Technical Report

TR-99-08

Background report
to SR 97

SR 97

Waste, repository design and sites

October 1999

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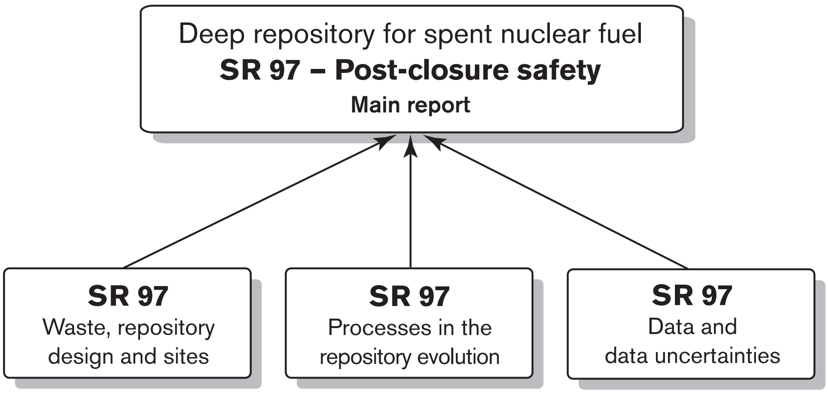
Foreword

The present report, SR 97 – Waste, repository design and sites – comprises a main reference for the main report: Deep repository for spent nuclear fuel; SR 97 – Post-closure safety. The SR 97 report is a comprehensive safety assessment of the KBS-3 method for the deep disposal of spent nuclear fuel. A further precondition is that geosphere data be taken from three actual sites in Sweden.

The figure below shows the relationship between the main report and the most important background reports.



SR 97 – Main report summarizes the entire safety assessment. It can be read independently of the others and contains all essential results. The report is available in both Swedish and English.

SR 97 – Waste, repository design and sites describes the waste, the repository design with canisters and buffer/backfill material, the three sites and the site-specific adaptations that have been made of the repository layouts. The report is available in both Swedish and English.

SR 97 – Processes in the evolution of the repository describes the thermal, hydraulic, mechanical and chemical processes in fuel, canister, buffer and geosphere that control the evolution of the repository system. The report is available in both Swedish and English.

SR 97 – Data and data uncertainties (in English only) contains a compilation of input data for calculations of radionuclide transport. It also contains an evaluation of uncertainties in input data.

This background report of SR 97 is based on background material furnished by Patrik Sellin, Anders Ström, Christer Svemar, Kaj Ahlbom, and Bengt Leijon of Svensk Kärnbränslehantering AB, Jan Hermanson of Golder Grundteknik, and Raymond Munier of Scandiaconsult. The material has been edited by Karin Pers, Kemakta Konsult.

Contents

1 Introduction

The SKB project SR 97 is a safety assessment for a repository for spent nuclear fuel in the Swedish crystalline basement. The purpose of the assessment is to demonstrate the feasibility of finding a site in Swedish bedrock which fulfils the demands of long-term safety and radiation protection as provided in SSI's and SKI's directives.

SR 97 is a comprehensive analysis of long-term safety of a deep repository for spent nuclear fuel. The repository is assumed to be designed according to the KBS-3 method. Assessments are performed in SR 97 for three fictitious sites: Aberg, Beberg and Ceberg. One premise is that data used for assessment of the fictitious sites are to be taken from sites that have previously been investigated. Data from Äspö in Småland is used for Aberg, data from Finnsjön in Uppland for Beberg, and data from Gideå in Ångermanland for Ceberg. The names of the sites have been chosen to indicate clearly that they are not the actual candidate sites for a deep repository.

With this point of departure, the analyses in SR 97 shall partly show that a deep repository with an acceptable safety standard can be designed in the bedrock that is prevalent in Sweden; partly show how a repository can be adapted to the data and situation at a particular site; and partly show how safety can be evaluated at the actual sites.

The three sites have been investigated by different experts at various times over a period of twenty years, on occasion using different systems and scopes. Thus, there are differences between the terms used in each of the sites (for example, to describe fracture zones), between what has been investigated and how the results have been analysed and presented. In SR 97 we have chosen to keep the original nomenclature and presentation method for the main part, which means that the terms and formats of maps differ in some respects between the three sites.

Sveriges Geologiska Undersökning has recently presented a geological overview of the counties where SR 97 data had been acquired. The results of these overviews have not been incorporated into SR 97.

The conceptual design of the repository system is based on the KBS-3 method for deep disposal in Swedish crystalline bedrock. The positioning of deposition tunnels and various access tunnels, tunnels for ventilation etc. is dependent on the local conditions and will therefore be different for the three areas.

The spent nuclear fuel is enclosed in copper canisters with an insert of cast iron. The canisters are emplaced in bored holes in the floor of the deposition tunnels. Around each canister, bentonite blocks are stacked which, after absorbing water and swelling, will isolate the canister from groundwater, hold the canister in place and retard transport of radionuclides from the canister to the surrounding rock. The spent nuclear fuel will emit heat for a long time, due to the decay heat. The maximum permissible temperature on the canister surface has been chosen at 100°C. The spacing between the deposition holes and between the deposition tunnels is adjusted site-specifically to meet this requirement. The thermal properties of the rock and the buffer material are of importance for how closely the deposition holes and tunnels can be spaced. After deposition, the deposition tunnels are backfilled with a mixture of bentonite and crushed rock.

SR 97 is a comprehensive safety assessment which above all examines the consequences of various scenarios and the handling of various types of uncertainties. The different repository sites illustrate normal properties for Swedish bedrock which are of importance for safety. To facilitate the work, the repositories on the three sites are configured as similarly as possible, which means for example that they are located at roughly the same depth and are fitted into the bedrock in a relatively similar fashion.

Apart from the siting of a repository for spent nuclear fuel, the site may need to house a separate repository for other long-lived waste. This possibility has been considered in the site-specific configuration of the repositories in Aberg, Beberg and Ceberg. However, other sitings of the repository for other long-lived waste are possible. Other long-lived waste consists of, among other things, waste from the research activities at Studsvik. The category of other long-lived waste also includes core components and reactor internals that have been situated close to the fuel core in the power reactors. Besides the waste from Studsvik and the reactors, the repository for other long-lived waste will also receive short-lived LILW (low- and intermediate-level waste) from CLAB and from the encapsulation plant that is produced after the closure of SFR 1 in Forsmark. SKB investigates the long-term safety of the repository for other long-lived waste in a separate safety report.

A brief description of the waste intended to be disposed of in a deep repository is given in Chapter 2. The information is based on data and assumptions in PLAN 98. Spent fuel equivalent to approximately 4,500 canisters is generated during operation of the nuclear reactors for 40 years. The BWR fuel SVEA 96 has been chosen as the reference fuel. The activity level in the reference fuel has been calculated for a period of 40 years after discharge from the reactor. The calculations include fission products, actinides and activation products.

Chapter 3 follows with a description of the conceptual design of the repository system based on the KBS-3 method for deep geological disposal in Swedish crystalline bedrock. The engineered barriers consist chiefly of copper canisters with an insert of cast iron, buffer and tunnel backfill plus plugs.

An account of the geoscientific properties of the different repository sites is given in Chapter 4. The account includes completed geoscientific investigations, geological/structural descriptions on a regional and local scale, descriptions of rock mechanics, hydrogeology and groundwater chemistry, and descriptions of the thermal properties of the different sites.

Site-specific adaptation of the repositories is dealt with in the last chapter. The main factors that influence the layout of the repositories are rock stresses and the direction and properties of individual fractures, and water-bearing zones.

Based partly on the information presented in this report, data comprising initial conditions for calculations and evaluations in the SR 97 assessment have been summarized in the main report. The main report also describes the uncertainty of the individual parameters.

2 Spent nuclear fuel

2.1 Quantities and burnup

Every year SKB submits an account of the production of energy and nuclear waste in the Swedish nuclear power programme, known as the PLAN report. The data on fuel quantities and burnup in this report are taken from PLAN 98 /1998/.

According to PLAN 98, total energy production with 12 reactors operating for 25 years, but at least until 1999, will be about 1,650 TWh. The total quantity of spent nuclear fuel to be disposed of then amounts to about 6,500 tonnes. Assuming 40 years of operation, total electricity production is 2,700 TWh and the quantity of spent nuclear fuel is about 9,500 tonnes. Both alternatives include 20 tonnes of spent fuel from the out-of-service heavy water reactor in Ågesta and from the R1 research reactor, plus 23 tonnes of spent MOX fuel.

If the reactors are operated for 40 years, 4,500 canisters will be required for the quantity of spent fuel produced.

2.2 Choice of reference fuel

A BWR assembly of type SVEA 96 with a burnup of 38 MWd/kg U has been chosen as the reference fuel in SR 97. The assembly consists of 96 rods divided into four bundles, enclosed by a fuel channel. The active fuel part in a BWR fuel assembly is approximately 3.7 m long.

The BWR assembly will be the dominating fuel in the repository and 38 MWd/kg U corresponds to the expected average burnup of all fuel in the repository. The higher the fuel burnup rate, the higher the radionuclide content and the higher the decay heat. Decay heat is the parameter governing the amount of fuel that can be deposited in a canister. This means that burnup will be “mixed” with low burnup fuel in order to obtain the right decay heat in the canister. Thus, the total radionuclide content will not differ too much between the different canisters.

PWR fuel is only marginally different from BWR fuel as regards radionuclide content and other differences e.g. geometry, are treated so simplistically in the safety assessment that the differences are of no importance.

The MOX fuel assembly (Mixed Oxide i.e. plutonium mixed with uranium dioxide) consists of a mixture of MOX rods and uranium rods. The differences between MOX and uranium fuel are given in /Forsström, 1982/. A MOX rod contains around 5 times as much plutonium and around 14 times more americium and curium than a uranium rod, calculated on a burnup of 33 MWd/kgHM. 40 years after discharge, the decay heat is 3 times higher in a MOX rod than in a uranium rod. The higher content of Cm-244 in a MOX rod means that the dose of neutron is around 20 times higher than in a uranium rod.

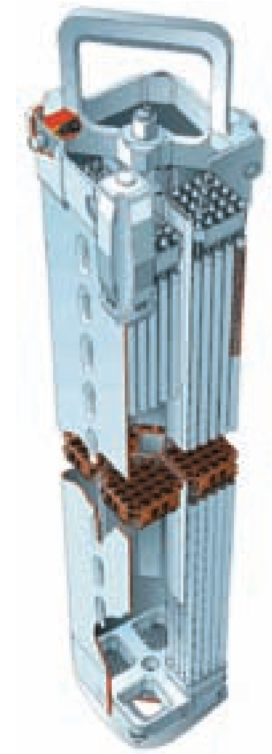


Figure 2-1. *Fuel assembly of type SVEA 96.*

The difference between a canister containing a "normal" uranium fuel and that containing MOX fuel depends totally on how many rods containing MOX fuel there are in the canister. The permitted decay heat sets the maximum number of MOX rods that can be deposited in a canister. However, the decay heat in a canister with MOX declines rather more slowly than in a canister containing reference fuel only. This has a local effect on the temperature, of several hundred years, near the MOX canisters. The highest temperature permitted in a canister will, however, never be exceeded. The local effects on rock temperatures and rock stress are insignificant, since the total amount of MOX fuel in the repository is less than 0.5%. The amount of actinides will always be higher in a canister containing MOX fuel, but this is of limited importance as regards the safety of the repository since the release of actinides from a defective canister is governed by solubility and is independent of the inventory. The higher dose of neutron from MOX fuel only marginally affects the radiolysis of pore water outside the canister and is meaningless as regards the canister's lifetime.

Figure 2-1 shows a picture of a Svea 96 fuel assembly. The reference fuel has been used to calculate radionuclide inventory and decay heat in the safety assessment.

2.3 Radionuclide content and decay heat

Decay heat, radionuclide content (disintegration rate, Bq/t U) and alpha, beta, gamma and neutron source strengths in the spent nuclear fuel have been calculated with the aid of the computer programs SCALE and ORIGEN-S /Håkansson, 1998/. The calculations have included times from one week to 300,000 years after irradiation in the reactor. The activity level in the reference fuel 40 years after discharge from the reactor is given in Table 2-1.

Table 2-1. Activity in the reference fuel (SVEA 96, 38 MWd/kg U) 40 years after discharge from the reactor /after Håkansson, 1998/.

Nuclide	Activity (Bq/t U) 40 yrs after discharge	Nuclide	Activity (Bq/t U) 40 yrs after discharge	Nuclide	Activity (Bq/t U) 40 yrs after discharge
Fission products		*Actinides*		*Activation products*	
H-3	$2.1\cdot10^{12}$	Ra-226	$9.3\cdot10^{8}$	H-3	$1.1\cdot10^{12}$
Se-79	$2.8\cdot10^{9}$	Th-229	$6.1\cdot10^{7}$	C-14	$5.0\cdot10^{10}$
Kr-85	$2.7\cdot10^{13}$	Th-230	$2.1\cdot10^{9}$	Cl-36	$5.5\cdot10^{8}$
Sr-90	$1.2\cdot10^{15}$	Th-234	$1.2\cdot10^{10}$	Fe-55	$9.3\cdot10^{9}$
Y-90	$1.2\cdot10^{15}$	Pa-231	$3.0\cdot10^{7}$	Co-60	$8.9\cdot10^{11}$
Zr-93	$5.0\cdot10^{10}$	Pa-233	$1.5\cdot10^{10}$	Ni-59	$8.8\cdot10^{10}$
Nb-93m	$4.2\cdot10^{10}$	Pa-234m	$1.2\cdot10^{10}$	Ni-63	$9.3\cdot10^{12}$
Tc-99	$5.7\cdot10^{11}$	U-233	$5.6\cdot10^{8}$	Sr-90	$2.6\cdot10^{7}$
Ru-106	$2.7\cdot10^{4}$	U-234	$4.6\cdot10^{10}$	Y-90	$2.6\cdot10^{7}$
Pd-107	$4.9\cdot10^{9}$	U-235	$4.8\cdot10^{8}$	Zr-93	$5.6\cdot10^{9}$
Cd-113m	$1.7\cdot10^{11}$	U-236	$1.0\cdot10^{10}$	Nb-93m	$2.3\cdot10^{10}$
Sn-121	$4.4\cdot10^{10}$	U-237	$1.9\cdot10^{10}$	Nb-94	$2.9\cdot100^{9}$
Sn-121m	$5.7\cdot10^{10}$	U-238	$1.2\cdot10^{10}$	Mo-93	$4.4\cdot10^{7}$
Sb-125	$1.1\cdot10^{10}$	Np-237	$1.5\cdot10^{10}$	Ag-108	$4.3\cdot10^{7}$
Te-125m	$2.7\cdot10^{9}$	Np-239	$1.2\cdot10^{12}$	Ag-108m	$5.0\cdot10^{8}$
Sn-126	$2.3\cdot10^{10}$	Pu-238	$9.5\cdot10^{13}$	Cd-113m	$3.4\cdot10^{10}$
Sb-126m	$2.3\cdot10^{10}$	Pu-239	$9.5\cdot10^{12}$	Sn-121	$1.4\cdot10^{10}$
I-129	$1.3\cdot10^{9}$	Pu-240	$1.2\cdot10^{13}$	Sn-121m	$1.7\cdot10^{10}$
Cs-134	$9.1\cdot10^{9}$	Pu-241	$7.7\cdot10^{14}$	Sb-125	$1.2\cdot10^{9}$
Cs-135	$2.1\cdot10^{10}$	Pu-242	$1.0\cdot10^{11}$	Te-125m	$3.0\cdot10^{8}$
Cs-137	$1.8\cdot10^{15}$	Am-241	$1.5\cdot10^{14}$	Eu-154	$3.2\cdot10^{11}$
Ba-137m	$1.7\cdot10^{15}$	Am-242m	$4.5\cdot10^{11}$	Eu-155	$1.3\cdot10^{10}$
Pm-146	$9.8\cdot10^{8}$	Am-242	$4.5\cdot10^{11}$	Ho-166m	$7.5\cdot10^{7}$
Pm-147	$1.5\cdot10^{11}$	Am-243	$1.2\cdot10^{12}$	*Total*	$1.2\cdot10^{13}$
Sm-151	$9.4\cdot10^{12}$	Cm-242	$3.7\cdot10^{11}$		
Eu1-52	$3.3\cdot10^{10}$	Cm-243	$4.4\cdot10^{11}$		
Eu-154	$1.8\cdot10^{13}$	Cm-244	$2.8\cdot10^{13}$		
Eu-155	$7.6\cdot10^{11}$	Cm245	$9.4\cdot10^{9}$		
Total	$6.0\cdot10^{15}$	Cm-246	$2.9\cdot10^{9}$		
		Total	$1.1\cdot10^{15}$		

Most of the radionuclides in the fuel assemblies are found inside or on the outside of the uranium dioxide; smaller quantities are found on the inside of the cladding tubes and in the fuel-clad gap. In addition there are activation products in the other metal parts in the fuel, e.g. fuel channels and grab handles. Part of the inventory of certain radionuclides (e.g. iodine and cesium) can find its way to the fuel's surface when the reactor is in operation. The typical value of the inventory on the surface is 3–6%.

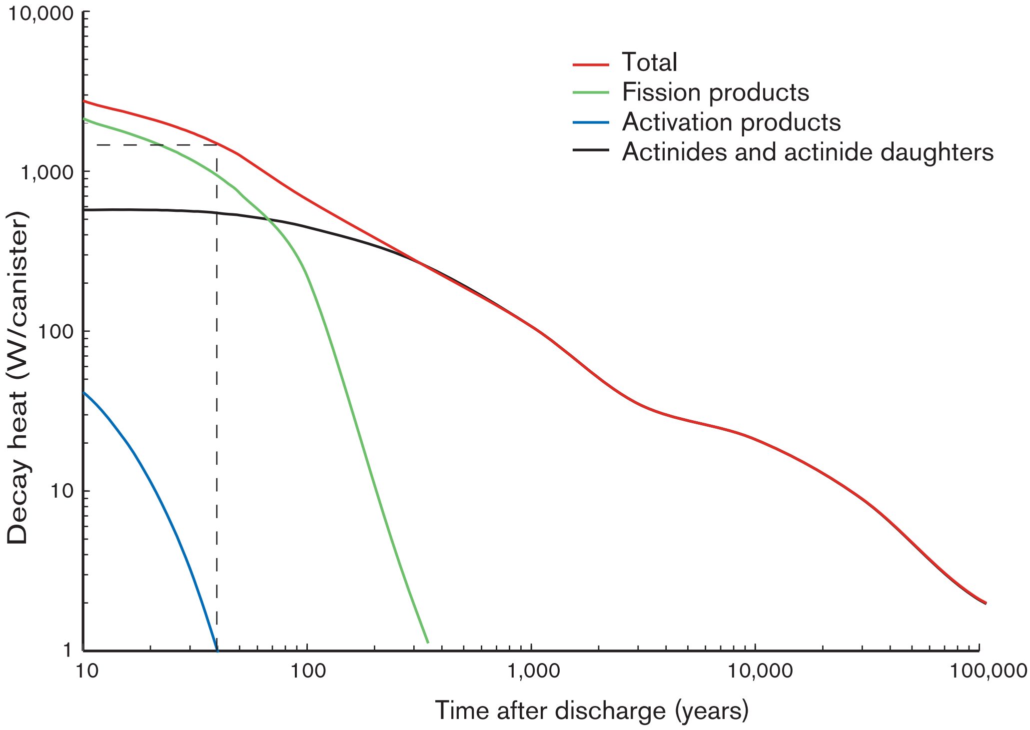


Figure 2-2. *Decline of decay heat as a function of time in the reference fuel.*

The decay heat in the fuel stems from the radioactive disintegration that takes place in the fission products and varies more or less in proportion to the fuel's burnup. The decline of the decay heat is shown in Figure 2-2. The decay heat refers to a canister containing 12 BWR assemblies with 170 kg of spent fuel each (uranium content).

Since burnup affects the composition and structure of the fuel, and future burnups will differ from those assumed for the reference fuel, calculations have also been done for a higher burnup of 55 MWd/kg U /Håkansson, 1998/.

2.4 Chemotoxicity of the waste

Besides potential health effects due to the waste's content of radionuclides, in describing the safety of a repository it is also necessary to consider its content of chemotoxic substances. Often a substance can be both potentially harmful due to its radioactivity and at the same time toxic as a chemical substance.

General inventories of chemically toxic substances in canisters of spent fuel have been done by Wiborgh and Markström /1991/. The spent fuel mainly contains uranium, but also small quantities of other elements produced by the nuclear reactions. Some of the chemical elements formed in the fuel may be toxic or harmful to the environment, aside from the harmful effects due to radioactivity. Examples of such elements are silver, barium, cadmium, antimony and selenium. Harmful metals such as chromium and nickel are present in the metallic parts of the fuel assemblies, and even the copper of which the canisters are made can be said to possess some toxicity. However, uranium is the dominant component, owing to the large quantities of uranium in the repository and to the relatively high radiotoxicity of uranium.

3 Repository design

The conceptual design of the deep repository is based on the KBS-3 method (see figure 3-1). The spent fuel is enclosed in canisters consisting of a cast iron insert for mechanical stability and a copper shell for corrosion protection. The canisters are deposited in bored holes in the floor of a system of deposition tunnels. Each hole accommodates one canister. The canisters are surrounded by a buffer of bentonite clay, which holds them in place and isolates them from circulating groundwater and protects them from minor rock movements in the surrounding rock. The clay also retards the transport of various substances to and from the canister.

Based on the requirements made on different parts of the system, the design of the repository system is described with an emphasis on the underground sections for spent nuclear fuel. Canister, bentonite buffer and backfill material are then dealt with.

3.1 Conceptual design of the repository system

3.1.1. General

The layout and design of rock caverns, tunnels, deposition positions for spent nuclear fuel etc. in the repository system is based on principles presented in the KBS-3 study /KBS-3, 1983/.

Several alternative solutions have previously been studied for repository systems located at mine depth, i.e. between 400 and 700 m below the ground surface /PASS, 1992; Skagius and Svemar, 1989/. In comparisons with the KBS-3 method, some of these alternatives have exhibited economic advantages. The ongoing development work leads to constant changes in the repository design and the choice of working methods. However, the modified solutions in the present study always satisfy at least the same functional requirements as in previous designs. The following description of the repository system is based on the current design /PLAN 98, 1998/.

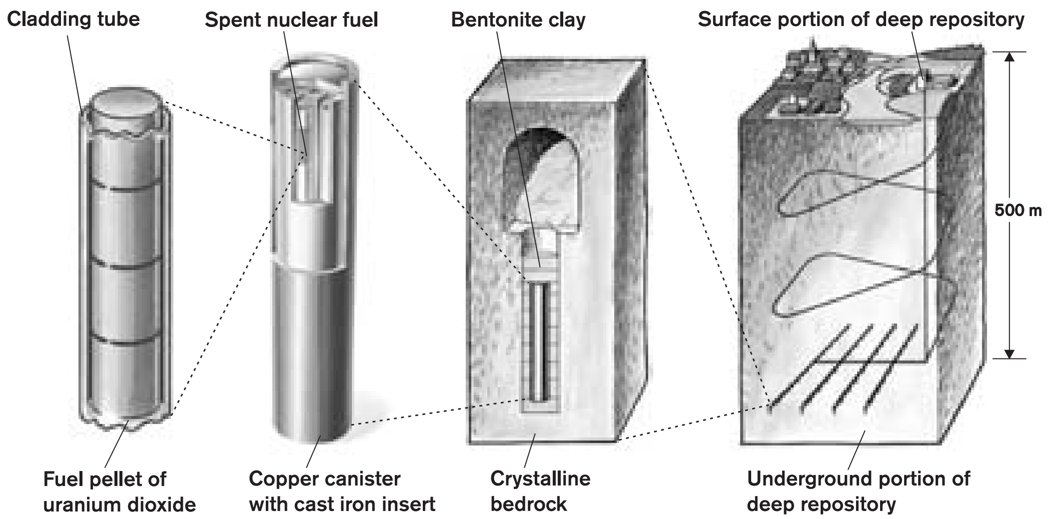


***Figure 3-1**. Illustration of KBS-3 method for the storage of spent nuclear fuel.*

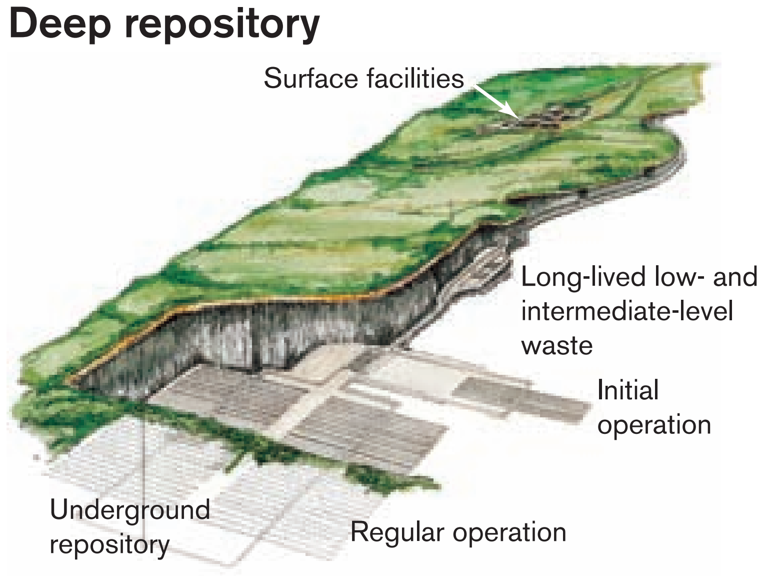


***Figure 3-2**. Schematic drawing of the deep repository with access ramp.*

The different parts of the deep repository are:

– surface facility

– access (shaft or ramp)

– central area under ground

– area for deposition of spent nuclear fuel

The design is adapted to the chosen site. An example of the design is shown in Figure 3-2.

The repository for spent nuclear fuel consists of a large number of parallel deposition tunnels with deposition holes bored in the bottom. Canisters with spent nuclear fuel are emplaced in the holes, together with a surrounding clay buffer.

