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Excavation damage and disturbance in crystalline rock – results from experiments and analyses

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November 2008

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This report concerns a study which was conducted for SKB. The conclusions and viewpoints presented in the report are those of the author and do not necessarily coincide with those of the client.

Preface

Despite extensive desk studies, laboratory experiments and field tests, there is still a lack of qualified data on the hydraulic effects due to excavation damage and disturbance around the excavation openings in the repository. Such data are important for the execution of safety assessments in Sweden in support of the application to site and construct a final repository for spent nuclear fuel.

The zone of excavation damage and excavation disturbance has been studied by all major nuclear waste management organisations worldwide, and in view of SKB's need for data to support the license application to construct the final repository for spent nuclear fuel, the time was now ripe to summarise the present knowledge base with regard to:

- the conceptual understanding of the excavation damage and disturbance;
- the safety implications of the excavation damage and disturbance;
- methods to infer or measure the pertinent properties in situ.

This report presents the results of a literature study, a workshop organised by SKB and work conducted by the author to establish a process description and a preliminary "control programme" to verify the assumed properties of excavation damage and disturbance during repository construction.

The work has substantially benefitted from the willingness of many colleagues to share their knowledge and experience beyond written contributions, and their individual assistance is hereby gratefully acknowledged.

The project was initiated by the SKB Spent Fuel Project and supervised by a steering committee chaired by Mr. Christer Svemar, SKB. The final draft of the report was reviewed by experts from Canada and Sweden who have been involved in relevant experiments and analyses concerning excavation effects.

Summary

SKB plans to submit the application to site and construct the final repository for spent nuclear fuel in 2010. It is estimated that approximately 12,000 tonnes of spent nuclear fuel will be generated by the Swedish nuclear power programme, which corresponds to roughly 6,000 canisters in the repository. One important basis for the application is the results of the safety assessments, for which one particular dataset is the axial hydraulic properties along the underground openings used to calculate the transport resistance for radionuclide transport in the event that the canister is impaired.

SKB initiated a project (Zuse) to be run over the period 2007–2009 to:

- establish the current knowledge base on excavation damage and disturbance with particular focus on the axial hydraulic properties along the underground openings;
- provide a basis for the requirements and compliance criteria for the excavation damaged and disturbed zone;
- devise methods and instruments to infer or measure the excavation damage and disturbance at different times during the repository construction and operation before closure;
- propose demonstration tests for which the methods are used in situ to qualify appropriate data for use in the safety reports.

This report presents the results of the first stage of the Zuse project. Previous major experiments and studies in Canada, Finland, Japan, Sweden and Switzerland on spalling, excavation damage and disturbance was compiled and evaluated to provide the SR-Site report[1] with a defendable database on the properties for the excavation damage and disturbance.

The previous SR-Can safety report[2] shows that the enhanced axial transmissivity along the deposition tunnels, for the conditions assumed, will have limited importance in comparison to other transport routes for radionuclides, even for very pessimistic assumptions about the EDZ. In preparation for the SR-Site report, a number of sensitivity studies were conducted in which reasonable ranges of values for spalling and damage were selected in combination with an impaired backfill. These calculations also showed that enhanced axial transmissivity along the deposition holes and deposition tunnels will have a very limited impact on the transport parts and performance measures selected.

The report here describes the construction of the repository in eleven steps and for each of these steps, the potential evolution of THMCB (Thermal, Mechanical, Hydraulic and Chemical/Biological) processes are reviewed.

In this work it was found that descriptions of the chemical and microbiological evolution connected with excavation damage and disturbance was lacking. A preliminary study was then carried out to find out whether results from underground tests could possibly be biased due to fracture clogging or dissolution. In the models, representative groundwaters at the Laxemar and Forsmark sites were assumed and fracture precipitation/dissolution studied for a number of pressure and temperature conditions. The preliminary calculations using the PHREEQC code showed that fracture precipitation is unlikely to clog the water-conducting fractures within a 10-year period, even when considering construction material such as cement and nitrogen compounds. For most of the modelling cases, the estimated decrease in hydraulic conductivity over a period of ten years is less than 2% due to mineral precipitation. As a general trend, the largest hydraulic conductivity variations (largest mineral precipitation amounts) correspond to the mixtures of groundwaters with the shallow infiltration waters.

[1] The SR-Site report provides the basis for demonstration of long-term safety of the repository for spent nuclear fuel. The report is included in SKB's application for siting and constructing the repository for spent nuclear fuel.

[2] SKB has already prepared a safety report SR-Can to make a first assessment of the safety of potential KBS-3 repositories at the two candidate sites Forsmark and Laxemar, to provide feedback to the programme and to foster a dialogue with the authorities.

It was further concluded that it is more likely for fractures to clog (self-heal) than become open as a result of fracture mineral dissolution.

With respect to the microbial evolution it was concluded that the potential for microbial iron hydroxide production will be large in all groundwater with ferrous iron. The potential for microbial calcite formation production will be large in all groundwater with high concentrations of Dissolved Organic Carbon (DOC) and methane, irrespective of oxygen, E_h (redox) or pH. The kinetics of the microbial evolution is not yet known, and microbes may then play a role in fracture clogging which could possibly be the factor that contributes to lower groundwater inflow to underground facilities with time, as shown for many facilities, for instance the Swedish final repository for medium level waste (SFR) and the Äspö Hard Rock Laboratory.

The compilation of results from the relevant tests at underground laboratories in Canada, Japan, Sweden and Switzerland was focussed on the extent of the damage and its dependency on excavation methods and measurements of the hydraulic properties.

The most important factor controlling the excavation damage is the choice of excavation method. Use of mechanical excavation may create irreversible damage less than 30 mm from the rock wall where increased micro-fracturing contributes to an increase in hydraulic conductivity. Several methods and sample scales have been used to characterise the damage zone. For a Tunnel Boring Machine < 5 mm of damage was recorded at Äspö HRL and at Grimsel in Switzerland. The hydraulic conductivity was at e.g. Äspö HRL determined to be in the range of 10^{-9} m/s over a distance of 1–2 mm from the tunnel periphery, $2 \cdot 10^{-11}$ m/s at a depth of 5 mm, and 10^{-13} m/s at a distance of 30 mm from the tunnel wall. The virgin crystal matrix of the rock is in the range of 10^{-13} m/s to 10^{-14} m/s.

Excavation of the deposition tunnel using drill and blast would create much more widely dispersed damage (several tens of centimetres) than using a TBM (a few centimetres). The extent of the damage caused by using drill and blast is very dependent on the accuracy of drilling for the blast holes, the amount of explosives used in the blast holes close to the periphery and the local geological conditions. Several methods have been used to estimate the hydraulic conductivity of the damage zone, and the values depend somewhat on the method used. Based on this study it is suggested that the hydraulic conductivity of the damage zone should be an absolute value and not presented in relation to the undisturbed hydraulic conductivity. A reasonable value for the hydraulic conductivity of the damage zone is 10^{-8} m/s. This magnitude has been obtained during several tests in crystalline rocks, where excavation was of good quality and measured by integrating measurement under saturated conditions along the tunnel floor. Point observations of the hydraulic conductivity have provided both lower and higher individual results. This is due both to the natural variability of the rock properties as well as to the fact that damage is correlated to the amount of explosives, which varies along the periphery of the opening and also along the longitudinal section of the tunnel.

A matter of discussion is still whether the EDZ is continuous or discontinuous over adjacent blast rounds, and the results are inconclusive based on the data from experiments at AECL's Underground Research Laboratory and Äspö Hard Rock Laboratory (HRL).

The compilation in this report shows that spalling is the most important factor that will contribute to an extended axial transmissivity along a deposition tunnel. Measured hydraulic conductivity based on spalling in a test tunnel in crystalline rock at the AECL URL in Canada was in the order of 10^{-6} m/s and significantly higher than the increased hydraulic conductivity due to the damage caused by the excavation process.

Much experience has been collected on useful and feasible methods to characterise spalling, excavation damage and disturbance, and many studies are still in progress. The findings so far have been used to outline a "control programme" to verify that the established requirements are complied with during repository construction. It is suggested that three factors should be considered, namely the excavation process itself, the rock response to the excavation and finally the evolution of the rock response with respect to time, where e.g. the heating and cooling are processes to consider. The control programme suggested here covers the period up to the start of backfilling.

The reference design for the deposition tunnels is to excavate them by drill and blast in two steps. The first step is to blast the top heading and the second is to slash the bottom bench; both steps are executed using smooth blasting. Wire-sawing is an alternative to drilling and blasting of the bench. In this context, the transmissivity of the EDZ can be tested at the positions of the deposition holes. The planned, nominal distance between deposition holes (around 6 m at Forsmark and 8–10.5 m at Laxemar) will not coincide with the length of the drill and blast round. Therefore, over the length of the deposition tunnel, a number of EDZ tests will be performed at the start, in the middle and at the end of the round. It is also more likely that the saturated conditions will prevail in the floor rather than in the roof and walls, which will simplify test evaluation. Typical tests suggested within the area of the deposition holes are e.g.:

- ultrasonic measurements;
- testing of hydraulic transmissivity by multi-packers;
- occasional laboratory tests on rock cores.

It is further assumed that site-specific relations are created between damage and density of explosive charge as the basic parameter for estimation of damage extent and properties. Such relations need to be established for different rock types and fracturing, etc. To measure the connectivity of the excavation damaged zone, such tests can tentatively be made using ground penetrating radar. After backfilling a deposition tunnel, micro-seismics from neighbouring tunnels that have not yet been backfilled can be used to track the micro-seismic evolution after closure of the deposition tunnel. It is concluded that a proper “control programme” could be established, but that additional work is necessary to calibrate measurement methods and to verify the reliability of such a programme.

Sammanfattning

SKB planerar att under 2010 lämna in en ansökan om lokalisering och anläggning av slutförvaret för använt kärnbränsle. Det är beräknat att omkring 12 000 ton av använt kärnbränsle kommer att genereras i det svenska kärnkraftsprogrammet, motsvarande cirka 6 000 kapslar i förvaret. En viktig grund för ansökan, är resultaten från säkerhetsredovisningar, där en uppsättning data rörande de axiella hydrauliska egenskaperna längs med undermarksutrymmena används för att beräkna transportmotståndet för radionuklider, i det fall att kapseln är skadad.

SKB skapade ett projekt (Zuse) att löpa under perioden 2007–2009 för att:

- upprätta den nuvarande kunskapsbasen om skador och störningar från berguttag med särskild inriktning mot de axiella hydrauliska egenskaperna längs undermarksutrymmen;
- skapa en grund för krav och acceptansvillkor med avseende på den skadade och störda zonen;
- ange metoder och instrument för att anta eller mäta skadad och störd zon vid olika tidpunkter under förvarsutbyggnad och drift före förslutning;
- föreslå demonstrationsförsök, där metoder används i fält för att uppfyller krav på relevanta data för användning i säkerhetsredovisningarna.

Denna rapport redovisar resultaten från Zuse-projektets första steg. Tidigare större försök och utredningar i Finland, Japan, Kanada, Schweiz och Sverige rörande spjälkning, skadad och störd zon har sammanställts och utvärderats för att ge en försvarbar databas på skadad och störd zon till SR-Site rapporten[3]. Den tidigare säkerhetsrapporten SR-Can[4] påvisar att högre axiell transmissivitet längs deponerinstunnlar, för gjorda antaganden, har en begränsad betydelse i förhållande till andra flödesvägar för radionuklider, äver för mycket pessimistiska antagande för den skadade zonen. Som förberedelse för SR-Site rapporten, har ett antal känslighetsanalyser genomförts, där rimliga intervall av värden använts för spjälkning, skadad zon i kombination med en försämrad återfyll. Även dessa beräkningar visar att förhöjd transmissivitet längs deponeringshål och deponeringstunnel har en mycket begränsad påverkan för de flödesvägar och prestandamått som använts.

Denna rapport beskriver förvarsutbyggnaden i elva steg. För vart och ett av dessa steg, diskuteras den möjliga utvecklingen av THMKB (termiska, mekaniska, hydrauliska, kemiska/biologiska) processer.

I detta arbete blev det klart att det saknades beskrivningar av den kemiska och mikrobiologiska utvecklingen med avseende på den skadade och störda zonen. En inledande utredning genomfördes därför för att bedöma om tolkning av resultat från undermarksförsök kunde påverkas av sprickutfällningar eller upplösning. I modellerna antogs representativa grundvatten för Forsmark och Laxemar. Sprickutfällning/-upplösning studerades för ett antal tryck- och temperaturförhållande. De preliminära beräkningarna med PHREEQC-koden visade att det inte är troligt att sprickutfällning kommer att täta vattenförande sprickor under en 10-årsperiod, även med hänsyn till byggnadsmaterial som cement och kväveföreningar. För de flesta beräkningsfallen, är den minskade hydrauliska konduktiviteten mindre än 2 % med hänsyn till mineralutfällningar. Som en allmän trend, så hänförs den största förändringen av den hydrauliska konduktiviteten (störst andel mineralutfällning) till blandning av grundvatten med ytligt infiltrerande vatten. En ytterligare slutsats som dragits är att det är troligare att sprickor sätts igenom (självläkning) än att de öppnas som ett resultat av upplösning av sprickmineral.

[3] SR-Site rapporten är en grund för att påvisa att slutförvaret av använt kärnbränsle är säkert över långa tidsrymder. Rapporten ingår i SKB:s ansökan för att lokalisera och bygga slutförvaret av använt kärnbränsle.

[4] SKB har tidigare tagit fram en säkerhetsrapport SR-Can för att göra en första utvärdering av säkerheten för ett tänkt KBS-3 förvar vid de två kandidatplatserna Forsmark och Laxemar, att ge återkoppling till programmet och för att uppmuntra till dialog med myndigheterna.

Med hänsyn till den mikrobiologiska utvecklingen drogs slutsatsen att möjligheten till produktion av mikrobiologisk järnhydroxid är stor i alla grundvatten med löst tvåvärt järn (Fe^{2+}). Möjligheten till mikrobiologiskt bildad kalcit är stor i alla grundvatten med hög koncentration av löst organiskt kol och metan, oberoende av halt av syre, E_h (redox) eller pH. Den mikrobiologiska utvecklingens kinetik är ännu okänd och mikrober kan tänkas spela en roll för spricktätning, som möjligtvis kan vara en faktor som begränsar grundvatteninflöde till undermarksanläggningar över tiden, som vid slutförvaret för medelaktivt avfall (SFR) eller vid Äspölaboratoriet.

Sammanställningen av resultat från relevanta försök vid underjordslaboratorier i Japan, Kanada, Schweiz och Sverige var inriktad mot skadezonens utbredning och dess beroende av uttagsmetod samt mätningar rörande hydrauliska egenskaper.

Den viktigaste faktorn som kontrollerar skadezonen är val av uttagsmetod. Mekaniska metoder kan ge upphov till irreversibla skador upp till 30 mm från bergväggen, där ökad mikrosprickbildning bidrar till en ökning av den hydrauliska konduktiviteten. Ett flertal metoder och testskalor har använts för att beskriva skadezonen. Vid Äspölaboratoriet och Grimsel i Schweiz mättes skadan till mindre än 5 mm vid användande av tunnelborrningsmaskin (TBM). Den hydrauliska konduktiviteten som uppmättes vid Äspölaboratoriet, bestämdes att vara i storleksordningen 10^{-9} m/s för ett avstånd av 1–2 mm från tunnelperiferin, $2 \cdot 10^{-11}$ m/s för 5 mm djup och 10^{-13} m/s vid 30 mm avstånd från tunnelväggen. Den ostörda konduktiviteten av bergets matrix är i storleksordningen 10^{-13} m/s till 10^{-14} m/s.

Uttag av deponeringstunneln med borrning och sprängning ger en större skada (tiotals centimetrar) än med en TBM (några centimetrar). Skadezonens djup med borrning och sprängning är starkt beroende av borrningsprecision för spränghålen, mängden sprängämnen i borrhålen närmast kontur och lokala geologiska förhållanden. Ett flertal metoder har använts för att uppskatta den hydrauliska konduktiviteten i skadezonen och värdena är delvis beroende av den metod som använts. På basis av denna utredning föreslås att skadezonens hydrauliska konduktivitet ska vara ett absolutvärde och inte presenteras som en relation i förhållande till det ostörda bergets hydrauliska konduktivitet. Ett realistiskt värde på den hydrauliska konduktiviteten i skadezonen är 10^{-8} m/s. Denna storleksordning har erhållits i flera försök i kristallint berg, där berguttaget skett med god kvalitet och integrerad mätning skett under mättade förhållanden längs med tunnelbotten. Punktobservationer av den hydrauliska konduktiviteten har gett både lägre och högre värden, vilket dels beror på den naturliga variationen av bergegenskaper samt att skadan är korrelerad till sprängämnesmängden, som både varierar över bergutrymmets kontur liksom längs med tunneln.

Det är en fortfarande en fråga för diskussion, om skadezonen är sammanhängande eller inte över angränsande sprängsalvor och resultaten från AECL:s underjordiska forskningslaboratorium (URL) och Äspölaboratoriet är inte entydiga.

Sammanställningen i denna rapport visar att spjälkning (spalling) är den viktigaste faktorn för ökad axiell transmissivitet längs en deponeringstunnel. Den uppmätta hydrauliska konduktiviteten, baserat på spjälkning i kristallint berg vid AECL:s URL i Kanada, var i storleksordningen 10^{-6} m/s och avsevärt högre än den ökade hydrauliska konduktiviteten orsakad av de skador som skapats av sprängningen.

Stor erfarenheter har samlats in rörande användbara och genomförbara metoder att karakterisera spjälkning, skadezon och störd zon och flera studier är ännu pågående. Resultaten så långt, användes för att ta fram ett första utkast på ett kontrollprogram för att verifiera att man uppfyller de ställda kraven under förvarsutbyggnad. Det föreslås att tre faktorer hanteras, nämligen byggprocessen, bergets respons på berguttaget och slutligen bergets respons över tiden, där bland annat uppvärmning och avsvalning ska beaktas. Det föreslagna kontrollprogrammet täcker tiden fram till start av återfyllning.

Referensutformningen är att deponeringstunnlarna tas ut med borrning och sprängning i två steg. Första steget är att spränga tunnelns övre del och det andra att strossa ut tunnelbotten; båda stegen sker med skonsam sprängning. Linsågning är ett alternativ att ta ut tunnelbotten. I detta sammanhang kan den skadade zonens transmissivitet bestämmas i lägen för deponeringshålen. Det planerade, nominella avståndet mellan deponeringshålen (omkring 6 m i Forsmark och 8–10,5 m i Laxemar) sammanfaller inte med längden på sprängsalvorna, vilket medför att vissa av testerna sker i början, vissa i mitten och andra i slutet av sprängsalvorna. Det är också mer troligt att mättade förhållande är rådande i tunnelbotten, jämfört med väggar och tak, vilket underlättar försöksutvärderingen. Typiska tester som föreslås inom områden för deponeringsområden är t ex:

- ultraljudsmätningar;
- mätning av hydraulisk transmissivitet med multi-packer;
- enstaka laboratoriemätningar på borrkärnor.

Det antas vidare att de platsspecifika relationerna mellan skadezon och laddningsdensitet är den grundläggande parametern för att skatta utbredningen av skadezonen och dess egenskaper. Sådana relationer behöver upprättas för olika bergarter och sprickighet etc. För att mäta skadezonens konnektivitet, kan preliminärt markradar användas. Efter återfyllning av en deponeringstunnel, kan man använda mikroseismik i angränsande tunnlar för att följa utvecklingen i den återfyllda deponeringstunneln. Slutsatsen dras att ett passande kontrollprogram kan upprättas, men att ytterligare arbete är nödvändigt för att kalibrera mätmetoder och för att verifiera tillförlitligheten i ett sådant program.

Contents

1 Introduction

1.1 The KBS-3 method

The plan to construct a geological repository for spent nuclear fuel in Sweden has reached the final phase of site investigations at the two candidate sites at Forsmark and Laxemar. A general description of the overall programme to implement the final repository in Sweden is found in the latest Research, Development and Demonstration Programme /SKB 2007/. An outline description of the geological repository is found at the website of the Swedish Nuclear Fuel and Waste Management Co (SKB), www.skb.se. Basic engineering of the repository for the KBS-3 method is being developed in parallel with the site investigations with the overall objective that the repository should be a safe and effective facility which fully complies with international guidelines and standards, national regulations and the general design requirements for the facility /SKB 2002/. The KBS-3 method (see Figure 1-1) is based on the multi-barrier principle and geological emplacement, and has been developed by SKB as a basis for planning for the final disposal of spent nuclear fuel.

The principle of the KBS-3 method is that the spent nuclear fuel is encapsulated in a copper canister with a cast-iron insert. The canister is placed in a deep repository constructed in crystalline host bedrock about 500 metres below surface. The canister is surrounded by highly compacted bentonite clay and the tunnel system is backfilled with bentonite clay. Vertical emplacement has been SKB's reference design for the last 30 years, see Figure 1-1.

SKB plans to submit the application to site and construct the final repository in 2010. It is estimated that approximately 12,000 tonnes of spent nuclear fuel will be generated by the Swedish nuclear power programme, which corresponds to some 6,000 canisters in the repository.

The evolution of the natural system and the effects caused by the excavation of underground openings are discussed in SR-Can /SKB 2006a/.

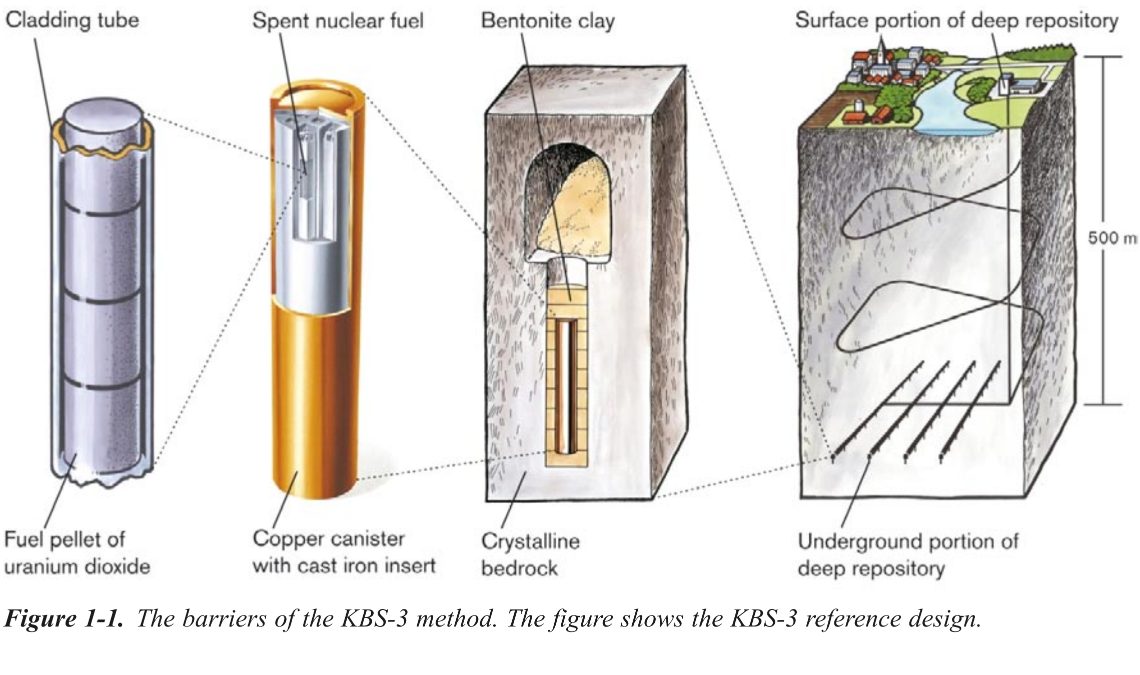

***Figure 1-1.** The barriers of the KBS-3 method. The figure shows the KBS-3 reference design.*

1.2 The Zuse project

SKB initiated a project (Zuse) to be run over the period 2007–2009 to:

- establish the current knowledge base on excavation damage and disturbance with particular focus on the axial hydraulic properties along the underground openings;
- provide a basis for the requirements and compliance criteria for the excavation damaged and disturbed zone;
- devise methods and instruments to infer or measure the excavation damage and disturbance at different times during the repository construction and operation before closure;
- propose demonstration tests in which the methods are used in situ to qualify appropriate data for use in the safety reports.

The Zuse Project is split into a set of work packages. The objectives of Work Package 1, which are reported here, were: to find consensus on a defendable data base on excavation damage and disturbance to be used in the SR-Site report and deliver the data and reasoning in the spring of 2008; to make a preliminary evaluation of the useful methods and instruments to describe the excavation damage and disturbance in situ based on previous work in Canada, Finland, Japan, Sweden and Switzerland; and to develop a plan to find methods and instruments to characterise the axial hydraulic properties along the underground openings and otherwise assist in increasing the understanding of the phenomena.

A main activity in the first Work Package, was to compile the results and experience from previous studies and experiments on the excavation damage and disturbance. The results of the literature study were discussed and reviewed in connection with a workshop and the comments received have been accounted for. The work also comprised a preliminary review of previously used methods and instruments as well as initial work to plan for future experiments. In the spring of 2008 it was also decided that the report should outline a preliminary "control programme", i.e. to describe how the excavation damage zone would be characterised during repository construction up to the backfilling of a deposition tunnel in the KBS-3V reference design.

1.3 Definitions

The review conducted has been restricted to crystalline rock studies and several definitions exist for the zones around the opening. The paper by /Bäckblom and Martin 1999/ employed the definitions used by Atomic Energy Canada Limited (AECL), namely to distinguish a failed zone, a damaged zone and a disturbed zone. In practice there are no distinct boundaries, but a gradual transition from the damaged to the disturbed zone and from the disturbed zone to the virgin rock mass. The extent and shape of these zones is mainly dependent on excavation method, shape of opening, size of the opening, rock mass strength, in situ stresses as well as the thermal load from the encapsulated spent nuclear fuel.

In /Bäckblom et al. 2004/ the damaged zone was defined as "*the part of the rock mass closest to the underground opening that has suffered irreversible deformation where shearing of existing fractures as well as propagation or development of new fractures has occurred*".

/Tsang and Bernier 2005/ defined the EdZ and EDZ in relation to the importance to the safety assessment. In this context, EdZ is the *Excavation Disturbed Zone with hydromechanical and geochemical modifications, without major changes in flow and transport properties*, whereas the EDZ, the Excavation Damaged Zone is a *zone with hydromechanical and geochemical modifications including significant changes in flow and transport properties. These changes can, for example, include one or more orders of magnitude of increase in flow permeability.*

Some authors define EDZ as the Excavation Disturbed Zone, some use EDZ to denote the Excavation Damaged Zone and others use damage and disturbance as synonyms.

The nomenclature used for this report is shown below and illustrated in Figure 1-2.

- **Damaged zone** is a zone closest to the underground opening that has suffered irreversible deformation and in which shearing of existing fractures as well as propagation or development of new fractures has occurred. Spalling, with blocks/slabs detached completely from the rock mass, will only occur in high-stress situations, whereas damage and disturbance will always occur due to creation of the underground opening.
- **Disturbed zone** is a zone dominated by change of state (e.g. stress, hydraulic head). The changes in rock mass properties are insignificant or reversible.

The members of the international modelling exercise DECOVALEX[5] have been elaborating on modelling of the EDZ /Hudson et al. 2008/ and have in this context prepared a graph, Figure 1-3, in which certain governing factors and suitable methods for characterisation are outlined. Several of these factors and methods are further explored in this report.

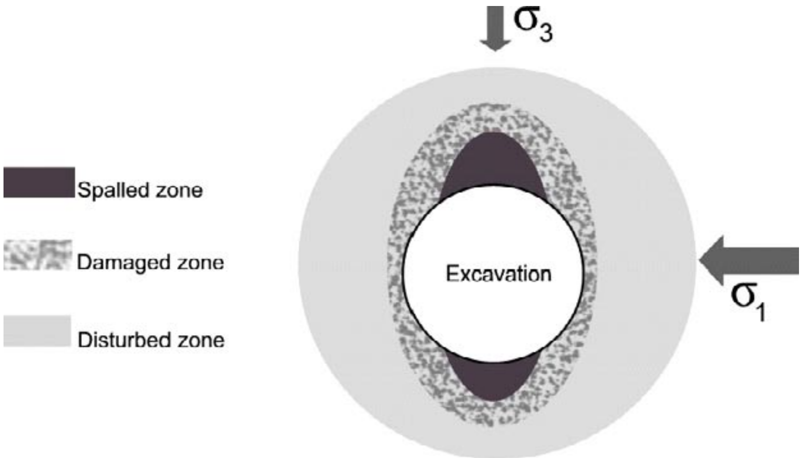

Figure 1-2. *Sketch of the Damaged and Disturbed Zone around an underground opening in a virgin stress field where the maximum principal stress is horizontal and the minimum is vertical. Spalling will only occur in high-stress situations, whereas damage and disturbance will always occur due to creation of the underground opening. Revised after /Bäckblom and Martin 1999/.*

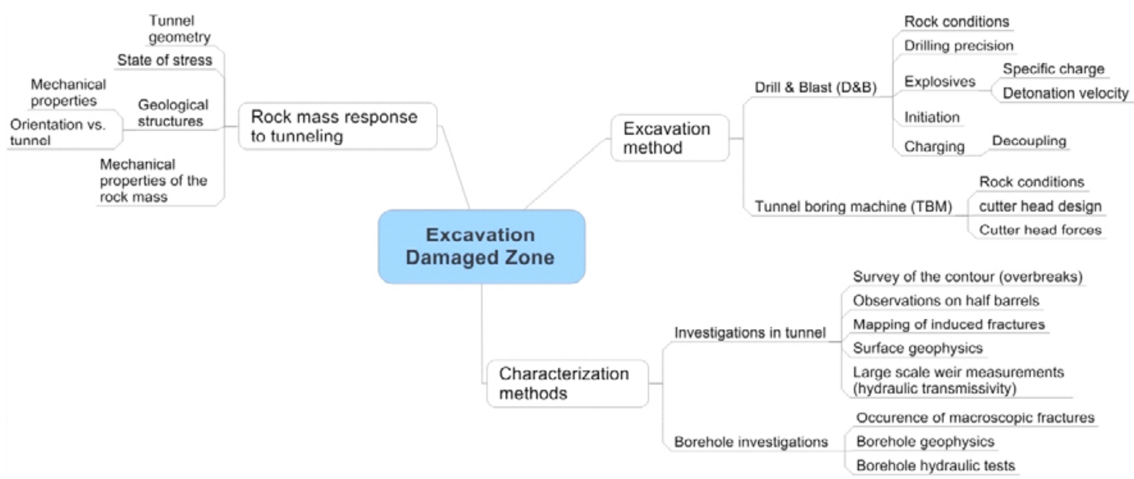

Figure 1-3. *Overview of rock mass response to tunnelling, its dependence on excavation method and characterisation methods for the EDZ. /Hudson et al. 2008/.*

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[5] (DEmonstration of COupled models and their VALidation against EXperiment: Research funded by an international consortium of radioactive waste regulators and implementers.

