the Äspö Hard Rock Laboratory was evaluated in /Stille and Olsson 1990/ and an outline of this philosophy is given below.

The stability of the underground openings will depend on a number of factors; both geometrical factors such as size and shape of the opening as well as structural and rock mechanical conditions will influence the stability. The underground openings of Äspö HRL are of a moderate size that does not normally cause any significant stability problems in Scandinavian rock. The division of the rock mass into different rock mass classes showed the predicted variations in structural and rock mechanical conditions. The laboratory tests proved that the strengths of both rock and fractures were normal for this type of rock in which stability problems are not usually expected.

The pre-dominant fracture pattern showed an east-west orientation with a steep dip of 70–90° with the horizontal plane, although gently dipping fractures were also observed.

The RMR system does not take into consideration the initial rock stress conditions. The relation between the maximum horizontal stress and the vertical stress, K_0 was stated to be around 1.7. The orientation of the maximum horizontal stress was measured to be E-W or N40°W. Both the magnitude and the orientation of the measured horizontal stresses were judged to be favourable for the stability. The measured stress orientation with the maximum horizontal stress almost perpendicular to the first 1,475 m of the decline was assessed to give the best possible conditions for the creation of a stable arch.

Both the expected stress conditions and fracture orientation were judged to be very favourable for the development of a stable arch without any major support measures being required. With the given rock mechanical conditions and the moderate tunnel area, the stability of the decline and ramp was assessed as being very good and potential problems limited to specific areas only.

Support measures were assessed as being related to the structural conditions and the rock quality. If gently dipping fractures with reduced friction would occur in the roof installation of un-tensioned grouted rock bolts should be applied to achieve a favourable arching effect. If the excavations would encounter areas with fractured to highly fractured rock with reduced strength and friction, it was assessed necessary to apply shotcrete to create an artificial arch that should support the surrounding rock enabling a rock arch to be performed.

The following stability philosophy was evaluated to apply to the different rock classes (Table 6-8).

Table 6-8. Estimated stability philosophy to apply to the different rock classes /Stille and Olsson 1990/.

Rock class	Characteristics
A RMR > 72	Instability of single blocks
B RMR 60–72	Instability of single key blocks that may progress to failure of the roof arch. Orientation of fractures will determine the amount of support necessary
C RMR 40–60	Instability in the roof. Difficult to locate all unstable areas. Both small and large blocks have to be supported. Bolts support large blocks and smaller blocks are supported by shotcrete. Bolts and shotcrete are applied systematically
D RMR < 40	General instability in walls and roof. A rock arch has to be established to make the tunnel stable. This necessitates systematic installation of bolts and shotcrete
E RMR < 40	As rock class D

Based on the stability philosophy and the rock mass classification presented in Section 6.2.2, an evaluation of the amount of rock support necessary was made. The evaluation concerned the final support of the tunnel (Table 6-9).

As mentioned before, the rock support philosophy for the underground structures was partially based on the requirements set by the research and scientific staff in their request to avoid or significantly minimize disturbances on the experiments; i.e. to use as little shotcrete as possible, or preferably use wire mesh instead of shotcrete, and to completely avoid grouting. Due to these demands, /Stille and Olsson 1990/ evaluated alternatives to shotcrete and pre-grouting and made the following proposals:

- In rock classes C and D shotcrete could be replaced by chain link mesh and supplementary bolting using alternating bolt lengths. In rock class E, the shotcrete must be replaced by steel arches and lagging. Occasionally the steel arches must be complemented with invert struts and lagging at the bottom.
- The dimensions and frequency of the steel arches and bolting replacing the shotcrete and mesh-reinforced arches cannot be determined until the actual rock conditions are known. Other support methods are possible to use in place of shotcrete, although this will effect the tunnelling operation and increase the demands on measurement and documentation.
- According to the evaluation, pre-grouting will be necessary in rock classes D and E, where inflow of water is expected. Pre-grouting will reduce the water inflow and enable the use of grouted bolts.
- If pre-grouting is not applied, the suggested rock support measures must be modified. Grouted bolts must be replaced by Swellex bolts or similar bolts treated with anti-corrosive agent.
- If steel arches or chain links are installed in areas with water inflow, the long-term stability will be limited due to risk of corrosion.
- Special treatment is necessary to avoid problems with water inflow at the front. Sumps must be excavated frequently depending on the amount of water inflow.

Table 6-9. Evaluation of rock support in different rock classes /Stille and Olsson 1990/.

Rock class	Roof	Walls
A RMR > 72	Spot bolting 1 bolt/15–20 m ²	No support required
B RMR 60–72	Spot bolting 1 bolt/10–15 m ²	No support required
C RMR 40–60	Systematic bolting 1 bolt/5 m ² Shotcrete 60 mm	Shotcrete 30 mm
D RMR < 40	Systematic bolting 1 bolt/3 m ² Shotcrete with fibres 60 mm Pre-grouting	Systematic bolting 1 bolt/4 m ² Shotcrete with fibres 60 mm Pre-grouting
E RMR < 40	Systematic bolting 1 bolt/2 m ² Shotcrete with fibres 90 mm Occasionally mesh-reinforced arches Pre-grouting	Systematic bolting 1 bolt/4 m ² Shotcrete with fibres 60 mm Occasionally mesh-reinforced arches Casting of bottom slab Pre-grouting

6.6 Water-bearing fractures and grouting of water-bearing fracture zones

Besides ordinary demands on water tightness to achieve occupational safety and an acceptable pumping volume, it was desired to achieve a limited spread of the grout to avoid excessive groundwater contamination around the tunnel. The total allowed inflow to the 1,500 m tunnel was specified to 3,000 l/min, corresponding to a water inflow of 78 l/min and 100 m of tunnel. Demands were also raised to show that the tunnel could pass highly water-bearing zones under full control of stability and water inflow. The demand on limited grout penetration was successively changed from the start with no limitation to a penetration of around 10 m for the passage of NE-1 that was found to be a reasonable value to achieve occupational safety and pumping volume. In the zones with poor rock conditions (highly fractured/crushed and partly clay altered) the minimum penetration was 6–8 m in order to reduce the negative effect of the water pressure on the stability /Stille et al. 1993/.

The evolution of total water inflow to the tunnel from 1991 to 2006 is shown in Figure 6-5, and the water inflow l/min and 100 m tunnel from section 1,372–1,584 to 2,840–2,994 is presented in Figure 6-6. A plan of weirs along the tunnel is presented in Figure 6-7.

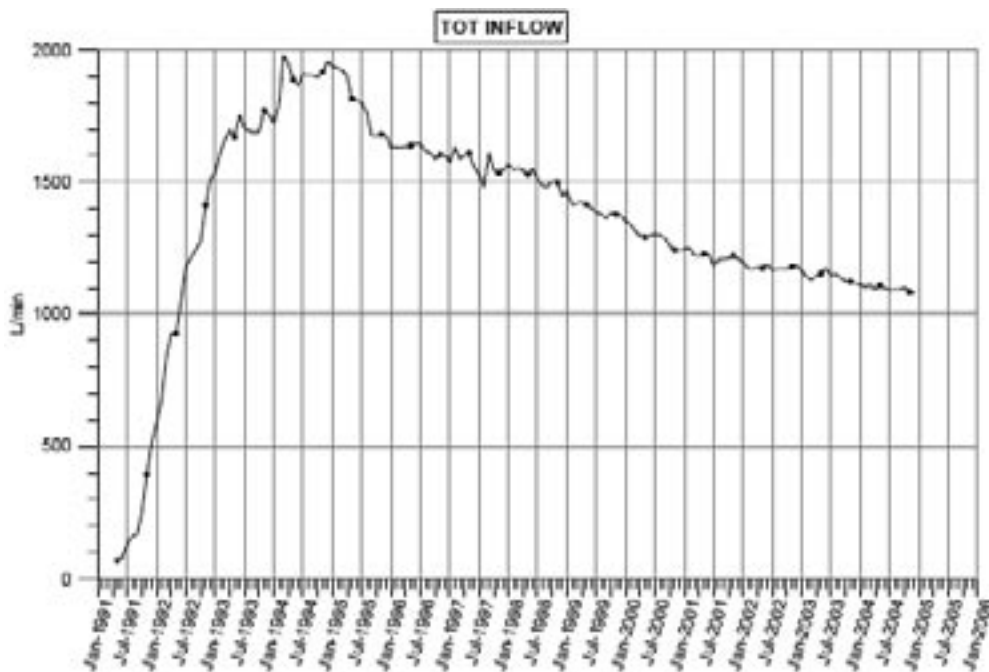


Figure 6-5. The evolution of total water inflow to the tunnel, Äspö HRL.

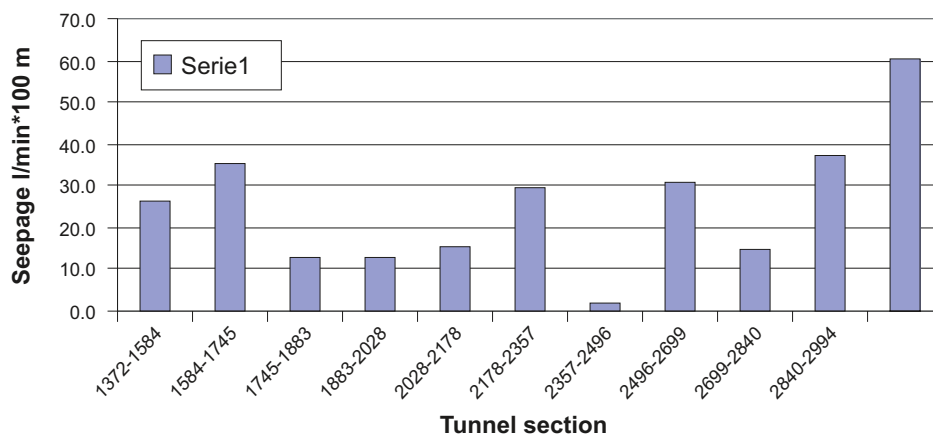


Figure 6-6. Water inflow l/min and 100 m tunnel, Äspö HRL.

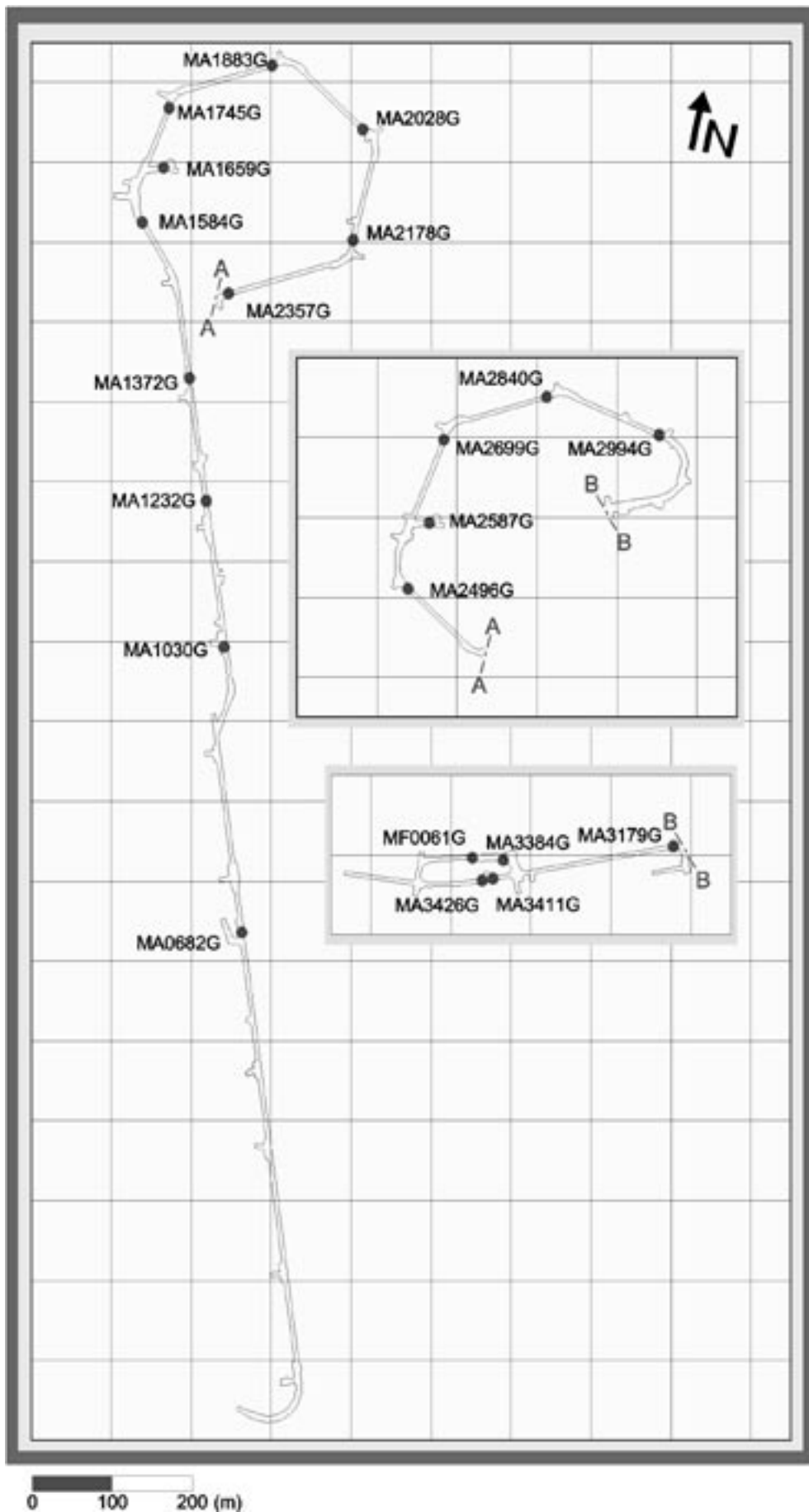


Figure 6-7. Plan of weirs along the tunnel, Äspö HRL. N = Magnetic North /Rhén et al. 1997b/.

Comparison of predicted and measured entities is presented in /Rhén et al. 1997b/. The prediction and outcome of the flow into tunnel section 700–2,875 m are shown in Figure 6-8. The outcome is 84–93% of the prediction. The outcome and predictions of flow from zones into the tunnel section 700–2,875 is shown in Figure 6-9.

The outcome and predictions of water flows into the tunnel legs (cf Figure 6-10) along tunnel section 700–2,875 are presented in Figure 6-11. As can be seen in Figure 6-11 the inflow rates are high in legs intersected by fracture zones EW-7, NE-3, NE-4 and NE-1 both in prediction and outcome /Rhén et al. 1997b/.

