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Construction experiences from underground works at Forsmark

Compilation Report

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February 2007

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Preface

The main objective with this report, the Construction Experience Compilation Report (CECR), is to compile experiences from the underground works carried out at Forsmark, primarily construction experiences from the tunnelling of the two cooling water tunnels of the Forsmark nuclear power units 1, 2 and 3, and from the underground excavations of the undersea repository for low and intermediate reactor waste, SFR. In addition, a brief account is given of the operational experience of the SFR on primarily rock support solutions.

The authors of this report have separately participated throughout the entire construction periods of the Forsmark units and the SFR in the capacity of engineering geologists performing geotechnical mapping of the underground excavations and acted as advisors on tunnel support; Anders Carlsson participated in the construction works of the cooling water tunnels and the open cut excavations for Forsmark 1, 2 and 3 (geotechnical mapping) and the Forsmark 3 tunnel (advise on tunnel support). Rolf Christiansson participated in the underground works for the SFR (geotechnical mapping, principal investigator for various measurements and advise on tunnel support and grouting).

The report is to a great extent based on earlier published material as presented in the list of references. But it stands to reason that, during the course of the work with this report, unpublished notes, diaries, drawings, photos and personal recollections of the two authors have been utilised in order to obtain such a complete compilation of the construction experiences as possible.

Contents

1 Introduction

The Forsmark nuclear power plant and the repository for low and intermediate reactor waste, the SFR, are situated on the east coast of Sweden some 130 km north of Stockholm (Figure 1-1). The nuclear power plant comprises three units, and the cooling water from the units is discharged through two submarine tunnels. The repository for reactor waste is located undersea, connected by two access tunnels (Figure1-2).

The open cut excavations and the underground excavations for the three units of the power plant and for the SFR were carried out in the course of several time intervals from 1972 to 1986.

In total, about 1.2 million cubic meters of rock were excavated of which about 775,000 m^3 refer to underground excavations. The total length of tunnels is approximately 11,000 m. The underground excavations at Forsmark is normally sited on a depth between 50–140 m below sea bottom. The extension of the construction area of the facilities is about 8 km^2. The target site for the Final Repository at Forsmark is bordering but is also partly situated within this construction area.

The Forsmark power station has been built and is being operated by Forsmark Kraftgrupp. Forsmark Kraftgrupp commissioned Vattenfall as the main contractor for the plant.

The discharge tunnel for units 1 and 2 was designed and administered by Vattenfall (former Swedish State Power Board). Hagconsult was the consultant on permanent supports. The contractor for the tunnelling was Forsmarkstunnlar, a joint venture between Armerad Betong and Vägförbättringar.

Vattenfall undertook the design, the tunnelling work and the assessment of the permanent support for the discharge tunnel for unit 3, and Vattenfall made the geological follow-up of the two Forsmark tunnels.

The repository for reactor waste (SFR) is owned by SKB, which commissioned Vattenfall to plan, design and build SFR, and the design and tunnelling work was carried out by Vattenfall in-house resources. The consultant for the geological follow-up at the site was VIAK AB.

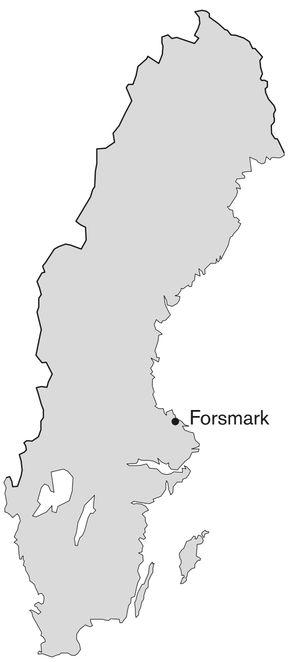


Figure 1-1. *Geographical location of Forsmark.*

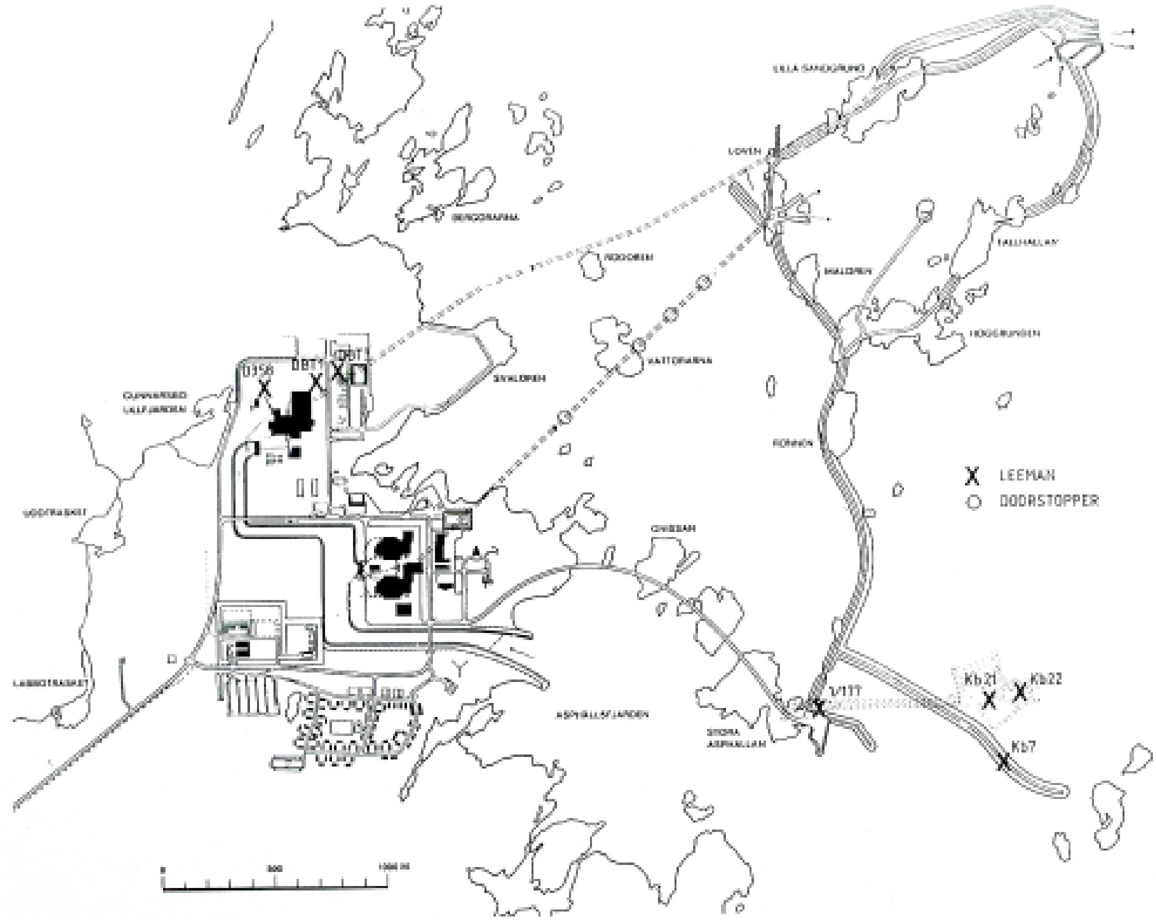


Figure 1-2. *Site plan of the Forsmark Power Plant and the SFR.*

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 Forsmark, 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 briefly 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 construction at the site has been to present the prerequisites for each of the three major construction projects at the Forsmark site. This includes the actual understanding on the site conditions, the requirements on each of the underground projects and the construction methods used in each of the projects.

There is a variation in documented data concerning the two discharge tunnels and the SFR. The variation comprises amount of data as well as quality and accessibility of the data. The authors have in principle focussed on published material, Swedish and internationally published papers on the Forsmark facilities, but also on Vattenfall and SKB internal reports. The main reason for using already published material is that the content, to a varying degree has been checked and reviewed before publication. But as mention in the Preface, also unpublished notes etc. and personal recollections of the two authors have been utilised in preparation of this report.

2 A learning process

Each of the major construction projects started with the specific experiences and requirements at the time being. The first design and construction works at site for the Forsmark 1 and 2 tunnel was to a high degree based on the experiences that the Swedish State Power Board (Vattenfall) had earned from decades of hydropower projects in Sweden. 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.

This report concludes the experiences gained up to completion of the SFR project. The formal periodic review of the status of the SFR facility has also been used as a reference for the design of a final repository at the Forsmark site from the lessons learned from maintenance of the underground facility during 20 years.

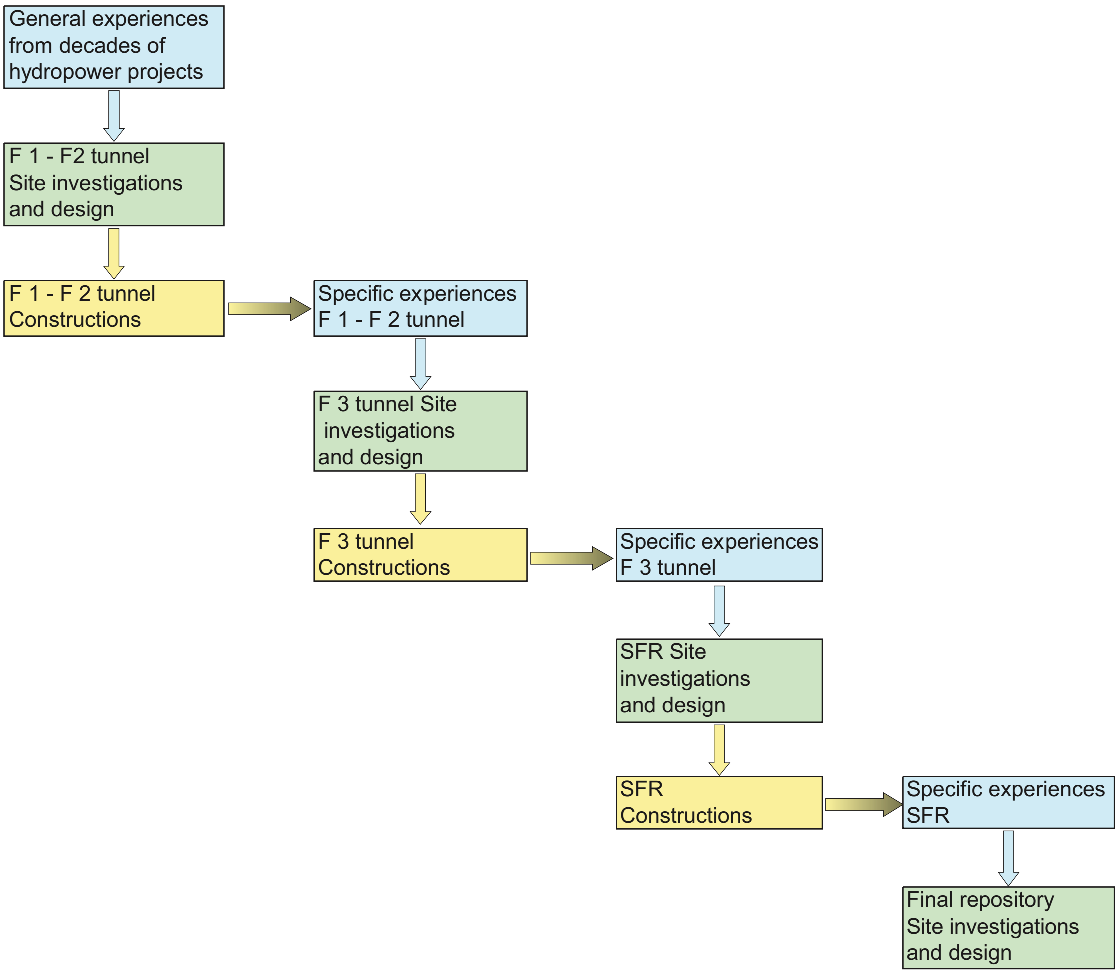


***Figure 2-1.** The sequential process of feedback from a construction project to the following.*

3 Geological overview

3.1 References

This brief, simplified geological overview is based on investigations and interpretations conducted during the course of the construction work at Forsmark, and published in among others, /Carlsson 1979, Carlsson and Olsson 1977, 1980, 1981, 1982abc, 1983ab, 1984, 1986, Hansen 1985, Carlsson and Christiansson 1986, 1987, 1988, Christiansson and Bolvede 1987/; i.e. the description should be seen in the light of how the geological, tectonic and hydrogeological conditions, from the engineering point of view, were understood, interpreted and used at the time of the construction and the completion of the Forsmark facilities.

Instead of including new data, such as new rock stress measurement data, hydrogeology data, etc., the authors believe it to be more relevant for the Reader of this Construction Experience Compilation Report to give the findings as they actually were presented to the end users at that time. However, adjustments of the geological terminology have been made, as far as reasonable, in order to follow the terminology of the ongoing descriptive modelling activities mentioned below.

This brief geological overview should therefore not be seen to be in any conflict whatsoever with the exceptional comprehensive, geo-scientific and detailed site descriptions, which the SKB is currently undertaking at Forsmark, with the objective of siting a geological repository for spent nuclear fuel. In order to give a comprehensive picture of the geology, the bedrock geological map of the Forsmark area (Figure 3-5) and the regional tectonic map (Figure 3-6) are derived from the SKB site descriptions mentioned below. The site descriptive modelling activities for the Forsmark area are so far documented in the SKB Report R-05-18, Preliminary site description, Forsmark area version 1.2, and in the SKB Report R-06-38, Site descriptive modelling, Forsmark stage 2.1 /SKB 2006/.

3.2 Site investigations

Site investigations and rock excavation works for the Forsmark nuclear power plant and the SFR started in 1971 with the site investigations for units 1 and 2 and were practically in continuous progress until 1986 when SFR was commissioned. The continuation of investigations was a consequence of the size of the plants and the special requirements imposed for nuclear power and reactor waste disposal (Figure 3-1).

The investigations included both regional and detailed investigations, which were undertaken simultaneously. The regional investigations were started with a review of earlier geological studies made locally together with appropriate material, such as topographical maps and aerial photographs. These investigations were then supplemented by a geological survey to elucidate the regional stratigraphy and the local rock distribution and occurrences of deformation zones.

Detailed studies were then instituted with a network of seismic profiles, after which the deformation zones indicated by the regional and seismic refraction investigations were examined closely by means of rock drilling. The seismic investigations were then intensified, further networks of seismic profiling being made, and the intensity of the rock drilling was increased.

In total, for the construction area of the Forsmark nuclear power plant, including SFR, more than 120 km of seismic profiles and more than 15 km of exploratory drilling, mainly core-drilling were carried out. In addition to standard core logging routines, a wide variety of hydraulic tests and geophysical logging were performed in the boreholes.

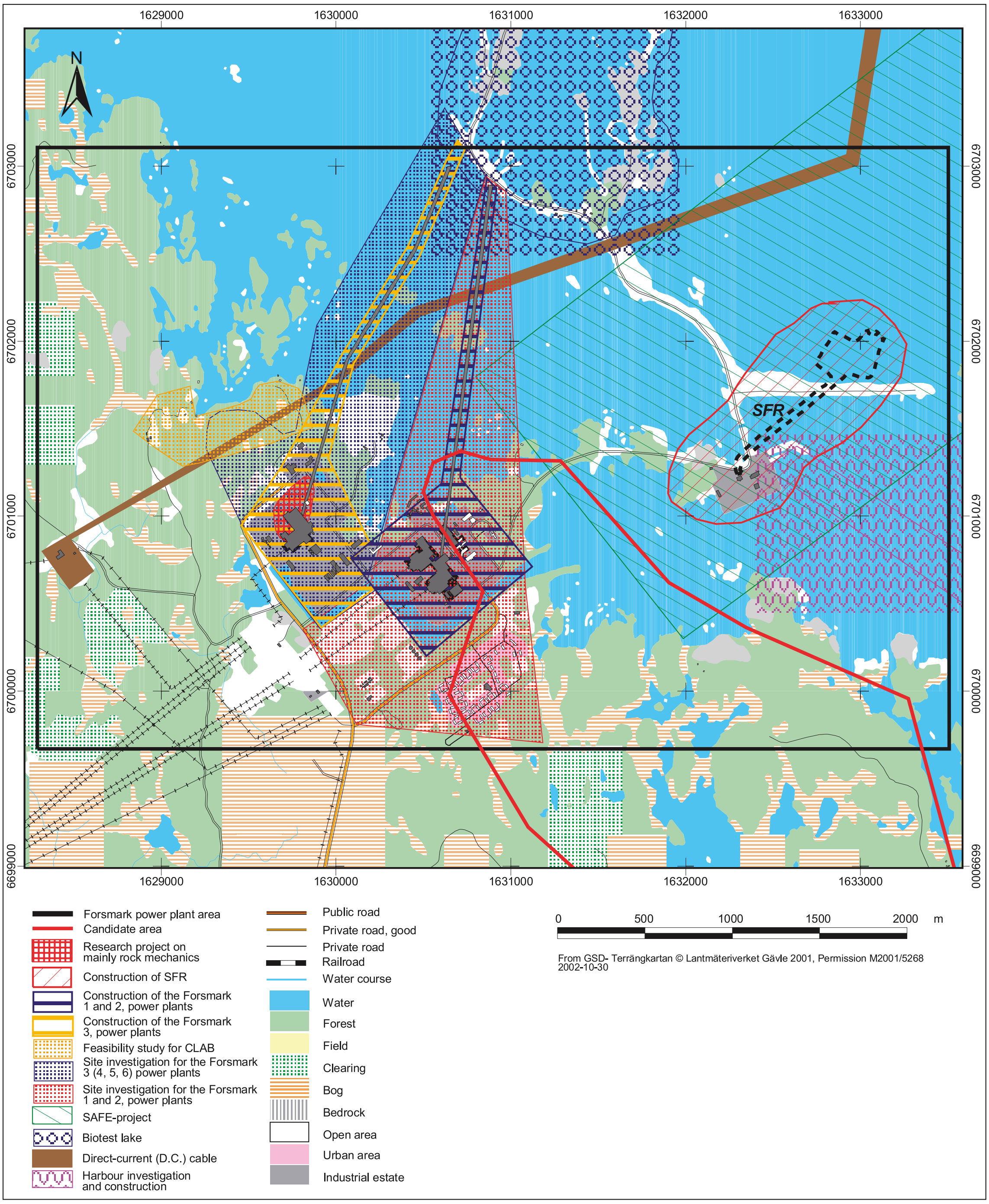


Figure 3-1. *Areas of investigations for the Forsmark nuclear power plant and the SFR are shown approximately by the scattering, /Carlsson and Christiansson 1987/. See also Figure 3-5.*

Concurrently with the construction work of the units, comprehensive engineering geological investigations were carried out comprising such as fracture surveys of the open cut excavations, tunnel mappings, groundwater studies in tunnels and in boreholes, in situ rock stress measurements, rock mass deformation measurements for the foundation of the turbine and reactor buildings. The in situ rock stress measurements and the tests to determine deformation characteristics were carried out primarily during construction /Hiltscher et al. 1984/.

Furthermore, a number of laboratory tests were performed to determine rock mass properties. The seismic profiling and the core drilling for the submarine tunnels and the submarine reactor waste repository were performed from the ice covering the sea and from offshore drilling platforms. Extensive drillings was also performed from underground excavations for various reasons. Figure 3-2 illustrates the drilling campaigns for the SFR, from platforms, ice-cover and from underground excavations. The drilling programme for the Singö deformation zone in the SFR access tunnels is shown in plan and section in Figure 3-3.

In conclusion, the extensive site investigations carried out at Forsmark during such a long period of time constituted a successive build-up of knowledge of the geological, geotechnical and hydrogeological conditions of the rock mass. Not only the investigations for the tunnels, but also the investigations and mapping of open cut excavations and drilling operations for the units significantly increased the knowledge and understanding of the geological conditions for the tunnels.

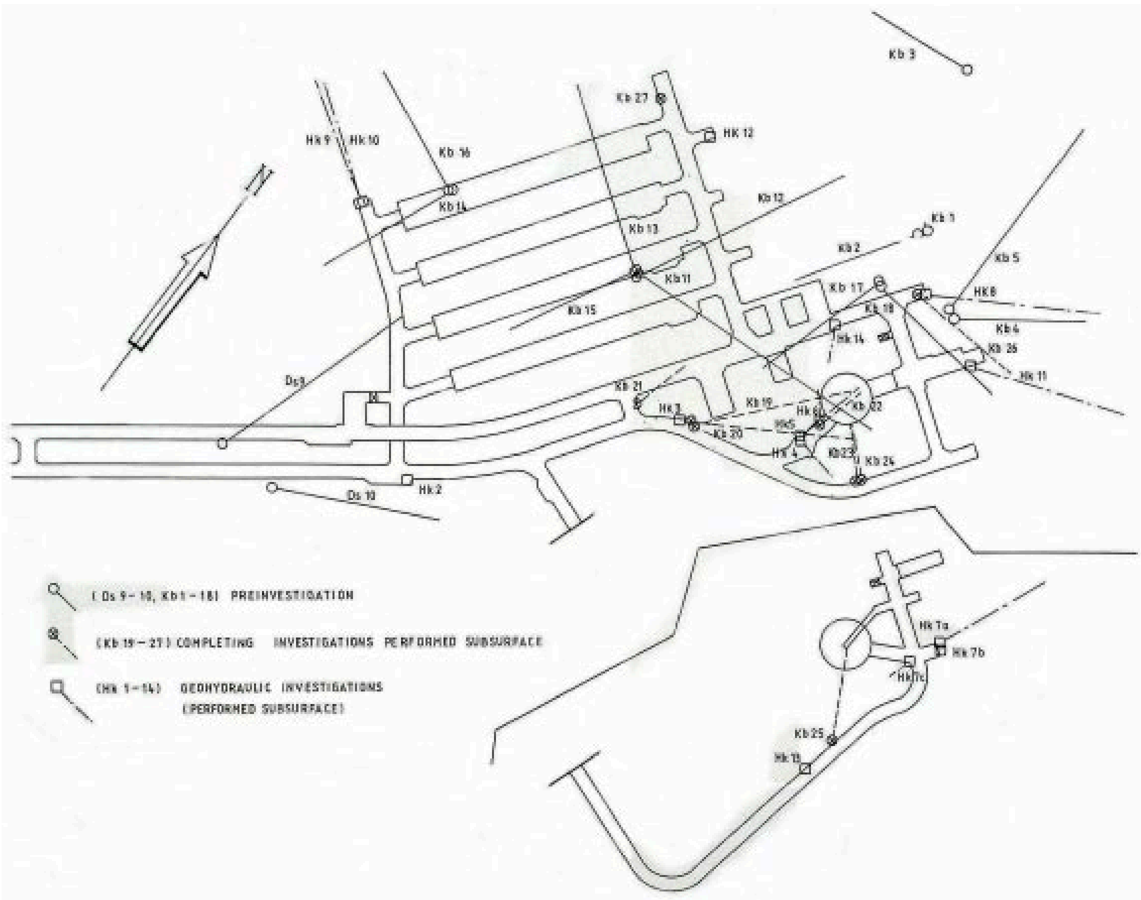
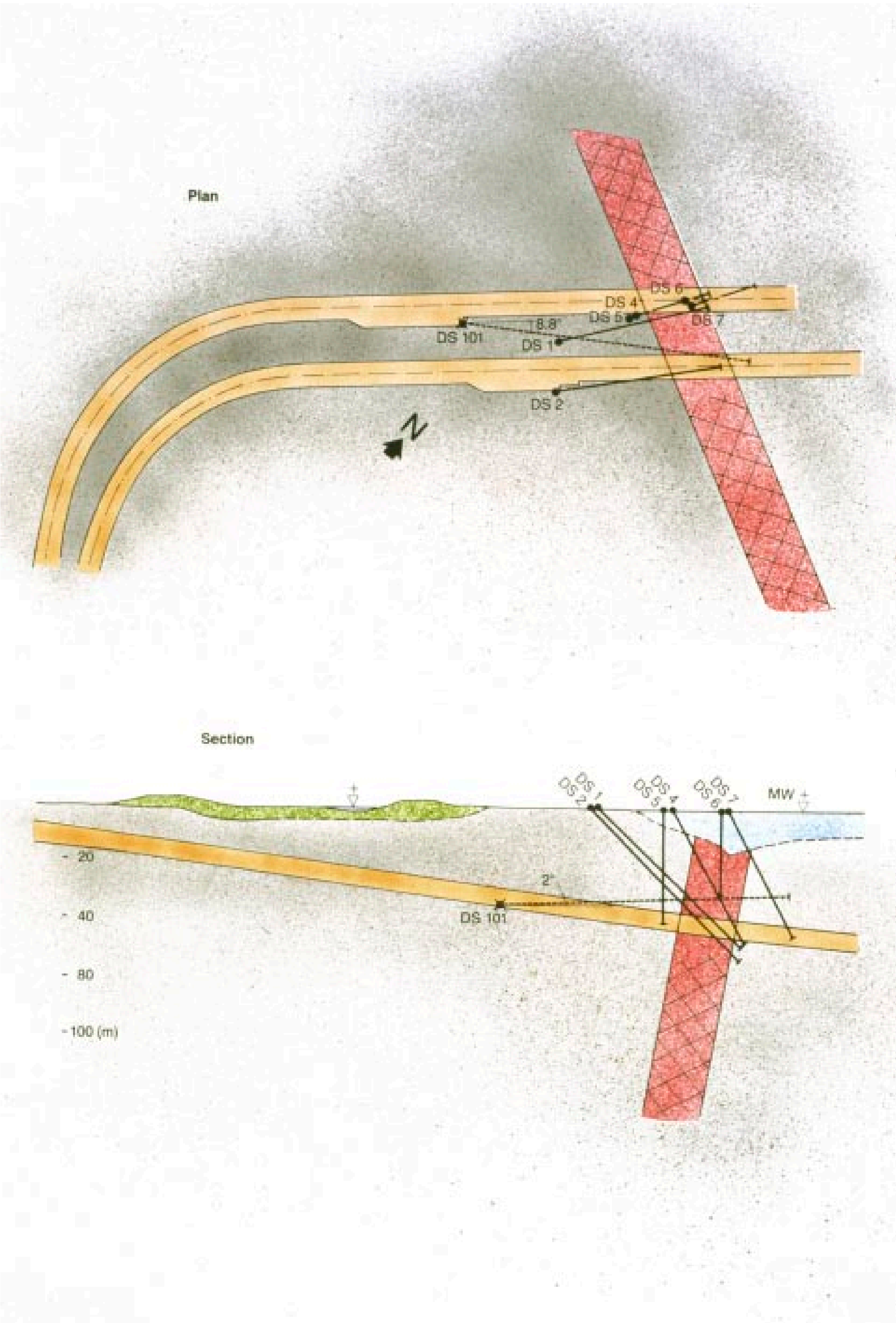


Figure 3-2. *Borehole layout for the repository, SFR /Carlsson and Christiansson 1987/.*


Plan
DS 6
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DS 1
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- 40
- 80
- 100 (m)
2°
DS 101

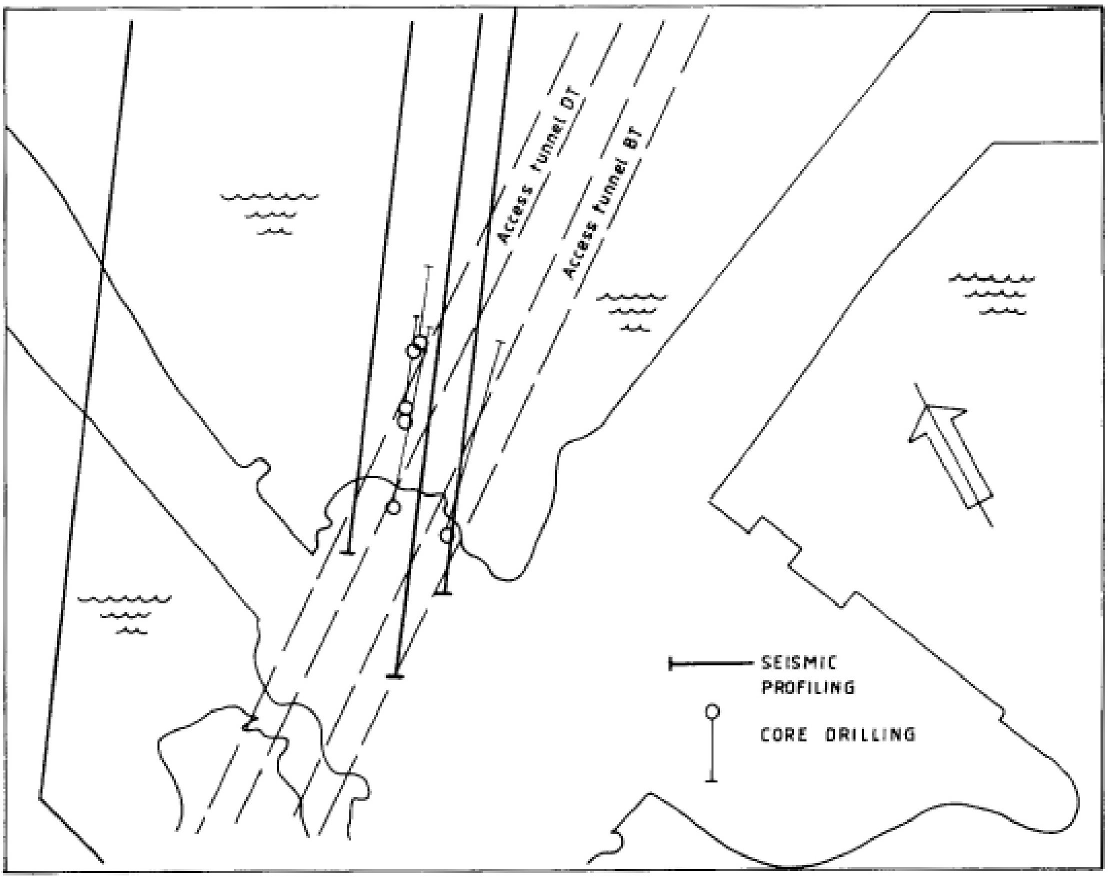


Figure 3-3. Borehole layout, in plan and section for the Singö deformation zone in the area of the access tunnels to the repository, SFR.