Figure 3-3 shows a cross-section through a canister position. The dimensions are determined by the size of the canister, the space needed for rock works, operation and deposition, and the desired performance and safety after closure /Svemar, 1995/. In practice, certain deviations will be made from the indicated dimensions, depending on such factors as the choice of rock excavation method and the method used for lowering the canisters and bentonite into the deposition holes.

The deposition tunnels are connected by tunnels for transport of fuel canisters, materials and personnel, as well as tunnels for ventilation and utility lines. The transport tunnels are connected to a central area underground and via tunnels and shafts to the ground surface.

The decay heat in the deposited spent nuclear fuel will lead to a heating of the repository. This means that the placement of the deposition tunnels, as well as the spacing between the positions for the deposited canisters, is determined in reference to a limitation of the temperature on the canister surface to a maximum of 100°C. The thermal properties of the local rock also determine how closely canisters can be spaced /Ageskog and Jansson, 1999/. Local rock and groundwater conditions will ultimately determine how large a portion of the theoretically conceivable number of canister positions can be utilized.

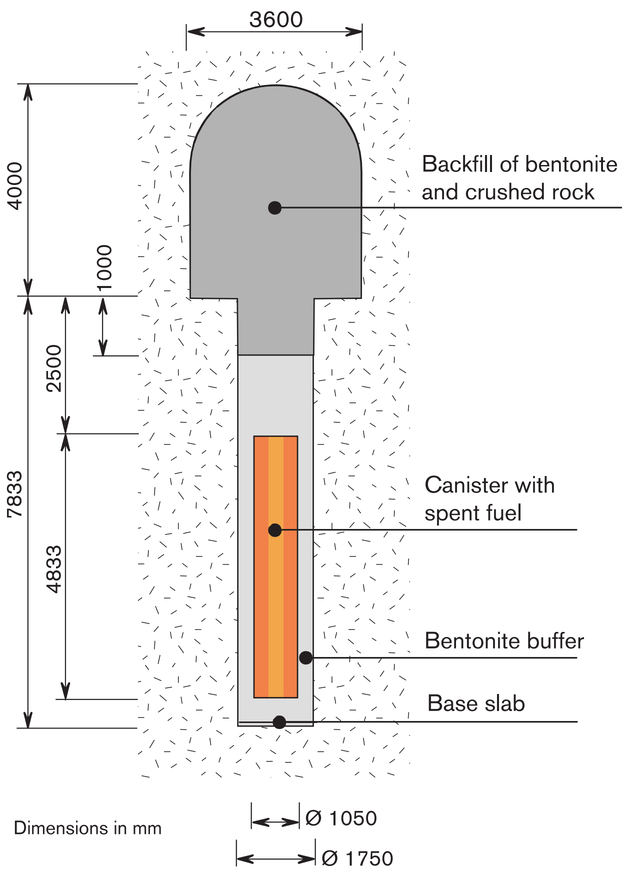


Figure 3-3. *Cross-section of deposition hole.*

The above design assumes that all tunnels are bored or blasted using conventional methods (careful blasting) and that the deposition holes are shaft-bored.

3.1.2 Dimensions of deposition tunnels

Since the canisters are transported down to the deposition positions surrounded by a radiation shield, the height of the deposition tunnels must accommodate the equipment used. Furthermore, natural ventilation alone is assumed in the tunnel with no space-taking ventilation ducts.

3.1.3 Dimensions of deposition holes

The distance between the bottom of the tunnel and the top of the canister is determined by:

– the requirement that the canister be located outside the excavation-disturbed zone under the tunnel floor

– influence of stress redistribution zone

– influence of bentonite swelling, which is dependent on the backfill material in the tunnel

– influence on the bentonite of any concrete on the floor of the tunnel

– possible grouting in the rock under the tunnel

- possible bolting for rails on the tunnel floor
- resistance to radionuclide transport in buffer and backfill
- radiation shielding permitting human presence in the tunnel.

The requirement that the canister be located outside of the excavation-disturbed zone entails a distance of 2.5 m between the top of the canister and the floor of the tunnel. To achieve low hydraulic conductivity in the buffer around the canister and prevent excessive loss of imperviousness in the buffer above the canister, the thickness of the buffer has been chosen to be 1.5 m. Above the buffer, 1.0 m of the deposition hole is filled with bentonite and crushed rock up to the floor of the deposition tunnel.

The thickness of the buffer under the canister is determined by:

- influence of the base pad on the bentonite if concrete is used
- bearing capacity of the buffer
- swelling capacity of the buffer
- hydraulic conductivity of the buffer
- resistance of the buffer to nuclide transport
- heat transport through the buffer.

To allow for the possibility of a concrete bottom pad, which can have a negative influence on the properties of nearby bentonite buffer, and to ensure that the barrier to nuclide transport has at least the same capacity as other parts of the buffer, the thickness of the buffer underneath the canister is chosen to be 0.5 m.

The diameter of the hole is the sum of the diameter of the canister, the original thickness of the bentonite blocks, and the tolerances between the canister and the blocks and between the blocks and the rock. The thickness of the bentonite buffer is in turn determined by the desired mechanical, chemical and hydraulic properties and the thermal performance of the buffer. Allowance has also been made for the desired gas transport capacity.

The permissible heat output of the canister is limited by the temperature that can be tolerated on the canister surface. A thicker buffer will reduce the quantity of spent fuel that can be loaded into each canister since the buffer material has lower heat-conducive properties than the surrounding rock. In view of this and the requirement of maintaining a diffusion barrier around the canister that will last for the whole lifetime of the repository, the buffer is made 0.35 m thick, which leads to a hole diameter of 1.75 m for the actual canister design.

3.1.4 Spacing between canisters and between deposition tunnels

The spacing between deposition holes is primarily determined by:

- quantity of fuel per canister
- thickness of the buffer
- permissible temperature on the canister surface and in the buffer
- original temperature of the rock

– thermal conductivity of the buffer
– thermal conductivity of the rock
– spacing between deposition tunnels
– permissible thermal load per unit of horizontal surface area
– strength of the rock against thermally induced stresses
– requirement on limited hydraulic connection between deposition holes.

The design basis factor is the permissible temperature rise on the canister surface. For the chosen canister, using thermal data for ordinary granite, and assuming an original temperature of 15°C in undisturbed rock and a maximum permissible temperature of 100°C on the canister surface, and with an economic optimization between canister spacing and tunnel spacing, the canister spacing is 6.0 m and the tunnel spacing 40 m. Examples of factors that could permit a shorter spacing between canisters and tunnels are low original temperature in the rock, higher thermal conductivity of the buffer and higher quartz content of the rock and thereby better thermal conductivity than in ordinary granite.

3.1.5 Deposition positions

It is unlikely that all deposition holes in the deposition pattern described above can be utilized. Local properties in the bedrock can, for example, lead to poorer properties for safe long-term isolation of the waste in certain positions. The factors that have been identified as being of importance in deciding whether a canister position can be utilized or not are /Rosén and Gustafson, 1995/:

– lithology (composition of the rock mass)
– inflow of water to the deposition hole
– stability from a construction viewpoint
– long-term stability
– probability of tectonic impact
– thermal properties of the rock.

A span of acceptable values can be determined for each factor. By assessing and setting bounds on these factors and then applying them to the specific repository site, a degree of utilization of the rock volume can be calculated.

3.2 Impact on properties of near-field rock

3.2.1 General

The degree to which the host rock is affected by the repository for spent nuclear fuel is dependent on such factors as the geological conditions within the area, the rock excavation methods used and the thermal load to which the rock is subjected. Blasting, drilling, boring and extraction of rock will affect the properties of the rock in walls, roofs and floors around shafts, tunnels and rock caverns. Shaft boring of deposition holes is expected to affect the rock to a lesser extent than blasting. Heating and subsequent cooling of the rock near emplaced canisters with spent nuclear fuel also affect the properties of the rock.

The impact on the properties of the near-field rock is described below, assuming the deposition holes are shaft-bored and the deposition tunnels are drilled and blasted in the conventional manner (careful blasting). The effect of heat generation in the emplaced canisters is described briefly.

3.2.2 Shaft boring of deposition holes

Shaft boring of 1.75 m diameter deposition holes has very little impact on the surrounding rock. The new fractures that form during boring will probably not extend more than 100 mm into the wall rock /Lindqvist et al., 1994/. To this must be added the elasto-plastic change undergone by the near-field rock as a consequence of the stress redistribution that occurs in conjunction with boring. Experimental boring of 1.52 m diameter holes at Olkiluoto /Autio, 1997/ has shown that a change in the form of increased porosity in the rock can be discerned up to a depth of about 20 mm from the hole wall.

In Äspö, the rock around a TBM (Tunnel Boring Machine) tunnel with a diameter of about 5 m has been investigated. The vibrated zone with elevated porosity is limited to about 30 mm. Further out from the tunnel, stress redistributions lead to changes of a less serious nature (see Figure 3-4). In Grimsel, Switzerland, drill cores have been extracted from the wall of a TBM tunnel at a depth of around 450 m. Changes could be noted up to a depth of 30 mm from the tunnel wall in these samples as well /Winberg, 1995/.

The conclusions drawn thus far are that a small zone is formed near the rock wall during shaft boring that may have elevated hydraulic conductivity in the axial direction of the deposition hole. This is advantageous for the water saturation of the buffer, since an even distribution of water along the entire deposition hole is thereby achieved, which facilitates an even water saturation of the bentonite buffer.

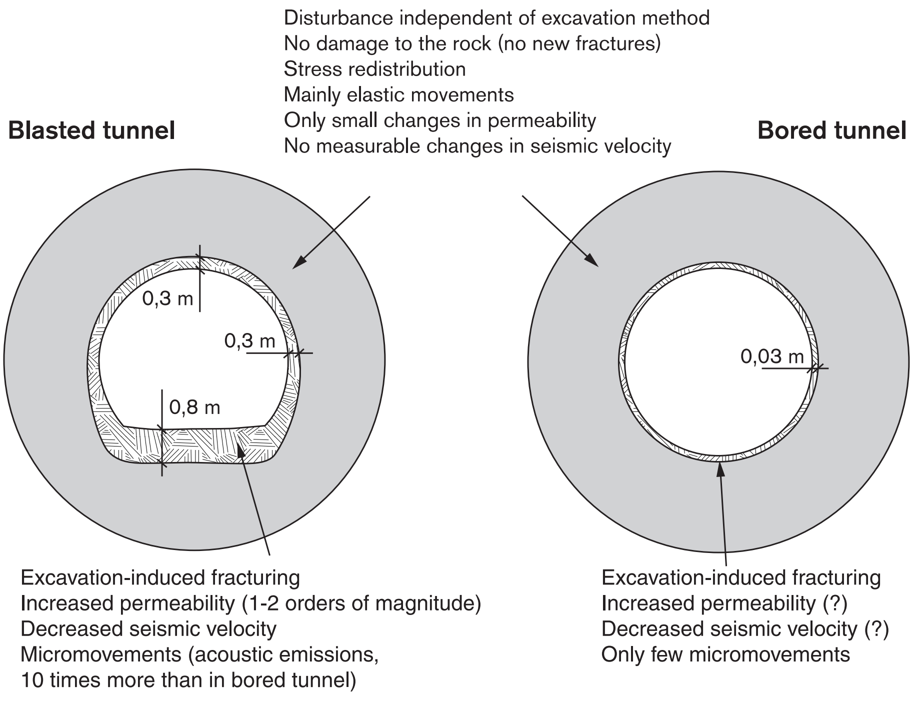


***Figure 3-4**. Extent of the disturbed zone (EDZ = excavation-disturbed zone) around a drill-and-blast tunnel as opposed to a TBM tunnel. Compilation of results from ZEDEX experiments in the Äspö HRL.*

3.2.3 Blasting of deposition tunnels

A disturbed zone will occur around the deposition and other tunnels causing a slightly different fracture geometry and permeability in the area around the tunnels. The size of the disturbed zone is greater in a blasted tunnel than in a drilled one. Blasting damage tests in the Äspö ramp /Pusch and Stanford, 1992/ and in the ZEDEX tunnel /Emsley et al., 1997/ confirm the preliminary estimates of the extent of the damages that were made in the Stripa project /Börgesson et al., 1992/, as well as the theoretically calculated damages /Andersson, 1994/. The tests in the Äspö ramp and the ZEDEX tunnel indicate a 0.3 m zone outside the walls and roof and a 0.8–1.5 m zone beneath the floor that are significantly affected. The results of hydraulic measurements in the ZEDEX tunnel show an increase in permeability in the blast-damaged zone. However, it was not possible to quantify this increase on the basis of the results. Further out from the tunnel, stress redistribution leads to changes of a less serious nature (see Figure 3-4).

Pusch and Börgesson /1992/ estimate that the axial hydraulic conductivity increases by a factor of 100 to 1,000 in the excavation-disturbed zone (on average 100 in a 1 m thick zone) and by a factor of 10 in the stress redistribution zone outside (one tunnel diameter out). Few measurements are available which confirm this estimate, however. Some results indicate instead that hydraulically interconnected structures only extend one or two metres in the axial direction /Read, 1996/.

3.2.4 Grouting

The safety of the deep repository is not predicated on any measures to seal the rock or otherwise improve its properties. It is, on the other hand, important to keep groundwater seepage under control during construction and operation. A possible method to reduce groundwater seepage is to grout the water-bearing fractures with e.g. cement.

3.2.5 Heating

Heating of the rock increases the stresses. Since the bentonite in the deposition hole exerts a pressure on the rock wall, there is little probability of movements along the fracture plane. After cooling due to radioactive decay, the thermally induced stresses will have ceased and the rock around the deposition holes will presumably resume the original properties of the near-field rock.

3.3 Canister

3.3.1 General

The canister comprises a fundamental engineered barrier in the repository system. A detailed discussion of the design premises and function requirements for the canister can be found in Werme /1998/. The canister has two primary functions in providing the necessary isolation in the deep repository:

1. The canister must retain its integrity over a long time. This imposes requirements on:
   - initial integrity
   - chemical resistance in the environment that exists or will exist in the deep repository
   - mechanical strength under the conditions expected to prevail in the repository for a long time.

2. The canister must not exert any harmful effect on the other barriers in the deep repository. This imposes requirements on:
   - choice of material that does not adversely affect the functions of the buffer and rock
   - limitation of heat and radiation dose in the near field
   - a design such that the configuration of fissile material in the fuel remains subcritical even if water enters the canister
   - limitation of the canister's downward pressure against the bentonite.

3.3.2 Design

The canister consists of two components: a cast insert and a copper shell /Werme and Eriksson, 1995/. The cast insert with individual channels for each fuel assembly lends the canister the necessary mechanical strength to withstand external pressures in the deep repository and loads arising in handling. The approximately 50 mm thick copper shell lends the canister corrosion resistance. Besides providing the necessary protection against corrosion for a long time after the repository has been water-saturated, the copper shell must also withstand atmospheric corrosion before the canister is emplaced and during the repository's saturation phase after deposition.

The design of the canister has not yet been finalized in detail, but may be modified in response to the requirements made by the fabrication and encapsulation processes. One possible design is shown in Figure 3-5. This design, with an outer copper shell and a cast iron insert with room for 12 BWR assemblies, serves as the basis for SR 97. The copper shell can be fabricated either as a seamless tube by extrusion, or from two plates that are formed into tube halves and welded together by two longitudinal welds. Lid and bottom can be fabricated by machining from thick plate or can be forged and machined to their final shape.

***Figure 3-5**. Design of canister shell and insert.*

The total weight of a canister filled with 12 BWR assemblies is about 24.5 tonnes with this design. The insert weighs about 13.5 tonnes, the copper shell 7.5 tonnes and the fuel assemblies 3.6 tonnes.

Aside from the insert, a canister for PWR assemblies is identical to a canister for BWR assemblies. This means that the copper shell has the same height and diameter. The equivalent total weight of a canister filled with 4 PWR assemblies is 27.6 tonnes. The insert weighs 16.5 tonnes in this case, the copper shell 7.5 tonnes and the fuel assemblies 3.6 tonnes.

3.3.3 Quality inspection

Fabrication and sealing of the canister will be inspected and verified by means of nondestructive testing. The detailed design of the testing programme has not yet been finalized. It is assumed that the stock material will be checked by ultrasonic testing and the welds by a combination of radiography and ultrasonic testing. The welds will also be inspected by a suitable method to ensure that the copper shell does not have any minor surface-breaking defects. Such defects would not be of any importance for the mechanical integrity of the canister, but could serve as starting points for crevice corrosion. Similarly, it is assumed that the sealing weld on the lid will be inspected by radiography, ultrasonic examination and a suitable method for detecting surface-breaking cracks.

3.4 Buffer and backfill

3.4.1 General

The deposited canisters will be surrounded by a buffer that will protect the canister from flowing groundwater, hold the canister in place and greatly retard the transport of radionuclides. After deposition, tunnels, rock caverns and shafts will be backfilled in such a way that increased groundwater flows are prevented and chemical changes of the repository's barriers are inhibited and delayed. These somewhat similar functional requirements on buffer and backfill are of importance in material selection.

3.4.2 Requirements when choosing buffer and backfill materials

Buffer

The buffer is meant to create such conditions around the canister that it brings about a long-term isolation of the spent fuel. For this reason a buffer material is chosen in which diffusion is the dominant transport mechanism for solutes in water to and from the canister. The buffer material must:

– completely envelop and protect the canister for a long time and keep it centred in the deposition hole

– prevent groundwater flow through the deposition hole around the canister and thereby prevent corrosive substances from being transported to the canister except by diffusion.

This in turn requires that the buffer remain in the deposition hole and that it be chemically stable for a long time.

It is also important that the buffer does not have any properties that could cause damage to other barriers. This requires that the buffer:

- possess sufficient thermal conductivity so that the temperature on the canister surface does not exceed 100°C
- possess sufficiently high density to provide the requisite bearing capacity
- possess sufficiently low density to avoid excessively high loads on the surrounding rock and canister due to swelling
- be sufficiently soft to permit rock displacements without the canister being subjected to excessively high stresses
- allow gas that may form in a damaged canister to migrate out.

In addition, there is an economic requirement that the material to be used as a buffer should be readily available.

Different types of swelling clays with a high smectite content have good prospects for meeting both the technical and the economic requirements. By choosing such materials, several other favourable properties are also obtained, such as:

- limitation of transport of radionuclides by sorption on the surfaces of the clay mineral,
- ability to filter and limit the growth of microorganisms,
- ability to filter colloids,
- chemical buffering capacity by impurities in the material.

The sorption process is of great safety-related importance which can be influenced by altering the buffer thickness. Certain long-lived radionuclides such as col-14, chlorine-36 and iodine-129 build up negative ions and do not sorb the mineral. The buffering properties of the clay mineral are influenced by its content of calcite, which buffers the pH, and pyrite, which "buffers" the redox conditions.

Important measurable properties of the buffer that can be partly influenced, include:

- hydraulic conductivity
- swelling pressure
- swelling capacity
- shear strength
- pore volume
- mineral composition
- diffusion and sorption properties
- thermal conductivity.

Backfill

Different mixtures of bentonite and crushed rock are intended to be used for backfilling tunnels and rock caverns in the areas for deposition of spent nuclear fuel and in other excavated rock chambers underground.

The backfill materials used should contribute towards keeping the tunnels stable and holding the buffer around the canisters in place. The backfill should also prevent or limit the water flow around the canister positions. Furthermore, the material in the backfill should be chosen so that the quality of the groundwater is not degraded, and should be chemically stable over a long period of time.

Compressibility is an important property for the backfill material. It determines how the buffer can be kept from swelling out of the deposition holes.

3.4.3 Material selection

Buffer

A suitable buffer material is a smectite-containing clay. However, the swelling function of this type of clay could be jeopardized in the long run by conversion of smectite to illite. A study of different buffer materials /Pusch, 1995/, based on a model for degradation of the clay /Karnland et al., 1995; Hökmark 1995/, has shown that the original smectite content should be at least 50 percent to guarantee the function of the buffer. The study shows that only the smectite types montmorillonite and saponite with sodium as the primary adsorbed ion should be considered as buffer materials. Such clays, bentonites, are available in large quantities in many countries. Even higher smectite contents than 50 percent are very valuable for effective self-healing and homogenization of the clay. Regardless of smectite content, the content of sulphur minerals and organic substances in the buffer material should be low.

Bentonites with a montmorillonite content of 70–90 percent and with sodium as the dominant adsorbed cation are commercially available. MX-80 is used as a reference material for the buffer in the deposition holes in SR 97 /Bäckblom, 1996/. MX-80 is also used in the KBS-3 project and is of a commercially-available bentonite quality.

Backfill

The backfill's requirement is to prevent the tunnel from being a dominating watershed and that the swelling pressure stabilises the tunnel wall.

The preliminary choice of backfill material is a mixture of 10-30 percent bentonite and the remainder crushed rock that is deposited and compacted in place. After water saturation, it is estimated that the backfill will have a hydraulic conductivity of not more than 10^{-10} m/s /Bäckblom, 1996/, which is on a par with the conductivity of very impervious rock. For environmental and economical reasons, reuse of the rock that is excavated from the deep repository is recommended. The hydraulic conductivity, compressibility and expandability of this material have been found to be comparable to the equivalent properties of quartz sand as aggregate /Pusch, 1995b/.

Alternative backfill materials are crushed rock and glacial till without bentonite. However, even after compacting the hydraulic conductivity of these materials will be higher than that of hydraulically impervious rock (Moreno, 1995).

The salinity of the groundwater affects the conductivity and swelling capacity of the bentonite. This means that a higher proportion of bentonite may be necessary at a repository site with more saline groundwater in order to achieve the right conductivity.

Use of a mixture of 15 percent by weight bentonite and 85 percent by weight crushed rock for backfill of tunnels in the repository for spent nuclear fuel is assumed in SR 97 on all repository sites. In a real repository, the composition of the backfill will be adapted to the properties of the site. A mixture containing 25–30 percent bentonite is required at Aberg because of the high salinity of the groundwater.

3.4.4 Technology for application of buffer and backfill

Buffer

Pre-compacted large bentonite blocks, pressed to a high degree of water saturation into whole rings or sills with a full diameter, are first placed in the deposition holes. The canister is then deposited, after which the gaps between the rock and the blocks are filled with water with a low electrolyte content so that a high pore water pressure in the bentonite is reached quickly. Alternatively, the gaps can first be filled with bentonite pellets so that the total quantity of bentonite will be as great as possible.

Backfill

The backfill material is deposited in inclined layers and compacted with a vibrating plate mounted on a movable arm, or similar equipment. Bentonite blocks may be used for backfilling of the upper part of the tunnel against the roof. Compaction of such blocks with a weight of about 15 kg has been done with good results /Johannesson et al., 1995/.

3.5 Plugging and final sealing

3.5.1 Temporary plugs

When fuel assembly canisters have been emplaced in all deposition positions in a deposition tunnel and the tunnel has been backfilled, the tunnel is sealed with a plug whose function is to:

– prevent swelling-out of the backfill material

– permit rapid build-up of a high water pressure in the deposition positions.

The latter requirements can be met if the tunnel has been sealed off effectively against water flow, both in the tunnel itself and in the surrounding disturbed zone. The function of the plug is important up until the time the transport tunnel is backfilled. The plug can then either be taken away or left in place.

3.5.2 Permanent plugs

The change in the original mechanical and hydraulic properties of the rock brought about by blasting-out of tunnels and rock caverns may in some cases require corrective measures in the form of sealing-off and backfilling in strategic places. Such measures aim at limiting hydraulic transport pathways by:

– sealing tunnels

– limiting flow paths in the disturbed zone around the tunnel

- separating repository areas hydraulically from fracture zones and other conductive structures that are in close or direct contact with the biosphere
- limiting flow paths along shafts and ramps that are connected to the ground surface.

Moreover, these measures help to deter human intrusion.

SR 97 does not include the analysis of the effects of permanent plugs.

In backfilling of shafts, allowance must be made for the fact that the backfill material will eventually settle, which can lead to cavities in the backfill behind rigid plugs. The backfill material in shafts must therefore consist of a swelling material that balances this settlement. An example of such a material is a mixture of bentonite and crushed rock.

The uppermost part of ramps and shafts near ground level is sealed to deter human intrusion. For example, a suitably long section is fitted with a concrete plug.

3.5.3 Plugging of boreholes

Boreholes connected to the ground surface are sealed before the repository site is abandoned. One method for sealing of boreholes that has been tried in Stripa is to insert perforated copper pipes filled with compacted bentonite into the boreholes. In the parts of the boreholes that intersect discontinuities or poor rock, pretreatment of the holes may be necessary, whereby the holes are filled with cement and drilled up again before the concrete fill is fed down.

Another method for plugging of boreholes has been studied by Nagra, Switzerland, where the borehole is filled with bentonite pellets through a tube by means of compressed air.

3.6 Retrieval of deposited canisters

The design of the deep repository assumed in SR 97 provides for the possibility to retrieve the deposited canisters at all stages. This possibility is not detrimental to the long-term performance of the repository.

4 Properties of the repository sites

4.1 General

This chapter describes the properties of the three repository sites. Data have been chosen from actual sites where SKB has conducted investigations namely Äspö (Aberg), Finnsjön (Beberg) and Gideå (Ceberg). All sites are relatively near the coast, and Äspö is an island in the coastal archipelago, as shown in Figure 4-1.