2 Development of the KBS-3V repository and preliminary evaluation of the implications of excavation damage and disturbance

As shown in Figure 2-1, the spent fuel will be deposited at a depth of approximately 500 m. The deposition tunnels are linked to tunnels for transport and communication. One ramp and several shafts connect the surface facility to the underground repository. The ramp is used for transporting the shielded canister down into the repository and for the heavy and bulky transports. The shafts are for ventilation, for utility systems and for the transport of excavated rock, backfill and staff. The different parts of the final repository are sketched in Figure 2-1 with some details in Figure 2-2.

In the following, the stepwise development of the repository is outlined as the development sequence is later used to describe the potential evolution of the excavation response over time (see Chapter 4).

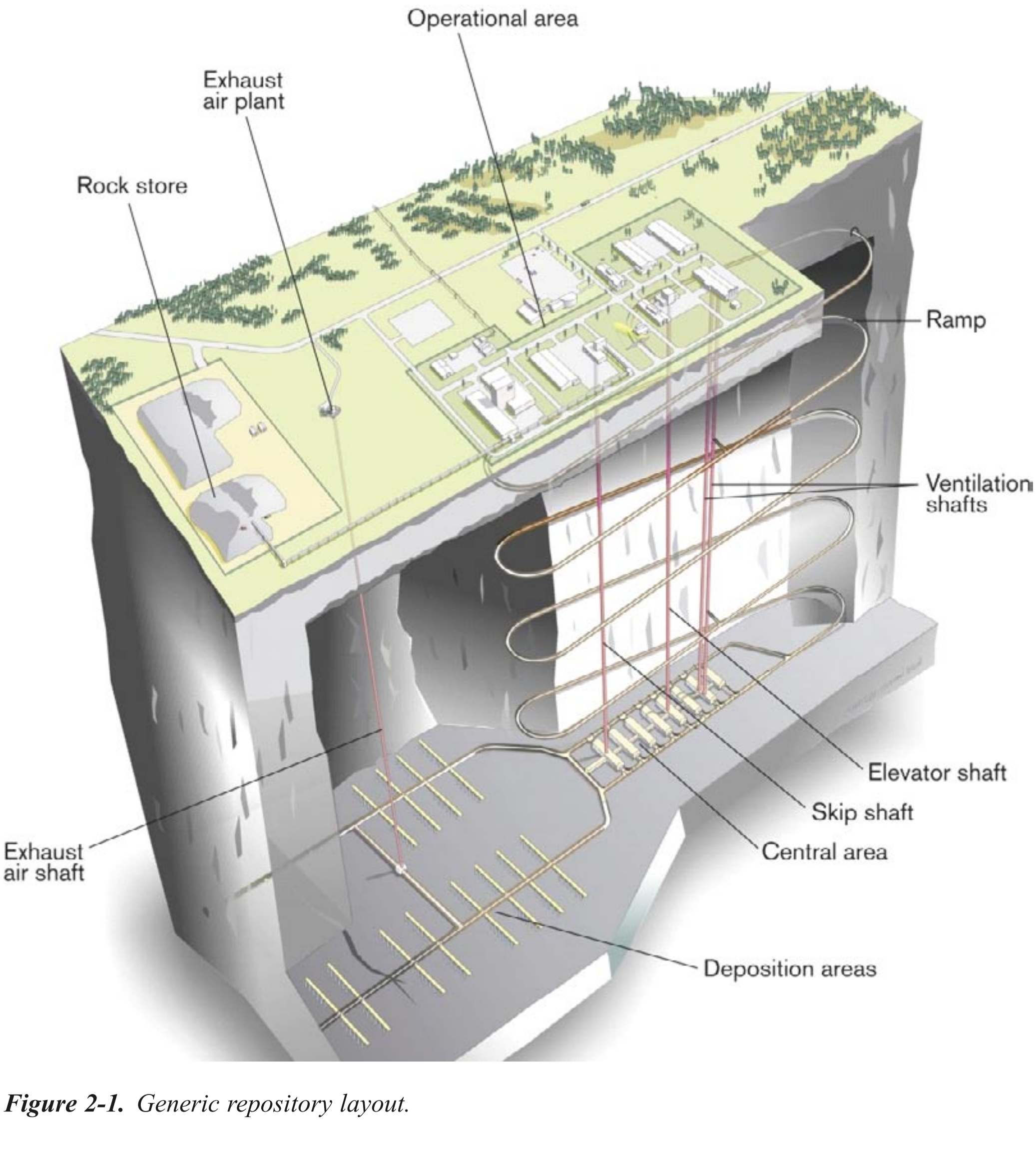

Figure 2-1. *Generic repository layout.*

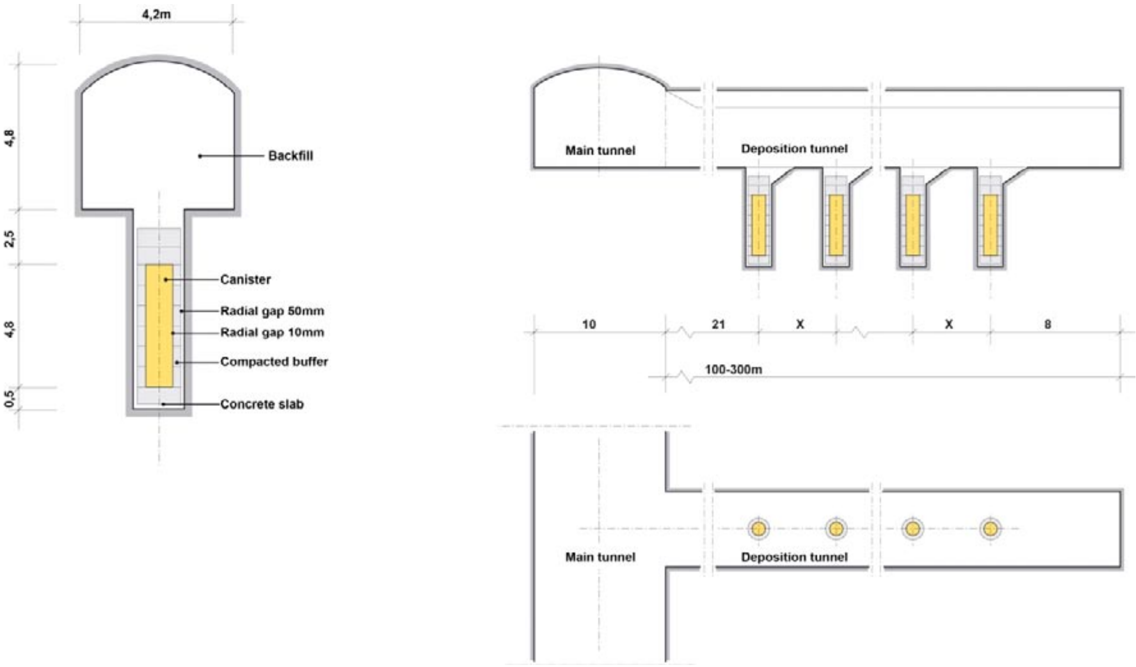

Figure 2-2. Geometry of the deposition tunnel and deposition holes in which the canisters are emplaced.

2.1 Stepwise development of the repository

The KBS-3 repository will be implemented in steps, and each step will provide the opportunity to collect data and to test models and site-specific assumptions with respect to the excavation response. It is assumed in this context that the repository will be developed in eleven steps, see Table 2-1 for a simplified outline, developed from a description by /Pettersson and Lönnerberg 2008/ and for the purpose of this report.

The stepwise construction of the repository will need to fulfil all the specified requirements. The design, manufacture and construction will have to be verified in order to underpin that the solutions are in compliance with the stipulated requirements. Finally, there will need to be a "control programme" during the implementation to ensure that the intended initial state, as assumed for the analyses of the long-term safety, can be fulfilled and to identify any quality deviations that may occur.

Table 2-1. Outline of the development of the repository.

Step	Development	Description
1	Site investigations prior to start of excavation	The site conditions are described in Site Description Models before construction. The site investigation results are used for repository design and safety assessment reports included in the application for permits to site and construct the repository
2	Repository development, i.e. excavation of access ramps, shafts, transport tunnels and underground openings required for the revised safety assessment.	After receiving all pertinent permits, construction of the facility may start. The underground portion is developed in steps by different excavation methods, see for example /Bäckblom et al. 2004/ and /Brantberger et al. 2006/ The ramp and blind shafts are excavated by drill and blast. After the central area has been excavated (around 30,000 m²) and necessary equipment installed, the transport, investigation and main tunnels are excavated. Transport tunnels are the link between the central area and deposition areas as well as the link between the different deposition areas. Investigation tunnels are excavated to open up new deposition areas and to locate the boundaries of deposition areas and to characterise the deposition areas from such tunnels. Investigation tunnels will later be enlarged for transport purposes or to serve as main tunnels. The main tunnels are situated within each deposition area. Deposition tunnels are excavated perpendicular from the main tunnels. Repository development work is a stepwise process. The detailed site characterisation is executed in parallel with all excavation work underground.

Step	Development	Description
3	Excavation of deposition tunnels	Deposition tunnels, 100–300 m long, are excavated at c/c 40 m with an inclination of 1–2°. The cross-sectional area is approximately 20 m^2. Several tunnels (around 10) will be excavated simultaneously using very careful drill and blast techniques. SKB is planning to excavate deposition tunnels in two steps: top heading and bottom bench, the bench having a thickness of 0.8–1.0 m. The reference method is drill and blast with wire-sawing as an alternative method. Each deposition tunnel will contain 10–45 canisters. Note that the groundwater inflow to the tunnel (and deposition holes) should be very low for robust emplacement of the backfill material in the tunnels and buffer in the deposition holes: Therefore, it is assumed that grouting the rock before excavation may occasionally be necessary.
4	Excavation of deposition holes	The deposition holes, approximately 8 m deep with a nominal diameter of 1,750 mm, are likely to be excavated by downhole push reaming. The c/c distance between the deposition holes is 6–10.5 m depending on the thermal properties and other local key properties of the bedrock. Excavation of deposition holes will commence in a deposition tunnel after the full length of the tunnel has been excavated and investigated.
5	Lead time to deposition work	After a number of deposition tunnels and deposition holes have been excavated, the area is cleared for deposition of the canisters. The switch between “construction” and “deposition” is expected to occur at least one year after a maximum of 200 deposition holes have been prepared, as the design capacity is “one canister a day”. The current planning is that the time between completion of deposition tunnel excavation and plugging of the same tunnel shall be less than five years.
6	Deposition	After receiving the authorities’ permission to deposit spent fuel, the canisters are transported from the encapsulation plant to the disposal site and then down to the central area for reloading into the deposition machine that moves the canisters from the reloading station into the deposition holes. The emplacement of the buffer in the deposition holes up to the planned top of the canister is done while the canister is transferred from the canister transport cask to the deposition machine in the reloading station. After correct positioning of the deposition machine at the deposition hole by using a navigation and positioning system, a gamma gate can be opened and the deposition of the canister will take place. The gate is then closed until the upper buffer blocks are emplaced up to the deposition tunnel floor level.
7	Backfilling	When all canisters in one deposition tunnel have been emplaced, backfilling and final sealing of the tunnel can start. The backfilling will be done with pre-compacted blocks of swelling clay and with some additional pellets for filling the void between the blocks and the rock wall and the roof of the tunnel. The backfilling of one tunnel is estimated to take 10–12 full weeks working twenty-four hours a day every day of the week.
8	Drift end plug	After backfilling has been completed, a cast low-pH concrete plug is constructed at the entrance of the deposition tunnel.
9	Sealing of deposition area	The sealing of the repository has not been decided in detail. The underground portion is split into several “deposition areas” and it may be the case that different deposition areas and interconnecting transport tunnels are sealed stepwise.
10	Sealing of repository	When all the spent nuclear fuel has been encapsulated and disposed of, and all the deposition tunnels have been backfilled and sealed, the repository will be closed. Backfilling and closure of all openings in the underground area will require a new permit from the authorities.
11	Post-closure phase	The post-closure phase starts after the sealing of the repository has been finished. It is possible that rationales may be developed for monitoring of the post-closure phase, such as verification of safeguard requirements. The extent of the post-closure monitoring programme will essentially be dictated by the decisions made at closure.

2.2 Treatment of the excavation damaged zone in SR-Can

The SR-Can project /see SKB 2006a/ was a preparatory stage for the SR-Site assessment to support the licence application to site and construct the final repository for spent nuclear fuel. The purposes of the safety assessment SR-Can were to:

- make a first assessment of the safety of potential KBS-3 repositories at Forsmark and Laxemar to dispose of canisters as specified in the application for the encapsulation plant;
- provide feedback to design development, to SKB's R&D programme, to further site investigations and to future safety assessment projects;
- foster a dialogue with the authorities that oversee SKB's activities, i.e. the Swedish Nuclear Power Inspectorate, SKI, and the Swedish Radiation Protection Authority, SSI[6], regarding interpretation of applicable regulations, as a preparation for the SR-Site project.

One of the issues dealt with in the SR-Can was the excavation damaged zone (EDZ) as the EDZ may influence the overall long-term safety due to lower transport resistance and higher fracture transmissivity around the deposition holes and the deposition tunnels.

The SR-Can deemed it reasonable to assume that in general, provided that proper excavation techniques and quality control are applied, the EDZ, if it develops at all, will be limited to a narrow zone (a few decimetres) adjacent to the tunnel and that it will not form a continuous hydraulically conductive path. It was also assumed that possibilities for more extensive fracturing would only occur as a consequence of poor engineering and inadequate QA practices, including the possibility that the tunnel is excavated parallel to a joint, etc so that the EDZ fractures link to the joint set.

Despite these assumptions, SR-Can explored the potential importance of the EDZ and the necessity for controlling its impact. For these reasons two different cases were studied, as listed below.

- The expected conditions are that the deposition tunnel excavation work is performed with the intention of limiting the EDZ and with the application of the necessary QA. In this case, the EDZ is likely to be limited in spatial context and to be discontinuous between different rounds of blasting. Based on the observations made at an experimental tunnel (TASQ) at the Äspö Hard Rock Laboratory (HRL), it was assumed that the rock permeability parallel to the axis of the tunnel will be increased by about half an order of magnitude over a thickness of 0.3 m, but due to the drill and blast techniques used, the EDZ will occur in 5 m sections with 0.5 m breaks of undamaged rock between them. However, in the flow modelling the EDZ was cautiously assumed to be continuous, but of low permeability.
- A limiting case was to assume the use of conventional drill-and-blast techniques for the tunnel, without applying any special QA procedures for controlling the EDZ. This may possibly create a continuous damaged zone, primarily at the bottom of the deposition tunnel. This case was assessed by increasing the hydraulic conductivity of the EDZ in the flow model, but keeping the size by two orders of magnitude. This increase in hydraulic conductivity was selected rather arbitrarily, but was judged as being typical, in order to assess the importance, if any, of a significant EDZ.

The reader is referred to the SR-Can Main report /SKB 2006a/, the so-called Data Report / SKB 2006b/ or the Model Summary Report /SKB 2006c/ for general descriptions of the overall modelling. Extracts from some EDZ-specific modelling are given below.

[6] SKI and SSI merged into one authority in July 1, 2008 under the name of the Swedish Radiation Safety Authority (in Swedish: Strålskyddsmyndigheten), see www.ssm.se.

The geometry for the near-field model is shown in Figure 2-3. Radionuclide release from the near-field is assumed to occur along three characteristic transport paths Q1–Q3 where:

- Q1 corresponds to a fracture intersecting the deposition hole. In the Discrete Fracture Network model, DFN, used for hydrogeological modelling, several fractures may intersect the deposition hole and these could be located anywhere along the longitudinal axis of the hole. However, to simplify the near-field migration model, the flow rates of all fractures intersecting the deposition hole were assigned to a single fracture. This fracture was placed on the opposite side of the buffer to the canister defect, hence minimising the transport distance and the diffusional transport resistance.
- Q2 corresponds to the excavated damaged zone, EDZ, and was in the hydrogeological model treated as a thin conductive layer located at the bottom of the deposition tunnel. As explained above, the extension of the EDZ in the longitudinal direction depends on the quality control applied during excavation and the excavation method.
- Q3 corresponds to a larger fracture zone intersecting the deposition tunnel. The deposition tunnel was, in the hydrogeological model, intersected by several fractures and fracture zones with different properties and the location of the Q3 fracture zone was obtained by tracking advectively transported particles released in the centre of the deposition tunnel just over the deposition hole. As the distance between the deposition hole and this fracture zone differs, the longitudinal dimensions of the modelled deposition tunnel may be different for different deposition holes.

As regards the EDZ, a sensitivity analysis was made for Forsmark and Laxemar by assuming a conductive zone on the floor of the deposition tunnel that was continuous and at a hydraulic conductivity that was a few orders higher than the hydraulic conductivity of the bedrock as evaluated for a 30 m scale.

Typical hydraulic data assumed for Forsmark /Hartley et al. 2006a/ and Laxemar /Hartely et al. 2006b/ are shown in Table 2-2.

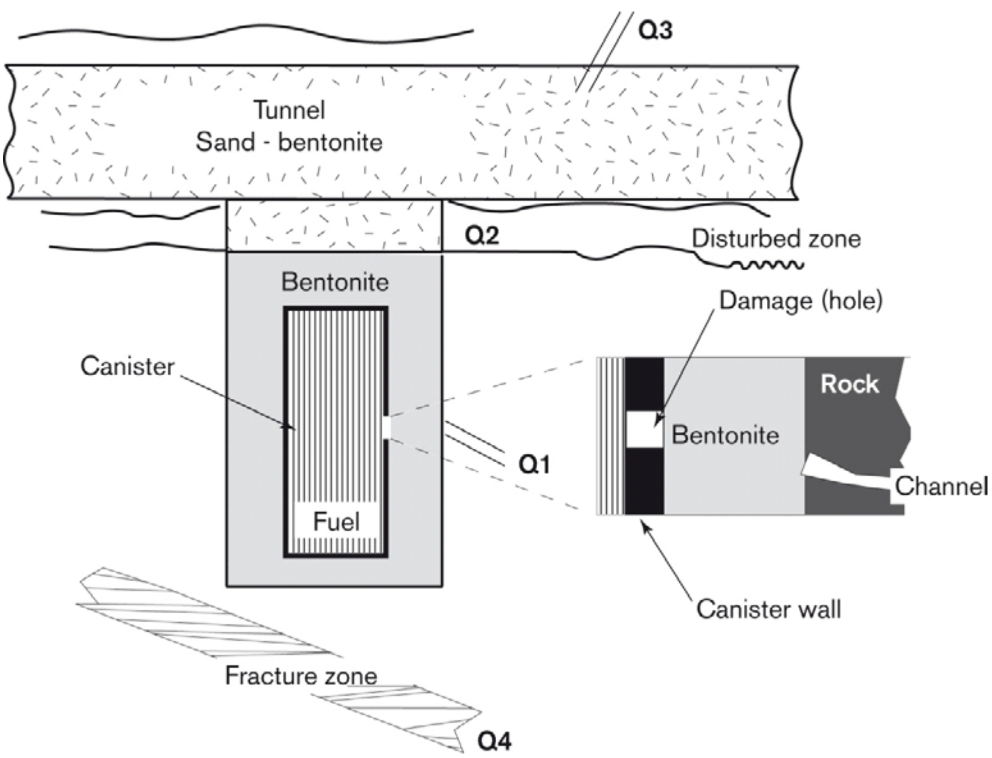

Figure 2-3. *Geometry of the near-field /Figure 2-15 in Hartley et al. 2006b/ It was assumed that there was no EDZ along the deposition hole and that the Q4 transport route was not used.*

Table 2-2. Hydraulic data for Forsmark and Laxemar. Revised after /Hartley et al. 2006ab/.

Parameter	Forsmark Value	Laxemar Value
Deposition tunnel rock conductivity[1]	10^{-10} m/s	10^{-10} m/s
Deposition hole hydraulic conductivity[2]	10^{-11} m/s	10^{-11} m/s
Backfill porosity	0.35	0.35
Thickness of EDZ	0.3 m	0.3 m
Hydraulic conductivity	$3 \cdot 10^{-10}$–$3 \cdot 10^{-9}$ m/s	$3 \cdot 10^{-8}$–$3 \cdot 10^{-7}$ m/s
Kinematic porosity	10^{-4}	10^{-4}
Flow-wetted surface	0.0 m^2/m^3	

[1] Conductivity of the backfill.

[2] Buffer conductivity in the deposition hole.

For the Forsmark case, an EDZ variant was selected /Hartley et al. 2006a/ by choosing a hydraulic conductivity for the EDZ that was so high that it started to have an impact on flow-paths. The value used was $3 \cdot 10^{-9}$ m/s.

An example of the modelling approach is shown in Figure 2-4. In this approach the tunnels and the EDZ were simulated by equivalent fractures that were connected to the fracture network. The EDZ fracture was subdivided into 6 m elements. Another modelling approach which was also used in SR-Can was a pure Continuum Porous Medium (CPM) model, in which the EDZ was modelled as a thin strip.

Based on the assumptions, models and data used, the conclusion was that the importance of the EDZ around deposition tunnels is limited in comparison to other transport routes for radionuclides, even for very pessimistic assumptions concerning the EDZ in relation to the reference excavation method, i.e. drill and blast. Cautious excavation methods were still recommended for the deposition tunnels, because competing transport routes may be assessed as being less important with additional data and because the conclusion regarding the EDZ is based on simplified, stylised modelling.

An example of calculation results is shown in Figure 2-5. The Darcy velocity Q1 path was not affected by the EDZ, which is as expected. The most notable change was an increase in the initial Darcy velocity for the Q2 path of over half an order of magnitude and an increase in the distance travelled in the EDZ from a range of 0 to about 60 m with a median of about 16 m.

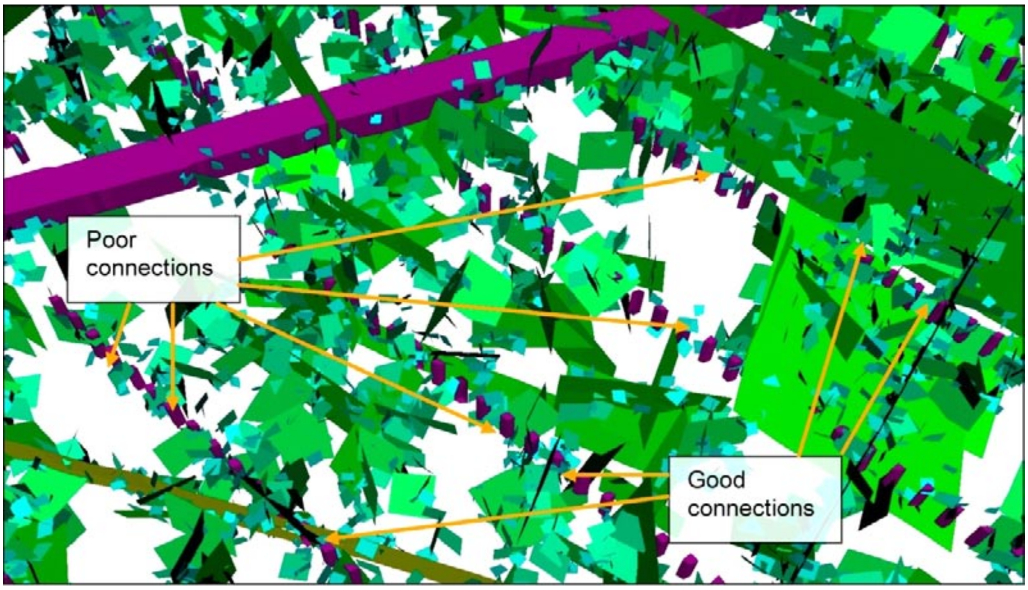

Figure 2-4. *Connected fracture network surrounding the canister in a combined Discrete Fracture Network/Continuum Porous Medium approach. The deposition holes and transport tunnel are coloured purple and the fractures are coloured by transmissivity. /Figure 4-8, Hartley et al. 2006a/*

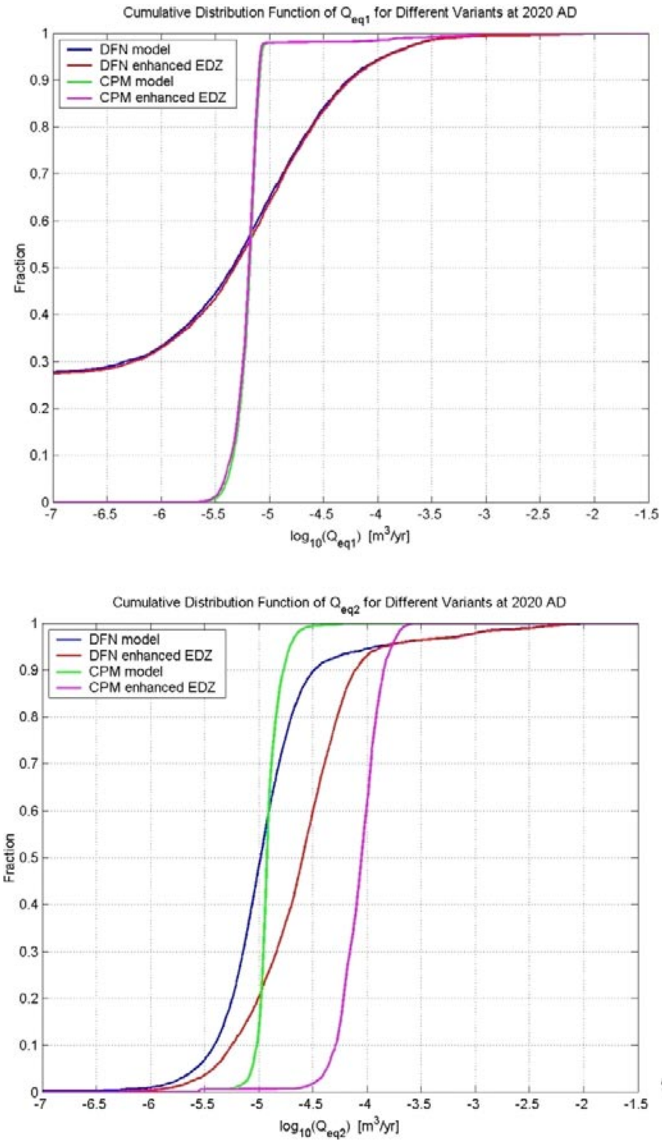

Figure 2-5. *Comparison of cumulative distribution plots of equivalent flow rates Qeq for paths Q1and Q2 at release time 2020 AD for the alternative Discrete Fracture Network (DFN) and Continuous Porous Medium (CPM) for to EDZ data. Redrafted after Fig 4-51 in /Hartley et al. 2006a/.*

To summarise, the sensitivity of the EDZ properties was not noticeable since the system of deposition tunnels was arranged orthogonal to the head gradients. Therefore, flow tends to be limited by what the fracture system can supply and paths have to leave the tunnel or EDZ after a relatively short distance in order to find a flow path to the surface through the fracture network.

For the Laxemar case /Hartley et al. 2006b/ a case was selected with a hydraulic conductivity ten times higher in the EDZ than for the reference case, i.e. $3 \cdot 10^{-7}$ m/s. Cumulative distribution plots of travel time, initial Darcy velocity and F-factor[7] for path Q1, path Q2 and path Q3 at release time 2020 AD were compared. The travel time distributions for this variant were very similar to those for the reference case. For example, the median travel time for path Q1 at 2020 AD was 57 years for the reference case, and 51 years for the enhanced EDZ variant case. For the initial Darcy velocity, the distributions obtained for path Q1 from the reference case ($2.5 \cdot 10^{-4}$ m/y) and from this variant case ($2.8 \cdot 10^{-4}$ m/y) were similar. However, for the initial Darcy velocity for the other two paths, path Q2 and path Q3, the median values ($2.5 \cdot 10^{-3}$ m/y and $2.9 \cdot 10^{-4}$ m/y respectively) was higher compared to

[7] Transport resistance along flow path [T/L], expressing the relation between flow wetted surface and ground-water flow. It controls retention of nuclides in the geosphere.

the reference case ($4.0 \cdot 10^{-4}$ m/y and $1.1 \cdot 10^{-4}$ m/y respectively) and the path Q2 distribution was now similar to the path Q1 distribution. Comparisons of the F-factor distribution showed that the F-factor was not particularly sensitive to the increase in EDZ conductivity.

These calculations assumed that no spalling occurred in the deposition holes. /Hökmark et al. 2006/ investigated spalling and drew the following conclusions:

- At both Forsmark and Laxemar, there will be spalling in the deposition holes because of the thermal load, unless there is some supporting bentonite swelling pressure. However, if the bentonite buffer would have time to take up water and begin to close the buffer-rock gap before the thermal stresses have reached levels that may cause spalling, then the bentonite support pressure will probably be sufficient to prevent spalling altogether, or to limit the growth of spalled rock regions.
- An indication of the geometry of this spalled zone can be assessed from the APSE experiments at Äspö /Andersson 2007/. The APSE experience indicates that the failure, once it has been initiated, redistributes the stresses continuously in such a way that the rock eventually stabilizes outside a notch-shaped region of much smaller volume. Similar observations have been made at the URL in Canada, cf /Martin 2005/.
- At present, there is no way of directly calculating the actual shape or depth of thermally induced failures. Experience indicates that the failures will be notch-shaped and that the notch will self-stabilize at some depth that depends on the prevailing stress at the time of the failure. Once the notch is stable, subsequent increases in stress will not significantly increase the depth of the failure. It is anticipated that the notch will be 0.1 m deep and 0.14 m wide, i.e. it is likely that the notch will be formed and stabilized much earlier, and at a lower tangential stress, than in the simulated results. However, the experience of brittle failures induced by continuously increasing thermal stresses is not extensive at present.

The effects of spalling were evaluated within SR-Can /Appendix D in Hartley et al. 2006b, Neretnieks 2006/.The results indicate that spalling may increase equivalent flow rates by more than one order of magnitude. Example calculations for the Laxemar site are shown in Figure 2-6.

Despite the relatively major impacts on the performance measures for the cases where it was assumed that EDZ forms a continuous band of higher transmissivity along the floor of the deposition tunnel, SR-Can demonstrates /SKB 2006a, report Section 10.5.7/ that this has no impact on dose.