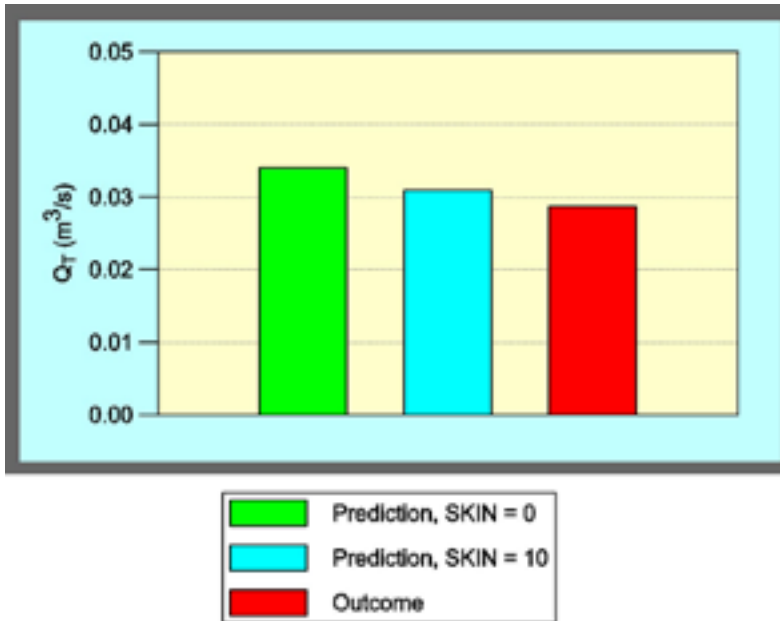


Figure 6-8. Water flow into the tunnel section 700–2,875 m. Predictions were for section 700–2,790 m /Rhén et al. 1997b/.

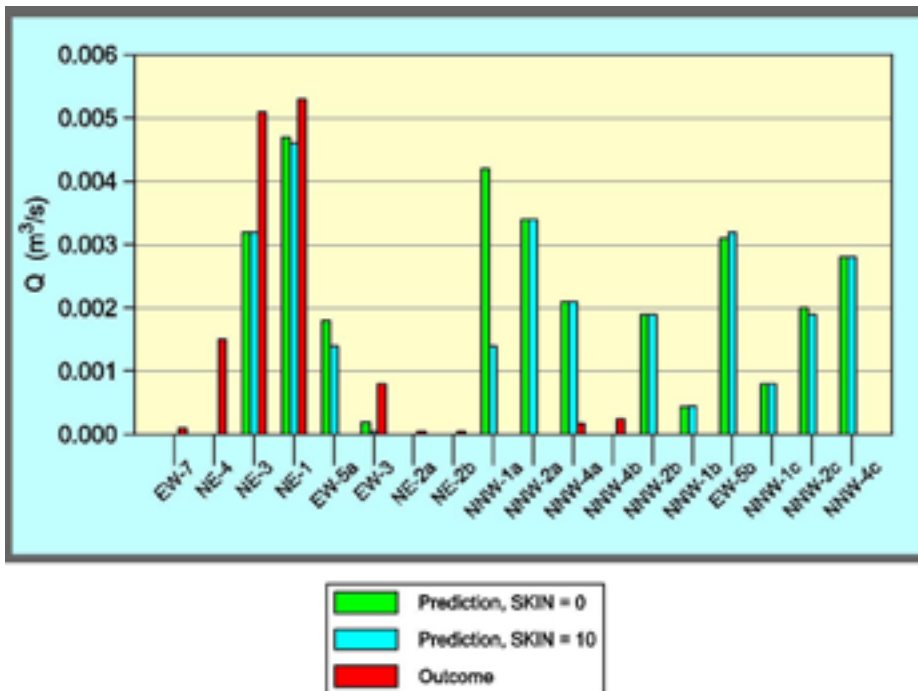


Figure 6-9. Water flows out of fracture zones along the tunnel section 700–2,875 m /Rhén et al. 1997b/.

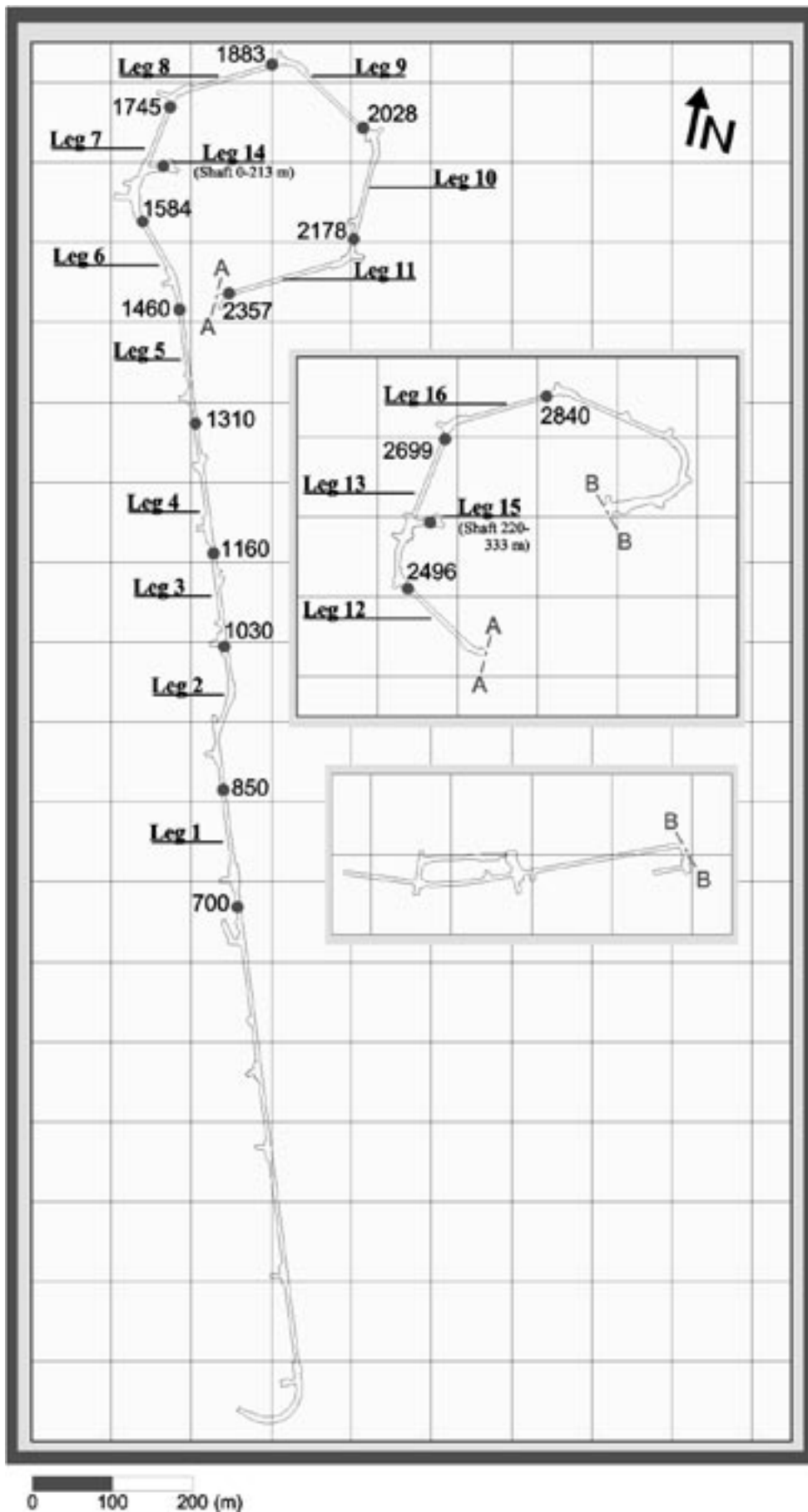


Figure 6-10. Plan of tunnel sections called 'legs' in the prediction, which were used for predictions of the flow into the tunnel /Rhén et al. 1997b/.

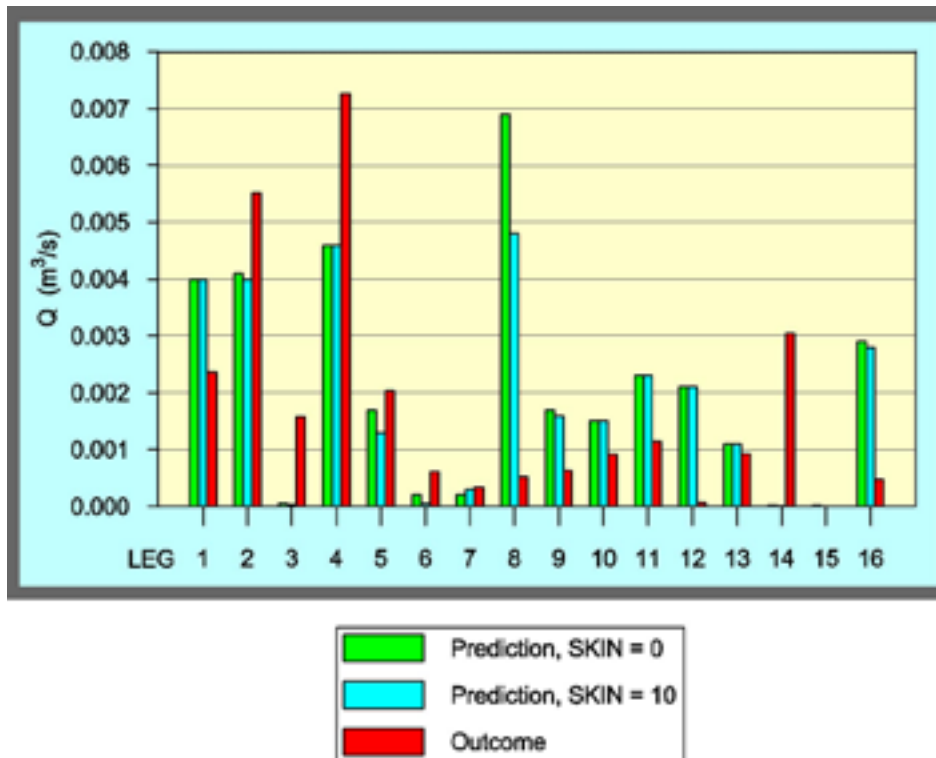


Figure 6-11. Water flows into tunnel legs along tunnel section 700–2,875 m /Rhén et al. 1997b/.

Mapped large, single, open water-bearing fractures in the tunnel are described in Figure 6-12. The fractures are mainly sub-vertical. All fractures, water-bearing fractures and fractures with grout respectively from 750 m to the end of the TBM tunnel, 3,600 m are shown in Figure 6-13 by Schmidt nets with lower hemisphere projection of Kamb contoured poles to fracture planes.

/Hermansson 1995/ carried out an investigation of the structural geology of water-bearing fractures and found that the entire fracture system could be grouped into five main sets (cf Figure 6-13). The mapped water-bearing fractures and the fractures filled with grout from the pre-grouting ahead of the tunnel face were dominated by a sub-vertical fracture set striking WNW-NW. But /Rhén et al. 1997b/ noted that the relevance of the orientation of the mapped water-bearing fractures can be questioned as the zone closest to the tunnel wall was damaged to some extent by the excavation giving increased fracturing and possibly a change in the hydraulic properties. Due to this the flow paths near the tunnel wall may be different than those of the undisturbed rock mass.

The mapping of major larger (intersecting more or less the entire tunnel) water-bearing fractures in the spiral showed that all mapped fractures either had a substantial water inflow and/or grout and often gouge, brecciation or ductile precursors /Hermansson 1995, Rhén et al. 1997b/. They were not in any case classified as zones and their widths ranged from millimetres to centimetres. According to /Hermansson 1995/ the fault system trending NW and NNW generally appears as sub-planar fractures with a central water-bearing fault plane that often contains fault breccias and/or fault gouge as well as mineral assemblage.

Prediction and outcome of the hydraulic conductivity for different rock types along the tunnel are also presented in /Rhén et al. 1997b/ and the result is shown in Figure 6-14.



Figure 6-12. Mapped large, single, open, water bearing fractures in the tunnel. The fractures are mainly sub-vertical. N = Magnetic North, x = North in the Äspö coordinate system /Rhén et al. 1997b/.

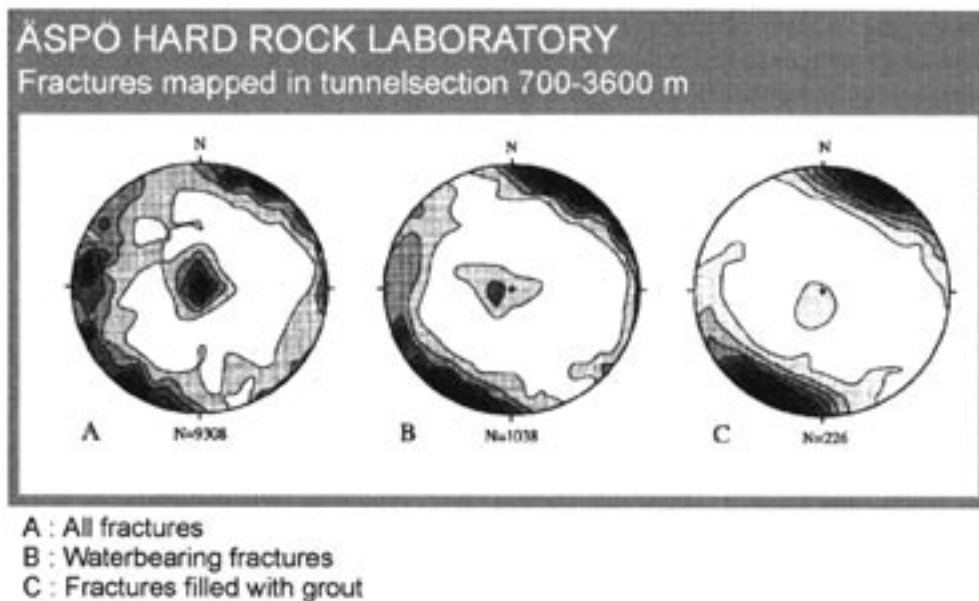


Figure 6-13. Schmidt nets with lower hemisphere projection of Kamb contoured pole to fracture planes. Contour interval 2.0 sigma. N = sample size /Hermansson 1995, Rhén et al. 1997b/.