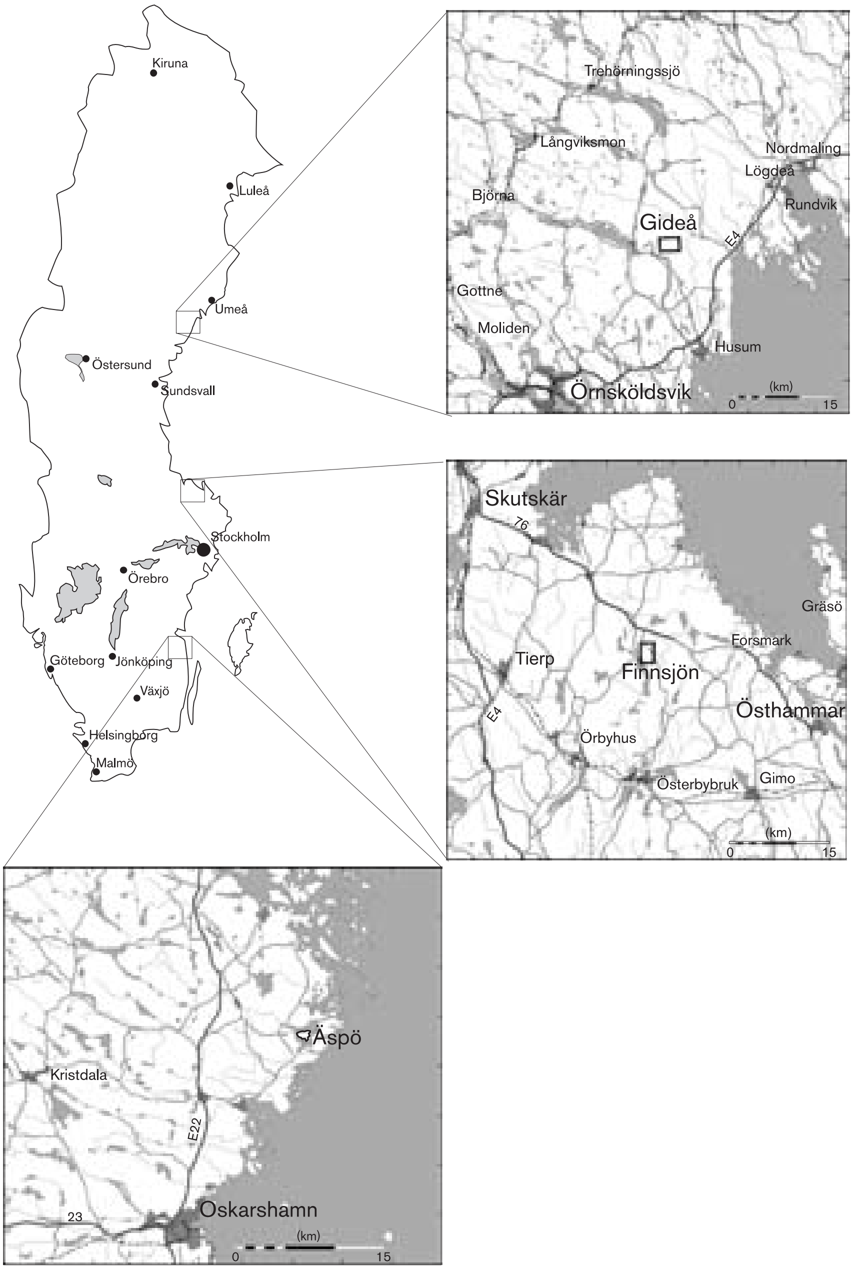


Figure 4-1. Data on the fictitious repository sites Aberg, Beberg and Ceberg are taken from Äspö, Finnsjön and Gideå.

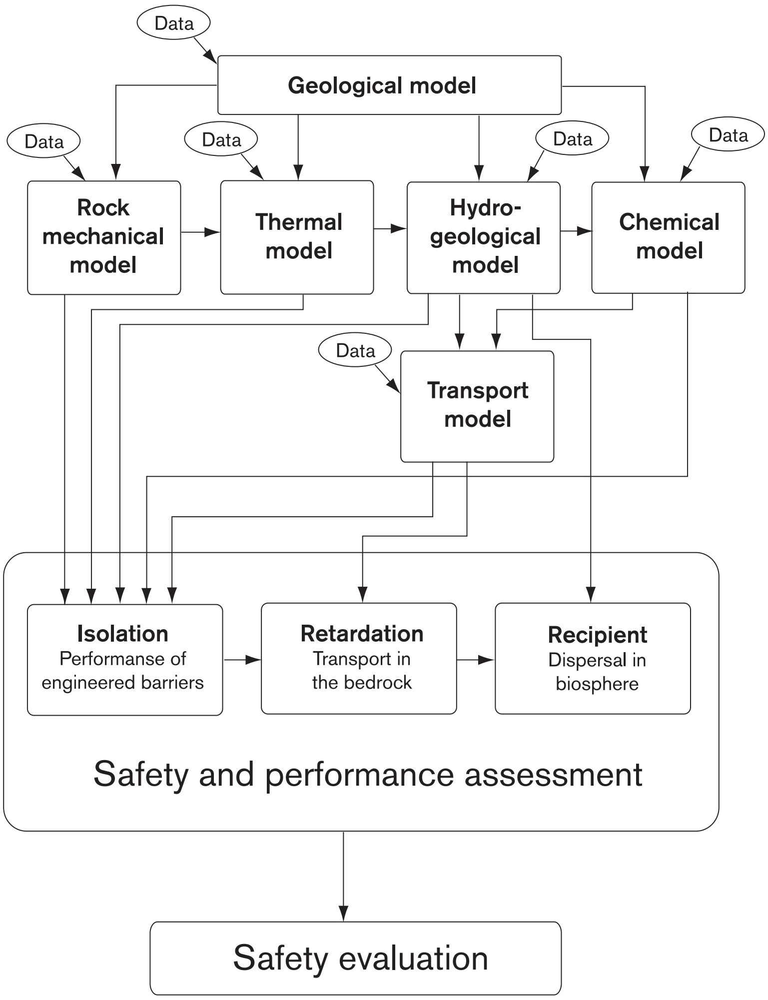


***Figure 4-2**. Schematic illustration of how information is transferred between different geoscientific models and how these models are used in evaluating the safety and suitability of a repository site.*

The geological model serves as the basis for a geoscientific description and forms a basis that the other models can develop (see Figure 4-2). It describes the evolution of the soil cover and rock with regard to the type of rock, fracture zones and other structures. The geological model also provides a geometric basis, from which the repository design can be decided upon. As a rule, the geological description is not used directly in a safety assessment, but is used primarily as a basis during rock-mechanical, hydrological, thermal and geochemical modelling of the site /Andersson et al, 1996/.

The geological descriptions of the three sites Aberg, Beberg and Ceberg are given below. Since research has been carried out by different geologists at different times, the terminology differs between the three sites, particularly as regards the different type of structures in the bedrock. It should be borne in mind that on occasion, this report does not differentiate between interpreted and verified fracture zones in the geological model. To a certain extent, this also holds true for the water-conducting properties that they have acquired. For information on what constitutes the basis for the assessment of each fracture zone's existence and properties, please refer to the reports of the different investigation areas.

On the whole, uncertainties in the collection of data and interpretation of results are a natural part of all geoscientific investigations. The uncertainties can be attributed in part to limitations in the collection of data from the study site, and in part to the errors and uncertainties that exist in the collected database and the interpretation of collected data, as well as in the conceptualization of the rock into a suitable model. Due to the fact that the investigation material varies from site to site and that the purpose of the site investigations performed may vary, additional uncertainties arise when geographically separate sites are compared.

The uncertainties are summarized below and refer for the most part to other reports.

4.2 Geoscientific investigation material

The investigations on the three sites Äspö, Finnsjön and Gideå – which serve as a basis for the fictitious sites Aberg, Beberg and Ceberg – have been carried out in different stages since the beginning of the 1970s. The scope of the investigations varies, in part due to the fact that they were performed for different purposes. The investigation material, which is most extensive for Äspö (Aberg) and least for Gideå (Ceberg), is summarized in Tables 4-1 and 4-2.

On Äspö, SKB built a Hard Rock Laboratory (HRL) during the period 1986–1995 at a depth of 500 m for the purpose of conducting studies, research and demonstration trials at repository depth in a previously undisturbed area. A large quantity of data were gathered before and during construction of the underground facility. The Äspö HRL is used today for research purposes, and data on the rock mass are gathered continuously

Table 4-1. Scope of surveys on a regional scale conducted in Äspö, Finnsjön and Gideå /after Saksa and Nummela, 1998/.

Investigation method	Äspö	Finnsjön	Gideå
Topography	5 m contour map	12.5 m contour map	5 m contour map
Lineament analysis	Satellite image analysis and aerial photograph analysis, including relief maps (resolution 5–50 m)	Satellite image analysis and aerial photograph analysis, including relief maps	Aerial photograph analysis and topographical maps
Lithology-tectonics	1:50,000 base map with rock types and major fracture zones, detailed survey above ground 1:10,000, underground 1:500	1:50,000 base map with rock types and major fracture zones	1:50,000 base map with rock types and major fracture zones
Fracture maps	20×30 km fracture maps and maps of Simpevarp Peninsula (about 2×2 km)	Not available	5 local area maps and investigations in Gissjö tunnel
Test drillings outside of detailed characterization area	2 cored boreholes and 7 percussion boreholes	Boreholes in SFR in Forsmark and in regional lineaments (Singö fault)	Not available
Aerogeophysicss	Aeromagnetic map	Not available	Not available
Ground geophysics	Magnetic and gravimetric mapping, VLF survey, seismic refraction survey	Not available	Magnetic mapping, slingram and VLF surveys

Table 4-2. Scope of surveys on local scale conducted in Äspö, Finnsjön and Gideå /after Saksa and Nummela, 1998/.

Investigation method	Äspö	Finnsjön	Gideå
Topography	1-2 m contour map	2 m contour map	2 m contour map
Lineament analysis	1:10,000 and 1:4,000 map from satellite, DTM and contour maps	1:20,000 aerial photo and as IR photo	Available of the repository area, co-interpreted with structural geology
Bedrock maps	Available of the areas Äspö, Ävrö, Bussvik, L. Laxemar and Glostad on a scale of 1:10,000. Detailed characterization on a scale of 1:10,000.	Available of an area of about 6–30 km^2, repository area is shown in detail	Available of an approx. 6 km^2 area
Fracture maps	Three profiles and in outcrop maps of Äspö	Two profiles and in outcrop maps	Two profiles through the repository area
Cored holes (number, max. depth)	18 holes, 993 m	11 holes, 691 m	13 holes, 701 m
Percussion boreholes (number, max. depth)	37 holes, 175m	20 holes, 459 m	24 holes, 153 m
Ground-geophysical measurements	5 km^2	1.6–2.4 km^2	5 km^2
Ground-geophysical surveys	Slingram, VLF, ground radar, resistivity and magnetic surveys plus seismic surveys	Slingram, VLF, resistivity and magnetic surveys plus seismic surveys	Slingram, VLF, resistivity and magnetic surveys plus seismic surveys
Borehole-geophysical surveys	Radar, flow logging, natural gamma radiation and seismic surveys	Radar, natural gamma radiation, flow logging and resistance measurement in fracture zones	Radar, natural gamma radiation, flow logging, resistance measurements in fracture zones and seismic tomography

in various research projects in the laboratory. However, only data from the pre-investigations and from the construction phase are used in SR 97.

Finnsjön was investigated during the period 1977–1978 within the framework of the KBS project for the purpose of finding geologically suitable sites for the final disposal of HLW /KBS 1977; KBS 1978/. During 1985–1991, studies were conducted of a low-dipping fracture zone to obtain detailed knowledge of the importance of flat zones for transport of groundwater and solutes (substances dissolved in the groundwater). Furthermore, the area has been utilized for several research projects, tests of new downhole instruments and a study of fallout from Chernobyl /Ittner, 1989/.

Equivalent KBS studies were performed in Gideå during the period 1981–1983. These investigations have been supplemented by several new investigations during the 1990s plus a thorough review of the original data, where new geological-structural and geohydrological descriptions have been formulated /Hermanson et al, 1997/.

4.3 Geological description

4.3.1 Aberg

Regional scale

Topography

The regional study site covers about 170 km^2, see Figure 4-3.

The regional area around Äspö is low-lying coastal terrain with an elevation difference of around 30 m from sea level to the area's western side. The area is forested with thin or no soil cover on highlands and bogs or fens in lowlands.

Rock types

The bedrock in the region is dominated by around 1,800–1,770 million year-old granites commonly called Småland granite. Of minor importance are larger basic rock intrusions, such as gabbro and diorite, as well as smaller xenoliths of older basic rock types. Younger granites, aged about 1,400 million years, have penetrated into the older granite belt and occur as larger rock bodies both north and south of Äspö. Dykes and rock bodies of fine-grained granite are also common in the region.

Regional fracture zones

The region is dominated by north-westerly, north-easterly and north-southerly steeply-dipping regional fracture zones (called structures) with a horizontal extent of more than 10 km (Figure 4-3) /Rhén et al, 1997b/. These fracture zones are often interpreted as having a width of hundreds of metres, with a central fractured portion which can be up to ten or so metres wide. The north-southerly regional fracture zones are presumably one of the latest formations and are interpreted to be the most water-bearing according to results of geophysical measurements.

It can be noted that SGU is carrying out regional interpretation of deformation zones in ongoing feasibility studies for the municipality of Oskarshamn. It has not been possible to include the results of this interpretation in the present report.

Local scale

Topography

The local study site covers an area of about 1 km^2 (Figure 4-4).

Rock types

On the local scale, four types of rock are dominant: Ävrö granite (Småland granite), Äspö diorite, greenstone and fine-grained granite. A bedrock map of Äspö is shown in Figure 4-4. A vertical cross-section showing the different rock types in parts of south-east Äspö is shown in Figure 4-5.

Both Äspö diorite and Ävrö granite can be seen as variants of Småland granite /Kornfält and Wikman, 1988; Wikman and Kornfält, 1995/. The Äspö diorite has been dated at about 1,800 billion years old and dominates the rock type distribution compared with the somewhat younger Ävrö granite. The Äspö diorite has a somewhat more basic characteristic (varying between granodiorite, quartz monzonite, and quartz diorite) than

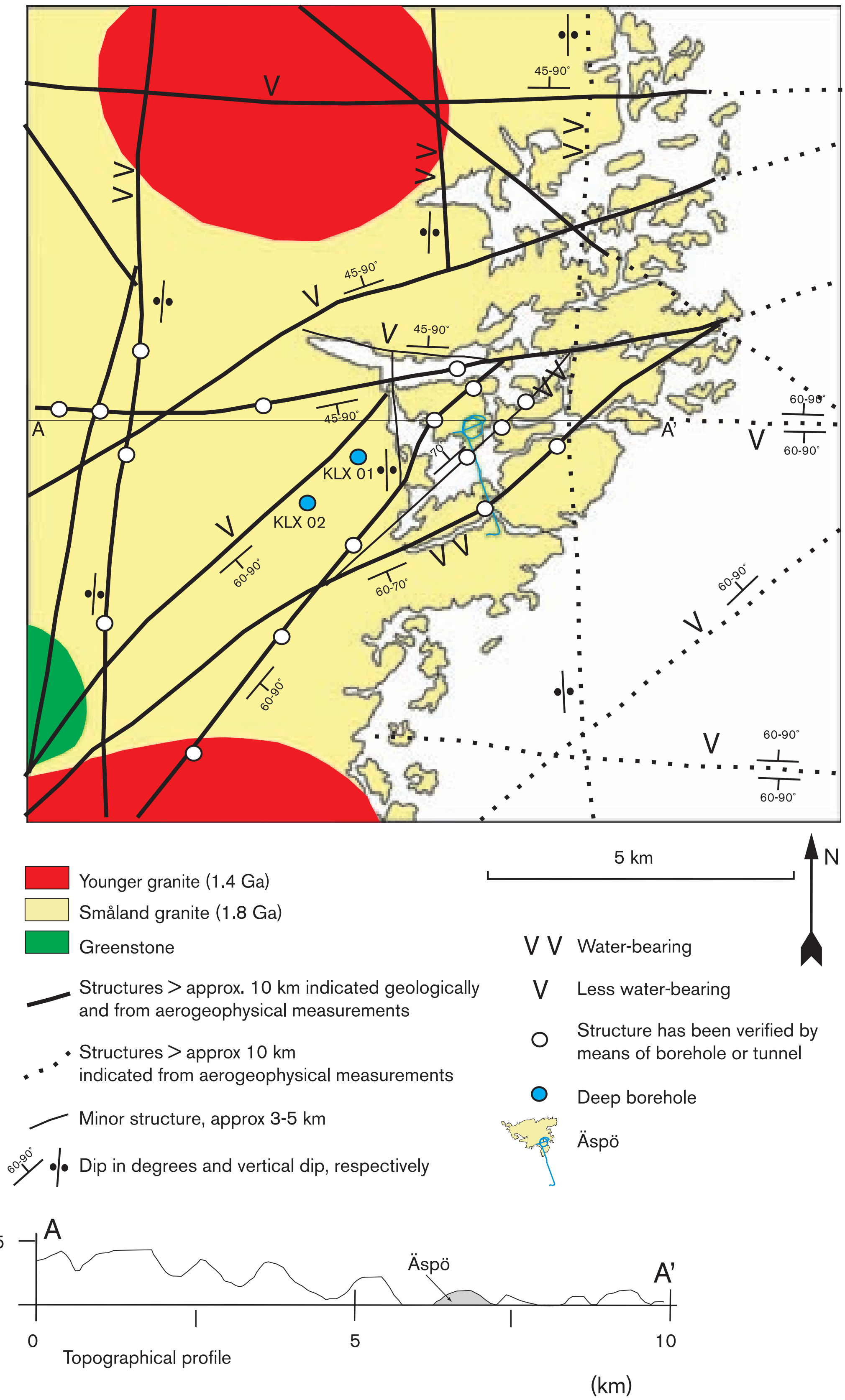


***Figure 4-3**. Regional geological description of Äspö /after Rhén et al, 1997b/. Topographical profile in east-westerly direction /after Walker et al, 1997/.*

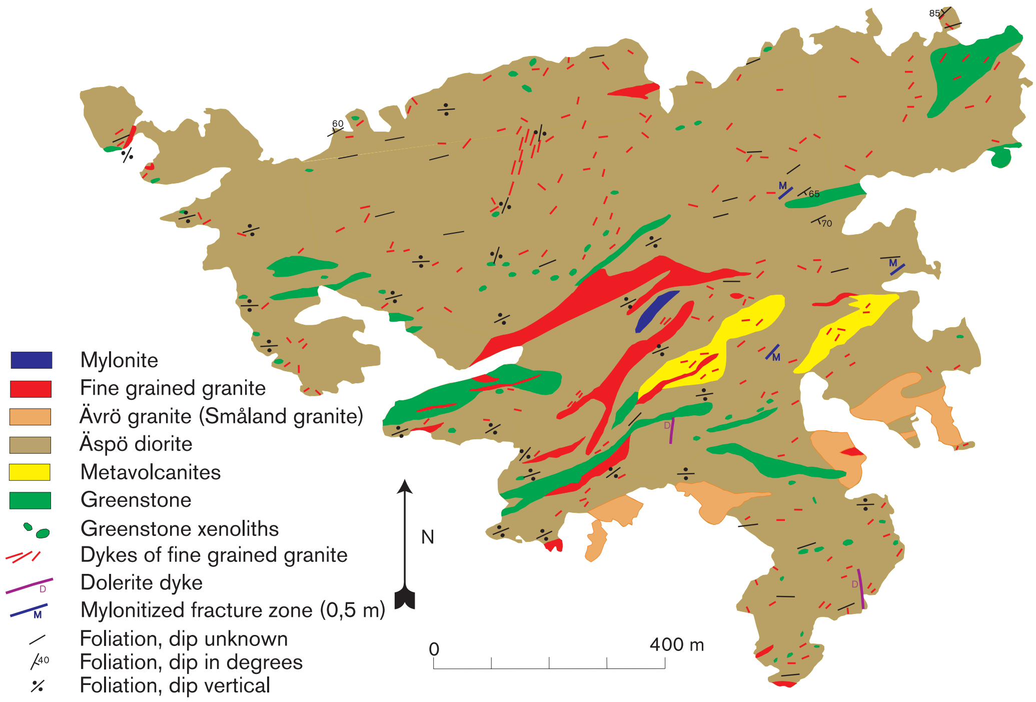


Figure 4-4. *Bedrock map of Äspö /after Kornfält and Wikman, 1988; Wikman and Kornfält, 1995/.*

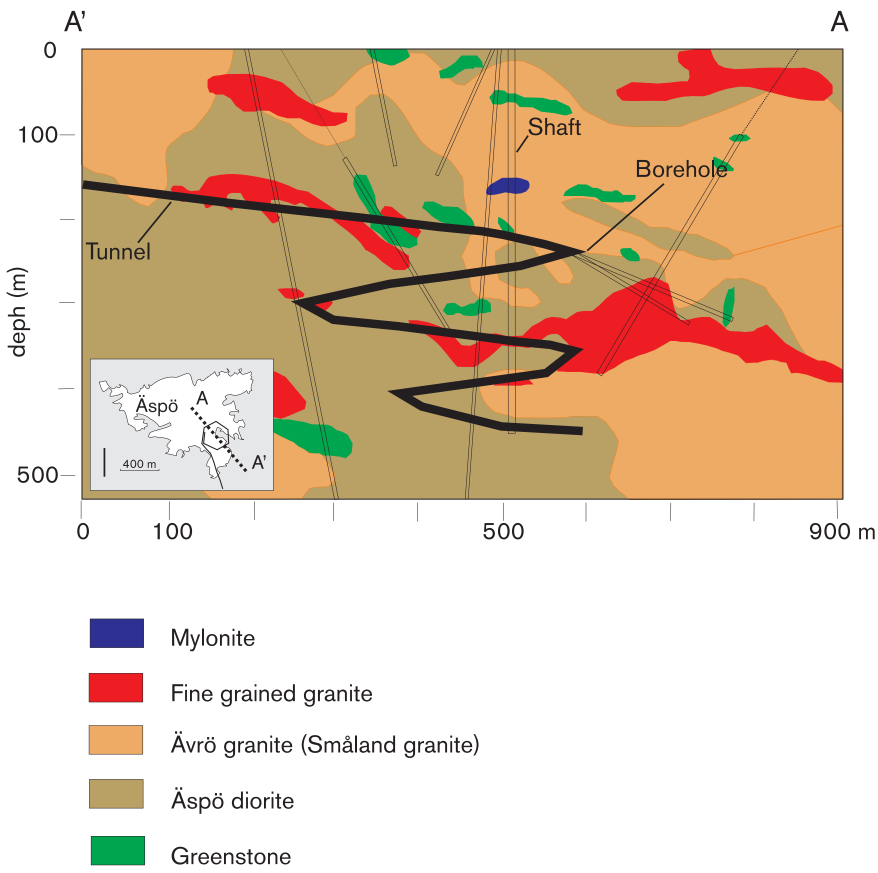


Figure 4-5. *Vertical cross-section of southeastern Äspö. Based on borehole and tunnel data /after Rhén et al, 1997b/.*

the Ävro granite and is normally of a grey to greyish red colour. The Ävrö granite differs from the Äspö diorite by virtue of its relatively light, redder colour.

Greenstones occur as elongated xenoliths in the Ävrö granite and Äspö diorite usually following the foliation. The larger occurrences of greenstone are often penetrated by fine-grained granite.

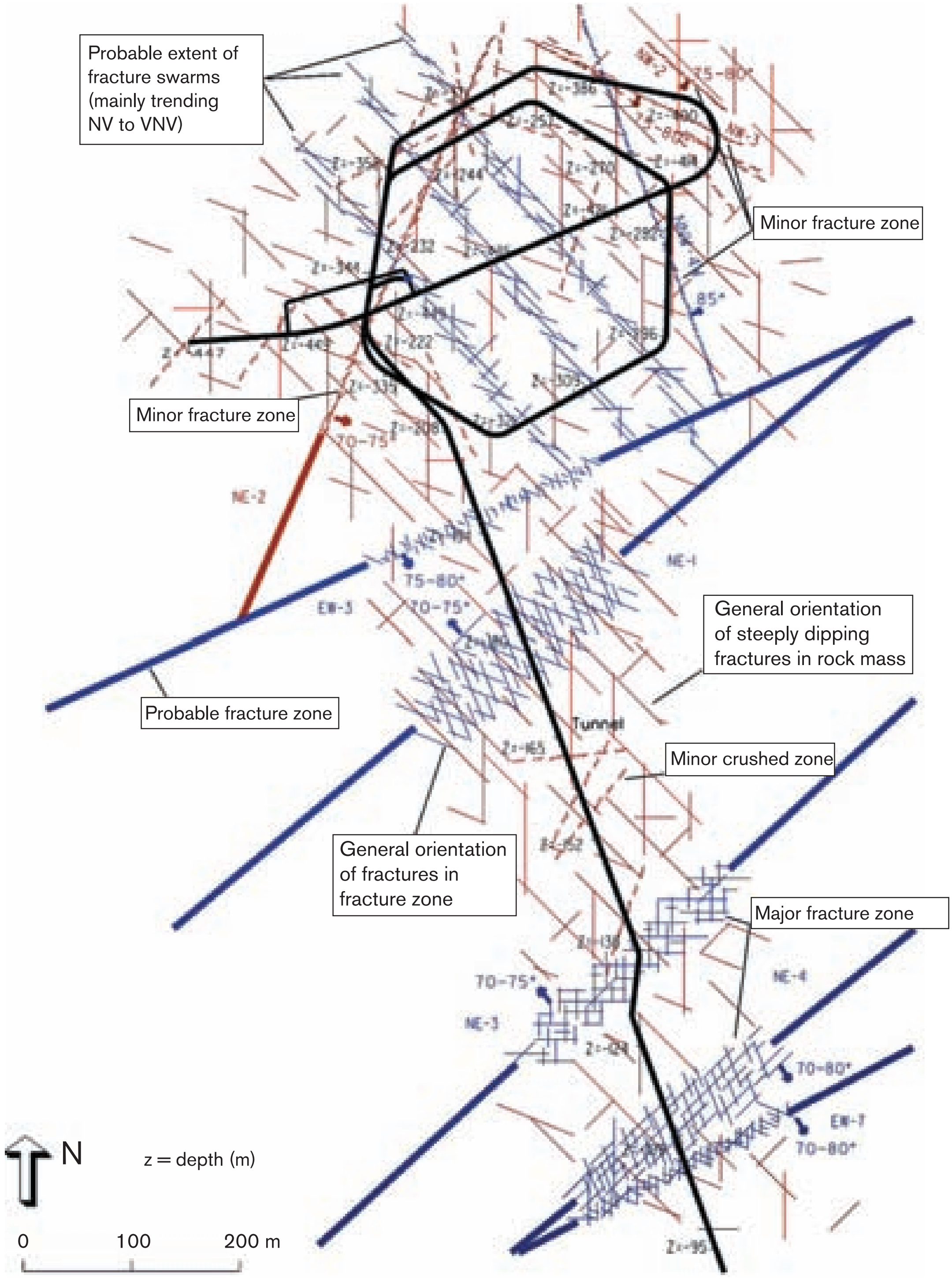


Figure 4-6. Geological-structural model of study site /after Rhén et al, 1997a/.