3.3 Morphological features

From a morphological point of view, the Pre-Cambrian peneplain of the Forsmark area is remarkably flat with only minor topographic variations. The ground level in the area investigated lies 1 to 5 m above sea level. The ground rises slightly and, about 2.5 km west of the plant, heights of about 25 m above sea level are occasionally found (Figure 3-4). It is, however, necessary to go about 15 km to the west to find more continuous areas reaching 20–30 m above sea level.

3.4 Quaternary deposits

Almost throughout the whole area investigated, the bedrock is covered by Quaternary deposits, mainly consisting of till and peat and rock outcrops on the mainland are therefore a rare occurrence. Till is the predominant Quaternary deposit within the construction areas, with a mean thickness of about 3 to 5 m and reaching a maximum of 14 m.

A characteristic feature is the concentration of the thickest layers of till to the lowest levels of the bedrock, which results in a levelling of the local topography. This morphological effect is thereby a contributory cause of the flatness of the area.

During the geological explorations four trial pits were investigated in the area of Unit 3. A sandy, silty till predominated and in one pit the till is superposed by a clay bed roughly 0.5 m thick. The rock surface in all four pits was found at a depth of about 3 m below the ground surface. Approximately 400 m to the east of Unit 3, three trial pits were excavated to a depth

Figure 3-4. *Aerial photo from 1980 showing the Forsmark construction area (Forsmark 3 is under construction); in the foreground, Forsmarks Bruk. Photo G Hansson/N.*

of 5 m. The upper part invariably consisted of a 0.5 m thick layer of wave-washed gravely, sandy till and in one pit wave-washed till superposed fine-grained sediments of a silty, sandy composition with a high degree of compaction which in turn rested on a very hard silty clay. The sediment beds were about 2 m thick and underlain by a hard clayey, silty till, which seemed to indicate the presence of double moraine. At the site of Unit 3, a blue-grey till also appeared. The till had a high compaction and was fine-grained with a low frequency of boulders and stones. The thickness of the till reached a maximum of nearly 4 m. This type of till was also found in a depression of the rock surface at the bridge abutment on the eastern side of the inlet channel. Because of the high degree of compaction blasting was necessary for the excavation of the till. Seismic investigations made during the geophysical explorations showed a seismic velocity of about 4,500 m/s in this particular area /Carlsson 1979/.

Next to the till, peat deposits are the most common soil type. The thickness of the peat deposits varies considerably; with an observed maximum of 2 m. Shore sand and clay occur as surface soils in the low levels of the terrain.

The offshore deposits consist generally of till covered by fine sediments such as clay. During the investigations for the harbour at Forsmark, it was found that a 2–4 m thick layer of clay rested on a 3–6 m thick layer of till /Axelsson 1986/. Evaluation of seismic surveys made at the SFR site revealed great variability in sediment thicknesses and layering. Finer sediments overlaying till predominate in areas where the till cover is thick. This is usually the case in depressions in the bedrock.

The sediments covering the bedrock above the repository site consist of coarse material such as sand and gravel in layers that are about 1 m thick /Axelsson 1986/.

3.5 Bedrock

Intrusive igneous rocks dominate the Forsmark Site. Supracrustal rocks, which are predominantly volcanic in origin and contain calc-silicate rocks and iron oxide mineralization, form a subordinate component. Apart from some younger granite, granite and pegmatite, all rocks are affected to a variable extent, by penetrative ductile deformation. This deformation is associated with recrystallization that occurred under amphibolite-facies metamorphic conditions and at depths probably greater than 15 km. For this reason, most of the rock names are prefixed with the term “meta” /SKB 2005/.

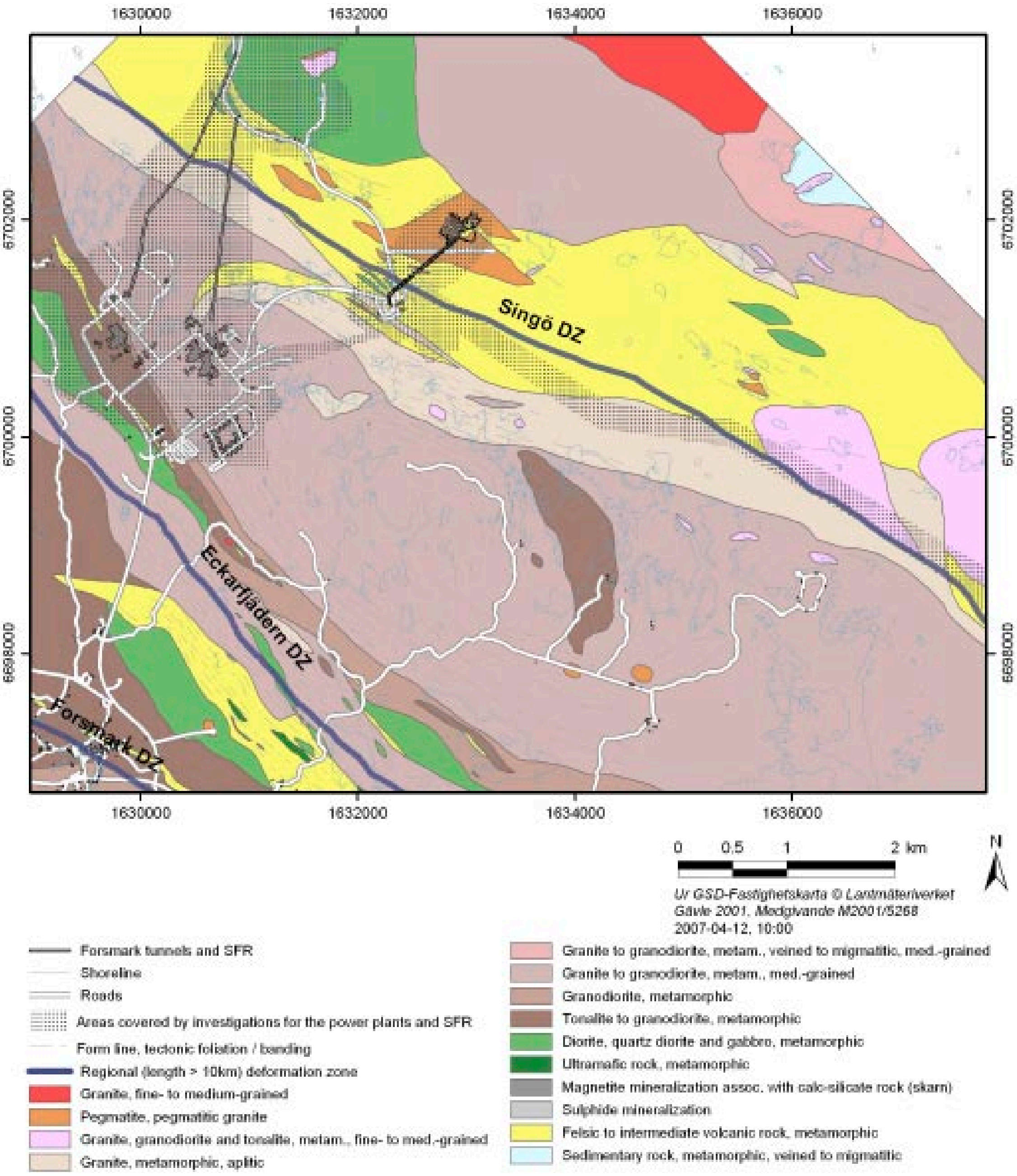


Figure 3-5. *Bedrock geological map of the Forsmark area, based on /SKB 2006/.*

Four major groups of rock types are present within the Forsmark Site /SKB 2005/:

- Fine-to medium-grained granite and aplite. Pegmatite granite and pegmatite.
- Fine-to medium-grained granodiorite, tonalite and subordinate granite.
- Biotite-bearing granite (to granodiorite) and aplitic granite, both with amphibolite as dykes and irregular inclusions. Tonalite to granodiorite with amphibolite enclaves. Granodiorite. Ultramafic rock. Gabbro, diorite and quartz diorite.
- Volcanic rock, calc-silicate rock and iron oxide mineralization. Subordinate sedimentary rocks.

The dykes are often found to run in the direction of the dominant schistosity, sub-vertical and trending NW-SE.

Larger areas with pegmatite are found on outcrops, in the open cuts of the power plant, in the discharge tunnels and in the SFR access tunnels. Pegmatites also occur as minor dykes. They represent at least two generations in the bedrock.

At a detail scale (tunnel and borehole observations) the lithological variation is complex, and folding is common. The contacts between different rock types are normally well healed.

According to the performed tests of cores the uniaxial strength of the granite-granodiorite was measured to 220–320 MPa, of the pegmatite 120–200 MPa and of the amphibolite 120–200 MPa. The Youngs's moduli were determined to 70–80 GPa, 40–80 GPa and 70–80 GPa respectively.

3.6 Tectonics

3.6.1 Deformation zones

The regional tectonic pattern within the Forsmark construction site is shown in Figure 3-6. Three main types of deformation zones have been identified in the Forsmark area. The most obvious of those are regional deformation zones, which strike WNW to NW with a vertical or steep dip orientation. This Singö deformation zone is the most well known of these structures, and exists over a length of at least 30 km /SKB 2005/. The Singö deformation zone, about 300 m to 1,000 m off the coast, is a regional fault line, that was penetrated by both the cooling water discharge tunnels from the power plants and the access tunnels for the SFR. The vertical or steeply dipping Singö deformation zone is about 200 m wide (plus/minus 50 m) in its central part, and complex, with smaller crushed zones running in different directions, partly altered rock and veins of clay. The general strike and dip of the zone is 120/90.

There are other regional deformation zones in Forsmark area, but none of them had any rock engineering impact on the construction of the Forsmark plant and the SFR.

It is known from core drilling and tunnelling through deformation zones in the Forsmark area that some structures are brecciated, with 3–4 different minerals, which are partly crushed and sealed with younger minerals. It was concluded, that the rock had been subjected to tectonic forces of a magnitude high enough to move large blocks in relation to one another on at least four occasions.

3.6.2 Joint sets

The blocky pattern found in the large-scale tectonic features was also found in the joint distribution (Figure 3-7). The average frequency based on examination of cores drilled in different directions is about 4 joints/m in the SFR area. The predominant sub-vertical joint set in the NW-SE direction includes the schistosity, and is sub-parallel to the Singö deformation zone. The joints are mainly healed by chlorite. The orthogonal vertical joint set gives rise to some

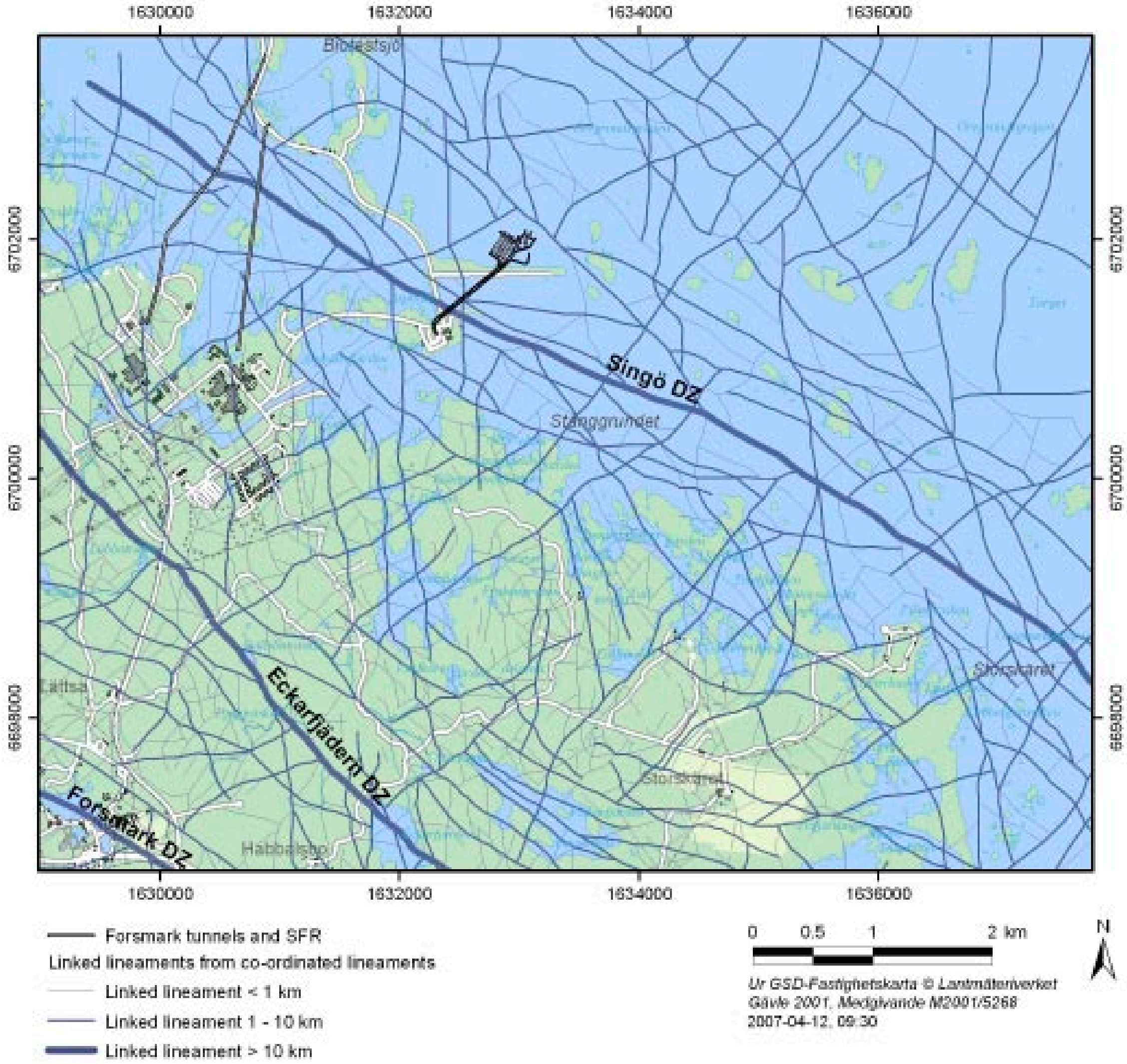


Figure 3-6. *The regional tectonic pattern within the Forsmark construction site. Lineament map based on /SKB 2006/.*

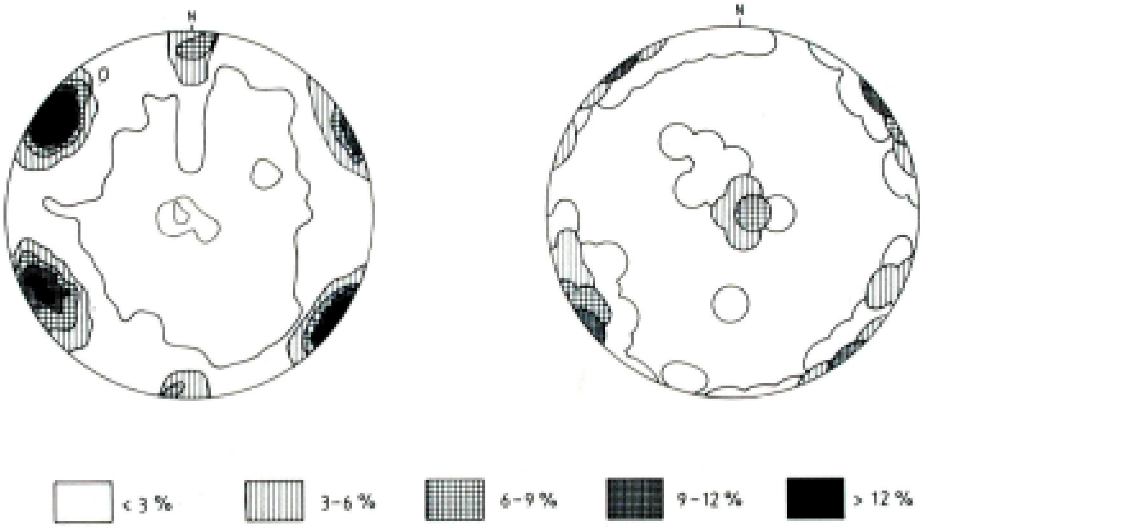


Figure 3-7. *Stereographic projection from Forsmark Unit 3 and the SFR, /Carlsson and Christiansson 1988/.*

zones with a dense fracture frequency. Individual fractures in such zones were measured to be 10–30 m long. The zones are seldom wider than about 100 mm. The rock is slightly altered, and the fractures are sealed with calcite, partly as crystals. Although orthogonal to the schistosity, these zones are referred to as schistose zones in the NW-SE direction.

A most striking feature of the rock mass at Forsmark is the difference in fracture behaviour between the uppermost part of the rock mass and deep-seated masses. Generally speaking, the actual difference is the occurrence of superficial horizontal and sub-horizontal fractures (dip normally less than 10^0) with wide apertures (locally 500–800 mm) most of them filled with unconsolidated sediments and rock fragments (Figure 3-8). These fractures may be described as a structure found primarily in granitic rocks, which is commonly termed sheet structures /Carlsson 1979/.

Both continuous and discontinuous horizontal fractures in the Forsmark area running almost parallel to the rock surface divide the rock mass into sheets or slabs of varying thickness. The fractures show a marked depth dependency, such that frequency, extension and aperture decreased with depth. Fracture lengths varied widely; tens of metres were often recorded and the maximum value was 170 m. A large portion of the horizontal and sub-horizontal fractures at depths down to about 5 m was filled with sediments. Below this depth, sediment fillings were less common. The deepest occurrence of sediment fillings was documented at 13 m. The glacial fracture fillings suggest that these fractures either formed or were reactivated (opened) during late- or post-glacial time /Carlsson A 1979/.

Figure 3-8. *Photograph of shaft walls of the storage basin for industrial water showing continuous horizontal fractures containing filling material of sediments. The length of the shaft is approximately 100 m. Photo G Hansson/N.*

3.7 Hydrogeological conditions

As mentioned earlier, from a morphological point of view, the Forsmark area is remarkably flat with minor topographical variations. This gives a low groundwater gradient – about 0.5 m/km. Fresh groundwater does not even reach the SFR repository area because of the salt-water interface just off the shore. The groundwater in the repository area is more saline than the water in the Baltic. The groundwater is stagnant and estimated to be about 3,000 years old /Wikberg 1986/.

The land up-lift in the Forsmark area is about 6 mm per year. In more than 500 years the sea-bed above the SFR repository will be dry land. But Finite Element Calculations carried out for the SFR demonstrated that the Singö deformation zone, with a hydraulic conductivity of between 10^{-5} m/s and 10^{-7} m/s acts as a discharge zone for most of the groundwater flow from the land. The dominant deformation zones outside and under the repository have a hydraulic conductivity between 10^{-6} m/s and 10^{-8} m/s. They act as a hydraulic cave for most of the groundwater flow through the SFR area.

Here, SFR is used as an illustrative example of the hydraulic conditions in the Forsmark area. Figure 3-9 shows the variation in hydraulic conductivity along the access tunnels of the SFR, calculated from inflow measurements in exploratory boreholes during the excavation of the tunnels. This illustrates how different water-bearing units exist side by side without noticeable interaction.

The total stationary inflow to the SFR after the excavation work (1986) was about 720 l/min. The access tunnels (including the Singö deformation zone) contributed about 300 l/min, and the seepage from a sub-horizontal zone into the service tunnel under the silo was about 100 l/min. The total seepage into the repositories (rock caverns and silo) was measured and found to be about 55 l/min. The seepage into the 45,000 cubic metres excavated silo was only 2 l/min.

Within the repository area grouting was performed only to a minor degree in one of the rock caverns. It was estimated that about one third of all groundwater flow came through the floors of tunnels and rock caverns.

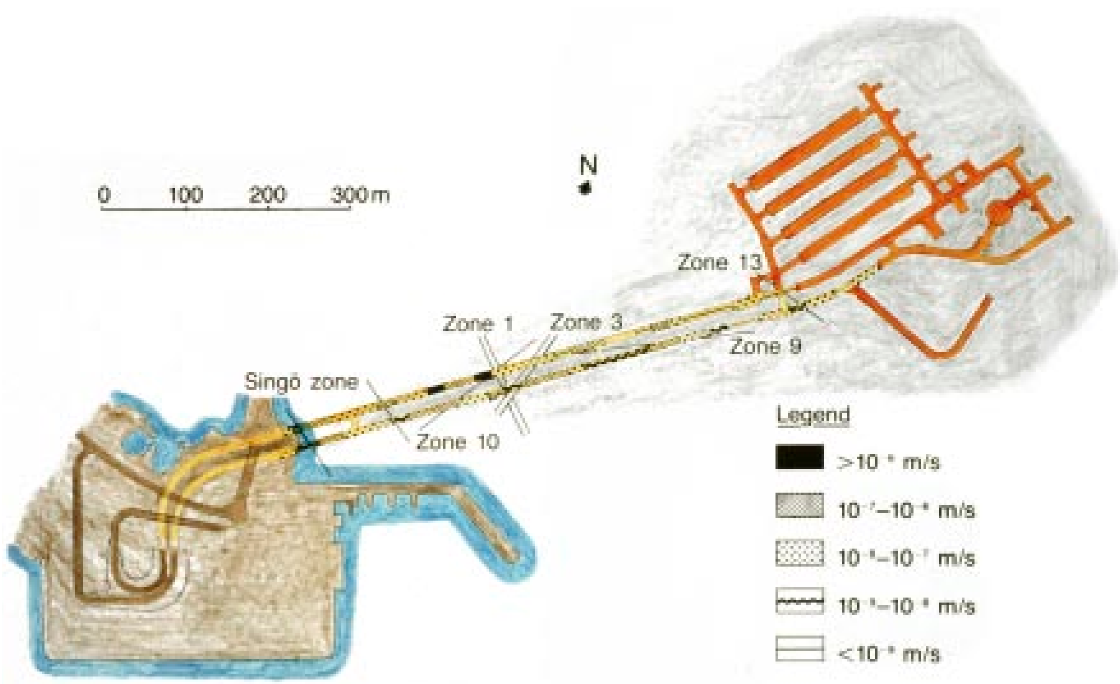


Figure 3-9. *The hydraulic conductivity distribution in the access tunnels, SFR, calculated from inflow measurements in exploratory boreholes during tunnelling, /SKB 1985/.*

The seepage in the repository area was mainly related to three predominant joint sets (cf. Figure 3-7). About 6 per cent of the visible joints in walls and roofs within the repository area exhibited seepage after excavation (mainly moist rock or dripping of water). The distribution of estimated seepage from individual joints is shown in Figure 3-10.

3.8 Rock stresses

In situ rock stress measurements were made in 10 boreholes at Forsmark, of which three boreholes were drilled for the SFR.

The measuring method used was developed by the Swedish State Power Board (Vattenfall) and allows measurements of the directions and magnitudes of the principal stresses, whereby the three-dimensional stress tensor can be determined. The measurement technique also allows measurements in deep water-filled boreholes /Hiltscher et al. 1979/.

The results of all the rock stress measurements in the ten boreholes at Forsmark (maximum measuring depth of 500 m) give a mean direction of the principal stresses in the horizontal plane of NW-SE, which corresponds to the direction of the fracture zones, fracture orientation data and the rock foliation.

The measurements taken at Forsmark, with the exception of the SFR area, reveal a high horizontal stress level in the superficial rock mass. In the upper 35 m of the rock mass, the stresses in the horizontal plane are in the range of 20–30 MPa. The stress distribution vs. depth indicates an increase in the vertical stress vector, which is approximately in accordance with the weight of the overburden, although tensile stresses exist even at great depth.

At SFR, the magnitude of the measured horizontal stresses measured from boreholes drilled from the underground caverns is lower, revealing divergence from the overall magnitude of the horizontal stresses in the area. Horizontal stresses for different measurement levels are shown in Figure 3-11.