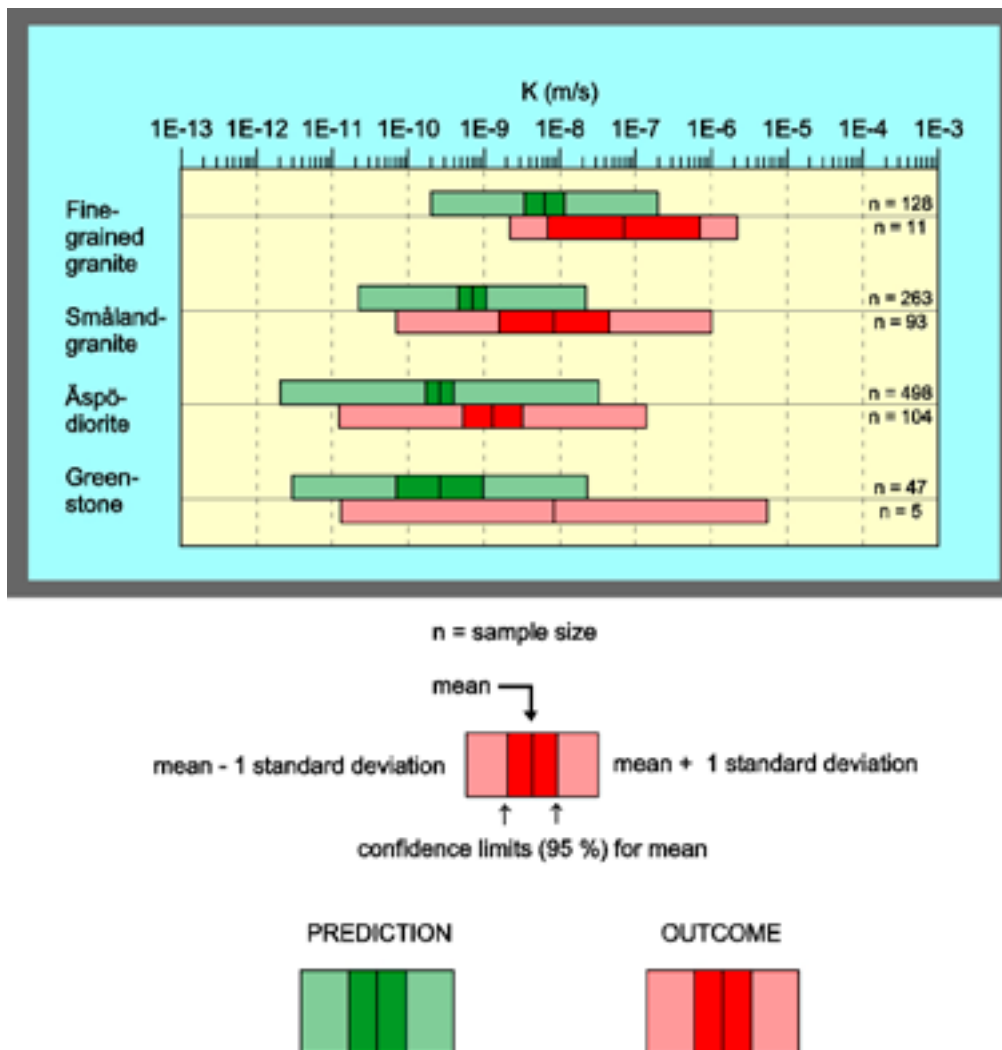


Figure 6-14. Hydraulic conductivity for different rock types along the tunnel. Detail scale. Test scale 14 m. The outcome is for tunnel section 1,475–2,875 m except for greenstone which is for tunnel section 700–2,875 m /Rhén et al. 1997a/.

Grouting became with time the main interest in the art of tunnelling at the Äspö Hard Rock Laboratory. It presented many new problems since so called new rules for the grouting was set for the Äspö Hard Rock Laboratory. Those rules stated that the materials used must have a limited impact on the groundwater chemistry and the spreading of the grout must be controllable. The allowed types of grout was cement with calcium chloride, cement and bentonite and certain chemical grouts, such as Taccs /Hamberger 1993/ (cf Section 6.4.5).

Six major fracture zones intersect the tunnel, section 1–1,400, with widths varying between 5 to 50 m, trending NE-ESE and dipping more or less vertical. Two of these zones, NE-3 and NE-1, have been particularly complicated and comprehensive grouting and rock support work were carried out. In addition narrow fracture zones occur, normally less than 5 m wide and single open fractures, mostly dipping vertical and trending approximately N-S and NNW. These structures have often been documented as structures with very high transmissivity. The experiences of the grouting are described in detail in /Stille et al. 1993/.

A total of 103 pre-grouting fans were accomplished along the first 1,500 m of the tunnel where 52 of the grouting, or 52%, were related to the two fracture zones NE-3 and NE-1. Of all grouting performed, 43% were carried out as re-grouting. In particular, in the fractures zones the degree of re-grouting was considerable.

The total grouted volume was approximately 267,000 litres where the fracture zones alone consumed 137,000 litres, or 53% of the grouted volume. The mean consumption for each grouting fan was about 2,700 litres but the variation between different grouting fans was however significant. A total of 17,300 m grout holes were drilled, which gives a mean value of 170 m grout hole per grouting fan. The variation between different grouting fans was, as mentioned for the volumes, considerably (cf Table 6-10).

6.6.1 Geological and hydrological conditions of the deformation zones NE-3 and NE-1

The fracture zone NE-3 was observed as a 45 m wide zone composed of a number of sub-zones. The zone was dominated by fine-grained granite with some intersections of Smålands granite and greenstone. The joint spacing was estimated to 5 to 20 cm, but crushed parts were found locally. A joint set with through-going fractures, oriented perpendicular to the tunnel intersected the zone. The major water-bearing structures appeared to have a north-south orientation and a possible contact with the zone NE-1 /Stille et al. 1993/.

The geological character of deformation zone is primarily based on the character of the intersection in the tunnel. The reason is that it is only here the zone has been named NE-1, and it is also here where the fracture sets and width have been determined /Berglund et al. 2003/.

/Rhén et al. 1997c/ give the following description of NE-1 (the location of the deformation zone NE-1 is shown in Figure 6-15).

The zone consists of three branches, which together are about 60 m wide. It is, moreover, related to rock types described as Äspö diorite, fine-grained granite and greenstone. Two planar water-bearing branches were used to approximate the major part of the zone. The northern of these branches is the most intense part of the zone and is approximately 28 m wide in the tunnel and was proved to be highly water bearing. In this northern branch an about 1 m wide central zone was found to be completely clay altered. Outside the central zone an approximately 5–8 m wide, partly clay-altered zone was found, including about 1 cm wide fractures and cavities. Towards the boundaries of the 28 m branch the fracture frequency varies with a distinct boundary to the surrounding rock. The observed fracture geometries in NE-1 were found to be similar in the zone as the average geometries of the Äspö HRL (cf Section 3.4).

A contribution to the complex character of NE-1 was the presence of a set of highly water bearing structures, gently dipping towards North. Approximate division of the transition and core zone of NE-1 used for estimation of the hydraulic conductivities in /Chang et al. 2005/ is described in Figure 6-16.

/Chang et al. 2005/ also present hydraulic conductivities based on data from the passage of deformation zone NE-1 at a depth of about 200 m (Table 6-11).

Table 6-10. Hole length and grout volume at different parts of the tunnel, section 1–1,400 /Stille et al. 1993/.

Volume/length	NE-3	NE-1	Remaining parts of the tunnel
Grouted volume	49,000 l	86,000 l	132,000 l
Drilled hole length	2,820	6,179	8,310
Grouted length of tunnel	31 m	23 m	560 m
Grout volume per metre of tunnel	1,580 l/m	3,780 l/m	240 l/m
Drilled length per metre of tunnel	91 m/m	270 m/m	15 m/m

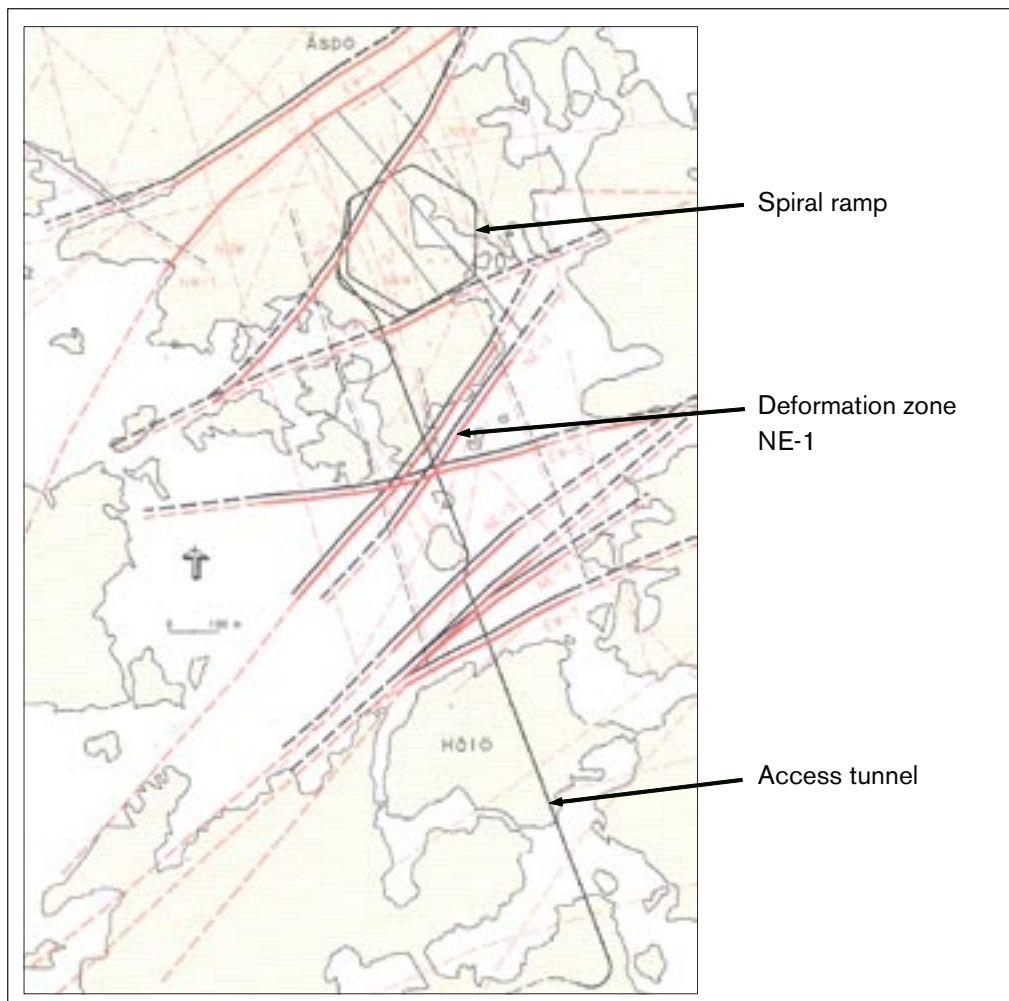


Figure 6-15. Location of deformation zones and ramp system in the Äspö area /Chang et al. 2005/.

Table 6-11. Hydraulic conductivities from the passage of deformation zone NE-1 /Chang et al. 2005/.

	Hydraulic conductivity K (m/s)	
	Transition zone	Core of fracture zone
Min	2×10^{-6}	1.2×10^{-5}
Mean	1.8×10^{-5}	8.8×10^{-5}
Max	4.9×10^{-5}	2.2×10^{-4}

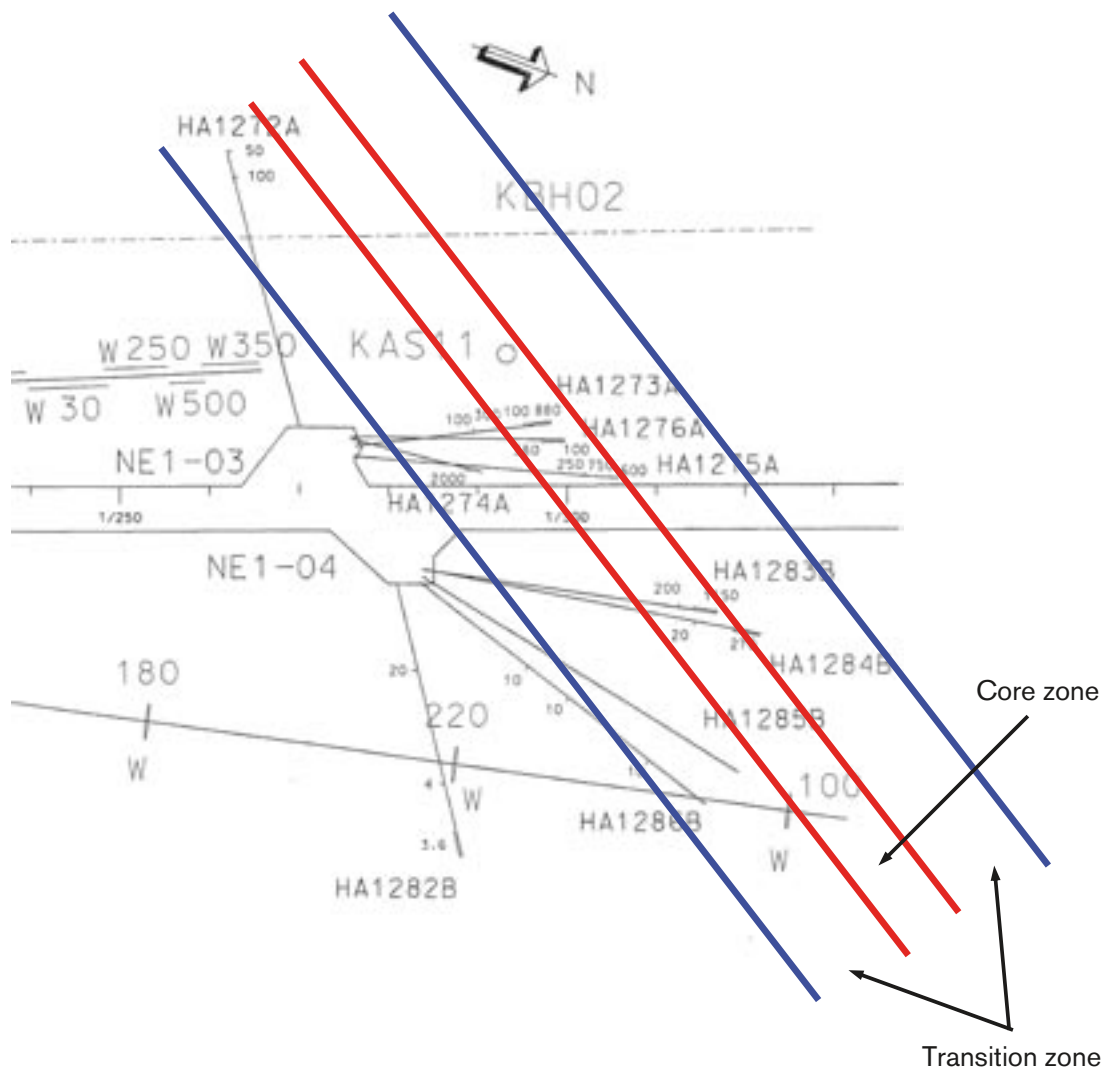


Figure 6-16. Approximate division of the transition and core zone of NE-1 used for estimation of the hydraulic conductivities /Chang et al. 2005/.