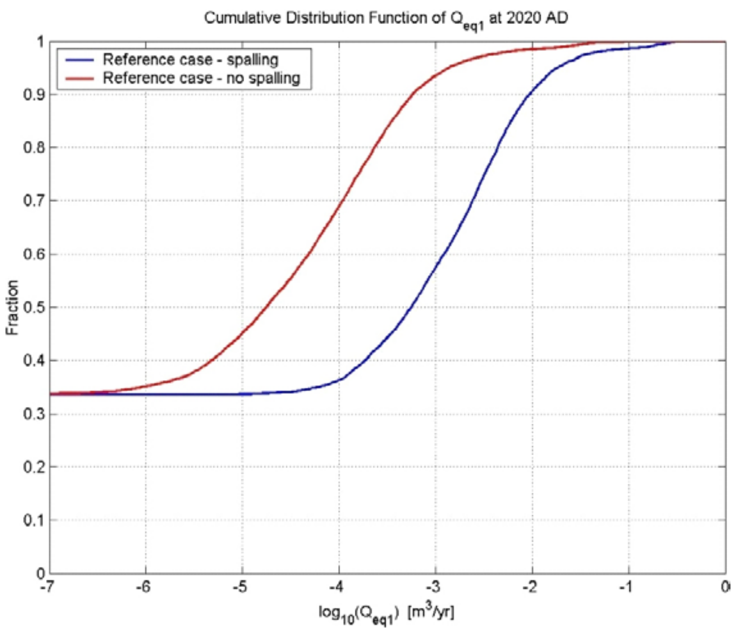

Figure 2-6. *Cumulative distribution plots of Q_{eq} for path Q1 in the combined repository and regional scale models at time 2020 AD in Laxemar for conditions with and without spalling. /Figure 9-37 in SKB 2006a/*

2.3 SR-Site pre-modelling: Sensitivity studies of hydrogeological model variants for the Laxemar site

As part of the SR-Site modelling project it will be necessary to investigate a number of variants as a means of identifying which model components and parameters will have an impact on site performance. Therefore, a number of pre-model variants were studied to understand the sensitivity of a number of performance measures to the chosen parameters. The report by /Joyce et al. 2008/ presents the results of varying tunnel backfill properties, the importance of a void in the backfill in combination with varying excavation damage zone properties and adding excavation damage and spalling to the deposition hole as well.

The "base case" for this task was a backfill of 10^{-10} m/s in the tunnels at repository depth, but 10^{-5} m/s (representing gravel) in the central area of the repository, in the access ramp and in the shafts. Based on a draft of this report, appropriate excavation damage zone properties have been redefined to be a transmissivity of 10^{-8} m^2/s and 0.30 m thick beneath the tunnels.

The properties used for repository features in the base case model are given in Table 2-3. Other model properties are as described in /Hartley et al. 2006b/.

Three particles corresponding to three path types are released around each canister:

1. Q1 in the fracture with the highest flux that intersects the deposition hole;
2. Q2 in the tunnel EDZ fracture adjacent to the deposition hole;
3. Q3 in the Continuum Porous Medium tunnel 1 m directly above the deposition hole.

A number of performance measures were used: the travel time (t), path length (L), initial Darcy velocity (U) and F-factor (F), see /Hartley et al. 2006b/. These are sub-divided into accumulated values when particles are within a tunnel (T), within the EDZ zone along the tunnel and within the rock (r).

Several cases were tested and are briefly explained here: *"Degraded backfill"* means that the hydraulic conductivity of the main, transport and deposition tunnels was changed from 10^{-10} m/s to 10^{-8} m/s. *"Less transmissive central area, ramps and shaft"* means the hydraulic conductivity of the central area, ramp and shaft backfill was lowered from 10^{-5} m/s to 10^{-8} m/s. *"Crown space"* means that the backfill at the crown subsided due to consolidation at the top of the tunnel. This was represented in the repository scale model by changing the rock properties of a narrow band, 0.1 m thick, at top of the deposition tunnels to a high hydraulic conductivity of 10^{-3} m/s. *"Less transmissive tunnel EDZ"* means that the deposition tunnel EDZ was changed from 10^{-8} m^2/s to 10^{-10} m^2/s. "*Deposition hole EDZ"* is a case represented by a transmissivity of 10^{-9} m^2/s for a zone of 0.1 m and *"Deposition hole EDZ and spalling"* by a transmissivity of 10^{-5} m^2/s for an extent of 0.1 m. Finally a case *"Deposition hole EDZ and spalling, degraded deposition tunnel EDZ"* was investigated where deposition hole EDZ and spalling variant from above is used, but where the transmissivity of the deposition tunnel EDZ was increased from 10^{-8} m^2/s to 10^{-6} m^2/s.

Table 2-3. Base case properties for repository features.

Parameter	Value
Main tunnel hydraulic conductivity	10^{-10} m/s
Transport tunnel hydraulic conductivity	10^{-10} m/s
Deposition tunnel hydraulic conductivity	10^{-10} m/s
Central area hydraulic conductivity	10^{-5} m/s
Ramp and shaft hydraulic conductivity	10^{-5} m/s
Deposition hole hydraulic conductivity	10^{-11} m/s
Deposition tunnel EDZ transmissivity	10^{-8} m^2/s
Tunnel, central area, ramp and shaft backfill porosity	0.35
Deposition tunnel EDZ porosity	10^{-4}
Deposition tunnel EDZ thickness	0.3 m

In general, for all cases, the performance results are not significantly changed irrespective of the EDZ properties selected. It was found that varying the backfill properties in the tunnels had an effect mainly on the tunnel performance measures and indicated that there would be greater flow in the tunnels when the backfill was degraded. However, changing the backfill properties in the central area, ramps and shafts only had minor effects on performance measures for those property values chosen.

The presence of a crown space in the deposition tunnels had an effect on the tunnel performance measures and less of an effect on the performance measures for the rock and the tunnel EDZ. The effect on tunnel performance measures could have been even greater if a crown space had also been modelled in the main and transport tunnels. The effect of degraded tunnel backfill when a crown space was present had an effect on mean travel times and path lengths in the tunnels. The less transmissive tunnel EDZ also had an effect on the tunnel and tunnel EDZ mean travel times and path lengths in the presence of a crown space.

The effect of the deposition hole EDZ and spalling was to increase the travel times and path lengths in the tunnels. Where there was also a degraded deposition tunnel EDZ, there were faster and longer paths in the deposition tunnel EDZ and a reduction in the F-factor[8] in the rock. The increased availability of flowing fractures provided by the deposition hole EDZ and spalling increased the number of successful paths for the Q1 release points and provided additional flow pathways around the deposition holes and tunnel EDZ.

[8] Transport resistance along flow path [T/L], expressing the relation between flow-wetted surface and groundwater flow. It controls the retention of nuclides in the geosphere.

3 Main results from previous major studies and experiments

Several international meetings and workshops have been arranged to discuss the concept of the excavation damage zone, to discuss data and experiments and how to treat the excavation damage in the safety assessment, for example the meetings in Canada 1988, 1996 and 2003 /NEA 1989, CNS 1996, Martino 2003/, the workshop in Paris /NEA 2002/ and in Luxembourg 2003 /EU 2005/.

In the latter workshop, the potential role of the EDZ in system performance and its assessment were here defined /Zuidema 2005/ as:

- "The EDZ exists when emplacing the wastes, the backfill material and the seals and is thus an integral part of the system that will evolve with time. There may be a need to consider the EDZ when assessing the *temporal evolution of the overall repository system* (engineered barriers, access routes, host rock) to define the relevant system conditions at the time when eventually some radionuclides may be released. The EDZ may e.g. affect the hydrogeological, geochemical, geomechanical and thermal conditions but also phenomena and processes related e.g. to the release of gas generated within the repository in ways that may be both favourable and/or detrimental to system performance.
- The EDZ may form a *potential pathway for migration of radionuclides* from the waste emplacement rooms to the surface environment. However, this requires an evaluation of the behaviour of the EDZ for the time and conditions when radionuclide release actually might occur (after canister breaching and breakthrough of radionuclides through the buffer which may in the case of a repository for SF/HLW be tens of thousands of years or even more).
- The behaviour of the EDZ and the changes with time need to be assessed on a system-specific basis. As mentioned above, the behaviour and the processes occurring and their magnitude may be such that in some systems the EDZ may be only of very limited *importance to post-closure performance* whereas in other systems it may be a critical factor."

The results of previous workshops and results over the period 1988–1998 were summarised by /McEwen 2005/. He described the evolution of nomenclature for different rock types. The conclusions for crystalline rock are, for example, that the processes forming the EDZ are well understood, that suitable characterisation methods exist and have been tested in various URLs, the geometry of the EDZ in all rock types is sufficiently well known but that it is difficult to determine the hydraulic characteristics of EDZs and it is even more difficult to determine radionuclide transport characteristics of the EDZ. He also reported that methods exist to limit the extent of an EDZ (e.g. the use of a TBM in crystalline rocks. For future works/McEwen 2005/ distinguished work in crystalline rock in low and high stress environments. Unclear issues for low stress environment were e.g. to what extent the EDZ is significant in providing a more transmissive pathway through the rock and how easy it is to demonstrate convincingly that low permeability seals or plugs in the rock can prevent such an effect. For the high stress environment, McEwen thought that it was necessary to demonstrate convincingly that low permeability seals or plugs in the rock can intercept the EDZ and limit its significance.

An overview of major EDZ experiments and treatment of the EDZ in safety assessments were published by /Bäckblom and Martin 1999/. They concluded that there is a good understanding of the mechanical response for the conditions studied and the effect of excavation by drill and blast and mechanical excavation respectively. They pointed out that heating, cooling and time aspects are factors which contribute to the excavation response and that these matters have not been dealt with explicitly in full scale previously, and looked forward to experiments in progress, e.g. the prototype repository at the SKB's Äspö Hard Rock Laboratory (HRL). /Bäckblom and Martin 1999/ also concluded that the hydraulic responses close to an underground opening may warrant further studies to demonstrate convincingly that all physical processes are understood. Quite a wealth of data is necessary to deduce whether changes are significant compared to the natural variability of properties. A particularly useful method for the collection of mechanical data on the EDZ is the micro-seismic network in combination with accurate stress measurement and careful descriptions of the discontinuities.

Several methods to mitigate the excavation response were discussed in the paper by /Bäckblom and Martin 1999/. They suggested that the development of spalling can be avoided by proper siting, choice of repository depth, orientation and shape of opening. The extent of damage caused by excavation can be reduced to a few centimetres by the use of mechanical excavation.

In spite of several workshops and papers, it was deemed necessary to revisit the major experiments to extract and evaluate the results and information provided. The investigation was focused on the crystalline environment, as the Swedish repository will be located in crystalline rock. During the course of investigation, many activities have been conducted in parallel by other organizations.

Posiva at ONKALO conducted several tests related to EDZ. One aspect was to improve excavation methods and procedures so that the excavation damage is more predictable. A second aspect was to develop investigation methods to verify the excavation damage zone properties (depth, continuity, fracturing, etc) in different parts of the tunnel profile (walls and roof, floor). The third aspect was to improve the interpretation and modelling methods for estimation of EDZ properties. The results of these tests are summarised in /Öhberg et al. 2008/. As regards investigation methods, the ground penetrating radar, for example, has been tested to study the connectivity of the EDZ /Silvast and Wiljanen 2008/.

Further on, the DECOVALEX project elaborated on uncertainties correlated to EDZ characterisation, uncertainties connected to modelling and outstanding issues and further work /Hudson et al. 2008/. In the work carried out it was demonstrated how widely different modelling approaches can be adapted to simulate the evolution of EDZ around a heat-releasing nuclear waste emplacement tunnel in fractured rock. It was also concluded that if the models are properly calibrated and validated after excavation, then a reasonable estimate can be made of how the EDZ will progress during the heating period after emplacement.

SKB has also pursued Zuse project activities in parallel with the compilation of previous knowledge, e.g. methodology for assessing the transmissivity of blasting-induced fractures and field methodology for measuring transmissivity and connectivity in the periphery of the deposition tunnels. SKB has also continued studies concerning the thermomechanical process to further strengthen the understanding of rock spalling.

3.1 Stripa experiments

The international Stripa Project, 1980–1992, studied the natural and engineered barriers in an abandoned iron ore mine in the central part of Sweden. One of the concerns during the period 1986 to 1992 was the Excavation Damaged and Disturbed Zone. The summary report by /Gray 1993/ provides an overview of the results.

The set up for a pertinent EDZ-experiment is shown in Figure 3-1. The Buffer Mass Test tunnel was excavated in 1978 and was later used for macro-permeability tests over the period 1978–1980 followed by several other tests as well. The excavation process is described in detail in the report by Börgesson et al. 1992. The drilled depth was 3.6 m using 35 diameter holes. Lower charging (Gurit, around 0.3 kg/m) was used in the periphery holes. The helper holes were charged with 0.8 kg/m detonating cord and the rest of the blast holes by 0.8 kg/m ANFO. More explosives were used in the floor.

It is interesting to note that the tunnel was heated twice. The first heating was for a period of two years when the tunnel was backfilled. The temperature was as high as up to 98°C in the floor; the second heating was for a period of a hundred days. The setting for the test is extraordinary as the tunnel has been open for some 10 years after excavation (resembling the lead time from excavation to deposition for a final repository) and has also been heated to fairly substantial levels.

The evaluations of the EDZ are not only based on actual measurements but on combined circumstantial evidence of iterative modelling (see Figure 3-2) and repeated testing procedures. The evaluation of this Stripa test is condensed in a number of main data for the "blast damaged zone" and the "stress disturbed zone" as shown in following Table 3-1.

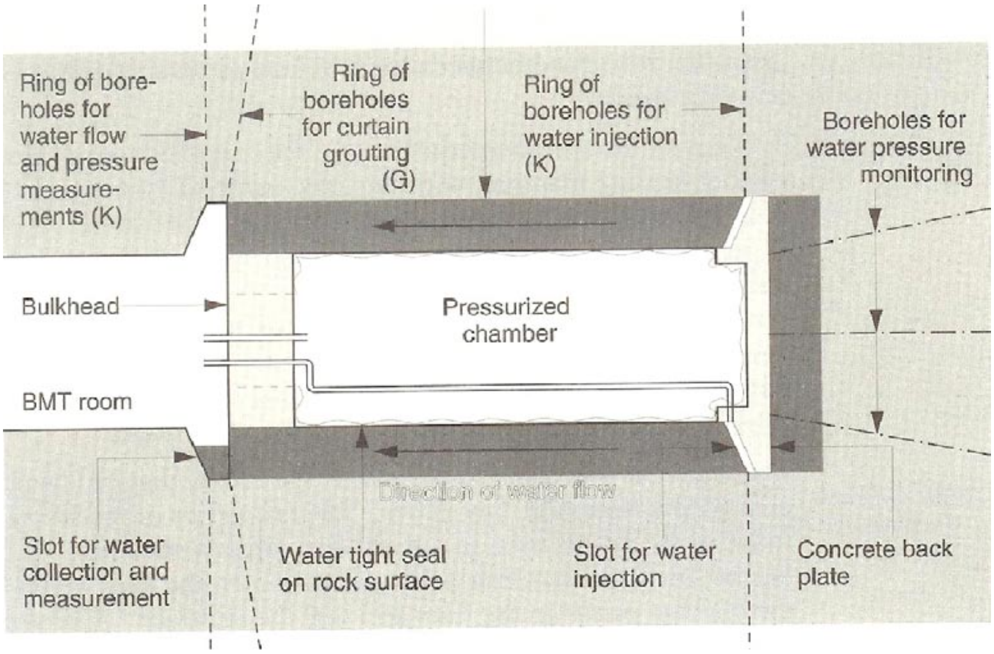

Figure 3-1. *Layout of an experiment to characterise and grout the excavation disturbed zone around the inner Buffer Mass Test Tunnel. The length of the tunnel is 12 m. After /Gray 1993/.*

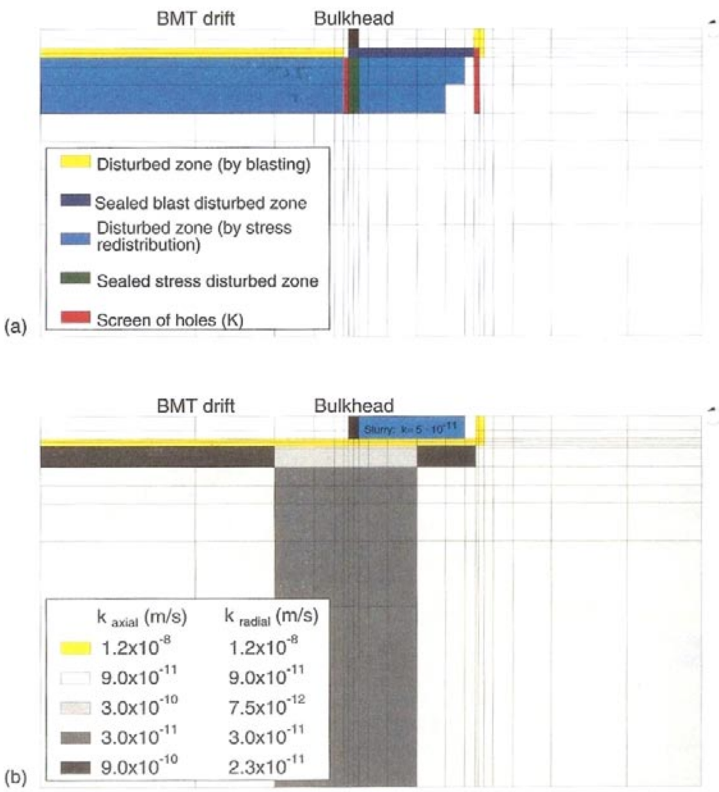

Figure 3-2. *a) the predictive model b) the calibrated model derived on test results: After /Börgesson et al. 1992/.*

Table 3-1. Main results from the Stripa EDZ tests /Gray 1993/.

Item	Hydraulic conductivity k [m/s]	Thickness of the zone [m]
The Damaged Zone ("Blast Damage Zone")	10^{-8} m/s (local variation in the scale of metres within the range $10^{-11} < k < 10^{-6}$ and maybe anisotropic	0.3 m in the walls and 1.2 m in the floor
The Disturbed Zone ("Stress Disturbed Zone")	Axial hydraulic conductivity $5 \cdot 10^{-10}$ (Local variation in the scale of tens of metres in the range $3 \cdot 10^{-10} < k_a < 9 \cdot 10^{-9}$ m/s). Radial hydraulic conductivity $5 \cdot 10^{-11}$ (Local variation in the scale of tens of metres in the range $7.5 \cdot 10^{-12} < k_r < 2.3 \cdot 10^{-11}$)	Up to 12 m but tested up to 7 m from the opening
Undisturbed rock	Isotropic with hydraulic conductivity in the range $3 \cdot 10^{-11}$ to $9 \cdot 10^{-12}$ over a scale of tens of metres	

In addition to measuring the hydraulic properties of the EDZ, the original intentions of the experiment included two separate phases in which the blast damaged zone and the stress disturbed zones were to be grouted. Before grouting, a series of pressure-flow tests were carried out. The Lugeon tests were carried out by packing off the outer 0.2 m and pressurizing the remainder of the hole within the inner end of the packer and the tip of the hole, which in the floor was 1.2 m from the surface and in the walls and ceiling was 1.0 m from the surface. The tests in the percussion drilled boreholes on the western wall were supplemented by tests in six supplementary holes which were core drilled close to the junction between the floor and the wall. The results of the tests are presented by /Börgesson et a1. 1992a/

The hydraulic conductivity of the blast damaged zone, as reported by /Gray 1993/, was measured to vary significantly within metres and in the range of $10^{-11} < k < 10^{-7}$ m/s. The tested volume of rock in the western wall had the highest mean hydraulic conductivity, generally in the order of m/s with specific values as high as $5 \cdot 10^{-5}$ m/s. The ceiling had the lowest mean hydraulic conductivity, which, generally, was in the order of 10^{-10} m/s. In the walls and the floor there was a general tendency for the hydraulic conductivity to increase with distance from the junctions between the vertical and horizontal faces of the excavations. The results clearly showed that, in planes normal to the boreholes (i.e. with the main axis of the tunnel), the hydraulic conductivity of the blast damaged zone is two to four orders of magnitude higher than that of the undisturbed rock mass and the stress disturbed zone.

However, when the quoted report /Börgesson et al. 1992/ is studied, the authors divided the results into three reliability levels, see Table 3-2. Note that the conclusions on hydraulic properties deviate from the conclusions drawn in the compilation /Gray 1993/; here the report by /Börgesson et al. 1992/ takes precedence.

The main conclusions based on the Stripa tests were that the hydraulic conductivity is interpreted to be $1 \cdot 10^{-8}$ m/s down to 0.3 m in the roof and walls and $2 \cdot 10^{-8}$ m down to 0.8 m in the floor. The results are from a tunnel covering a length of approximately three drill and blast rounds in saturated conditions and from a tunnel that was twice considerably heated. Furthermore, the tunnel was not newly excavated, but close to 10 years old when the test was performed, which make the Stripa tests even more interesting as an analogy to a deposition tunnel; the lead time between excavation and backfilling of deposition tunnels could be some 5 years.

3.2 AECL – Room 209 excavation response test

In the following sections, several tests conducted at the Atomic Energy Canada Limited (AECL) Underground Research Laboratory in Manitoba Canada are presented focussing on the flow properties of the rock. An excellent overview of the mechanical excavation response can e.g. be found in /Read 2004/.

Table 3-2. Conclusions from hydraulic parameter estimations. From /Börgesson et al. 1992/.

Reliability level	Hydraulic conductivity k [m/s]
Level 1: Parameter determined with complete certainty, i.e. no data or indications that allow for any other interpretation.	The hydraulic conductivity is on average $1 \cdot 10^{-8}$ m/s in the shallow zone 0–0.8 m from the rock surface. The conductivity in the floor is higher than in other sectors, i.e. $2 \cdot 10^{-8}$ m/s. The hydraulic conductivity of the virgin rock is $3 \cdot 10^{-11}$–$1 \cdot 10^{-10}$ m/s. The stress-disturbed zone extending to about 3 m from the surface has a decreased radial hydraulic conductivity with a factor of 4.
Level 2: Parameter determined with a great deal of confidence. Various data strongly support the evaluated parameters but there are one or two different interpretations that make them somewhat uncertain.	The average axial hydraulic conductivity in the stress-disturbed zone has increased with a factor of about 10.
Level 3: Best fit parameters. The measurements and evaluations yield the given parameter values but there are various other interpretations that are possible, although less likely.	The hydraulic conductivity of the most shallow 1–2 dm rock is $5 \cdot 10^{-8}$–$1 \cdot 10^{-7}$ m/s. In the inner part of the tunnel the conductivity of the virgin rock is $9.0 \cdot 10^{-11}$ m/s, while the conductivity of the zone extending 0.8–3.0 m from the rock is Axially $9.0 \cdot 10^{-10}$ m/s Radially $2.3 \cdot 10^{-11}$ m/s In the outer part of the tunnel the conductivity of the undisturbed rock is $3 \cdot 10^{-11}$ m/s and conductivity of the zone reaching 0.8–3.0 m form the rock surface is Axially $3.0 \cdot 10^{-10}$ m/s Radially $7.5 \cdot 10^{-12}$ m/s The possibility of having a highly permeable blast disturbed zone that goes deeper than 1.2 m cannot be disregarded.

The Room 209 Excavation Response Test was executed over the period 1985–1989 /Lang 1989, Simmons 1992/. Excavation was by drill and blast using a pilot tunnel and slash sequence. Pilot and slash rounds were 2.8 m in length drilled using 32 mm diameter holes, but no attempt was made to limit damage in the floor. Lifters were charged by "Forcite 75" explosives and the final perimeter holes by Primaflex. One method used at URL to quantify the damage was counting the percentage of visible blast hole traces (half-barrels) observed on the excavation profile. The charging of explosives was 3.15 kg/m^3 which included the floor of the room; charging was 1.55 kg/m^3 for the slash /Chandler et al. 1996/

The rock around Room 209 was essentially unfractured with only one natural, water-bearing, en echelon fracture intersecting the test tunnel. The responses to excavation through the tunnel were predicted and compared to the measured responses during and after excavation. The predicted mechanical response was very much in line with the measured response, but it was only possible to reconcile the hydrogeological response assuming that the permeability decreased in the walls and increased in the floor /Winberg et al. 1989/. The reasons for the decrease were hypothesised as being either the effects of partial desaturation of the permeable fractures, due to debris clogging the fracture, or chemical or biological factors.

The increase of permeability in the floor was attributed to the higher concentration of explosives at the floor needed to excavate the tunnel. This result is in line with the results of many other experiments indicating that inflow is less than predicted, cf the results from the macro-permeability tests and the site-characterisation and validation project at Stripa /Olsson and Winberg 1996/ where all modelling teams, although taking excavation disturbance effects into account, over-predicted the total inflow by a factor of between 3 to 8. The inflow to the averagely fractured rock outside of the fracture zone producing the bulk of the inflow was over-predicted by a factor of 20–30.

3.3 AECL – Room 209 connected permeability experiment[9]

The Room 209 Connected Permeability Experiment is one of the most relevant experiments, as the main objective of the experiment was to quantify the connectivity of excavation-induced fractures in the axial direction of the floor. The test was conducted in "unfractured" rock during the period 1988–1992. The test was conducted in the same location as the Excavation Response Test, where in situ stresses were sufficiently low /Martino et al. 1997/ for fractures to be formed around the excavation as a result of the excavation method. The test set-up was a reservoir between a pair of concrete dams and a monitoring slot 2 m deep, see Figure 3-3.

The concrete dam was first cast as 2 m in length within the boundaries of a single blast round, the dam was then extended to 4 m in length and spanned the intersection of two blast rounds. The right part of Figure 3-3 shows the flow as measured in the monitoring slot when the concrete pads were 2 m (Test #2) respectively 4 m (Test #4). The flow over two rounds (4 m) was 50 times less than for the flow within one round. The hydraulic conductivity based on flow measurement was estimated to be approximately 10^{-8} m/s for an EDZ area 0.3 m deep and 2 m wide. /Chandler et al. 1996/.

By using the information in the paper, the following detailed data are obtained[10]:

$K_{Test\ 2} = q/i \cdot A = 5 \cdot 10^{-7}\ m^3/s$;
$Q_{Test\ 2} = 1.8 \cdot 10^{-2}$ l/min $= 3.0 \cdot 10^{-7}\ m^3/s$; $i_{Test\ 2} = 1.0$; $A = 0.6\ m^2$

$K_{Test\ 4} = 1.6 \cdot 10^{-8}$ m/s
$Q_{Test\ 4} = 4.1 \cdot 10^{-4}$ l/min $= 6.8 \cdot 10^{-9}\ m^3/s$; $i_{Test\ 4} = 0.7$; $A = 0.6\ m^2$

After completion of the flow tests, 16 boreholes were drilled in the floor and tested at 100 mm intervals by using vacuum permeability apparatus (10 boreholes) and transient pressure pulse (6 boreholes). The latter were tested during saturated conditions.

During vacuum permeability testing, the majority of the boreholes showed a vacuum permeability in the range of 10^{-8} m/s to 10^{-11} m/s /Martin et al. 1992/ for intervals 0–500 mm below the rock-concrete interface but very low permeabilities below 500 mm.

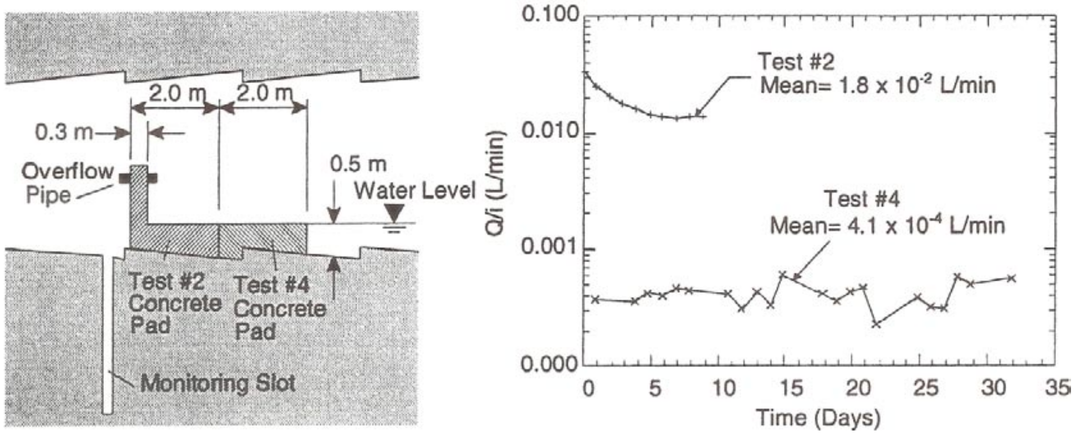

Figure 3-3. *Arrangement of the Room 209 Connected permeability experiment and summary of test results /Chandler et al. 1996/.*

[9] The suggested main introductory reference for the AECL EDZ work up to 1999 is the report by /Martino 2000/, where experiments and methods used are presented and evaluated, and the paper by /Martino and Chandler 2004/ and /Read 2004/.

[10] Note: The paper by /Martin et al. 1992/ provides the data $K_{Test\ 2} = 4 \cdot 10^{-6}$ m/s and $K_{Test\ 4} = 2 \cdot 10^{-7}$ m/s, but does not provide information on flow area.

The hydraulic measurements showed transmissivity values of 10^{-12} m^2/s to 10^{-13} m^2/s to a depth of 0.26 m below the rock/concrete interface and in the range of 10^{-14} m^2/s to 10^{-16} m^2/s below 0.26 m to full testing depth 0.86 m for the borehole.[11] The test section length used was 100 mm so the anticipated "hydraulic conductivity" would be 10^{-11}–10^{-12} m/s for the zone closest to the opening.

The report by /Martino 2000/ states that it was not possible to register any pressures at the rock/concrete contact when the hydraulic borehole measurements were conducted. The reason for this was that the length of the packer did not allow the zone closest to the tunnel wall to be tested. Butyl bentonite strips under the dam made for an effective seal. No flow occurred at the interface, although some may have occurred in EDZ immediately adjacent to it.

3.4 AECL – The Mine-by tunnel connected permeability test

The Mine-by Experiment and Mine-by tunnel connected permeability test were conducted between 1991 and 1992 at the 420 Level of the URL /Chandler et al. 1996, Martino 2000/. The in situ stresses were higher at this depth /Martino et al. 1997/ and the Mine-by tunnel shape and orientation were deliberately selected to produce a large excavation response. The circular 3.5 m diameter tunnel was excavated without explosives by using pilot holes around the periphery that were reamed with a larger diameter drill effectively connecting adjacent boreholes. The interior rock stub was removed by hydraulic rock splitters. Thus this tunnel has no blast-induced damages. The rock exhibited slabbing behaviour and the circular shape was progressively changed by extending notches. These notches extended 1.3 times the radius of the tunnel measured from the tunnel centre (0.5 m from the periphery). Source location of micro-seismic events indicated that most activity occurred within 0.8 m of the tunnel perimeter and 2/3 of the events occurred within 0.4 m. The results and experience from the Mine-by experiment were later used to plan the ASPE project at the Äspö HRL, see Section 3.8.