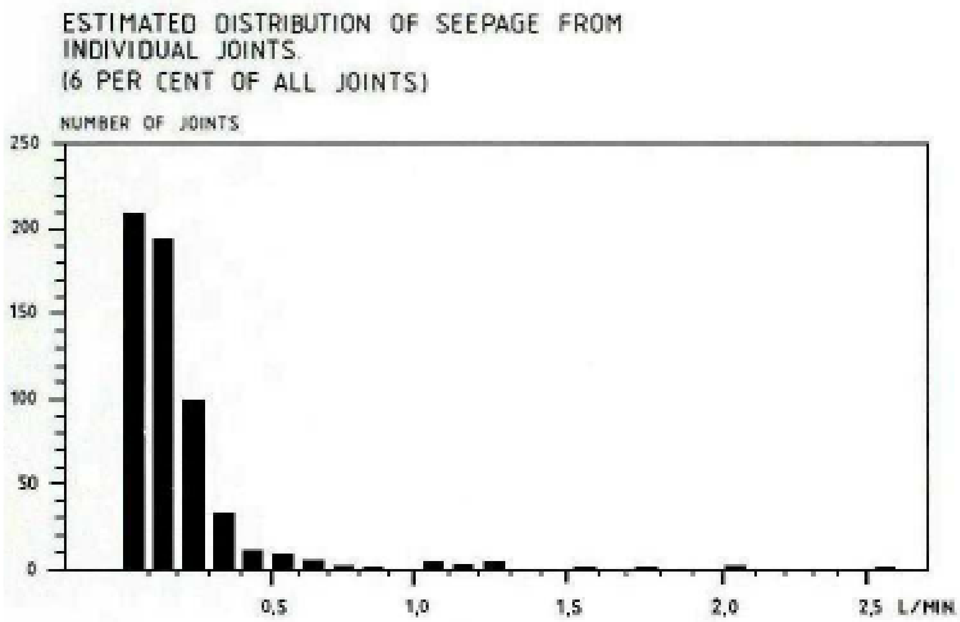


Figure 3-10. Estimated distribution of seepage from individual joints. The diagram is based on data from tunnel mapping, and represents the water-bearing joints in the repository, SFR /Carlsson and Christiansson 1988/.

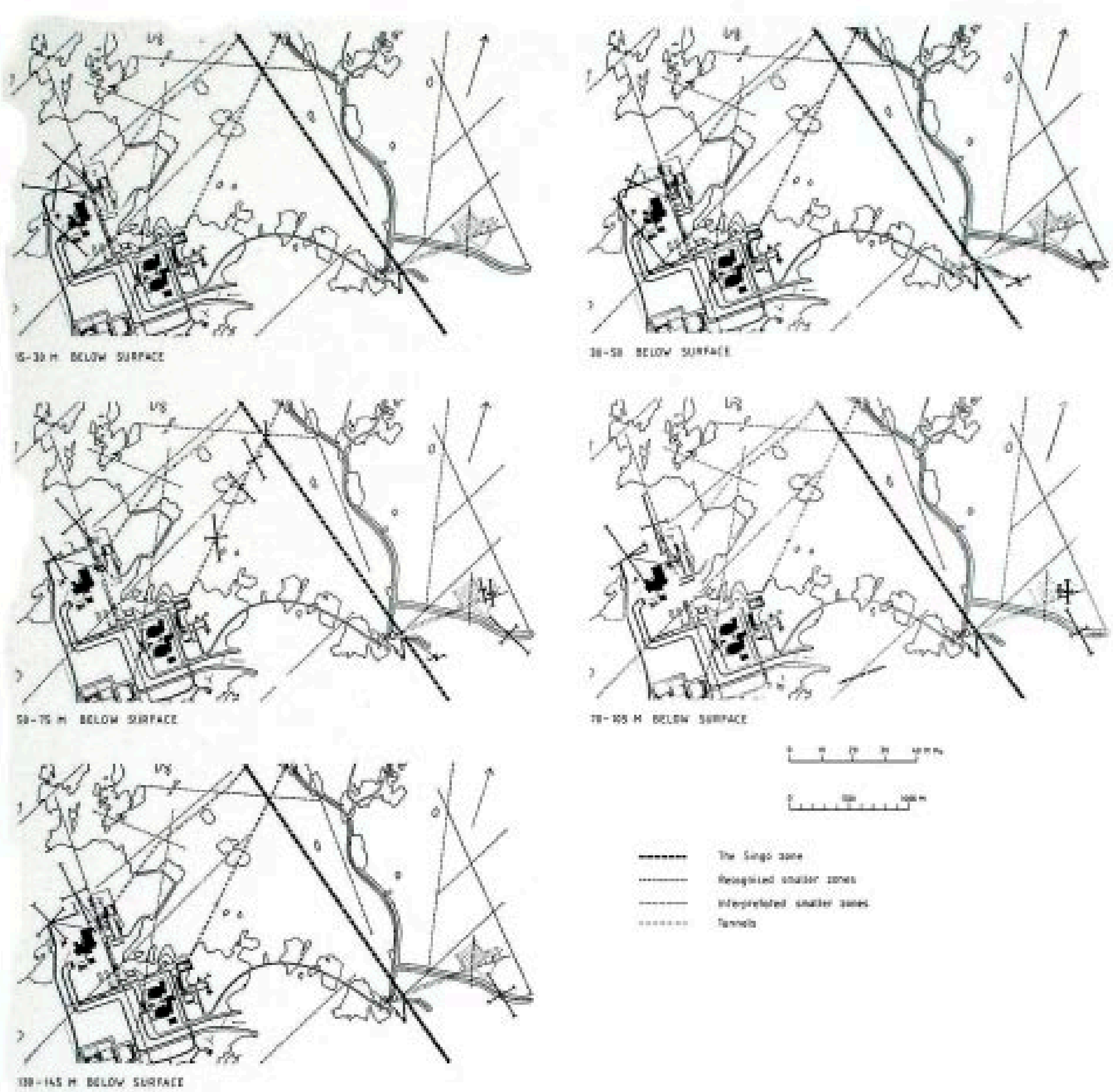


Figure 3-11. *Horizontal stresses for different measurement levels. The figure represents all measurements carried out at Forsmark up to 1986, /Carlsson and Christiansson 1986/.*

In conclusion, the rock stress measurements at Forsmark (Units 1, 2 and 3 and SFR) indicated that there is no significant difference in stress field between different rock types (local scale) or between larger intrusions.

Larger structures, such as the Singö deformation zone, seem to influence the direction of the principal stresses. There is a tendency for stress directions to be parallel to an adjacent structure. Minor structures, such as folds and geological irregularities (for example increased joint frequency), influence the magnitude as well as the direction of the rock stresses. There seems to be a significant relationship between rock stresses and depth below the rock surface. The superficial rock mass down to a depth of 50–60 m below the rock surface shows a stress situation with large deviation in stress magnitude and direction. This may be an effect of the latest glaciation.

The relationship between the average horizontal and vertical stresses seemed to be almost constant (2.9) from the rock surface down to a depth of 500 m.

Based on the rock stress conditions at Forsmark, it was judged that the tectonic stresses are favourable from an overall stability point of view. However, the stress distribution was considered to create relatively high tangential stresses in the walls and the roofs of the silo of the SFR. This situation was estimated to possible lead to rock spalling phenomena.

In view of this, the brittleness of the rock types was tested. The results of this investigation show that the gneiss granite and the pegmatites have a brittleness coefficient greater than 1. In spite of the relatively high brittleness coefficient and the rock stress situation, no rock spalling phenomena occurred during the excavation. However, it is to be noted that rock spalling occurred in the recirculation tunnel of Forsmark 3 after some metres of excavation. The rock cover at the sections of spalling was only 10–15 m. Figure 3-12 shows the portaling of the recirculation tunnel of Unit 3, where rock spalling occurred after some metres of excavation.

Figure 3-12. *A view of the portaling of the recirculation tunnel of Forsmark Unit 3, where rock spalling occurred after some metres of excavation. Photo G Hansson/N.*

4 Experiences from the construction of the cooling water discharge tunnels at Forsmark

The discharge tunnel for the cooling water from Forsmark 1 and 2 has a total length of 2,300 m and runs from the station area via a surge basin to discharge at Loven Island into an artificial bio test lake (Figure 1-2). The cross sectional area of the tunnel is about 80 m^2, and was excavated with a top heading of 50 m^2 and a bench of about 30 m^2. The maximum depth of the tunnel below sea level is about 75 m, the rock cover being approximately 55–60 m thick as maximum. The construction of the cooling water tunnel of Forsmark 1and 2 was started in 1974 and finished in 1976.

The discharge tunnel for cooling water from Forsmark 3 is about 3,000 m long. Over most of its length, it runs under the Baltic and reaches a maximum depth of about 70 m below the sea level (Figure 1-2). The tunnel is 10 m wide and 6.2 m high, giving a theoretical area of 55 m^2. The construction of the cooling water tunnel of Forsmark 3 started in 1980 and was completed in 1982. The distance between the two tunnels is approximately 400 to 700 m.

The bio test lake, some 90 hectares in size connected with the outlet of the discharge tunnel Forsmark 1 and 2 was constructed in order to allow investigation of the effect of the discharge of the heated cooling water on marine-flora and fauna. The dam incorporates material from the earth and rock excavation in the area (Figure 4-1).

Figure 4-1. *Aerial view of the bio test lake with the outlet of the Forsmark 1 and 2 tunnel in the foreground, and the outlet of the Forsmark 3 tunnel to the left (on the sea-side of the dam). Photo G Hansson/N.*

4.1 Site Investigations

The geological and geophysical exploration for the Forsmark 1 and 2 tunnel was started in 1971 and was completed in 1973 /Larsson 1973, Moberg 1974/. It included the geological mapping of exposed rocks, seismic investigations and diamond-core drillings. A number of seismic profiles were made along the tunnel and, on the basis of these investigations; nine diamond-drilled boreholes were placed in zones of low seismic velocity. In seven of these boreholes, a systematic test of water loss was made. In parallel with the excavation of the Forsmark 1 and 2 tunnel, site investigations with a similar scope were carried out for the Forsmark 3 tunnel /Larsson and Moberg 1975/.

4.2 Geological follow-up, tunnelling and rock support work

The rock works in the cooling water tunnels were followed up continuously to obtain information on the geological and tectonic conditions in the area. This work was carried out to collect basic information for assessment of permanent support required and to provide documentation. The geological surveying comprised both geological structure assessments as well as characterising of rock types, fracture fillings and evaluating of other factors of importance to the stability of the rock and to the driving of tunnels.

4.2.1 Forsmark 1 and 2 tunnel

A longitudinal section through the Forsmark 1 and 2 tunnel is shown in Figure 4-2. The section illustrates the main rock types, deformation zones, rock excavation classes, seismic velocity, location of water samples, and water inflow. During the engineering geological mapping, the tunnel was divided into nine rock excavation classes, and the characteristic parameters of each class are presented in Table 4-1.

Table 4-1. Characteristic parameters of the rock excavation classes in the Forsmark 1 and 2 tunnel /Carlsson and Olsson 1977/.

Class	Main rock type – structure of the rock mass	Joint sets	Estimated RQD	Max compressive stresses in the horizontal plane. Magnitude and direction	Leak intensity L/min m
A	Metagranite – Massive	N80°W; 80°S Horizontal	75		0.5
B	Metagranite – Slaty	E-W; 300 S Horizontal	45		3.2
C	Metagranite – Massive	N80°W; 80°S E-W; 10°S	100	15 MPa; N24°W	1.9
D	Metagranite – Blocky	N80°W; 70°S N80°E; 50°W Horizontal	25		8.3
E	Metagranite – Massive	N80°W; 70°S Horizontal	80	13 MPa; N60°W	0.4
F	Metagranite – Massive	N70°W; 80°S N80°W; 40°S	80	14 MPa; N45°W	0.5
G	Brecciated metagranite	Brecciated	30		3.9
H	Paragneiss – Slaty	E-W; 70°S	95	9 Mpa; N24°W	1.0

Class	Main rock type – structure of the rock mass	Joint sets	Estimated RQD	Max compressive stresses in the horizontal plane. Magnitude and direction	Leak intensity L/min m
I	Paragneiss – Blocky	E-W; 60°S N20°E; 80°N	30		3.5
K	Paragneiss – Slaty	N45°W; 70°S	100		0.4

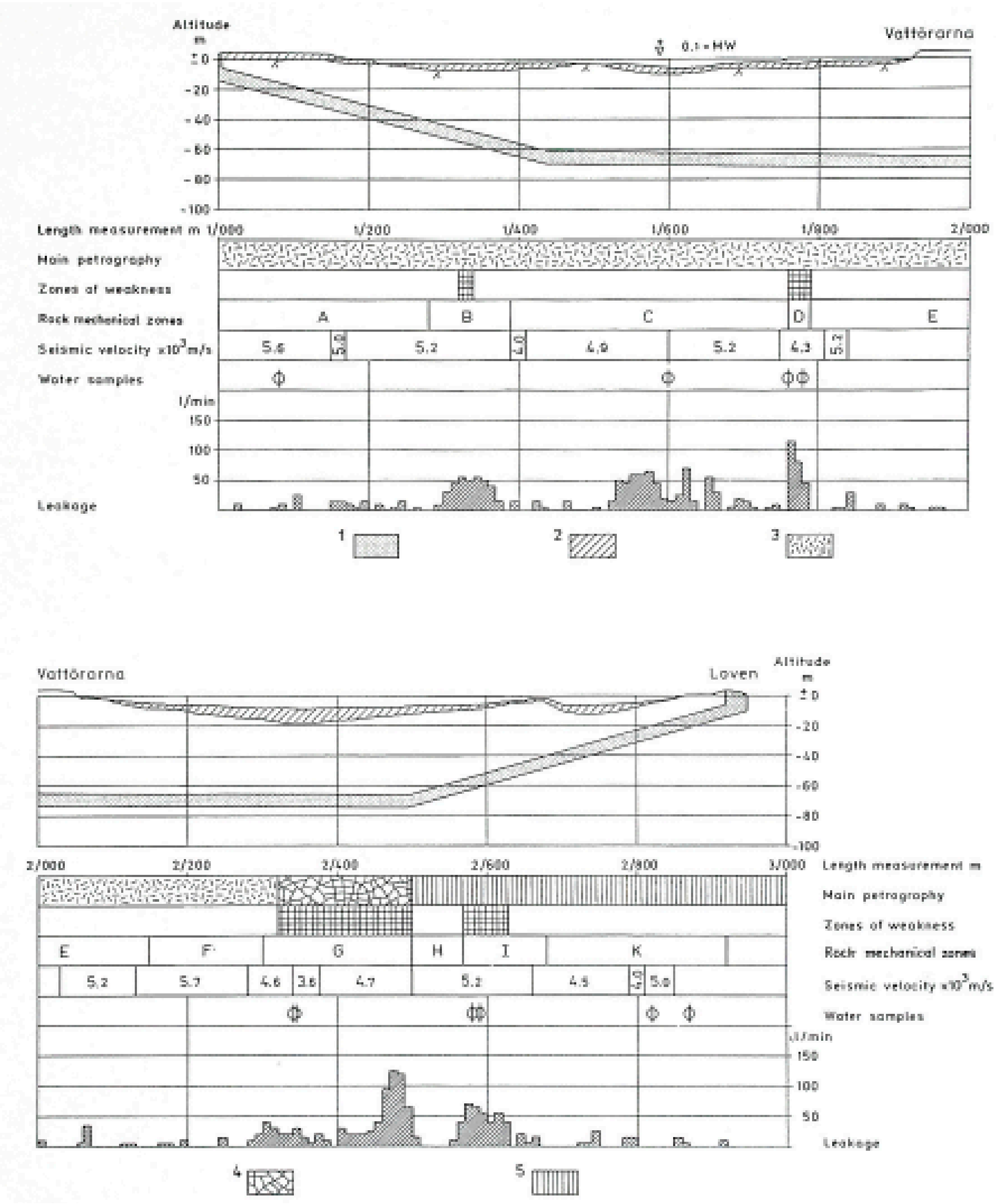


Figure 4-2. *Longitudinal section through the Forsmark 1 and 2 tunnel, /Carlsson and Olsson 1977/.*

Disturbances in tunnelling occurred in rock excavation class B, G and I. However, it must be emphasised that the rock engineering problems were only minor, and could be handled by applying conventional support practice (pattern and spot bolting, un-reinforced shotcrete, mesh-reinforced shotcrete, and reinforced shotcrete arches). The shotcrete thicknesses normally used, varied between 80 to 200 mm depending on whether the shotcrete was reinforced or not.

Rock Excavation Class B – The driving problems in class B were a consequence of the occurrences of horizontal fractures in combination with water inflows, mainly in the roof and abutment. Minor rock falls occurred and pattern bolting and reinforced concrete arches were carried out in the section.

Rock Excavation Class D – Rock excavation class D comprises partly crushed rock with a comparatively high water inflow. However, no driving problems occurred mainly because the tunnel intersected the crushed zone almost perpendicularly. In addition, the zone did not contain any clay-filled or weathered fractures, and thereby no risk for washout of material.

Rock Excavation Class G – The problems, which occurred within rock excavation class G (part of the Singö deformation zone), which comprises, brecciated metagranite of aplitic type, and resulted of minor weathering and water inflows along the rock contacts. The maximum water inflows occurred in the rock contacts. Within rock excavation class G, there were also a few sub-vertical clay-filled fractures with fracture openings of up to 500 mm (Figure 4-3). These fractures were supported with dental treatment and reinforced shotcrete arches; i.e. weak material was excavated for full width of zone to firm material or to depth equal to width of zone, the zone was backfilled with shotcrete extended approximately 1 m of either side of the clay-filled zone on to sound rock, where after permanent mesh/shotcrete layers were applied. The frequency of open fractures within the brecciated zone showed an increase between section 2/450 and 2/500 (cf. Figure 4-2; longitudinal section), while the fractures are mainly sealed in the remaining part of the zone (Figure 4-4).

Rock Excavation Class I – The fracture orientation within rock excavation class I caused driving problems due to unfavourable intersecting fractures, which resulted in rock falls and minor arching in the roof. The support constituted spot bolting and mesh-reinforced shotcrete.

Figure 4-3. *Clay-filled joint in brecciated metagranite within the Singö deformation zone, Forsmark 1 and 2 tunnel, /Carlsson A, Olsson T 1977/. Photo G Hansson/N.*

Figure 4-4. Brecciated metagranite within the Singö deformation zone with numerous veins of quartz and calcite, Forsmark 1 and 2 tunnel /Carlsson and Olsson 1977/. Photo G Hansson/N.

In total, 13% of the tunnel length is characterised as deformation zones, and the water inflow was about 0.03 l/s and m tunnel. About 60% of the total water inflow could be attributed to these mentioned deformation zones. The rock mass in between these zones did only exhibit negligible water inflows or was completely dry. The hydraulic conductivity and corresponding values of the leak intensity for the rock mass in the various rock excavation classes are shown in Table 4-2. The calculations are based on water pressure tests in the exploratory boreholes drilled during the site investigation phase of the Forsmark 1 and 2 tunnel.

During the top heading operations, pre-grouting with about 115 tons of cement was carried out. The locations and the quantities of pre-grouting are shown in Figure 4-5.

Table 4-2. Hydraulic conductivity and corresponding values of the leak intensity for the rock mass. The calculations are based on water pressure tests in borings along the F1 and 2 tunnel during site investigations /Carlsson and Olsson 1977/.

Borehole No.	Mean values, whole borehole		Mean values, excl. surface rock		
	Hydraulic conductivity m/s	Calculated leak intensity, l/min m	Hydraulic conductivity m/s	Calculated leak intensity, l/min m	Rock excavation classes penetrated by the borings
D61	1.36×10^{-6}	2.53	6.1×10^{-7}	1.13	D–E
D 62 A	2.13×10^{-7}	0.40	2.28×10^{-7}	0.42	K
D 63	5.02×10^{-7}	0.93	5.11×10^{-7}	0.95	G
D 64	6.50×10^{-7}	1.21	2.48×10^{-7}	0.45	E
D 66	3.38×10^{-7}	0.63	3.38×10^{-7}	0.63	F–G
D 67	1.06×10^{-6}	1.98	1.06×10^{-6}	1.98	H–I
D 68	5.5×10^{-7}	1.02	7.69×10^{-8}	0.14	E
Mean values	6.68×10^{-7}	1.24	4.4×10^{-7}	0.81	

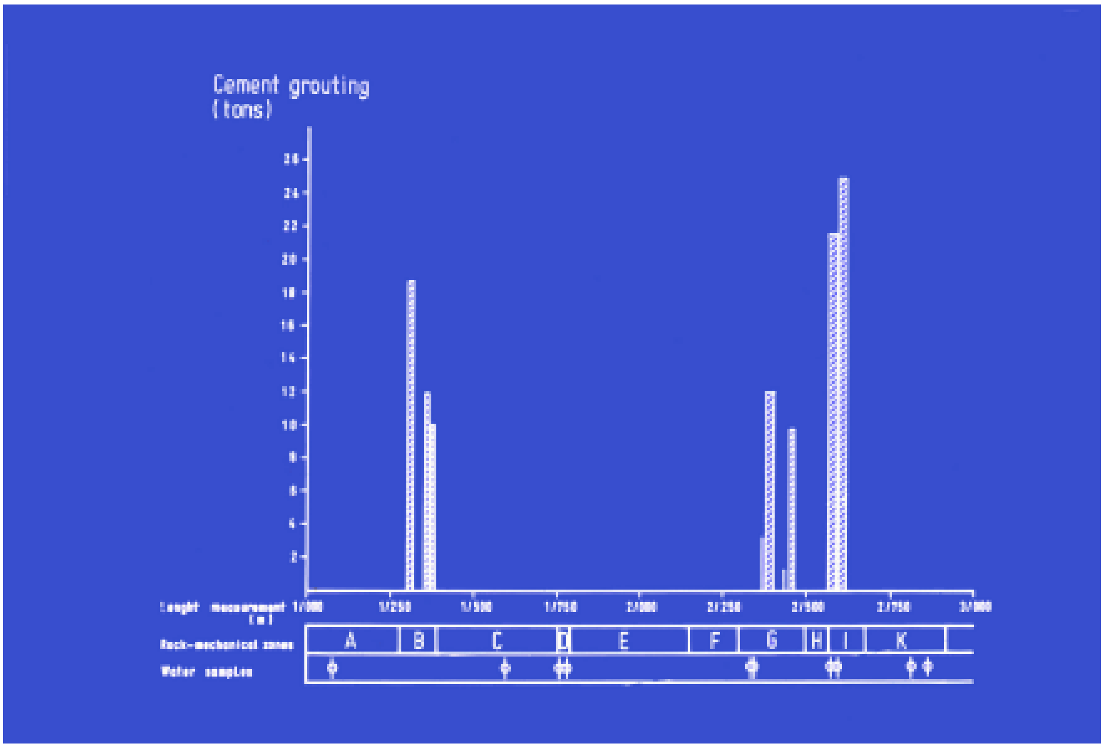


Figure 4-5. Cement grouting in the Forsmark 1 and 2 tunnel. The sites of the grouting and the quantities of cement used are shown in the diagram /Carlsson and Olsson 1977/.

The total measured inflow to the Forsmark 1 and 2 tunnel was 3,000 l/min.

/Carlsson and Olsson 1977/ conducted a special study on the general occurrence of the individual water inflows in the Forsmark 1 and 2 tunnel. They concluded that the water inflows seemed to follow certain patterns in relation to the different joint sets and joint system of the rock mass. Also the varying leakage volumes and the degree of tectonic disintegration were in well-defined agreement with each other. Normally, a high degree of breakage corresponded to a high leakage volume.

These connections between the water inflows and the joint pattern made it possible to classify the occurrence of the leakage in relation to each single water-bearing fracture. Five types of leakage emerged from the observations, as dominating the water inflow. The principle features of each type and, in consequence, the differences between the types are schematically illustrated in Figure 4-6. The characteristic features of the five types are as follows:

1. The leaking is concentrated in apertures in an otherwise tight fracture. This condition mainly occurs in horizontal fractures in the gneiss granite. This type of water inflow is illustrated in Figure 4-7.
2. The water inflow occurs along horizontal or sub-horizontal fractures, often as a distinct flow without noticeable apertures.
3. The water leakage occurs in steep dipping fractures, generally as distinct inflows at a lower boundary of the fracture or on the tunnel floor. This type of inflow is illustrated in Figure 4-8.
4. Distinct inflows at crossings between two or more fractures. Generally, no inflows or only very small ones occur along individual fracture planes outside the crossing-points.
5. The water leakage occurs as a diffuse inflow or flows out of the rock on a brad front. In general, no distinction between different inflows is noticeable and no correspondence between a certain inflow and a certain fracture is to be found. This type of inflow is illustrated in Figure 4-9.

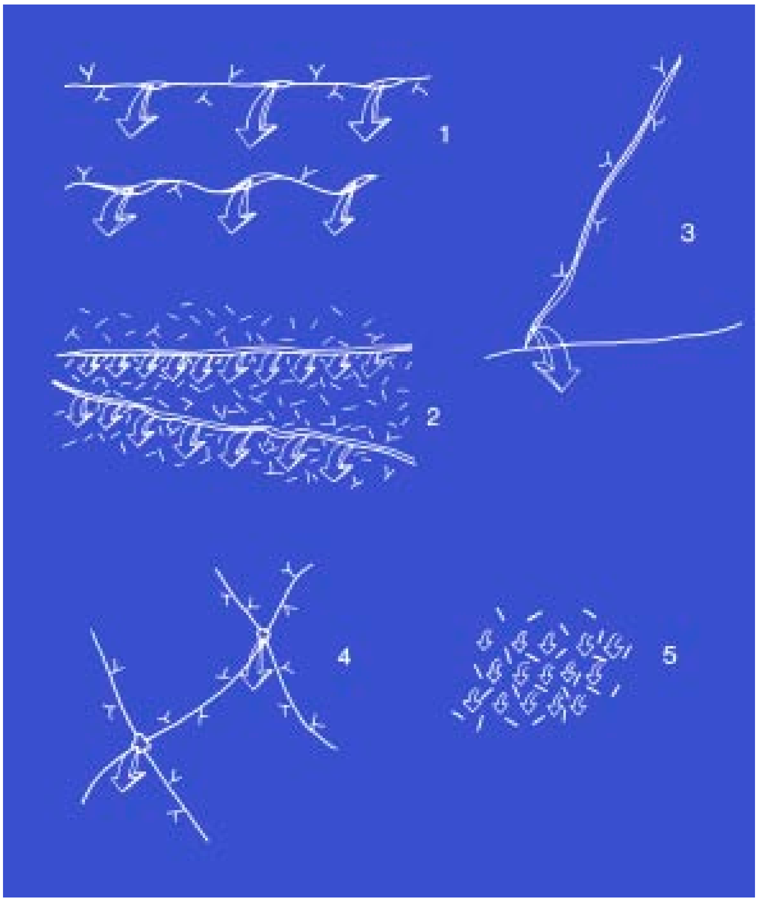


***Figure 4-6.** Schematic illustration of the different appearances of water inflows in the Forsmark 1 and 2 tunnel /Carlsson and Olsson 1977/.*

One of these types of leakage was found to usually predominates in the inflow pattern in each of the ten zones described in Table 4-1. But, while it was obvious that one of the types predominated, this type was not alone responsible for the whole inflow. In most zones, all the five types of leakage were to be found.

Rock support in percentage of the total length of the tunnel is shown in Table 4-3. A 6-metre long reinforced in situ cast concrete arch supports the portal in the surge basin, and an 8-metre long concrete arch was concreted at the outlet in the bio test lake.

Table 4-3. Rock support* in percentage of the total length of the Forsmark 1 and 2 tunnel (1,921 m). 65% of the tunnel is unsupported.