/Chang et al. 2005/ argue that the data review for their study suggests that the correlation between hydraulic conductivity and depth is weak, and assumed that the hydraulic conductivity does not vary with depth. Thus, the hydraulic conductivity values as presented in Table 6-11 are, according to /Chang et al. 2005/ assumed to be representative for the depth interval 200–600 m.

The hydraulic conductivity of the rock mass along the first 1,500 m of the tunnel was in general low and in the range of 10^{-8} m/s. The water bearing structures were judged to be associated to the fracture zones and to the N-S, NNW trending fractures.

Transmissivities were recorded in the probe holes drilled every 20 m of the tunnel. Evaluation of the data obtained by interference testing in the probe holes prior to grouting was carried out by /Rhén and Stanfors 1993/. The evaluation of the results is presented in Table 6-12.

The rock quality was in general evaluated as good or fair to good according to the RMR classification system. Poor rock was with some exceptions only observed in connection with the fracture zones NE-3 and NE-1 /Stille et al. 1993/. The RMR-values for the deformation zone NE-1 determined by the geological tunnel mapping are shown in Table 6-13.

Table 6-12. Transmissivity of NE-1 prior to grouting operations /Rhén and Stanfors 1993/.

Transmissivity of NE-1, evaluation based on	Range	Geometric mean
Early time response	4×10^{-5} – 1×10^{-3} m ² /s	4.5×10^{-4} m ² /s
Late time response	8×10^{-5} – 7×10^{-4} m ² /s	4.0×10^{-4} m ² /s

Table 6-13. RMR-values for deformation zone NE-1 determined by the geological tunnel mapping /Markström and Erlström 1996/.

Chainage	Description	RMR
1/285–1/298	Fair	41–60
1/298–1/301	Poor	21–40
1/301–1/303	Very poor	< 21
1/303–1/310	Poor	21–40
1/310–1/320	Fair	41–60

6.6.2 Results from grouting

The water inflow to the grouted tunnel was regularly measured at different sections along the tunnel (cf Table 6-14). The higher values of measured inflow of water for sections 0/672–1/030 and 1/233–1/370 were estimated to depend on the much higher leakage in the two major water bearing zones, NE-3 and NE-1 respectively than the rest of the rock mass. The given average inflow expressed per 100 m of tunnel can be misunderstood if not these local inflows are taken into consideration /Stille et al. 1993/. The total measured inflow to the tunnel between sections 0/672 and 1/370 was approximately 25 l/s corresponding to about 205 l/min and 100 m.

The water inflow from walls and roof were mapped to location, volume and character between section 0/700 and 1/475. The mapping was performed after the grouting work was completed. The mapping proved that the total inflow from the walls and roof was about 7.8 l/s, which represented roughly 35% of the total measured inflow along this part of the tunnel. In correspondence with this, approximately 65% of the inflow to the tunnel must have occurred in the bottom of the tunnel. The result from the mapping also showed that approximately 55% of the observed inflow from walls and roof were related to the two major fracture zones NE-3 and NE-1 /Stille et al. 1993/.

The penetration of the grout was also measured by use of video scope, which made it possible to identify the grout in the fractures but also to observe water inflow into the boreholes and thus giving an identification of the grout penetration /Rhén and Stanfors 1992, Stille et al. 1993/. The achieved penetration of the zones NE-3 and NE-1 was less than the demand for stability. A detailed stability analysis was therefore carried out. The rock support was theoretically slightly overloaded and it was recommended to follow up the stability by convergence measurements and measurements of the water pressure.

Table 6-14. Measured inflow for different sections of the tunnel /Stille et al. 1993/.

Tunnel section m	Measured inflow l/s	Litre min×100 m
0/000–0/672	1.2	10
0/672–1/030	7.8	138
1/030–1/233	3.0	89
1/233–1/370	12.4	543

The effect of grouting on the transmissivity of the deformation zone NE-1 was calculated by measuring water inflow from the grout holes. Only the effects of the first grouting cycle for each tunnel section were evaluated /Chang et al. 2005/. The results of these measurements are shown in Table 6-15. The measured transmissivity in the first fan of NE-1 resulted in a higher transmissivity than the following measurements. According to /Chang et al. 2005/ this is most likely due to the fact that it was the first grouting fan in unaltered rock, and later fans that benefited from the effects of grouting conducted in the earlier fans.

/Chang et al. 2005/ present a graph of the results from flow measurements conducted in the boreholes at various sections of the tunnel after grouting (Figure 6-17). They conclude that a comparison of the transmissivities measured for the entire deformation zone NE-1 prior to grouting, and for the various sections after grouting suggest that the transmissivity of NE-1 is reduced by an order of 10^{-2} m²/s due to the first grout process. Re-grouting of sections that were deemed to be un-sufficiently watertight would, according to the authors, have further reduced the transmissivity /Chang et al. 2005/.

Table 6-15. Median transmissivities after grouting for different sections of NE-1 /Stille et al. 1993/.

Fan (no.)	Section (m)	Median transmissivities (m ² /s)
65	11/290	18.0×10^{-6}
71	1/292	0.3×10^{-6}
75	1/294	1.3×10^{-6}
84	1/297	3.3×10^{-6}
89	1/299	4.0×10^{-6}
91	1/301	4.5×10^{-6}
93	1/306	2.5×10^{-6}

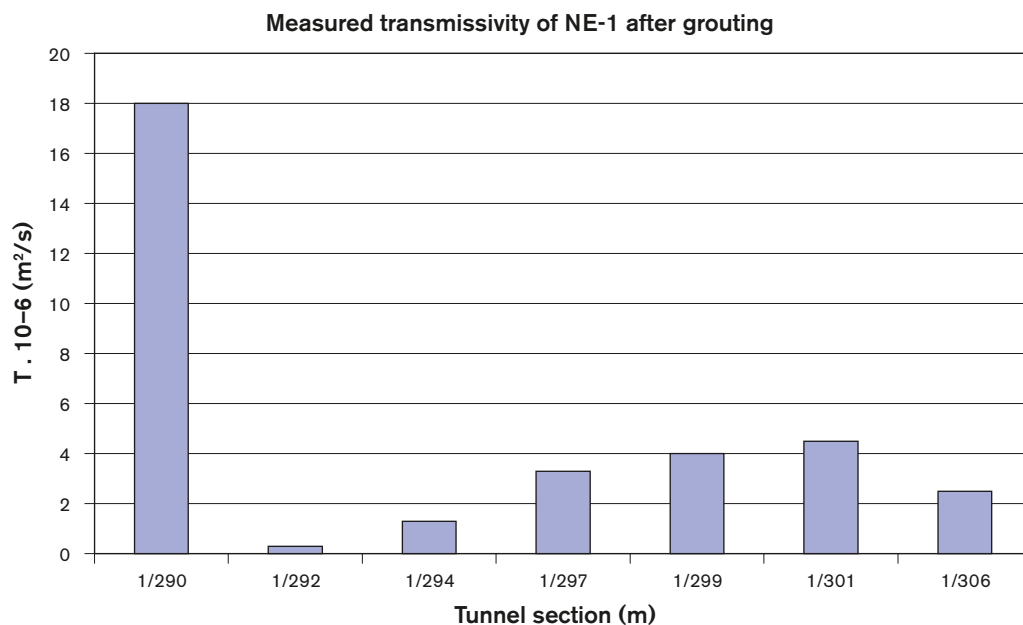


Figure 6-17. Median transmissivity after grouting of NE-1 as measured at each tunnel section during tunnel excavation /Chang et al. 2005/.

6.6.3 Concluded remarks on the grouting activities

The preparations of the grouting campaigns, performing and conclusive aspects on the grouting activities in the Äspö HRL tunnel are comprehensively described in /Stille et al. 1993/ from which some aspects are derived, and presented in this section.

The only demand given in the tender documents was the maximum allowable inflow of 3,000 l/min. The other demands like limited penetration but still with acceptable stability of the rock mass, and a reduced amount re-grouting were set up during the tunnelling work. The different demands were not related to each other.

The demand on maximum allowable inflow of water was not transformed to a demand on maximum allowable transmissivity in the pre-sounding holes to guide the decision of grouting. The criterion on decision of grouting was taken by the engineer in cooperation with the contractor based on their experience; a working method which must be regarded as wise and consistent.

Detailed rules were set up and revised for choosing the grouting methodology by the research group. These rules were found not to be adequate and were gradually simplified and giving the engineer an increasing responsibility.

The orientation and number and length of the boreholes followed normal praxis in Sweden for a grouting fan. At high water leakage it was found to be favourable to make a rough tightening first with a few boreholes before the rest of the fan was drilled and grouted. Long boreholes that penetrated the whole water bearing zones had for NE-1 a very good effect to tightening the zone.

To optimize the grouting work it was judged favourable to be able to vary the shear stress of the grout but also to speed up the hydration of the grout in order to reduce the grout take or pumping time as well as be able to dismantle the packer earlier. The first statement was theoretically analysed with input from the Äspö tunnel and was found to be valid. The second statement was practically observed when the water exceeded about 1.5 MPa and implied development of an accelerated cement grout with calcium chloride.

In order to get refusal and prevent outwash of the grout, the grouting pressure was gradually increased. From a theoretical point of view, it was found that the grouting pressure must exceed the double groundwater pressure to obtain a good refusal and prevent fingering of the grout. This was confirmed by practical observations.

The grouts such as polyurethane (Taccs) and Muring were sparsely used, but they did not have the desired effect and were soon abandoned. Some fans were grouted with high water cement ration. It was found that this type of grout was not stable and also the sealing effect was not specially observed.

The flow properties and hardening time of the grouts were tested both in laboratory and in the field. The test results became very important for a better understanding and development of suitable grouts for the Äspö tunnel.

The following specific conclusions from the grouting work in NE-3 /Stille et al. 1993/ are:

- The problem of front leakage was not foreseen and treated in the programme. Sealing of the tunnel face is very important to be able to carry out a proper grouting fan.
- Water loss measurements seemed to be much better tool for choosing grout type than measured amount of inflow of water.
- The maximum amount of grout takes for a borehole was increased from 80 to 600 litres. The background of the assumed value of 80 litres was an assumption of just a few conducted fractures.

The passage of NE-1 during the construction of the access tunnel for the Äspö Hard Rock Laboratory provided a wealth of information on the process of driving a tunnel through a water-bearing deformation zone. /Chang et al. 2005/ have compiled and summarized the experiences found in the SKB's reports as follows:

- At high water flow, it was found to be expedient to carry out rough sealing first with a few boreholes before the rest of the fan is drilled and grouted /Bäckblom et al. 1994b/.
- Long boreholes that penetrate the whole water-bearing zone were very effective for grouting and enabled the entire deformation zone to be mapped /Stille et al. 1993, Bäckblom et al. 1994b/.
- Both theoretical and practical evidence confirmed that the grouting pressure must exceed twice the water pressure to get a good refusal and prevent fingering of the grout. This tended to cause an additional problem as the packers began to creep out /Bäckblom et al. 1994b/.
- Grout holes injected with high water-cement ratios did not have the desired sealing effect /Stille et al. 1993/.
- The high water pressure increases the demands for a quicker hardening of the grout /Stille et al. 1993/.
- The use of *Stabilo grout* with a mix of cement, bentonite, plasticizer and silicate increased the initial shear strength of the grout when in a *fluid* state, which can shorten the time for removal of the packers /Bäckblom et al. 1994b/.
- The use of calcium chloride as an accelerator improved the sealing effect, however, long-term durability of the grout when using this accelerator is a concern /Bäckblom et al. 1994b/.
- Flushing out the gouge material in the zone could increase the effectiveness of the grouting (jet washing) /Bäckblom et al. 1994b/.
- The tunnel face should have enough distance to the zone in order to have sufficient stability /Rhén and Stanfors 1992/.
- Geophysical measurements in boreholes give valuable supplementary information, especially regarding the orientation of a structure /Rhén and Stanfors 1992/.
- It is very important to pay attention to hydraulic conductors with a possible orientation more or less parallel to the tunnel. Different hydraulic tests are necessary to confirm the orientation and character of identified hydraulic conductors /Rhén and Stanfors 1992/.
- A special drilling arrangement using a special type of casing with a valve arrangement was used to cope with the high water pressures /Rhén and Stanfors 1992/.

6.7 Grouting of the rock mass at depth

In 2003, a grouting field experiment was carried out at Äspö Hard Rock Laboratory in connection with the construction of a tunnel (TASQ) for the Äspö Pillar Stability Experiment (APSE). The tunnel is situated in connection with the elevator shaft landing at 450 m depth and runs in direction NE. The grouting was carried out as part of the ordinary construction work, but was accompanied by extra investigations and analyses during operations and an active adaptation of a basic grouting design to the encountered conditions. A special project "APSE Grouting" was initiated. The characterisation, design and execution of the two grouting fans at 450 m level are described in detail in /Emmelin et al. 2004/.

As described in /Emmelin et al. 2004/, the main objectives of the APSE Grouting project were to:

- Investigate what can be achieved with best available technology, material and knowledge under the current conditions, i.e. a relatively tight crystalline rock mass at great depth.
- Collect data and evaluate theories resulting from previous research projects on characterisation and predictions on grout spread.
- Collect data to further develop those above-mentioned theories.
- Contribute to the achievement of good conditions at the experimental site for the pillar stability experiment.

It should be noted that there was no level set for what would be the accepted inflow after sealing work.

A short summary of the work is given below and for a detailed description of the methodology and results, the Reader of this report is directed to apply to /Emmelin et al. 2004/.

The characterization method was based on analyses of stepwise investigations consisting of investigations in an initially drilled core-drill hole followed by probe and grouting boreholes with pressure-build-up tests and measuring of inflow during drilling, all aiming at identifying the singular fractures that were to be sealed.

The decision about grouting design was based on the successively up-dated rock description from the characterization and iterative selection and testing of grouting design and grout in a numeric model, resulting in an expected grout spread and sealing effect.