Dykes or irregular bodies of fine-grained granite are quite common, particularly in the Äspö diorite. The dykes usually have a northeasterly direction. Fractures are far more frequent in the fine-grained granite than in other rock types. Contact between the fine-grained granite and the Äspö diorite is usually diffuse and well smelted.

Äspö has a steeply-dipping foliation in a northeasterly to east-northeasterly direction.

Fracture zones and fractures

Fracture zone maps based on lineament studies and geophysical measurements show a concentration of fracture zones in and around Äspö.

Fracture zones on a local scale have been divided into major and minor zones by Stanfors and Stille /1997/. Major fracture zones are those wider than 5 m, whereas minor zones a narrower than 5 m. A total of 16 fracture zones have been defined within the study site (Figure 4-6). A list of these is given in Table 4-3. The fracture zones are dominated by major zones in an east-westerly and north-easterly direction, and minor zones in a north-northwesterly to north-easterly direction, see Figure 4-6.

The largest single structure zone is a regional north-easterly plastic deformation zone that cuts straight across the island. It is called zone EW-1 in the geological model. The zone is over 100 m wide and is clearly indicated by geophysical measurements and in the borehole. It is characterized by heavy foliation and metre-wide mylonites. The Äspö HRL together with the access tunnel is located south of the zone.

Table 4-3. Major fracture zones in Äspö /after Rhén et al, 1997b/. Fracture zone width in parentheses indicates estimated, unobserved average width. T = transmissivity, S = storativity.

Fracture zone	Dip	Width (M)	T mean (m^2/s)	S (log T) (–)
EW-1N	8 SE	(30)	$5.2 \cdot 10^{-7}$	1.6
EW-1S	78 SE	(30)	$1.2 \cdot 10^{-5}$	1.2
EW-3	79 S	15	$1.7 \cdot 10^{-5}$	0.5
EW-7	81 SE	(10–15)	$1.5 \cdot 10^{-5}$	1.3
NE-1	70-75 NW	30	$2.2 \cdot 10^{-4}$	0.5
NE-2	77 SE	5	$1.2 \cdot 10^{-7}$	2.1
NE-3	70-80 NW	50	$3.2 \cdot 10^{-4}$	0.5
NE-4	71-78 SE	40	$3.1 \cdot 10^{-5}$	0.8
NW-1	30 NE	(10)	$4.1 \cdot 10^{-7}$	1.1
NNW-1	(vertical)	(20)	$8.6 \cdot 10^{-6}$	0.8
NNW-2	(vertical)	(20)	$2.4 \cdot 10^{-5}$	1.1
NNW-3	(vertical)	(20)	$2.0 \cdot 10^{-5}$	–
NNW-4	85 NE	10	$6.5 \cdot 10^{-5}$	1.5
NNW-5	(vertical)	(20)	$4.0 \cdot 10^{-6}$	0.8
NNW-6	(vertical)	(20)	$1.4 \cdot 10^{-5}$	–
NNW-7	85 NE	(20)	$7.5 \cdot 10^{-6}$	0.9
NNW-8	(vertical)	(20)	$8.4 \cdot 10^{-6}$	0.1

The fracture zones NE-1, NE-3, EW-3 and NE-4 are all clearly identified by surface-geophysical measurements, in boreholes and tunnels. The zones are characterized by heavily fractured, 10–40 m wide central sections, more or less water-bearing in association with the occurrence of fine-grained granite, greenstone and diorite. NE-2 is a steeply-dipping minor zone that follows a previous mylonite/plastic deformation zone and varies in width between 1 and 5 m.

The minor north-westerly to north-easterly fracture zones are steeply-dipping, with a width that is normally less than 5 m. The also are less spread out than the above-mentioned zones. They are locally highly water-bearing and usually consist of one or more zone sets.

Besides superficial stress-relief fractures, only two minor low-dipping fracture zones have been documented from the underground investigations (Rhén et al, 1997a). They both have a width of about 0.5 m and are visible a hundred or so metres alongside the tunnel.

The number of measured fractures amounts to over 25,000. Of these, about 10,000 have been mapped on outcrops and in trenches. The rest have been mapped in conjunction with the HRL. The fracture system is divided into four fracture sets with steeply-dipping fractures in north-south, north-northwesterly and west-northwesterly directions, plus a low-dipping fracture system. Water-bearing fractures tend to be steep and strike west-northwest. Mineral fillings in these fractures consist mainly of epidote, quartz and iron oxides.. The length of fractures is measured in outcrops and tunnel walls and is independent of rock type.

4.3.2 Beberg

Regional scale

Topography

Finnsjön is situated just north of Österbybruk in a relatively low-lying terrain with flat outcrops, mires and small lakes (Figure 4-7). The elevation above sea level varies between 0 and 60 m. The mean elevation in the region is about 30 m.

Rock types

The oldest rock types in the region consist of alternating sedimentary and volcanic supracrustal rocks aged around 1,900-1,880 million years. It is usual for mineralization, primarily of iron minerals, to occur in these volcanic rock types. Mining was previously common (e.g. Dannemora Mine). However, the mine has now ceased to operate. The sedimentary and volcanic rock types were formed at the same time as gabbro, diorite and granitoids (tonalite, granodiorite and granite). All these rock types have also been penetrated by basic dykes. Most of the plastic deformation, i.e. the formation of stronger or weaker foliation, lineation and folding of the bedrock, occurred 1,850–1,780 million years ago. During this deformation, the rock types underwent change and some of them started to smelt. These smeltings hardened to form younger granite, aplite and pegmatite.

Fracture zones and fractures

This review is primarily based on a study by Ahlbom and Tirén, 1991. As stated earlier, SGU has recently presented an interpretation of the plastic and brittle deformation zones in the regions (Overview of Uppsala county). These interpretations have not been included in SR 97.

Regional fracture zones form a block-like network (Figure 4-7). The ground surface in the blocks that are bounded by north-northeasterly fracture zones inclines towards the east-southeast, while the ground surface inclines towards the northeast and in the blocks that are bounded by the northwesterly fracture zones. This block inclination is interpreted as being a result of a system of regional faults with a steep dip near the ground surface /Ahlbom and Tirén, 1991/.

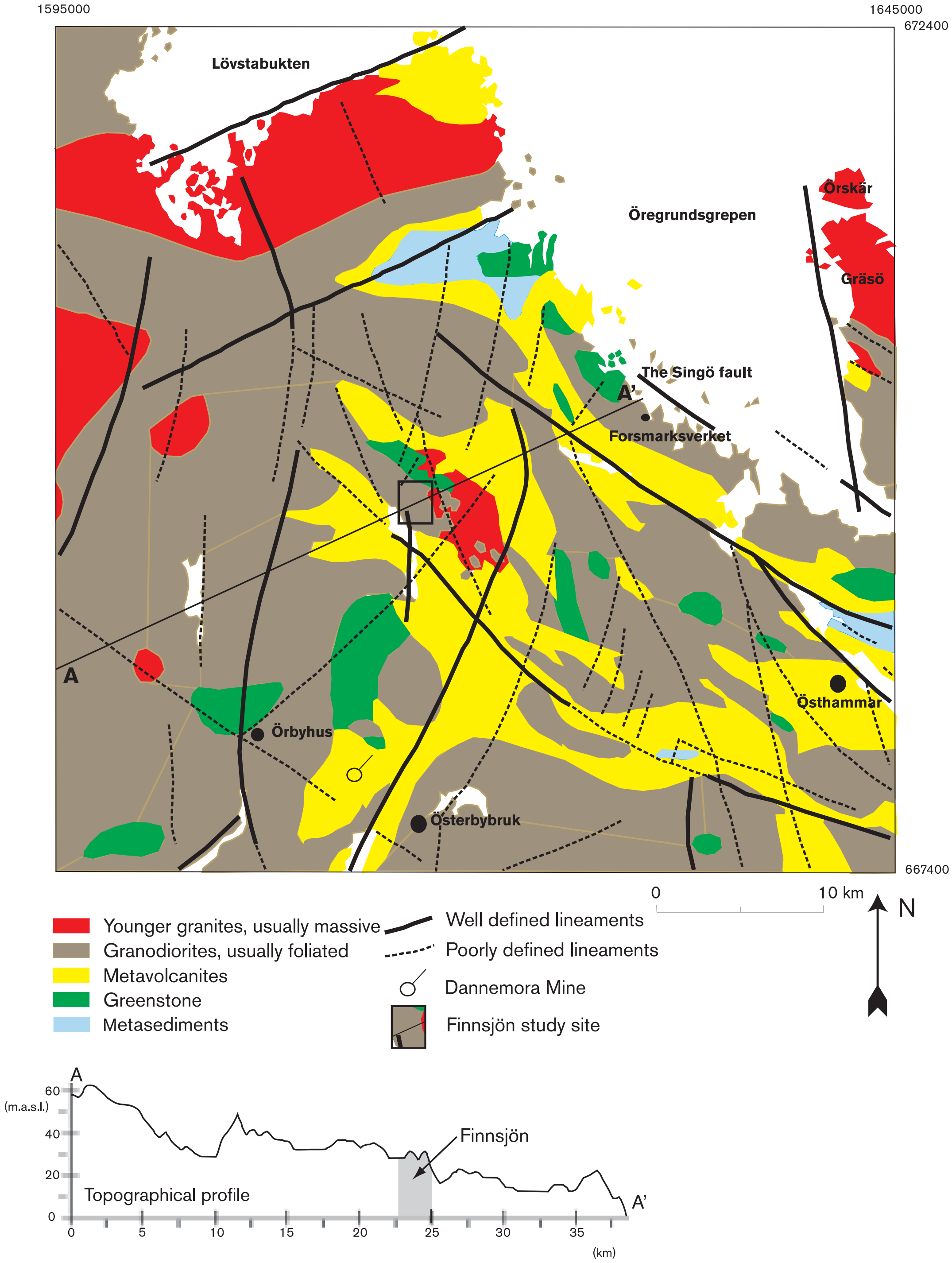


Figure 4-7. Regional geological description of Finnsjön /after Ahlbom and Tirén, 1991/. Topographical profile in northeasterly direction /after Walker et al, 1997/.

Only one of the regional fracture zones, the Singö fault, has been studied in detail in conjunction with the investigations for cooling water tunnels to the Forsmark nuclear power plant and prior to the construction of a repository for radioactive operational waste (SFR) in Forsmark. The Singö fault is a steeply-dipping, roughly 100–200 m wide zone with a complex inner structure consisting primarily of mylonites and an approximately 15 m wide, highly fractured core.

Local scale

Topography

The study site is defined as a 6 km^2 rectangular region immediately north of Finnsjön. The area is intersected by regional west-northwesterly lineaments and has an even topography with an elevation difference of less than 15 m. Outcrops are relatively common, but approximately 85 percent of the ground surface is covered by Quaternary deposits of glacial till and peat bogs. Sand is found in some depressions in Gåvastbo /Gustafsson et al, 1987; Ahlbom et al, 1991/.

Rock types

The bedrock is dominated by a grey granodiorite with a depth of at least 700 m (deepest borehole). The granodiorite has a steeply-dipping north-westerly foliation, Figure 4-8. Dykes of basic rock types, pegmatites and aplites, intersect the granodiorite. Older altered acid volcanites occur west and south of the area. Basic rock types occur as elongated xenoliths running along the foliation.

Fracture zones and fractures

A steeply-dipping fracture zone, Zone 1, running form north-east to south-west divides Finnsjön into two halves, the northern block and the southern block. Zone 1 is documented at a length of around 5-6 km and a width of around 20 m. The boreholes across the zone show a increased frequency of fractures and altered reddish granodiorite. Asphalt is a characteristic mineral in the zone.

A sub-horizontal fracture zone, Zone 2, occurs in the northern block. The zone is 100 m wide and has been identified in nine boreholes where the top of the zone has been found at a depth of between 100 and 295 m (see vertical cross-section in Figure 4-9). There are no indications that Zone 2 intersects Zone 1 in the southern block and it has not been identified in outcrops. Zone 2 is believed to have been formed by reactivation of a previous plastic structure. The most intensively fractured sections occur along the upper and lower boundaries of the zone.

In addition to the above-mentioned zones, a further 12 fracture zones have been defined in Finnsjön. They are summarised in Table 4-4 and Figure 4-8. Most of these fracture zones are less well-known, particularly in the southern block. Minor shear zones running in a north-westerly direction also regularly occur in the northern block. These zones are usually around 1-5 m wide and extend over several hundred metres.

The fracture system is dominated by steeply-dipping northeasterly and northwesterly fractures and by low-dipping fractures. The mean fracture frequency in the southern block is about 3 fractures per metre, measured along two perpendicular profiles along the ground surface and in the uppermost 100 m in the cored boreholes /Olkiewicz and Arnefors, 1981; Ahlbom et al, 1992/. No decrease in the fracture frequency has been noted with depth. In the northern block, on the other hand, the measured frequency is only 1.5 fractures per metre based on measurements in outcrops. Boreholes in this block exhibit a similar fracture frequency.

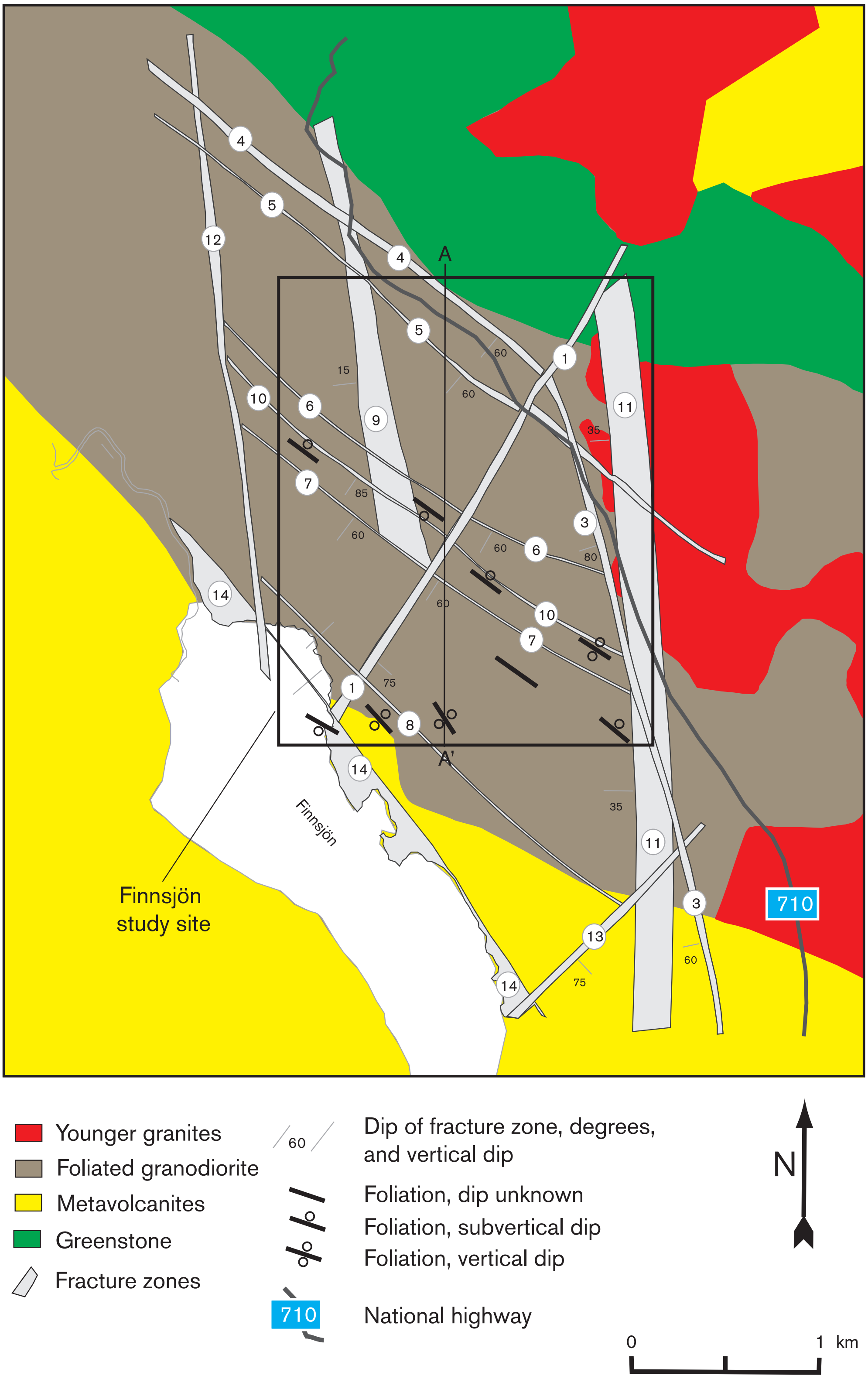


Figure 4-8. *Bedrock map and generalized picture of fracture zones across Finnsjön /after Ahlbom and Tirén, 1991 and Ahlbom et al, 1992/.*

Table 4-4. Fracture zones in Finnsjön /after Ahlbom et al, 1992; Walker et al, 1997/. Conductivity values based on 3 m packer tests, upscaled to 100 m scale in certain cases. K = hydraulic conductivity (m/s).

Fracture zone	Strike	Dip	Length (km)	Width (m)	100 m scale median log K	3 m scale median log K
1	N30E	75SE	5	20	–4.3	–5.7
2	N28W	16SW	1.5	100	–5.2	–6.3
3	N15W	80W	5	50	–5.6	–6.8
4	N50W	65SW	1	10	–5.2	–6.4
5	N50W	60SW	5	5		–6.4
6	N55-65W	60SW	2	5		–8.4
7	N55W	60SW	2	5		–7.4
8	N50W	90	3	5		–7.4
9	N10W	15W	2	50		–7.9
10	NW	85SW	2.5	5		–8.3
11	N5W	35W	2	100		–7.2
12	N-S	90	6	25	–4.9	–6.1
13	N30E	75SE	7	20	–4.3	–5.7
14	NW	90	>50	100	–4.9	–6.1

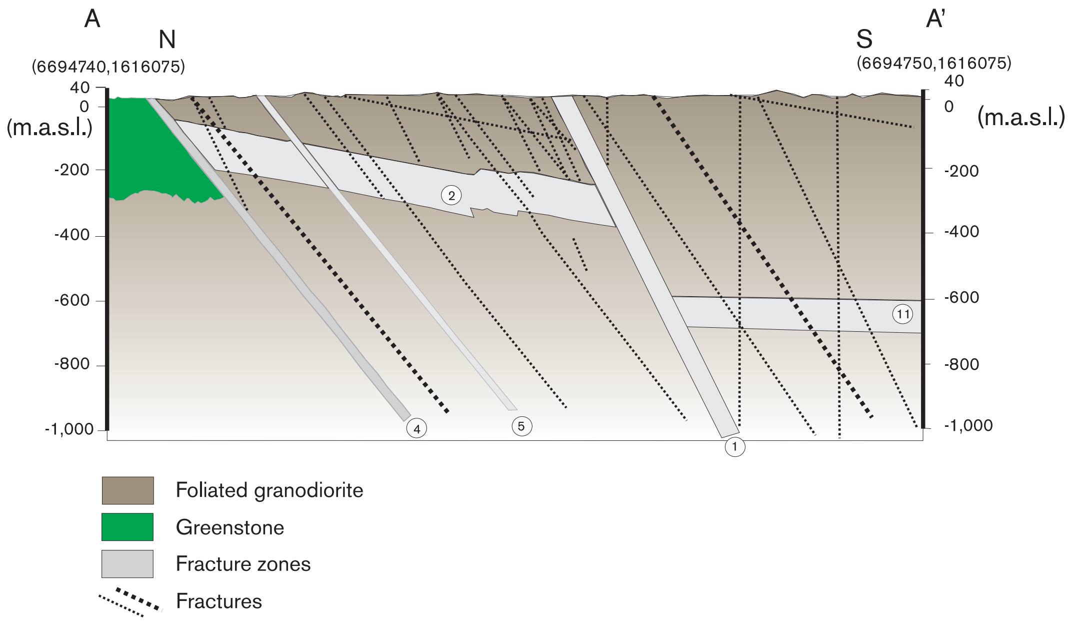


Figure 4-9. North-southerly vertical profile through the study site /after Ahlbom and Tirén, 1991/. For the path of the profile, see the profile A–A' in Figure 4-8.

Northeasterly fractures are most frequent. The fracture walls, which are often reddish due to earlier hydrothermal solutions, are filled with iron-rich, 1.2 billion-year-old prehnite /Wickman et al, 1983/. The northwesterly fracture set is older than the northeasterly one. These fractures have probably been formed by reactivation of early plastic structures and exhibits the longest fractures in the area. The flat fractures tend to incline to the southwest. The drill cores often contain the fracture minerals chlorite, calcite and laumontite.

4.3.3 Ceberg

Regional scale

Topography

In contrast to Äspö and Finnsjön, the area around Gideå has a significant topographical relief with an elevation varying between about 300 m and sea level. The investigation area lies between the Husån and Gideälven rivers and has a relatively even topography locally. It is forested and the soil is overlain by a thin layer of glacial till, with peat bogs and mires in depressions in the terrain.

Rock types

The regional geology is dominated by sedimentary gneiss, a metagreywacke formed by the depositing of sand mixed with a relatively large amount of clay. This rock type was formed around 1,950–1,870 million years ago. Gneiss foliation is generally steeply-dipping towards the northwest. South of Gideå is an older granite of around 1,890–1,850 million years old. West of the area is a younger granite (Revsund granite), 1,800–1,770 million years of age. Dolerite sills and dykes, aged around 1,270-1,215 million years old, intersect the bedrock. The sills appear as low-dipping sills which can be several hundred metres wide /Welin and Lundquist, 1975/. Dolerite dykes are often one or several metres wide. They often occur in groups, with a distance of approximately 200–300 m between the dykes. In the region around the site, the dykes are steeply dipping, in an east-westerly direction.

Fracture zones and fractures

The regional fracture zones (lineaments) are interpreted as being steeply-dipping with a predominantly west-northwesterly to northwesterly direction /Ahlbom et al, 1983; Askling, 1997; Walker et al, 1997/, Figure 4-10. According to Askling /1997/, a regional fracture zone can be found in the northeast corner of Gideå. However, no boreholes intersect the fracture zones, and geophysical surveys in the area have failed to indicate the fracture zone. A small number of other lineaments cross the site and can be correlated to existing fracture zones. Despite the fact that several boreholes intersect these zones, their geological and hydraulic properties should be regarded as uncertain /Ahlbom et al, 1983; Askling, 1997; Hermanson et al, 1997/.

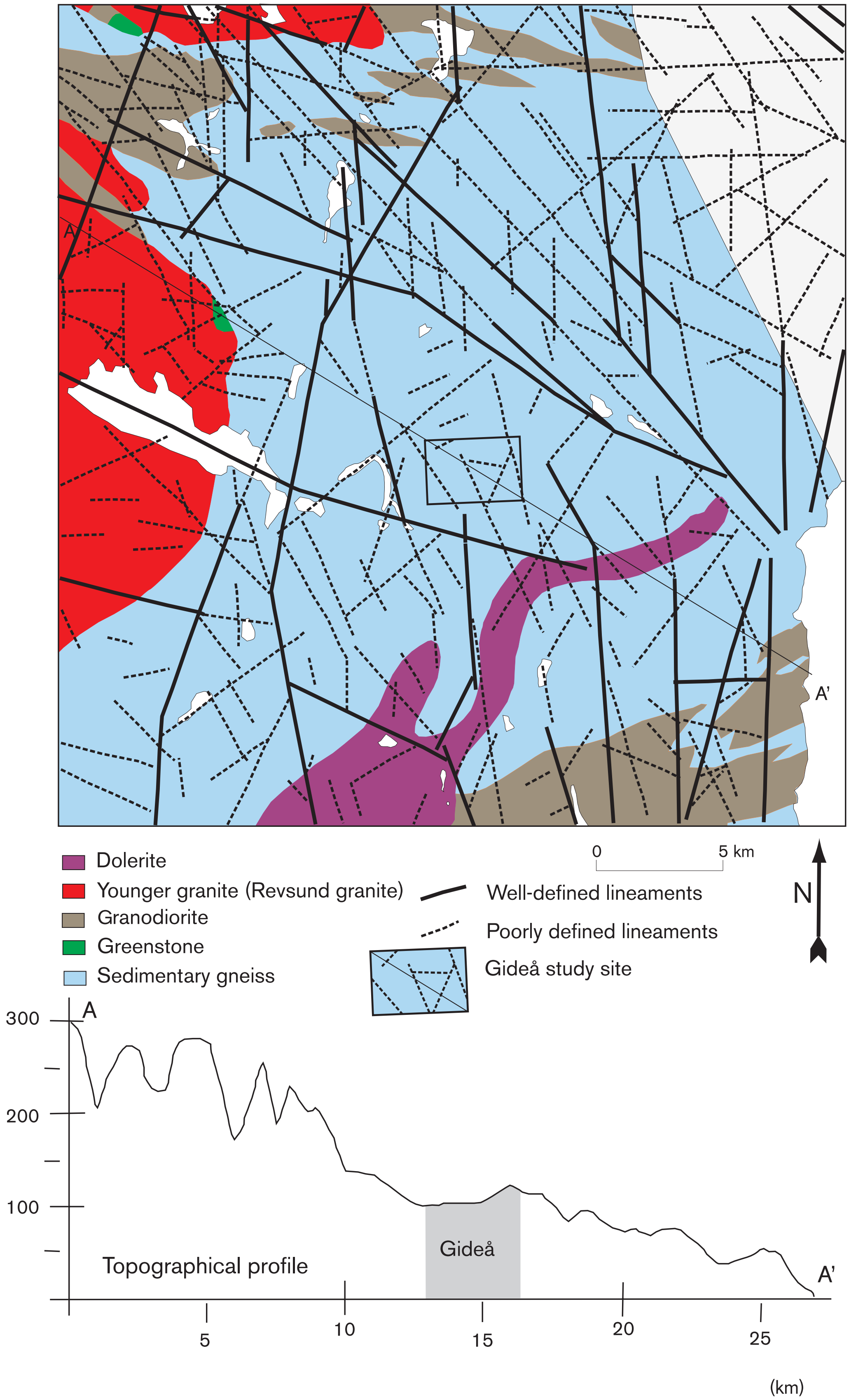


Figure 4-10. Regional geological description of Gideå /after Ahlbom et al, 1983; SGU, 1981; Walker et al, 1997/. Topographical profile in northwesterly direction /after Walker et al, 1997/.