Rock support	Thickness, mm	Reinforcement	Percentage (%)
Un-reinforced shotcrete	50–60		1
Reinforced shotcrete	80–90	# φ 6 c 150	23
Reinforced shotcrete	150	# φ 6 c 150	8
Reinforced shotcrete arches	200–300	# φ 6 c 150 # φ 8 c 200 # φ 12 c 150	3
Total			35

* Note: Temporary (expansion-shells and grouted bolts) and permanent bolting not included. Type of rock bolts normally used for the permanent pattern and spot bolting: Grouted dowels, ϕ 25 KS40S, L: 4.0–4.8 m. Pattern bolting: 1 bolt/5 m^2 to 1 bolt/3 m^2. In total, about 16,000 permanent bolts were used in top heading and bench, and about 1,200 temporary rock bolts.

***Figure 4-7.** Water inflow from an aperture in a horizontal fracture. Large quantities of iron and manganese sediments, Forsmark 1 and 2 tunnel. Photo G Hansson/N.*

***Figure 4-8.** The water leakage in steep dipping fractures as distinct inflows at a lower boundary of the fracture, Forsmark 3 tunnel.*

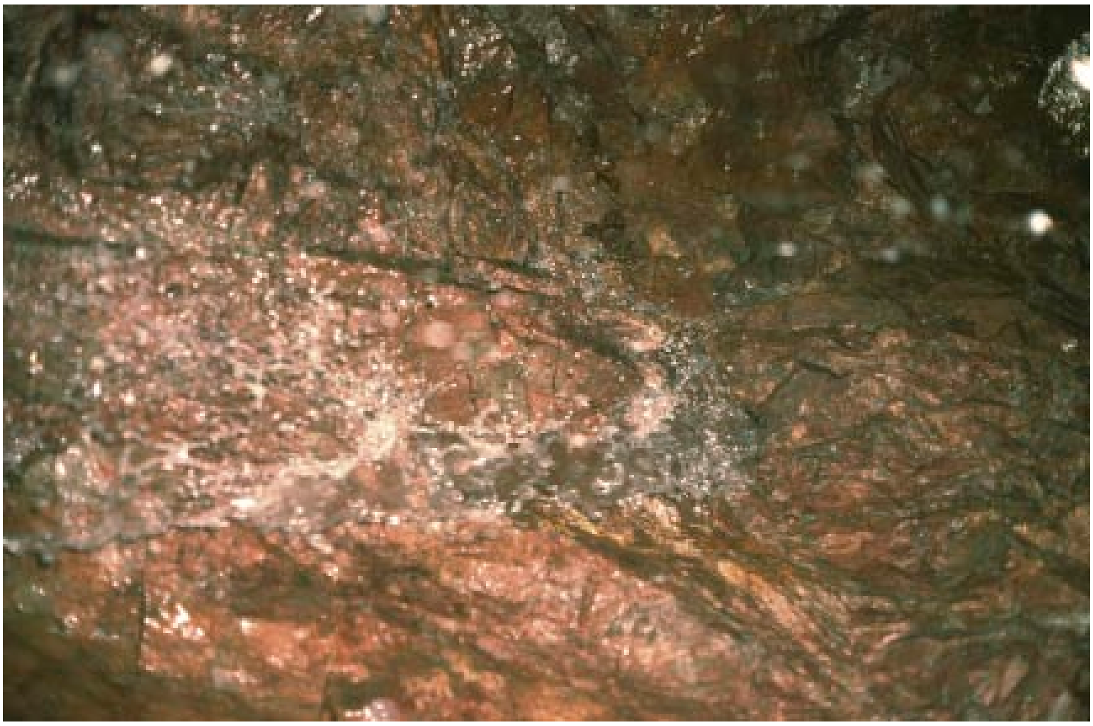


***Figure 4-9.** The water leakage occurs as a diffuse inflow or flows out of the rock mass on a broad front, Forsmark 3 tunnel.*

4.2.2 Forsmark 3 tunnel

Geological surveying, as for the mapping in the Forsmark 1 and 2 tunnel, was made in step with the tunnelling work, and observations were noted in records, which specified the geological parameters, and some sections also type extent of rock support. The surveying was made by an engineering geologist, a designer and countersigned by the work supervisor responsible for the relevant section of the tunnel /Hansen 1982/. Similar working procedure was also later applied for the SFR.

The tunnelling was made simultaneously from the shore and from the seaward side. During the driving of the discharge tunnel, pilot holes were employed, with percussion drilling and diamond core drilling from the face of the tunnel, to locate sections of inferior rock and to assess the need for grouting. A total of 1,290 m of percussion holes and about 320 m of coring holes were drilled. The percussion holes were drilled parallel to the bottom of the tunnel, mainly during normal shift work, whereas the coring holes were drilled upwards at an angle of about 5 degrees, at weekends.

Supports normally consisted of grouted dowels 25 KS40S of which the majority have a length of 4 m. A total of 7,268 permanent bolts were installed (121 tons). Mesh-reinforced and reinforced shotcrete, and in exceptional cases, in situ cast concrete structures were also employed. In the case of concrete structures, two steel shutters were made and used in section 2/511–2/545 for casting 9 concrete arches (described below in Section 4.3). The steel shutters were later employed for constructing eight, 4 metre wide permanent in situ cast arches, in fracture zones within section 2/147 and section 2/394. The characteristics of these zones are high fracture frequency (block size less than 200 mm), fracture fillings of clay, chlorite, sand, occasionally zeolite, and water inflows. The rock mass in which the fracture zones are embedded is mainly a brecciated granite with a banded structure striking E-W, dipping 60° S.

Table 4-4. Rock support* in percentage of the total length of the Forsmark 3 tunnel (approximately 2,920 m). 58% of the tunnel was unsupported.

Rock Support	Thickness, mm	Reinforcement	Percentage (%)
A. Un-reinforced shotcrete	80–90		13
B. Reinforced shotcrete	80–90	# φ 6 c 150	10
C. Un-reinforced shotcrete	100–200		3
D. Reinforced shotcrete	100–200	# φ 6 c 150	9
E. Reinforced shotcrete arches		# φ 6 c 150# φ 12 c 150	5
F. In situ cast concrete arches			2
Total:			42

*Note: Temporary and permanent bolting not included. Type of rock bolts used for the permanent pattern and spot bolting: Grouted dowels, φ 25 KS40S, L: 4–4.5 m.

On the basis of the observations made from the pilot hole drilling, pre-grouting with 66,825 kg of cement was carried out in 2,500 m of drill holes (26.7 kg per m drill hole) over a total length of 85 m (786 kg per m tunnel). The grouting was primarily cement, but cement-bentonite mixtures were also used. The normal number of holes drilled in the face was 10. No water-loss measurements were made after grouting, but leakage observations were made in conjunction with drilling for blasting. The total water inflow to the tunnel was 4,000 l/min.

In total, 675 m^3 of concrete was used for the concrete arches, 2,950 m^3 for shotcreting, and 70 tons of wire mesh were installed. Rock blasting along the whole line of the tunnel was carried out as careful blasting, using stick charges and Gurite around the contour of the tunnel section. This careful blasting left a better surface structure that, in turn, resulted in better stability and more favourable conditions for shotcreting. The average rate of driving per round was 4.9 m from the seaward side of the tunnel and 3.8 m from the shore. The difference in the driving rates was related to the use of different drilling rigs at the two faces, but the variation in rock structure had also its affect. The breakthrough took place on the 17th of May, 1982.

4.3 Rock falls in the Singö deformation zone – Forsmark 3 tunnel

In the section 2/511–2/545, which forms a part of the regional fault line, the Singö deformation zone, the rock consists mainly of reddish metavolcanics and aplitic metagranite, which has been brecciated and partly altered to clay. The average frequency of fractures was estimated at 10 per metre. In section 2/515–2/530, the rock was characterized as fragments of fresh rock separated by layers of altered, crushed and disintegrated rock, and in section 2/530–2/545, large amounts of rock had been transformed into clay. Nine samples of clay were taken and analysed at the Swedish Geological Survey, by means of X-ray diffraction, showing the minerals hydro-mica, chlorite, hematite-stained plagioclase feldspar, and quartz, but no swelling clay minerals. In the remainder of the tunnel and in the discharge tunnel for Forsmark 1 and 2, no swelling clays were found.

After a round had been blasted at section 2/545, there was a tendency for rock to break away above the theoretical roof of the tunnel, so an approximately 2 m wide, singly reinforced arch of shotcrete was constructed and singly reinforced shotcrete was also placed back up the tunnel, over a distance of about 10 metres from the arch, to protect those working in the area. Rock ahead of the face was investigated by drilling a 23 m long pilot hole collared just above the bottom of the tunnel. On the basis of the observations made from the pilot hole, a decision was taken to blast a new, 4.5 m deep round. The holes for the round were located one metre inside the theoretical contour of the tunnel section and the size of the charge was reduced.

After the round had been blasted, the drilled holes (half pipes) could be seen clearly, but the rock started to give away after about three hours. Rock falls started at the tunnel face, then from the roof and finally from the walls. All surfaces were shotcreted, but rock falls continued and arching started. A decision was then taken to make steel arches. Shotcreting was then carried out on three shifts while the arches were being made and the arch was stabilized after a time. Further rock falls were observed, however, and before the steel arches were placed in position, cleaning work was carried out and the bottom slab for the arches was cast.

One 4 m section of steel shuttering was erected and casting could be started. But new rock falls occurred during casting, of which the largest fall involved 200 m^3 of rock from the face and the left-hand side of the tunnel. The shuttering was cleaned and filling the arch with concrete prevented further rock falls. At the worst point, the arch extended to 7.5 m above the theoretical roof of the tunnel. The total cover of rock (to the sea bottom) at this section amounted to about 58 m. The rock fall and the support are illustrated in Figures 4-10, 4-11, and 4-12. The method used to support this section is described below and in Figure 4-13.

The area ahead of the steel shutter was filled to the upper edge of the shutter with rock, and wooden stop ends were constructed between the steel shutter and the arch of shotcrete at the roof of the tunnel. Several concreting pipes were introduced into the arch and it was filled with concrete. Supporting work was carried out about 7 m ahead of the steel shutter, with reinforced shotcrete on the roof and walls, down to the bottom of the tunnel.

Holes were drilled to the top of the arch and the top surface of the concrete was checked. Excavation was performed to release the shutter and the shotcrete support was continued down to the bottom of the tunnel. The steel shutter was removed and blocks of rock projecting into the cast concrete were chiselled out and the holes were repaired with shotcrete. Excavation was started in stages, alternated with shotcreting between section 2/538 and the tunnel face.

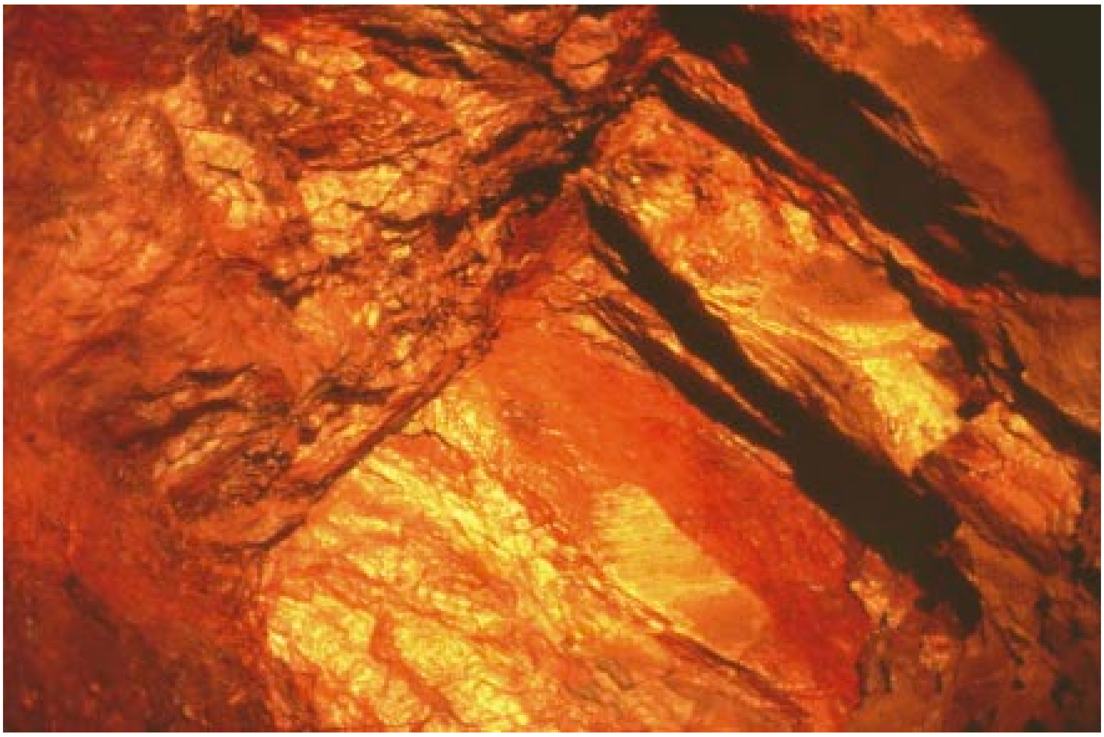


Figure 4-10. *Overbreak of some metres starts to develop in section 2/545 in the Singö deformation zone, Forsmark 3 tunnel.*

Figure 4-11. a. The steel arch placed in position within the overbreak area in section 2/545 in the Singö fault, Forsmark 3 tunnel. b. Further rock fall occurred after the erection of the steel arch; section 2/545 in the Singö deformation zone, Forsmark 3 tunnel.

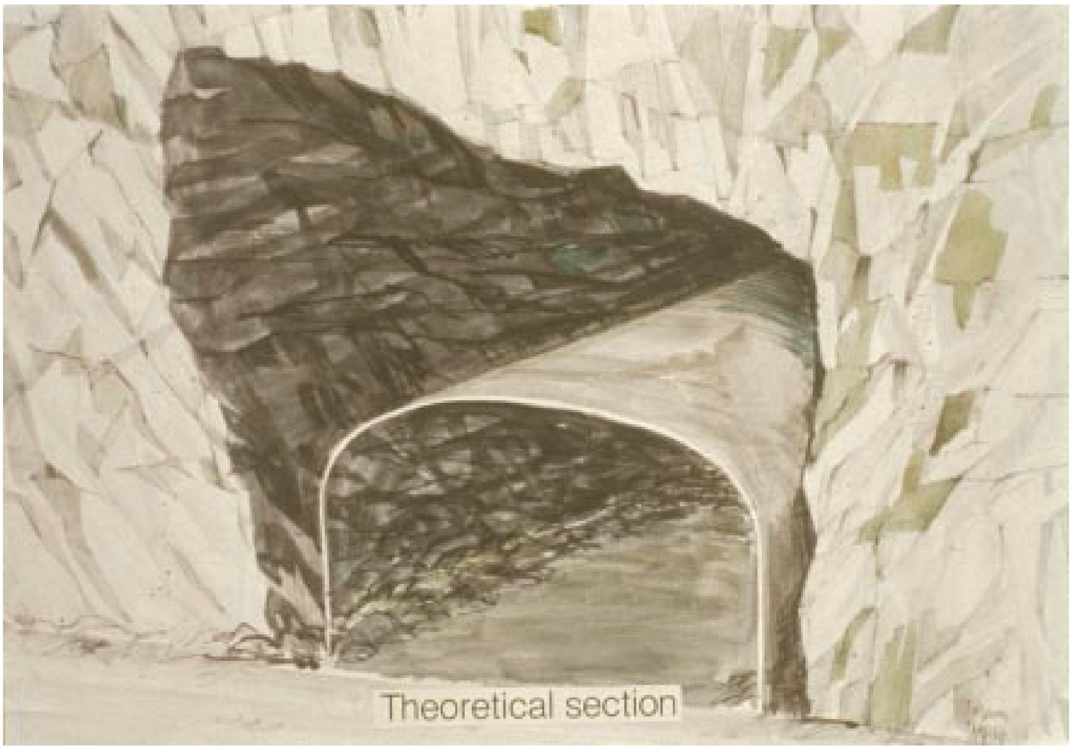


Figure 4-12. *Maximum overbreak amounted to 7.5 m above the theoretical roof of the tunnel; section 2/545 in the Singö deformation zone, Forsmark 3 tunnel. The artist's view of the overbreak should be seen in the direction of driving. /Carlsson and Olsson 1983/.*

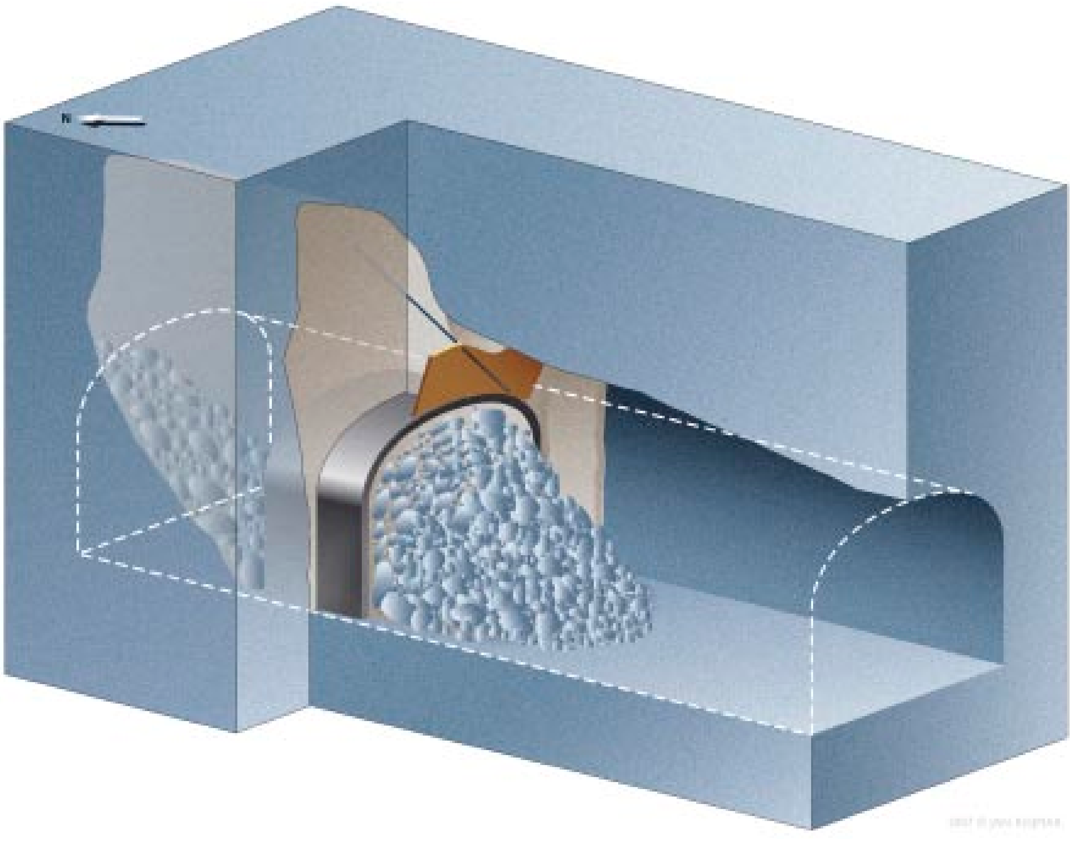


Figure 4-13. *The method used to support section 2/545 in the Singö deformation zone, Forsmark 3 tunnel /modified after Carlsson and Larsson 1983/.*

The concrete inside the section of the shutter was removed by careful blasting. Bolts were fixed in a radial form from the face of the tunnel at an upward angle of 45 degrees from the horizontal. Three 25 m long pilot holes were drilled, of which two were in the sides and one in the middle of the tunnel. The shotcreting unit was in position and a new steel shutter was ready before driving was continued.

The next round was blasted with careful blasting, the steel shutter was erected and a 400 mm, un-reinforced concrete arch was cast immediately. The remaining driving through the Singö deformation zone was achieved without noteworthy problems /Carlsson and Larsson 1983, Carlsson and Olsson 1984/. The rock support applied within the Singö deformation zone in the Forsmark 1, 2 and 3 tunnels is shown in Table 4-5.

Table 4-5. Supported tunnel length as percentage of total length of Singö deformation zone (200 m) in the tunnels of Units 1, 2 and 3 respectively /Carlsson et al. 1985/.

Type of support	Permanent rock Bolts* %	Shotcrete %	Shotcrete %	Shotcrete arches %	In situ cast concrete arches** %	Grouting %
Description	φ 25; L: 4 m	Un-reinforced; T: 100–200 mm	T: 80–90 mm; # φ 6 c 150	T: 80–90 mm # φ 6 c 150 alternatively T: 200 mm # φ 12 c 150		
Units 1 and 2 (top heading)	67	65	4	16	–	10
Unit 3	66	36	24	17	17	13

* Note: Number of permanent bolts: 460; 1bolt/ 8 m^2.

** Note: Steel shuttering.

Note: Temporary bolts, normally 1 bolt/ 13 m^2.

5 Site location and underground layout of SFR

5.1 Introduction

In 1980, SKB started the planning of the SFR and in 1981, an area in the Baltic Sea close to the Forsmark harbour was selected for further site investigations.

In June 1983, the Swedish Government granted the Swedish Nuclear Fuel and Waste Management Company (SKB) a license to build and operate a facility to be known as the SFR for the final disposal of low and intermediate level reactor waste from all the Swedish nuclear power plants.

SKB commissioned the Swedish State Power Board (Vattenfall) to plan, design and build SFR. The underground excavation works started in October 1983, and was finalised in May 1986. The repository is prepared for an extension and may be built in two phases, if needed, and the first phase was commissioned in 1988 with the first deposition of waste in April 1988.

5.2 Site selection

The results of investigations of possible and alternative sites meeting the requirements indicated that Forsmark offered the overall the best conditions for location of the SFR. In addition, the geological conditions in the Forsmark area were well known as a result of extensive investigations during the planning and construction of the Forsmark nuclear power plant, which also included the excavation of the two undersea discharge tunnels for cooling water as described earlier.

The rock caverns are built under the sea, about 1,000 m from Forsmark harbour. The rock cover is approximately 60 to 65 m from the top of the caverns to the seabed. The lowest level of the repository comprising the bottom of the silo, and a rock drainage basin is located about 140 m below seabed.

5.3 Site investigations

A comprehensive site investigation programme, which included both regional and detailed investigations, was carried out. The regional investigations commenced with a review of earlier geological studies of the area, these studies then being supplemented by a geological survey to elucidate the regional stratigraphy and the local rock distribution and occurrences of zones of weakness.

Detailed studies were then instituted with a network of seismic profiles, after which the zones of weakness indicated by the regional and seismic investigations were examined more closely by means of rock drilling. The seismic investigations were then intensified, further networks of seismic profiles were made, and the intensity of the rock drilling was increased.

The seismic profiling and core drilling were performed from the ice-covered sea and from offshore platforms (Figure 5-1). Seventeen diamond drill holes, with a total length of about 2,800 m, were sunk for the SFR. In all, about 15 km of seismic profiling has been carried out within the area of the SFR. Exploratory drilling was also performed from rock caverns and tunnels, penetrating a total of more than 1,600 m, were made from underground excavations (cf. Figures 3-2 and 3-3).

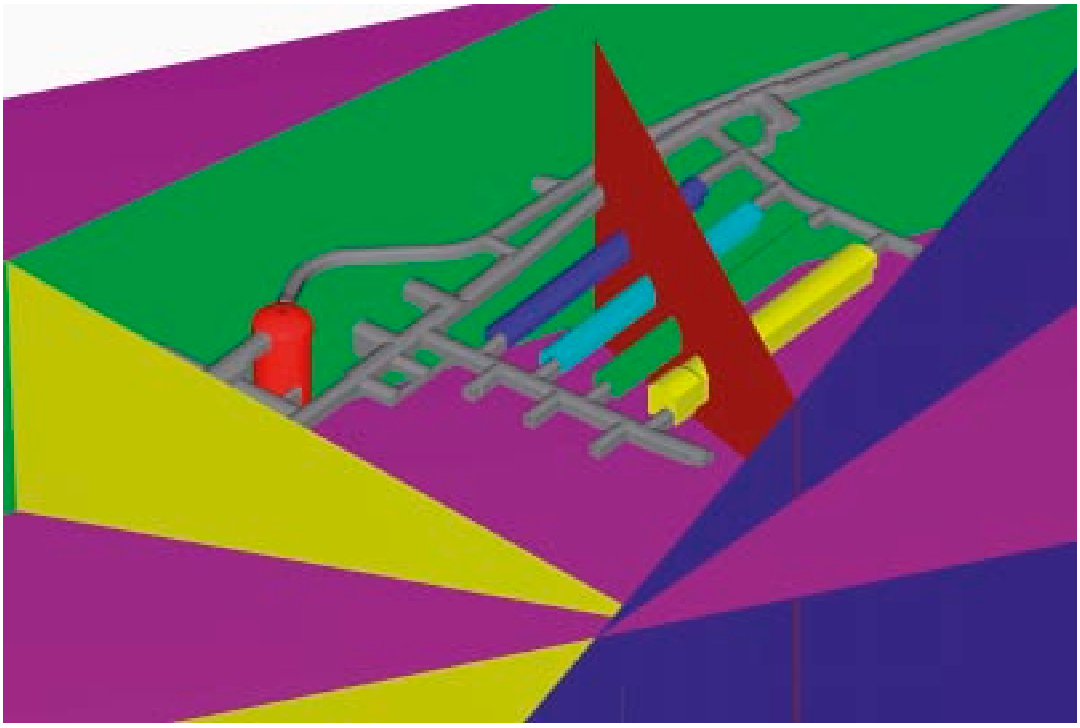
***Figure 5-1.** Offshore platforms were used for the exploratory core drillings for the SFR.*

In addition to standard core-logging routines, conventional water pressure tests, falling-head tests, transient injection tests and pressure build-up tests were utilized. In some of the boreholes, geophysical logging and radar reflection surveys were also used. In situ rock stress measurements were made in three boreholes, of which two were performed from underground excavations.

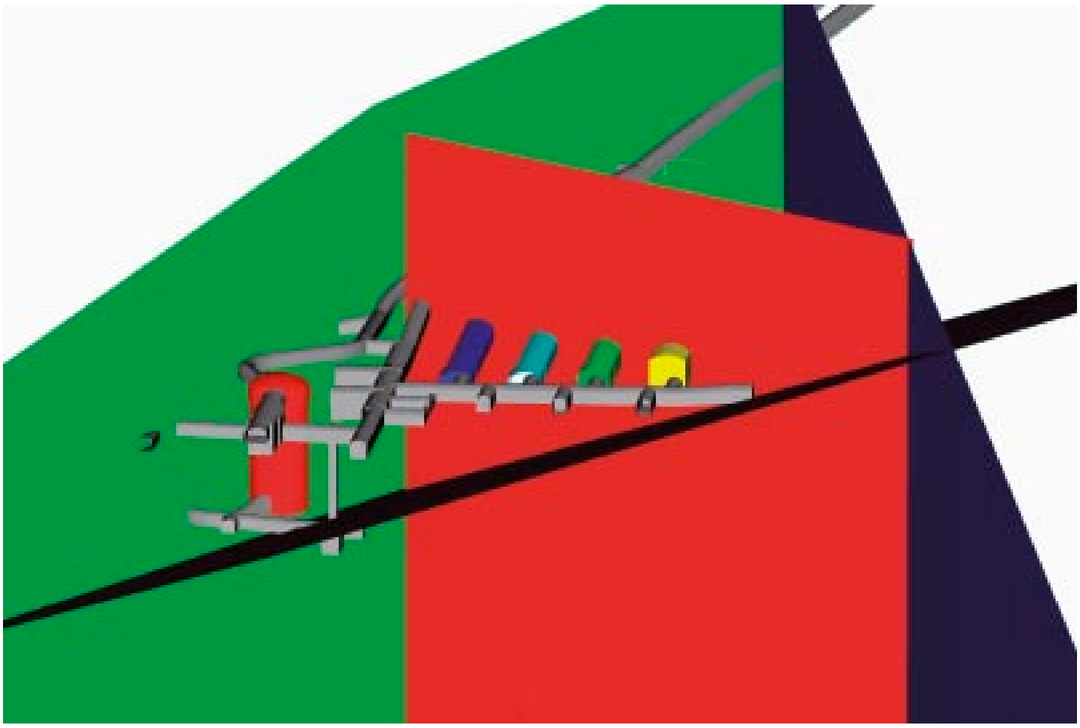
Engineering geological surveying consisting of investigations of structural geology, fractures, water inflows, etc has followed the tunnelling work continuously /Christiansson and Eriksson 1984, Christiansson 1985, 1986, Christiansson and Granlund 1986/. The work involved detailed mapping of tunnel walls and faces, as well as geological follow-up of exploratory drilling and pilot drilling ahead of the excavation faces. Deformation measurements were also included in the surveying to check the stability of the SFR tunnels within the Singö deformation zone and to confirm the design of the silo. Measurement sections were installed in the central part of the fault in both access tunnels, where the rock is of poor quality. One measurement section was located in a part of the deformation zone that shows a rock mass of good quality to enable comparative measurements to be made /Christiansson and Bolvede 1987/.