Based on investigations and analysis of results from investigations of a core-drilled hole at the site, a basic design was set up, together with conditions for application. Probe boreholes covering the first anticipated fan gave substantially larger inflows than expected, and subsequently the design was changed. A first round was drilled and grouted, sealing off the larger fractures. This was followed by a round, drilled and grouted according to the basic design, taking care of the smaller fractures. The other fan carried out was grouted according to the basic design. The sealing effects in both fans were high and according to calculations.

The application of the coupled methodology for characterisation and design implied that a systematic pre-grouting could be avoided due to a detailed characterisation and that an early assessment could be made concerning what is a suitable grouting methodology.

7 Engineering aspects on the rock mass conditions in the Oskarshamn area

7.1 Introduction

Several authors have reported the extensive site investigations in the area as well as the geoscientific research carried out at the Äspö HRL. These results and the observations made in totally approximately 8 km tunnels down to a maximum depth of 450 m make the rock mass conditions one of the best understood in Sweden, at least outside mining areas. The Simpevarp peninsula and the southern part of the Äspö Island that cover the area for underground excavation works is approximately 5 km² (cf Figure 7-1). The most comprehensive description of engineering experiences from the underground works is given by /Larsson and Leijon 1999/. Further information on the mechanical behaviour of the rock mass at the Äspö HRL is evaluated by /Andersson and Söderhäll 2001/.

In addition, the ongoing site investigations for a final repository have given more information to the regional geological conditions, including tectonic development, and show a structural difference along the coastal area with the Simpevarp peninsula and the archipelago towards north compared to the main inland called the Laxemar area /SKB 2006a/. The divider seems to be the so-called Äspö shear zone /Rhén et al. 1997b/, later named ZNSMNE005A in the site investigations /SKB 2006a/. This structure has been described as a several 100 m wide, vertically dipping structure with a ductile and complex nature. Later brittle deformations intersect the structure. Results from the ongoing site investigations show a significant difference in the dominant orientations of lineaments west respectively east of the Äspö shear zone, Figure 7-2.

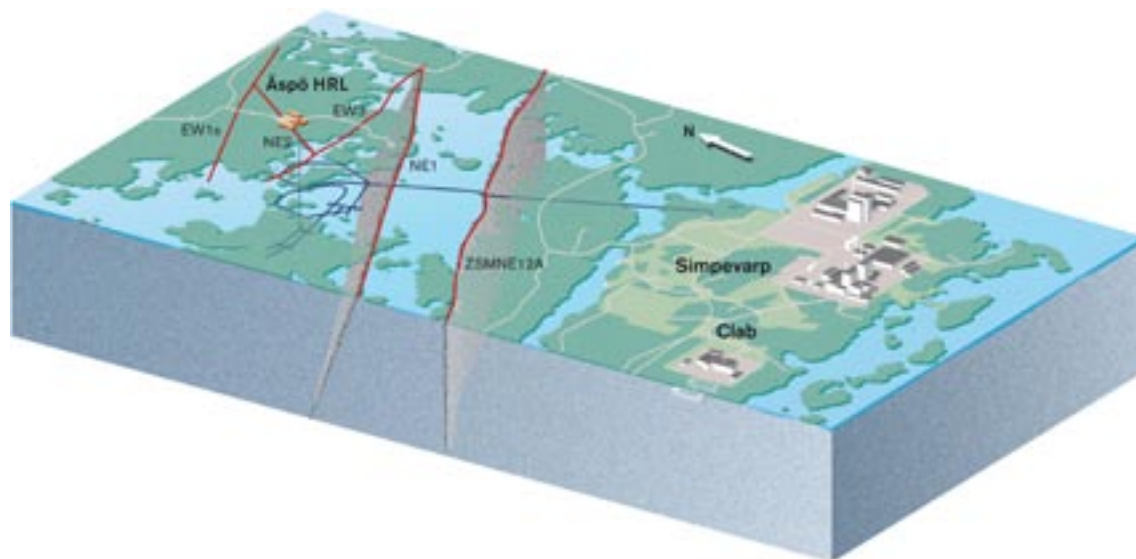


Figure 7-1. Overview of the locations of underground excavations and general structural features.

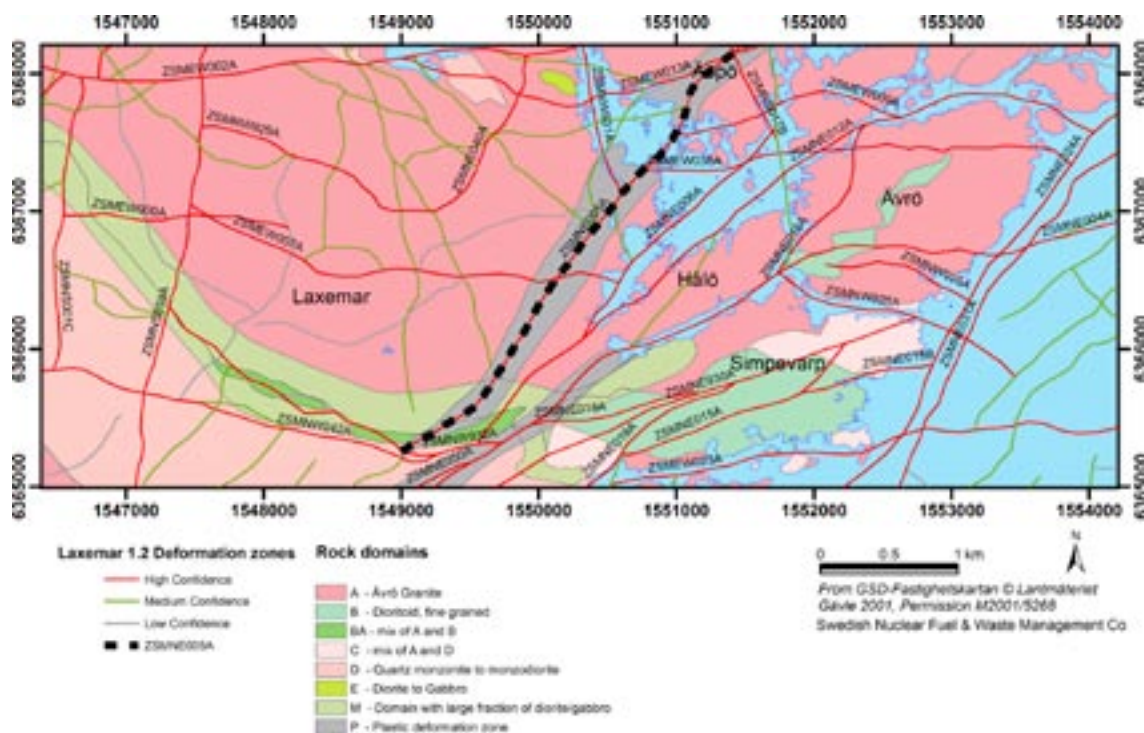


Figure 7-2. The Äspö shear zone and lineament orientations west and east of the structure /SKB 2006a/.

There is an uncertainty if the indicated large-scale structural difference on each site of the Äspö shear zone has affected the rock engineering conditions differently between the Simpevarp peninsula and the Äspö HRL. A difference in fracture orientation at depth for the Äspö HRL is reported (see Figure 7-3), but on the other hand, the upper 200 m of the access tunnel to the Äspö HRL is located south of the Äspö shear zone (NE-1 in Figure 7-1). The general notion that rock conditions may differ between the more superficial rock mass and deeper seated rock can not be evaluated in this report due to the lack of shallow excavations at the Äspö Island and lack of deep excavations at the Simpevarp peninsula.

7.2 General engineering description of the rock mass

7.2.1 Lithology

The different rock types described in Section 3.1 and Figure 3-2 are normally homogenous and show only local minor foliation. The dominant different rock types of 1,810–1,760 Ma are typically mingled into each other and show mechanically very competent rock boundaries. However, the dykes of younger fine-grained granite display a more distinct contact to the older host rock. This contact is sometimes less well sealed.

Minor hydrothermal alteration is found around both ductile and some brittle structures. Although limited in volume around structures, the total amount of these alterations is estimated to be some 15–20% of the total volume of rock. The ongoing site investigations have shown that the strength of the different rock types may vary due to different rock types and degree of alteration. However, the reported observations from the tunnelling projects in the area do not report any significant mechanical problem with any of the rock types, unless alteration occurs in fractured and water-bearing rock.

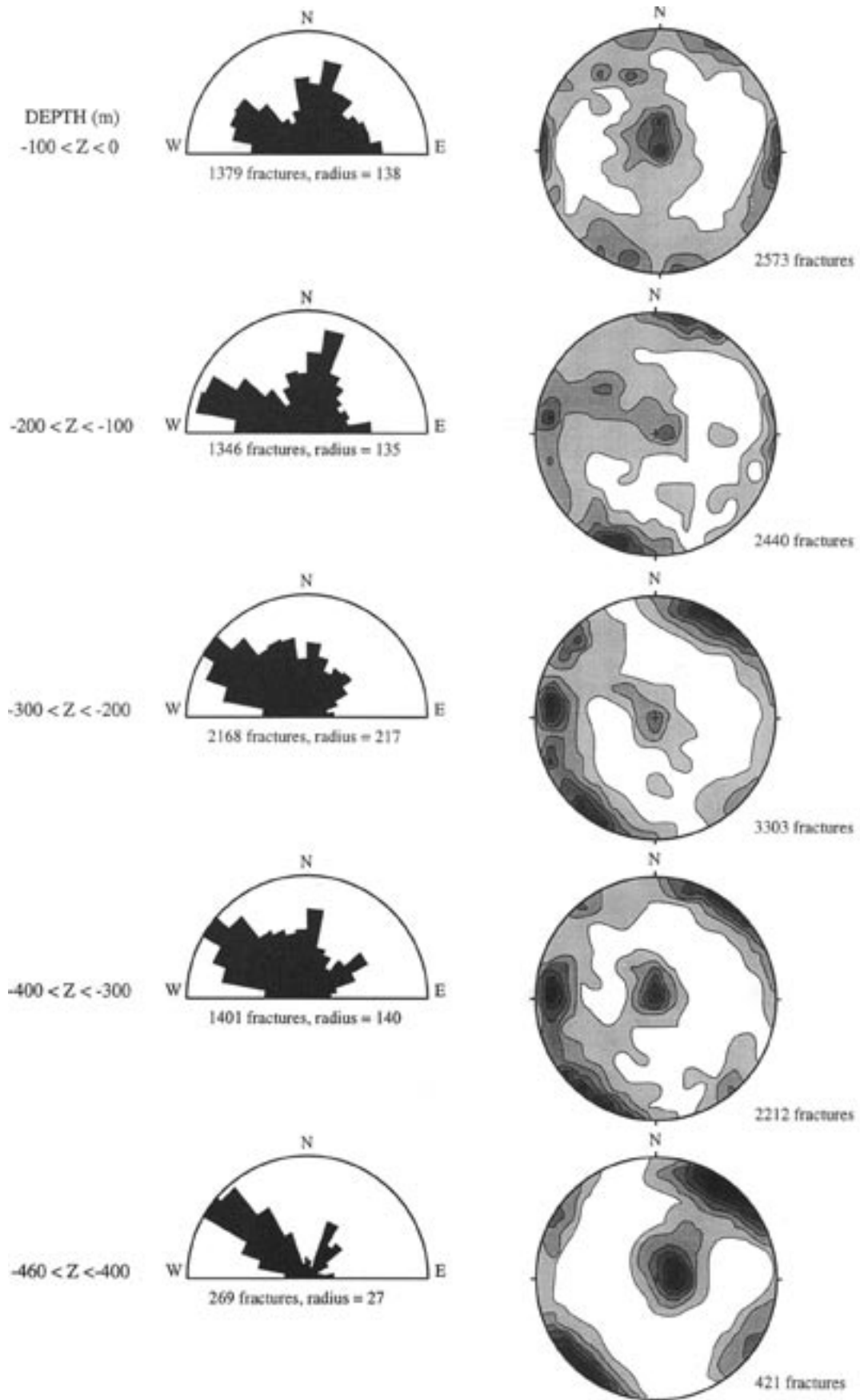


Figure 7-3. Fracture orientations in 100 m depth intervals for the Äspö HRL /Rhén et al. 1997c/.

7.2.2 Fracture distribution

A general illustration of fracture distribution is shown in Figures 3-7 and 6-13. In addition, the larger structures within the different construction areas are often trending NE (cf Figures 3-8 and 7-2). These structures have a ductile origin, but are frequently reactivated and show brittle deformation as well. A well-described example is the deformation zone NE-2 at Äspö /Rhén et al. 1997c, Berglund et al. 2003/. This structure is roughly 5 m wide at the surface but decreases in thickness, alteration and fracturing at depth. Down at the 450 m level, it is interpreted that NE-2 terminates into some splays in terms of minor occurrence of mylonite with red stain (weak alteration) and no brittle fracturing. The steeply dipping NE trending fracture set is infrequently persistence outside the minor deformation zones and sealed with chlorite precipitation. This set shows rarely any sign of leakage outside deformation zones at depth in the Äspö HRL, but gives significant over-break along the fracture surface if the tunnel wall is aligned close to the trend of the fracture. The steeply dipping NE fracture set occurs seldom as swarms in the Äspö HRL outside the ductile deformation zones, but could be found as clusters in Clab (cf Figures 3-4 and 3-5). The brittle fracturing within the ductile deformations zones is however more frequent, and often water bearing. An alternative location for water-bearing dense fracturing in NE is associated to dykes of younger granites. A simplified illustration of water bearing structures is given in Figure 7-4.

The steeply dipping fracture set trending WNW-NW is the dominant water-bearing fracture set, especially at the Äspö HRL. This fracturing occurs normally as narrow bands with sub-parallel fractures. The fracturing displays a pattern with significant splay of sub-parallel fractures (Figure 7-5). A scan line across such a cluster could show fracture frequency in the range of one to tenth of fractures within a width of less than a metre. The fracture orientation (trend and dip) can vary locally within at least $\pm 20^\circ$ due to the large undulation. Locally, this may give an indication of a bi-modal orientation of the fracture set (cf Block 8 in Figure 3-11).