Local scale

Topography

The study site is a 3x2 km area with a relatively even topography (about 150 m above sea level). The site lies between two rivers, Gideälven and Husån. About 40 percent of the ground surface is covered with glacial till deposits. In depressions the till is overlain by sand on about 30 percent of the study site. Outcrops cover about 25 percent of the ground surface. The rest of the area is covered by peat bogs.

Rock types

Gideå is dominated by sedimentary gneiss (metagreywacke) as well as sections of older granite. Vertical dolerite dykes, which can be up to 15 m wide, intersect this bedrock in an east-westerly direction, see Figure 4-11. Thicker dolerite sills have probably overlaid the site previously. Even if no sills have been identified at a depth of 0–700 m, Saksa and Nummela /1998/ do not, however, rule out the possibility that they could be present at greater depths.

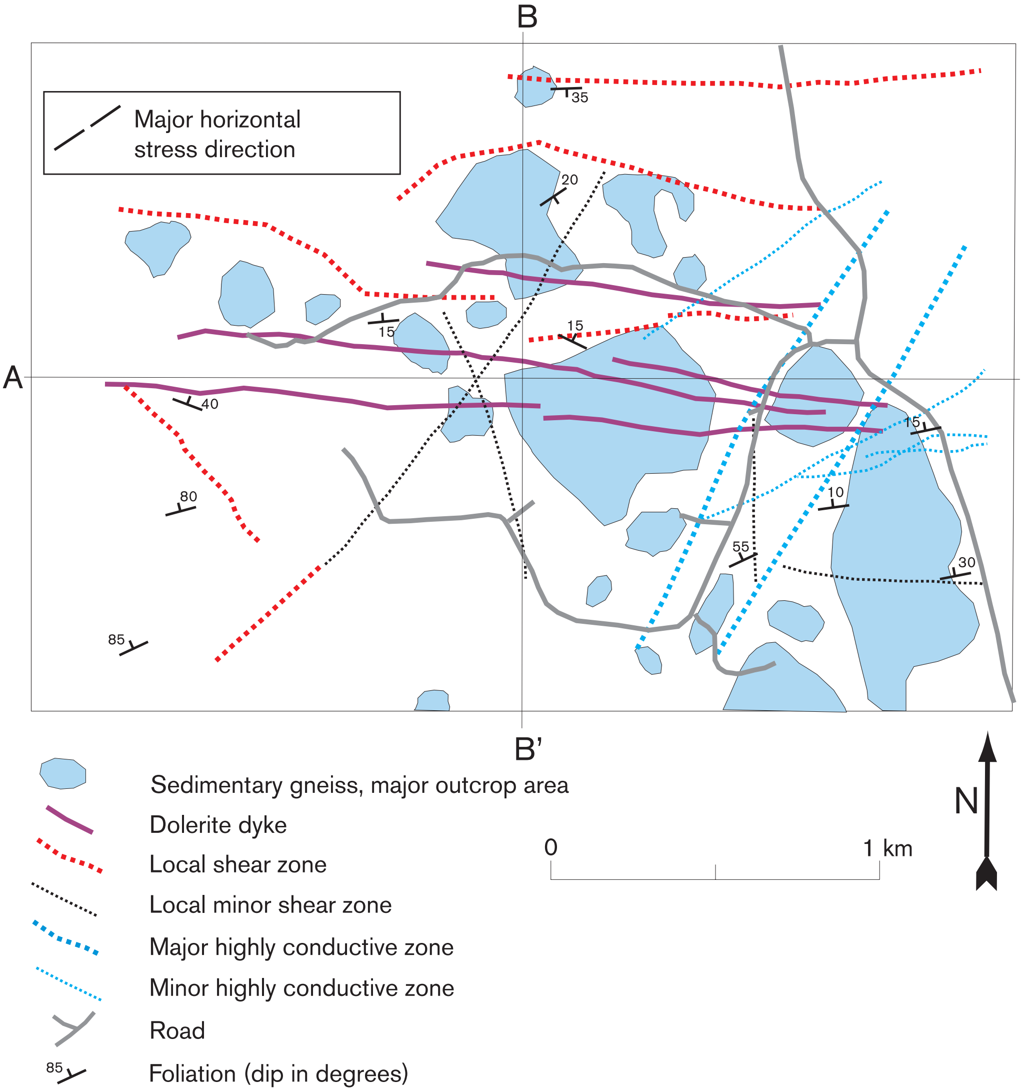


Figure 4-11*. Geological-structural model of Gideå /after Ahlbom et al, 1991, modified by Hermanson et al, 1997/.*

Table 4-5. Fracture zones in Gideå /after Hermanson et al, 1997/. Conductivity values are based on 25 m packer tests, upscaled to 100 m scale /after Walker et al, 1997/. K = conductivity (m/s).

Fracture zone	Strike	Dip	Width (m)	Fracture frequency (fractures/m)	K (m/s)
1	NNE	steep E	41	12	$<5 \cdot 10^{-12}$
2A	NNE	80 E	7–16	6–18	$1 \cdot 10^{-8} - 2 \cdot 10^{-6}$
2B	NNE	60 E	35	6	$1 \cdot 10^{-7} - 5 \cdot 10^{-6}$
3A	E-W	30 N	11–30	10–19	$1 \cdot 10^{-7} - 1 \cdot 10^{-10}$
3B	E-W	steep N	4–12	3–10	$7 \cdot 10^{-11} - 7 \cdot 10^{-12}$
4	E-W	steep N	9	11	$7 \cdot 10^{-12}$
5	E-W	steep N	50		
6	NNE	70 SE	3–10	11–17	$5 \cdot 10^{-9} - 5 \cdot 10^{-12}$
7	NNW	75 E	8	9	$7 \cdot 10^{-10} - 7 \cdot 10^{-11}$
8	NW	steep SE	10	23	
9	E-W	N	5		
10	N-S	90	5		
11A	ENE	75 SE	25	9	$5 \cdot 10^{-7}$
11B	ENE	75 SE	13	8	$2 \cdot 10^{-6}$
12	ENE	75 SE	10		$5 \cdot 10^{-6} - 2 \cdot 10^{-10}$

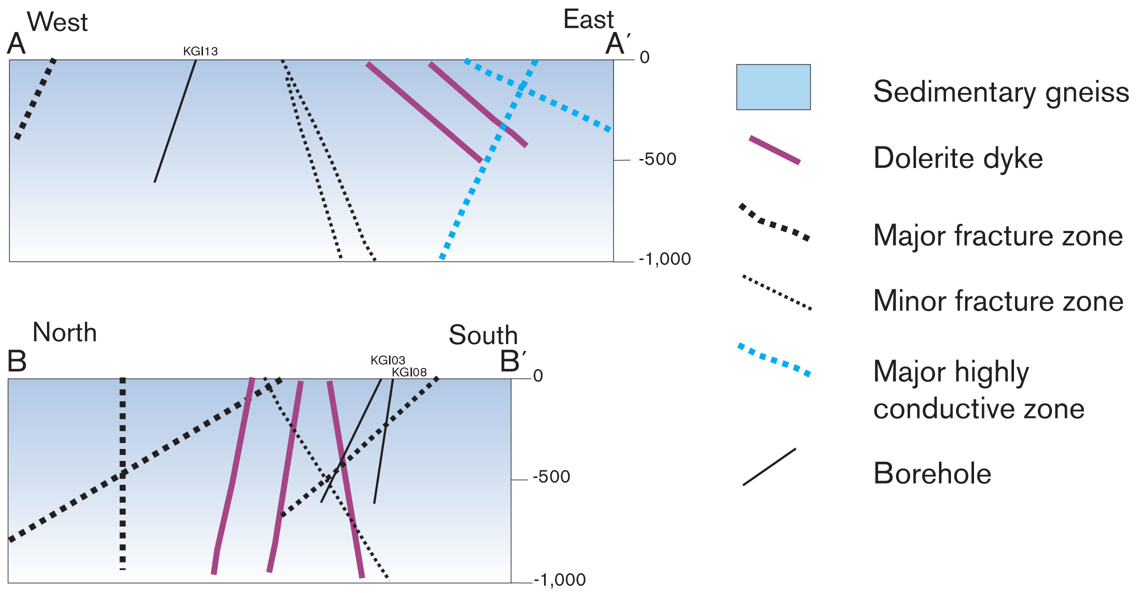


Figure 4-12. Vertical profiles in north-southerly and east-westerly direction (see A-A', B-B' in Figure 4-11) /after Ahlbom et al, 1991/.

Fracture zones and fractures

A total of 15 fracture zones with a width of 5 to 50 m have been identified in the study site, see Table 4-5 /Hermanson et al, 1997/. Most zones have been located by means of geophysical measurements and verified in percussion boreholes and later also in cored boreholes. Several of these fracture zones are water-conducting, Figure 4-11. The fracture zones intersect all rock types in the area and do not follow any rock type boundaries, but may have been affected by the penetrating foliation. Low-dipping zones have not been observed at a depth of 0–700 m. Saksa and Nummela /1998/ point out, however, that low-dipping fracture zones may occur in conjunction with possible flat dolerite dykes at depths below 700 m. Vertical profiles in Figure 4-12 show both dolerite dykes and fracture zones.

Fracture zones have generally been identified by means of an elevated fracture frequency, sections of crushed core and water losses. The most commonly occurring fracture-filling minerals are calcite, chlorite, laumontite, pyrite and the clay minerals smectite and illite. Both fracture zones and dolerite dykes usually contain clay-altered sections. According to Ahlbom et al /1991/, this is the foremost reason why certain fracture zones have low conductivity.

4.3.4 Uncertainties in the geological descriptions

The three sites are located in different geological settings, all of which are representative of Swedish bedrock outside the Caledonide mountains.

Andersson /1999/ summarizes the uncertainties that influence the geological description. The author states that many geophysical investigation methods are indirect and few analysis methods give unambiguous results. This can lead to uncertainties in data processing and use of the wrong concepts.

– However, the main uncertainty depends on the limited availability of data used to describe a large volume. A limited quantity of data may be due to many reasons for example, inaccessible terrain with few outcrops, time limitations that do not permit a complete geological mapping, or surveys that only cover a part of the site.

Table 4-6 shows an overview of the parameters for which input data exist and some form of analysis and interpretation has been carried out on the different sites. Saksa and Nummela /1998/ and Hermanson et al /1997/ give an account of the types of analyses and interpretations that have been carried out.

Site-specific uncertainties are discussed by Saksa and Nummela /1998/:

- The geological models of rock mass and fracture zones used for the sites in SR 97 are largely devised using the same methodology. The greatest uncertainty is the limited description of fracture zones on the scale from 10 to 300–1,000 metres length.
- Äspö lacks a delimited study site. If the island of Äspö is used to delimit the site, there are indications that the rock volume within the site comprises a highly tectonized portion of the region. This means that borehole data from southern Äspö are not necessarily typical for the entire area.
- There are relatively great uncertainties in the geological description for Finnsjön and Gideå.

Table 4-6. Used and analyzed parameters in the structural and geological description in SR 97.

Discipline	Aberg	Beberg	Ceberg
Topography			
Topography	•	•	•
Lithology			
Lithology of the rock mass			
Distribution	•	•	•
Xenoliths	•	•	
Dykes	•	•	•
Contact surfaces	•		•
Age	•	•	
Potentially ore-bearing mineralizations			
Description of rock type			
Mineralogy		•	
Susceptibility/gamma radiation etc.			•
Weathering and alteration		•	•
Structural geology			
Plastic deformation			
Folding	•		•
Foliations	•	•	•
Schistosity			•
Mylonites			•
Banding			•
Age	•	•	
Regional and local discontinuities			
Position	•	•	•
Orientation	•	•	•
Length	•	•	•
Width	•	•	•
Movement (size, direction, age)			•
Number of fracture sets	•	•	
Fracture frequency	•	•	
Block size	•	•	
Mineral infillings			•
Alteration			
Local minor discontinuities			
Position	•	•	
Orientation	•	•	
Length	•	•	
Width	•	•	
Number of fracture sets	•		
Fracture frequency	•		
Block size	•		
Individual fractures			
Frequency (different sets)	•	•	
Orientation	•	•	
Length			•
Mineral infilling	•	•	
Alteration and weathering	•	•	

Five uncertainty indices have been developed by Saksa and Nummela /1998/ based on uncertainties on the regional and local scale, borehole data, spatial extent and variation of surveys and estimates of the structural intensity in the area. The uncertainty concerning the structures in the local study site is great on all sites, but greatest for Äspö, probably partly due to the fact that large parts of the local site are covered by water, which prevents a detailed interpretation. Saksa and Nummela /1998/ believe that the thoroughly investigated rock mass in Äspö is relatively small and may comprise an unusually heavily tectonized portion of the rock that is not representative of the entire area. To handle the uncertainties, Saksa and Nummela /1998/ propose alternative interpretations of the structures in Finnsjön and Gideå. No alternative interpretation is proposed for Äspö since the area chosen for the modelling is naturally delimited and well researched.

4.4 Rock-mechanical description

The site specific body of rock-mechanical data consists primarily of results from rock stress measurements and laboratory determinations of deformation and strength properties of drill core specimens. Rock mass classification results are also available to a varying extent. The classification consists of measurable parameters (e.g. rock type strength, fracture frequency and fracture properties) which are empirically combined to index values. The main aim is to assess the quality of the rock mass from a construction-related viewpoint. However, the classification provides indirect opportunities to appraise the deformation properties of the rock mass as well.

Apart from data relating to loads and mechanical rock properties, rock-mechanical evaluations in a safety analysis must also be based on geological information, thermal property data, and a series of repository-related parameters (geometry, heat production). For example, in analyses of rock movements during an earthquake and its possible consequences for a deep repository, the geological-structural situation, particularly the deformation zones' positions and character, is of crucial importance. The description below is, however, limited to current rock-mechanical data i.e. loads (rock stress) and the mechanical properties of the rock types.

4.4.1 Aberg

The body of rock-mechanical data from Äspö differs from the other sites in two respects. The first is the much greater scope of investigations and data. The second, and more important difference, is that both data from borehole investigations as well as practical experience from construction and operation of the underground rock facilities are available from the area.

Rock-mechanical data from Äspö has been presented in many different contexts. For more comprehensive reports please refer to Stille and Olsson /1996/ and Rhén et al /1997b/.

Rock stress

Prior to the constrution of the Äspö HRL, rock stress measurements were made to a maximum of around 900 m depth in three near-vertical boreholes /Bjarnason et al, 1989/. The methods used were hydraulic fracturing and overcoring. The measurements were supplemented by rock-mechanical laboratory tests of drill cores. Further measurements were then carried out during excavation and operation of the facility, exclusively by overcoring, in a number of short (up to about 20 m) boreholes at different levels, down to a maximum depth of around 450 metres.

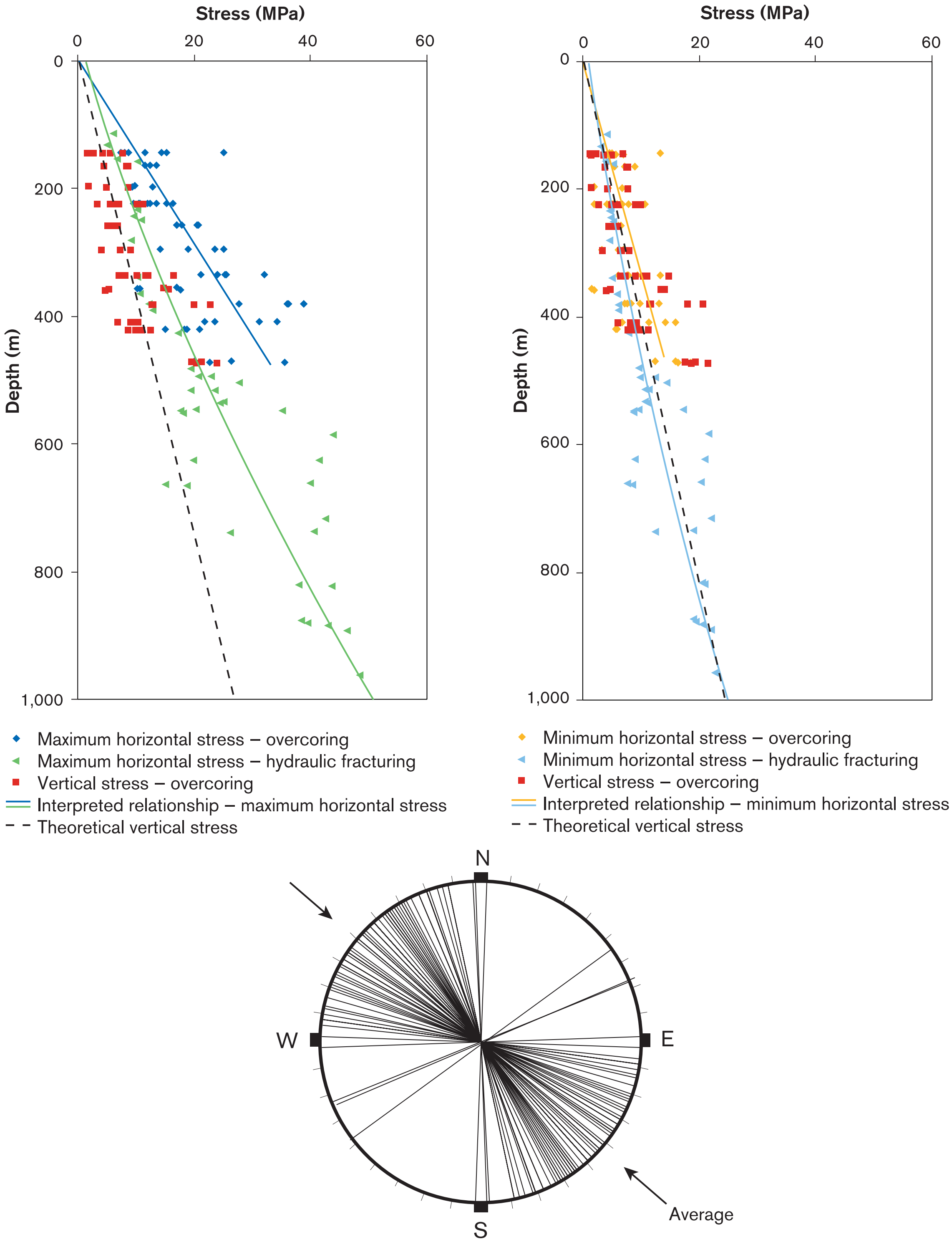


Figure 4-13. *The rock stress situation in Äspö. Above: Magnitude as a function of depth for measured, horizontal and vertical stress components. Below: Orientation of maximum horizontal stress.*

Altogether, a large quantity of rock stress data are thus available from Äspö. Figure 4-13 summarizes the available data. The figure shows that the state of stress in the horizontal plane is decidedly anisotropic throughout, i.e. the maximum horizontal stress is much greater than the minimum ($\sigma_H >> \sigma_h$). The values of σ_H, as well as the rate of increase with depth, are slightly higher than average for Swedish crystalline bedrock, but cannot be said to be abnormally high. The direction of the maximum horizontal stress is northwest-southeasterly with some variation, regardless of depth. This agress well with both measurements in the nearby Laxemar area /Ljunggren and Klasson, 1997/ and trends on a national scale /Stephansson et al, 1991; Ljunggren and Persson, 1995/. The vertical stress is much lower than the maximum horizontal stress and agrees, at least on average, fairly well with the lithostatic load.

Rock stress data from Äspö generally show a broad range /Ljunggren et al, 1998/. More or less systematic differences are also found between the data taken from the different measurements. For example, some of the overcore measurements in the facility show higher values for the maximum horizontal stress than the equivalent measurements using hydraulic fracturing in the boreholes from the surface. The variations and differences can be ascribed to a combination of local variations in the stress field and pure measurement errors in the measuring methods used /Ljunggren et al, 1998/.

Figure 4-13 shows the state of stress in the horizontal plane as well as vertically. The picture does not really change when viewing the complete, three-dimensional state of stress. This is because the principal stress trends are relatively well connected to the horizontal and vertical trends. This is particularly so for the maximum principal stress (i.e. σ_1 approx. equal to σ_H).

Mechanical properties

Investigations into mechanical properties of the rock types, individual fractures and rock mass as a whole have been aimed at providing a basis for technical site forecasts and judgements /Stille and Olsson, 1990/. During the investigation phase, forecasts were made regarding tunnel stability, reinforcement and grouting needs which could then be compared with the actual outcome in the construction phase. In addition, certain tests have been performed as part of the experimental operations at the facility.

Laboratory determinations have been made regarding the deformation and strength properties of the four dominant rock types, Äspö diorite, Ävrö granite, fine-grained granite and greenstone. Core samples were subjected to uniaxial compression and the load-deformation relationship was registered. Thereafter, the uniaxial compressive strength (σ_c), modulus of elasticity (E) and Poisson's ratio (ν) were determined. The mechanical properties of the fractures have been determined to a lesser extent, partly through laboratory shear tests, and partly through measuring the properties of the fracture surface in the field.

The measurement results were not surprising as regards mechanical properties. The highest, average compressive strength was noted for fine-grained granite and Ävrö granite (ca 250 MPa) and the lowest for Äspö diorite (ca 170 MPa). Both the mean values and ranges are normal for the actual rock types.

The construction and operation of the facility to date, has provided a good picture of the rock-mechanical conditions from a practical construction-related viewpoint. Stability in the tunnel and other excavations has generally been good. For example, 96% of a 1,200 m long tunnel section complied with the "quite good or better" rock classification requirements, according to the current rock mass rating norms. Apart from the tunnel passages through the fracture zones, the rock only needed limited reinforcement. Stability problems including spalling or overloading were not encountered, despite the relatively high stresses and partly brittle rock.

4.4.2 Beberg

The rock-mechanical measurements for Finnsjön consist of rock stress measurements and some laboratory tests of core samples. In addition, rock-mechanical analyses were made when the site was chosen as a study site, and for research on fracture zone properties. The rock stress measurement data and the laboratory test data have been summarized by Ahlbom et al /1992/. Rock-mechanical analyses of the fracture zones have been reported by Leijon and Ljunggren /1992/, Rosengren and Stephansson /1990/ and Israelsson et al /1992/.

Rock stress

Bjarnason and Stephansson /1988/ reported rock stress measurements in a vertical borehole. The method used was hydraulic fracturing which means that the results are limited to the stress components in the horizontal plane. The results are shown in Figure 4-14, and are an overview of the stress situation that is very typical of Swedish crystalline bedrock. Thus, the horizontal stress is somewhat higher than the theoretical

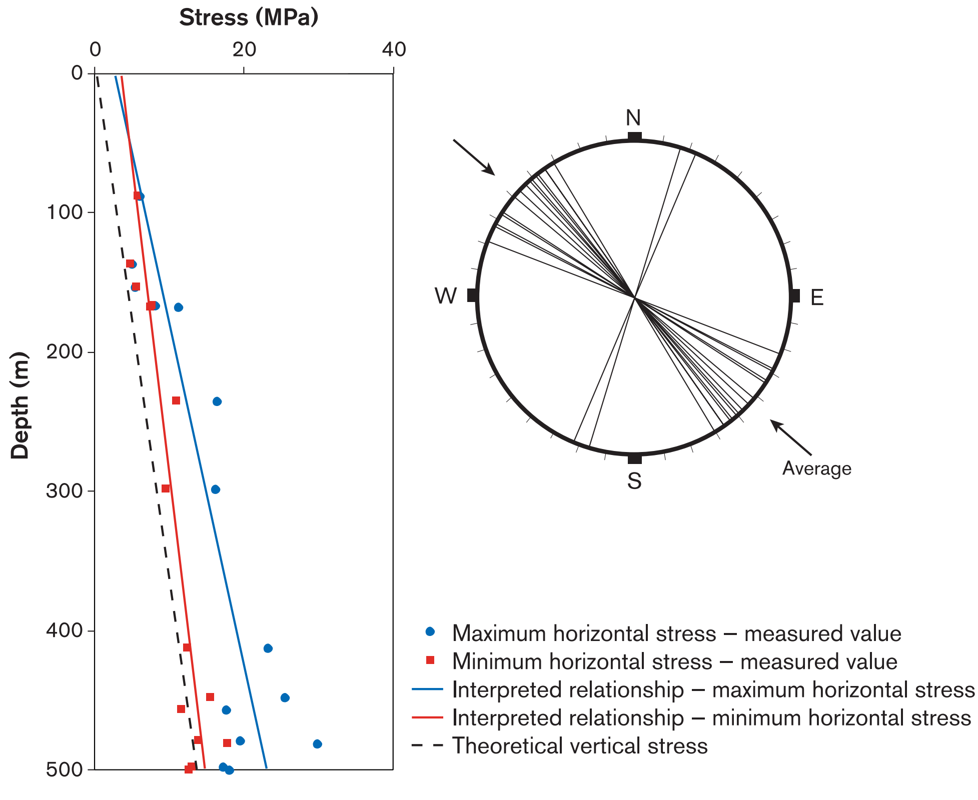


Figure 4-14. The rock stress situation at Finnsjön. Magnitude as a function of depth for the measured horizontal stress components (theoretical vertical stress is shown for comparison purposes) can be seen on the left. The orientation of the maximum horizontal stress – average and for each point of measurement – can be seen on the right. All data are from hydraulic fracture measurements in a vertical borehole.

vertical stress (lithostatic load) and increases gradually with depth to reach 15–25 MPa at a depth of 500 m. With very few exceptions, the principal horizontal stress direction is northwest to southeasterly.

Mechanical properties

Laboratory tests have been carried out to determine deformation and strength properties of the granodiorite that is the dominant rock type /Ahlbom et al, 1992/. As an average value, a compressive strength of 240 MPa and a tensile strength of 14 MPa was maintained, which is relatively normal for this rock type. The modulus of elasticity was set at 82 GPa which is relatively high, but within the variation range for granite rock types.