The purpose of the SFR demanded a safety assessment to be carried out. A separate investigation programme of geological, hydrogeological and hydro-geochemical conditions was carried out in parallel to the ongoing constructions. This included the drilling of in total 12-cored holes with the purpose to more in detail determine the locations and properties of various deformation zones identified during the site investigations and avoided by the layout of the facility. These deformation zones gave together with the sea above the facility the hydraulic boundary conditions. Monitoring for hydrogeological and hydro-geochemical purposes was initiated in many of the boreholes, and some of them are still included in the monitoring system of the SFR.

The borehole results together with the results from the engineering geological surveys were used to establish a 3D model of the site (cf. Figure 5-2). The integration of tunnel and borehole information to conclude a site model relied to a high degree on experiences gained from applied research by SKB and sister organizations. The works ongoing by that time on geosphere studies in the abundant mine in Stripa in central Sweden provided valuable experience to the investigations and modelling of the SFR site and boundary conditions.



View I. A close-up view of the fracture zones and the layout of the deposition tunnels at SFR.
ZONES: Purple= H2. Dark blue= 3. Dark red= 6. Yellow= 8. Green= 9.
TUNNELS: Grey= Access. Red= SILO. Dark blue= BTF1. Light blue= BTF2. Green= BLA. Yellow= BMA



View II. A close-up view of zones H2 and 6 and the layout of the deposition tunnels. Zone H2 is a sub-horizontal zone, which intersects the access tunnels below the SILO. Zone 6 is a vertical zone that intersects the following deposition tunnels: BTF1, BTF2, BLA and BMA.
ZONES: Black= H2. Dark blue= 3. Red= 6. Green= 9.
TUNNELS: Grey= Access. Red= SILO. Dark blue= BTF1. Light blue= BTF2. Green= BLA. Yellow= BMA

Figure 5-2. *An example of a 3D model of SFR based on results from surfaces borings and engineering geological survey in the tunnels and underground caverns.*

5.4 Underground layout of the SFR

The portaling of the access tunnels was located on the rocky islet, Österblänkarna, situated between the Forsmark harbour and the Stora Asphällan Island (cf. Figure 5-3). The islet constitutes a continuous rock outcrop with an area of 150×250 m in an N–S direction. In the layout, the portaling of each tunnel was placed close to each other in the same open cut (cf. Figure 5-4). The tunnels were sited parallel to each other with a short distance between themselves (15 m). The rock caverns at repository depth were consequently placed beside the tunnels and with their longitudinal axis parallel to the access tunnels. The advantages with the location of the open cut on the Österblänkarna were

- Short access tunnels.
- Cross passages between the tunnels improved the escape safety and, in addition, a flexible traffic situation during e.g. installation works in one tunnel.
- A common and uniform industrial area for the harbour and the surface facilities of the SFR admitting an effective operation of activities, but also simplified the security and guarding of the area.
- The SFR industrial area could be filled and levelled with surplus rock material from the underground excavations which resulted in short transport distances.
- A more environmentally friendly solution in that, the forest stand of the Stora Asphällan Island could be spared which also was a desirable requirement from the architects of the administrative province and municipality.

The disadvantage with the location on the Österblänkarna was that the tunnels would intersect the Singö deformation zone only 300 m from the collaring. In order to have a sufficient rock cover, the operation tunnel had to be driven with a slope angle of 12%, and the construction tunnel of 14% along the first 300 m. Later, during the operation of the underground facility, problems with the waste transport vehicles came up along the first 300 m due to the steep

Figure 5-3. *The location of the portaling of the SFR access tunnels on the rocky islet, Österblänkarna. Photo G Hansson/N.*

slope angle. The vehicle has seven pair of wheels with driving on three shafts. The tyres were skidding on the concrete paving which resulted in an overload of the driving system. However, the problems were completely overcome by re-designing different components of the vehicle, and asphalting of the concrete pavement. Another disadvantage is, since the two tunnels has a jointed open cut, a theoretical, somewhat increased risk for flooding of the facility at extreme high sea water levels /Larsson 1996/.

The layout of the access tunnels as described above has proved to be successful by giving a logical and a well-arranged plant with a good possibility for expansion.

5.4.1 Repository area

The principles for storage of the waste were established in an early stage of the project, and was not changed or modified during the course of the project implementation. Even the design of the rock caverns and the silo was generally kept unmodified during the planning and the construction phases, and this lead to a smooth and relatively speedy detailed design. Since SFR was the first facility of its kind, many detailed design solutions had to be elaborated without using any specific models acquired by previous experience, but most of the problems could be solved by routine /Larsson 1996/.

The siting in level of the repository caverns was, however, adjusted to the results of the supplementary drilling investigations that were carried out during the summer and autumn of 1983. In addition, a complete re-location of the relative position of the repository caverns as a result of the revised layout and design of the access tunnels, but also due to the results of the exploratory drillings.

The length and the cross-sections of the rock caverns were somewhat adjusted, mainly with respect to the equipment for handling of the waste packages, but the layout of the silo was kept unmodified.

Figure 5-4. *The access tunnels of the SFR are sited parallel to each other with a short distance in between (15 m).*

In spite of the fact that the access tunnels became shorter in length, the repository area could be lowered and the rock cover was thereby increased, resulting in better rock quality and most probably less water inflow. The lowering was limited to 6 m (a lowering up to 20 m was possible) depending on an expected sub-horizontal zone with inferior rock quality below the bottom of the silo. When the zone was encountered in the lower rock drainage basin and in the gable of the lower construction tunnel, heavy water inflow occurred. The dip direction of the zone will influence the level of the bottom of a future Silo No 2, but Silo No 1 was not influenced at all. Drillings were carried out to investigate the extension of the zone, and the result of the drillings showed that the zone was situated at a safe distance below the bottom of the silo, and in addition it decreased rapidly in thickness to finally disappear below the centre of the silo /Larsson 1996/.

5.4.2 Flexibility

One of the objectives of the layout work was to elaborate a flexible layout enabling

- Adjustment to potential changes in space requirements during the planning and construction stages. The more detail layout for the repository area was commenced at the same time as the work started with the open cut for the access tunnels. The layout of the repository area was presented at the time when the access tunnels had reached half of their length
- Adjustment to the successive build-up of knowledge of the geological conditions obtained from the excavation works and supplementary geological investigations. The supplementary investigations were carried out from offshore platforms concurrently with the excavation works of the open cut for the access tunnels. In addition, pilot drillings were to be carried out from the tunnel faces, and from niches in the tunnel walls to locate possible water-bearing fractures and faults.
- To allow several possibilities for future expansion of the plant. At the time of the layout work there were a number of uncertainties regarding the amount type of waste as well as regarding future waste.

The philosophy behind the flexible layout work resulted in several positive results, such as

- The length of the rock caverns did not have to be decided before the excavation had reached the cross passage. There was even a possibility to allow minor adjustments of the length of the caverns after the excavation of the cross passage had started.
- From a layout point of view, there was a possibility to increase the number of rock caverns. However, one water-bearing fracture zone is present behind the rock caverns for intermediate rector waste that limits the possibility for expansion in this direction.
- The size of the underground service buildings could be changed if needed.
- The distance between the silo and surrounding tunnels and buildings was set by a comfortable margin. Therefore, it was possible to re-locate the silo by 10 m in the horizontal plane when a steep dipping discontinuity was found that might have caused increased seepage and maybe also stability problems in the wall.
- There is a substantial amount of freedom to expand the repository in optional directions. However, the layout will to a great extent be governed by the geological conditions.

The most important prerequisite of the economical and technical use of the flexible layout was that the project organisation was adapted to such an approach. It may be concluded, that for the planning and construction of the SFR, the procedure of decision-making and information was effective and speedy. A good cooperation and mutual understanding prevailed between the Owner and the Contractor. It was also important to apply a remuneration system for additional work, so that changes as described above could be accomplished at reasonable costs, as well as allowing the Owner credit for a reasonable price reduction at reduced workload /Larsson 1996/.

No additional costs or loss of time came up from the re-location of the silo. The reason for that was that the decision of the re-location was made one day and revised drawings were available the following day. The revision comprised adjusted direction of the connection tunnels for the silo and, to some degree, a prolongation of the tunnels. The latter involved of course certain additional costs. The excavation of the upper connection tunnel to the silo was interrupted for a shorter period of time in order to carry out core drillings, and this operation caused a certain delay of the silo works. However, the resources could be used for excavation of other tunnel faces /Larsson 1996/.

There are also examples when the flexibility was insufficient. The dimension of the open cut for the access tunnels was too small resulting in a too small ventilation building. The design as well as the installations became unnecessarily complicated. The reason for this was that the system engineering was not developed sufficiently enough at the time for start of the construction works. Another example is that the conduits for ventilation were changed in the operation tunnel from plate to concrete at the same time as the airflow was increased. This resulted in larger space requirement for the conduits, which in turn adversely reduced the traffic space. On the other hand, advantages were gained by lower construction cost, reduced maintenance and increased life of the conduits and in addition, more environmentally cleaner solution /Larsson 1996/.

Figure 5-2 shows the rock excavations included in the first construction phase. The quantity of rock excavated for the first phase amounts to 430,000 m^3 of which 115,000 m^3 is attributed to the access tunnels down to the cross passage No.3, 110,000 m^3 relates to the four rock caverns, 50,000 m^3 to the silo, and 140,000 m^3 relates to other spaces such as service facilities, communication tunnels and shafts; in all, the combined length of tunnels, rock chambers, silo and shafts is 4,500 m.

As shown by Figure 5-2 the underground structure of the first construction phase is composed of communication tunnels, four rock chambers of varying size and shape, and one silo. The layout of the storage facilities is dependent on the nature and quantity of the waste, its composition and handling considerations. Different types of storage space for various kinds of waste have therefore been found to be appropriate.

Most of the waste, primarily solidified ion-exchange resins are stored in the silo repository. This part is designed in the form of a rock cavern, 69 m high and 30 m in diameter, housing a 50-m high slip-formed concrete silo. The concrete silo is built on a bed of sand and bentonite, and the space between the walls and the rock is filled with pure bentonite. Figure 5-5 shows the slip-forming work seen from the top of the silo. Other parts of the repository containing waste with low activity contents have no barriers other than the surrounding rock and the waste package itself.

Figure 5-5. *Slip-forming work, SFR. The photo shows the slip-forming work viewed from the top of the silo. Photo G Hansson/N.*

6 Experiences from underground works – SFR

6.1 Construction period and costs

The total time for completion of the first phase of the SFR – i.e. the time from the start of the planning and design work until disposal of the waste can begin – was seven years. The first phase of the SFR was commissioned in 1988. Work on a detailed testing programme and safety studies was being carried out concurrently with the construction and excavation work.

The total cost of the first phase of the SFR amounted to about 740 MSEK of which 75% represents the cost for civil works (rock excavation: 24%, other civil works: 26%, installation and vehicles 29% and management, supervision etc: 21%). The costs are quoted at 1988 prices /Larsson 1996/.

6.2 Construction requirements

As may be understood from the description of the method of driving the access tunnels given in Section 6.5, the method of driving incorporated unusually high safety allowances; allowances which were not in direct proportion to earlier experience or to the results of investigations which applied to these access tunnels. It may be assumed on good grounds that problems of a normal nature that usually occur in conjunction with, for example, the driving of a hydropower tunnel, or something of that kind, would be judged in a considerably different light if the same type of problem were to occur in these access tunnels.

6.3 Tunnelling, construction equipment and personnel

The two parallel, about 1,000 and 1,200 m long access tunnels run at an inclination of 1 in 10 (except for the first 300 m), beginning at the open cut at Österblänkarna and ending about 50 below the sea bed. The theoretical cross-sectional areas of the tunnels are 48 m^2 (construction tunnel) and 64 m^2 (operation tunnel). The various rock caverns in the repository are linked to each other by a tunnel system (see Figure 5-2). The cross-sectional area of these tunnels varies from 50 to 80 m^2. Four rock caverns were excavated in the first construction phase. Each is 160 m long and varies in width from 14 to 20 m; and in height, from 10 to 19 m. The cross-sectional area of the largest cavern is approximately 320 m^2.

Work was started simultaneously on the two access tunnels. According to Swedish State Power Board practice, consecutive working was employed during tunnelling, part of the work force engaged in drilling, charging and blasting also helping in mucking out, but there were also additional truck drivers. Tunnelling was carried out in two 7.2-h shifts five days a week; scaling and rock support work normally was carried out on a third shift during the night. The total number of personnel working underground was 65, of whom 22 were occupied with rock support work /Carlsson and Hedman 1986/.

The tunnels were driven to full section (except for the rock caverns) using Atlas Copco TH531 drilling jumbos equipped with four BUT35 booms fitted with COP 1038 HD rigs with 18-ft feed. Each rig was fitted with charging cradle on a hydraulic boom. The largest rock cavern was excavated with one top heading and two benches, 90 m^2 each. The other three chambers were excavated with one top heading and one bench. The heading and benching were carried out with the aid of ordinary Atlas Copco jumbos.

Electrical powered BröytX-4 excavators were used to muck out rock; and Engsson 666B, Kiruna K250 and K501 trucks were used for rock haulage. The mechanized scaling was carried out using a Liebherr 941A and Åkerman H12 equipped with a Montabert BRH125 (Figure 6-1).

Atlas Copco Boltec 540/22 mechanized rock bolting equipment was used for the bolting operations (Figure 6-2), and Furuholmen and Hoyer-Ellefsen Robocon remote-controlled shotcreting system were chosen for the shotcreting. Most of the shotcrete was applied as ready-mixed fibre shotcrete. Grouting was carried out by a mobile unit fitted with a working platform and equipment for simultaneous grouting of four holes.

Figure 6-1. *Mechanized scaling with a hydraulic hammer in the SFR tunnels.*

Figure 6-2. *Mechanized rock bolting equipment used for bolting operations in the SFR.*

The tunnels were driven using contour blasting with a damage zone limit of 0.3 m for the operation tunnel and 0.6 m for the construction tunnel. The blasting pattern adopted in the construction tunnel consisted normally of 83 plus 3 holes, and 109 plus 3 holes in the operation tunnel. Figure 6-3 shows the basic blasting pattern employed in the operation tunnel, but the pattern was, of course, modified continually during the tunnelling in order to optimize the fragmentation. Consequently the number of holes varied from section to section.

The borehole depth was 4.9 m with an advance per round of 4.7 m in each tunnel. The borehole diameter was 48 mm and the burn holes were 102 mm in diameter. The specific drilling was amounted to 1.8–1.9 m/m^3.

The consumption of explosives per cubic metre of rock was 1.3 kg in the tunnels. The explosives used were the Nitro Nobel products Dynamex, Nabit and Gurit (for contour holes only). The explosive Nitro Nobel Emulite was also used in the excavation of the silo.

The cycle time for advancing the faces of the access tunnels was 11–14 h, including drilling and charging which took 5 h. Work started on the charging of the round before the drilling was finished. Manual and mechanized scaling and mucking out took about 5–6 h. The cycle time should be viewed in the light of the fact that consecutive working was employed during the tunnelling.

The ventilation system was mainly based on 1.6 m diameter pipes delivering fresh air to the faces using 75 kW axial fans delivering 1,800 m^3 per min. After blasting the fumes were cleared in about 20 min /Carlsson and Hedman 1986/.

Table 6-1 summarizes the construction equipment used for the driving of the access tunnels and cross passages. The construction equipment was chosen for achievement of the highest capacities as known at that time, and was dimensioned in that way that all separate activities could be carried out simultaneously in the two tunnels (except for the mechanized scaling and shotcreting.

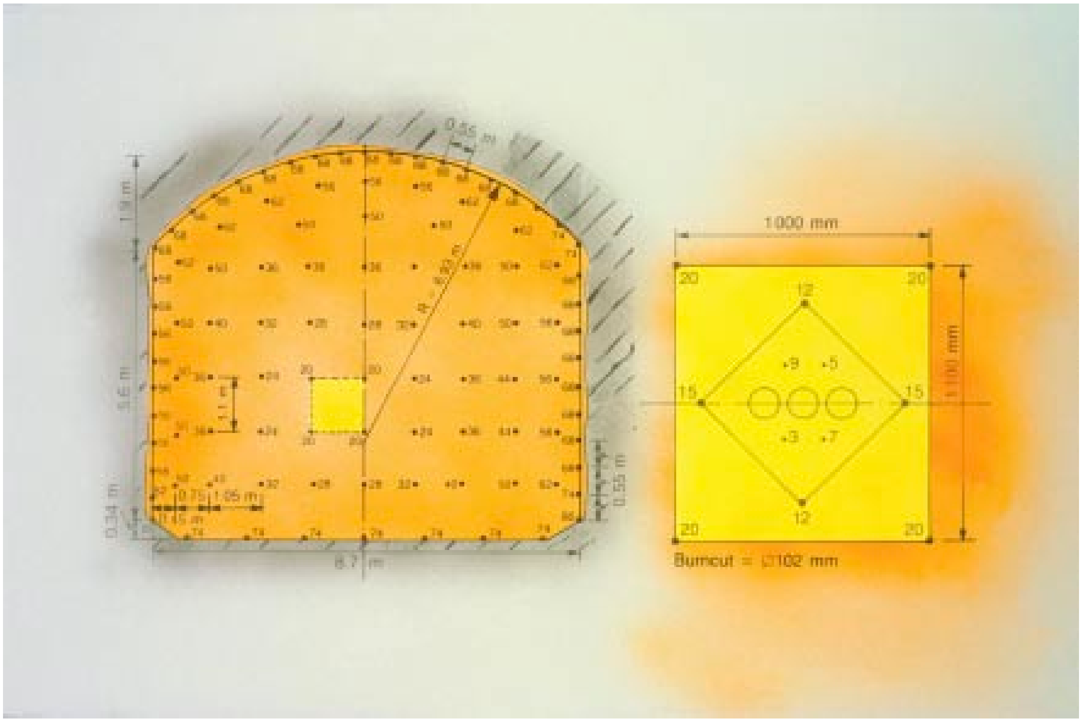


Figure 6-3. *The blasting pattern in the operation tunnel of the SFR and a close-up of the burn holes /Carlsson and Hedman 1986/.*

Table 6-1. A summary of the construction equipment used for the driving of the access tunnels and cross passages in the SFR.

Construction activity	Type of construction equipment
Drilling	Three Atlas Copco TH531 equipped with 4 BUT35 booms fitted with 18-ft feed and fitted with a charging cradle on a hydraulic boom for 5 m advance.
Mucking Out	Two electrical powered Bröyt X4 excavators with 2.0–2.5 m^3 bucket capacity.
	One Cat 988 A wheel-mounted loader with 4.5 m^3 bucket capacity.
	One Cat 966 C wheel-mounted loader with 2.3 m^3 bucket capacity.
Scaling	One Liebherr 941 A hydraulic excavator and one Åkerman H12 diesel hydraulic excavator both equipped with Montabert BRH125 hydraulic scaling rod.
Rock Haulage	Two Kiruna trucks, 35 tons.
	One Kiruna truck, 50 tons
	Four to seven Engsson 666B.
	A number of Cat trucks on stand-by.
Rock Support	One Atlas Copco Boltec 540/22 mechanized rock bolting equipment.
	One "Furuholmen – Forsmark" remote-controlled shotcreting system.
	One mobile unit fitted with a working platform and equipment for simultaneous borehole grouting (the unit was developed on site).
	Grout holes and drilling of holes for the Atlas Copco drilling rigs drilled expander bolts.
	The shotcrete was mixed at the concrete plant, originally erected for the concrete works for Forsmark 1.
Ventilation	Two 1.6 m diameter coated fabric tubes.
	Two 75 kW axial fans AMF 1,520 at the collaring, and two fans at a distance of 450 m from the collaring.

6.4 Tunnel support

The rock support types used for the SFR was based on the gradually development of the rock support system applied in the Forsmark tunnels /Larsson 1996/. Before the start of the underground excavation works, a rock support system was established showing different rock support types to be used for the SFR.

The rock support types were mainly composed of shotcreting and grouted dowels in various combinations depending on local stability conditions. Special rock support design was assumed to be made in cases where the established rock support types were deemed to be insufficient or inappropriate, such as in situ cast concrete arches. However, no guidelines were shown on the rock support drawings when a specific rock type should be used; the only indications given were directions such as "good rock", "high fracture frequency", "blocky rock mass" or similar. Rock mass classifications like Q- or RMR classification systems were not used.

As for the Forsmark tunnels, the concept of active design (Observational Method) was employed throughout the underground excavations for SFR.

In principle, seven rock support types for shotcreting were used for the rock support with a rising scale from S1 to S7, and four types for rock bolting and drains respectively. In addition two types were defined for the support of clay-filled fractures. The tables below illustrate the various rock support types for shotcreting and bolting used in the SFR /Larsson 1996/.

Table 6-2. Rock support types – shotcrete, SFR.

Rock support type shotcrete	Description
S1	Un-reinforced shotcrete; T: 30 mm (one layer)
S2	Un-reinforced shotcrete; T: 50 mm (two layers)
S3	Fibre reinforced shotcrete; T: 50 mm, Type EE 18 mm, percent by volume: 75kg/m^3
S4	Fibre reinforced shotcrete; T: 80 mm, Type EE 18 mm, percent by volume: 75 kg/m^3
S5	Mesh reinforced shotcrete; T: 100 mm, # φ 6.5 c 150; alternatively
	Bars # KS 40S φ 8 c 200
S6	Shotcrete arch; T: 200 mm, including rock anchoring
S7	Shotcrete arch; T: 300 mm, including rock anchoring

Table 6-3. Rock support types – rock bolting, SFR.

Type rock bolts	Description
Bx	Spot bolting; grouted deformed bars, φ 25, L: 3.8*
B1	Pattern bolting; 1 bolt/4 m^2, grouted deformed bars, φ 25, L: 3.8
B2	Pattern bolting; 1 bolt/2 m^2, grouted deformed bars, φ 25, L: 3.8
	Plate top anchoring in the roof**
B3	Pre-bolting; c/c 1 m

* The length was chosen with respect to the performance of the rock bolting equipment.

** Normally, the dowels were not fitted with top plates, but in areas where the adhesion between rock and shotcrete in the roof was judged to be poor, the dowels should have threaded ends and fitted with plates, spheres and nuts.

6.5 Excavation of the access tunnels and driving through the Singö deformation zone

The SFR access tunnels intersect the regional fault line, the Singö deformation zone; i.e. the same deformation zone crossed by the two undersea discharge tunnels for Forsmark 1, 2 and 3. At the SFR, the zone is dominated by foliated metasediments/metavolcanics with pegmatite, and mylonitic structure is common.

Thus, experience gained during the driving of those tunnels, especially where they pass through the deformation zone, in which conditions ranged from good rock conditions to inferior, local zones of weakness (and hence, experience ranged from moderate support works to solid, in situ cast structures), provided a good basis for the planning and preparation of the driving of the SFR access tunnels.

To avoid problems such as those encountered in the Unit 3 tunnel, unusually large allowances were made for the driving of the SFR access tunnels. Based on earlier experience, an operational schedule was established. The schedule comprised a lowering of the tunnel elevation at its intersection with the Singö deformation zone from 15 m down to 23 m below the seabed, exploratory drilling, pre-grouting, pre-bolting, reduced advance of driving per round, systematic bolting and reinforced arches of shotcrete, and the performance of deformation measurements.

Detailed investigations from the ice cover and from the offshore drilling platforms were also carried out before driving through the Singö deformation zone (cf. Figure 3-3). Three seismic profiles and six core drillings were made within the area before driving through the zone.

When the tunnelling was approaching the critical area, two horizontal diamond drill holes, 100 and 120 m in length, respectively, were drilled ahead of the tunnel faces. In addition, pilot drilling with the jumbos was carried out continuously with three 20 m long holes into the roof and walls. The water inflow into the pilot holes was measured in such a way that at least one round (4.7 m) overlapped previous pilot holes. The engineering geologist on site evaluated the rock fragments from the pilot drilling, flushing water and water inflow.

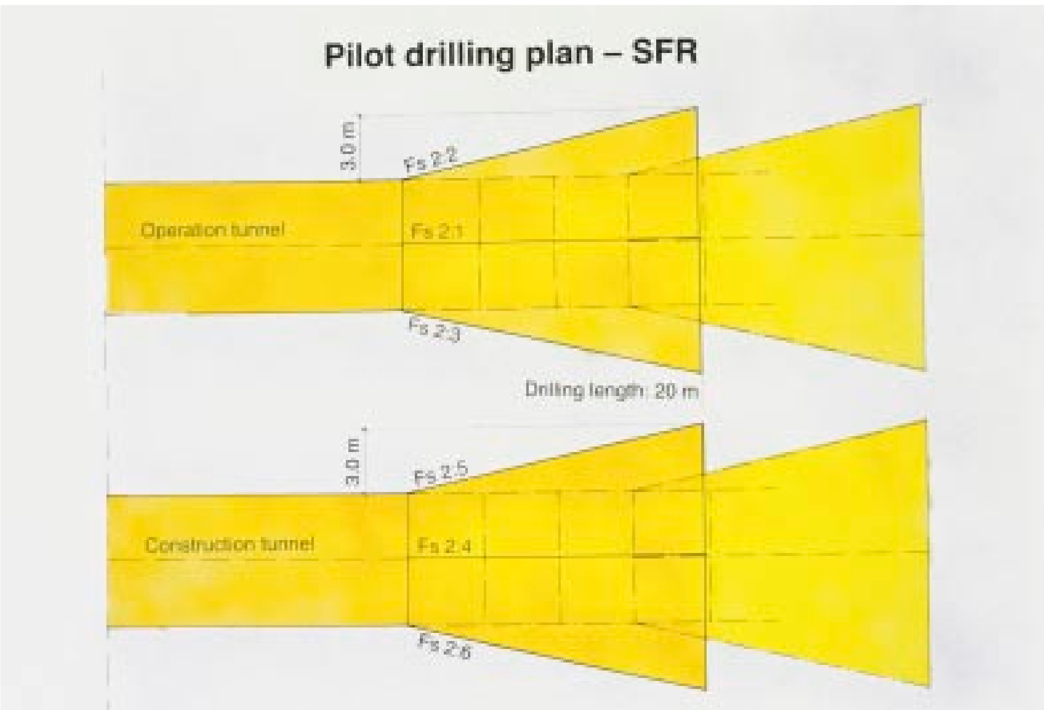
Based on information from the pilot drilling, pre-grouting was carried out from the tunnel face, with a double curtain at the crown and a single curtain at the bottom. Pre-grouting was performed when the quantity of water inflow exceeded 3–4 l per min from a 20-m borehole. The maximum water inflow in one single borehole amounted to 170 l/min and pre-grouting was carried out at 15 points along the deformation zone. The length of the boreholes was approximately 15 m and the grouting was carried out as campaign grouting for 2 h, with a final pressure of 1.5–2 MPa. About 1 T of rapid hardening cement was used for each round of grouting, on average.