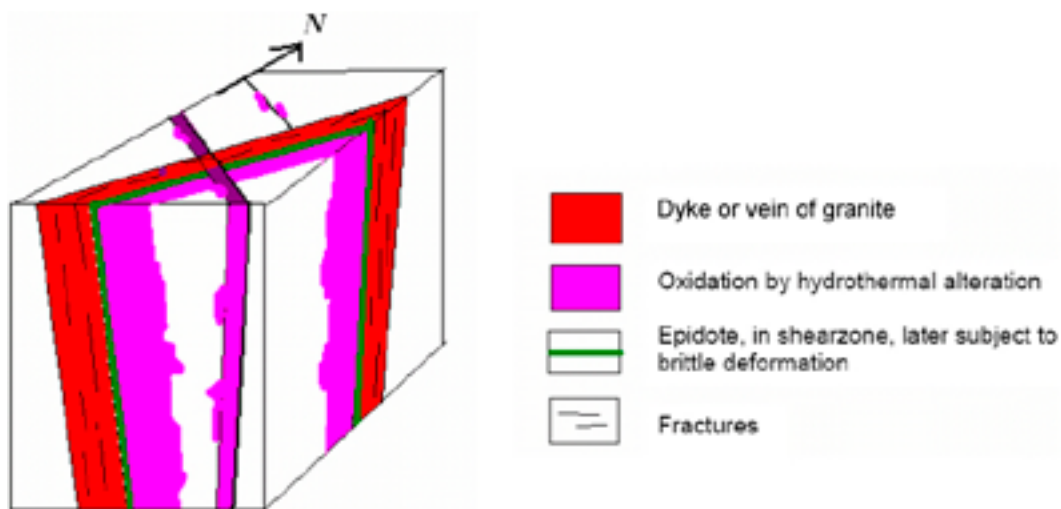


Figure 7-4. Simplified illustration of the water bearing structures in the area.

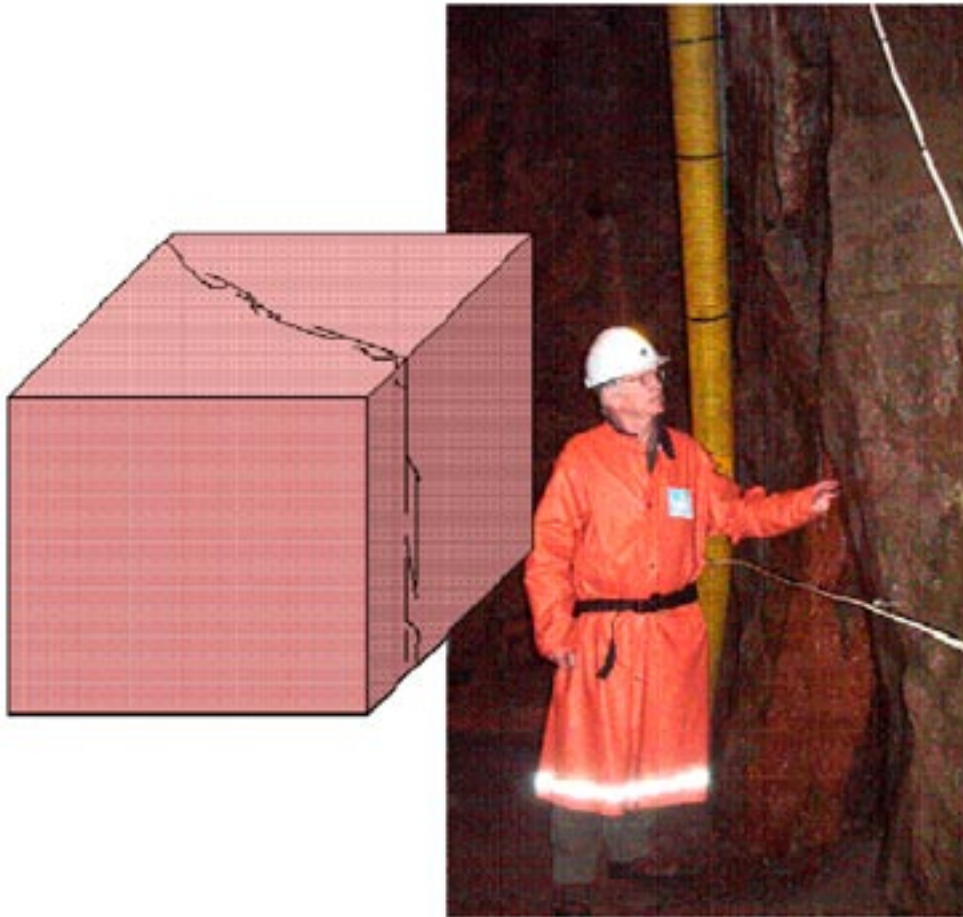


Figure 7-5. Photo along a NW-SE trending, steeply dipping fracture at the 450 m level of the Äspö HRL. The surface closest to the yellow pipe is also a fracture surface. The red stain at the lower part is caused by precipitation of iron oxide. The block diagram to the left illustrate the splay structure.

7.3 Description of the heterogeneity of the rock mass, block size 10×10×10 m

7.3.1 Rock class 1

Sparsely fractured rock. All dominant joint sets may occur, but seldom as significant clusters, Figure 7-6. The occurrence of hydrothermal alteration is very minor along few fractures. The fractures are normally well sealed with precipitation. Running or dripping water may occur, although the fractures are normally sealed with precipitation.

Rock support: Bolts: none, spot bolting or patten bolting 1 bolt/4 m².
Shotcrete: none to 50 mm un-reinforced.

The lower level of rock support is frequently found at depth in the Äspö HRL. The demand of allowing geoscientific studies after construction leads to the approach to minimize the use of shotcrete. Shotcrete is frequently replaced by wire mesh at the HRL. The maintenance records in terms of scaling frequency indicate stable conditions of the un-supported section in rock class 1.

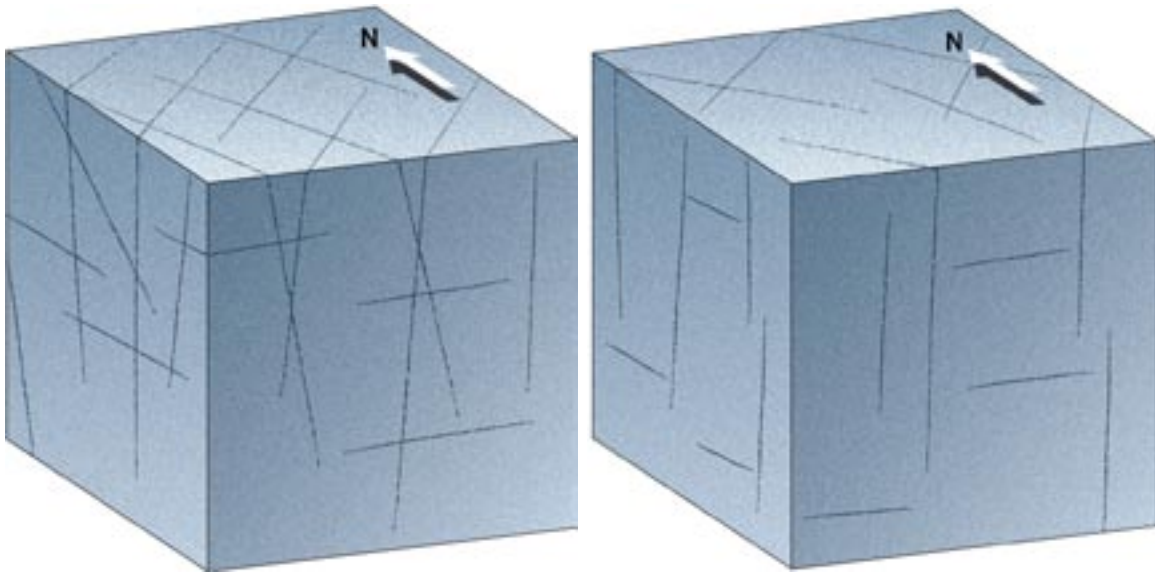


Figure 7-6. Sparsely fractured rock at Clab (left) and Äspö HRL.

7.3.2 Rock class 2

Clusters/minor deformation zones caused by the steeply dipping WNW-NW trending fracture set. Hydrothermal alteration of the fracture surfaces is insignificant. The fracture transmissivity can vary over large ranges. Mechanical strength of mineral precipitation on the fracture surfaces varies. The width of this structure is normally very limited (< 1–2 m). The impact on construction works is related to the orientation of tunnel walls relative to the structure, as well as the local hydraulic properties. Significant high flow has been experienced at depth. The hydraulic connectivity seems to be up to at least 200 m at depth in the Äspö HRL. Fracture precipitation can open up if penetrated by a borehole that intersects with a hydraulically open fracture further away, causing mechanical instability. Similar rock conditions at Clab, but less transmissive fractures seem to be experienced in steeply dipping NE-SW trending fractures. An illustration is given in Figure 7-7.

Rock support: Bolts: pattern bolting 1 bolt/2–4 m².

Shotcrete: 50 mm un-reinforced to 100 mm fibre reinforced.

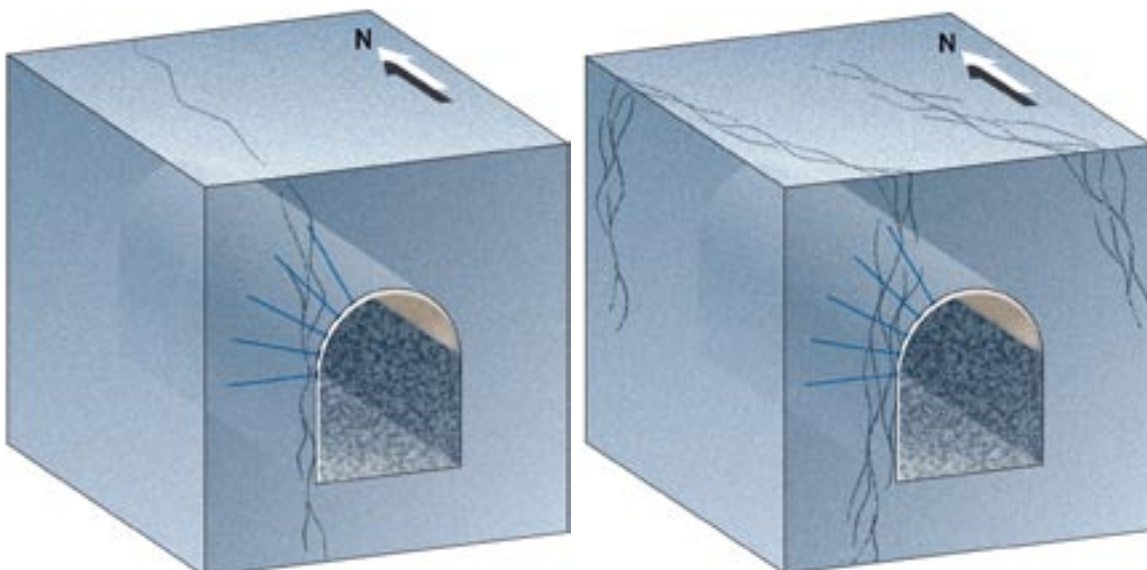


Figure 7-7. A competent rock mass cross-cut by steeply dipping fractures at Clab (left) and the Äspö HRL.

Experience at the Äspö HRL shows that the lower level of shotcrete support can be replaced by wire mesh anchored with coated Swellex, spacing 1 bolt/2 m². This level of rock support has also been applied in the shallow parts of accesses to the cavern at the Simpevarp peninsula.

7.3.3 Rock class 3

NE trending and steeply dipping ductile deformation zones, reactivated with brittle fracturing. There might be differences in properties if such structure is formed along a dyke of granite or not, Figure 7-8. The hydrothermal alteration may be locally significant. Sub-parallel splays can be expected, causing an uncertainty in the width of such structures. The width of this kind of minor deformation zone is however seldom more than 5 m (excluding splays). The brittle fractures are often open to some degree; local variation in conductivity can be expected.

Rock support: Bolts: pattern bolting 1 bolt/2–4 m².
Shotcrete: 50 mm un-reinforced to 100 mm fibre reinforced.

The highest level of support is applied in the Äspö HRL in section when similar conditions occur over several rounds.

7.3.4 Rock class 4

Rock class 4 refers to the experiences gained from the major deformation zone NE-1 (Section 6.6.1). Details are given in several references to this report, see for example /Chang et al. 2005/. The core zone was the most problematic. However, characterisation works show that at least for this actual structure the width and properties of the core may vary over short distances, and in particular, the amount of clay and transmissivity may vary considerably.

Rock support: Bolts: pattern bolting 1 bolt/2–4 m².
Shotcrete: 50 mm to 200 mm fibre reinforced.

Pre-bolting of the arch (spiling) was carried out for passage of the deformation zone NE-1. Plans and strategy for characterization and pre-grouting are fundamental measures before tunnelling into rock class 4.

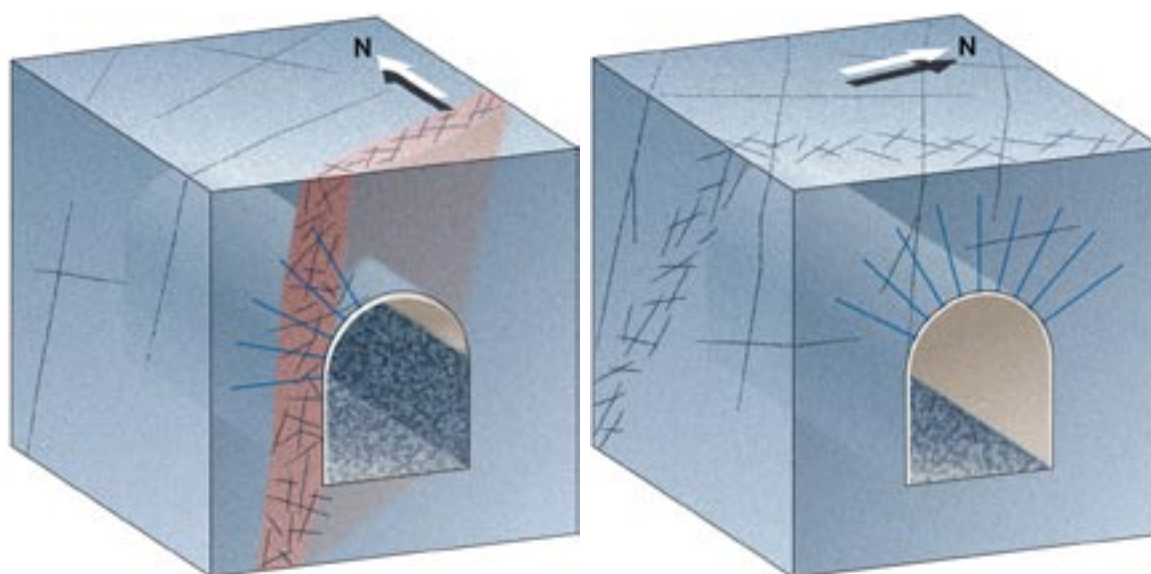


Figure 7-8. Minor steeply dipping deformation zones trending NE-SW. The block to the left is associated to a granitic dyke.