A rock massclassification overview of the rock mass at Finnsjön, based on rock-mechanical parameter values and data from core mapping, showed medium to good rock quality, although with some local variations. Properties of the dominant flat fracture zones (Zone 2) at Finnsjön have been specifically investigated in many respects /Ahlbom et al, 1989/. Assessments of the deformation properties of the zone have been presented by Leijon and Ljunggren /1992/. The deformation module for the whole zone was assessed at 20-25% of the deformation module for the intact rock. Rock stress measurements were made in a borehole that intersects the actual fracture zone. The results, in Figure 4-14, indicate that the zone does not have any large-scale influence on the stress field. Finally, it should be mentioned that data from Finnsjön have been used for analyses of large-scale stability in relation to glaciation /Rosengren and Stephansson 1990; Israelsson et al, 1992/.

In brief, there is nothing to indicate that the rock-mechanical situation in Finnsjön should deviate in any respect from what is normal for Swedish crystalline bedrock. Rock stress is modest and the limited data available on rock-mechanical properties indicate a good situation as regards rock quality.

4.4.3 Ceberg

The rock-mechanical data from Gideå is approximately the same in scope and type as that from Finnsjön. Rock stress measurements have been made in a borehole and laboratory determinations of the mechanical properties were made on core samples from the same borehole. The data has been compiled and reported by Ahlbom et al /1991/.

Rock stress

Rock stress measurements have been made by hydraulic fracturing, from the surface down to a depth of 500 metres, in a near vertical borehole. The results are shown in Figure 4-15. The measured stress magnitudes are normal throughout for Swedish crystalline bedrock. The magnitude and gradient of the minimum horizontal stress can be compared to the theoretical vertical stress. Data also shows that the increase of rock stress with depth, decreases below a depth of 300 m. This is the reason why the interpreted non-linear relationship can be seen in Figure 4-15. This interpretation is however uncertain and is not supported by any geological information.

According to Figure 4-15, the maximum principal stress orientation can be interpreted as east-westerly to northeast-southwesterly, albeit with large variations and without any tendency to rotation with depth. This deviates somewhat from the general trend with a dominant northwest-southeasterly stress orientation. Such local deviations have been noted in several sites and are not unusual. Furthermore, it cannot be ruled out that the

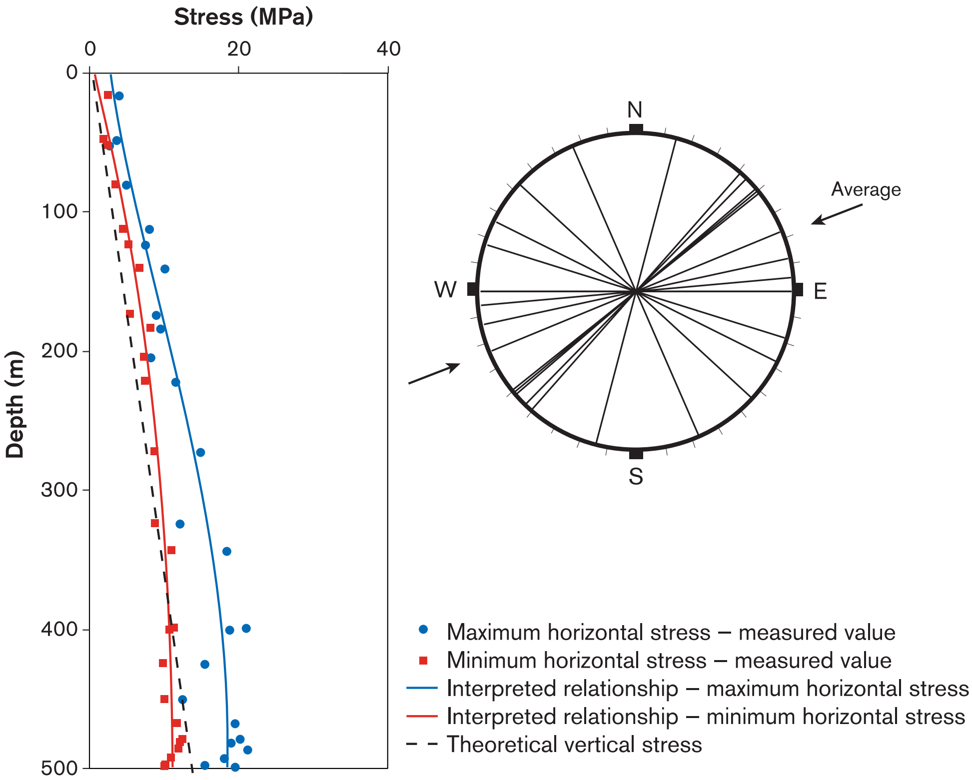


Figure 4-15. *The rock stress situation at Gideå. The magnitude as a function of depth for the measured horizontal stress components (theoretical vertical stress is shown for comparison purposes) can be seen on the left. The orientation of the maximum horizontal stress – average and for each point of measurement – can be seen on the right. All data are from hydraulic fracture measurements in a vertical borehole.*

measurements of the orientations at Ceberg could contain major errors. This is based on the fact that the rock type has a steeply dipping foliation which could have influenced fracture formation during hydraulic fracturing, and thereby the measurement values for the stress orientations.

Mechanical properties

Laboratory tests have been performed on two rock types – the dominant gneiss and granite – from a few core samples from Gideå. In addition to the current uniaxial compression tests, tests were made under confined conditions (triaxial load) and to determine wave propagation velocities (P and S waves), from which the dynamic elasticity properties can be calculated.

The tests showed that the value throughout for granite was normal for this rock type. As regards gneiss, a relatively low value was recorded for the uniaxial compressive strength, averaging 128 MPa. The reason could be the rock type's foliation and/or a relatively high content of clay minerals. Tests performed under confined conditions showed that the strength values increased rapidly in line with the surrounding load, up to around 300 MPa at a confining pressure of 25 MPa. This is normal for crystalline bedrock and indicates that the low uniaxial compressive strength values are attributable to foliation or fractures, unfavourably oriented in relation to the load orientation. As regards deformation properties, the tests showed normal values (average modulus of elasticity 56 GPa). The dynamic tests also showed normal values.

To sum up, the data from Gideå also does not show any deviation in rock-mechanical properties of Swedish crystalline bedrock on the whole. Rock stress is moderate and the limited data that exists on the mechanical properties of the rock indicates a good situation as regards rock quality. Of the properties contained on the actual test scale (core samples), one could perhaps mention mechanical anisotropy, that follows the foliation of gneiss, as a factor that could be important. However, it is unlikely that this anistropy would have negative consequences for stability on a larger scale. Experience shows that foliation and gneissic structure mean that the rock is less brittle than granite, for example, which can be advantageous to tunnel stability.

4.4.4 Uncertainties in rock-mechanical properties

Laboratory determinations of the rock types' mechanical properties are not marred by any major experimental uncertainties in themselves. They are, however, point measurements. Spatial variations and, above all, scale-dependency cause considerable uncertainties during practical applications. Therefore, the determinations of rock types' mechanical properties can in many cases be substituted by experience-based estimates, without particularly adversely affecting the reliability of the analyses' results. This is often the case in stability analyses of excavations in good rock subjected to low stresses.

Rock stress measurements also provide point values. The measurement methods for rock stresses are less exact than is the case for mechanical properties, at the same time as the spatial variation is greater and more difficult to link to the local geology. In measurements in good rock and at stress levels that are typical for depths around 500 m, the uncertainty of individual measurements is usually estimated at a few dozen percent. The uncertainties in the understanding of the rock stress data are rarely the weakest link in rock-mechanical analyses in practice, but there are some important exceptions. These are above all in cases where high loads in combination with sparsely-fractured and brittle rock can cause spalling problems or other forms of instability around the excavations. Thus, even minor errors in input values for stress and/or mechanical properties can result in serious judgmental errors.

As a general conclusion, one can say that the crucial uncertainties rarely lie in the knowledge of the rock types' mechanical properties or in the rock stresses, such as individual components. They lie in the understanding of how the mechanical system of rock mass with the excavations created, will behave under the loads and scale in question. The uncertainties usually apply to the bearing capacity of the rock mass (including fractures and fracture zones), deformation modes and possible failure mechanisms. Experience feedback from engineering applications is the principal method for reducing these uncertainties. The uncertainties therefore increase markedly where previous engineering applications against which forecasts and judgements can be verified are lacking. This is partly the case with the rock-mechanical assessments that are made in conjunction with performance and safety assessments of the deep repository, mainly because they involve timespans that vastly exceed those in the experience base.

4.5 Hydrogeological description

The hydrogeological model encompasses the hydraulic properties of the rock's fracture zones and of the rock mass, as well as describing the groundwater conditions on the site and the processes that control the natural flow of the groundwater.

The hydrogeological descriptions of Aberg, Beberg and Ceberg have been compiled by Walker et al /1997)/. The compilation is based on a large body of material from site investigations within the three sites and evaluations of the hydraulic conductivity of the rock on a regional and local scale.

4.5.1 Aberg

Regional scale

Surface hydrology and groundwater hydrology

The terrain around Äspö contains a large number of smaller runoff areas. These are described in detail in /Walker, 1997/. Average precipitation has been measured at 675 mm/year, of which about 18 percent falls as snow. Average runoff in the area has been calculated at 150 to 200 mm/year.

The regional groundwater table follows the topography and the regional flow follows the topographical level difference from the highlands in the west to the lower coast in the east. The groundwater table reaches its annual maximum level in the spring and its minimum level in August. Brackish water is found under the fresh surface water along the coast (Svensson, 1991).

Hydraulic conductivity

Rhén et al /1997b/ analyzed well data from SGU (Geological Survey of Sweden) in a 25 km radius around Äspö to estimate hydraulic conductivity on a regional scale.

Walker et al /1997/ have calculated hydraulic conductivity and its depth dependence in rock mass and fracture zones. Table 4-7 shows the conductivity of the rock mass regardless of rock type. Alternative cases, where conductivity varies with rock type, are presented by Rhén et al /1997b/ and discussed by Walker et al /1997/.

Table 4-7. Hydraulic conductivity K, m/s, for rock mass on regional scale for Äspö, upscaled to a 100 m scale down to 600 m depth, 300 m scale below 600 m depth /after Walker et al, 1997/.

Elevation (m.a.s.l.)	Arithmetic mean log K	Variance log K	No. of measurements
0 to −200	−6.9	0.92	264
−200 to −400	−6.7	0.42	30
−400 to −600	−6.6	0.62	9
−600 to −2000	−7.3	0.51	11
0 to −2000	−6.9	0.86	314

Only a few hydraulic measurements have been carried out in the regional fracture zones. Rhén et al /1997b/ have divided the zones into water-bearing and less water-bearing (see Table 4-8). The water-bearing zones on a regional scale are assumed to have similar hydraulic properties to the more conductive zones on Äspö and the less water-bearing zones are assumed to be slightly less transmissive than the mean value for the less conductive zones on Äspö

Table 4-8. Transmissivity of the regional fracture zones in Äspö /after Rhén et al, 1997b/. See also Figure 4-3.

Lineament	Transmissivity Mean (T) (m^2/s)
W (less water-bearing)	$0.3 \cdot 10^{-5}$
WW (water-bearing)	$10 \cdot 10^{-5}$

Local scale

Surface hydrology and groundwater hydrology

The Äspö local study site comprises an area of approximately 1 km^2 including the island of Äspö plus the surrounding marine areas. The maximum elevation difference is 14 m. Surface water is drained directly out into the sea or through peatland and sediment.

Figure 4-16 shows the groundwater table under undisturbed conditions (before tunnel construction). The maximum water table both before and after tunnel construction is about 4 m.a.s.l. (metres above sea level). In conjunction with the construction on southern Äspö, the maximum groundwater drawdown was to a level of 85 m below sea level. The pressure drop north of fracture zone EW-1 was considerably less, however, suggesting that the hydraulic connectivity between southern and northern Äspö is limited at depth /Rhén et al, 1997b; Svensson, 1995/.

Hydraulic conductivity

Hydraulic properties of rock mass and fracture zones have been calculated from 3 m packer tests and hydraulic tests from boreholes drilled from the tunnel. Walker et al /1997/ have upscaled the conductivity to the chosen scale for all three repository sites.

Conductivity is given for two types:

– Rock Domain (RD) = borehole sections outside the deterministically determined fracture zones,

– Conductor Domain (CD) = deterministically determined.

On a local scale, the rock mass has been divided into five domains with different conductivities according to Table 4-9. The rock mass has a dominant fracture direction towards the west-northwest, which means that conductivity may be greater in this direction. Based on investigations in percussion boreholes along the tunnel, Rhén et al

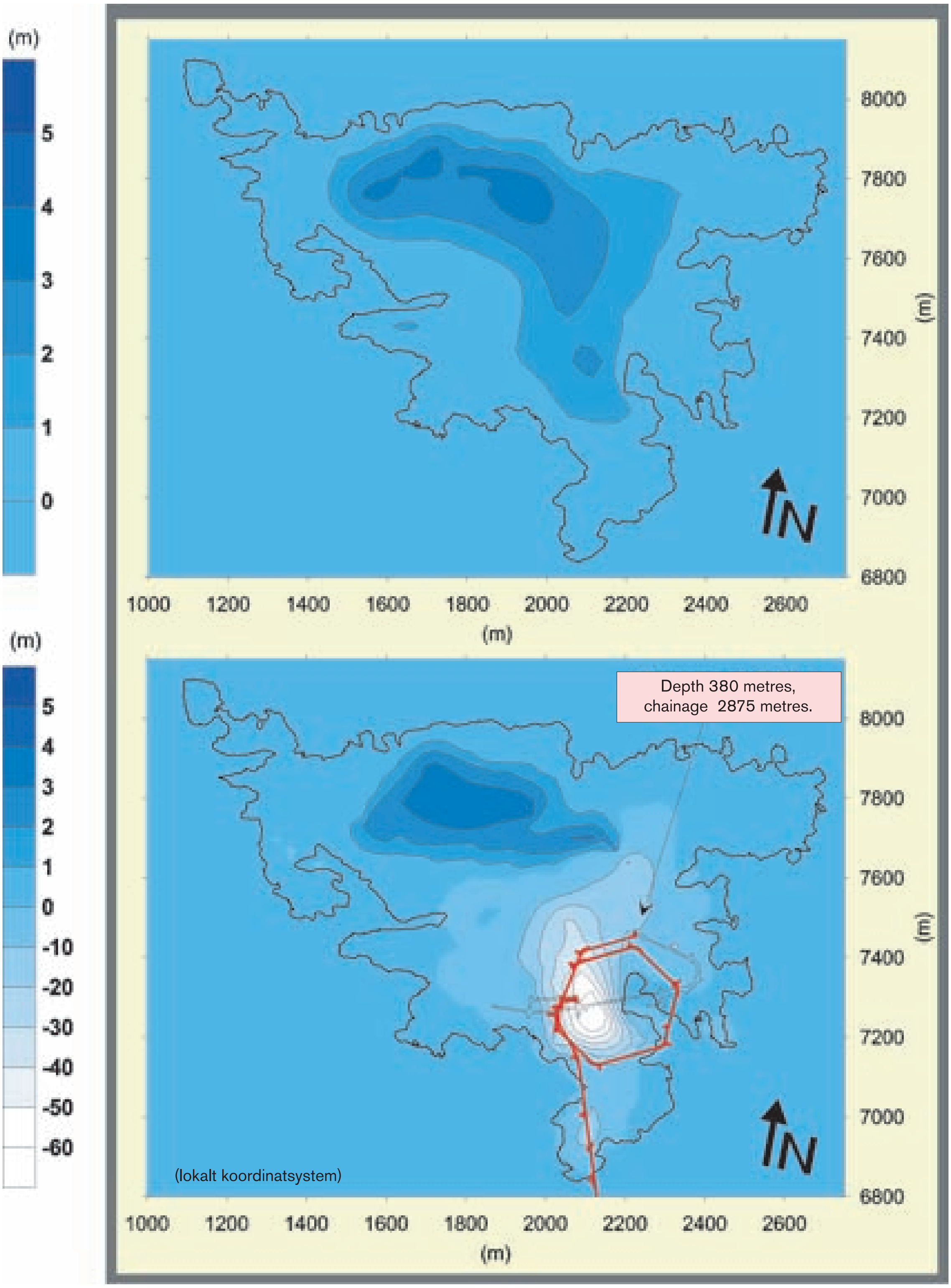


***Figure 4-16**. Observed groundwater table levels at Äspö before and after the construction of the HRL tunnel. The levels are based on measured values /after Walker et al, 1997/.*

/1997b/ have shown that the transmissivity is greatest in the dominant fracture direction. Alternative descriptions of the anisotropy of the rock mass are given by Walker et al /1997/.

Table 4-9. Hydraulic conductivity, m/s, for rock mass on local scale in Äspö. The rock mass is divided into five different domains which represent parts of bedrock containing different conductivities. The values are based on 3 m packer tests but are upscaled to a 24 m scale /after Rhén et al, 1997b, and Walker et al, 1997/. The upscaling was done prior to numerical modelling.

Domain	Arithmetic mean log K
RD1	–8.0
RD2	–7.1
RD3	–8.8
RD4	–7.5
RD5	–7.6
Other	–8.6

Most of the conductive fracture zones are indicated in the geological-structural model. Additional conductive structures are found hydraulically, but exhibit only a weak elevation of fracture frequency and can be regarded as fracture sets. The north-northwesterly structures, which intersect the central portion of the HRL, can also be regarded as fracture sets /Walker et al, 1997/. Most structures have been tested at least once, and in several cases a large number of hydraulic tests have been conducted. A more complete account of the hydraulic properties of the conductive fracture zones can be found in Rhén et al /1997b/ and Walker et al /1997/.

4.5.2 Beberg

Regional scale

Surface hydrology and groundwater hydrology

The northern part of the region around Finnsjön is drained by the Dalälven, Tämnarån, Forsmarksån and Olandsån rivers. The southern part is drained by Lake Mälaren via the Öresundaån and Fyrisån rivers. Average precipitation in northern Uppland is 670 mm/year. Runoff in the area is calculated at 240 mm/year.

The groundwater table in the region is relatively flat due to the flat topography. The regional groundwater flows from southwest to northeast /Andersson et al, 1991/. The regional gradient is estimated to be about 0.2–0.3%. Figure 4-17 shows a regional map of the water table in Beberg.

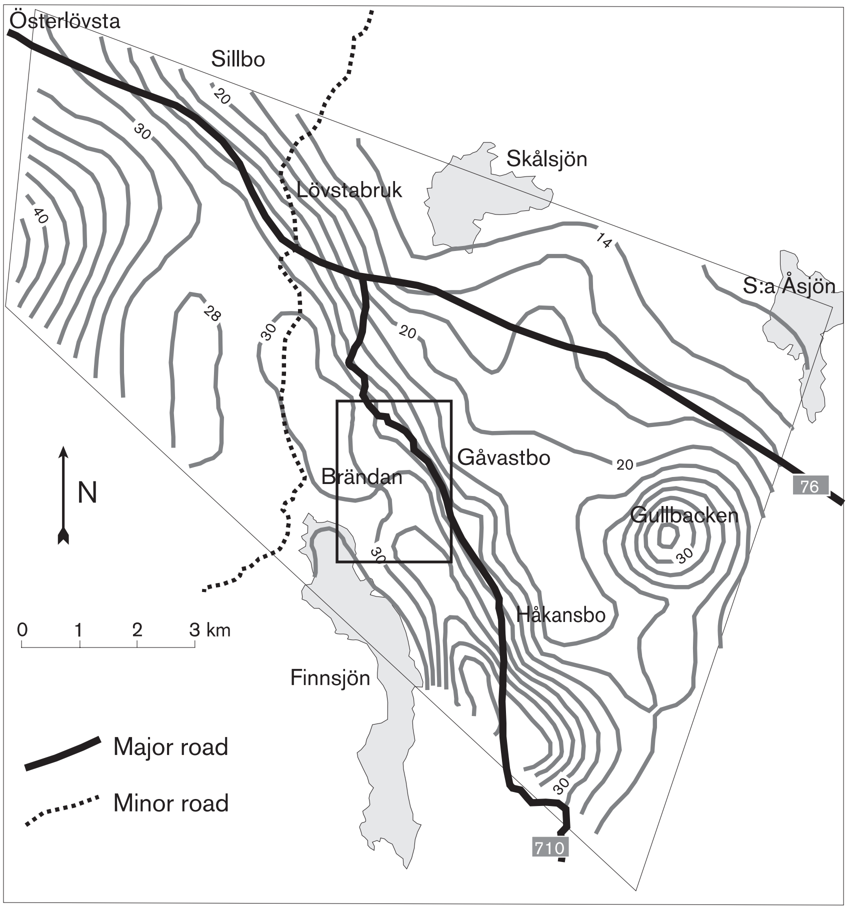


Figure 4-17. Regional map of groundwater table in and around Finnsjön /after Andersson et al, 1991/.

Hydraulic conductivity

Walker et al /1997/ have used data from SGU's well archive for wells within a 25 km radius of Finnsjön in an attempt to estimate the hydraulic conductivity on a regional scale. Since the wells are generally not deeper than 200 m, data are used from packer tests from the local study site in order to estimate the depth dependence of the conductivity and to quantify the difference between the conductivity in rock mass and fracture zones, see Table 4-10. There are, however, several possible interpretation cases where the hydraulic conductivity varies depending on assumptions concerning regional heterogeneity in the rock mass /Walker et al, 1997/.

Table 4-10. Hydraulic conductivity K, m/s, for rock mass on regional scale for Finnsjön based on SGU data and 3 m packer tests. The values are upscaled to a 100 m scale /after Walker et al, 1997/.

Elevation (m.a.s.l.)	Arithmetic mean log K	Variance log K	Number of measurements
Over –100	–6.8	0.21	142
–100 to –200	–7.7	0.15	119
–200 to –400	–8.1	0.097	325
Below –400	–7.8	0.042	265

Porosity measurements were made in the upper part of fracture zone 2. The flow porosity varies between $8 \cdot 10^{-4}$ and $1 \cdot 10^{-2}$ /Andersson et al, 1991/.

Several regional lineaments that cross through the study site have been investigated by boreholes. Further information on regional fracture zones is provided by the investigations around SFR. The hydraulic conductivity in the fracture zones is based on 3 m packer tests on the study site and in SFR /Andersson et al, 1991; Lindbom and Boghammar, 1992/, see Table 4-4 and Figure 4-7.

Local scale

Surface hydrology and groundwater hydrology

The ground surface on the study site is relatively flat and varies around 25 m.a.s.l. The southern block contains a watershed with drainage basins to the northeast and southwest. The northern block drains to the northeast.

The near-surface groundwater system is complex, with flow both to the northeast and the southwest. It is not known how deep this system penetrates. The maximum water table level occurs in November and December, with low water in August and September.

The deep groundwater system is relatively complex with subhorizontal fracture zones and abrupt salinity variations. Detailed investigations in fracture zone 2 in the northern block (see Figure 4-9) show that there is fossil saline groundwater underneath the subhorizontal structure. In the southern block there is no saline water down to a depth of about 600 m (maximum drilling depth), which shows that this part of the study site has good contact with superficial groundwater /Walker et al, 1997/. Just east of the Finnsjön area, however, saline water was encountered in structures that are also found in the southern block. This can be explained by the fact that impervious glacial clay overlies the rock surface here, obstructing throughflow of superficial groundwater.

Hydraulic conductivity

The hydrogeological model, which is based on a geological-structural model by Andersson et al /1991/, differs between the rock mass and fracture zones. The rock mass is in turn divided into a southern and a northern block. Table 4-11 shows hydraulic conductivity at different depths in the rock mass. The conductivity is calculated from 3 m packer tests and upscaled to 24 m blocks /Walker et al, 1997/ prior to modelling.

In addition to this base case, Walker et al /1997/ propose alternative hypothetical models in which the rock mass is regarded as anisotropic and dependent on the dominant fracture direction and the stress field in the area.

The positions and hydraulic conductivities of the fracture zones are given in Figure 4-8 and Table 4-4. The hydraulic properties are based on 3 m packer tests and tracer tests. In the base case, the conductivity is assumed to be the same at all depths. However, several alternative cases, with varying conductivity towards depth and with the same relative change as in the rock mass, are discussed by Walker et al /1997/.

Table 4-11. Hydraulic conductivity K, m/s, for rock mass on local scale for Finnsjön, based on 3 m packer tests. The values are upscaled to 24 m /after Walker et al, 1997/.

Elevation (m.a.s.l.)	Arithmetic mean log K
Northern block	
Above –100	–6.6
–100 to –200	–7.2
–200 to –400	–7.8
Below –400	–7.6
Southern block	
Above –100	–6.8
–100 to –200	–7.8
–200 to –400	–8.1
Below –400	–7.9

4.5.3 Ceberg

Regional scale

Surface hydrology and groundwater hydrology

Average precipitation for the area is 765 mm/year. Timje /1983/ has estimated the runoff to be 345 mm/year, with the peak runoff occurring in May. The mean annual net recharge of surface water to the groundwater system is 10 mm, but may vary locally depending on the topography /Walker et al, 1997/.

SGU's well data and data from 24 percussion boreholes have been used to assess the regional groundwater table /Timje, 1983/. The map in Figure 4-18 represents an estimated mean groundwater level and assumes that the groundwater table follows the ground surface. The seasonal fluctuation in the groundwater table is 0.2 to 3.9 m, with the annual low water levels occurring in January through March and the annual high water levels occurring in April and October /Ahlbom et al, 1983/. Both the geographic location of the wells and the contours of the groundwater table map indicate that the groundwater system on the central plateau in the region is dominated by a recharge area and that discharge takes place to streams in the fracture zone valleys /Walker et al, 1997/.

It is reasonable to assume that the regional groundwater system is driven by the topography and the groundwater runs from the uplands north and west of the site, through the study site, towards the Gulf of Bothnia.

Hydraulic conductivity

Walker et al /1997/ have used SGU's well data in a 25 km radius around Gideå in order to estimate the hydraulic conductivity of the rock mass and of fracture zones. Rock type and rock type contacts have no or little influence on the hydraulic conductivity, according to Hermanson et al /1997/.