The Mine-by connected permeability test area was in the last 12 m of the test tunnel, Figure 3-4. An observation trench was excavated using the same line-drilling and splitting method as for the tunnel. Infrared photography in the trench revealed fractures forming slabs of varying thickness up to 2 cm near the tunnel perimeter. The depth of the "process zone" was around 20 cm below the stable tip of the break-out notch that formed down to roughly 30 cm below the design level of the floor. The "process zone" is the volume of rock where fracture development is initiated. When the tunnel reaches a stable shape, activity in the process zone decreases and eventually stops. Changing conditions (temperature, moisture) can re-initiate activity in this zone.

In a way similar to the Room 209 Connected Permeability Test, a concrete dam was constructed and extended as required. The inflow results for different pad lengths led to the following suggestions and conclusions:

- The "process" zone is a connected pathway of high permeability along the length of the Mine-by tunnel.
- The hydraulic conductivity of the process zone is approximately 10^{-6} m/s. (All four tests converge on the same value for flow divided by the hydraulic gradient (c. 5 ml/min) divided by the cross-sectional area of the process zone [0.05 m^2].)

Tracer tests were later conducted in the "process zone" in the floor of the Mine-by tunnel. The tracer tests /Frost and Everitt 1997/ were performed over a length of 1.5 m. Based on an equivalent porous media approach, the hydraulic conductivity was calculated to be $7.4 \cdot 10^{-7}$ m/s and the transport porosity to be 2.7%. Using a one-dimensional advective-diffuse transport model the longitudinal dispersivity and transport porosity were estimated to be 0.6 m and 3.3% respectively.

[11] Note that /Chandler et al. 1996/ states that the hydraulic conductivity below 0.3 m was measured to 10^{-14}–10^{-13} m/s consistent with other URL measurements. Above 0.3 m the "hydraulic conductivity" increased one to two orders" (i.e. around 10^{-12}–10^{-11} m/s).

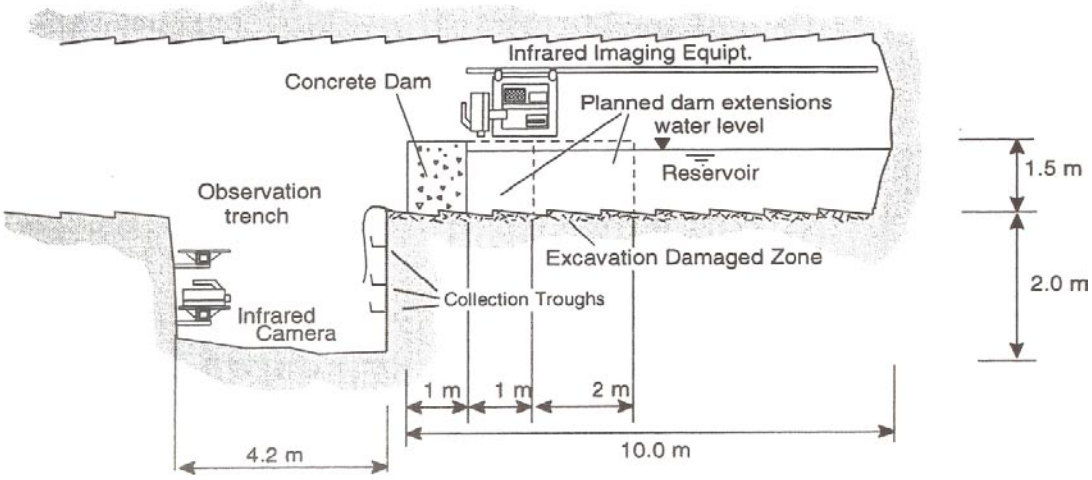

Figure 3-4. *Configuration of the Mine-by Connected Permeability Experiment /Chandler et al. 1996/.*

3.5 AECL – Tunnel Sealing Experiment

AECL in co-operation with JAEA in Japan, Andra in France and WIPP in USA executed a Tunnel Sealing Experiment (TSX) to study the constructability and ability of full scale tunnel seals to limit axial flow along a tunnel. A part of this experiment included a study of the EDZ and ways to cut off the EDZ by seals. The 40 m long tunnel was excavated by drilling and blasting at the AECL URL in Manitoba in Room 425 at the 420 Level. The tunnel was typically in rounds 3.5 to 3.8 m in length with 41-mm blast holes as a full-face operation /Chandler et al. 2002/. The perimeter holes were charged with Primaflex. The main test tunnel needed 13 blasts to excavate the 40 m long tunnel. The average half-barrel percentage was 67% and could not reach 100% due to rock peeling of the roof and floor of the tunnel regardless of whether the tunnel was excavated by drill and blast or by mechanical excavation.

Two keyed concrete/clay bulkheads were constructed and separated by a 12 m long sand-filled section, Figure 3-5. One bulkhead was 2.3 m in length with a 2 m key and was constructed of pre-compacted clay-sand blocks, and the second bulkhead was 3.5 m in length with 1.75 m long key and was a constructed of low-heat high performance concrete. A surface reservoir supplied the water head.

A suite of different methods were utilized to monitor the responses (see also Chapter 5). Before the bulkheads were constructed, a test was conducted to establish the hydraulic conductivity of the EDZ in the tunnel floor by using a similar approach to the one used in the Room 209 and the Mine-by connected permeability tests; two dams were constructed about 3 m apart and designed to provide

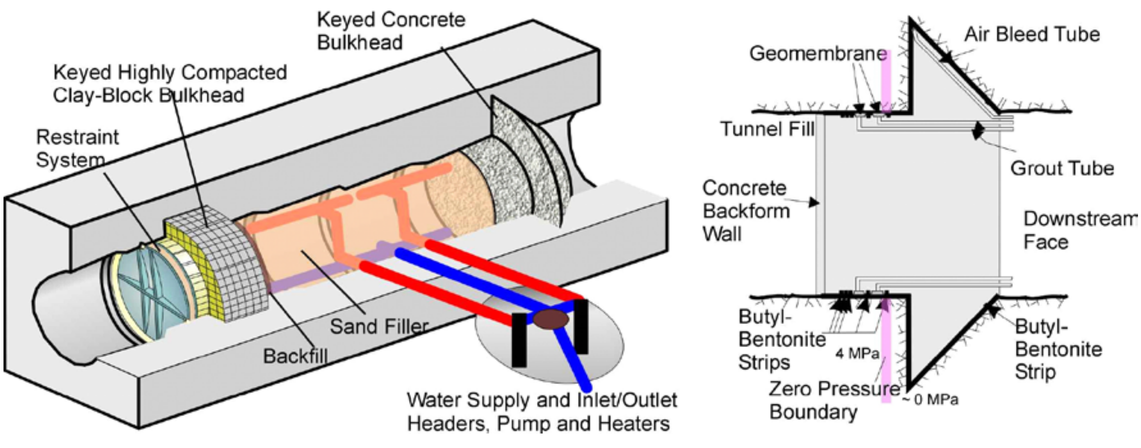

Figure 3-5. *The TSX tunnel and its installations (Courtesy J Martino, AECL).*

1 m of hydraulic heads. A slot was line-drilled into the floor and walls. To reduce the risk of EDZ-fracture clogging, the dams were constructed using AECL low-heat high performance concrete to eliminate calcium leaching, and potable water was sand filtered to eliminate the deposition of fines. A stable flow rate was reached 41 days after the reservoir was filled. The hydraulic conductivity of the EDZ fractures was estimated at $2.5 \cdot 10^{-8}$ m/s assuming that the EDZ was 0.1 m thick as estimated by visual inspection of the monitoring slot. The test set-up is illustrated in Figure 3-6.

The depth of the damaged rock as determined by a seismic refraction survey along the test tunnel was between 0.25–0.78 m from the periphery. Using other tools such as the micro-velocity probe and the SEPPI permeability probe (test length 50 mm) /Chandler et al. 2002/ typical results show influences close to one metre, see Figure 3-7. The figure illustrates a good correlation in between velocity and transmissivity increase. The visual inspection of damage by a borehole camera showed fractures of up to 0.3 m from the periphery as reported in /Chandler et al. 2002/

The extent of the damaged rock as measured by different measurement methods at the clay bulkhead characterisation array is shown in Figure 3-8.

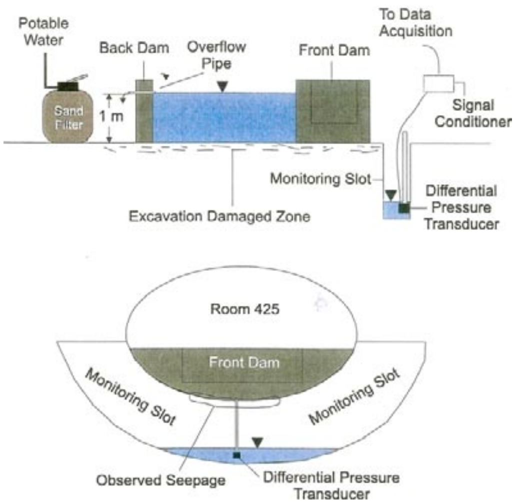

__Figure 3-6.__ Configuration of the Tunnel Sealing Experiment EDZ Seepage Characterisation Test /Chandler et al. 2002, Figure 3.18/.

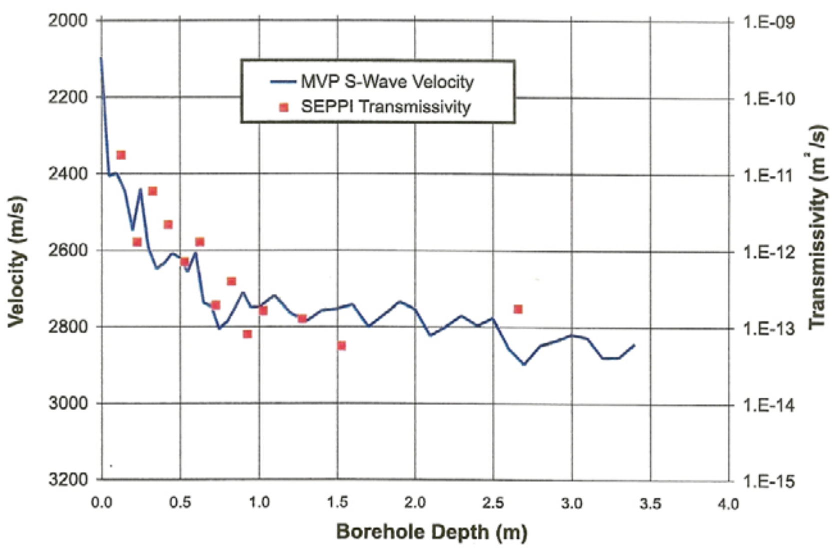

__Figure 3-7.__ Transmissivity as a function of depth for borehole MVP3 located in the wall of the tunnel for the Tunnel Sealing Experiment /Figure 3-14, Chandler et al. 2002/.

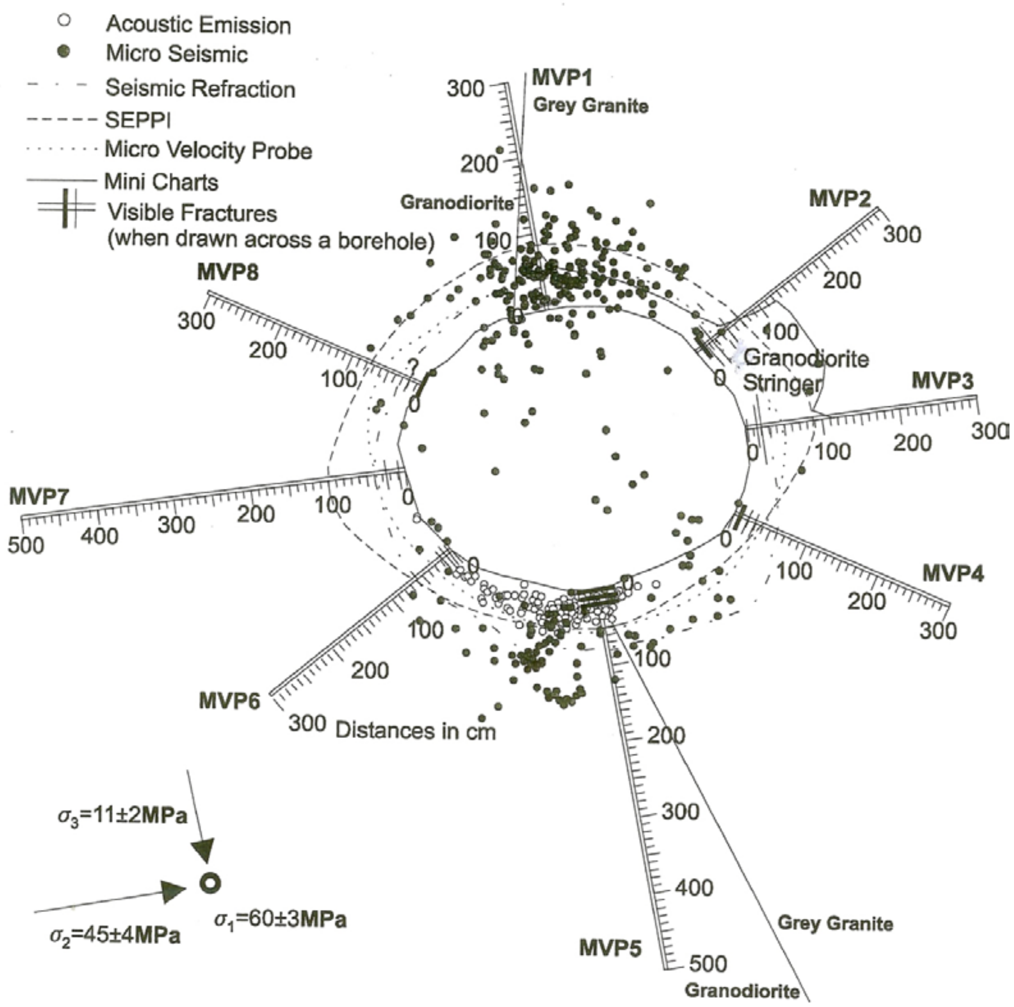

Figure 3-8. Extent of damaged rock at the clay bulkhead EDZ characterisation array. Summary from different measurement methods /Chandler et al. 2002, Figure 3.17/.

3.6 AECL – the Blast Damage Assessment Project

This project was developed at the AECL URL on the 240 Level in Room 220 over the period 2000–2003. The 240 Level is located in a region of lower stresses than the tests on the 420 Level (Mine-By and TSX) /Martino et al. 1997/. A tunnel 3.5 m in height and 4.5 m in width was excavated by drill and blast over a distance of 18 m and with the same shape as for the Tunnel Sealing Experiment tunnel and using the same full-face blasting method and controlled blasting as for the Tunnel Sealing Experiment tunnel. However, the visible half-barrels were higher, namely 74–91%. Connected permeability tests were conducted by the construction of dams and a trench in a similar fashion as for the Room 209 experiment (see Section 3.3) and the Mine-by experiment (see Section 3.4). The visible fractures as determined by a borehole camera were within 400 mm, the majority being hairline fractures and up to 0.6 m using a micro-velocity probe. After construction of the first dam, the hydraulic conductivity in the floor was between $1.8 \cdot 10^{-8}$ m/s $3.9 \cdot 10^{-8}$ m/s or $5.3 \cdot 10^{-8}$ m/s depending on the trench sector measured. After extension of the dam with a second dam, the test results for the same trench sectors were $2.4 \cdot 10^{-8}$ m/s, $19.6 \cdot 10^{-8}$ m/s and $79.9 \cdot 10^{-8}$ m/s respectively. The higher values of hydraulic conductivity for longer lengths contradict previous results and several hypothesises related to the impact of the 2nd dam construction were forwarded /Martino et al. 2004/. It was also suggested that the decrease in hydraulic conductivity measured for the Room 209 Connected Permeability Test (see Section 3.3) was a result of careful scaling[12] of the floor before construction of the 2nd dam rather than crossing a blast-round intersection.

[12] Scaling is the activity to identify and break away "loose rock", and is a standard maintenance operation in all underground unlined openings.

An interesting part of the blast damage assessment project was the studies on bioorganics and, specifically, how blasting residues (nitrates) might affect microbial viability and growth. It was concluded that microbial growth can be enhanced one to four orders of magnitude for nutrient-poor ground waters.

3.7 SKB – the ZEDEX project and other miscellaneous tests at Äspö HRL

The objectives of the **Z**one of **E**xcavation **D**isturbance **Ex**periments (ZEDEX) experiments performed during 1994–1997 /Emsley et al. 1997/ were to:

- understand the mechanical behaviour of the EDZ with respect to origin, character, magnitude of property change, extent and its dependence on excavation method;
- perform supporting studies to increase the understanding of the hydraulic significance of the EDZ;
- test equipment and methodologies for quantifying the EDZ.

The test set-up at the Äspö Hard Rock Laboratory was truly unique: Two parallel tunnels were excavated by drill and blast and a tunnel boring machine respectively. A series of measurements were performed before, during and after excavation. The drill and blast tunnel diameter 5 m, flattened in the floor (area 18 m^2) was excavated in ten rounds using two alternate blasting schemes. Normal smooth blasting and low-shock energy smooth blasting (both ~ 2.4 kg explosives per m^3) with charges in contour holes of around ~ 0.2 kg/m, except for the floor. The TBM tunnel was excavated by a TBM machine (diameter 5.03 m, area 20 m^2). 34 cutters were used with a force of ~140–190 kN per cutter.

The damage (i.e. increased fracturing) was concluded to be within 0.3 m in the walls and roof of the drill and blast tunnel and up to 0.8 m in the floor. For the TBM tunnel the damage was within 0.03 m.

It was not possible to make a set-up so the hydraulic conductivity could be measured in the close vicinity before and after the excavation. A large number of hydraulic pulse tests were collected from 26 tested short radial boreholes after the excavation work had been completed, but the results failed to indicate any clearly defined and significant increase in the permeability of the rock mass in the vicinity of the tunnels – the "damaged zone". The highest measured permeability measured close to the tunnel was approximately $1 \cdot 10^{-16}$ m^2, corresponding roughly to a hydraulic conductivity of 10^{-9} m/s, see Figure 3-10.

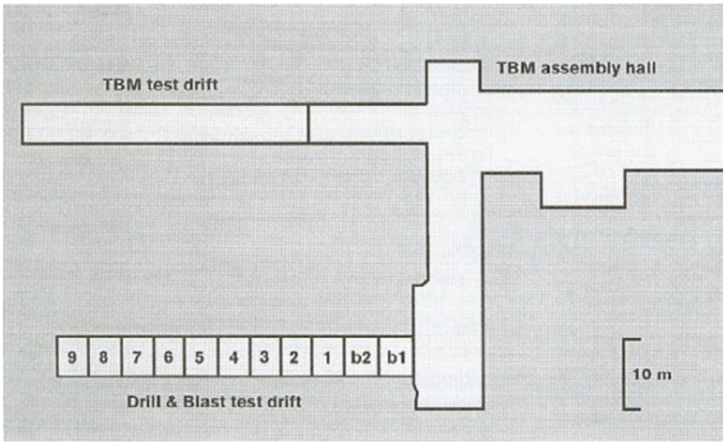

Figure 3-9. *Set-up of two test tunnels at the Äspö Hard Rock Laboratory at a depth of 420 m below the surface. Rounds 1-4 involved the use of low-shock explosives and Rounds 5-9 the use of "normal smooth blasting" techniques.*

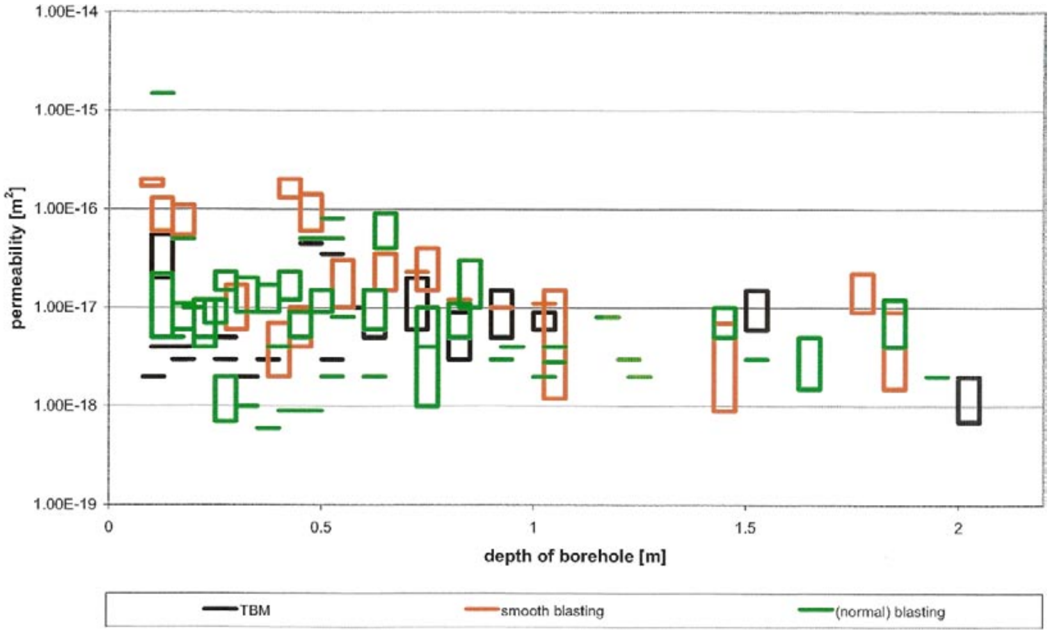

Figure 3-10. *Compilation of measured permeabilities in situ from the ZEDEX-project /Nowak 2005/.*

Detailed measurements of the hydraulic conductivity of the damage zone were also reported by /Sabet et al. 2005c/ and /Pusch 2008/ Samples were prepared from 100 mm cores with a length of 250 to 500 mm taken perpendicularly to the rock wall. Several series of 10 mm diameter cores were extracted by diamond drilling perpendicular to the large cores, i.e. parallel to the tunnel at different distances from the tunnel wall, and several series of 3 mm discs were sawn from the large cores to allow determination of hydraulic conductivity perpendicular to the tunnel wall. Hydraulic testing of the samples used a triaxial apparatus and a hydraulic gradient of about 100. The investigations showed that the isotropic hydraulic conductivity was 10^{-9} m/s for a distance of 1–2 mm from the TBM tunnel periphery, $2 \cdot 10^{-11}$ m/s at a depth of 5 mm, and 10^{-13} m/s at a distance of 30 mm. The virgin “crystal matrix” of the rock is in the range of 10^{-13} m/s to 10^{-14} m/s. Fluorescence microscopy of epoxy-impregnated samples gave the porosity and showed the pattern of fissuring caused by the bits. The majority of the fissures formed an angle of ±25–45° to the rock wall and were responsible for the increase in porosity from less than 0.5% of the undisturbed crystal matrix to 2–5.6% within a distance of 10 mm from the wall. A number of macroscopic fractures were also identified, their depth and spacing being 10–50 mm, and were assumed to cause an additional increase in hydraulic conductivity of larger volumes of the matrix than those represented by the small samples, and hence yield values in the same order of magnitude as the ones obtained from the small-scale packer tests.

The hydraulic measurements in the “disturbed zone” were conducted by packer tests and difference flow measurements in sub-parallel boreholes before and after the excavation. Out of 19 hydraulic build-up tests, three intervals showed a decrease in transmissivity by half an order of magnitude, and two intervals a decrease of two orders of magnitude. The remaining 14 intervals showed no significant change. The conclusion drawn from the evaluations was that no statement could be made regarding the trend in transmissivity properties before and after excavation.

The ZEDEX results for the “damage zone” are summarised in Figure 3-11 as they were used in the SR 97 Safety Assessment report /SKB 1999/. It should also be added that the ZEDEX project concluded that the “disturbed zone” could be characterised by changes in state, which could be considered to be reversible. It was also concluded that the changes in rock properties and rock stress with distance from the rock wall of excavation is gradational, and there is hence no distinct boundary between the zones.

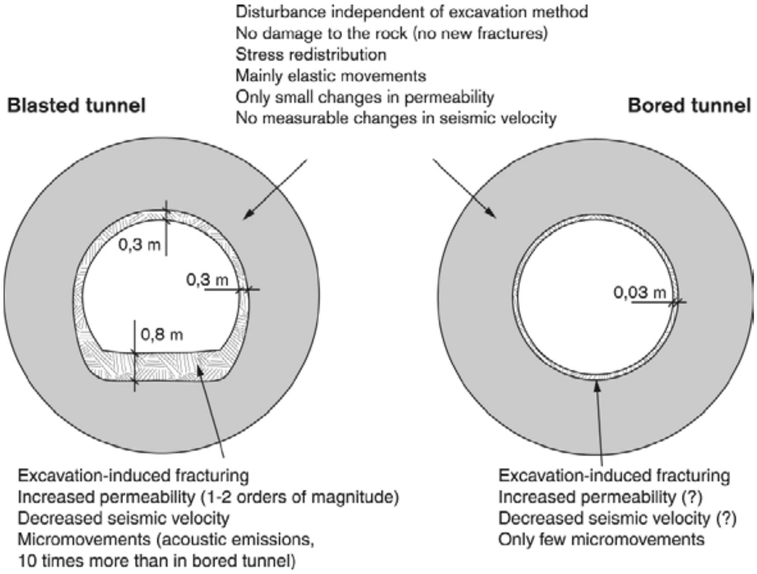

Figure 3-11. *Compilation of the ZEDEX results /SKB 1999/. The hatched area represents the "damage zone" and the grey area the "disturbed zone".*

In connection with the Backfill and Plug Tests at Äspö HRL situated in the drill and blasted ZEDEX tunnel, several instrumentation boreholes were hydraulically tested /Ludvigsson et al. 2001/. The test involved 23 boreholes of 1 m in length and 14 longer boreholes c. 5 m, 8 m and 25 m long. The Backfill and Plug Test was executed in the drill and blast tunnel from the ZEDEX-test (see Figure 3-9). The tested intervals in the short 1 m holes were 0.3–0.7 m in the rock, perpendicular to the tunnel face. However, during testing in some of the 1 m-holes, visible rock leakage occurred through superficial, probably blast-induced fractures in the vicinity of the tested boreholes, generally located in the tunnel floor. The hydraulic conductivity of these boreholes was high, above the practical upper measurement limit of the actual test system used ($K_{max} \approx 1 \cdot 10^{-7}$ m/s). The median value of the estimated hydraulic conductivities of all 1 m-boreholes tested (N=23) is $4 \cdot 10^{-10}$ m/s. The median hydraulic conductivity of the 1 m-holes in the roof (N=5) and walls (N=10) is $7 \cdot 10^{-11}$ m/s and $4 \cdot 10^{10}$ m/s, respectively. In the tunnel floor, four holes from a total of eight tested holes have hydraulic conductivities above the upper measurement limit. If the tests showing visible rock leakage are excluded, the geometric mean of the estimated hydraulic conductivities of the remaining 1 m-holes is $K_{gm} = 2 \cdot 10^{-10}$ (N=17). This value is also uncertain since several of the tests had conductivities below the practical lower measurement limit of the test system (K_{min} = c. $1 \cdot 10^{-10}$ m/s). The tests in the longer boreholes showed that the hydraulic conductivity further into the rock is in general below c. $1 \cdot 10^{-10}$ m/s. These results are apparently inconclusive with respect to the excavation damaged zone.

/Autio et al. 2005/ presents results from the Äspö TBM tunnel and the Äspö Prototype Repository deposition holes. Five rock samples from the TBM-tunnel at Äspö were studied. The porosity in the EDZ with respect to distance from the excavated surface was similar in all samples and decreased from around 1% to around 0.2% at a distance of 20 mm from the periphery. A total of 12 samples were taken from the experimental deposition holes. The thickness of the zone (EDZ) with a significantly higher porosity than in the undamaged zone extended about 20 mm from the TBM-excavated surface, which was similar to the results found in the experimental deposition holes in the Research Tunnel at Olkiluoto, see Section 3.11.

The paper by /Liedtke 2005/ presents a few test results from near-surface measurements in the TBM-drilled Äspö Prototype Repository Tunnel with a diameter of 5.0 m. Ten surface packer tests were carried out at five different locations. Each location was tested with water and then with gas. The results from the water and gas tests were quite similar and resulted in a permeability of $1–2.5 \cdot 10^{-17}$ m/s (hydraulic conductivity of $1–2.5 \cdot 10^{-10}$ m/s over a depth of 10 mm. The completely undamaged rock extending over this zone was assigned a value of 10^{-20} m^2.

3.8 SKB – Äspö Pillar Stability Experiment and associated EDZ studies in the TASQ tunnel

The Äspö Pillar Stability Experiment (APSE) was carried out during the years 2002–2006 to examine the failure process in a heterogeneous and slightly fractured rock mass when subjected to coupled excavation-induced and thermal-induced stresses /Andersson 2007/. A basic idea was to test the capacity to make predictions of spalling along a deposition hole and further build up the predictive confidence of spalling processes that was created from the Canadian Mine-by experiment, see also Section 3.4. The pillar at Äspö HRL was created by the excavation of two large boreholes (Ø 1.75 m, 6.5 m deep) so that a rock web of ~ 1 m was left in between them. The experiment was located in a tunnel excavated for the experiment denoted TASQ. The floor was arched to concentrate the excavation-induced stresses in the centre of the floor. Acoustic emission, displacement and thermal monitoring systems were installed to follow the yielding of the pillar as the temperature was increased. The pillar was heated by electric heaters so that thermal stresses were induced which caused the pillar wall in the open hole to yield gradually and in a controlled manner.

The yielding propagated down along the pillar wall and created a v-shaped notch, see Figure 3-12. The first of the two large holes was confined with a water pressure before the excavation of the second hole commenced. This was done to enable the effect of a confinement pressure on the response of the rock mass to increased loading to be studied.