7.4 Influence of geological structures on tunnelling

Except for problems that can be expected in rock class 4, there are very limited problems reported from tunnelling in the Oskarshamn area. Early experiences of local variability and the minor impact on constructability gained from the power plants have been proven frequently in different project in the area. This is also verified by the prediction outcome for the Äspö HRL (see Section 6.2.3). The major concern for rock engineering outside deformation zones is the hydraulic conditions and the need for pre-grouting in project with demands for high sealing levels.

Some illustrations related to tunnelling and specific geological features as experienced during underground works in the Oskarshamn area are given below.

7.4.1 Steeply dipping structures

Both the WNW-NW and NE trending steeply dipping structures cause significant over-break and need for bolt support when the tunnel wall is in a small angle to these structures. A relatively higher degree of support can be expected for the NE trending structures that display various degree of alteration. The NE trending deformation zones may require significant concern (rock class 3).

7.4.2 Gently dipping fractures

Gently dipping fractures caused frequent over-break in the superficial rock during the first 2–300 m of the access tunnel to the Äspö HRL. No such problems have been experienced deeper in the HRL or in Clab.

7.4.3 Distribution of water-bearing fractures

The WNW-NW steeply dipping fracture set and the ductile NE trending minor deformation zones are the major water-bearing structures in the area. Especially the former set has displayed locally high transmissivities, causing large local inflow at depth in the Äspö HRL.

Observations in the Äspö HRL show that water inflow often is related to channelling flow. Small flow in terms of spot of moisture can be found also in gently dipping fractures. At depth, the length of the gently dipping fractures is seldom more than some metres. These fractures terminate normally towards steeply dipping fractures.

7.4.4 Stress conditions

There are no experiences of high stresses at the Oskarshamn area. However, unsuitable geometry at intersecting tunnels at depth in the Äspö HRL shows locally unfavourable stress concentrations /Andersson and Söderhäll 2001/. Such minor problems shall be considered in the design.

8 Maintenance experiences from the operation of Äspö Hard Rock Laboratory

8.1 Introduction

The operating phase of the laboratory started in 1995. Documentation of maintenance measures and other related activities is summarized and discussed in /Andersson and Söderhäll 2001/. Activity records from the construction and operating phases have been stored in both ordinary archive and digitally in the Site Characterisation Database (SICADA).

The report of /Andersson and Söderhäll 2001/ focuses on the stability of the existing tunnels and niches at Äspö HRL, and one of the objectives was to correlate maintenance with rock mechanics factors such as stress concentrations, tensile stress zones, fracture frequency, tunnel orientation relative to in situ stress orientation. The records could be used to locate parts of the tunnels that have required more maintenance than average.

As described in Section 6.5 (Rock support), the rock support philosophy for the underground structures was partially based on the requirements set by the research and scientific staff in their request to avoid or significantly minimize disturbances on the experiments; i.e. to use as little shotcrete as possible, or preferably use wire mesh instead of shotcrete, and to completely avoid grouting.

In reality, conventional rock support measures were applied, not least considering safety aspects. However, there is no doubt that the contractor during the excavation adapted his rock support work, to the utmost possible extent, to the above mentioned requirements without waiving the safety aspects or lower his standards of quality. It should be emphasised that the contractor's operational support works in the tunnel was carried out after due consultations with SKB. Rock bolts, shotcrete with and without steel fibres and shotcrete arches were used for support, and for grouting cement and polyurethane based grouts were used.

The summarized descriptions given below on maintenance and other remedial measures carried out in the Äspö HRL are derived from /Andersson and Söderhäll 2001/. The numerical modeling, which was carried out by the authors and comprehensively presented in their work is not treated in this compilation report.

8.2 Rock bolting, scaling and shotcreting

/Andersson and Söderhäll 2001/ used two different methods to isolate areas in the tunnels where more rock maintenance than average had been carried out: interviewing one of the miners performing scaling and rock bolting (since autumn 1998), and secondly studying the records for bolting and shotcreting.

The location of areas, defined by the miner, which needed repeated maintenance, is illustrated in Figure 8-1. The maintenance comprised primarily of frequent scaling, bolting and partly fibre-reinforced shotcrete. Noise from micro-seismic events was occasionally heard during scaling.

/Andersson and Söderhäll 2001/ used three different parameters to investigate the long-term behaviour of the main tunnel, namely scaling, shotcreting and bolting. The parameters were sorted into nine different groups of which each group represented a leg of the hexagonal spiral down to approximately 390 m. The authors used only data from below a depth of about 200 m and downwards due to the assessment that the rock stresses above this level were too small to create stress related problems. The location of the nine different legs is shown in Figure 8-2, and the chainage for the legs is shown in Table 8-1.

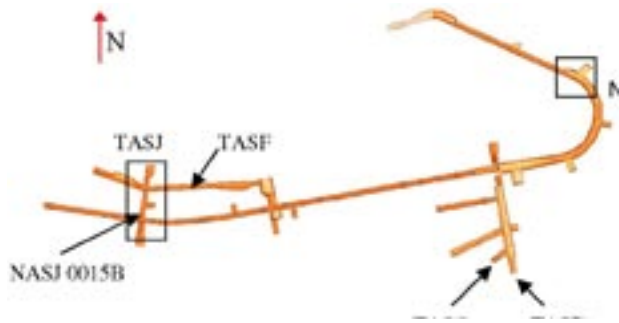


Figure 8-1. Areas in the tunnels needing additional maintenance according to interview /Andersson and Söderhäll 2001/.

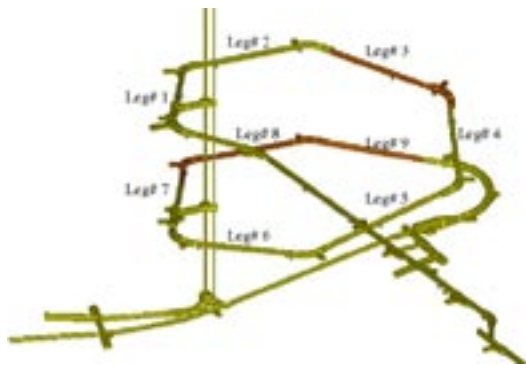


Figure 8-2. The nine different legs and the parts of the tunnel classified as fair rock (RMR 40–60) is shown in red colour. The rest of the tunnel within the legs is classified as good rock (RMR 60–80) /Andersson and Söderhäll 2001/.

Table 8-1. Chainage for the different legs presented in Figure 8-2 /Andersson and Söderhäll 2001/.

Leg #	From section (m)	To section (m)	Angle relative σ_1 trending 133°
1	1,584	1,745	65
2	1,745	1,883	60
3	1,883	2,028	0
4	2,028	2,178	55
5	2,178	2,357	60
6	2,357	2,496	0
7	2,496	2,699	65
8	2,699	2,840	60
9	2,840	2,997	20

/Andersson and Söderhäll 2001/ made a compilation of scaling, bolting and shotcreting records showing the relation between the three parameters together with the leg's angle to the major principal stress (Figure 8-3). The comments to the compilation made by the authors are summarized in Table 8-2.

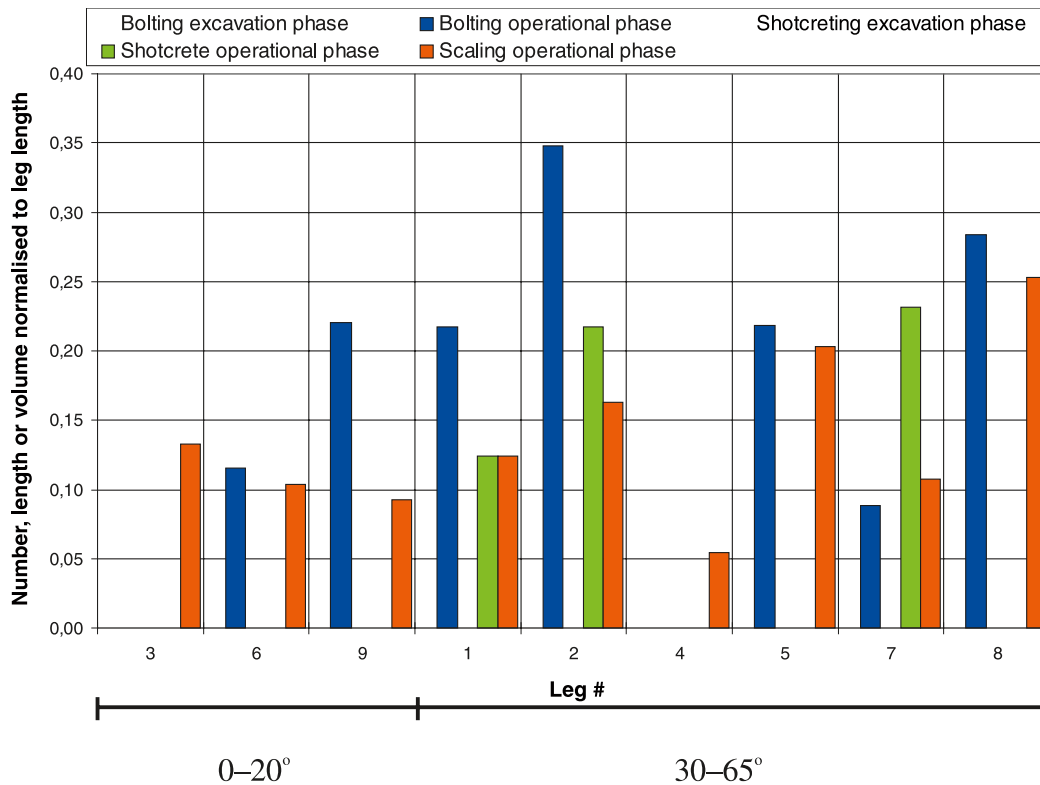


Figure 8-3. Compilation of records for scaling, bolting and shotcreting for the different tunnel legs and their relation to the major principal stress /Andersson and Söderhäll 2001/.

Table 8-2. Summary of comments made by /Andersson and Söderhäll 2001/ on the compilation of records for scaling, bolting and shotcreting (cf Figure 8-3).

Leg #	Comments
1	<i>Bolting, shotcreting:</i> 1,620–1,680 m. <i>Scaling:</i> 1,650–1,680 m. <i>Geology:</i> mixture of Äspö diorite, greenstone, fine-grained granite and some pegmatite
2	<i>Bolting, shotcreting:</i> 1,750–1,780 m. <i>Geology:</i> Äspö diorite with a thin mylonite zone in tunnel roof. <i>Scaling:</i> 1,820–1,883 m (65%). <i>Geology:</i> Småland granite with veins of fine-grained granite (50%), Äspö diorite with veins of fine-grained granite (50%)
3	<i>Bolting:</i> in fracture zone. <i>Scaling:</i> evenly distributed along the leg. <i>Geology:</i> Contact between Småland granite/fine-grained granite, Äspö diorite
4	<i>Shotcreting:</i> 2,120–2,178 m. <i>Scaling:</i> evenly distributed along the leg. <i>Geology:</i> Småland granite, Äspö diorite with veins of fine-grained granite
5	<i>Bolting:</i> 2,325–2,355 m. <i>Shotcreting:</i> 4 m. <i>Scaling:</i> evenly distributed but mainly around fine-grained granite. <i>Geology:</i> Äspö diorite with veins of fine-grained granite
6	<i>Bolting:</i> 2,360–2,380 m. <i>Scaling:</i> evenly distributed with some increase at curve ends and when tunnel is narrowed. <i>Geology:</i> Äspö diorite with veins of fine-grained granite
7	<i>Bolting, shotcreting:</i> 2,570–2,610 m. <i>Scaling:</i> 2,490–2,510 m, 2,621–2,660 m. <i>Geology:</i> Äspö diorite, fine-grained granite, Småland granite
8	<i>Bolting:</i> 2,710–2,735 m. <i>Scaling:</i> evenly distributed. <i>Geology:</i> increased fracturing, one minor fracture zone, Småland granite
9	<i>Bolting:</i> 2,860–2,870 m, 2,900–2,940 m. <i>Scaling:</i> 2,860–2,900 m, 2,970–3,000 m. <i>Geology:</i> Äspö diorite, fine-grained granite (2,860–2,870 m), Småland granite with larger intrusion of fine-grained granite in roof (2,900–2,940 m), 10-m fracture zone, Småland granite (2,970–3,000 m)

The maintenance records show that almost all of the bolting has been performed in the curves where the tunnel is widened. This concentration of bolting could be a consequence of increased fracturing due to the blasting when widening the tunnel. However, the records indicate that increased scaling only had to be performed in two of the curves /Andersson and Söderhäll 2001/. Furthermore, the authors assume that the additional bolting in the other curves could therefore partly have been made of psychological reasons because of the wider tunnel and perhaps more fractured surface than the actual need for maintenance. The tunnel contour in the outside of the curves is rougher than the contour in the straight part of the ramp due to the drill and blast and might therefore look somewhat more unstable.

Another conclusion made by /Andersson and Söderhäll 2001/ is that all bolting, shotcreting and scaling is located either in Äspö diorite areas with intrusions of fine-grained granite and larger areas with Småland granite. Increased fracturing or fracture zones often occur in the fine-grained granite and Småland granite. If the bolting in the fracture zone area in leg #9 is excluded there is hardly no maintenance in the tunnel legs almost parallel to the bearing of the major principal stress.

8.3 Water inflow measurements

In addition to geology, /Andersson and Söderhäll 2001/ studied water inflow records, and Figure 8-4 shows the relation between scaling and water inflow together with the leg's angle to the major principal stress. As can be seen, four of the legs have a significant larger inflow compared to the other legs and one leg has a much lower inflow. Leg #9 that is almost parallel to the major stress bearing has the highest water inflow of all the legs.