The layout in plan and section of the pilot drilling is illustrated in Figure in 6-4, and the pre-grouting in section is given in Figure 6-5.

The steady-state water inflow within the Singö deformation zone after excavation amounted to approximately 60 l/min in the two tunnels, and the leakage was concentrated in a 12 m long section.

An evaluation of the performed grouting works in the Singö deformation zone, SFR was carried out by /Carlsson et al. 1987/. They concluded that the grouting was effective, and normally more than a 75% reduction of the rock mass permeability was achieved. The study also showed that when the grouting was performed both from the construction tunnel and from the operation tunnel, a much more improved result was obtained due to variations in cut angles between grout-holes and water-bearing fractures. When grouting from the two tunnels, the grout penetrated two to three times more water-bearing fractures.

Pre-bolting of the next round was carried out regularly, using approximately eight 5-m long cement grouted bolts. The pre-bolting proved to be very successful, so that even in areas of clay-mineralized and weathered rock, the contour of the roof was good. On one occasion when pre-bolting was omitted, some stability problems occurred. However, no problems were encountered in the connection with the next round of corresponding rock mass quality when pre-bolting was employed.



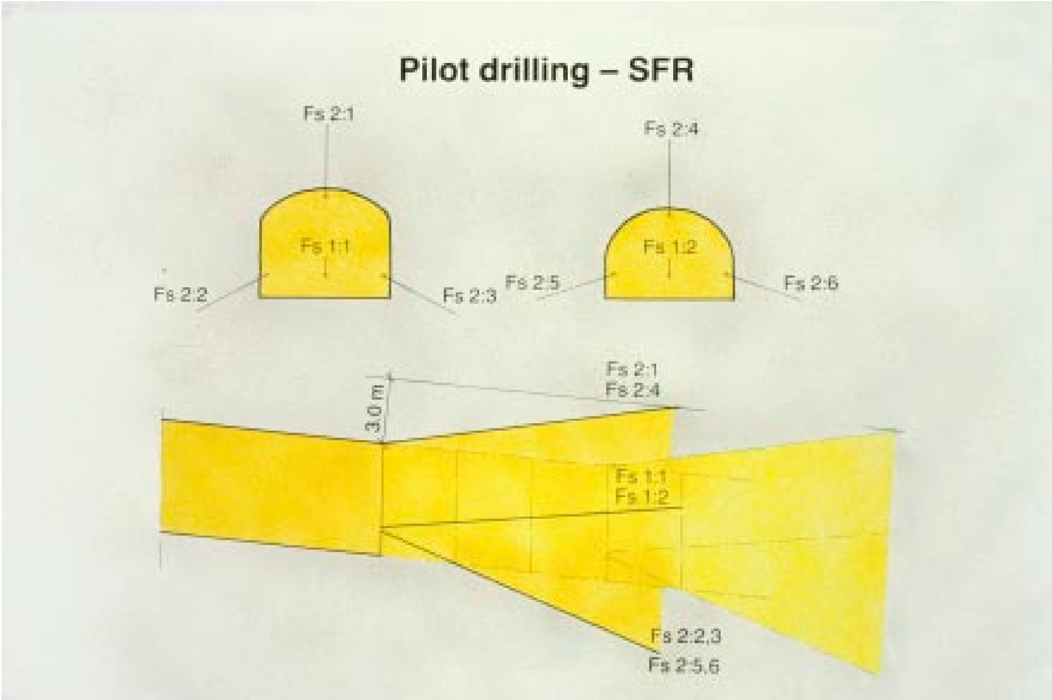


Figure 6-4. *Pilot drilling in plan (a) and section (b) through the Singö deformation zone in the access tunnels, SFR.*

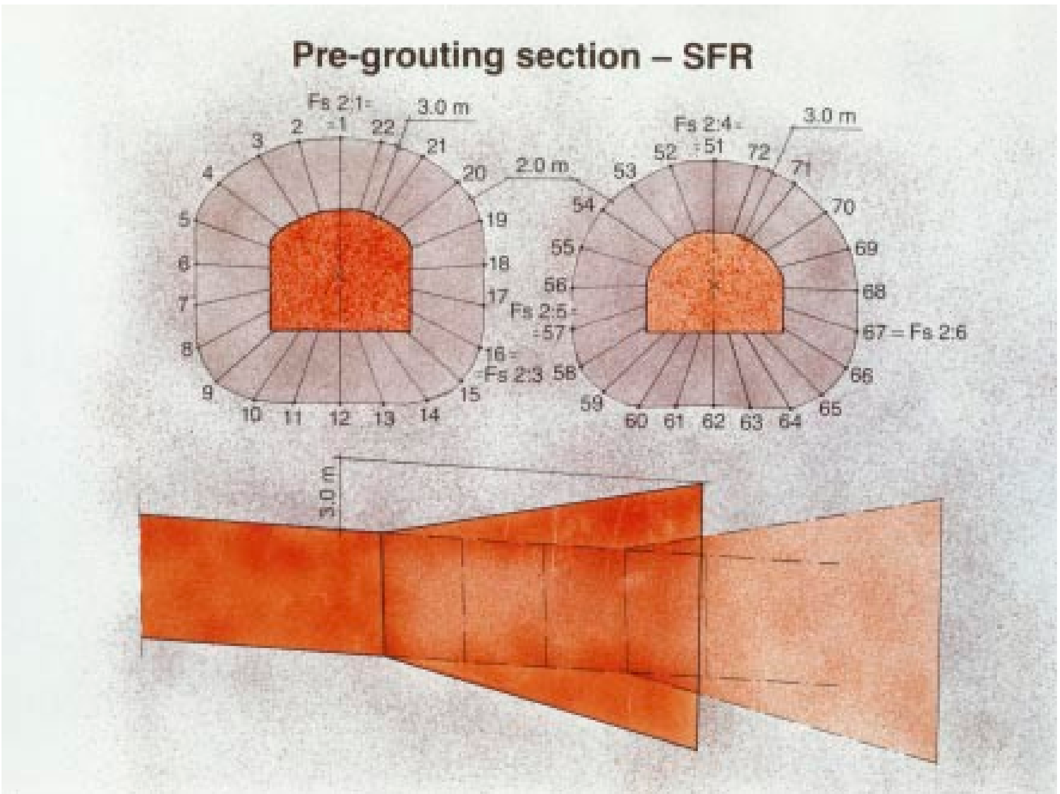


Figure 6-5. *Pre-grouting in section through the Singö deformation zone in the access tunnels, SFR.*

When the rock mass quality was judged to be inferior, the rate of driving per round was reduced from 4.9 to 3.0 m. A total of 13 rounds were reduced across the deformation zone due to poor rock in general and to difficulties in drilling and charging in particular, but also to suit the limits of a round in the transition from one rock type to another. However, due to the heavy hydraulic hammer used for scaling the cleaning of the tunnel front resulted normally in that the advance became longer than the intended. In the weakest parts, the amount of scaled-off rock was the only limit to proceed the scaling, resulting in up to 1- meter longer rounds than drilled.

The temporary support required consisted of systematic bolting and un-reinforced shotcreting; reinforced shotcrete arches were installed as quickly as possible after the rounds had been fired, followed by pre-bolting of the next round. The permanent support works comprised systematic bolting in roofs and walls, and the constructions of shotcrete arches containing reinforcing bars and mesh reinforcement. In addition, pre-fabricated steel arches were available for immediate use, although it was never necessary to use them.

In all, 2,500 m^3 of shotcrete was used within the deformation zone, of which approximately 1,400 m^3 was fibre shotcrete. About 20 T of 25-mm reinforcing bars and 1,000 m^2 of hot-welded nets (6 mm diameter) were used, especially in the shotcrete arches.

The cement in the ready-mixed fibre shotcrete used was basically a low-heat aluminasilica and sulphur resistant cement. The aggregate used was 0–8 mm with a fine material content less than 0.25 mm of 15–20%. The additives used were super-plasticiser and accelerator. Two types of fibres were used: 18 mm steel fibre with enlarged ends (known as EE-fibre and produced by Australian Wire Industries) and 30 mm Dramix steel fibres, glued together into small bundles. The fibre content was 1% by volume.

Deformation measurements were performed within the Singö deformation zone to check the stability. Tunnel convergences were measured with a Distometer, and tunnel deformations were also measured with 2-m and 6-m long extensometers, together with devices to measure the loads in the permanent shotcrete lining. Calculations of deformations and load on reinforcement were carried out using the finite element method.

The results of the measurements show that there were very small deformations. The vertical deformations of the roofs amounted to 1–4 mm, while the wall deformations were larger (convergence 7–8 mm). The reason for the larger wall deformation may be that a rock pillar divides the tunnels approximately 15 m wide. The deformations of the walls of the pillar were larger and not as superficial as those of the outer walls. One reason for this may be that the pillar is of a poorer rock quality than the rock in the outer walls; another reason could be that distressing in the horizontal directions makes the behaviour of the pillar more flexible than that of the more fixed outer walls.

Table 6-4. Supported tunnel length as percentage of total length of Singö deformation zone (120 m) in the SFR access tunnels /Carlsson et al. 1985/; cf. Table 4-4.

Type of support	Permanent rock bolts* %	Shotcrete %	Shotcrete %	Shotcrete %	Arches of shotcrete	Pre-grouting %	Pre-bolting
Description	φ 25, L: 3.8 (B1, B2)	Un-reinforced; T: 30–50 mm (S1, S2)	Fibre-reinforced T: 50–80 mm # φ 6 c 150 (S3, S4)	Reinforced T: 100 mm # φ 6.5 c 150 Bars # KS 40S φ 8 c 200 (S5)	T: 200 mm (S6)		(B3)
Operation tunnel	100	–	68	33	28	30	82
Construction tunnel	100	10	67	20	26	21	86

Note: Fibre shotcrete and reinforced shotcrete occasionally occur in combination.

The following conclusions may be drawn with regard to deformation measurements. The deformations in the roofs and the walls were small and the movements were slowing down. The loads on the reinforcement were low. FEM calculations showed stable conditions with deformations of the same magnitude as measured.

The method of driving the access tunnels with pilot drilling, pre-grouting, and pre-bolting of the next round, reduced rate of advance of driving per round, mechanical scaling, flushing and fibre shotcreting proved to be successful.

Because serious rock falls did not occur within the Singö deformation zone in the SFR access tunnels and there were no stoppages, it was possible to drive through the deformation zone according to plan. In total, 3.5 months were required for the driving of the operation tunnel through the zone (130 m), and about 3 months for the construction tunnel (140 m); i.e. the average rate of driving was approximately 50 m per month. The remaining driving of the access tunnels was achieved without noteworthy problems.

The driving result of the access tunnels is summarized in Table 6-5. The total time of excavation of the two tunnels amounted to 12 months. The rate of advance through the Singö deformation zone was 40 m/month, and the total time for the excavation of the Singö deformation zone was 3–3.5 months. During the excavation period of the access tunnels (12 months), the following rock support and stabilization were carried out:

- 38 pre-grouting operations.
- Installation of 3,300 rock bolts.
- 1,700 m^3 shotcrete.
- 1,780 drains.

The shotcrete used was mainly ready-mixed wet shotcrete; i.e. the shotcrete was mixed at the concrete plant above ground, and transported with concrete trucks down to a hutch at the location of the shotcreting. The use of accelerator was a minimum due to its quality reduction. Instead of using accelerator, several, thinner layers of shotcrete were applied /Larsson 1996/.

6.6 Excavation of the tunnel system and rock caverns

The tunnel system linking the different storage chambers and the four rock caverns were excavated according to plan, and no problems of special rock engineering interest occurred. Due to the dry rock conditions the amount of probe drilling was reduced. During tunnelling, pre-grouting was carried out as a selective action at one location only in the top heading in one of the rock caverns.

Table 6-5. The result of the driving of the access tunnels, SFR. It is to be noted that the average, maximum advance and average production include the excavation through the Singö fault, excavation of niches, and temporary and permanent support.

Access tunnels	Average advance m/month	Maximum advance m/month	Average production m^3/ month
Construction tunnel 48 m^2	74	135	4,400
Operation tunnel 64 m^2	61	110	4,000

The total water inflow into the underground excavations – i.e. along about 4 km of tunnels – amounted to a steady 720 l/min. This is a remarkably low inflow, especially considering the siting of the repository 60 m below the seabed. The water inflows are distributed as follows /Larsson 1996/:

- Access tunnels (L: 2,300 m) 520 l/min
- Rock cavern for intermediate reactor waste (L: 160 m) 15 l/min
- Other rock caverns including service tunnels and space (L: about 600 m) 120 l/min
- The silo 2 l/min
- Other tunnels including the lower construction tunnel (L: 900 m) 60 l/min

After the commissioning of the SFR, the water inflows have gradually decreased to a steady state. In 1994, the total water inflow was about 550 l/min /Larsson 1996/, and in 2006 the total water inflow amounts to 380 l/min.

6.7 Excavation of the silo

The silo is one of the largest vertical rock caverns excavated in Sweden, with a height of 69 m and a diameter of 30 m. In the upper part are two connecting tunnels – a construction tunnel and a tunnel for transportation during the operation period. A construction tunnel and a drainage tunnel have been excavated at the bottom of the silo.

A number of rock mechanical calculations were carried out in order to judge the rock stability and the permanent rock support measures to be employed for the silo and connecting tunnels. Different calculation models with both planar and axial symmetrical sections were considered. Thus, it was possible to simulate the silo with an anisotropic stress situation on the one hand, and an average stress on the other.

The stress and deformation behaviour indicated by the finite element calculations showed that the excavation of the silo could be carried out with obtained adequate rock stability. The expected movements were judge to be acceptable for rock caverns of this dimension located in rock of this specific quality.

A monitoring system employing installed extensometers was used to record both deformation of the rock during excavation of the silo and long-term deformation. The vertical deformation measured amounted to movements of only some millimetres.

Immediately after the excavation of the dome (before the stoping), an extensometer was installed in the central point of the roof.

The excavation sequence is shown in Figure 6-6. The first phase included excavation of the dome, which was excavated by using ordinary jumbos on the basis of the drilling scheme. The remaining rock, about 2 m, was then excavated by pneumatic pusher-leg drills in order to obtain as good as contour as possible. It is to be noted that no temporary support was needed for the excavation of the dome, indicating extremely good rock conditions and, thereby, a good stability of the rock mass. The permanent rock support consisted of shotcreting (80–120 mm thick in the roof and abutment and 50 mm thick on the walls) and systematic bolting of the roof with 3.7-m long bolts and a spacing of 1.75 m.

After the excavation of the dome and the construction tunnel at the base of the silo, work started on the long-hole drilling of the central hole. The excavation sequence of the silo is shown in Figure 6-7. The diameter of the central hole was 14 m. Forty-two long-holes of 76 mm were drilled using a Tamrock Zoomtrack DHA 600. The length of the vertical long-holes is 50 m.

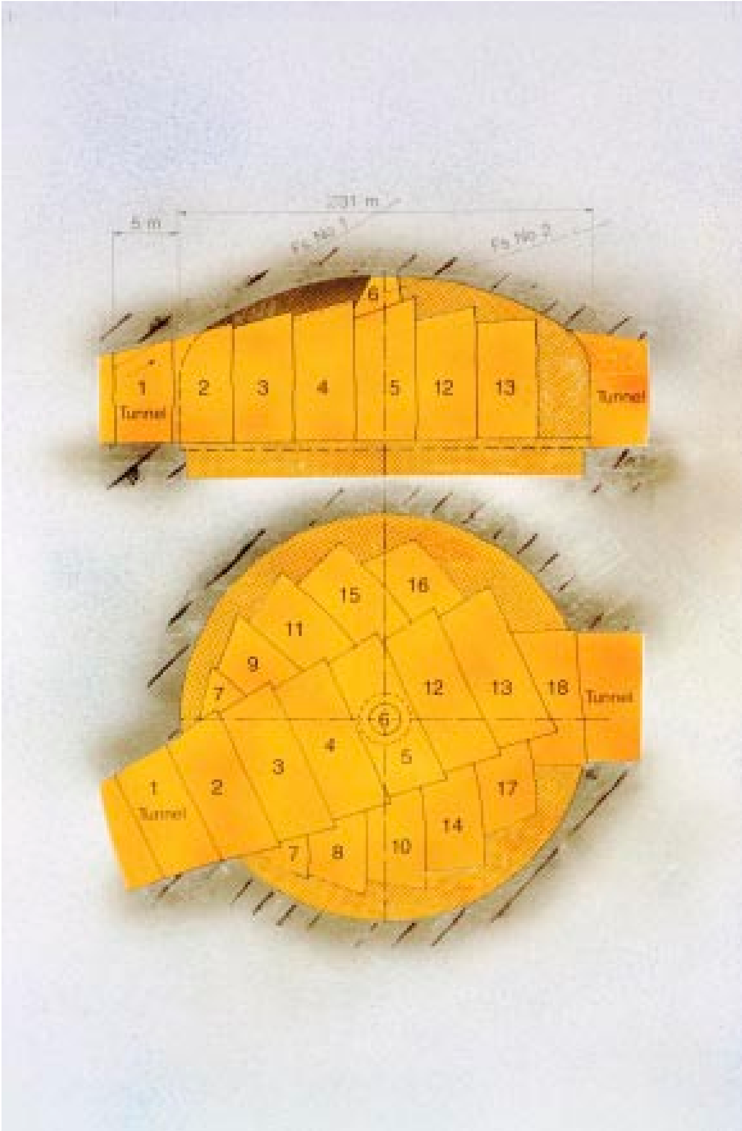


__Figure 6-6.__ The drilling scheme in the excavation of the dome, SFR, using Atlas Copco drilling jumbos /Schutz 1986/. The figures represent the orders of the individual rounds. Round No. 6 was made as soon as possible in order to install an extensometer. Fs Nos. 1 and 2 represent previously pilot drilling.

After control measurements of the location, inclination and deviation of the holes, the central hole was blasted from the bottom to the top with a 180-degree spiral, i.e. rising one bench height per half-turn spiral. The height of the spiral bench was 5 m, and mucking out was carried out successively.

The original idea was to use long-hole drilling for the entire excavation of the silo. However, because the deviation of the boreholes turned out to be too great a decision was made to use long-hole drilling only for the central hole. Bench blasting with a drilling depth of 6 m then excavated the rock remaining between the central hole and the silo wall. The mucking-out of the loose rock was then made in 2-m spiral benches.

The rock support of the silo walls consisted of shotcrete and rock bolting, and the support work was carried out successively, bench by bench /Larsson and Christiansson 1986, Schutz 1986, Carlsson and Hedman 1986, Carlsson et al. 1989/.

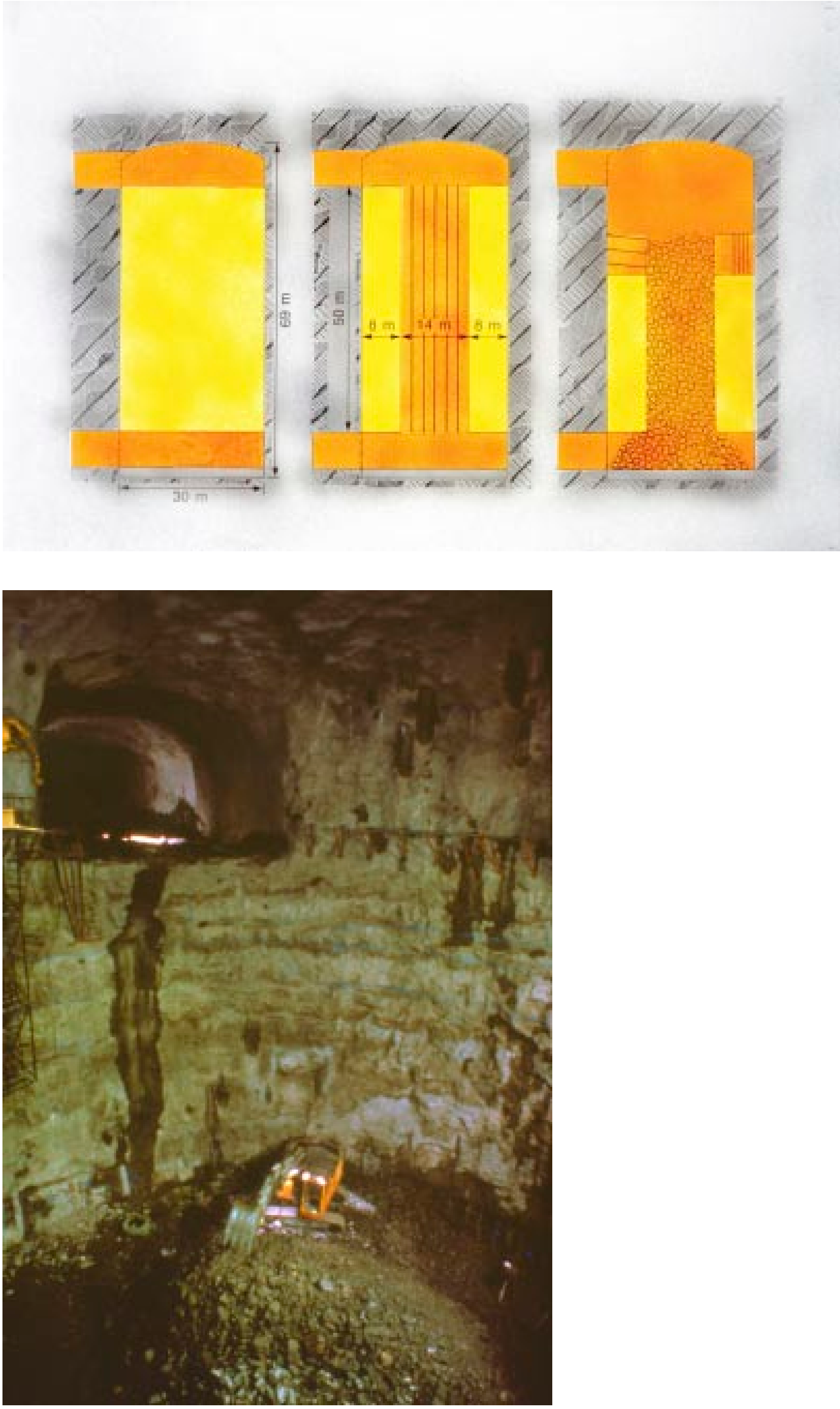


Figure 6-7. The excavation sequence of the silo, SFR, a. (1) Excavation of the dome. (2) Long-hole drilling and blasting. (3) Bench blasting. /Schutz 1986/.

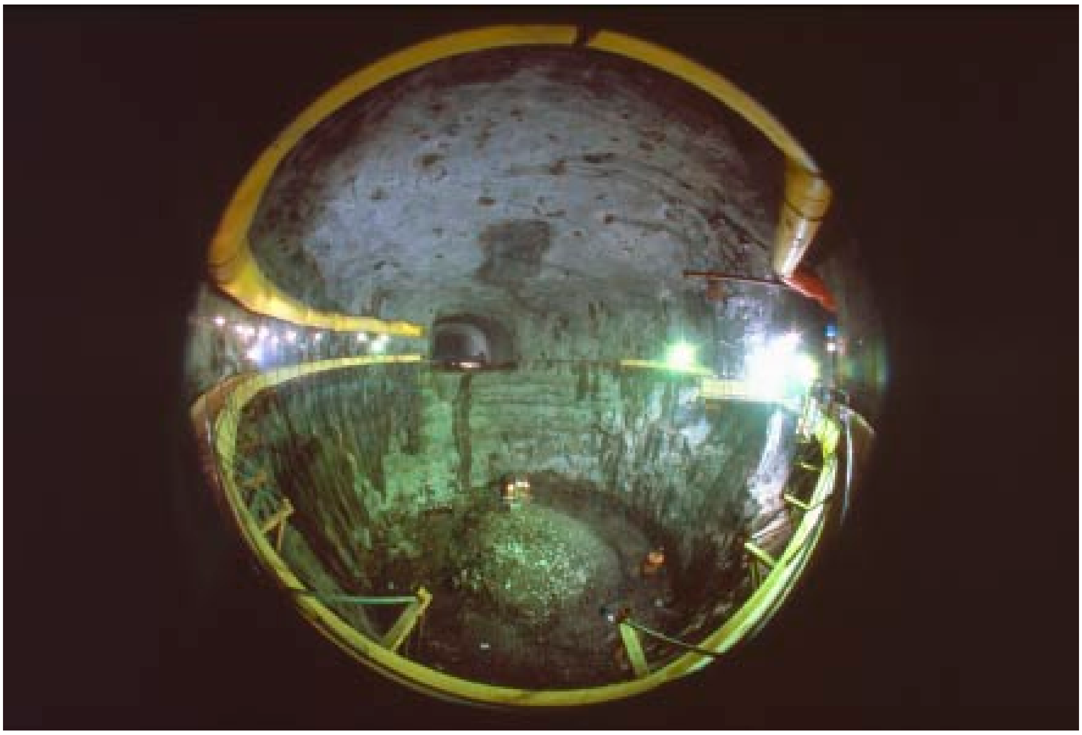


Figure 6-8. *A fish-eye view photo showing excavation work in step (3), SFR. PhotoG Hansson/N.*

In summary, the result of the excavation of the repository area was /Larsson 1996/

- Time of excavation: 18 months
- Average production: 18,300 m^3/month
- Maximum production: 32,300 m^3/month
- Installation of rock bolts: 17,160
- Pre-grouting operations: 29 (primarily in the sub-horizontal deformation zone under the silo)
- Shotcrete: 8,700 m^3
- Drains: 1,780

Table 6-6 shows a summary of construction equipment used for the excavation works in the repository area.

Table 6-6. Construction equipment in the repository area.