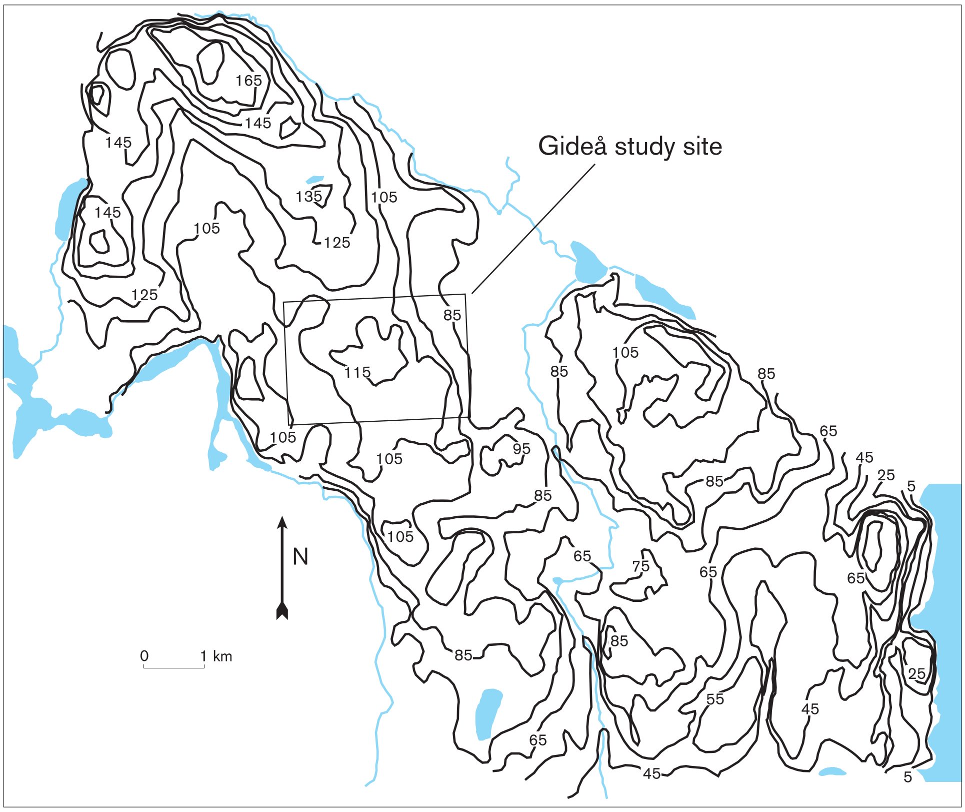


Figure 4-18. Regional groundwater map of Gideå /from Timje, 1983/.

Table 4-12 shows the conductivity of the rock mass upscaled from 25 m to a 100 m scale. The variation of the conductivity with depth is described by division of the domain into depth zones /Walker et al, 1997/. There are several possible conductivity domains for the rock mass depending on a number of alternative cases, for example Ericsson and Ronge /1986/ suggested a regional anisotropic conductivity field parallel to the direction of the principal stress. Ahlbom et al /1983/ suggested that the dolerite dykes may have a higher conductivity than the surrounding rock. Hermanson et al /1997/ believe that the opposite is true, i.e. that the dolerites are less conductive than the surrounding rock. According to Saksa and Nummela /1998/, however, an elevated conductivity cannot be ruled out in the dolerite dykes or along their contacts. This is why several variants with different assumptions have been made within the framework of SR 97.

Table 4-12. Hydraulic conductivity K, m/s, for rock mass on regional scale for Gideå. The values are upscaled to a 100 m scale /after Walker et al, 1997/.

Elevation (m.a.s.l.)	Arithmetic mean log K	Variance log K
+110 to 0	−7.2	0.38
0 to −100	−8.6	0.70
−100 to −300	−9.6	0.71
below −300	−9.8	0.99

Flow porosity data at depth are not available for Gideå. Data from the local scale in Äspö and Finnsjön show a flow porosity of $2 \cdot 10^{-4}$ to $1 \cdot 10^{-3}$ /Ahlbom et al, 1983/.

The hydrogeological model for fracture zones is based on the lineament map in Figure 4-10 and on 25 m packer tests from the local study site. Table 4-13 shows conductivity values for fracture zones after upscaling to 100 m with zonation for depth dependence /Walker et al, 1997/. The properties of the fracture zones are given with the assumption that the regional lineaments are hydraulically comparable and that they have a mean width of 20 m. The flow porosity in the zones is assumed to be the same as in the rock mass.

Table 4-13. Hydraulic conductivity K, m/s, for fracture zones on regional scale for Gideå. The values are upscaled to a 100 m scale /after Walker et al, 1997/.

Elevation (m.a.s.l.)	Arithmetic mean log K
+110 to 0	–6.4
0 to –100	–7.9
–100 to –300	–8.9
Below –300	–9.1

Local scale

Surface hydrology and groundwater hydrology

The land surface within the site varies between 80 and 120 m.a.s.l., with the lowest area situated in the northwest corner of the site. An extensive peat bog, Stormyran, is located in the northeast corner of the site /Ahlbom et al, 1983/. There are three watersheds on the site, one draining south and west to the Gideälven river, one draining north and west to the Gideälven, and a third draining north and east into Stormyran and the Husån river.

The groundwater table ranges from 0 to 10 m below the ground surface in lowland and upland terrain, respectively. Timje /1983/ mapped the groundwater hydrology on the repository scale based on the assumption that the water table follows the topography. Figure 4-18 shows that there is a recharge area to the groundwater system in the central portions and discharge to rivers and streams in the fault valleys.

The variation of the potential groundwater system with depth has been explored in 13 cored boreholes with a maximum depth of 700 m, showing that the local study site is situated in a recharge area.

Hydraulic conductivity

As on the regional scale, the site is divided into two different conductivity domains: the rock mass and fracture zones. The division is based on the structural model by Hermanson et al /1997/. The domains differ through relatively low conductivities in the rock mass compared with the zones /Walker et al, 1997/. Hydraulic properties for structures and domains are based on 25 m packer tests. Tracer tests have not been conducted aside from a small fallout study after Chernobyl /Gustafsson et al, 1987/.

The hydraulic conductivity of the rock mass, given in Table 4-14, is based on 25 m packer tests. There is no correlation between rock mass conductivity and rock type /Hermanson et al, 1997/.

Table 4-14. Hydraulic conductivity K, m/s, for rock mass on local scale for Gideå. The values are on a 25 m scale /after Walker et al, 1997/.

Elevation (m.a.s.l.)	Arithmetic mean* log K
+110 to 0	–7.6
0 to –100	–9.0
–100 to –300	–10.0
Below –300	–10.2

* Corrected values according to Wen (1994) and assuming normally distributed data.

The conductivity in the fracture zones is based on the 25 m packer tests within the borehole sections where the zones are identified. This leads to relatively few representative measurements, so hydraulic conductivity is assumed to be the same in all the zones. As in the rock mass, the conductivity in the conductor domain varies with the depth as shown in Table 4-15.

Table 4-15. Hydraulic conductivity K, m/s, for conductor domain on repository scale for Ceberg. The values are on a 25 m scale /after Walker et al, 1997/.

Elevation (m.a.s.l.)	Arithmetic mean* log K
+110 to 0	–7.0
0 to –100	–8.5
–100 to –300	–9.5
Deeper than –300	–9.7

* Corrected values according to Wen (1994) and assuming normally distributed data.

In addition to the above base case, Walker et al /1997/ also present a number of variations of the conductivity field in the fracture zones which take into account different configurations of regional lineaments that are interpreted as intersecting the study site, plus domains for the rock mass with deviant conductivity properties depending on dominant directions of the fracturing of the rock mass.

4.5.4 Uncertainties in the hydrogeological descriptions

The hydrogeological model in SR 97 is described as a rock mass with varying hydraulic conductivity, a so-called stochastic continuum, intersected by conductive zones. The hydraulic conductivity of the rock can also be described with the aid of alternative conceptualizations, with discrete fracture networks or channel networks. These alternative models are also analyzed within the framework of SR 97.

Andersson /1999/ identifies several possible uncertainties in the hydraulic description where coupled phenomena between flow and rock mechanics, thermal driving forces and density effects due to salinity comprise the main points. Silting-up or erosion of flow paths can also significantly affect the water flow.

On a regional scale, the hydraulic conductivity of the rock mass is based on well data, while the conductivity of fracture zones is based on interpretation of regional lineaments and the results of packer tests. On a local scale, conductivity in the rock mass is based on packer tests. Corresponding data for fracture zones is determined by means of packer tests and interpretations based on the selected geological-structural model. The upper boundary conditions of the hydrogeological model are determined by evaluation of the surface hydrology in the area.

Despite differences in investigation methodology, Walker et al /1997/ have compared the hydrogeological descriptions of the sites and drawn several qualitative conclusions:

– The differences between the hydrogeological characteristics of the sites can be described in reference to their general hydrological features, i.e. recharge and discharge areas, topographical relief, proportions of saline vs. fresh water.

– The hydraulic conductivities of Äspö and Finnsjön are on the same level. Gideå generally has lower hydraulic conductivity.

– The contrast in hydraulic conductivity between fracture zones and the rock mass is lower in Gideå than on the other sites.

Andersson /1999/ discusses uncertainties coupled to the hydrogeological description given by Walker et al /1997/. The following sources of uncertainties are mentioned:

– The distribution of recharge and discharge areas in the region and the formation of groundwater entails uncertainties in the upper boundary condition in the hydrogeological model.

– The division of conductivity domains into rock mass and hydraulically conductive fracture zones gives alternative interpretations on both regional and local scales.

– Interpretation of water injection tests imply simplifications and assumptions. Since the test lengths vary, this leads to influence from different-sized rock volumes. The availability of test data above the lower measurement limit is limited. This is particularly true for Gideå, where 36 percent of all injection tests lie below the measurement limit.

– Scaling of water injection tests to other scales than those that apply to the original data prior to modelling gives rise to uncertainties.

– Hydraulic conductivity can be direction-dependent.

For uncertainties that cannot be handled directly, a number of variants are formulated for the basic interpretation of data. In addition, cases are analyzed with interpretation of the hydraulic conductivity of the rock by using discrete networks and channel networks.

4.6 Groundwater chemistry description

The groundwater chemistry model describes the groundwater's chemical composition and distribution in the rock. Groundwater chemistry influences many of the analyses that provide input data to SR 97.

The groundwater chemistry description for use in SR 97 is summarized by Laaksoharju et al /1998. The geochemical composition of the groundwater is represented by samples taken from four boreholes from the sites Äspö (KAS02), Finnsjön (KFI07 and BFI01) and Gideå (KGI04). With the aid of principal component analysis and isotope ratio analysis of these waters, Laaksoharju et al /1998/ have described the water as glacial, meteoric, biogenic (Baltic Sea water altered by a bacterial sulphate reduction process), marine or brine (fossil saline water with a salinity of over 10 percent) (see Figure 4-20). The samples from the sites in SR 97 cover the different types of chemical groundwater composition measured at other places in Sweden. The chemical composition of the reference water is shown in Table 4-16.

4.6.1 Aberg

The geochemical composition of the groundwater at Äspö is represented by samples taken from the KAS02 borehole. The reference water sample indicates a saline, lime-poor and sulphate-rich water (see Table 4-16). The chloride distribution can be attributed to a complex mixture of waters of different origins. The reference water sample contains a mixture of meteoric water (31%), glacial water (29%), biogenic water (15%), brine (14%) and marine water (12%).

4.6.2 Beberg

The reference water samples show both a saline, lime-poor and sulphate-rich water (BFI01), and a lime-rich, chloride- and sulphate-poor fresh water (KF107), see Table 4-16. The chloride distribution in water samples from borehole BFI01 can be explained by the fact that the groundwater is a mixture of meteoric water (30%), glacial water (26%), biogenic water (19%), brine (16%) and marine water (10%). Water samples from borehole KFI07 indicate a mixture of meteoric water (50%), glacial water (15%), biogenic water (22%), brine (3%) and marine water (10%).

4.6.3 Ceberg

The geochemical composition of the groundwater at Gideå is represented by samples taken from the KGI04 borehole. The reference water sample indicates a lime-poor and sulphate-poor fresh water. Based on the chloride distribution, the water has been interpreted as a mixture of meteoric water (43%), glacial water (28%), biogenic water (17%), marine water (7%), and brine (5%). The chemical composition of the reference water is given in Table 4-16.

4.6.4 Uncertainties in the groundwater chemistry descriptions

The uncertainties in the groundwater chemistry descriptions include measurement uncertainties and sampling errors, as well as representativity of the water samples and spatial variability.

According to Laaksoharju et al /1998/, the reference water samples represent a typical water from each site at repository level (500 ± 100 m depth). The results of the principal component analysis shown in Figure 4-19 show that the four reference water samples have a composition that is comparable to relevant water samples from different places in Sweden.

Table 4-16. Chemical composition of reference waters /after Laaksoharju et al, 1998/.

Groundwater components	Unit	Aberg	Beberg		Ceberg
		KAS02	*BFI01*	*KFI07*	*KGI04*
Date of measurement		88-05-04	86-10-27	80-11-19	82-07-04
Depth	m	528	436	508	384
Drilling water content	%	0.19	0.02	–	11.03
Na^+	mg/l	2100	1700	275	105
K^+	mg/l	8.1	13.0	2.0	1.9
Li^+	mg/l	1.0	0.007	–	–
Ca^{2+}	mg/l	1890	1650	142	21
Mg^{2+}	mg/l	42	110	17	1.1
Sr^{2+}	mg/l	35	21	–	–
Fe^{2+}	mg/l	0.24	–	1.80	0.05
Mn^{2+}	mg/l	0.29	0.82	0.13	0.01
HCO_3^-	mg/l	10	47	278	18
SO_4^{2-}	mg/l	560	370	49	0.1
Cl^-	mg/l	6410	5500	555	178
I^-	mg/l	–	0.12	–	0.14
Br^-	mg/l	40	32	–	–
F^-	mg/l	1.5	1.2	1.5	3.2
HS^-	mg/l	0.15	0.01[1)]	–	0.01[1)]
NH_4^+ calculated as N	mg/l	0.03	0.35	0.09	0.012
NO_3^- calculated as N	mg/l	0.010[1)]	0.005[1)]	0.002[1)]	0.009
NO_2^- calculated as N	mg/l	0.001[1)]	0.005	0.010	0.001[1)]
$PO_{4.tot}$ calculated as P	mg/l	0.005	0.005	0.040	0.008
SiO_2 calculated as Si	mg/l	4.1	5.4	5.6	4.7
DOC (dissolved organic carbon)	mg/l	1.0	–	5.7	2.0
TDS (total dissolved solids)	mg/l	11107	9457	1339	338
U	mg/l	3.86	19.3[3)]	0.35	0.17
Th	mg/l	0.022[2)]	0.120[3)]	–	0.028
^{14}C age corrected with ^{13}C	years	–	8640	4610	–
D	SMOW	-97.2[2)]	-88.7	-89.0	-99.4
T	TU	8[2)]	3[2)]	8	8
^{18}O	SMOW	–12.30[2)]	-11.81	-11.90	-13.63
Representative Eh	mV	–308	–	-250	-202
Representative pH		7.73	7.04	7.90	9.30
Representative electrical conductivity	mS/m	1890	1610	1906	5.5
Ionic strength		0.240	0.210	0.0252	0.00619

1) under the detection limit
2) measured 95-05-05
3) oxidation during drilling

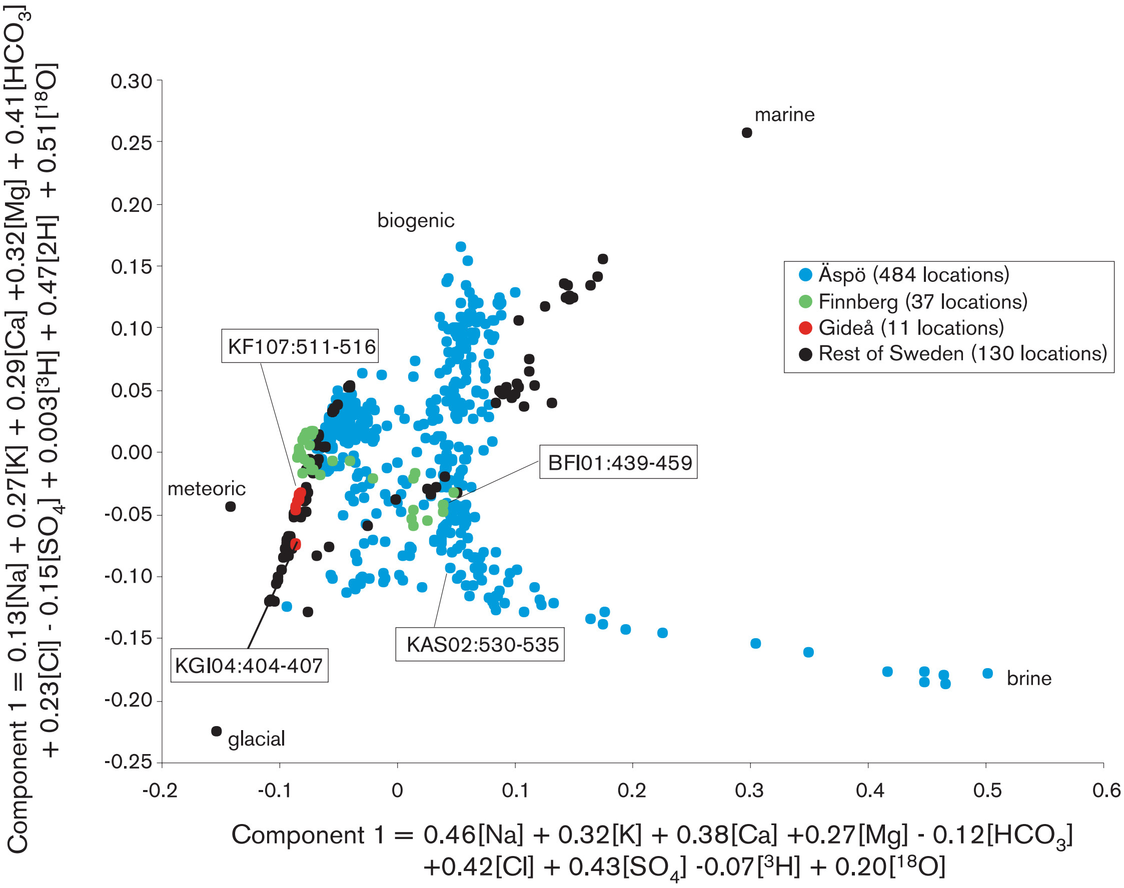


***Figure 4-19**. Analysis of the principal components contained in all water samples taken from Äspö, Finnsjön and Gideå as well as from other places in Sweden. Reference water samples are marked in the figure /after Laaksoharju et al, 1998/.*

Uncertainties in the geochemical model have been analyzed by Andersson /1999/ and include:

- Disturbances at the time of measurement due to tunnel construction and facility operation, which alters the flow pattern around the repository, with a risk of introduction of more saline groundwaters from deeper parts of the bedrock.
- Measurement errors in analysis of the principal components (Na^+, K^+, Mg^{2+}, HCO_3^-, Cl^-) in the order of 1–5% and pH that can vary by up to one pH unit.
- Future changes in groundwater movements, for example as a result of land uplift, which will affect water chemistry. The salinities are expected to change as the shoreline is displaced.
- Installation of engineered barriers (bentonite) that can affect the groundwater composition in the repository and the near-field rock.

Andersson /1999/ considers it reasonable to assume that the uncertainty in groundwater chemistry, with regard to both spatial variations and future evolution, is largely limited by the variation that exists between the four reference groundwater compositions.

4.7 Geothermal properties

The geothermal description for Äspö, Finnsjön and Gideå includes mean temperature, general temperature gradient and thermal conductivity of the rock types.

The temperature in Swedish rock varies with the latitude. Figure 4-20 shows a compilation of rock temperatures in Sweden at a depth of 500 m. It is based on information from some sixty (mostly deep) boreholes drilled in connection with mine prospecting, geothermal prospecting and SKB's siting studies. The figure shows that the temperature at a depth of 500 m is estimated to be 7.5–10°C in northern Sweden and 10–15°C in central and most parts of southern Sweden. The rock temperature in the southernmost parts of the country and on Gotland is estimated to be 15–20°C.

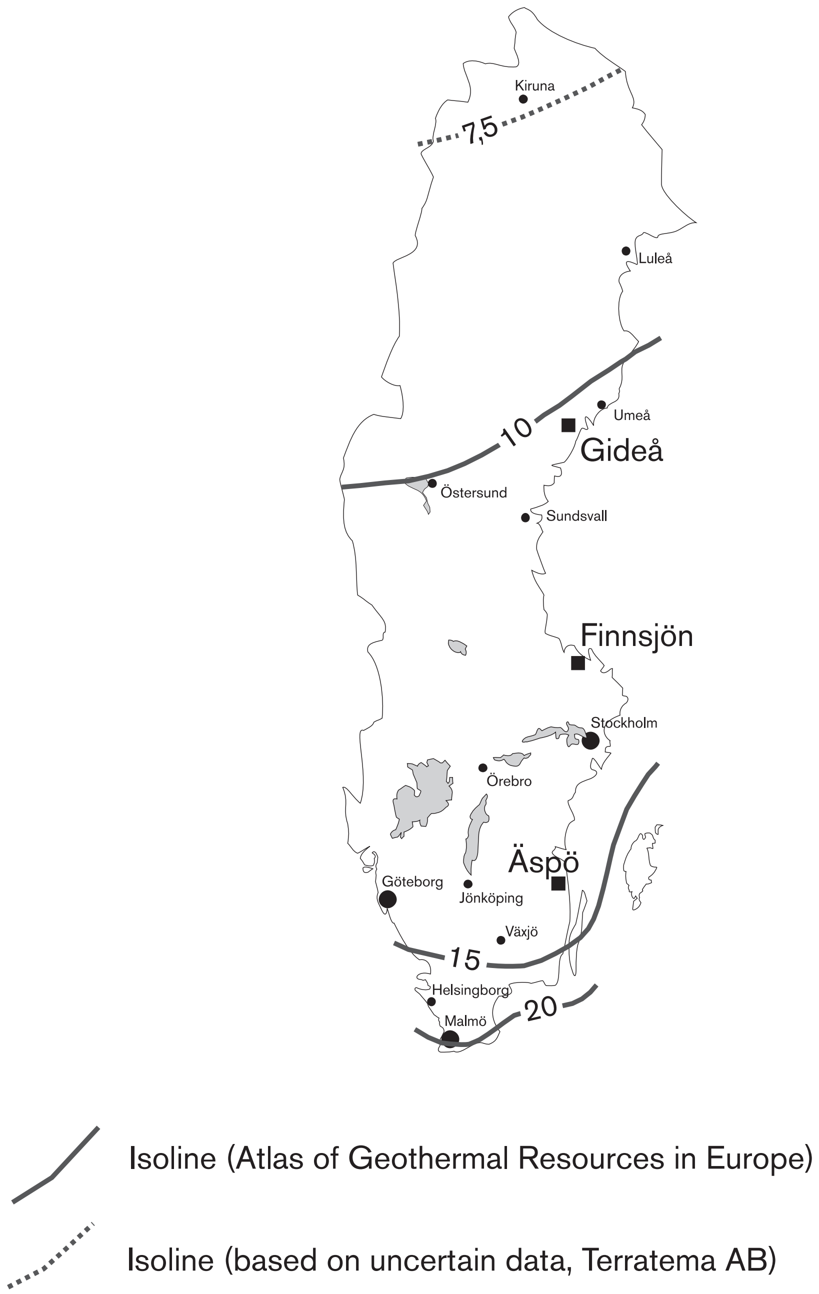


***Figure 4-20**. Temperature at 500 m depth in Sweden /after Sundberg, 1995/.*

Due to the process of nuclear decay in the spent fuel, deposited fuel canisters will generate heat. In the near field of the repository, maximum temperatures will be reached a few dozen years after deposition. The decay heat declines with time, and after 1,000 years heat production will have almost completely ceased. At first the canisters only cause local heating.

The course of the rock heating process is controlled by a number of rock and design parameters. The most important rock parameters are the original temperature distribution and the thermal conductivity of the rock.

The induced heating overlays the original temperature distribution. A higher initial temperature therefore gives roughly an equivalent increase of the absolute temperatures after deposition. Further, the better the thermal conductivity of the rock is, the lower are the maximum temperatures reached for a given layout of the repository.

Comprehensive parameter studies have been conducted to ascertain how temperature propagation varies with the rock and design parameters. An increase in the initial temperature by 7°C, which roughly corresponds to the difference in rock temperature between sites in northern and southern Sweden, would correspond to an increase in the size of the deposition area by 15–20%.

Temperature and its distribution are fundamental parameters of state in a deep repository and influence both the mechanical, chemical and biological environment and the groundwater flow through the repository.

Thermal conductivity is controlled by the mineral composition of the rock, mainly by its quartz content. Assuming isotropic and homogeneous conditions, the thermal conductivity of the rock can be calculated directly from its mineral composition. Table 4-17 shows calculated mean values of thermal conductivity for some of the most commonly occurring types of crystalline rock in Sweden.

4.7.1 Aberg

A mean temperature of 14.6°C at a depth of 500 m and a mean temperature gradient of 15.0°C/km have been measured in Äspö, see Figure 4-21. Collected data have, however, been reported to be slightly influenced by the groundwater flow in the boreholes /Ahlbom et al, 1995/.

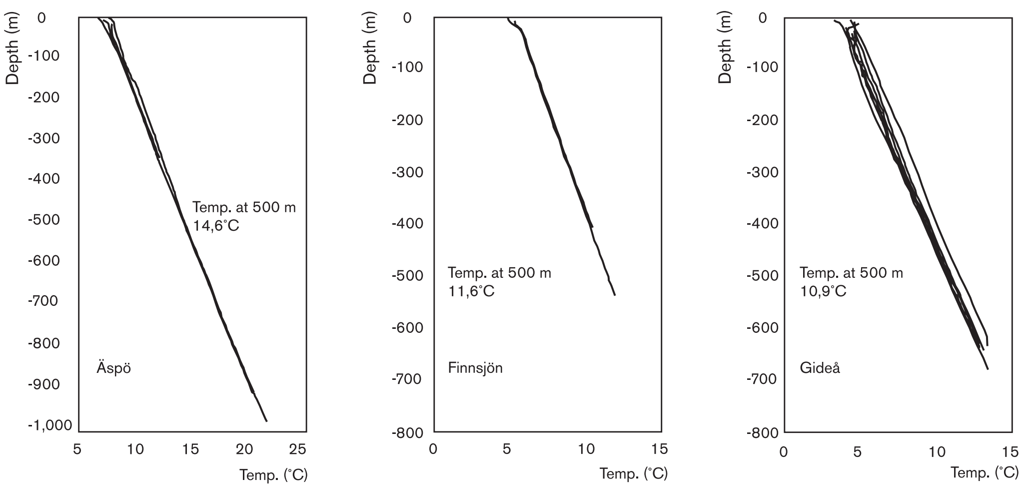


***Figure 4-21**. Geothermal gradient in Äspö, Finnsjön and Gideå /after Ahlbom et al, 1995/.*

The mean values of thermal conductivity for approximately 100 different rock samples from the site are presented in Table 4-18. The values are based on mineral properties calculations, and have not been measured from rock types samples. Discrepancies from the mean values for rock types in Sweden that are presented in Table 4-17 can be explained by local variations.