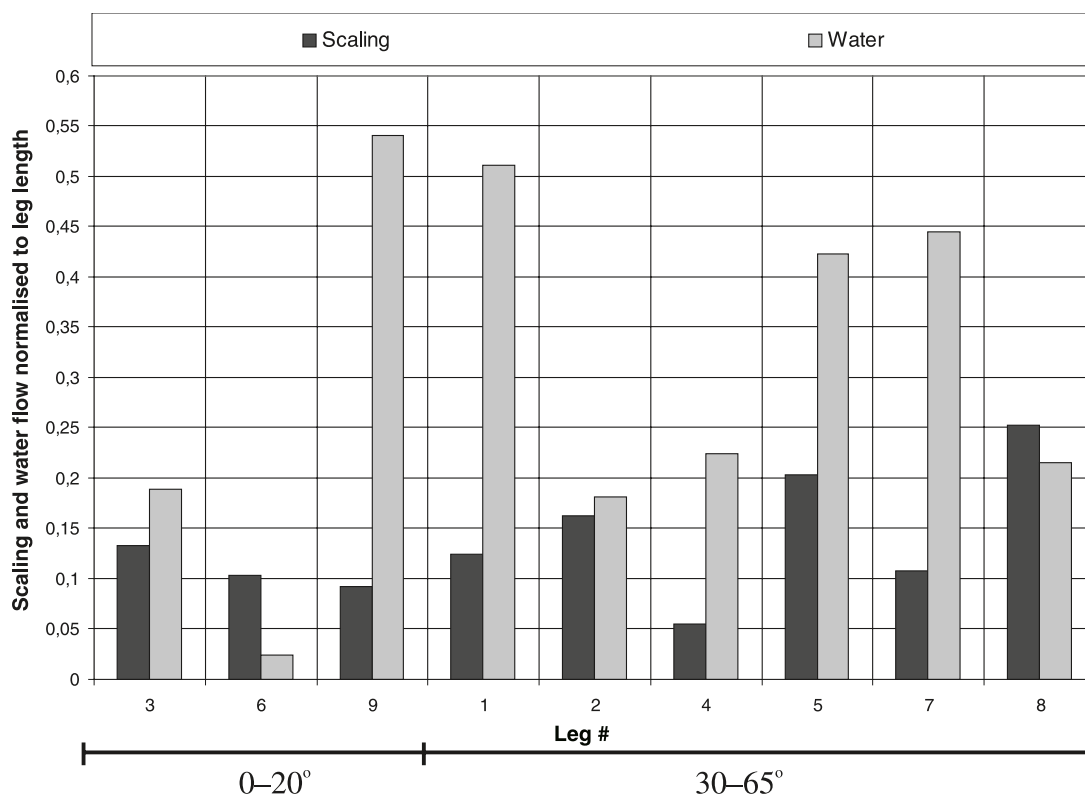


Figure 8-4. Compilation of records for scaling and water inflow for the different tunnel legs and their relation to the major principal stress /Andersson and Söderhäll 2001/.

The total water inflow to the tunnel from January 1991 to December 2004 is presented in Figure 6-5, and as is plain from the figure, the water inflow has decreased continuously since January 1995. When /Andersson and Söderhäll 2001/ studied the water inflow records for the operational phase (1995–2000) they concluded that there was a remarkably uniform decrease in almost all of the weirs (cf Figure 8-5). The decrease in water inflow from 1995 to 2000 was 22% or 4.4% per year.

8.4 Concluded remarks

Some of the conclusions drawn by /Andersson and Söderhäll 2001/ are:

- The study of the maintenance records indicates that the reason for the additional maintenance in different parts of the main tunnel is due to geological factors.
- The scaling, bolting and shotcreting are located in areas with intrusions or veins of fine-grained granite or Småland granite. Increased fracturing and fracture zones are typically associated to these two rock types.
- The support and scaling performed in the niches and side tunnels are most certainly caused by stress concentrations created by the choice of geometry.
- There is an indication that the tunnel alignment to the stress field has an impact on the need for maintenance.
- The comparison of a horseshoe shaped D&B tunnel with a TBM tunnel regarding the need for maintenance shows that the drill and blast tunnel requires more resources in that aspect.
- It is likely that the stability of the tunnel system would decrease if the tunnel were located at 500 m depth. The geometry is therefore of most importance when designing a facility at these depths in this kind of rock.

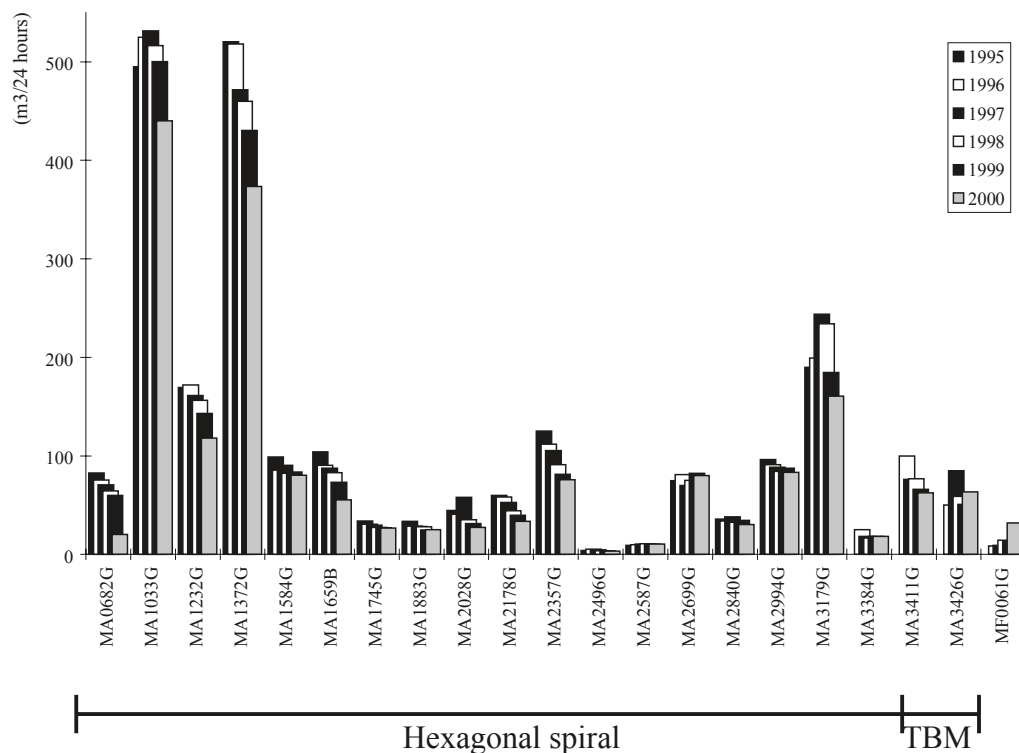


Figure 8-5. Average water flow through the weirs during five years. The measurements were made from October to December each year. The numbers MA0682 etc indicate the chainage /Andersson and Söderhäll 2001/.

9 Tunnelling experience at Oskarshamn

9.1 Overall conclusions

Approximately 8,000 m of tunnels including three major rock caverns with a total volume of about 550,000 m³ have been excavated. The excavation works of the various tunnels and rock caverns were carried out during the period of 1966–2000. In addition, minor excavation works were carried out at the Äspö HRL in 2003. The depth location of the underground structures varies from near surface down to 450 m.

As an overall conclusion it may be said that the rock mass conditions in the area are well suited for underground construction. This conclusion is supported by the experiences from the rock excavation works in the Simpevarp and Äspö area. These works have shown that no major problems occurred during the excavation works; nor have any stability or other rock engineering problems of significance been identified after the commissioning of the Oskarshamn nuclear power units O1, O2 and O3, BFA, Clab 1 and 2, and Äspö Hard Rock Laboratory. The underground structures of these facilities were built according to plan, and since then have been operated as planned. Thus, the quality of the rock mass within the construction area is such that it lends itself to excavation of large rock caverns with a minimum of rock support.

9.2 Tunnel driving

In rock construction, it is a well known fact that serious disturbances can occur not only in conjunction with major deformation zones of inferior rock, but also in conjunction with excavation through minor zones with unfavourable rock conditions. The geological environment and thus, the geological conditions are never uniform, and great differences may exist also between adjacent locations with similar geological conditions.

Consequently, the planning and execution of a tunnelling project must account for this variability, where also the unexpected is to be expected. The variation in properties may apply to hydraulic properties, variations in magnitude and orientation of rock stresses, changes in fracture frequency and fracture orientation, degree of alteration, etc. A single unfavourable factor does normally not result in a serious hazard, but a combination of several factors may result in a difficult disturbance to the tunnelling.

During construction work, the rock is encountered just as it is which may involve unwelcome surprises, but, just as easily, positive ones, in that the quality of the rock mass may be found to be considerably better than was forecasted or expected. Examples of both these cases were experienced in the Oskarshamn tunnels.

For instance, the conceptual rock support design of a shear zone located in the centre of the Clab 2 rock cavern, and identified from site investigation boreholes, had to be adjusted during the excavation of the cavern. The change in orientation and splay of the shear zone was not predicted wherefore adjustment of the rock support was made. Furthermore, the grouting methodology was significantly modified during construction since the rock mass turned out to be considerably tighter than predicted. The use of the Observational Method throughout the underground excavation of Clab proved to be successful adopting satisfactory technical solutions in a timely and costly acceptable manner.

There are no documentation available, which reports any particular difficulties due to rock conditions encountered during the tunnelling work for the Oskarshamn nuclear power units.

The most demanding undertaking, from a rock-engineering point of view, was perhaps the excavation work for the cooling water intake of O3. The work comprised rock excavation underneath a 5 to 6 m thick rock plug and a water head of 20 m. However, the excavation work was achieved without any special difficulty due to careful planning of remedial measures and the actual performance of the work.

The underground excavations of the Clab facility were carried out without any particular rock engineering difficulties. Except for pre-grouting, the rock support comprised spot and pattern bolting, reinforced and un-reinforced shotcrete and reinforced shotcrete arches at crossings between storage caverns and the transept.

A comprehensive rock inspection of the underground Clab facility was carried out in 2004 comprising inspection of the opening gaps between concrete and rock walls in the caverns and the auxiliary building, transport tunnel, and the opening gap above the ceiling in the caverns.

For each of the inspected elements in Clab 1 and 2, the inspection report concludes concordantly that: *“No observations of damages or other conditions of importance for the safety of the facility have been recorded.”*

In conclusion, the measurements and monitoring performed during the operation phase and the 2004 rock inspection of the Clab 1 and 2 underground facilities do not indicate any damages, deterioration or failure of installed rock support, nor any stability problems or unexpected deformation. No problems related to groundwater have been identified.

In general, the experiences gained from the tunnelling of the Äspö Hard Rock Laboratory have shown that no notably rock engineering problems occurred during the execution of the works. The stability conditions during the underground excavation works and during the present operational stage have been favourable. The rock support has mainly comprised spot bolting to secure rock blocks in the roof. Rock support could be omitted in large parts of the tunnel, and more significant rock support measures were only used in connection with larger fracture/deformation zones. Rock spalling was not observed during the tunnelling. Occasionally, click noise was heard after blasting, and minor tendencies of extraction of intact rock were noted.

The saline water, however, created problems for the construction equipment as well as the permanent installations. This implied high demands on daily maintenance on e.g. drilling rigs.

The tunnel down to the spiral intersected a number of fracture zones, and the passages of the deformation zones NE-3 and NE-1 were in particular time consuming due to high water inflows resulting in comprehensive investigations and grouting works. At NE-3 the excavation was in contact with a site investigation borehole running parallel with the tunnel roof. This together with unfavourably fractures zones motivated a minor displacement laterally, relatively to planned tunnel alignment.

The major difficulty experienced during the passage of NE-1 was related to the grouting and drilling operations due to the fractured nature of the rock mass and the relatively high water pressure. Initially grouting was carried out with restrictions on the maximum volume of grout that was to be pumped into each hole and the injection pressure in order to limit the extent of the grouted zone. In addition, the initial concept was based on relatively short grout holes that did not penetrate the entire deformation zone as well as using a grout mix that had a relatively low initial strength. This concept was planned with the aim of minimizing impact on hydraulic conditions.

Due to the unsatisfactory sealing results of the grouting, changes were made to the grouting strategy. Tests were conducted to determine the optimum grouting fan, injection pressure, grout medium and initial grout strength. It was found that long grouting holes penetrating the whole fracture zone and grouts with higher initial strengths gave more effective grouting results. Grouting holes with high water-cement ratios did not have desired sealing effects. Theoretical and practical evidence also indicated that the grouting pressure needed to exceed twice the water pressure to provide a sufficient dispersion of the grout cement. A new grouting concept based on

these findings was then employed and the water inflow was reduced to an accepted level. The tunnel excavation proceeded at a relatively rapid pace of about 2 m per day after implementation of the new grouting strategy.

Problems were encountered during the drilling due to water pressure acting on the drill rods and pushing them out of the hole at high speeds. This resulted in dangerous working conditions for the drill crews and caused partial flooding of the drill site. Introducing a special drilling arrangement that included a combination of drill niches, pump niches, borehole casing and a special water valve solved the problem. When the drill rod was extracted, the valve was closed in order to limit the inflow of water from the borehole, which meant that the drill site remained relatively safe and dry.

No indications of tunnel instability have been noted in the reports with regard to the passage of deformation zone NE-1. However, concerns relating to high water pressure destabilizing the tunnel face, particularly during the drilling and grouting have been noted. In order to guard against this, the distance between the tunnel face and deformation zone NE-1 was kept to about 20–30 m to ensure face stability during the drilling and grouting operations. The boreholes drilled from the face were cased to prevent water from being introduced to the fractures in the vicinity of the face. Rock support consisted of fibre-reinforced shotcrete, rock bolts and steel mesh installed in tunnel walls and roof.

The study of the maintenance records for the Äspö Hard Rock Laboratory indicates that the reason for the additional maintenance in different parts of the main tunnel is due to geological factors. The scaling, bolting and shotcreting are located in areas with intrusions or veins of fine-grained granite or Småland granite. Increased fracturing and fracture zones are typically associated to these two rock types. The support and scaling performed in the niches and side tunnels are most certainly caused by stress concentrations created by the choice of geometry. The comparison of a horseshoe shaped D&B tunnel with a TBM tunnel regarding the need for maintenance clearly shows that the drill and blast tunnel requires much more resources in that aspect.

9.3 Concluding remarks

It stands to reason that varying rock engineering difficulties and problems occurred during the course of the excavation works involving as much as about 8,000 m of tunnels and three major rock caverns. However, the majority of these problems must be considered as being of minor degree of difficulty and importance, and could generally be handled by day to day routine.

This conclusion is supported by the fact that there are no reports from the underground works, which indicate any serious consequences due to rock conditions encountered during the tunneling work. This opinion is also based on the total amount of rock support. The combination of the overall good rock quality and good construction practice has enabled rock reinforcement in the major caverns to be kept to very low levels. In addition, no stability or other rock engineering problems of significance have been identified after the commissioning of the Oskarshamn nuclear power units, Clab and the Äspö Hard Rock Laboratory, bearing in mind that the construction period extended over more than 30 years.

As a final conclusion of the construction and rock engineering experiences from the Äspö Hard Rock Laboratory, the authors of this Compilation Report would like to quote /Larsson and Leijon 1999/:

The underground excavation works for the Äspö Hard Rock Laboratory have not in any way been more difficult, more uncertain or more hazardous than underground works in general.

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