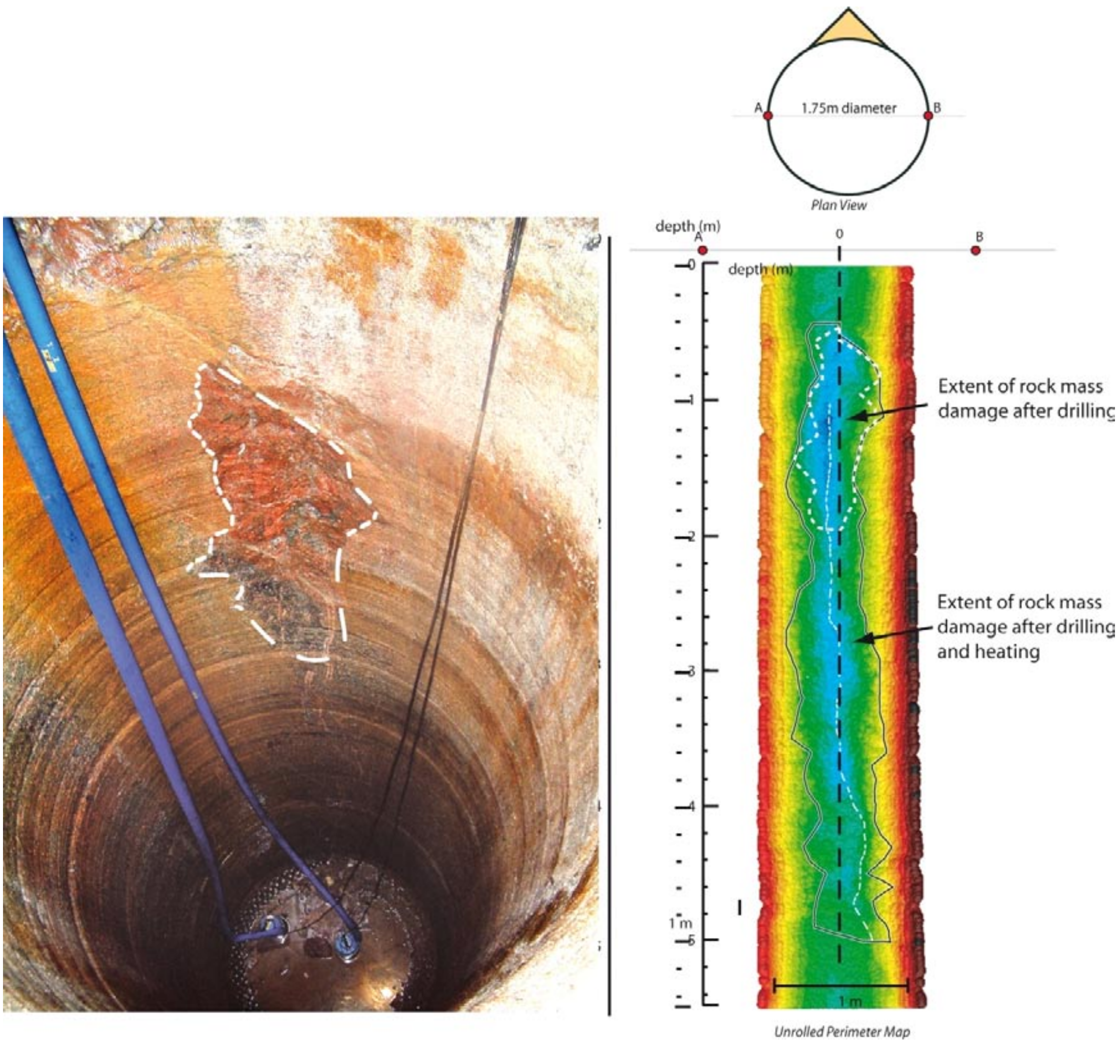

Figure 3-12. Photograph of the rock volume that spalled during the excavation of the second large hole. The total spalled area after heating as derived from a laser scanning of the pillar wall is presented in the right-hand part of the figure /Andersson 2007/.

The effect of the confinement pressure was obvious as soon as the excavation of the second hole started. Acoustic Emission (AE) events were recorded in the unconfined hole but not in the confined one. During the hole-heating period of the experiment, the AEs in the confined hole were only a fraction of those in the unconfined hole.

Acoustic emissions provided a good approximation of the general yielding rate in the pillar. However, it was found during the analysis of the data that the AEs could not be correlated to the amount of damage to the rock or to the monitored displacements. It was concluded that fracturing and displacements in many cases occurred without being registered by the AE system.

The monitored temperatures were used to back-calculate the temperature in the experimental volume. Coupled modelling was used to determine the increase in thermal stress in the pillar. By combining these stresses with the excavation-induced ones, the total stress in the pillar could be determined at all times. When correlated with these data, observations of when and where the rock yielded gave the yield strength of the rock. This strength was determined at 18 different locations on the pillar wall. The mean value was $0.58\sigma_c$, with a standard deviation of $0.04\sigma_c$, where σ_c is the unconfined compressive strength. This value was correlated to the crack initiation stress (CIS) determined by the volumetric strain method on core samples taken from the experimental volume. The mean value of the CIS was $(0.45 \pm 0.03)\sigma_c$ and /Andersson 2007/ then recommended that the crack volumetric strain method be used to estimate the yield strength of a rock mass in the absence of in situ data.

As mentioned before, the TASQ tunnel and the APSE experiments were also used to study the EDZ. The excavation for the APSE project in the Äspö Hard Rock Laboratory during year 2003 was specially designed to reduce damage in the tunnel floor. The limitation of excavation damage was achieved by excavating a traditional horseshoe-shaped tunnel, 5.5 m in length with separate blasting of the bottom bench with a radius of 2.5 m. The later excavation of two 1.75 m diameter and 6 m deep experimental holes for the APSE experiment using mechanical excavation by a Shaft Boring Machine allowed observations to be made of the excavation damaged zone in the floor on a larger scale than by core mapping and by cutting out slices of rock from the EDZ. Observations in the two large holes in the floor of the APSE tunnel seem to confirm the results of the previous core mapping; the EDZ has a varying depth and is to a high degree controlled by the presence of the pre-existing natural sub-horizontal fractures under the floor /Bäckblom et al. 2004/.

The experience from excavation of the TASQ-tunnel for the APSE experiment was reported in detail in /Olsson et al. 2004/. There were high requirements on borehole precision and of a minimized excavation damaged zone in the APSE pillar area. This included a maximum borehole deviation of 10 mm/m, a maximum overbreak due to the lookout angle of 0.3 m and a damage zone less than 0.3 m. To make the charge control feasible, cartridged explosives were prescribed. The conclusions with respect to excavation included:

- To excavate with a top heading and bench gives significantly lower damage in the floor compared to ZEDEX experience, even less than in the roof and walls. This is primarily caused by the difference in specific charge in the contour holes for the floor. Further development in blast design is needed to enable similar results in terms of a small excavation damage zone in the floor without excavation of a separate bench.
- For the "average" tunnel construction, based on current Swedish practice the observed excavation damage is similar to that observed in the ZEDEX drill and blast tunnel.
- A systematic use of an electronic initiation system in the contour seems to be promising for a further reduction in the extent of the excavation damaged zone.
- Large drill hole deviations have caused significant local damage.

Soon after the tunnel was completed, 13 cores were drilled in the floor for detailed planning of the APSE experiment, and for instrumentation. Possible induced fracturing was found to a depth of normally not more than 0.3 m. In addition, when drilling the two 1.8 m diameter holes in the floor for the APSE experiment the upper part of the floor could be studied, see Figure 3-13.

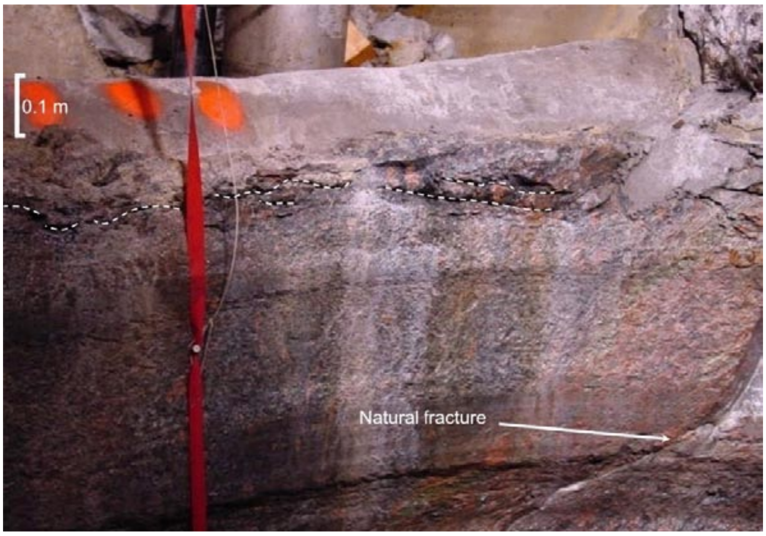

Figure 3-13. Photo of the upper part of a 1.75 m diameter hole in the floor of the APSE tunnel. The concrete slab on the floor was cast for setting up the Shaft Boring Machine. The slab is approximately 0.15 m thick. The damaged zone along the photo (covering 1 m) varies from 0.1 m (centre of photo) to 0.3 m (right side). /From Bäckblom et al. 2004/.

The observations from the large size holes support the observations from the cored holes. To further study the excavation damage due to the drilling and blasting, eight slots were prepared in the walls and in the floor. The slots were 0.5 m deep and made by sawing a number of parallel cuts into the rock. Normally two horizontal cuts and 4–5 vertical cuts have to be sawed. The distances between the cuts are adjusted to the force required to loosen the rock between the cuts. Then the slices are pried loose by wedges and removed. Finally a dye penetrant is sprayed on to the cleaned surface causing the cracks to appear very clearly, see the example in Figure 3-14.

Cracks were analysed with respect to crack length and direction of the walls relative to the vertical walls. No evidence was found of a continuous damage zone parallel to the tunnel wall.

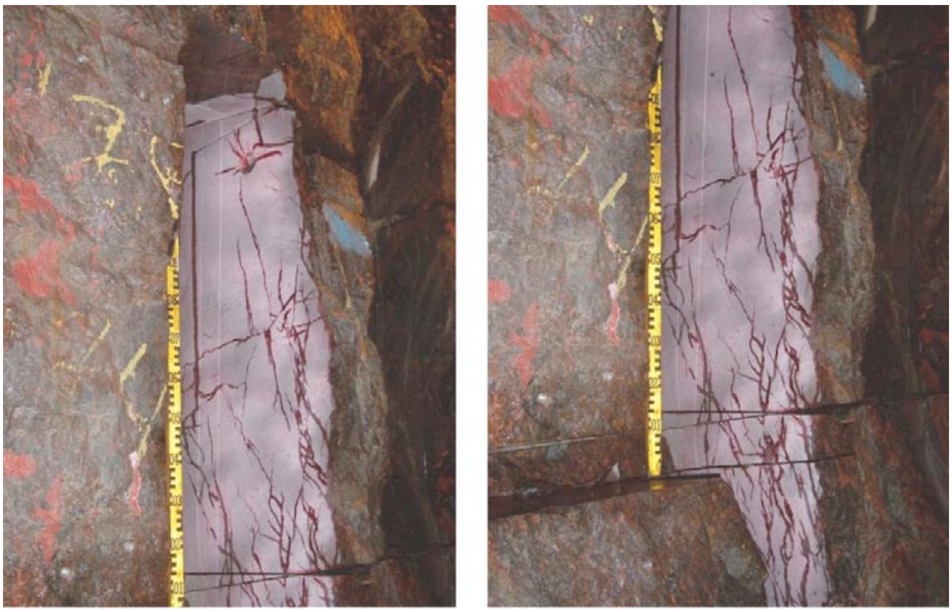

Figure 3-14. Slots are prepared and fractures dyed. The left-hand photo shows cracks in the upper part of the right wall and the right-hand photo cracks in the lower part of the right slot wall. After /Olsson et al. 2004/.

In the previous investigations /Olsson et al. 2004/ slots and slabs were cut in tunnel walls at the Äspö HRL to study the EDZ, see for example Figure 3-15. The slot and individual slabs were now investigated to establish the 3D fracture network. The report /Olsson et al. 2008/ describes the methodology tested and proposals for its developments. It was concluded that it is possible to prepare a 3D model of the blasting-induced fractures, but the data are likely to be biased as only the most prominent fractures would be modelled.

A typical result from the first methodology test is shown in Figure 3-16. Work is in progress at Äspö HRL to evaluate the connectivity of blasting-induced fractures over several blasting rounds, using the methodology presented.

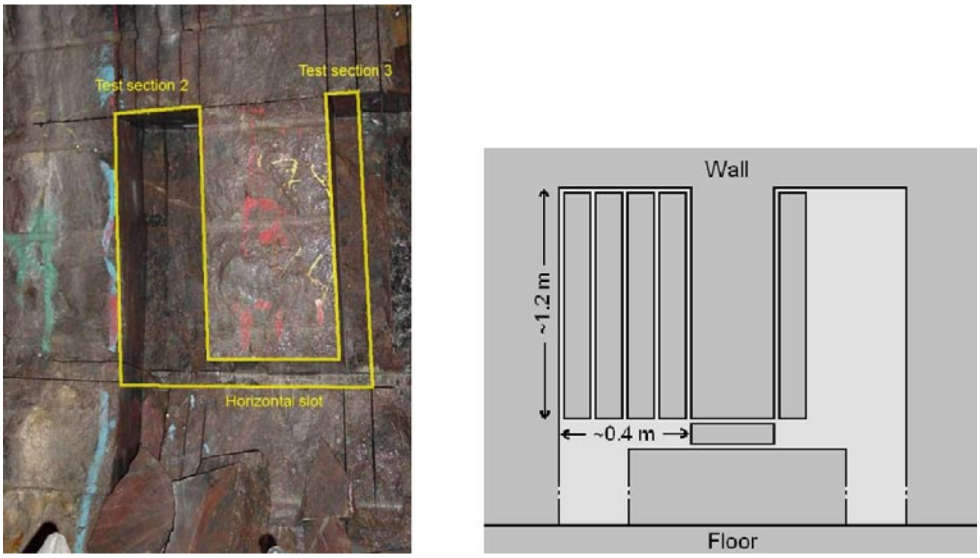

Figure 3-15. *Left: Photo of test area with some of the slab. Right: Schematic vertical view of the investigated area. After /Olsson et al. 2008/.*

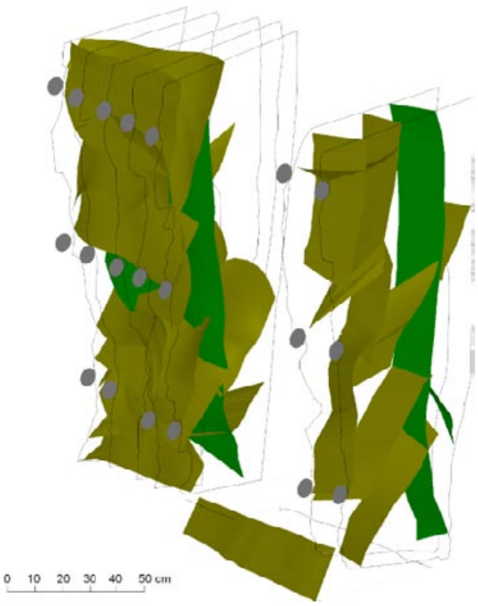

Figure 3-16. *The complete model created. Fractures interpreted as blast induced are marked in dark green and natural fractures in light green. The grey circles represent positions of blast holes at the periphery of the tunnel. After /Olsson et al. 2008/.*

The TASQ tunnel at the Äspö HRL was developed for the APSE project (see Section 3.8). A section along the TASQ analysis was scanned by a laser to obtain a high-resolution picture of the periphery of the tunnel. In this particular section of tunnel, ultrasonic measurement was previously carried out by the Federal Institute for Geosciences and Natural Resources (BGR) in eight boreholes. The data sets for the selected section were re-examined and cross-correlated as input for detailed 2D and 3D numerical modelling of the mechanical response to creation of the TASQ tunnel /Jonsson et al. 2009/. Based on the cross-comparisons, the authors suggest that mechanical damage can be detected with high accuracy using the ultrasonic method. Based on the numerical analyses of stress and strain due to creation of the opening, the authors also suggest that the uneven as-built geometry of the tunnel generated from blasting also contributes to the heterogeneous stress redistribution close to the periphery of the opening. These variations in stresses are local and differ strongly between nearby sections, which could induce local fracturing and asymmetry in the EDZ. The importance of minimizing borehole deviations is emphasized.

3.9 Nagra – Tunnel Near-Field Programme

During period 1994–1996, Nagra, in co-operation with other organisations, executed a major Tunnel Near-Field Programme (TNFP) at the Grimsel Test Site (GTS) /Marschall et al. 1999/. One important part of the TNFP was the study on the development of conceptual and numerical models of the rock mechanical and hydraulic behaviour of the EDZ around tunnels and caverns. The programme also, for example, engaged in development and testing of techniques, which are applicable in hydraulics, geophysics and rock mechanics for characterizing the near-field of tunnels and caverns. The overall geology for the locations of experiments is shown in Figure 3-17 and the TNFP research topics and projects in Figure 3-18. The EDZ investigations are from three test zones, the EDZ site in the WT tunnel being bored by a TBM and the ZPK site (drill and blast) between the BK and GS sites and within the BK site (drill and blast).

The EDZ tunnel was deliberately drilled in the direction of the expected minimum horizontal stress and in an area where an unusual level of tunnel wall damage occurred in order to be able to test "maximum impact" of the EDZ. The tunnel diameter is 3.5 m. The ZPK was excavated by drill and blast in a 4 m square section. The report by /Marschall et al. 1999/ unfortunately does not provide any information on excavation, but it is here assumed that smooth drill and blast was used to reduce the damage zone.

The investigations of the "rock matrix" were conducted using optical microscopy of thin sections from borehole cores, single and cross-hole sonic measurements, core gas permeability measurements and surface and short interval air and water packer tests. The fractures in the "disturbed zone" (supposedly also including the "damaged zone") were investigated using geological mapping from core and tunnel walls, borehole logging using television systems, sonic measurements in boreholes and packer tests using air and water in selected intervals containing fractures.

The studies at the different sites failed to show any significant differences in matrix properties for different excavation methods. The measurements showed the matrix permeability in the order of $3 \cdot 10^{-18}$ m^2 for 0 to 1 m around the tunnel wall based on 32 tests, which is a factor three times greater than the estimated undisturbed rock permeability. Three tests in the EDZ site at a distance of 2.0–2.5 m from the tunnel wall showed the permeability to be in the order of $2\text{–}3 \cdot 10^{-19}$ m^2. The fracture permeability of the damage/disturbed zone was also tested and was thought to extend up to 2 m from the EDZ tunnel. Based on 15 test intervals, the arithmetic mean permeability is $2.2 \cdot 10^{-15}$ m^2, (corresponding to the hydraulic conductivity of $2.2 \cdot 10^{-8}$ m/s) albeit with a large standard deviation. The measured permeability is 2,000 times larger than the background effective permeability of the rock mass (i.e. excluding shear and fracture zones). As noted in Table 6.3 in /Marschall et al. 1999/ these data are from a point "where an unusual level of spalling occurred". The results from the EDZ tunnel are shown in Figure 3-19.

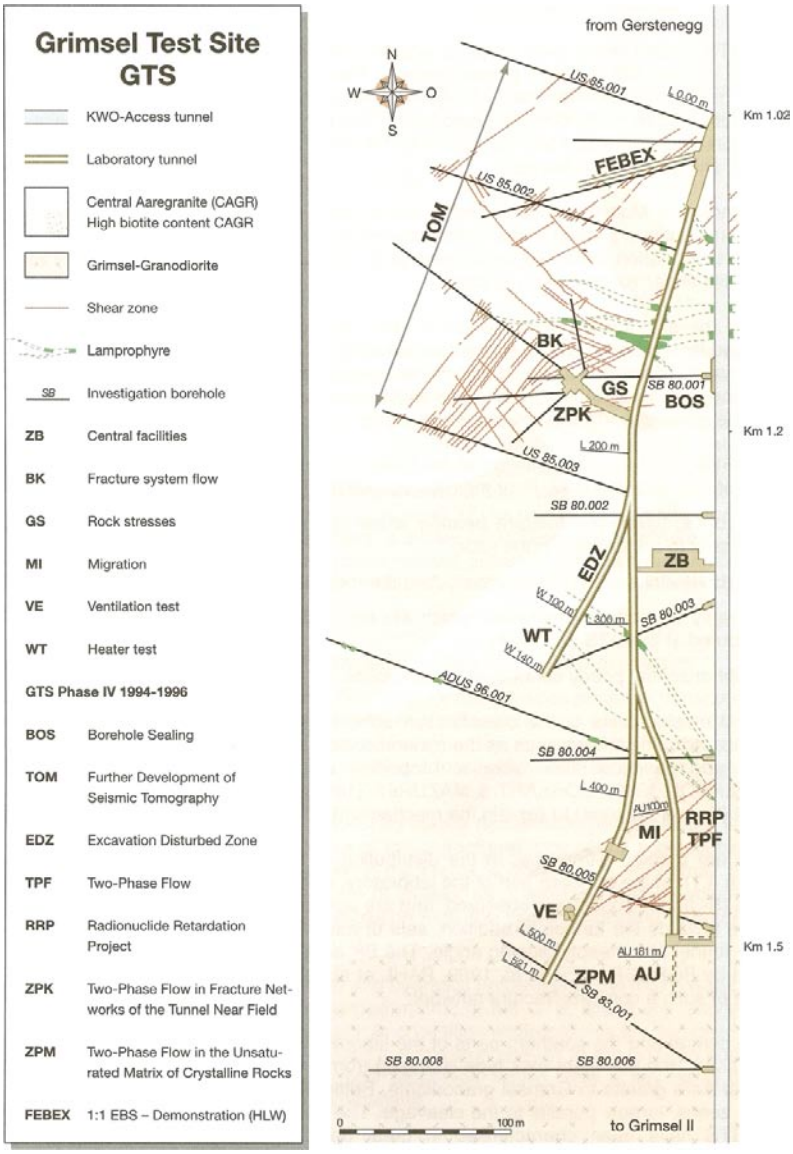

__Figure 3-17.__ Geological overview and locations of experiments at the GTS /Figure 4.1, in Marschall et al. 1999/.

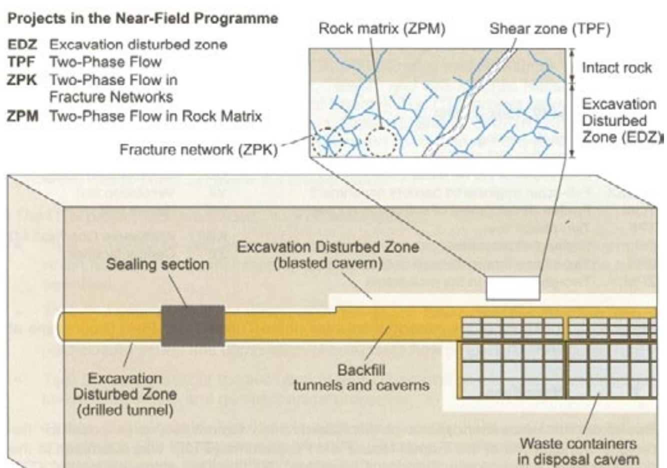

__Figure 3-18.__ Research topics and projects in the Tunnel Near-Field Programme at the GTS /Figure 1.2 in Marschall et al. 1999/.

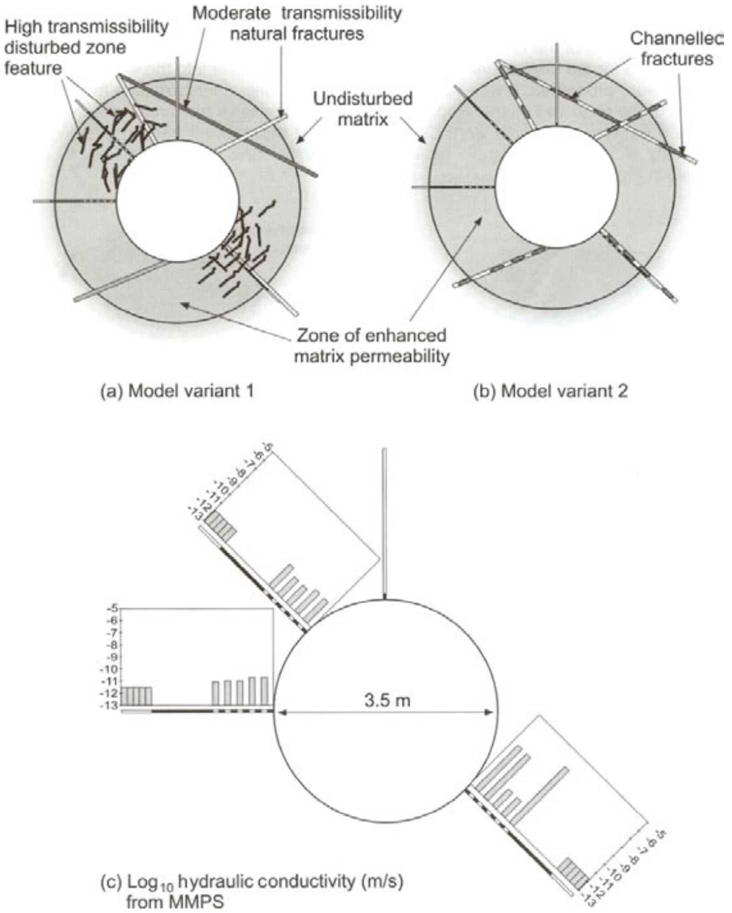

Figure 3-19. *Model variants used in axial flow calculations and field data from the GTS: (a) Variant 1. High transmissivity values are disturbed zone features. (b). Variant 2. High transmissivity values channels in natural fractures. (c). Hydraulic conductivity profiles as measured in the EDZ/WT area /Figure 5.8 in Marschall et al. 1999/.*

Two more comments are added here: The sonic measurements in the ZPK site showed considerable reduction on Pressure-wave and Shear-wave velocities in the regions with 0.5 m of the tunnel. However, after sealing and re-saturation of the rock, these effects were considerably reduced. The authors /Marschall et al. 1999/ also argue that many of the high permeability intervals measured are correlated to damage zone fractures along the tunnel. If *"these fractures are locally channelled and possibly stress-relieved parts of pre-existing fractures that cut across the tunnel rather than run along its length, then the effective axial permeabilities will be much lower and comparable to that of the enhanced matrix permeability of* $3 \cdot 10^{-18}$ m^2."

3.10 Enresa – FEBEX

Parts of the FEBEX (Full-Scale Engineered Barriers Experiment in Crystalline Host Rock) were executed at the Grimsel Test Site in Switzerland during 1994–2004. The FEBEX experiment was a 1:1 simulation of a High Level Waste disposal facility after the Spanish concept. A tunnel, 2.28 m in diameter and 70 m long, see Figure 3-20, was specifically excavated for this test (FEBEX tunnel) by using a mini-tunnel boring machine. In the vicinity of the FEBEX tunnel, the Aare granite is dominant. Performance of the experiment implied placing two electric heaters of a dimension and weight equivalent to those of the canisters in the concept. The entire space surrounding the heaters was filled with blocks of compacted bentonite to complete the 17.4 m of barrier for the test section. The test zone was closed with a concrete plug. The experiment was monitored by more than 600 sensors reading continuously relevant parameters. After a period of 6 years of heating, the experiment was partially dismantled in 2002 within the framework of the FEBEX II project, allowing the launching

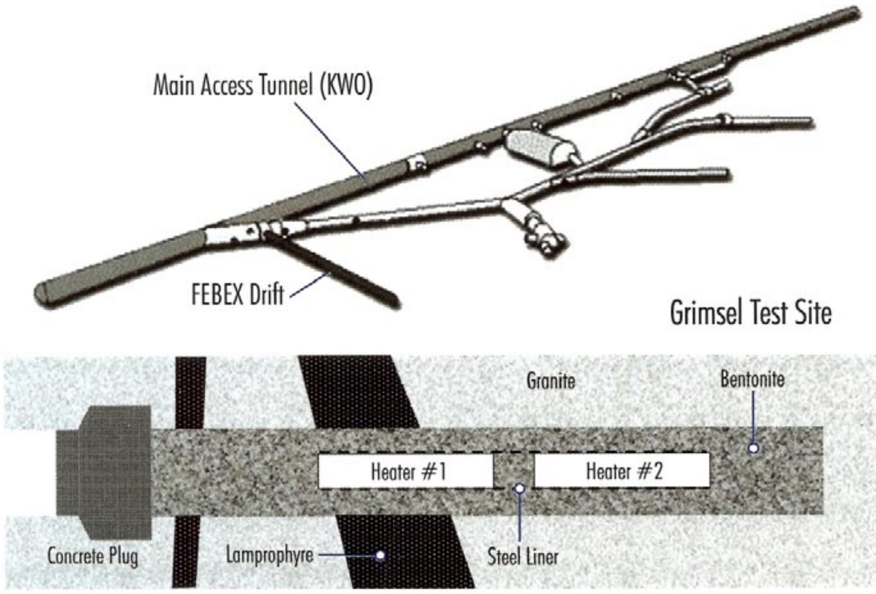

Figure 3-20. *General layout of the FEBEX experiment /Sabet et al. 2005b/, see also Figure 3-17.*

of a post-mortem analysis. Among the studies performed, a research group investigated the existence and the potential evolution of the excavation-damaged zone (EDZ) around the FEBEX tunnel. /Sabet et al. 2005ab/

Extensive investigations concerning hydrogeological studies were performed in the first phase of the FEBEX project (1994–1999). The hydraulic tests were performed in sections isolated by packers in the totally 23 boreholes. The most frequent hydraulic conductivity value in the tests was 10^{-11} m/s[13]. This value may be considered as representing the hydraulic conductivity of the rock matrix. Thus, it was concluded that the permeability of the granite matrix drilled by TBM is comparable to that of the undisturbed rock mass. It was also concluded that the probability of the existence of an axial flow pathway along the tunnel is low as the observed fractures are closed or filled with secondary materials.

Several methods were then used (surface packers, ^{14}C-PMMA, acoustic tomography; SEM/EDX) to gather data, and the conclusion was that a close look to the first millimetres of the granite matrix by using the SEM method indicates the existence of an EDZ in the granite matrix, which extends no further than 1–2 mm from the tunnel wall.

/Sabet et al. 2005a/ found no evidence of EDZ in the granite matrix close to the tunnel (< 10 cm), except for a 3 mm deep zone as determined by SEM/EDX analysis. The very low impact at the FEBEX tunnel was attributed to the TBM excavation, the low number of natural fractures and natural joints, the regularity of the structural texture of the Grimsel granite as emphasized by the homogeneity and the smallness of the cuttings observed during the excavation of the FEBEX gallery. These factors must also play an important role in the absence of EDZ beyond a skin of few mm /Sabet et al. 2005b/.

3.11 Posiva – Olkiluoto deposition holes

The damage caused by excavating deposition holes by hydraulic push-reaming was investigated by /Autio 1997/. A Research Tunnel was built at the VLJ repository and three mock-up deposition holes with a nominal diameter of 1,524 mm and a depth of 7.5 m were excavated in tonalitic rock. In addition to standard mapping methods, ^{14}C-PMMA and He-gas methods were deployed to reveal a distinct zone of changes in porosity, permeability and effective diffusion coefficient extending 8–34 mm from the periphery. The tests were made on 98 mm core samples at different locations in the hole (walls, floor).

[13] The detailed measurement results are not found in the references.

The disturbed zone in the walls was a crushed zone 1–3 mm from the surface, a fractured zone 6–10 mm from the surface and a fissured zone 12–31 mm from the surface. The measured values of the disturbed samples were an order larger than those for the undisturbed rock. The average value for the permeability was $7 \cdot 10^{-20}$ m^2 (hydraulic conductivity of $7 \cdot 10^{-13}$ m/s) and around $2 \cdot 10^{-9}$ m^2/s for the effective diffusions coefficient.