Construction activity	Type of construction equipment
Drilling	Three Atlas Copco TH531 equipped with 4 BUT35 booms fitted with 18-ft feed and fitted with a charging cradle on a hydraulic boom for 5 m advance.
	Charging was also carried out using a sky lift when the rigs were utilized for pendulum excavation.
	Long hole drilling was employed for the central shaft of the silo using a Tamrock Zoomtrack DHA 600. The wall benches of the silo were drilled by using a two booms bench rig, Atlas Copco 302.
Mucking out	A wheel mounted loader Cat 980 C with a 4.5 m^3 bucket for the main part of the mucking out in the silo and in the rock caverns.
	One Cat 988 A wheel-mounted loader with 4.5 m^3 bucket capacity.
	One Cat 966 C wheel-mounted loader with 2.3 m^3 bucket capacity.
	The excavated rock from the wall benches was dumped into the central shaft of the silo using a hydraulic excavator Åkerman H 14. The excavated was protected allowing it to remain in the silo during blasting.
Scaling	One Liebherr 941 A hydraulic excavator and one Åkerman H12 diesel hydraulic excavator both equipped with Montabert BRH125 hydraulic scaling rod.
Rock haulage	Two Kiruna trucks, 35 tons.
	One Kiruna truck, 50 tons
	Three to six Volvo BM trucks, 25 tons.
	A number of Cat trucks on stand-by.
Rock support	Atlas – Alimak bolting robot.
	One Atlas Copco Boltec 540/22 mechanized rock-bolting equipment.
	Robocon shotcreting equipment.
	Hand held shotcreting (for dry mixed shotcrete) of the silo walls and at inaccessible areas.
	One “Furuholmen – Forsmark” remote-controlled shotcreting system.
	One mobile unit fitted with a working platform and equipment for simultaneous hole grouting (the unit was developed on site).
	Grout holes and drilling of holes for the Atlas Copco rigs drilled expander bolts.
	The shotcrete was mixed at the concrete plant, originally erected for the concrete works for Forsmark 1.

7 Engineering aspects on rock mass conditions in the Forsmark area

7.1 Introduction

The extensive site investigations and the conditions encountered during the construction works at the Forsmark site were compiled by /Carlsson and Christiansson 1987/. At that time the authors focussed on the general construction experiences, and summarised the stress measurement data and the results from the hydraulic investigations. The authors also provided a general description of the rock mass in the area. Because the four tunnels at the Forsmark site all penetrated the Singö deformation zone, earlier reports focussed on the tunnelling conditions encountered in the vicinity of this major deformation zone, and only provided a brief description of the general "more normal" tunnelling conditions away from the fault zone. Thus, the documentation from the construction works has been reviewed again with the focus on providing a more detailed engineering-geological description on the general rock mass and those in vicinity of the Singö deformation zone.

In this description the rock mass is described in rock classes. Classes 1–3 covers the experiences gained outside the major deformation zone, whereas rock class 4 summarizes the conditions within the Singö deformation zone. The rock classes 1–3 are primarily based on a summary of the granitic portions of the rock mass. The largest construction area in granitic rock available for this description is the SFR area, cf. Figure 7-1. A summary of the engineering-geological conditions is therefore given as the observed variability within a block with the approximately dimensions: L = B = 250 m, H = 150 m.

It should be noted, that the following description of the rock mass conditions within the SFR area is also valid for the conditions in the cooling water tunnels as well as for the geological conditions of the walls in the open-cut excavations of the Forsmark units.

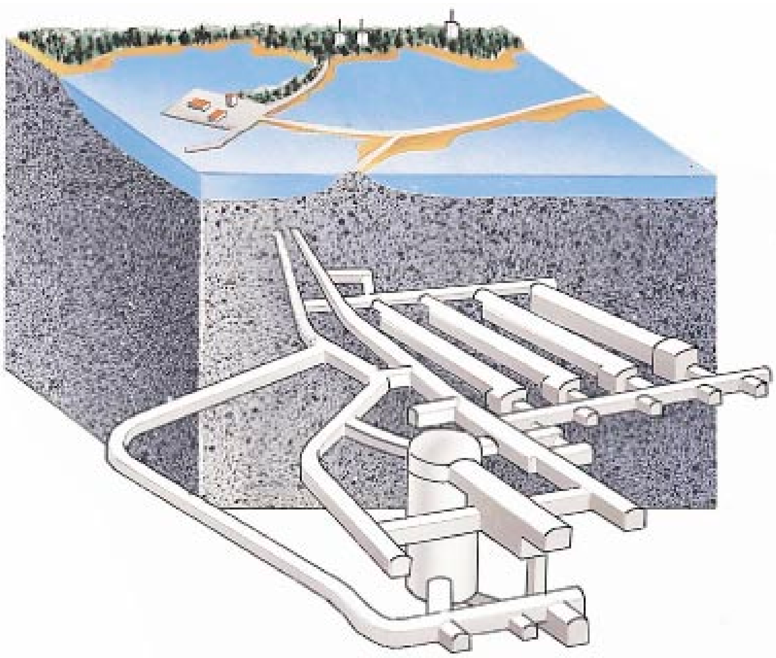


Figure 7-1. 3-D view of the SFR facility. The rock caverns are 180 m long. The total length across the four caverns and the two access tunnels are approximately 200 m. The deepest part of the facility is located 140 m below seabed.

7.2 General description, block size 250×250×150 m

7.2.1 Lithology

The rock mass is dominated by granite. The degree of foliation is relatively strong closest to the Singö deformation zone, and decreases with the distance from this major deformation zone. The origin of the granite is uncertain, but it could probably be classified as metamorphic granite. Minor foliated or lineated portions of the granite can occur at further distances from the Singö deformation zone. Minor xenolites of mafic rock were occasionally found. The relative age of the granite to the xenolites was never studied during construction.

Dykes of fine-grained granite as well as pegmatite occur frequently in the rock mass, and also within the Singö deformation zone. The pegmatite varies significant in grain size and could change frequently between typical pegmatite and very course grained granite over short distances. The pegmatitic dykes were subject to plastic deformations as large scale undulating folding, as well as small scale ptygmatic folding of these veins was observed. The pegmatitic rock occurred in both vertical and sub-horizontal orientations. Occasionally they formed large surfaces in the tunnel wall or in the roof (tens of meters along the tunnel). There are a few observations of a more homogeneous course grained pegmatite with less ductile deformation crosscutting the plastic deformed pegmatite – course grained granite. The dykes of fine-grained granite are likely a younger generation of pegmatite.

Dykes of amphibolites were also observed. They are normally sub-vertical, but appear to be of two kinds; one being more aligned with the large-scale deformation (NW-SE) and one being more N-S trending. Both orientations of the dykes display some ductile deformation. The NW-SE trending dykes are very homogenous and massive, whereas the other set sometimes display a significant influence of deformation. The N-S trending dykes often display intense fracturing parallel to its boundaries with fracture spacing in the range of tens of mm, and there is a distinct but moderate alteration. The alteration has formed biotite on the fracture surfaces. These dykes were described as minor deformation zones during construction and are sometimes wet from local dripping points or spots of moisture.

7.2.2 Fracture distribution

Local intense clusters of sub-horizontal fracturing, overprinting the more regular fracturing that occurs at greater depth characterize the upper 20–30 m. Sediment infillings may occur in this sub-horizontal joint set, Figure 7-2.

At depth, the rock mass primarily contains steeply dipping NW and NE joint sets, as well as sub-horizontal fracturing. Typically, the fracture frequency is irregular, clusters of each joint set are common, sometimes forming minor deformation zones and building up a blocky rock mass with typical block size of tens of meters. E-W trending fractures and minor fracture zones gently dipping towards south occur occasionally. A simplified illustration of the fracture distribution in a 250×250×150 m block covering the deposition and operational areas of the SFR is given in Figure 7-3. The heterogeneity of this rock mass is illustrated by the description of 10×10×10 m blocks for rock classes 1–3 in the following section. The rock support used in these rock classes is also summarized.

Figure 7-2. *a. Observed horizontal and sub-horizontal fracturing along the inlet canal for cooling water to Forsmark nuclear the power plants. b. Continuous horizontal fractures in the northern wall of the inlet channel. Photo G Hansson/N.*

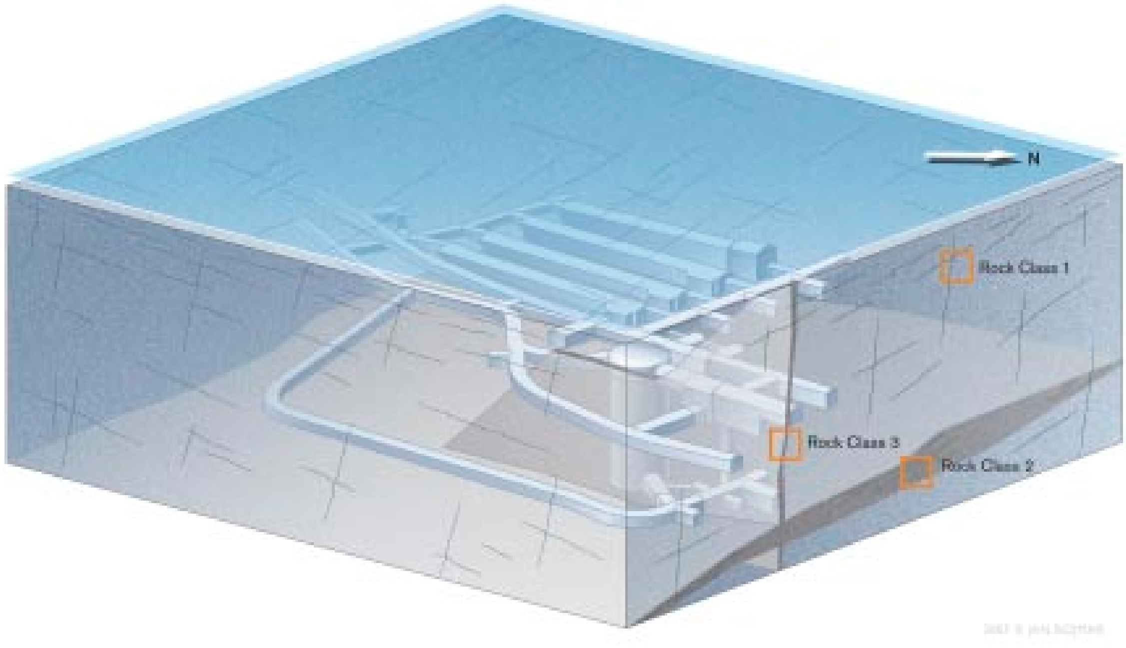


Figure 7-3. Simplified illustration of the large-scale fracture distribution in a 250×250×150 m block, covering the rock mass of the SFR.

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.

If pegmatite is frequent, especially in the roof, the rock mass may appear to be more fractured. This is commonly due to the coarse-grained rock being more sensitive to the excavation- induced damage. The pegmatite can occur both as vertical and gentle dipping dykes (Figure 7-4).

The fractures are well sealed with precipitation. Dripping water and spots of moisture are observed.

Rock support: Bolts: none to pattern bolting 1 bolt/ 4 m^2 (Bx–B1)

Shotcrete: none to 50 mm un-reinforced shotcrete (S2).

The lower levels of rock support in rock class 1 are primarily applied in the construction tunnels of SFR, and in the cooling water tunnels. The use of shotcrete in this rock class was more frequent in areas of the SFR due to long-term purpose of the facility and hence the need for future maintenance of rock surfaces.

7.3.2 Rock Class 2

Clusters of sub-horizontal to gently dipping fractures. The fractures occur often as clusters as discrete minor deformation zones. The frequency of vertical joint sets seems to locally increase across the clusters of sub-horizontal to gently dipping fractures. Locally, the intersection of high frequency of vertical and gently dipping fracture give the appearance of crushed rock. Lenses of crushed rock were observed in the minor deformation zone H2 under the silo at the SFR (Figure 7-6a and 7-6b). In contrast, the same deformation zone was not detectable in some vertical boreholes drilled in the vicinity of the observations in the tunnel, indicating a heterogeneous and possibly discontinuous fracturing pattern.

***Figure 7-4.** Pegmatite dykes at the face of the Forsmark tunnel. Host rock is mafic rocks.*

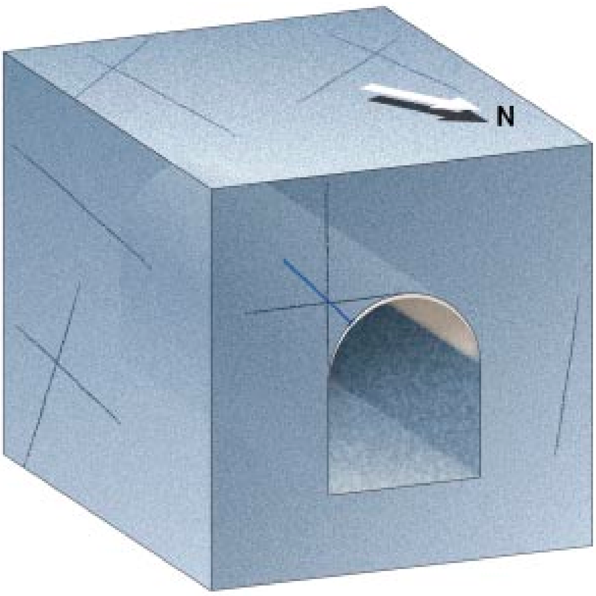


***Figure 7-5.** Simplified illustration of a 10×10×10 m block of the rock mass in rock class 1 and applied support measures.*

Figure 7-6a (left). Gentle dipping fractures within the deformation zone H2 under the silo of SFR. A section with relatively low fracture frequency.

Figure 7-6b (right). A nearby section with relatively high fracture frequency.

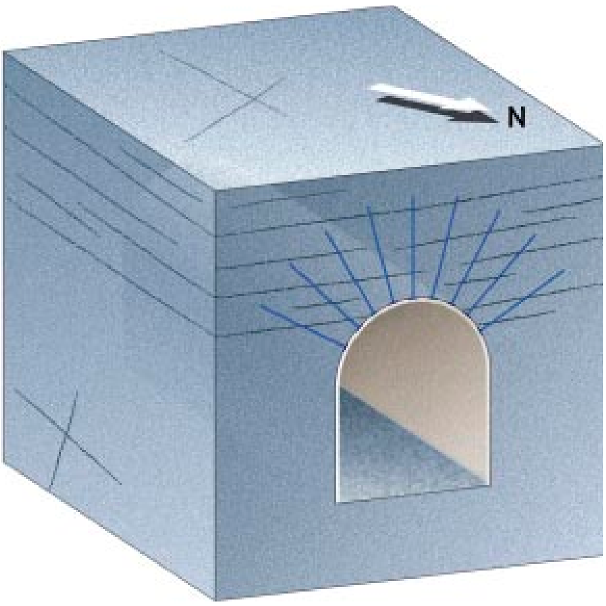


Figure 7-7. Simplified illustration of a 10×10×10 m block of the rock mass in rock class 2 and applied support measures.

This rock class has locally a high transmissivity, especially within gently dipping fractures and fracture zones.

Rock support: Bolts: 1 bolt/ 4 m^2 (B1) to 1 bolt/ 2 m^2 (B2)

Shotcrete: 50 mm un-reinforced shotcrete (S2) to 80 mm fibre reinforced shotcrete (S4).

The highest levels of rock support in rock class 2 were used when gently dipping fracture zones intersect the roof and abutments. Extensive grouting was occasionally required.

7.3.3 Rock Class 3

Clusters of steeply dipping fractures occasionally form minor deformation zones. The NE trending joint set usually formed these cluster and dominated this type of fractured zones. The fractures were relatively well sealed with calcite and laumontite. Crystals of calcite together with asphaltite were observed.

Also the NW trending, steeply dipping joint set occasionally formed minor deformation zones, but less frequent outside the area of the Singö deformation zone and the more gneissic part of the rock mass where the ductile deformations are significant. The aforementioned N - S trending, altered amphibolitic dykes sometimes appear as minor deformation zones and hence may present similar engineering challenges as the minor deformation zones formed by the clusters of NE and NW trending steeply dipping joints.

Along the minor deformation zones formed by the clusters of NE and NW trending steeply dipping joints other joint sets can occasionally be found with increased frequency.

Figure 7-8. *An altered amphibolitic dyke, highly schistose.*

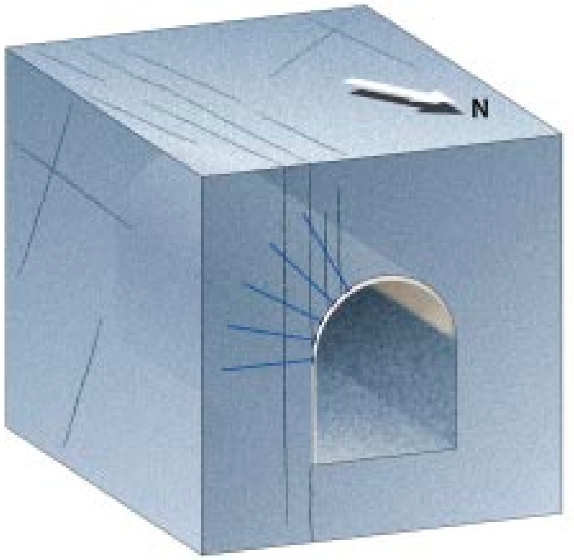


Figure 7-9. Simplified illustration of a 10×10×10 m block of the rock mass in rock class 3 and applied support measures.

Typical widths of these minor deformation zones are 0.5–1.5 m, and less than 0.5 m width for the minor deformation zones formed by altered amphibolitic dykes (Figure 7-8). The length of the NE trending minor deformation zones was frequently observed along the tunnels and caverns of the SFR, because the layout had the main underground openings aligned approximately parallel to that joint set. More or less continuous length of up to approximately 100 m was observed for the NE trending, steeply dipping minor deformation zones. In caverns with high walls oriented at a small angle to the strike of NE joint set commonly resulted in local overbreak.

The transmissivity was normally rather small in rock class 3. Spots of moisture and occasionally dripping water were sparsely distributed. However, more discrete channelling has been observed. The highest detected inflows in accordance to Figure 3-9 are related to such channelling.

Rock support: Bolts: 1 bolt/ 4 m^2 (B1) to 1 bolt/ 2 m^2 (B2)

Shotcrete: 50 mm un-reinforced shotcrete (S2) to 80 mm fibre reinforced shotcrete (S4).

The highest level of rock support in rock class 3 was used when the vertical dipping minor deformation zones were intersecting a tunnel or cavern wall. This normally required a decrease in bolt spacing. The use of shotcrete in sections with rock class 3 was more a result of the span of the opening than the occurrence of the minor deformation zone.

7.3.4 Rock Class 4

Rock class 4 refers to the major deformation zone intersected by the tunnels in the area of the Singö deformation zone.

This major deformation zone was composed of several sectors that exhibited different geological characteristics and large heterogeneity in terms of rock mass strength and hydraulic transmissivity. The appearance of the zone differed somewhat between the tunnels, but transition zones, zones of intense fracturing and core zones, the latter characterized by clay alteration and crushed rock, with cubic blocks; 2–20 cm in size were encountered in all four tunnels.

The core zone was the most consistent part and intersected in all of the tunnels. It was characterised by a 2–12 m wide zone of crushed rock, showing high degree of alteration and disintegration. Matrix consists of silty, sandy and gravely material. On one or both sides of the crushed rock, several clay filled fractures were found, with a thickness of a few cm to approximately 1 metre.

The clay resulted from rock alteration. The number, order of occurrence and thickness of these elements varied between the tunnels.

Rock support: Bolts: 1 bolt/ 4 m^2 (B1) to pre-bolting (B3)

Shotcrete: 80 mm fibre reinforced shotcrete (S4) to shotcrete arch, T = 300 mm, including rock anchoring of the reinforcing mesh.

The rock support of the Singö fault in the Forsmark tunnels is given in Table 4-5, Section 4.3.

Illustration of rock support in rock class 4 is shown in Figure 7-10.

7.4 Influence of geological features on tunnelling

It is essential to highlight that virtually all-geological features of importance for tunnelling were foreseen such as the spatial locations of the deformation zones penetrated by the tunnels, rock boundaries, and occurrences of fracture zones. However, for some features, on a local scale, their properties and their engineering behaviour were not fully foreseeable when encountered during the excavation. When such ground conditions were encountered, the method of working and choice of support adopted by the contractors and designers were adequate. The advance of tunnelling was only marginally affected and the temporary and final stability of the tunnels were perfectly satisfactory.

Some illustrations related to tunnelling and specific geological features as experienced during the underground works at Forsmark are given below. Examples of such geological features are gently dipping fractured zones with a high transmissivity, the influence of pre-dominant fractures, distribution of water-bearing fractures, and stress conditions.

7.4.1 Gently dipping fracture zones with a high transmissivity

The occurrence of gentle dipping fractures in the superficial rock mass has been described by /Carlsson 1979/. This fracturing is to a high extent release fracturing closest to the surface. Some of these fractures are filled with sediments of sub-glacial origin.

Figure 7-10. *Rock support in rock class 4. The wire mesh was later covered by shotcrete. Forsmark 1 and 2 tunnel.*

In addition, gently dipping deformation zones with high transmissivity were met in the discharge tunnel for units 1 and 2, as well as in the lower construction tunnel of the SFR. The depth at which these structures were met was 50 m and 140 m respectively. In addition, the site investigations for the SFR drilled through a similar structure at some 40 m depth. The structure at the bottom of the SFR has been described by /Carlsson and Christiansson 1987/. The horizontal fractures dip 10–15° and occur locally with very high frequency. Locally, also vertical fractures increase in frequency at the intersection with these "clusters" of sub-horizontal fractures. The fracturing was locally rather high, displaying lenses with more or less crushed rock approximately 0.3–0.5 m thick and some meters long (cf. Figure 7-6b). These lenses of crushed rock occurred step-wise, being the irregular core of a deformation zone dipping approximately 25° towards SE. The nature of this kind of gentle dipping deformation zone indicates another origin than the sub-horizontal release fracturing commonly found in the superficial rock mass in the Forsmark area. The extension of these gentle dipping deformation zones is less known. Exploration drilling during construction of the SFR intended to more in detail find the location of the lower deformation zone failed, maybe due to the possible heterogeneity of the structure, or just because its extension was limited.

The tunnelling through the gentle dipping fracture zones in the first discharge tunnel and in the SFR required significant efforts of grouting. At the SFR the total use of cement for grouting was some 9,000 kg for approximately 40 m of tunnel. The area is the part of the SFR facility that has the highest seepage. This is due to several reasons:

- There was not put any high requirement to seal this part of the construction area, which is sited below all operational areas.
- The significant difficulty to seal a gently dipping fractured zone by pre-grouting from the tunnel.

The complex layout at this lower part of the SFR with auxiliary tunnels for the drainage under the silo, a cavern for the deepest pump station and vertical shaft for the discharge of drained water and other installations.

7.4.2 Influence of predominant fractures

Apart from the major deformation zones experienced during construction of the tunnels at the Forsmark site, the fracturing in the rock mass is in general favourable for tunnelling. The rock mass is rather blocky with vertical joint sets trending NW–SE respectively NE–SW and a sub-horizontal set. In addition, a minor set dipping some 40–50° towards S–SE was observed occasionally. The fracture frequency was found to be relatively low in the tunnels, but not evenly distributed. The fractures within the dominant three sets occur often in clusters, forming minor deformation zones. Because of the extensive mineral precipitation in the most of the fractures these minor structures are of less importance for the construction of stable tunnels. There are two situations that in general required systematic bolting:

1. The occurrence of sub-horizontal fractures in the roof normally required systematic bolting. Overbreak in the crown and towards the abutments was commonly observed in this situation. The extension of the more pronounced cluster of sub-horizontal fractures was observed for example in the access tunnels to the SFR to have a length of > 30–40 m.
2. Tunnel walls aligned parallel to the NE trending joint set displayed also over-breaks up to the abutment. This occurs also if there is a 5–20° difference in the trend of the wall and the fracturing, indicating that the resistance of the strength of the fracture is low to the forces caused by the tunnelling. This joint set often displays calcite and laumontite infillings. Dripping water occurs only occasionally. The extension of these NE trending minor deformation zones could be followed up to some 100 m in the deposition area of the SFR.

7.4.3 Distribution of water-bearing features

Except for the major deformation zones experienced during construction of the tunnels at the Forsmark site, the rock mass is in general rather dry, also at shallow depth. In the upper 30–40 m of the superficial rock mass, the gentle dipping release fracturing is contributing significantly to the seepage. At larger depth all pre-dominant fractures are contributing to the seepage, especially when they form "clusters", which might be defined as "minor deformation zones.

The distribution of all seeping fractures in the deposition and operational area of the SFR was evaluated /Carlsson and Christiansson 1987/. The data included roughly 10,000 observed fractures in roofs and walls. Of the total population, 6% was seeping water; the smallest observation was defined as “a spot of moisture, some dm^2 large”. The distribution of the observed seepage is given in Figure 3-10. The largest seepage (2.5 l/min) originates from the minor deformation zone. This zone crosscuts all rock caverns and the operational tunnel, but is not intersecting the construction tunnel. Selective grouting on that structure was done in one of the caverns. This was the only grouting carried out in the whole deposition and operational area of SFR. This measure had probably very little influence on overall distribution of seepage into the deposition and operational area of the SFR. The silo illustrates the low transmissivity of the rock mass. This cavern has a diameter of 30 m and a height of 69 m.

The excavated volume was 45,000 m^3. After completed excavation 1.4 l/min was measured at a temporary weir where the water coming into the silo discharged in to the lower construction tunnel. In addition, by measuring the humidity in the ventilation air going into and out of the silo it was estimated that the ventilation evacuated approximately 0.6 l/min, giving a total seepage of 2.0 l/min.

The water seepage into the SFR except for the inflow in the major deformation zones occurred primarily as spots of moisture and locally dripping water. The most predominant structures for the seepage was

- Areas were cluster of NE trending vertical fractures occur. This may be defined as minor deformation zones. The longest such structure was observed in the access tunnel to the silo roof, along the roof of the silo and further into the cavern for handling of the waste packages. The total observed length was 100 m, even although the fracture frequency was irregular over the observed distance. Spots of moisture and dripping water occurred.
- Areas were cluster of NW trending fractures. These cross most of the tunnels and caverns in large angle, so the length is difficult to estimate. But because this structure seldom can be traced from one rock cavern to another, the length is probably limited. The water occurs mainly as spots of moisture.
- Amphibolitic dykes that has been strongly deformed and schistose. These dykes occur more or less as minor deformation zones. The width is limited to 1–2 dm and its length is probably seldom-exceeding 100 m. Water occurs as moisture/dripping along large stretches of the areas where the structures have been observed. In the SFR, these structures are sub-vertical and trends N–S to NW–SE.

The improvement of air quality during the operation of the SFR facility has included measures to decrease the humidity underground. This may be one of the reasons for the decreased total seepage into the facility (Figure 8-2). An overview of the distribution of seepage points in the deposition and operational area today, indicate the type of structures that contributed to the most of the seepage during construction were occurrence of spots of moisture on the shotcrete surfaces still can be found.

7.4.4 Stress conditions

The experiences of high stresses in underground works are limited from the Forsmark area. /Carlsson and Olsson 1982a/ reported stress induced spalling in the roof of the recirculation tunnel of Unit 3. This is related to a very limited rock cover over the tunnel, indicating significant stress concentrations, most likely very limited.

Any construction problems due to high stresses were never experienced at the SFR. Only in the upper part of the lover construction tunnel when the tunnels was driven in an direction close to the orientation of the major horizontal stress, loosening up of the tunnel face was experienced for some rounds. The tunnelling went through a body of pegmatite in that area.

8 Inspections and rock engineering experiences from the operation of SFR

8.1 Overview of rock inspections

After taken into operation, the underground facility of the SFR has been subjected to recurrent inspections with regard to rock quality and rock support. Special attention has been given to operational aspects, such as safety and health for personnel as well as function of barriers.