Table 4-17. Mean value of thermal conductivity for the most commonly occurring types of crystalline rock in Sweden /Sundberg, 1995/.

Rock type	Number of observations	Thermal conductivity (W/m°C)
Granite, gneissic granite	848	3.47
Granodioroite	255	3.34
Tonalite	171	3.16
Aplite, pegmatite	44	3.31
Quartz-diorite	122	2.87
Syenite, diorite, gabbro	188	2.67
Porphyry	95	3.55
Rhyolite, dacite	119	3.37
Trachyte, andesite, basalt	70	2.83
Quartzite	32	6.62
Altered sediments: greywackes, mica schist	122	3.58
Leptite	726	3.58

Table 4-18. Mean value of thermal conductivity for rock types samples from Äspö /Sundberg, 1991/.

Rock type	Thermal conductivity (W/m°C)
Greenstones	2.58
Diorites	2.55
Quartz-monzodiorite-granodiorite	2.63
Granodiorite-granite	3.03
Granite	3.48
All rock samples from Äspö	2.96

4.7.2 Beberg

The temperature has been measured in three deep boreholes in Finnsjön. The measurements give a mean temperature of 11.6°C at a depth of 500 m. The mean temperature gradient has been calculated to be 12.7°C/km, see Figure 4-21 /Ahlbom et al, 1995/.

Local data for the thermal conductivity of the granodiorite are not reported for Finnsjön. From Tables 4-17 and 4-18, conductivity values for granodiorite in Sweden can be estimated to be 3–3.3 W/m°C. Discrepancies from the mean may be due to irregular properties in the rock mass such as foliation, dominant fracture directions and major conductive zones.

4.7.3 Ceberg

The mean temperature at a depth of 500 m in Gideå is reported by Ahlbom et al /1995/ to be 10.9°C, with a mean temperature gradient of 15.5°C/km, Figure 4-21.

The thermal properties of the rock mass in Gideå have been reported by Ahlbom et al /1991/. Gneissic granite, which is the dominant rock type, is characterized by irregular thermal properties and has a thermal conductivity that varies between 3.14 and 5.51 W/m°C. This range is somewhat high compared with the values reported by Sundberg /1991/ in Table 4-17.

4.7.4 Uncertainties in thermal descriptions

The thermal conductivity in a rock is dependent on its mineral composition. Different minerals differ significantly in thermal conductivity. Quartz, the most common mineral in granites, has a thermal conductivity that is three to four times higher than the second most common mineral, feldspar. An uncertainty factor is thus how the composition of the rock mass varies within a repository. Thermal conductivity is also affected by anisotropies in the rock mass in the form of foliations and extended, parallel rock bodies, as well as by discontinuities in the form of fracture zones and fracture systems, which are always incompletely known. Fractures oriented vertical to the flow of heat reduce the heat transport slightly, since they act as barriers to the heat flow /Sundberg, 1995/. If the fractures are water-bearing they also transport energy.

Sundberg /1995/ emphasizes that the anisotropy of the rock leads to a directionally dependent thermal conductivity. The arithmetic and harmonic mean can be used to specify an upper and lower limit for the thermal conductivity, parallel and perpendicular to the foliation, respectively.

5 Site-specific adaptation of the repository

The repository design described in Chapter 3 is general in nature, i.e. independent of detailed knowledge of the repository site. In practice, the layout of a repository always has to be adapted to the characteristics of the specific site.

The site adaptations that have been carried out for Aberg, Beberg and Ceberg are described in Chapter 5. The adaptation process concerns how the different repository sections are situated in relation to each other, how tunnels, shafts and deposition areas are positioned, and how deep down in the rock different parts of the facility are placed. Adaptation has to take into account such factors as fracture zones and zones of weakness, rock quality, groundwater flow paths and biosphere recipients for deep groundwater. Consideration must also be given to conditions on the surface, such as municipal planning, other activities in the area and infrastructure.

The possibility of siting the repository for other long-lived waste adjacent to the repository for spent nuclear fuel has been taken into account in designing the repositories in Aberg, Beberg and Ceberg.

5.1 Repository design

5.1.1 General

Design is the integrated process where the technical planning and development work is coordinated to produce concrete documentation for construction and operation. The process embraces engineering of facilities and equipment above and below ground, plus planning of the activities during the construction and operating phases. Design must be closely coordinated with research, safety assessment and siting.

The facility description is the first coordinated proposal for the layout of the deep repository and serves as the platform for the continued design work. It provides important premises for the development of system solutions for vital functions underground, such as ventilation, drainage and electricity supply, as well as for the design of machines and vehicles.

5.1.2 Detailed adaptation of repository layout

When site investigations have been commenced, the work of adapting the layout (configuration) and design (engineering) of the repository to conditions on the investigated site begins (see Figure 5-1). Construction analyses are performed to evaluate the solutions for important construction-related factors, possible building methods, resource needs, etc.

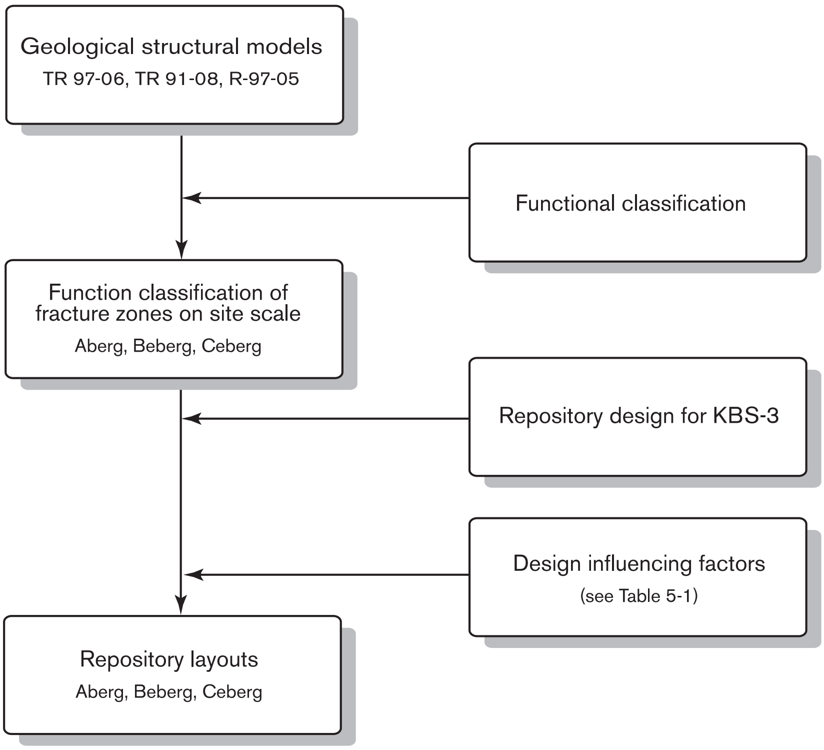


Figure 5-1. *Schematic diagram of links between data from site investigations, design and safety assessments. (Does not show any feedback, but how it was done in SR 97, see further in section 5.3).*

The design work becomes increasingly detailed as data become available from the site investigations. In an initial phase, the locations and layout of the surface facility and underground repository are adapted in rough terms to the characteristics of the site. The manner in which the surface and underground portions are to be connected (shaft or ramp) is also determined. Layout drawings of surface facilities, tunnel systems and access ramps/shafts are prepared. In a second phase, drawings and other documents are produced describing both the layout and function of individual facility parts and how the parts are tied together into a whole. A preliminary design of the deep repository is included in the supporting documentation for an application for a permit for detailed characterization and construction.

5.2 Factors that influence layout

5.2.1 General

A number of site-specific factors influence the layout of the deep repository and its different parts. The factors that are deemed to be of the greatest importance for repository layout underground are rock stresses and the direction and properties of individual fractures and water-bearing zones. Some of these factors interact, while others are independent.

5.2.2 Functional classification of discontinuities

Based on the geoscientific characterization, alternative layouts of the repository are analyzed and evaluated with respect to construction aspects and operation, as well as long-term performance and safety. A functional description is prepared of the rock and the repository, whereby the rock and its structures are described and classified. The classification of structures is revised as the available geoscientific information is updated and changed.

Discontinuity refers to a mechanically deviating structure in the rock mass, e.g. a fracture or schistose plane or zone – most often with the main length being two dimensional – with a lower strength than the surrounding rock. This term is used throughout to differentiate between structures classed by function and those described through identification or through their own specific geological characteristics.

The discontinuities are classified from a functional viewpoint /Almén et al, 1996/ as follows:

D1: Discontinuities with such properties that they may not occur in the repository volume and may only be passed by an access tunnel in exceptional cases. Discontinuities of functional class D1 comprise large-scale mechanical and/or hydraulic boundaries for the deep repository. Discontinuities of functional class D1 often manifest themselves as lineaments many kilometres in length.

D2: Discontinuities with such properties that they may not be passed by deposition tunnels. Discontinuities of functional class D2 may, however, serve as boundaries between different main sections of the repository or deposition areas. Discontinuities of type D2 have properties similar to those of D1 but are in general of a more local and less intensive character.

D3: Discontinuities with such properties that they are permitted to occur within repository sections and are passed by both transport tunnels and deposition tunnels. Discontinuities of type D3 usually consist of minor fracture zones, but individual fractures, for example highly conductive fractures of great length, may also exhibit properties that warrant their being assigned to this class.

D4: Discontinuities with such properties that they are permitted to intersect deposition positions and thus do not affect the degree of utilization of the repository. Discontinuities of functional class D4 normally consist of single fractures or schistose planes whose principal directions may nevertheless influence the directions of the repository tunnels. Discontinuities of functional class D4 may also be of importance for the function of the near field with respect to groundwater flow and nuclide transport; they may also be of importance for the rock-mechanical stability of the canister hole.

Fractures vary in age, size, direction, density, extent and hydraulic conductivity as well as mechanical properties. A rock mass usually contains several fracture sets.

Water-bearing fractures of functional class D3 influence how the rock can be utilized, since canister positions intersected by them cannot be used. Such discontinuities are site-dependent, and their influence on the repository layout can therefore vary both between different repository sites and between different blocks within the same area.

The deposition tunnels should be oriented in such a manner that their intersection area with conductive fractures is minimized, i.e. so that the fractures are intersected at as obtuse an angle as permitted by the shape of the rock blocks.

5.2.3 Respect distance

An influence area around a discontinuity may be defined as the volume of the rock mass on a given scale beyond which a given influence is considered improbable. A respect distance can therefore be defined as the distance from an interpreted discontinuity that is required to ensure that requirements on long-term safety for a canister position are met.

Coupling the respect distances to defined functional classes makes it possible to delimit and judge the size of the blocks in a given area for positioning of the different repository sections at an early stage in the design process.

5.2.4 Geometry of rock blocks

Fracture zones in the structural models govern the shape and size of the rock blocks. Blocks that can accommodate tunnels with lengths in the span 250 to 500 m are considered advantageous from a construction-related point of view.

5.2.5 Rock stresses

Rock stresses mainly influence the mechanical stability of rock caverns, tunnels and deposition holes. The stresses are site-specific and can vary within a repository area in both direction and magnitude. Rock stresses are, however, normally moderate at repository depth. A basis for siting a depository at a depth of between 400–700 m is that rock stresses should be maintained at reasonable levels.

Rock stresses can affect transport tunnels, deposition tunnels and canister holes differently depending on their direction, intersection and operating methods. From a stability viewpoint, deposition tunnels should normally be oriented parallel to the maximum horizontal stress or at as acute an angle as possible to it

5.2.6 Temperature

The spent nuclear fuel in the canisters emits heat, which leads to elevated temperature in relation to surrounding rock masses. This can give rise to changes in the stress field in the rock and affect groundwater movements in the rock and the properties of the bentonite buffer. The thermal properties of the rock are taken into consideration when designing the layout of the repository. Extending the repository in a larger rock volume by increasing the distance between the canister positions limits the maximum temperature in the repository and surrounding rock.

5.2.7 Flow pattern

The hydrogeological conditions on the site will be taken into account in locating deposition tunnels, ramps and shafts. The repository's different components should be positioned so that the natural groundwater flow from the repository for other long-lived waste, which could affect the groundwater chemistry, is not allowed to affect the deposition area for spent nuclear fuel. The repository layout will therefore be adapted to the results of the analyses of groundwater movements that are performed.

5.3 Adaptation of repository at Aberg, Beberg and Ceberg

5.3.1 Premises for SR 97

The main purpose of SR 97 is to carry out a complete (in most respects) safety assessment, where the results are illustrated with real, site-specific conditions from three sites in Sweden. The repositories have been adapted realistically to each site, even though the layout has not been optimized with respect to each site. The principal purpose has instead been to create equivalent premises in order to be able to conduct site-specific consequence analyses.

The premises that apply to the repository layouts according to KBS-3 for Aberg, Beberg and Ceberg are summarized below /Munier et al, 1997/. The following in particular can be noted:

– A repository depth of around 500 m has been striven for, unless otherwise warranted by the geological conditions on the site. The repositories have a single-level layout unless otherwise warranted by the geological conditions.

– No allowance is made for the unacceptability of canister positions, i.e. no extra area is set aside so that deposition holes can be rejected. The distance (spacing) between the deposition holes has been assumed to be 40 m and between the canisters at least 6 m /Pettersson et al, 1993/. No attempts have been made to adapt the spacings to a specific site by carrying out thermal or rock-mechanical calculations as a part of the repository design procedure. Such calculations will be carried out later with a given repository layout.

– The point of departure for the repository layout is the geological-structural models for each site.

– The fracture zones have been classified. As a basis for modelling of the rock volumes, the respect distances to structures of functional classes D1 and D2 have been set at 100 and 50 metres, respectively. No respect distance has been assumed for structures of functional class D3.

– The repository for other long-lived waste is located at a distance of about 1 km from the nearest canister position in the repository for spent nuclear fuel and in such a direction that the groundwater flow is not directed towards the deposition area for canisters.

The site-specific factors that influence the layout of the deep repository and its different sections in SR 97 are summarized in Table 5-1.

Table 5-1. Factors that have influenced repository layouts for Aberg, Beberg and Ceberg /after Munier et al, 1997/.

Factor	Importance for repository layout in SR 97
1. Respect distance	Structures that have been judged to belong to functional classes D1 and D2 have been given a respect distance of 100 and 50 metres, respectively, on both sides of the centre of the interpreted zone. No respect distance has been assumed for structures of functional class D3.
2. The local stress field	A stress field with respect to direction has been assumed for each site, the magnitudes have not been taken into consideration.
3. Conductive structures, local scale	For Aberg there is evidence that conductive structures strike parallel to the maximum horizontal stress. For Beberg and Ceberg this has merely been assumed. Deposition tunnels have been oriented perpendicular (or at an obtuse angle) to the maximum horizontal stress for the main alternatives on all sites.
4. Shape of blocks	The zones in the structural model govern the shape and direction of the blocks. Blocks that can accommodate long tunnels (200–500 m) have been preferred to blocks in which only short tunnels can fit.
5. Hydraulic barriers	Major conductive discontinuities within the repository volumes have been used wherever possible to separate the repository for other long-lived waste from other repository components. The basic assumption has been that they can function as hydraulic barriers.
6. Distance to repository for other long-lived waste	The repository for other long-lived waste has been located at least 1 km from the nearest canister position so that the groundwater flow from this repository is not deemed to reach any deposition tunnel.

A number of engineering designs have been devised for each site, of which the main alternative for each site is presented in the following sections. For a comprehensive account, see /Munier et al, 1997/. The deposition tunnels are oriented perpendicular to the maximum horizontal stress. This orientation has been chosen to avoid long intersections with water-conducting fractures that have the same direction as the horizontal stress.

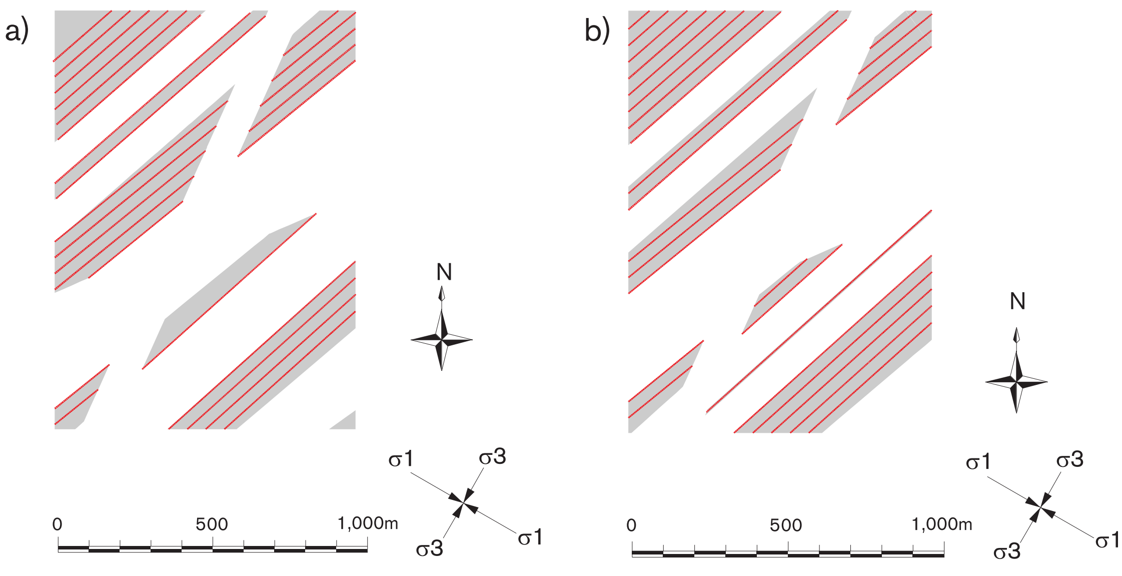


Figure 5-2. Adaptation of layout for deposition tunnels in the blocks in Aberg. The figure shows the two levels of the repository a) –500 m and b) –600m.

5.3.2 Aberg

Comprehensive investigations have been conducted in Aberg. The geological model is based not only on information from the surface investigations, but also on information from the ramp that has been driven from the ground surface down to the 450 m level. In Aberg, the proposed layout is split into two levels situated at depths of 500 m and 600 m due to limited rock volumes on one level (figure 5-2). Access is via a ramp, and the repository for other long-lived waste could, if it adjoins the deep repository, be connected to the ramp at 300 m. The deposition tunnels are oriented perpendicular to the maximum horizontal stress.

Thermal calculations carried out for the proposed repository layout in Aberg show that the minimum spacing between the canisters must be increased from 6 m to about 7.5 m if the tunnel spacing is 40 m in order to keep the temperature on the canister surface from exceeding the maximum permissible temperature /Ageskog and Jansson, 1999/.

5.3.3 Beberg

The investigation of Beberg has primarily been conducted to investigate the area as a part of SKB's study site investigations and to study the thick, flat structure (Zone 2) that has been identified in the area between –100 and –295 m. Relatively good knowledge thus exists concerning the site's regional geology and the rock mass above Zone 2. Considerably less knowledge exists concerning the rock mass below Zone 2. In Beberg, the proposed layout is situated at the –600 m level (Figure 5-3) in order to avoid the

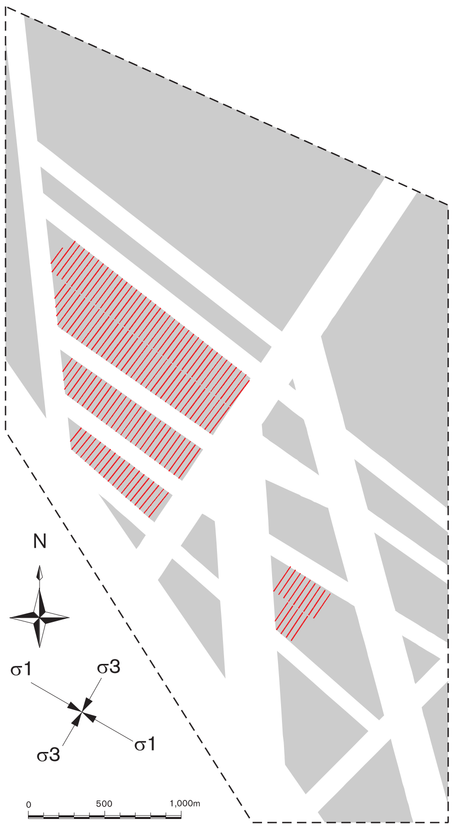


Figure 5-3. *Adaptation of layout for deposition tunnels in the blocks in Beberg at the –600 m level.*

horizontal structure with good margin. Access is via a ramp, and the repository for other long-lived waste could, if it adjoins the deep repository, be connected to the ramp at the –360 m level. The deposition tunnels are oriented perpendicular to the maximum horizontal stress. The layout coincides well with the one devised in conjunction with SKB 91 /SKB 91, 1992/.

5.3.4 Ceberg

Ceberg is the site on which the least information is available; for one thing, there are fewer boreholes than in Aberg and Beberg. In Ceberg the repository is proposed to be located at –500 m (Figure 5-4), i.e. around 600 m below the ground surface. Access is via a ramp, and the repository for other long-lived waste could, if it adjoins the deep repository, be connected to the ramp at the –375 m level. The deposition tunnels are oriented perpendicular to the maximum horizontal stress.

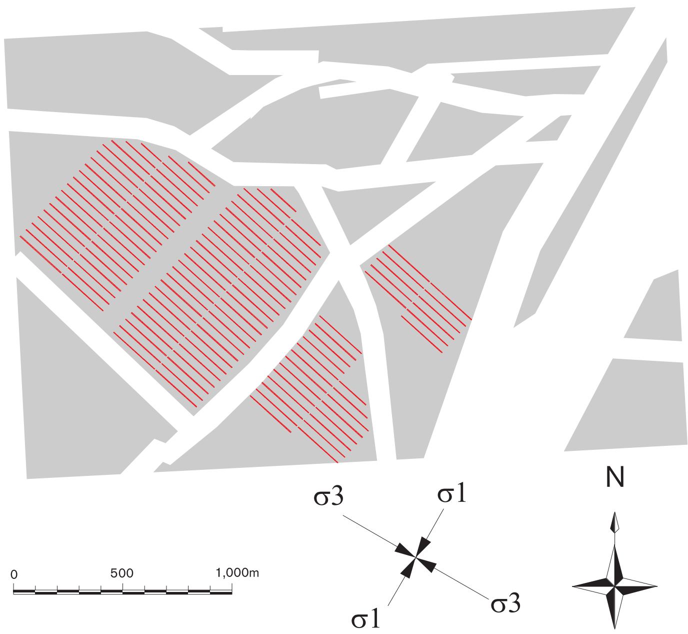


Figure 5-4. Adaptation of a layout for deposition tunnels in the blocks in Ceberg at the –500 m level.

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Abbreviations

BIOMOVS	=	Biosphere Model Validation Study
BIOMASS	=	Biosphere Modelling and Assessment
BWR	=	Boiling Water Reactor
CLAB	=	Central interim storage facility for spent nuclear fuel
EDZ	=	Excavation-Disturbed Zone
EIA	=	Environmental Impact Assessment
EIS	=	Environmental Impact Statement
EQUIP	=	Evidence from Quaternary Infills for Palaeohydrogeology
FEBEX	=	Full-scale Engineering Barriers Experiment in Crystalline Host Rock
GPS	=	Global Positioning System
HRL	=	Hard Rock Laboratory
ILW	=	Intermediate-level waste
LILW	=	Low and intermediate-level waste
LLW	=	Low-level waste
HLW	=	High-level waste
HRL	=	Hard Rock Laboratory
IAEA	=	International Atomic Energy Agency
KASAM	=	Statens råd för kärnavfallsfrågor (Swedish National Council for Nuclear Waste)
KBS	=	Kärnbränslesäkerhet = Nuclear Fuel Safety
KTH	=	Kungliga Tekniska Högskolan (Royal Institute of Technology)
MLH	=	Medium Long Holes
MSEK	=	Millions of Swedish kronor
NEA	=	Nuclear Energy Agency
NPP	=	Nuclear Power Plant
OECD	=	Organization for Economic Cooperation and Development
PAGEPA	=	PAlaeohydrogeology and GEoforecasting for Performance Assessment
PWR	=	Pressurized Water Reactor
P&T	=	Partitioning and Transmutation
RD&D	=	Research, Development and Demonstration
REX	=	Redox Experiment on detailed scale
RMR	=	Rock Mass Rating
RPV	=	Reactor Pressure Vessel
RVS	=	Rock Visualization System
SAFE	=	Safety Assessment of Final Repository for Radioactive Operational Waste
SEK	=	Swedish kronor
SFR	=	Final repository for radioactive operational waste
SGU	=	Geological Survey of Sweden
SKB	=	Statens Kärnbränslehantering AB (Swedish Nuclear Fuel and Waste Management Co)
SKI	=	Statens Kärnkraftinspektion (Swedish Nuclear Power Inspectorate)
SKN	=	Statens Kärnbränslenämnd (National Board for Spent Nuclear Fuel)
SSI	=	Statens Strålskyddsinstitut (National Radiation Protection Institute)
SFR	=	Final repository for radioactive operational waste

TRUE	= Tracer Retention Understanding Experiments
TBM	= Tunnel Boring Machine
TDS	= Total Dissolved Solids
VDH	= Very Deep Holes
VLH	= Very Long Holes
VSP	= Vertical Seismic Profiling
ZEDEX	= Zone of Excavation Disturbance Experiment

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