The results were later extended by additional investigations /Autio et al. 2005, Autio et al. 2006/ comparing them with Äspö data and drawing conclusions with respect to gas migration in the EDZ for a KBS-3H type repository – *"The results of the analysis show that the EDZ is capable of transporting the gas gradually away from tight hole sections without significantly disturbing the isolation characteristics of the repository system."*

Based on the study of samples taken from the surface of TBM tunnel and experimental deposition holes at Äspö HRL, see Section 3.7, it was concluded /Autio et al. 2005/ that the EDZ caused by TBM excavation and deposition hole boring is very similar.

3.12 JAEA – Kamaishi excavation disturbance experiment II

Over the period 1993–1998, JAEA (formerly, JNC, formerly PNC) conducted a series of experiments in the Kamaishi mine related to EDZ /Matsui et al. 1998/. It was concluded that the intrinsic properties of the rock were changed up to 1.0 m from the tunnel periphery. A connected permeability test was conducted in a similar fashion as the tests at AECL. Based on connected permeability tests and hydraulic tests in short sections (10 cm) it was concluded that there is low correlation between low velocity and hydraulic conductivity except for measurements in the floor, where the hydraulic conductivity is two orders higher (around 10^{-5} m/s) than in the undamaged part (10^{-7} m/s). The values in the damaged zone are higher than in the similar AECL connected permeability test. This is thought to a consequence of the high-stress situation. The EDZ II tunnel had an overburden of more than 800 m and spalling occurred in the roof of the tunnel (Matsui H, JAEA, pers. comm.). The situation in that case is similar to the AECL Mine-by test, in which the EDZ was due to spalling. The EDZ II results could be interpreted as a combination of spalling effects and blasting damage effects.

3.13 Compilation of results from major experiments

The measured results from the major experiments on stress- and excavation-induced damage are compiled in Table 3-3. The table distinguishes excavation method, an estimated equivalent test scale as well as the mechanical and hydrogeological results. It is apparent that many of the consequences can be mitigated by design either by selection of excavation method or by selection of rock stress environment.

For rock in a lower stress state, i.e. no spalling environment, the typical damage is dependent on the excavation method. Using mechanical excavation the damage is only a few centimetres and with hydraulic conductivity in the order of 10^{-10} m/s (Äspö) or lower (Olkiluoto and Grimsel, FEBEX). The values of the overall permeability (conductivity) are higher in the Near-Field Tunnel Programme at the Grimsel Test Site, where the test sections are deliberately situated in areas with an unusual level of tunnel damage. It is assumed that the damage for some parts of the tunnel is more likely to be a stress-induced phenomenon than excavation method-induced. The extent is up to 2 m and with a corresponding hydraulic conductivity of around $2.2 \cdot 10^{-8}$ m/s.

A typical result for the openings excavated by drill and blast is a damage zone up to several tens of centimetres wide in which the damage progressively diminishes with the distance from the opening. The extent of the damage is dependent on the design and execution of the drill and blast operation, but it is assumed that the damage is insignificant beyond 0.8 m from the periphery. The measured hydraulic conductivity is in the order of 10^{-8} m/s.

In the case of spalling, the hydraulic conductivity is substantial compared to the virgin rock properties even if mechanical excavation is used, and the results suggest a hydraulic conductivity in the order of 10^{-6} m/s for a "process zone" some 200 mm deep.

Table 3-3. Overview of main results for the "spalled "or "damage" zone; See Sections for references

Experiment	Excavation method	Equivalent Test Scale [m]	Mode	Mechanically defined extension of damage [m]	Hydraulic parameter results
Stripa – Rock Sealing Experiment	Drill and Blast,	~ 10–15 m The test covers approximately four rounds	Excavation-induced	0–0.8 m	Hydraulic conductivity $1\cdot10^{-8}$ m/s but $2\cdot10^{-8}$ m/s in the floor. The hydraulic conductivity of the most shallow 1–2 dm rock is probably $5\cdot10^{-8}$–$1\cdot10^{-7}$ m/s.
AECL – Room 209 Connected Permeability Test	Drill and Blast	~ 0–5 m for connected permeability tests ~ 0.03 m for borehole permeability tests. The test covers 1–2 rounds	Excavation-induced	0.3 m in the floor	Hydraulic conductivity "increased 1 to 2 orders" (i.e. around 10^{-12}–10^{-11} m/s) for the 0.3 m closest to the periphery based on permeability tests and transient pulse tests. However, the hydraulic conductivity based on flow measurement was estimated to be approximately 10^{-8} m/s.
AECL – Mine-by Tunnel Connected Permeability Test.	Mechanical excavation using line-drilling	~ 0–5 m for connected permeability tests	Stress-induced with spalling	"Process zone" due to spalling of width 0.2 m (extending beyond the break-out notch 0.3 m below the design level of the floor	Hydraulic conductivity 10^{-6} m/s based on flow measurements and $0.7\cdot10^{-6}$ m/s based on tracer tests. Transport porosity 3.3% and longitudinal dispersivity of 0.6 m.
AECL – Tunnel Sealing Experiment	Drill and Blast	~ 0–5 m The test covers 1–2 rounds	Excavation-induced	< 1 m: The blast-induced damage is 0.3 m as concluded by AECL	Hydraulic conductivity of $2\cdot10^{-8}$ m/s.
AECL – Blast Damage Assessment Project	Drill and Blast	~ 0–5 m The test covers 1–2 rounds	Excavation-induced	< 0.6 m	Hydraulic conductivity in the order of $2\cdot10^{-8}$ m/s to $8\cdot10^{-7}$ m/s.
SKB – ZEDEX	Drill and Blast	~ 1 m The test covers 11 rounds	Excavation-induced	0.3 m in walls and 0.8 m in the floor	Permeability of approximately 10^{-16} m^2 for a few measured sections (corresponding to a hydraulic conductivity of 10^{-9} m/s).
	Tunnel Boring Machine	~ 1 m	Excavation-induced	0.03 m	Hydraulic conductivity in the range of $2\cdot10^{-9}$ to $5\cdot10^{-12}$ m/s from the tunnel wall to 4.5 mm depth. $5\cdot10^{-12}$ to $5\cdot10^{-13}$ m/s from 4.5 to 10 mm depth. The undisturbed rock matrix was estimated to be about $5\cdot10^{-13}$ m/s. (Data from the Äspö HRL TBM tunnel).
SKB – APSE	Drill and Blast	~ 0–5 m	–	–	–
SKB – Prototype repository	Tunnel Boring Machine	~ 1 m	Excavation-induced	10 mm	Hydraulic conductivity of 1–$2.5\cdot10^{-10}$ m/s.
Posiva – Olkiluoto	Mechanical excavation of deposition holes	<< 1 m	Excavation-induced	< 21 mm	Hydraulic conductivity of $7\cdot10^{-13}$ m/s.
Nagra – Near-Field Tunnel Programme	Drill and blast and mechanical excavation using Tunnel Boring Machine	0–3 m	Excavation-induced and possibly stress-induced	0–2 m	Matrix permeability of $3\cdot10^{-18}$ m^2 and effective permeability of $2.2\cdot10^{-15}$ m^2 (corresponding to a hydraulic conductivity of $2.2\cdot10^{-8}$ m/s.
ENRESA – FEBEX	Mechanical excavation using Tunnel Boring Machine	<< 0.01 m	Excavation-induced	< 3 mm	–
JAEA; EDZ II	Drill and Blast	~ 0–5 m	Excavation-induced and stress-induced	1 m. Damage possibly due to spalling and blasting damage	Hydraulic conductivity in the order of 10^{-5} m/s.

4 Potential evolution of processes close to the underground openings

The major experiments on excavation damage have mostly been executed in connection with the excavation work. The results of such experiments are necessary but not sufficient as the evaluation of excavation response should not only cover the present but also the properties for a long time after closure of the repository when the canister and other engineered barriers have been potentially impaired. The intention of the description in this chapter is to outline processes of importance or potential importance to the excavation response over time from start of construction to the post-closure phase. The main rationale to describe the evolution of physical, chemical and biological processes over time is to provide the background for the later discussions on feasible and relevant methods to measure the excavation response at different time periods.

The repository developed is executed in steps (see for example Section 2.1) and each step may have some impact on the excavation response. The time-evolution of the repository, with changes or potential changes in the ambient conditions, is therefore slotted into thermal, mechanical, hydrogeological and chemical/biological processes. A hypothetical time-evolution is shown in the following Table 4-1–Table 4-2 and the potential effect on the zone around the underground openings is outlined in Table 4-3 and Table 4-4. In the future work it will then be essential to decide on what processes to capture and the most appropriate time for testing. The tables contain generic information on measurements, and it should be noted that monitoring may occur but never in such a way that the barriers could be harmed. The processes are discussed in some detail in the following sections after relations between the excavation process and damage have been presented.

4.1 Relation between charging density, vibration and damage

The civil engineering and construction companies in Sweden have agreed to approach the subject of blasting damage in the following way: Commonly used explosives are listed in order of their equivalent linear charge concentration in terms of kg Dynamex per metre and a damage zone for each explosive is established /Olsson and Ouchterlony 2003/. The authors believe the relations have many shortcomings and that they have only been verified for very few explosives and under specific circumstances. Furthermore, no consideration is given to the influence of blast hole pattern, scatter in initiation and coupling ratio, etc.

In general, much of the R&D carried out has been focussed on the relation between damage and Peak Particle Velocity (PPV), and it was established that damage (creation of new cracks) was created at PPV of around 700 mm/s. The relation between PPV (mm/s) and charge W (kg) and distance R(m) is:

$PPV = K \cdot W^{\alpha}/R^{\beta}$

where α and β are parameters to be decided in the field. Typical relations for density of charge and depth of damage are shown in Figure 4-1.

Furthermore, several field tests were conducted during open pit blasting, and these results were used to correlate fracture length with density of charge /Ouchterlony 1997/, Figure 4-2.

The authors /Olsson and Ouchterlony 2003/ argue that depth of damage from blasting should be defined as the "maximum crack length of the radial fractures originating from the half-barrels" and they proposed a formula and its application as:

$R_c = R_{co} \cdot F_h \cdot F_t \cdot F_w \cdot F_r$

where R_{co} is crack length and $F_{h,t,w,r}$ are correction factors for hole distance, initiation delay, water and rock respectively.

Table 4-1. Time evolution from ambient conditions to before deposition. The text in red is a potential means of analysing/measuring the evolution.

Processes	Site investigations	Repository development	Excavation of deposition tunnel	Excavation of deposition hole	Lead time to deposition
External					
Thermal	-	Small seasonal variations of temperature underground Temp measurements of air	Small seasonal variations of temperature underground Temp measurements of air	No thermal load during excavation of the hole (vertical push-reaming using water flushing)	Small seasonal variations of temperature underground Calculations, temp measurements of air
Mechanical	-	Local EDZ around the openings due to stress change and excavation Survey batch	Local EDZ around the openings due to stress change and excavation Survey batch	Local EDZ around the openings due to stress change and excavation Survey batch	Time-dependent loosening of rock evident by scaling Monitoring (hydraulic, Acoustic Emission)
Hydrogeological	-	General lowering of the groundwater table Monitoring	Lowering of the groundwater table. Zero pressure at the opening. Two-phase flow phenomena Survey batch	Lowering of the groundwater table. Zero pressure at the opening. Two-phase flow phenomena Survey batch	Drying out of fractures etc. close to the openings. Two-phase flow phenomena Monitoring (hydraulic, Acoustic Emission)
Chemical/Biological	-	Precipitation, dissolution of minerals (close to openings) due to changes in pressure, presence of oxygen Monitoring	Precipitation, dissolution of minerals (close to openings) due to changes in pressure, presence of oxygen Survey batch	Precipitation, dissolution of minerals (close to openings) due to changes in pressure, presence of oxygen Survey batch	Precipitation, dissolution of minerals (close to openings) due to changes in pressure, presence of oxygen Monitoring

Table 4-2. Time evolution from deposition to post-closure. The text in red is a potential means of analysing/measuring the evolution.

Time evolution of the ESDDZ - KBS-3V						
Processes	**Deposition**	**Backfilling**	**Drift end plug**	**Sealing of deposition area**	**Sealing of repository**	**Post-closure**
External						
Thermal	Slow heating around the deposition hole and in the deposition tunnel Calculations, temp measurements of air	Slow heating around the deposition hole and in the deposition tunnel Calculations, temp measurements of air	Slow heating around the deposition hole and in the deposition tunnel Calculations, temp measurements of air	Slow heating around the deposition area Calculations, temp measurements of air	Slow heating around the repository Calculations, temp measurements of air	Heating and cooling of the repository Calculations, monitoring
Mechanical	Cyclic point loads in deposition tunnel due to heavy vechicles (deposition machine),Static load due to canister. Monitoring (hydraulic,Acoustic Emission)	Static load due to backfill, Slow build-up of pressures due to swelling of the buffer/backfill Monitoring (hydraulic,Acoustic Emission)	Slow stress build up around the openings due to swelling Monitoring (hydraulic,Acoustic Emission)	Slow stress build-up around the openings due to heating and swelling Monitoring (hydraulic,Acoustic Emission)	Slow stress build-up around the openings due to heating and swelling Monitoring (hydraulic,Acoustic Emission)	Slow stress build-up around the openings due to heating and swelling, de-stressing due to cooling Monitoring (hydraulic,Acoustic Emission)
Hydrogeological	The deposition holes are drained by pumping during until backfilling is approaching the hole. Thereafter slow increase of local water pressure and fully saturated rock in the deposition hole Monitoring (hydraulic,Acoustic Emission)	Slow increase of local water pressure and fully saturated rock in the deposition tunnel Monitoring (hydraulic,Acoustic Emission)	Slow increase of local water pressure and fully saturated rock in the deposition tunnel Monitoring (hydraulic,Acoustic Emission)	Slow increase of local water pressure and fully saturated rock in the deposition area Monitoring (hydraulic,Acoustic Emission)	Slow increase of ground water table and fully saturated rock in the repository Monitoring (hydraulic,Acoustic Emission)	Fully saturated conditions and flow controlled by gradients (incl. density, thermal) Monitoring (hydraulic,Acoustic Emission)
Chemical/Biological	Precipitation, dissolution of minerals (close to openings) due to changes in pressure, presence of oxygen, contact with buffer Monitoring	Precipitation, dissolution of minerals (close to openings) due to changes in pressure, temp, contact with buffer Monitoring	Precipitation, dissolution of minerals (close to openings) due to changes in pressure, temp, contact with buffer/backfill Monitoring	Precipitation, dissolution of minerals (close to openings) due to changes in pressure, temp, contact with buffer/backfill Monitoring	Precipitation, dissolution of minerals (close to openings) due to changes in pressure, temp, contact with buffer/backfill Monitoring	Precipitation, dissolution of minerals due to changes in pressure, temp, salinity etc. Monitoring

Table 4-3. Possible effects on the underground openings from ambient conditions to before deposition. The text in red is a potential means of analysing/measuring the evolution.

Time evolution of the ESDDZ - KBS-3V					
Processes	**Site investigations**	**Repository development**	**Excavation of deposition tunnel**	**Excavation of deposition hole**	**Lead time to deposition**
Potential effects on the EDZ					
Thermal		Ventilation air will be heated during winter and no freezing is expected in the shafts Monitoring	-	-	-
Mechanical		Reversible closure and opening of fractures close to opening, irreversible micro-cracking and creation of new fractures in the vicinity of the opening Survey batch	Reversible closure and opening of fractures close to opening, irreversible micro-cracking and creation of new fractures in the vicinity of the opening. Potentially local spalling due to high stresses Survey batch	Reversible closure and opening of fractures close to opening, irreversible micro-cracking and creation of new fractures in the vicinity of the opening. Potentially local spalling due to high stresses Survey batch	Opening of fractures and a potential "loose zone" around the opening Survey batch
Hydrogeological		Increased local transmissivity around the openings. Potentially creation of a new connected hydraulic network. Erosion Survey batch	Increased (due to opening of fractures)/decreased(due to clogging and 2-phase flow conditions) local transmissivity around the openings due to Potentially creation of a new connected hydraulic network Survey batch	Increased (due to opening of fractures)/decreased(due to clogging and 2-phase flow conditions) local transmissivity around the openings due to Potentially creation of a new connected hydraulic network Survey batch	Increased (due to opening of fractures)/decreased(due to clogging and 2-phase flow conditions) local transmissivity around the openings due to Potentially creation of a new connected hydraulic network Survey batch
Chemical/Biological		Clogging of existing, opened fractures or new fractures. Channelling Survey batch	Clogging of existing, opened fractures or new fractures Survey batch	Clogging of existing, opened fractures or new fractures Survey batch	Clogging of existing, opened fractures or new fractures Survey batch

Table 4-4. Possible effects on the underground openings from deposition to post-closure. The text in red is a potential means of analysing/measuring the evolution.

Time evolution of the ESDDZ - KBS-3V						
Processes	**Deposition**	**Backfilling**	**Drift end plug**	**Sealing of deposition area**	**Sealing of repository**	**Post-closure**
Potential effects on the EDZ						
Thermal	Rock slabs from spalling might develop and fall down in the slot between buffer blocks and the wall of the deposition hole Monitoring	Possible closure/opening of fractures due to heat load Monitoring	Possible closure/opening of fractures due to heat load Monitoring	Possible closure/opening of fractures due to heat load Monitoring	Possible closure/opening of fractures due to heat load Monitoring	Possible closure/opening of fractures due to heat and cooling Monitoring
Mechanical	Slow closure/opening of fractures due to stress changes, Potential forming of slabs that drops to the bottom Monitoring	Slow closure/opening of fractures due to stress changes. Spalling before swelling pressure is high enough Monitoring	Slow closure/opening of fractures due to stress changes Monitoring	Slow closure/opening of fractures due to stress changes. Possible spalling due to time-effects and heating Monitoring	Slow closure/opening of fractures due to stress changes. Possible spalling due to time-effects and heating Monitoring	Closure/opening of fractures due to stress changes. Possible spalling due to time-effects and heating Monitoring
Hydrogeological	Change of flow pattern due to change of gradients, fracture aperture changes, no 2-phase flow, and due to chemical effects Monitoring	Change of flow pattern due to fracture aperture changes, no 2-phase flow, and due to chemical effects Monitoring	Change of flow pattern due to change of gradients, fracture aperture changes, no 2-phase flow, and due to chemical effects. Possible changes of transmissivity Monitoring	Change of flow pattern due to change of gradients, fracture aperture changes, no 2-phase flow, and due to chemical effects. Possible changes of transmissivity Monitoring	Change of flow pattern due to change of gradients, fracture aperture changes, no 2-phase flow, and due to chemical effects. Possible changes of transmissivity Monitoring	Change of flow pattern due to fracture aperture changes, no 2-phase flow, and due to chemical effects. Possible changes of transmissivity Monitoring
Chemical/Biological	Clogging of existing, opened fractures or new fractures Monitoring	Clogging of existing, opened fractures or new fractures Monitoring	Clogging or dissolutions of minerals in existing, opened fractures or new fractures Monitoring	Clogging or dissolutions of minerals in existing, opened fractures or new fractures Monitoring	Clogging or dissolutions of minerals in existing, opened fractures or new fractures Monitoring	Clogging or dissolutions of minerals in existing, opened fractures or new fractures Monitoring

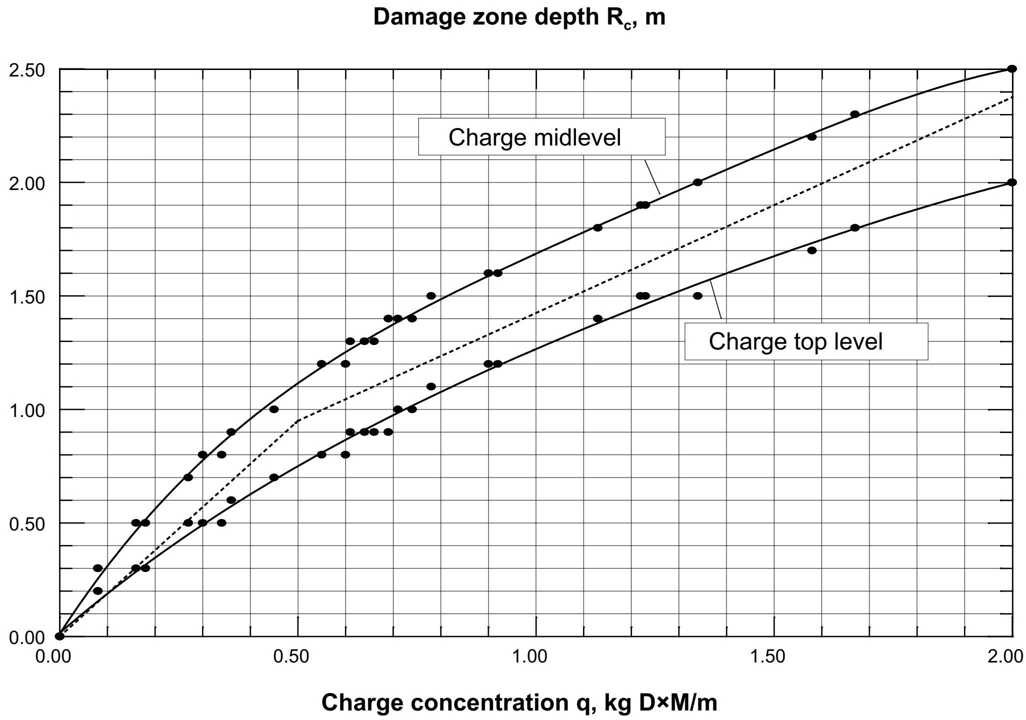

Figure 4-1. *Depth of damage (R_c) in [m] as a function of density of charge [equivalent kg DxM/m]. Lower curve: At the top of the charge. Upper curve: At the centre of the charge. Curve in the middle: Empirical relation damage depth and density of charge. After /Ouchterlony et al. 2002/.*

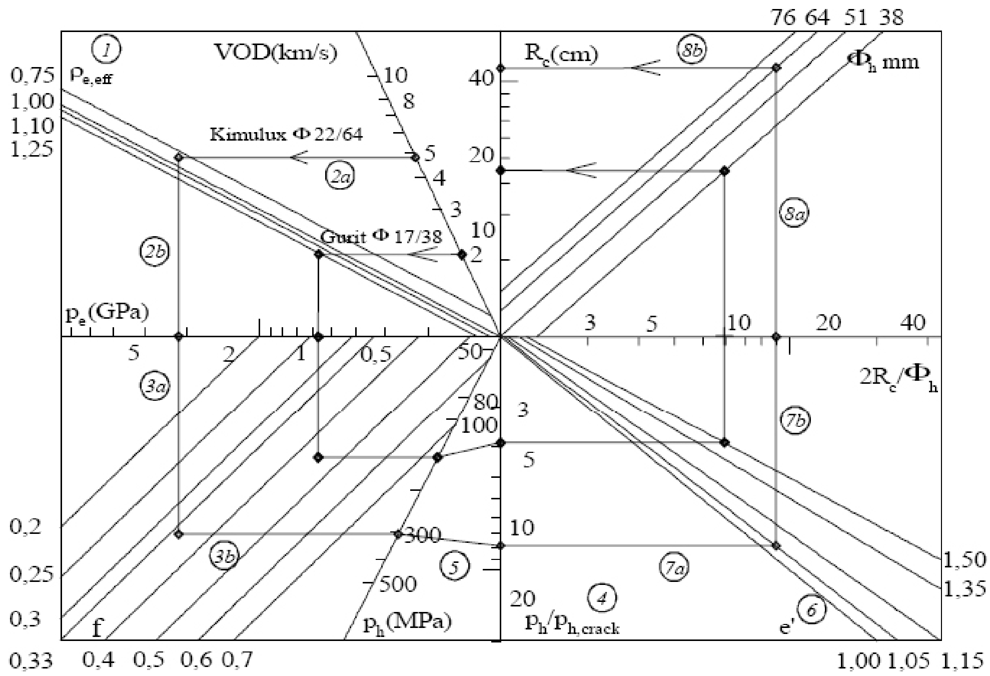

Figure 4-2. *Nomogram for determination of crack-length behind half-barrels /After Olsson and Ouchterlony 2003/ ρ is charge density [kg/m³], VOD velocity of detonation [m/s], Q, explosion energy of the explosive [J/kg], Ø diameter, p borehole pressure.*

The general approach described above is based on decades of R&D in blasting damage and is, despite many shortcomings, attractive as there is a clear coupling between controllable factors, such as density of charge, peak particle velocity with distance, etc and the approach would be useful for a "control programme" to verify properties during repository excavation, see Section 5.3.

4.2 Thermal evolution

Many of the descriptions related to the thermal evolution of the deep geological disposal facility are related to the source term due to the radioactive decay of the spent nuclear fuel and the heat dissipation throughout the disposal system after deposition of the canister. The heat will, of course, have an impact on the mechanical, hydrogeological and chemical/bacteriological processes. The reader is referred to the SR-Can report /SKB 2006a/ for detailed descriptions of the thermal evolution. The repository is designed so that the maximum temperature in the bentonite will never exceed 100°C. A typical evolution of repository temperature is shown in Figure 4-3 indicating a maximum rock temperature of around 60°C at the wall of the deposition hole.

In the SR-Can reference scenario, a future plausible glacial cycle was used to indicate the unlikelihood of permafrost (0°C isotherm) developing to a depth greater than 250 m at Forsmark and 160 m at Laxemar. Therefore, the evolution of spalling and damage due to permafrost conditions is not discussed in this report.

As excavation commences, the temperature variations underground at repository level are insignificant. However, as the repository access routes and shafts are opened, the rock closest to the opening will experience small annual variations in temperature and humidity (summer and winter), daily variations (day and night) and also shorter impacts due to traffic, etc. The variations are to some extent controlled by the arrangement of the underground ventilation system, and it needs to be determined whether these variations produce a measureable impact on the evolution of spalling, damage and disturbance.

After excavation of the deposition tunnels by drill and blast, minimal heat will be generated when the deposition holes are excavated by downward push-reaming as the intention is to use water-flushing for removal of the debris. The high level of heat generated by excavation is not likely to be transferred to the walls of the deposition holes; the mechanical excavation technique is similar to a TBM. In the ZEDEX experiment, the TBM used 1,680 MW in order to break the rock, but when measuring the temperature effect in the walls of the tunnels during excavation, the change was found to be insignificant and to have a radiated seismic energy of around 400 W /Emsley et al. 1997/.

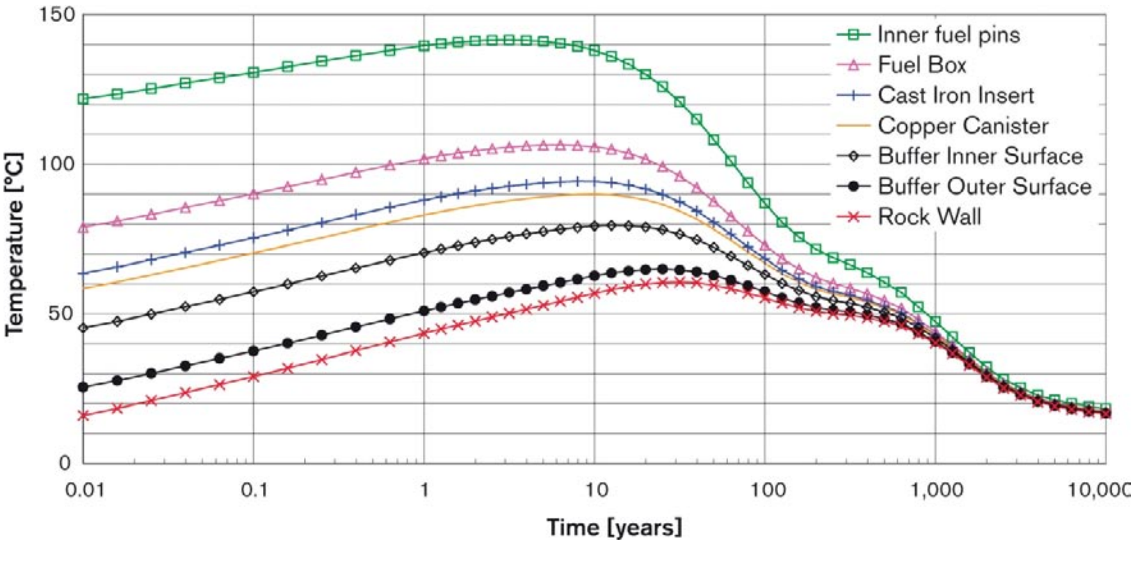

Figure 4-3. *The thermal evolution for a number of points at canister mid-height. /Figure 9-17 in SKB 2006a/.*

The repository will heat up to a certain extent over the period of time prior to installation of the spent fuel due to the operation of equipment, presence of personnel and circulation of air in the tunnels. This temperature change will be relatively small.

4.3 Mechanical evolution

The creation of an underground opening will redistribute stresses and strains around the opening. Evaluation of the stability of underground openings is very much based on experience in combination with modelling. Over the course of time, a design practice has been developed and its application to a Swedish final repository for spent nuclear fuel was outlined in the report by /Martin et al. 2001/. Detailed assessment findings for stability are reported in e.g. /Martin 1995/. The results of model testing for the excavation response were published in /Shin et al. 2007/. The models for thermally-induced spalling were tested in detail at Äspö as part of a PhD dissertation work /Andersson 2007/ and applied for the SR-Can /Hökmark et al. 2006/. The SR-Can report also discusses many types of natural phenomena that could have an impact on the mechanical evolution, such as glaciation and earthquakes. The potential effects of these phenomena are not discussed in this report.

The issues of interest for the mechanical evolution of the underground openings are the overall design of the openings to avoid spalling, the selection of excavation method to minimize or optimise the damages and the evolution due to the thermal sources and saturation of the buffer.

One aspect that has not been discussed to any great extent in the experiments and tests already conducted is, for example, the loosening of rock that occurs over time and prompts the need to scale the rock. Scaling is the activity employed to identify and break down "loose rock" and is a standard maintenance operation in all unlined underground openings. It is evident that the unlined openings will need scaling maintenance to scale down the loose rock. Scaling should be minimized and done for safety reasons only (operational and personnel) and should be performed with care. The need for scaling is dependent on rock and stress conditions, but also on excavation method. For example, at Äspö HRL records are kept of this type of maintenance /Andersson and Söderhäll 2001/ and there is a substantial need for maintenance scaling. However, in comparison there was no need for scaling in the Äspö TBM-tunnel.

One important issue is whether the deposition tunnels that are unlined will be subject to rock loosening, and whether after backfilling and swell pressure this phenomenon will be prevented. Data at AECL URL from the Heated Failure Test /Read et al. 1997/ and the Tunnel Sealing Experiment /Martino et al. 2008/ show that confining pressure does restrict damage development even at a low pressures of 100 kPa. Another issue to be settled is the maximum time in which deposition tunnels are to be kept open after excavation to avoid loosening rock.