An instruction for the Inspection Team was issued in 1988 /Larsson et al. 1988/ and was later replaced by a revised instruction in 2003 /Bodén and Lundin 2003/. The instruction describes field of responsibility and how inspection and maintenance of rock surfaces and rock support shall be performed. By studying groundwater inflows, rock deformation, shotcrete, rock anchors etc., the status of the underground facility can be assessed.

The tunnels and rock caverns of SFR are, like any other tunnels in Sweden in which personnel is working, subject to the rules of the Swedish Work Environment Authority (Arbetsmiljöverket). The directions imply, among other things that inspections should be carried out as often as the result from previous inspections give rise to. The time interval applicable for SFR has in agreement with the Authority been set to 5-year inspection interval. This inspection has been given the term Large Inspection. In addition, recurrent safety evaluations have to be performed according to conditions set forward in the Swedish Act for Nuclear Facilities (Kärntekniklagen).

Thus, in order to initiate and evaluate the yearly inspections and testing, that are included in the overall programme, SKB has appointed an inspection team composed of personnel from SKB (key person responsible for operation), personnel from SFR 1 (key person responsible for maintenance of SFR), and specialists on rock engineering and concrete for the 5-year Large Inspection. At this inspection, SKI's and SKB's responsible persons for the long-term safety should be provided to participate.

Members of the inspection team, and if necessary seconded by specialists on certain subjects constitute the so-called Rock Inspection Group, and carries out yearly inspections and yearly reporting to SKB.

8.2 Control actions

The control actions contain visual inspections, measurements and readings. The maintenance actions comprise in the first place remedial measures to increase personnel safety, and to guarantee the combined function of different rock support system.

The following control actions are carried out in all rock openings of the facility. Adjustments to the availability of a certain repository opening with respect to radiation level is however being made before entering the actual space.

Control actions	Responsible
Internal check-up	SFR Organization
Yearly rock inspection	Rock Inspection Group
Testing and investigations	Rock Inspection Group
Maintenance	SFR Organization
Large inspection	Inspection Team

Internal check-up

As a part of the systematic work environmental efforts, the SFR organization is responsible for the internal check-up. The Rock Inspection Group is notified if loose rock blocks in the walls or on the tunnel floors are found or if any rock support element is suspected to have a reduced function. The activities and measures included in the internal check up are reading of pumped groundwater out from the facility once a month, and general notes on rock maintenance.

Yearly rock inspection

The activities included in the yearly inspection are shown below. A compilation report including protocols on measures being made and measurements carried out is presented yearly.

Activity	Measures	Documentation	Programme
Deformation measurements	Reading – compilation of result	Report number	
Measurement of groundwater inflow	Reading – compilation of result	Yearly report	Report number
Rock mass quality and rock support	General visual inspection	Yearly report	

Testing and investigations

Except for the yearly rock inspection, various testing and investigations are carried out with an extended space of time, such as

Time interval	Programme	
Measurement of groundwater inflow		Report number
– Evaluation of groundwater inflow	5 years	
Testing of rock anchors		Report number
• Boltometer testing	6 years	
• Visual inspection	6 years	
• Corrosion investigation	15 years	
Investigation and testing –shotcrete	10 years	Report number

Maintenance

An efficient maintenance is of significant importance for the functioning of the support and drainage system. The following maintenance actions are carried out in accordance with the time interval given below, or more often if deemed necessary.

Object	Measures	Documentation	Time interval
Free rock surface	Scaling	Protocols, estimated scaled rock volume	5 years
Shotcrete	Scaling	Protocols, estimated scaled shotcrete volume	10 years

Large inspection

At the Large Inspection, the results from all other inspections, measurements and measures, which have been carried out in the facility during the previous operation period, are compiled and summarised. Furthermore, changes of the instructions and programmes are proposed, if any, for the next inspection period. Special attention is given to questions related to durability and safety in the underground facility. Until further, the Large Inspection is conducted with a 5-year interval.

8.3 Rock engineering experiences from the operation of SFR

8.3.1 General

The underground excavation works for SFR was finalised in 1986; i.e. the rock support in the underground tunnels and caverns was applied for more than 20 years ago. In conclusion, the result from the various inspections, testing and measurements given above, no significant deterioration or failure of installed rock support has been notified. The host rock mass after excavation has performed as foreseen, and no stability problems or unexpected deformation have been reported, nor has any problems related to groundwater been identified.

In order to illustrate the status of the rock support and rock mass behaviour, a brief presentation is given of the result from the deformation measurements, testing of rock anchors, and groundwater inflow measurements.

8.3.2 Deformation measurements

By using 18 extensometers, deformation measurements have been performed in the Singö deformation zone and in the silo. Since 1997, all extensometers are electrically read, and the latest readings were made in 2006. The result of the deformation measurements in the Singö zone and the silo is shown in Table 8-1 and 8-2 respectively for the measuring periods 2005–2006 and 1997–2006.

Table 8-1. Deformation measurements in the Singö deformation zone, SFR.

Extensometer no.	Deformation 2005–2006 mm	Deformation 1997–2006 mm
Ext. 1	+0.02	+0.03
Ext. 2	+0.03	+0.17
Ext. 3	–0.01	+0.01
Ext. 4	± 0	–0.04
Ext. 5	± 0	+0.03

Note: The accuracy of measurement is 0.02 mm.

Table 8-2. Deformation measurements in the silo, SFR.

Extensometer no.	Deformation 2005–2006 mm	Deformation 1997–2006 mm	Notes
M009	± 0	+ 0.01	
M010	–0.01	–0.02	
M011	± 0	–0.01	
M014	± 0	+0.01	
E1	0.00	± 0	Total deformation since 1989: ± 0 mm
E2	–0.02	–0.30	Total deformation since 1989: –1.06 mm
E3	0.00	–0.02	Total deformation since 1989: –0.01 mm
E4	± 0	± 0	Total deformation since 1989: –0.01 mm
E5	–0.01	+0.18	Total deformation since 1989: +0.11 mm
E6	± 0	± 0	Total deformation since 1989: –0.12 mm
E7	± 0	+0.07	Total deformation since 1988: +0.15 mm
E8	–0.01	+0.02	Total deformation since 1988: +0.09 mm
E9	± 0	+0.01	Total deformation since 1988: +0.03 mm

Note: The accuracy of measurement is 0.02 mm.

The extensometers M009–M014 were rebuilt for electrical reading in 1997. Extensometers E1–E5 were installed 1989, and the extensometers E7–E9 were installed 1988.

It is evident that the deformation in the Singö deformation zone as well as for the silo is very small with measuring values within or close to the accuracy of measurement; e.g. extensometer no. 2 in the Singö zone has registered a deformation of 0.17 mm towards the operation tunnel.

The measured deformation in the silo demonstrates stable conditions with measuring values within the accuracy of measurement. The total deformation since 1997 is 0.28 mm, and the total upward deformation trend of extensometer E2 since 1989 is approximately 1 mm.

8.3.3 Testing of rock anchors

In 1984, it was decided to start a programme on testing cement-grouted rock bolts on a long-term basis in the SFR 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/.

During the construction period 1984–1986, a total of 316 cement-grouted rock bolts were tested /Knape 1987/. These bolts were production bolts, and were later covered with shotcrete and thereby not accessible for future testing. Therefore, in 1987, a number of easily accessible reference rock bolts were installed to enable long-term testing and quality control. A total of 42-reference rock bolts (Ks 40 φ 25, L = 3.65) were installed in the southeastern wall of the construction tunnel plus three production bolts (Ks 40 φ 25, L = 3.65). The cement grout for all bolts has a water-cement ratio of 0.28. Seven reference bolts were reamed and extracted at two occasions. In order to replace these bolts plus future reaming and extraction, an additional 12 reference bolts (Ks 500ST φ 25, L = 3.6) were installed in 2004.

The principle of Boltometer measurement is that a specially designed sensor containing piezoelectric crystals is pressed against the free planar end surface of the rock bolt. Compression and flexural elastic waves are transmitted into the metal bolt. When the waves travel along the bolt, some energy is transferred through the grouting into the rock and thus the wave amplitude decreases. At the inner end of the bolt the waves are reflected. The reflected waves are recorded at the outer end of the bolt by means of the piezoelectric crystals. If the grouting surrounds the bolt fully and is of good quality, the amplitude of the reflected wave is damped more than if grouting is deficient or lacking. The reflected waves will therefore have a lower amplitude in the case of good grouting, than in the case of imperfect grouting. The time interval between the excited and the reflected wave gives a possibility to calculate the length of the bolt. The amplitude of the reflected wave or sometimes successive reflections can be analysed and the probable condition of the rock bolt can be estimated on the basis of calibration tests /Bergman et al. 1983/.

The result of the Boltometer testing of the 316 production bolts indicated that 73% of the bolts could be classified as having optimal performance, 21% reduced performance, 5% insufficient performance and 1% very poor or non-existent performance /Knape 1987/.

The Boltometer measurements in 1987 were carried out in two steps: one test 33–34 hours after grouting and one test 34 days after grouting. The 34-day test was set as the zero point for the evaluation of the future testing.

The result from the Boltometer measurements on 49 reference rock bolts carried out in 2006 shows that all 49 bolts have optimal performance /Lundin 2006/. In comparison with an identical test on the same rock bolts in 1999, no difference in performance can be perceived from the 2006-testing; i.e. all the bolts can be classified to have optimal performance.

A corrosion study of reference cement-grouted rock bolts were performed in 2004 /Lundin 2004/. Three bolts were selected based on their location; one rock bolt in an area with water inflows (in the Singö deformation zone), one bolt located in a moist environment, and the third bolt in dry environment.

The reaming of the reference bolts was made using drilling equipment giving a rock core of φ 130 mm. The cores were photo documented and mapped with regard to rock types and fractures, and two rock cores were cut by water jet for identifying any corrosion.

No corrosion could be noticed on the rock bolts, and the space between steel and rock was well filled up with few pores, nor could any water channels be found and the grout was of a very good quality.

Thus, no corrosion could be found on any of the bolts in spite of the salinity of the groundwater. This indicates that the rock bolts were perfectly grouted with a high quality cement grout. In addition, the stagnant water surrounding the grouted rock bolts has not been corrosive due to the alkaline process since the installation of the bolts.

8.3.4 Groundwater inflow measurements

The measurements of groundwater inflow to the SFR underground facility are carried out with fixed time intervals in order to demonstrate changes in inflow volumes. The groundwater inflow measurements have been in progress since January 1988. The measurements are made in a number of measuring sections, total inflows from rock drainage basins and estimations of inflows. Table 8-3 shows the groundwater inflows measured and estimated in 2006. The locations of the various measuring points are shown in Figure 8-1.

As shown by Table 8-3, of the total inflow to the repository area, ≥ 18 l/min comes from the rock caverns and the silo. With respect to the relatively, evenly distributed extension of the dripping and moist spots occurring within the area that drains to pump pit UB, it is probable that the measuring points in the rock caverns do not succeed in measuring all of the inflows. Assuming a relatively, evenly distributed inflow within the area that drains to pump pit UB (approximately 17,000 m^2), the total water inflow to the rock caverns should be about 50 l/min, which corresponds to an average inflow of about 7 l/min 100 m rock cavern. This inflow should relate to that only one selective pre-grouting was carried out in 1BLA, namely in the minor deformation zone that extends diagonally over all of the rock caverns. An estimation of water inflows from roof and walls is given in Figure 3-10.

The measuring point Access tunnels in Table 8-3 is located in the upper part of the lower construction tunnel (NDB). Of the measured 200 l/min, about 170 l/min is coming from the first 700 m of the construction and operation tunnels, which represents near-surface excavation and excavation through the Singö zone. The water inflow to the remaining parts of the access tunnels is approximately 8 l/min and 100 m tunnel.

The total inflow to the pump pit in the lower construction tunnel (NDB) is 280 l/min of which 200 l/min is attributed to the access tunnels (cf. Figure 8-1). Assuming a water inflow of about 7 l/min and 100 m tunnel also in the lower construction tunnel, the contribution of water inflow from the gently dipping deformation zone H2 in the lowermost 30 m in the lower construction tunnel should be about 64 l/min.

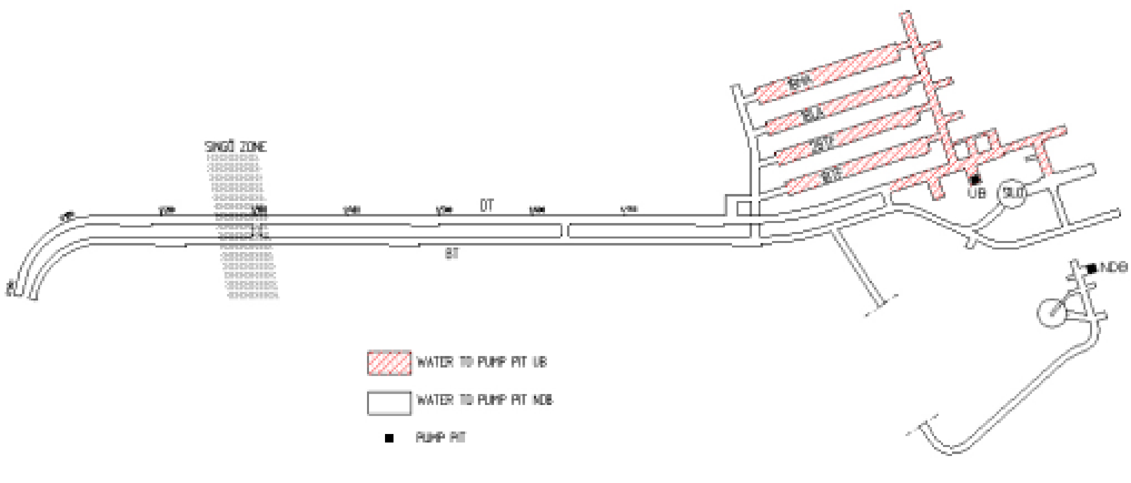


Figure 8-1. The locations of measuring points of groundwater inflow to the SFR underground facility.

In conclusion, about 60% of the total volume of the groundwater inflow comes from the 31% of the tunnel and rock cavern lengths, which were systematically pre-grouted (two deformation zones and the uppermost part of the access tunnels). The average groundwater inflow from other parts of the underground facility is 7–8 l/min and 100 m tunnel, primarily the repository and operation area with adjacent tunnels. In those areas, only a few selected grouting activities were performed.

The recurrent measurement of groundwater inflow to the SFR from January 1988 to date (2006) is shown in Figure 8-2. As demonstrated by the figure, there is a downward trend of inflow; the total water inflow has decreased from 720 l/min in 1988 down to about 320 l/min in autumn 2005 after which there is a sudden upward bend of the curves (380 l/min). This is most probably a consequence of the calibration of the water meters, which took place at that particular time.

Table 8-3. Measured groundwater inflow to the underground SFR facility, 2006.

Point of measuring	Inflow	Estimated accuracy of measurement	Fraction of total leakage	Notes
BMA	5 l/min	± 1 l/min	< 2 %	
1BLA	0 l/min	± 1 l/min	0%	Installed 2003. All water possibly not passing the measuring point.
2BTF	9 l/min	± 1 l/min	< 3 %	Installed 2003. All water possibly not passing the measuring point.
1BTF	3 l/min	± 1 l/min	< 1 %	Installed 2003. All water possibly not passing the measuring point.
Water to pump pit UB	75 l/min	± 1 l	~ 20 %	Water through siphon to the lower pump pit (NDB) and measured by water meter (installed in 1996). Earlier only pumping time was measured.
Access tunnels	200 l/min	± 10 l/min	~ 70 %	Collection dam constructed in 1992, measuring rule changed in 1994, calibration and new equation in 1997.
Silo top	0.1 l/min	± 0.1 l	< 1 %	Water meters often out of order due to iron precipitations.
Silo walls and bottom	0.8 l/min	< ± 0.1 l/min	< 1%	
Water to pump pit NDB	280 l/min	± 10 l	~ 80 %	Water from non-operational area. Water is measured by water meter on ground surface installed in 1996. Earlier the rise of water in the blasted pump pit NDB was measured. Rebuilt in 1999. Calibration of water meter in September 2005.

Note: The locations of the measuring points see Figure 8-1.

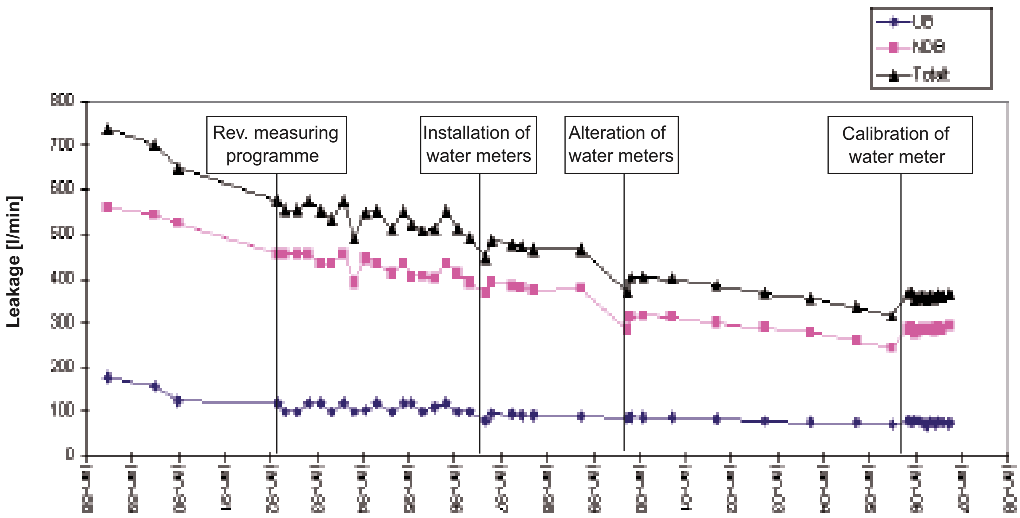


Figure 8-2. Measurement of groundwater inflow to the SFR underground facility.

9 Tunnelling experience at Forsmark – concluding remarks

9.1 Overall conclusions

In general, the experiences gained from the rock excavation works in the Forsmark area have shown that no major problems occurred during the execution of the works; nor have any stability or other rock engineering problems been identified after the commissioning of the Forsmark Plant and the SFR. The underground rock excavation works involve about 775,000 m^3, and a total tunnel length of about 11,000 m and effectively, all tunnels are sited undersea. The time after completion of the rock excavation works is 30 years (Forsmark 1 and 2 tunnel), 24 years (Forsmark 3 tunnel) and 20 years (the SFR) respectively.

It stands to reason that a number of various rock engineering difficulties and problems occurred during the works. In this context, the use of active design (Observational Method) throughout the underground excavations at Forsmark proved to be successful adopting satisfactory technical solutions in a timely and costly acceptable manner could solve the absolute majority of these problems. In addition, the overall good outcome of the underground excavations at Forsmark was much due to the smooth and professional cooperation between the contractors and the design teams. This cooperation was the basis of an efficient use of active design.

The only severe rock-engineering problem, which occurred during the construction at Forsmark, was the rock fall within the Singö deformation zone in the Forsmark 3 tunnel. However, the problem was restricted to a 10–20 m section of the tunnel, and the remedial measures used to stabilise the section was successful and professionally executed, and no evidence of instability has been registered ever since.

The experience from the Forsmark works, in particular the excavations of the SFR rock caverns and silo, shows clearly that 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 and deformation zones

In many cases, the choice of support type, support method and the synchronization of blasting with the support installation may be of greater importance for the efficiency and safety of the tunnelling than the fact that the rock is inferior. It other words, it is imperative to have the appropriate equipment in the correct location at the right time. At the same time, the choice of support methodology and preparatory measures are dependent on several factors, the most significant of which should be earlier precedence in experience of rock behaviour during tunnelling in a similar geological environment.

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 stress, changes in the fracture frequency and fracture orientation, clay 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.

The cooling water tunnel for Forsmark 1 and 2 passes through two major deformation zones, which are 50 and 200 m wide respectively. Together these deformation zones stretch over about 13% of the total length of the tunnel. Of the excavated 200 m of the Singö deformation zone, only about 40 m was found to have more adverse rock conditions, including open and clay-filled fractures with a maximum width of at most 500 mm. The rock mass in the remaining 150 m of the Singö zone may be characterized as having closed and healed fractures making up a competent rock from a construction point of view. The clay-filled fractures were supported using reinforced shotcrete arches, as well as mesh-reinforced arches. The rate of advance of the tunnelling operations was not affected significantly.

In contrast to the rock conditions in the Forsmark 1 and 2 tunnel, the rock mass conditions of a minor section in the Singö zone in the Forsmark 3 tunnel was significantly more inferior, larger quantities of water inflow, and as a consequence complex construction efforts were required (Figure 9-1).

In conjunction with the driving of the discharge tunnel for Forsmark 1 and 2, a general philosophy was developed for the reliability level of the tunnel's stability. This philosophy then formed the basis for the evaluations made in assessing the support measures required. This philosophy was then also implemented when the driving of the Forsmark 3 tunnel started, and the good conditions experienced from Forsmark 1 and 2 was also expected to be at hand in the Forsmark 3 tunnel. The unexpected problems encountered in the Forsmark 3 tunnel resulted in thorough planning and implementation of a pre-defined working methodology for the driving of the SFR tunnels.

A working method statement was elaborated for the driving through the Singö zone in the SFR tunnels /Carlsson, and Larsson 1983/. The method statement was based on the site investigations of the zone /Hagconsult 1982/ and on the experiences gained from the driving through the Singö zone in the Forsmark 1, 2 and 3 tunnels. The site investigations indicated that the rock mass quality was sufficiently good for tunnelling, and by applying appropriate working and support methods it was concluded that the driving through the zone could be accomplished

Figure 9-1. *Illustration of the difference in rock mass conditions within the Singö deformation zone between Forsmark 1 and 2 tunnel (upper pictures) and the Forsmark 3 tunnel (lower pictures).*

in a safe way. Large allowances were made for the driving of the SFR tunnels, which proved to be successful. Another factor that is also reflected in the rock support figures is the function of the tunnels. The SFR tunnels are transportation tunnels, whereas the discharge tunnels are water tunnels – a difference that has an effect on the total support measures that were taken.

Thus, it may be seen that it is difficult to clearly define and forecast geological phenomena of the type described above from pre-investigations and from feedback of experience within one and the same area. However, it must be emphasised that the actual location of the Singö deformation zone was foreseen, but the rock mass conditions in the specific inferior section of the deformation zone in the Forsmark 3 tunnel was unexpected due to a combination of unfavourable factors, which was in contrast to the experienced gained during excavation of the Forsmark 1 and 2 tunnel, and core-drillings through the deformation zone.

During the 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 that was expected. Examples of both of these cases were found in the tunnels described above.

9.3 Summary and concluding remarks

The two discharge tunnels, Forsmark 1 and 2 and Forsmark 3, are now completely water filled, both during normal operation and in the event of any stoppage. Emptying of the tunnel is not planned as a normal event during the service lifetime of the plant. Under unfavourable conditions, certain events may require the tunnels to be emptied, such as a gradual increase in friction losses as a result of deposits, increased roughness caused by blocks falling from the walls and roof, and local rock falls which results in more or less total blocking of the tunnel.

In the event of a sudden blocking of the tunnel resulting in a pressure loss of more than 2.5 m w.g. at full flow, immediate emptying and cleaning of the tunnel would be required, in which case the financial consequences due to the loss of energy production would be immense.

Against this background, the demand on the stability of the tunnel, expressed as the risk of events with disastrous consequences, should be lower for the Forsmark tunnels than for a corresponding hydropower tunnel, by a factor in proportion to the expected cost for loss in energy production.

This support philosophy formed the basis for the support directives in the Forsmark cooling water tunnels. It is important to point this out, because the intensity and extent of the support work cannot be justified compared to normal water tunnels. In general the tunnels are over-supported compared to conventional hydro tunnels of the same age. This support level was decided on in order to mitigate the risk for a serious consequence resulting from production loss of any of the nuclear units served by the tunnels.

The results from site investigations and feedback of experience from the driving of the Forsmark 1 and 2 tunnel formed the basis for the chosen support types, support methodology and stand-by equipment for installing supports in the Forsmark 3 tunnel. The set of measures planned also contained a certain degree of flexibility, to be able to meet and cope with special and unforeseen conditions.

The level at which this preparedness was set depended on a number of factors, such as the probability of a certain event occurring and the consequences of such an event on safety, the progress of work and the economy. In the case of the Forsmark 3 tunnel, on the basis of the observations made, the probability of an event that would result in long stoppage of tunnelling was judged to be small. Against this background, it was decided not to have steel arches as a contingency measure.

At the time when the tunnels were completed and, with the knowledge of the actual conditions acquired during tunnelling, this assessment may in hindsight be regarded as having been too optimistic. When the assessments were made, however, the feedback of experience from the driving through the Singö deformation zone in the Forsmark 1 and 2 tunnel, only about 400 m to the east of the Forsmark 3 tunnel, did not indicate that any severe construction problem would occur within the Singö deformation zone. Neither did the geotechnical site investigations forecast any significantly different conditions to be encountered. Thus, the experience from the earlier driving through the Singö deformation zone and the result from the site investigations constituted the basis for making the decisions and, taking this into account, the decision at that time may be regarded as having been realistic.

It is possible that the rock fall at tunnel section 2/545 would have been contained or completely prevented if pre-fabricated steel arches had been available as a contingency measure. About two weeks elapsed from the time that arching started to the time the steel arches were in position. The standing time that occurred is probably the single and primary explanation of the size of the rock fall. Another construction procedure utilizing shortened excavation rounds may also have reduced the problems.

The tunnelling and the underground excavations for SFR were carried out without any particular rock engineering difficulties. The method of driving the access tunnels proved to be successful, and the driving through the Singö deformation zone could accomplish according to plan.

In summary, it may be said that the result of the underground excavations at Forsmark was good, with the exception of the passage of one single, local zone of weakness in the rock mass within the Singö deformation zone in the Forsmark 3 tunnel. The measures, which were taken, resulted, however, in reassuring safety from a working point of view and perfectly satisfactory total stability of the tunnel. Thus, by applying active design (Observational Method) during the excavation works, the absolute majority of geological obstacles along the approximately 11,000 m tunnel length could be handled in a safe, efficient and economical way.

In the summer of 2006, there was a unique opportunity to inspect the F1 and F2 tunnel owing to a simultaneous standstill of the two nuclear units. The purpose of the inspection was to investigate and document potential rock falls or any apparent damages on rock support. The tunnel was completely water filled, and using a remote controlled U-boat carried out the inspection. On its way into the tunnel, the tunnel bottom was video recorded (in total 1,921 m), and on its way back sonar investigations were made in selected sections based on observations made during the video recording. In addition profile measurements for checking the cross-sectional areas were made each 10 m, supplemented with profile measurements in sections where special observations had been made during the video recording.

The result of the inspection demonstrates that the tunnel is in good condition. No occurrences of rock falls or larger falls of shotcrete could be identified /Forsmarks kraftgrupp AB 2006/.

The underground excavation works for SFR was completed in 1986; i.e. the rock support in the underground tunnels and caverns was applied for more than 20 years ago. In conclusion, the result from the various inspections, testing and measurements carried out during the operation since 1988 do not indicate any significant deterioration or failure of installed rock support. The host rock mass after excavation has performed as foreseen, and no stability problems or unexpected deformation have been observed, nor has any problem related to groundwater been identified.

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