Data from the AECL Tunnel Sealing Experiment showed some remarkable results. A situation was created in which the temperature and humidity variations were kept to a minimum at the test site, and during this period all changes in rock parameters (acoustic emissions, deformations) etc were non-existent (Jason Martino, AECL, pers.comm).

4.4 Hydrogeological evolution

The hydrogeological evolution has some distinct phases:

1. the ambient situation before construction of the underground parts of the repository;
2. the draw-down of the groundwater table and up-coning of lower groundwater during the construction and operation of the repository;
3. re-saturation of the repository once pumping of the open tunnels has stopped;
4. the evolution of the saturated repository up to the start of the next glacial period.

During the draw-down, parts of the rock, especially near the openings, will be partially or completely unsaturated, which will have a potential effect on inflow to the underground facility, see Figure 4-4. The reasons for the lowering of inflow over time are not fully understood, and several ideas have been forwarded, for example de-gassing. However, /Järsjö et al. 2001/ concluded that groundwater degassing will not give rise to considerable inflow reductions in fractures that intersect open boreholes under conditions which are normal for Swedish granitic bedrock. However, a situation with fully or partially unsaturated conditions complicates any hydrogeological testing programme; the situation being much simpler in the case of saturated conditions. Subsequently, desaturation could dry out fracture gouge material close to the openings, which may give rise to rock loosening, see also the discussion in Section 3.2 and the following Section 4.5 The repository conditions for different hydrogeological phases are dealt with in detail in the SR-Can report /SKB 2006a/

In the case of a resaturated repository, groundwater flow directions and magnitudes are of interest for studying the transport of solutes potentially harmful to the repository, and for studying potential releases of radionuclides from defective canisters. Groundwater flow paths provide information on where in the bedrock transport would take place and where exfiltration to the biosphere occurs. Properties along flow paths provide information on transport and retention characteristics of potentially migrating radionuclides. The overall objective is to assess the implications of site hydrogeological and hydrogeochemical conditions on repository performance. Generally, this is assessed by simulating the groundwater flow, identifying migration paths and calculating travel time and transport resistance for these paths, /SKB 2006a/.

4.5 Chemical and biological evolution

Previous experiments at e.g. Stripa, AECL URL and Äspö HRL concluded that chemical and biological effects could influence the inflow to an underground opening due to fracture clogging by mineral precipitation. If such clogging proved to be temporary and dissolved in the post-closure phase, there is a risk that the data collected underground during repository construction would be biased, i.e. that the measured hydraulic transmissivities are not relevant.

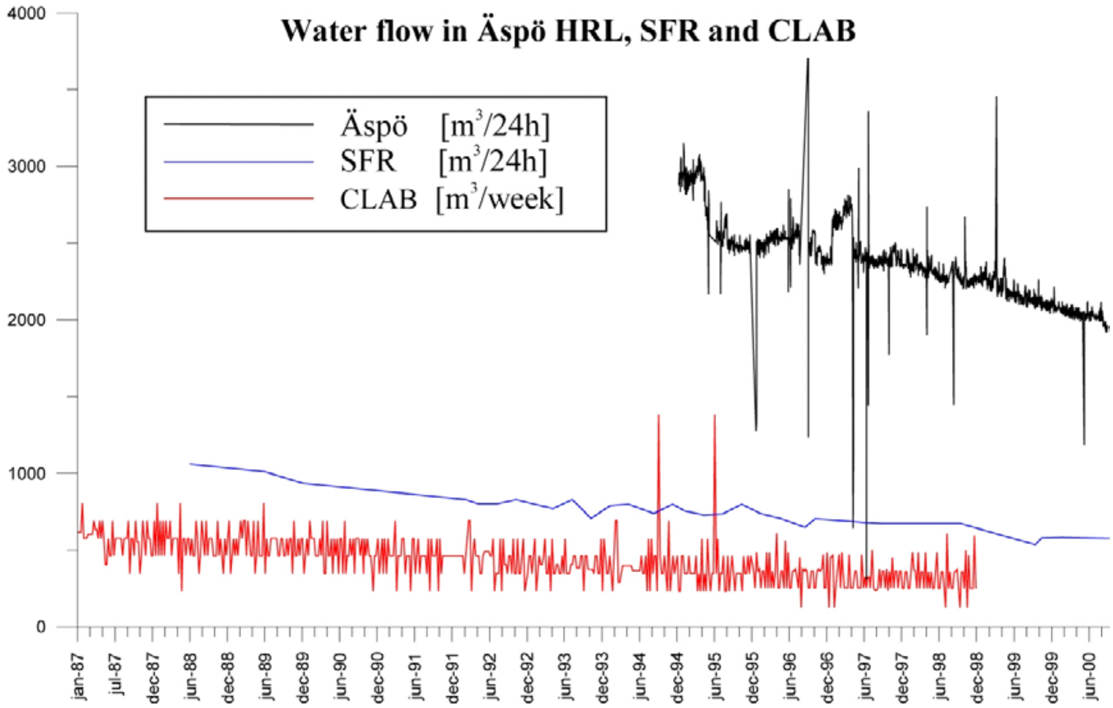

Figure 4-4. Water inflow rates to Äspö HRL, SFR and CLAB over time /Figure 3.5 in Andersson and Söderhäll 2001/.

As already noted, most previous EDZ studies have discussed the possible chemical and biological influences on the short- and long-term evolution of the EDZ, but no direct studies were quoted that could be used for the purpose of this Zuse-study. SKB therefore contracted a preliminary study, the results of which, /Laaksoharju et al. 2008/, are extracted here. The idea of the study was to describe the chemical and biological evolution during repository implementation and to determine whether the reactions could possibly affect the set-up of an EDZ-experiment.

During the excavation/construction stage, the rock and groundwaters at repository depth will be exposed to open atmospheric conditions, and this exposure will shift their redox character towards more oxidising conditions and will change their carbon content due to re-equilibration with atmospheric partial pressures of CO_2. Moreover, the excavation of underground galleries and the pumping of ingressing groundwater will facilitate the access of diluted infiltration waters downwards and deep saline groundwater upwards to the repository system. Furthermore, the dissolution of the cementitious material used for grouting and ground support will generate high-pH waters which can mix with the repository groundwater. All these processes, which can also take place after the repository closure, will lead to geochemical disequilibria with respect to the original conditions. Besides these processes, other geochemical changes are expected to occur when closing the repository. Firstly, the repository closure will lead to isolation with respect to atmospheric gases and to the recovery of reducing conditions. Secondly, the temperature will increase due to the thermal impact of spent nuclear fuel on the surrounding rock, and this may also influence the hydrochemistry.

The compilation of information and simplified modelling using the PHREEQC code were conducted according to the following steps:

1. Identification of representative groundwater samples from repository depths at Laxemar and Forsmark.
2. Modelling and discussion of the potential for calcite and iron hydroxide, and microbial sealing during prevailing conditions.
3. Modelling and discussion of the potential for calcite and iron hydroxide, and microbial sealing and dissolution during open repository conditions (atmospheric pressure and oxidising conditions) by assuming: a) the groundwater composition is unchanged (except for redox potential E_h and pH), b) a deep water is 50% mixed, c) a shallow water is 50% mixed, e) the pH is affected by a low-pH cement according to the discussion in SR-Can, and d) the groundwater is contaminated by nitrogen compounds (300 g/m^3 rock) from explosives.
4. Modelling of the temperature effects during emplacement and after closure. This was carried out in three steps: at 100°C, 50°C and at ambient temperature (i.e. 15°C). The 100°C and 50°C cases are modelled by simulating closed repository conditions with full water pressure (500 m depth) and the 15°C situation by assuming open repository conditions (zero pressure).

Based on the site-specific conditions and the preliminary modelling, it was concluded that for most of the modelling cases, the estimated decrease in the hydraulic conductivity over the space of ten years is less than 2% due to mineral precipitation. The largest variations in the hydraulic conductivity are caused by calcite precipitation. The precipitated volumes for this mineral are generally between one and three orders of magnitude larger than for precipitated ferric phases. As a general trend, the largest hydraulic conductivity variations (largest mineral precipitation amounts) correspond to the mixtures of groundwater with the shallow infiltration waters. For those mixtures, the estimated decrease in hydraulic conductivity is always larger than 1%. It was further concluded that it is more likely for fractures to clog (self-healing) than for them to open as a result of fracture mineral dissolution.

With respect to the microbial evolution, it was concluded that the potential for microbial iron hydroxide production will be substantial in all groundwater containing ferrous iron. The potential for microbial calcite formation production will be significant in all groundwater with high concentrations of Dissolved Organic Carbon (DOC) and methane, irrespective of oxygen, E_h or pH.

- As deep-lying groundwater generally has less ferrous iron and DOC than intermediate and shallow groundwater, it is assumed that the rate of microbial iron oxide and calcite formation will decrease, relative to the ambient conditions before construction, except for cases in which deep groundwater contains a high concentration of methane that mixes with a sulphate- or oxygen-rich groundwater.
- Shallow groundwater generally has more ferrous iron and DOC (and less methane) than intermediate (unchanged) groundwater. It can, therefore, be assumed that the rate of microbial iron oxide and calcite formation will increase in relation to the ambient conditions before construction.
- The contamination explosives with nitrogen compounds will not influence the concentrations of ferrous iron, DOC or methane, and the situation will therefore not change in relation to ambient conditions before construction.
- The increase in pH from cement grouting will not have a large effect on the microorganisms. However, observations made in tunnels elsewhere, in which low-pH cement has been injected, reveal vivid microbial growth and the formation of up to 5 cm thick layers of slime on the tunnel walls, which was found to be clogging water conducting fractures.
- There is no information available about the potential for microbial growth and activity at 50°C and 100°C. As a general rule of thumb, biological processes double with every 10°C increase in temperature. Increasing temperature will then most probably speed up the bio-precipitating processes.

To summarise, it is unlikely that mineral precipitation will affect the test programme for an EDZ experiment. On the other hand, microbes may play a role in fracture clogging, but the kinetics of the microbe evolution is not presently known.

5 Analysis and evaluations of methods and instruments

5.1 General discussion

The excavation response as related to final disposal has now been studied with a multitude of methods[14] and instruments over some decades. This section contains a general discussion and overview of methods and instruments used in previous R&D projects as well as a first proposal for a "control programme" to be used during repository construction.

The main methods used to study the **mechanical damage** are:

- visual inspection of the tunnel surfaces by mapping and fractures close to the periphery, either in boreholes or by cutting slots in the rock;
- using volume methods such as micro-seismic networks or acoustic emissions to pinpoint the locations of slip, extended or new fracturing;
- using geophysical methods at the tunnel surfaces (such as refraction surveys) or in boreholes as indicators of increased fracturing;
- standard tools to measure the elastic response by convergence or by displacement measurements in boreholes.

The main methods used to study the **hydrogeological response** to the excavation are:

- packer tests in short boreholes after the excavation has been completed;
- construction of dams and filling parts of an excavated study tunnel with water to study the connectivity of the damaged zone over some distance.
- The Stripa Buffer MassTest method, comprising measurements of the axial hydraulic conductivity by recording the water flux from a gallery of closely spaced, radially bored holes to an adjacent gallery.

Chemistry and microbiology were not considered in previous excavation damage and disturbance experiments, and adequate methods have therefore not been specifically tested.

A simplistic view of potential general methods for exploring the excavation response has previously been outlined in Table 4-1 to Table 4-4. It is noted that in general, remote monitoring methods will become more and more important, or rather the only viable method, after the deposition of canisters.

A large number of methods and instruments have been employed during the experiments discussed in Chapter 3 and in many instances experience has been drawn. However, this experience has often been related to the individual instruments rather than being a critical review of the overall method employed.

This report is not intended to critically evaluate possible methods and instruments for describing changes in the bedrock, but rather to present some lessons that have been learnt and also to discuss the relevance of methods and – based on a preliminary evaluation outline – a possible "control programme". The main references for evaluations of instruments used here are the reports by /Martino 2000/, summarizing the AECL work and the report by /Emsley et al. 1997/, summarizing the ZEDEX experience and /Marschal et al. 1999/, where experience from the Grimsel Test Site was compiled. These results are discussed in detail in Section 5.2.

With respect to the connectivity of the flowing fracture network, previous tests have studied the connectivity over several metres. However the relevant scales may be much larger. At the Swedish potential repository site Laxemar, the typical highly conductive fractures are separated by a distance

[14] "Systematic procedure, technique, or mode of inquiry employed by or proper to a particular discipline or art"; Merriam-Webster.

of 0–10 m, and at Forsmark the typical separation distance is 20–100 m. Therefore, the relevant scale for connectivity is site-dependent, and this has to be accounted for when relevant test designs are prepared. Such tests are likely to be staged, the first stage being to understand flow on the local scale and the second stage on the larger scale so that the influence of the fracture network can be separated from local flow driven in the tunnel.

From a methodological point of view, the hydrogeological tests are key to the input to the safety case. It is important to note that the test results are somewhat method-dependent. For example, the AECL Room 209 connected permeability experiment (Section 3.3) showed hydraulic conductivity based on short borehole tests in the order of 10^{-12}–10^{-11} m/s, whereas the hydraulic conductivity based on flow measurements was estimated to be approximately 10^{-8} m/s. The conclusions with respect to connectivity based on the construction of dams are not conclusive as the connectivity measured was possibly dependent on the dam construction, cf Section 3.6.

The methods and equipments to be used should be targeted to the intended use of the tests. In this case, the results will be used to infer the excavation response for a closed repository possibly several thousand years after closure: What are the most relevant and practical methods for inferring the present and future conditions around the underground opening? After closure, the rock and repository will eventually be fully saturated and for this reason it is more relevant so make tests in which the rock is fully saturated than to perform tests in drained tunnels where two-phase conditions will probably prevail, and which will make interpretation of the tests more complex. For any tests underground, biases due to possible clogging as a result of precipitation/dissolution of fracture minerals or microbes should be understood.

As regards mechanical damage, a large number of methods can be used. These include direct methods such as cutting slots and dying fractures, which are useful for phenomenological studies of damage. However, they cannot easily be transferred into interpretations of changes in physical parameters such as porosity and or transmissivity.

Direct fracture mapping can be made from wall mapping, cutting slots and dying, by inspecting core samples and by in-the-hole investigations. A common problem for all these methods is that, due to the geometrical situation, they can seldom be used to compare fracturing before and after excavation. When the methods are used after excavation it is difficult to clearly separate “old” natural fractures from “new” and induced fractures if the old fractures do not show some type of marker such as alterations. The most practical method used for fracture mapping is to map fracture intensity from the periphery and outwards from the periphery and assume that the damage extends until the background level is reached. One factor open to discussion is the cut-off length for fracture mapping as well as the possibility to connect fractures over distances.

AECL used, for instance, core logging and borehole camera surveys for direct fracture mapping whereas SKB in the ZEDEX experiment used core logging, borehole TV, and the dying of fractures.

On a much smaller scale, the Scanning Electron Microscope (SEM) – a type of electron microscope capable of producing high-resolution images of a sample surfaces – has been used, for example, at the FEBEX to deduce where the damage zone terminates.

Indirect fracture mapping

Several standard geophysical methods are useful to deduce fracturing by geophysical methods. SKB makes frequent use of in-the hole radar, which also provides a reasonable 2.5D picture close to the borehole, and Posiva has started to test a ground penetrating radar along the periphery of the ONKALO tunnel. Testing of the deposition holes at Olkiluoto and at FEBEX was performed with porosity measurements to infer the increase of micro-cracking using the ^{14}C-polymethylmethacrylate (^{14}C-PCCMA) method where impregnated rock samples a few cm^3 in volume are investigated by autoradiography.

Seismic and acoustic emission methods to pinpoint locations of crack initiation and measurement of seismic velocity have been used extensively. These methods are efficient for describing the mechanical evolution in a volume and also useful, with sensible locations of sensors, for tracking the evolution during and after the excavation. Acoustic emission (AE) is the study of elastic waves. Based on studies in good quality crystalline rock, transducers are in the frequency range 50–5,000 kHz and typically involve volumes from cm^3 up to maybe 1,000 m^3. Micro-seismic systems use geophones for measuring velocities or accelerometers for measuring acceleration. The typical range is 0.1–40 kHz and the typical volume up to the scale of the underground facility, whereas the seismic systems for the seismic monitoring of earthquakes operate in the low frequency range, typically below 50 Hz.

AECL and SKB (ZEDEX) used micro-seismic systems (typically used for rock volumes < 100,000 m^3) and acoustic emission systems (typically used for rock volumes 1,000 m^3): Both systems have been very useful in measuring crack events and fracture displacements during the excavation. SKB used the micro-seismic system to develop high-resolution seismic tomograms before and after excavation. AECL and SKB (ZEDEX) also successfully used seismic refraction systems to measure the primary seismic wave (P-wave velocity) and the depth of the zone with lower velocity along the tunnels. Several cross-hole measurement tools are also available. AECL for instance, used a system named CHARTS (Cross-Hole Audio Frequency Rock Testing System) in the frequency range 3–40 kHz to look for small-scale structures in boreholes over distances of up to a couple of metres. Also, single hole measurements are available and AECL used a micro-velocity probe (MVP) to measure the ultrasonic interval velocity between transducers of a fixed spacing.

Several methods are also applicable for analysing rock core samples in the laboratory, such as the propagation of ultrasonic waves to estimate velocity as an indicator of extent of fracturing.

Hydrogeological measurements

With respect to hydrogeological readings, measurements can be performed, for example, on rock samples, in boreholes and by seepage tests.

Gas permeability tests were used, for example, on rock samples from the FEBEX site. The test section length was typically 1 cm along a hollow rock cylinder with an internal diameter of 24 mm. The test is apparently aimed at studying the rock matrix rather than the natural rock fractures on a larger scale. Gas permeability tests were also conducted by using a surface packer developed by Bundesanstalt für Geowissenschaften und Rohstoffe (BGR) in Germany. This type of equipment can measure permeabilities down to 10^{-21} m^2. Such measurements were also conducted at Äspö HRL.

To measure the hydraulic conductivity of very tight rock, different borehole probes have been used. One example is the SEPPI probe (Système Expérimental de mesure de Perméabilité par Pulse *In Situ)* which were developed by the universities in Nancy and Lille in France and used by AECL. A system of hydraulic and mechanical packers is used to saturate the rock and also achieve a stiff system as the injected volume is measured with a precision of 0.1 mm^3 over a 50 mm packed-off test interval. Also "standard equipment" with single-packer and double-packers has been used extensively at Stripa, AECL and at SKB. However, it is important for the systems to have measurement limits that are adequate. Nagra has also developed single-hole measurements for low-permeability rock whereby a surface packer is combined with a Modular Mini-Packer System (MMPS). Nagra has also developed and used a Time-Domain Reflectometry (TDR) unit used for measuring the water content in rock with porosities as low as 0.1% /Marschall et al. 1999/. AECL has also developed and used an excavation damage zone (EDA) packer designed with multiple narrow rubber glands to isolate short sections of the near field rock /Martino et al. 2008/.

Both AECL and JAEA have used seepage tests, in which a much larger area/volume is investigated than in the borehole tests. Indications show that there are magnitude differences between results in borehole tests and seepage tests, but it should be remembered that both methods are marred significant degrees of uncertainty.

Weaknesses of the hydrogeological methods are, for example:

- difficulty to measure and compare results before and after excavation;
- the rock on a smaller scale is very tight, and therefore strains the measuring limits of the equipment;
- the rock is heterogeneous and quite a few tests are needed to make conclusive statements;
- the tests are made on a scale that is not relevant for the problem at hand;
- the tests are conducted in partially saturated conditions instead of fully saturated conditions that are more relevant for geological disposal.

In the site investigation programme, SKB has used power law between structure size and transmissivity for natural fractures, see for example /Dershowitz et al. 2003, Follin et al. 2007/. As described previously, work is being conducted by SKB on adopting a similar approach but where the relation between excavation-induced fractures and transmissivity is studied. Such data is important for Discrete Fracture Network modelling of the EDZ.

Chemistry and microbiology

The standard site characterisation programme with groundwater sampling will provide data on the groundwater chemistry, including all its constituents. The evolution of the chemistry and microbiology is very dependent on the ambient and disturbed hydrogeological situation where different types of groundwater mix. Other factors need to be accounted for in the modelling, such as pressures and temperatures that may change over time. Construction and stray materials added during repository construction, for example cement due to grouting and ground support, and nitrogen compounds due to explosives, may also impact the reactions. The "methods" are not only the instruments used for investigations, but also laboratory studies, integrated modelling and good book-keeping of all site-specific data that may influence the chemistry and microbiology.

The construction process

In general, there is a need to strengthen documentation on the construction process for the purpose of EDZ evaluation. In many instances it is difficult or impossible to track how the EDZ test sites were actually excavated. At best, plans are provided in the reports, but not always. Records and documents of the final product "as-built", including misfires, confirmation of drilling pattern, borehole deviations, charging etc are often lacking. For some tests, such as the ZEDEX test, the TBM site was grouted before excavation as the main purpose was to study the mechanical response. If data was collected before construction without grouting, and it was later found necessary to grout, it would then be even more difficult to compare prior data with data after excavation. For proper evaluation of EDZ during repository construction, it is very important that the openings are constructed "as planned" as the "control programme" might be very different for a flawed construction process.

5.2 Basic experience from Grimsel Test Site, AECL URL and Äspö HRL

As mentioned previously, the compilation of experience has been more on the "instrument level" than the "method level". The Tunnel Near-Field Programme run by Nagra at the Grimsel Test Site recommended a suite of methods/instruments to characterise the near-field for different stages of the repository implementation phase (investigations during excavation, test phase and long-term monitoring phase), see Table 5-1. The recommendations are based on data requirements, and on technical, economic and time constraints.

AECL in the report /Martino 2000/ presented a practical "method assessment" of methods that were feasible and useful. These were seismic tomography, seismic refraction, the micro-velocity and SEPPI probe, core logging, borehole camera logging, micro-seismic systems, acoustic emission and connected permeability tests. A recent paper /Kuzyk and Martino 2008/ also shares experience from the construction process at the URL.

In the ZEDEX project /Emsley et al. 1997/, 17 different methods were used to study the damaged and disturbed zone, such as high resolution seismic tomography, permeability measurements (pressure build-up tests), difference flow measurements, stress measurements, acoustic emission monitoring, multi-point borehole extensometers, converge measurements (pins), high-resolution permeability measurements, laboratory testing of cores with respect to velocity, downhole P-wave velocity logging, acoustic resonance measurements, micro-velocity logging (high-frequency), mini-sonic velocity logging (low frequency), seismic refraction, detailed mapping of the half barrels from blast holes in the periphery, crack discrimination in rock samples using a penetrant and finally dye penetration tests of cuts in the tunnel walls.

Table 5-1. Proposed field methods for tunnel field characterisation. Field methods in *italics* show the method that also was applied at the Grimsel Test Site before the Near-Field Programme /After Marschall et al. 1999/ Notation: WCF stands for Water Carrying Fractures.

			Geology						Hydrogeology										Rock mechanics			
Exposure	Exploration Phase	Purpose / Field method	Classific. of struct. elements	Orientation WCF	Frequency WCF	Size WCF	Deformation state matrix	Microstructural properties	Interv. Transmissiv. profiling	Internal variability TWCF	Correlation length TWCF	Head profiling	Saturation profiling	Matrix potential profiling	TPF parameters of WCF	Inflow distribution	Water balance (total)	Longterm monitoring head	Stress distribution	RM parameters - matrix	RM parameters - fractures	Extension damaged zone
Tunnel wall	1,2	*Geological / structural mapping*	x	x	x	x	x															
	1,2	*Hydrogeological mapping*								x	x					x						
	1,2	*Seismic surveys*		x	x	x																
	1,2	*Thermography*		x		x				x						x						
	2	*Water sampling /plastic sheets*														x	x					
	2	*Evapometer*														x	x					
	2	*Surface packer*				x			x	x	x				x							
	1,2,3	Convergence measurements																	x	x	x	
Boreholes	2	Petrophysical logging	x	x	x																	
	2	*Core inspection*	x	x	x		x															
	2	*Comb. Short Interval Packer*							x	x	x	x			x	x						
	2,3	*MMPS*							x	x	x	x						x				
	3	*Minipiezometers*											x					x				
	1,2	*Fluid logging* / flow logging														x						
	2	*Ultrasonic borehole seismics*											x							x	x	x
	2,3	*Time Domain Reflectometry*											x		x	x						
	2,3	*Thermocouple psychrometers*												x								
	2	*In-situ stress / overcoring*																	x			x
	2	*Borehole slotter probe*																	x			x
	2	*Borehole camera (bh imager)*	x	x	x		x	x														
Core samples	2	*Thin sectioning*	x				x	x														
	2	*Porosimetry*						x														
	2	*Capillary pressure tests*													x							
	2	*Relative permeability tests*													x							
	2	*Rock mechanical lab tests*																		x	x	x
	2	NMR-pore space visualisation					x	x		x												

Exploration phase:
1 - investigations during excavation
2 - test phase
3 - longterm monitoring phase

All the methods used at ZEDEX, except the acoustic resonance measurements and high resolution seismic tomography, were in some respect useful. The most sensitive method for the damage zone was considered to be the monitoring of acoustic emission (AE) events. AE and seismic velocity measurement were judged to be the most robust methods.

5.3 Methods and measurements within the framework of a "control programme" during repository construction

The final repository for spent nuclear fuel is a nuclear facility and will have to fulfil the requirements for a nuclear facility in general as well as the specific requirements established for the geological disposal of spent fuel.

During repository construction, a "control programme" will be necessary in order to verify that the established requirements are being complied with. The control programme is a description of:

- what aspects are to be verified and/or reviewed;
- who should execute the verification/review;
- how, or in what order, the verification/review should be executed.

With respect to the excavation response, it is suggested that three factors be considered, namely the excavation process itself, the rock response to the excavation and finally the evolution of the rock response with respect to time where, for example, the heating and cooling are one factor to consider. SKB has decided that the particular control programme for EDSSZ should cover the period prior to start up to backfilling. It is also evident that the control programme for excavation is not part of the Zuse project. Considerations are nevertheless provided in this context which might be useful in the planning of the other control programmes, before the outline of the control programme is presented in Section 5.3.4.

The preliminary modelling (Section 2.3) indicates that the spalling and excavation-induced damage only has a very minor influence on the long-term safety, which means that the control programme should be somewhat limited, as the ambitions of the "control programme" should be in balance with the importance of a factor with respect to safety.

5.3.1 Aspects of excavation control

As described in Section 4.1, there is a complex interaction between density of charge, blast hole pattern, scatter in initiation, etc and damage. However, there is a high level of theoretical understanding and experience to couple these parameters with the extent of the damage. The excavation damage should be studied for the case in which excavation goes according to plan, but also for a number of cases in which possible deviations in charge density, borehole deviation, etc are studied. Site-specific relations can then be prepared for, for example, damage and charge density as the basic parameter for the estimation of damage extent and properties. Such relations need to be established for different rock types and fracturing, etc. It is here assumed that the design of the opening will minimize periphery stress concentrations, that a proper quality control is in place and that the results from such control are systematically presented to the construction management.

After excavation, the rock is to be scaled, i.e. loose rock is removed from roof and walls. Standard practice today in civil engineering and mining is to use a mechanical hammer, but high-pressure water-scaling is also used. The latter practice is recommended as the damage caused by this scaling method is very limited.

To establish the damage-density of charge relations, it is assumed that a standard batch of methods will be used, e.g.:

- extent to depth using ultrasonic measurements in boreholes combined with studies on thin sections;
- measurement of the transmissivity of the damaged zone by laboratory measurements on cores in combination with field tests in boreholes;
- connectivity of the EDZ by using, for example, ground penetrating radar, refraction seismics or hydraulic interference testing, or other methods under development.

It is to be further assumed that pre- or post-grouting (i.e. grouting before or after excavation) will be utilised locally to limit the ingress of groundwater flow. Such measures will hamper the possibility of measuring the "true" hydraulic transmissivity of the natural fractures and also the "true" hydraulic characteristic of the blasting-induced fractures in the event that post-grouting is used.

5.3.2 Aspects of rock response to excavation

The excavation of an underground opening creates mechanical, hydrogeological, chemical and biological reversible and irreversible changes. Here it is assumed that the control programme is limited to the irreversible changes in the damage zone. In case the relations between density of charge and damage are established and verified, the control programme on mechanical damage shall verify the density of charge, which is a matter for the excavation control programme.

The transmissivity of the EDZ can be tested at the positions of the deposition holes. The nominal distance between deposition holes (approximately 6 to 11 m) will not coincide with the length of each drill and blast round, nominally 4 m. Therefore, over the length of the deposition tunnel, some EDZ tests will be conducted at the start, in the middle of the round and also at the end of a round. As EDZ should be more prominent in the floor, such values will also provide conservative values of the EDZ in the walls and floor. It is also more likely that the saturated conditions will prevail in the floor rather than the roof and walls, which will simplify test evaluation.

Typical measurements at the locations of deposition hole and at the top m should cover, for example:

- ultrasonic measurements;
- testing of hydraulic transmissivity by multi-packers;
- occasional laboratory tests on rock cores.

The tests should be integrated with the control programme for the deposition hole that will be described in the Rock Line Report[15].

To test the connectivity of the EDZ, such tests can tentatively be conducted using ground penetrating radar. Fracture connectivity can also be tested by hydraulic interference testing within the area of the deposition hole (diameter 1.75 m). Some part of the deposition tunnel, close to the drift-end plug, could perhaps be tested for connectivity when the deposition tunnel has been backfilled and the drift-end plug emplaced.

As described in Section 4.5, chemical effects are not expected to bias the hydraulic programme. However concerns are raised with respect to the microbiological evolution as the kinetics is basically unknown. Clogging of fractures by microbes could make the measurements non-conservative.

5.3.3 Aspect of excavation response after the canister is emplaced

The properties of the damaged zone are mainly of interest for the distant post-closure phase when the Engineered Barrier System could be impaired. In SKB's work, the Evolution Reports are prepared to describe the evolution after deposition of the spent fuel and therefore these aspects are only commented here:

- Are the data compiled in connection with excavation stationary with time?
- What are the effects of water saturation of buffer and backfill and the heating and cooling due to radioactive decay?
- Chemical effects seem to clog the fractures rather than to dissolve fracture minerals. Can this self-healing effect be utilized as a factor that contributes to long-term safety as the near-field flow is reduced?

Concerning spalling, this phenomenon is treated in the ongoing (November 2008) SKB project CAPE, and it is here assumed that the result will be used to establish the control programme aspects. As shown in the work by /Andersson 2007/, standard uniaxial compressive tests on cores can be used to assess at what stress levels spalling will be initiated.

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[15] The "Line reports" were outlined in the SKB RD&D Programme 2007 /SKB 2007/ and the Rock Line Report will be entitled "The underground opening construction report" (in prep.)

Another method that would be useful is to use for instance micro-seismics from neighbouring tunnels that have not yet been backfilled to track the micro-seismic evolution after closure of the deposition tunnel. Micro-seismic monitoring is a technology that is undergoing strong development due to deep mining (> 2 km depth), see for example /Reyes-Montes et al. 2008/, and the theoretical understanding and coupling to the mechanical behaviour is rapidly expanding. The benefit of using a future deposition tunnel or investigation tunnel for the monitoring of a deposition area should be evaluated. Another important aspect could be to track that the backfill is behaving as expected, as the results of the SR-Site preliminary modelling exercise (see Section 2.3) show that the backfill behaviour could be more important than the stress-induced or excavation-induced damage behaviour with respect to the long-term safety function.

5.3.4 Outline of a control programme

This section outlines a tentative control programme concerning the excavation response, i.e. the methods to be used during repository construction, on condition that the methods have been verified.

What aspects are to be verified and/or reviewed;

For the **deposition tunnels** the following applies:

The main properties for which substitute parameters are to be established are the hydraulic transmissivities around the tunnel periphery (< 1 m distance) and the potential connectivity of hydraulic transmissivities along a deposition tunnel over a stretch of more than 10 metres. By measurement < 1 m, the extent of the EDZ, typically 200–300 mm, will also be captured.

It should be verified/reviewed that the excavation process is within its authorised event space. In case the excavation is within the established ranges, the extent of the EDZ and its properties are assumed based on the empirical relation between density of charge and damage, and accounting for a range of rock conditions. In the event that the excavation process is conducted beyond the established event space, a deviation order will be prepared and a special programme developed and authorised. Such a programme may aim to extend the previously authorised event space, i.e. to confirm that the relation between density of charge and damage is established for the broader range of the event space.

The potential for spalling due to heating/cooling or stress changes is to be evaluated.

It should be verified that the groundwater chemistry is as expected so that no rapid clogging due to precipitation of fracture minerals will occur. The potential for clogging due to microbial growth should be clarified to avoid biased data.

The evolution of potential micro-cracking with time should be monitored.

For the **deposition holes:**

The properties of the EDZ (hydraulic transmissivities) are to be defined within 0.1 m and along the periphery of the deposition hole. Excavation will typically be by hydraulic push-reaming. The damage zone is dependent on the design of the reamer and cutters. Relations between micro-fracturing and hydraulic transmissivities are established for the equipment design in use and in consideration of typical cutter wear and for a range of anticipated rock conditions.

It shall be verified that the equipment design and rock properties are within the authorised event space.

The potential for spalling due to heating/cooling or stress changes shall be evaluated.

It should be verified that the groundwater chemistry is as expected so that no rapid clogging due to precipitation of fracture minerals will occur that could interfere with the interpretation of hydraulic measurements. Also, the potential for clogging due to microbial growth should be evaluated.

For the **drift-end plug:**

The verification of the tightness of the drift-end plug will be part of the control programme for the Rock Line Report and will not be further discussed here.

Who should execute the verification/review;

There are two steps to consider. The first step is to verify that the methods using direct or substitute parameters are reasonably correct and robust. The second step is to put the methods into daily practice during repository construction

SKB has in the past conducted a long series of experiments that in general will be useful for verifying methods, but in this case it is proposed that site-specific relations be established for the repository site. The verification of these relations can most likely be through the traditional scientific/technical peer review.

The second step is covered by the quality system through method descriptions and work orders.

Potential work flow

It is here assumed that in the future, a Rock Line control programme will be available, in which the excavation and "detailed site characterisation" programme are integrated with the particular tasks necessary for studying the excavation response.

The first step is to verify that the methods proposed, what they are intended to measure and this verification should be established no later than during the ramp access excavation (see points 1. and 2. below).

A potential flow of work within the control programme for a deposition tunnel is[16]:

1. Establish how the deposition tunnel will be designed and constructed. Estimate the typical event space due to design flaws, construction flaws and range of rock conditions.
2. Establish damage in relation to density of charge and other factors that are to be accounted for in the authorised event space. Such relations can be established during ramp access excavation. Several tests methods are applied, and the most useful and feasible ones are used in the following control programme.
3. During excavation of the deposition tunnel, the excavation process is recorded and damage estimated.
4. The periphery of the deposition tunnel is mapped with respect to lithology, natural fractures, potential blasting-induced fractures, half-barrels and break-outs. The floor, roof and walls are investigated by ground penetrating radar and/or possibly refraction seismics.
5. It is assumed that the location of the deposition hole is verified by a cored borehole. The standard batch of investigations methods is used at the deposition hole. For the upper first m of the deposition hole, detailed investigations are performed on the core and in the borehole, to describe the natural and blasting-induced fractures. Methods foreseen to be used are ultrasonic measurements and detailed hydraulic testing. The data are also used to calibrate methods such as ground penetrating radar and reflection seismics that might be used to study the connectivity.
6. At every 8th deposition hole, a small-scale interference testing programme is executed within the periphery of the deposition hole. The central cored hole is pumped and 3–4 percussion holes < 2 m deep are instrumented to measure the pressure response.
7. When the deposition of the engineered barrier systems starts (canister, buffer, backfill), it is assumed that separate control programmes will commence for example to ensure that the expansion of the buffer will not create a damaging upheaval of the deposition tunnel floor.
8. It is assumed that the study of EDZ around the drift-end plug is part of the control programme to ensure the overall function of the plug.
9. During the development of the deposition tunnel and the deposition holes, deposition of spent fuel and backfilling, every 4th adjacent deposition tunnel is used for monitoring purposes before being utilized for the deposition of spent fuel. The positions of deposition holes are core-drilled and instrumented with multi-packers. A micro-seismic network is operated to capture the events during the work and also to monitor the initial development after backfilling and plugging the adjacent deposition tunnels. It is assumed that the "monitoring tunnels" are filled with water, but not to full water pressure. The detailed logistics between the excavation, deposition, backfilling and monitoring operations are to be detailed.

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[16] The procedure for e.g. the deposition holes is quite analogous and will not be described here.

6 Discussion

A number of issues are further discussed in the following sections:

6.1 Mitigation of spalling and damage

The excavation of an underground opening will always create changes, many of which, but not all, can be eliminated or mitigated by proper design and construction.

6.1.1 Mitigation of spalling

Spalling at a free surface underground in crystalline rock occurs when the local stress is greater than the local strength of the rock. The local stress in the underground opening is dependent on several factors. Examples include the depth to the surface, the orientation of the opening in relation to principal stresses and the shape of the opening, but also the local geological conditions. In the case of the repository, two more factors will contribute: the stresses created by the thermal load from the emplaced canisters and the supporting pressures created by the swelling of the buffer in the deposition holes and swelling of the backfill materials in the deposition tunnels and other underground openings.

The risk of spalling can be decreased by selecting the depth of the repository and orienting the deposition of the deposition tunnels in the direction of maximum horizontal stress. However, the design should account for the fact that the overall stress situation may change due to glaciation, variations in the tectonic forces and other regional stress changes that may occur over tens of thousands of years.

The shape of the opening is important for the local stress field, but the final shape is also dependent on other factors not related to the rock mechanics, such as the design of deposition equipment, the interest in minimizing the volume of backfilling, etc and the overall optimisation needs to address all these factors.

The stress situation is also dependent on the local geological conditions since the heterogeneity of the rock and its varying rock properties will also lead to a certain variability of the virgin stresses.

The thermal load from the canisters will cause local stress changes in the vicinity of the deposition holes and the deposition tunnel. The modelled transient temperature distribution of course has uncertainties such as variability of the burn-up rate of the spent fuel, the water content of the buffer and the closure of potential gaps of air before the state of full saturation and complete contact between the canister, the buffer and the rock is achieved. The time to saturate the buffer (and develop full swelling pressure) was in SR-Can estimated to be in the region of 5 to 2,000 years depending on the hydraulic conductivity of the rock. The time to arrive at the maximum temperature at the wall of the deposition hole is estimated to be approximately 30 years (see Figure 4-3). Therefore, for "dry" rock (the hydraulic conductivity of the rock matrix ~ 10^{-13} m/s, assuming no contribution of water from fractures in the deposition holes or from the damage zone or backfill material) it can be expected that the thermal pulse will move faster than the increase in swelling pressure with the associated higher risk of spalling due to little or no support pressure. One of the most important factors in evaluating the risk of spalling is the variability of the rock strength, and it is assumed that variation in local strength properties is greater than the variation of the imposed load. As the position of the deposition hole can be decided after detailed site characterisation, rock of lower strength would be avoided. Based on the extensive studies (Mine-by in Canada and APSE in Sweden), the process understanding and predictive capability of spalling is deemed to be solid.

6.1.2 Mitigation of the zone of damage

The relation between damage and the excavation method has been demonstrated in several studies and experiments.

By using mechanical excavation, the damage is limited to a few centimetres. The damage is mainly a crushed zone of a few millimetres from the rock wall, a zone of increase fracturing of a few millimetres followed by a zone of increased micro-cracks as evidenced from the excavation of the deposition holes at Olkiluoto. The damage is, however, dependent on the equipment used, such as cutter design, and how it is operated, such as the thrust used. It is assumed that the damage caused will not contribute to any significant axial conductivity along the opening.

Excavation of the deposition tunnel by a Tunnel Boring Machine (TBM) will also cause minor damage to the rock at the position where the grippers push into the wall so that a thrust of approximately 300 kN per cutter may be applied. Here too, the damage to the rock is very minor as most of the energy is applied at the tunnel face. It was noted in the ZEDEX experiment that the seismic energy radiated into the walls of the tunnel was only in the order of 400 W.

Excavation by drill and blast creates a significant amount of damage as well as an irregular, rough shape to the opening. There are many studies that correlate damage with the concentration of explosives used, and therefore the general principle is to use small amounts of explosives close to the periphery. Usually, there is a thicker damaged zone below the floor caused by the heavier explosive charges used close to the floor. Factors that contribute to lower specific charging and less damage are drilling the blast holes both close to each other and straight in order to lower the maximum burden, as well as using exact ignition times by utilising electronic detonators. The damage is, of course, also very dependent on the local geological conditions. SKB is planning to excavate the deposition tunnel in parts by using a pilot for the upper part of the galley and afterwards slashing the floor of the tunnel. This is because a split of this type diminishes the damage caused by the use of less explosives. As the distribution of explosives varies along the blast holes (extra primers at the bottom of the blast holes and no charge at all close to the tunnel face) and the start and end positions of the boreholes for each blast round is different owing to the look-out of the blast holes, it is considered likely that the damage is discontinuous between adjacent rounds. The absence of axial transmissivity along adjacent blast rounds has been indicated in the AECL tests and in tests performed by SKB, but additional studies are needed in order to be able to draw firm conclusions. The alternative method of wire-sawing the floor of the deposition tunnel to create a smooth surface for the deposition equipment would of course also remove a large part of the EDZ around the tunnel. Assuming a width of damage of 0.3 m in the roof and walls, and 1.0 m in the floor, and tunnel geometry as in Figure 2-2, the total volume of the damage zone per m of tunnel is reduced from around 8 m^2 ($12 \cdot 0.3 + 4.2 \cdot 1.0$) to approximately 4 m^2, i.e. 50% of the damage zone area remains. However, the damage in the floor is more important than in the walls and roof, as the floor has a shorter distance to the canisters.

The overall effect of the EDZ on the axial flux as a result of choice of Tunnel Boring Machine (TBM) or drill and blast was also discussed in the paper by /Pusch 2005/. Based on the set assumptions it was concluded that the total axial flux over an 80 m^2 near-field section area was about 100 times higher for the drill and blast tunnel compared to a tunnel excavated by a TBM.

The discussion so far has been restricted to the excavation process, but consideration should also be given to other types of damage. As the maximum inflow to the deposition tunnels and deposition holes should be very low in order to permit emplacement of the buffer and the backfill, the use of grouting is foreseen. The most typical grouting operation is to inject a grout into long boreholes before excavation. SKB is planning to use low-pH cement as well as silica sol for grouting very narrow fractures. To plan and execute the grouting, long holes (10–20 m) will be drilled in front of the tunnel face which will give rise to damage if the grout holes are drilled outside the periphery of the tunnel. However, the grout holes are unlikely to form continuous stretches over the full length of the tunnel.

The standard construction practice is to scale the rock manually or by using hydraulic hammers. The damage and impact on the rock caused by scaling have not yet been investigated. At present, the mining industry is also studying the use of high-pressure water to scale the rock. The objective is to take down any loose rock that could fall down. Scaling is normally performed immediately after excavation and is then repeated, where the tunnels are unlined, after a few years of operation. The need for maintenance scaling is a sign of the rock loosening. The frequency of maintenance scaling and the volume of rock scaled down are very much dependent on the geological conditions as well as on the excavation method used. Äspö HRL is no exception, but the need for scaling is much lower where a Tunnel Boring Machine was used /Andersson and Söderhäll 2001/.

The use of ground support elements, such as bolts and shotcrete (sprayed concrete), may be used to secure the ground. Bolts are typically made of steel in lengths of 1.5–4 m and are inserted in boreholes and grouted with cement grout. The borehole is a form of "damage" to the rock whereby the bolt holes could short-cut the excavation damaged zone. However, if bolts are needed for reasons of occupational safety, their use cannot probably be avoided.

With respect to shotcrete, it is anticipated that such material will be removed before backfilling is carried out for most cases. However, the removal of shotcrete, for example by using hydraulic hammers, could very well cause damage to the rock. This possibility needs to be evaluated.

6.1.3 Mitigation of the zone of disturbance

As noted previously, the zone of disturbance has been connected to a change in state due to the creation of an underground opening, and these changes are reversible. The effects correlated to the mechanically reversible changes can for crystalline rock be limited to a distance of a few equivalent radii of the underground opening. Therefore, a smaller opening creates a smaller influence radius. The stress changes will also be lowered due to the supporting pressures caused by the swelling of the buffer and the backfill material.

The underground opening creates a hydrogeological sink. The drawdown causes the mixing of different types of groundwater which could affect the zone closest to the opening due to precipitations, dissolutions and bacterial growth.

6.2 Properties of the backfill and the interface with rock.

The relative importance of the damage zone in relation to the backfill and the interface between the backfill and rock was discusses at the workshop in Canada 2002. /Chandler et al. 2003/.

Assuming a rock cylinder with a radius of 20 m, a circular tunnel with a radius of 2 m, and the extent of the damage zone to be 0.33 m, then the EDZ would need to be 300 times more permeable than the host rock or 3 times greater than the backfill to become the preferred hydraulic pathway. A smaller, but more permeable EDZ would need to be 1,000 times more permeable than the host rock or 10 times more permeable than the backfill to become the preferred flow path /Chandler et al. 2002/. The backfill-tunnel interface was also discussed. It was concluded that an interface between the tunnel and carefully placed pre-compacted blocks of sand-bentonite materials and voids filled with a pneumatically-placed mixture of sand and bentonite, had a transmissivity of 10^{-11} m^2/s or less, but an open air gap of only 0.6 mm would convert to a transmissivity of 10^{-4} m^2/s assuming a parallel plate model. Thus the tight interface is very important for guaranteeing the backfilled deposition tunnel a barrier function. Therefore, the "damage zone" is not only the properties of the damage outside the periphery but also the geometry of the periphery itself. A rugged surface due to poor drilling, poor blasting or excessive scaling will influence the ability to backfill the tunnel and if so, the consequences might be more severe than the EDZ in itself, see also Section 2.3. The rugged profile will also contribute to higher local stresses around the opening.

6.3 Testing the excavation response

The treatment of spalling and the excavation-induced damage zone in the safety case is mainly devoted to the zone being a potential pathway for the migration of radionuclides. However, this requires an evaluation of the behaviour during the period and under the conditions when radionuclide release could actually occur (after canister breaching and breakthrough of radionuclides through the buffer which may be tens of thousands of years or even more.

As described in Section 2.1, SKB needs to verify that the repository – the "product" – fulfils the expected functions and stipulated requirements. Such tests can generally speaking be at system, sub-systems or component level. During the implementation, quality plans are prepared and one aspect of these plans is the "control programme" which assures that the execution of the plans are as required. A damage zone test is thus basically a test of the sub-system "rock" with its associated requirements.

The purposes of a damage zone test are basically two-fold. The first is the need for a verification test that the construction activities will produce a damage zone with data that is within the range assumed in the requirements. The second purpose is to test and propose efficient methods that can be used in the daily operation of the repository, to infer the properties of the damage during repository construction or to conclude that the damage is within the assumed range.

A verification test should be relevant for and simulate the expected conditions for a saturated and heated repository. However such a large-scale test combining thermal mechanical, hydrogeological, chemical/axial processes and their internal couplings would be very difficult to set up and to interpret. It is therefore suggested that the focus be centred on the excavation-induced damage and coupling to the potential of axial connectivity of the transmissivity, that is an Excavation Damage Zone (EDZ) experiment focussed on hydrogeology but with supporting information from geology, geophysics, rock mechanics and chemistry/biology. Issues concerning spalling should be addressed in separate tests, although they may be connected to an EDZ test.

To meet the first purpose of the test, to verify that the construction activities produce an EDZ within stipulated ranges a number of conditions are to be met:

1. The assumptions to be made in the safety case concerning the EDZ and its evolution and properties are to be stated.
2. The excavation of the deposition tunnel (and deposition) hole should be excavated with design, methods and quality that are commensurate with the methods and quality to be used when the final repository is to be excavated. It is furthermore assumed that the test tunnel is so tight that grouting to limit seepage of groundwater would not be necessary. The effects on EDZ due to common excavation flaws should be studied or evaluated.
3. The test should be executed at a site location where bedrock properties, state of stress and other factors should be similar to the conditions at the repository site.
4. The test should as far as possible simulate the sealed and saturated repository. Therefore, the tests should preferably be performed in saturated conditions and the test scale should be relevant to support or reject the hypothesis that the axial conductivity is connected over several rounds for a deposition tunnel excavated by careful drilling and blasting.

To fulfil the second purpose of such a test – to devise methods and instruments to be used in daily operation to verify that the EDZ would be reasonably within the assumed range – it is here assumed that several methods and instruments will be used during the verification tests described above, to deduce particular individual methods or methods in combination which would infer that the properties of the EDZ are or are not as assumed in the safety case. It is further assumed that the "control programme" for construction will verify that design and excavation of the deposition tunnel (and deposition) hole is as required or report deviations. As indicated here, the planning of effective tests for the properties of the EDZ should be closely connected to the control programme for excavation. As long as the repository is constructed as planned, there should be no need to verify the properties of EDZ on a daily basis. However, when conditions change or deviate with respect to design, excavation, stress and bedrock properties, investigations of the EDZ should be undertaken.

The second purpose of EDZ testing as described above assumes that the EDZ mainly evolves as a result of the excavation process and can be studied in connection with the excavation. The inherent assumption is that the extent and properties of the EDZ will not be significantly modified when the deposition tunnel (deposition hole) is backfilled, the repository saturated, heated and cooled down. Here we can anticipate potential processes that contribute to fracture closure such as self-healing and processes that could open or extend fractures, such as loosening due to heating/cooling. The evolutionary processes are described in detail in the safety reports, and are not easily tested after the deposition tunnels have been plugged off. It is expected that a monitoring programme will be operating during the repository implementation, and a programme of this type could possibly track some of the processes, such as the mechanical development, by using micro-seismic and acoustic emission systems. However, it is unlikely that potential development of the EDZ will be able to be monitored directly.

7 Conclusions

Previous studies, tests and experiments in crystalline rock have been compiled and evaluated with an emphasis on the change in hydrogeological properties in a deposition tunnel and a deposition hole due to the creation of the underground opening.

The most important factor controlling the excavation damage is the choice of excavation method. The use of mechanical excavation may create damage less than 3 cm from the rock wall where increased micro-fracturing contributes to an increase in hydraulic conductivity. Several methods and sample sizes have been used to characterise the damage. For a Tunnel Boring Machine, < 5 mm of damage were recorded at Äspö HRL and at Grimsel in Switzerland. The hydraulic conductivity at Äspö HRL, for example, was determined to be in the region of 10^{-9} m/s for a distance of 1–2 mm from the tunnel periphery, $2 \cdot 10^{-11}$ m/s at a depth of 5 mm, and 10^{-13} m/s at a distance of 30 mm from the tunnel wall. The virgin crystal matrix of the rock is in the region of 10^{-13} m/s to 10^{-14} m/s. The results of hydraulic conductivities are dependent on the test methods (scales) used.

Excavation of the deposition tunnel using drill and blast would create much more extensive damage (several decimetres) than by using a TBM (a few centimetres). The extent of the damage depends very much on the accuracy of drilling for the blast holes, the amount of explosives used in the blast holes close to the periphery and the local geological conditions. Several methods have been used to estimate the hydraulic conductivity and the values depend somewhat on the method used. Based on the study reported here, it is suggested that the hydraulic conductivity of the damage zone should be an absolute value and not be presented in relation to the undisturbed hydraulic conductivity. A reasonable value for the hydraulic conductivity of the damage zone is 10^{-8} m/s. This magnitude has been obtained during several tests in hard rock (see Table 3-3) where excavation was of good quality and measured by integrating measurement under saturated conditions along the tunnel floor. Point observations of the hydraulic conductivity has provided both lower and higher individual results.

A matter of discussion is whether the EDZ is continuous or discontinuous over adjacent blast rounds, and the results based on the data from experiments at AECL URL and Äspö HRL are inconclusive.

The compilation shows that spalling is the most important factor which will contribute to an extended axial transmissivity along a deposition tunnel. Measured hydraulic conductivity based on spalling in a test tunnel in crystalline rock at the AECL URL in Canada was in the order of 10^{-6} m/s and significantly higher than the increased hydraulic conductivity due to the damage caused by the excavation process.

Several methods and instruments have previously been used to characterise the damage zone, and the work in progress will add to this knowledge. It is concluded that a proper “control programme” can be established, but that additional work is necessary to verify the reliability of such a programme.

References

Andersson C, Söderhäll J, 2001. Rock mechanical conditions at the Äspö HRL. A study of the correlation between geology, tunnel maintenance and tunnel shape. SKB R-01-53, Svensk Kärnbränslehantering AB.

Andersson J C, 2007. Äspö Pillar Stability Experiment. Final report. Rock mass response to coupled mechanical thermal loading. SKB TR 07-01, Svensk Kärnbränslehantering AB.

Autio J, 1997. Characterization of the excavation disturbance caused by boring of the experimental full scale deposition holes in the research tunnel at Olkiluoto. SKB TR 97-24, Svensk Kärnbränslehantering AB.

Autio J, Hjerpe T, Siitai-Kauppi M, 2005. Porosity, diffusivity and permeability of EDZ in crystalline rock and effect on the migration in a KBS-3 type repository. In EU, 2005. Impact of excavation disturbed or damaged zone (EDZ) on the performance of radioactive waste geological repositories. Proc European Commission Cluster Conf and Workshop. 149–155. Luxembourg Nov 3–5, 2003. European Commission EUR 21028 En.

Autio J, Gribi P, Johnson L, Marschall P, 2006. Effect of excavation damaged zone on gas migration in a KBS-3H type repository at Olkiluoto. Physics and Chemistry of the Earth 31, 649–653, Elsevier.

Brantberger M, Zetterqvist A, Arnbjerg-Nielsen T, Olsson T, Outters N, Syrjänen P, 2006. Final repository for spent nuclear fuel. Underground design Forsmark, Layout D1, SKB R-06-34, Svensk Kärnbränslehantering AB.

Bäckblom G, Martin C D, 1999. Recent experiments in hard rocks to study the excavation response. Implications for the performance of a nuclear waste geological repository. Tunnelling and Underground Space Technology. Vol 14, No 3, 377–394.

Bäckblom G, Christiansson R, Lagerstedt L, 2004. Choice of rock excavation methods for the Swedish deep repository for spent nuclear fuel. SKB R-04-62, Svensk Kärnbränslehantering AB.

Börgesson L, Pusch R, Fredriksson A, Hökmark H, Karnland O, Sandén R, 1992. Final report of zones disturbed by blasting and stress release. Stripa Project 92-08, Svensk Kärnbränslehantering AB, Stockholm.

Chandler N A, Kozak E T, Martin C D, 1996. Connected pathways in the EDZ and the potential for flow along tunnels. In: Martino J B, Martin C D (eds.), 1996: Designing the Excavation Disturbed Zone for a nuclear repository in hard rock. Proc. Can. Nucl. Soc. Sept 20, 1996, Winnipeg, Canada. Canadian Nuclear Society, Toronto. ISSN 0-919784-44-5.

Chandler N A, Cournut A, Dixon D A, Fairhurst C, Hansen F, Gray M, Hara K, Ishijima Y, Kozak E, Martino J, Matsumoto K, McCrank G, Sugita Y, Thompson P, Tillerson J, Vignal B, 2002. The five-year report of the Tunnel Sealing Experiment: An international project of AECL, JNC, ANDRA and WIPP. Atomic Energy of Canada Limited Report AECL-12727. Chalk River, Canada.

Chandler N A, Martino J B, Read R S, 2003. The EZD exists – So what? In: Martino, J.B. 2003. The 2002 international EDZ workshop – The Excavation Damage Zone – causes and effects. Ontario Power Generation, Nuclear Waste Management Division Report 06819-REP-01200-10105-R00. Toronto, Ontario.

CNS, 1996. Designing the Excavation Disturbed Zone for a nuclear repository in hard rock, Proceedings of the EDZ Workshop, International Conference on Deep Geological Disposal of Radioactive Wast*e*, Winnipeg, Canada, 20 September 1996, Canadian Nuclear Society.

Dershowitz W, Winberg A, Hermanson J, Byegård J, Tullborg E-L, Andersson P, Mazurek M, 2003. Äspö Task Force on modeling of groundwater flow and transport of solutes. Task 6C. A semi-synthetic model of block scale conductive structures at the Äspö HRL. SKB IPR-03-13. Svensk Kärnbränslehantering AB, Stockholm.

Emsley S, Olsson O, Stenberg L, Alheid H-J, Falls S, 1997. ZEDEX – a study of damage and disturbance from tunnel excavation by blasting and tunnel boring. SKB TR 97-30, Svensk Kärnbränslehantering AB, Stockholm

EU, 2005. Impact of excavation disturbed or damaged zone (EDZ) on the performance of radioactive waste geological repositories. Proc. European Commission Cluster Conf and Workshop. Luxembourg Nov 3–5, 2003. European Commission EUR 21028 En.

Follin S, Levén J, Hartley L, Jackson P, Joyce S, Roberts D, Swift B, 2007. Hydrogeological characterisation and modelling of deformation zones and fracture domains, Forsmark modelling stage 2.2. SKB R-07-48. Svensk Kärnbränslehantering AB.

Frost L, Everitt R A, 1997. Excavation damage zone tracer experiments in the floor of the Room 415 test tunnel. AECL-11 640, COG-96-321-1. AECL, Manitoba.

Gray M, 1993. OECD/NEA International Stripa Project 1980–1992. Overview Volume III, Engineered Barriers. Svensk Kärnbränslehantering AB, Stockholm

Hartley L, Hoch A, Jackson P, Joyce S, McCarthy R, Rodwell W, Swift B, Marsic N, 2006a. Groundwater flow and transport modelling during the temperate period for the SR-Can assessment. Forsmark area – version 1.2. SKB R-06-98, Svensk Kärnbränslehantering AB.

Hartley L, Hoch A, Jackson P, Joyce S, McCarthy R, Swift B, Gylling B, Marsic N, 2006b. Groundwater flow and transport modelling during the temperate period for the SR-Can assessment. Laxemar subarea – version 1.2. SKB R-06-99, Svensk Kärnbränslehantering AB.

Hudson J, Bäckström A, Rutqvist J, Jing L, Backer T, Chijimatsu M, Feng X, Kobayashi A, Koyama T, Lee H-S, Pengzhi Pan P, Rinne M, Shen B, 2008. Final Report of DECOVALEX-THMC Task B (EDZ Guidance Document) Characterising and modelling the Excavation Damaged Zone (EDZ) in crystalline rock in the context of radioactive waste disposal. SKI Report 2008:43 Svenska Strålskyddsmyndigheten.

Hökmark H, Fälth B, Wallroth T, 2006. T-H-M couplings in rock. Overview of results of importance to the SR-Can safety assessment. SKB R-06-88, Svensk Kärnbränslehantering AB.

Jarsjö J, Destouni G, Gale J, 2001. Groundwater degassing and two-phase flow in fractured rock: Summary of results and conclusions achieved during the period 1994–2000. SKB TR 01-13, Svensk Kärnbränslehantering AB.

Jonsson M, Berglund J, Bäckström A, Johansson M, Mas Ivars D, Olsson M, Quanhong F, 2009. Äspö Hard Rock Laboratory. Studies of factors that affect and controls the Excavation Damages/Disturbed Zone. SKB R 09-17, Svensk Kärnbränslehantering AB.

Joyce S, Hoek J, Hartley L, 2008. SR-Site Pre-modelling: Sensitivity studies of hydrogeological model variants for the Laxemar site using CONNECTFLOW. SKB R-08-108, Svensk Kärnbränslehantering AB.

Kuzyk G W, Martino J B, 2008. Development of excavation technologies at the Canadian Underground Research Laboratory. In: Bäckblom, G, (ed.) 2008. Proc. International Conference Underground Disposal Unit Design & Emplacement Processes for a Deep Geological Repository, Prague 16–18 June 2008. Andra, France.

Laaksoharju M, Gimeno M, Pedersen K, Auqué L F, Javier B, Gómez J B, Acero P, 2008. Hydrogeochemical and microbiological effects on EDZ fractures. SKB R-09-05, Svensk Kärnbränslehantering AB.

Lang P A, 1989. Room 209 excavation response test in the underground research laboratory. In: OECD/NEA, 1989: Excavation response in geological repositories for radioactive waste. Proc. NEA Workshop 26–28 April 1988, Winnipeg. OECD, Paris.

Liedtke L, 2005. Spread of contaminants in excavation disturbed zone based on results of hydraulic in-situ tests in jointed rock. In: EU, 2005. Impact of excavation disturbed or damaged zone (EDZ) on the performance of radioactive waste geological repositories. Proc. European Commission Cluster Conf and Workshop. 157–162. Luxembourg Nov 3–5, 2003. European Commission EUR 21028 En.

Ludvigson J-E, Nordqvist R, Ekman L, Hansson K, 2001. Äspö Hard Rock Laboratory. Backfill and Plug Test. Hydraulic testing of core drilled boreholes in the Zedex drift. SKB IPR-09-01, Svensk Kärnbränslehantering AB.

Marschall P, Fein E, Kull H, Lanyon W, Liedtke L, Müller-Lyda I, Shao H, 1999. Conclusions of the Tunnel Near-Field Programme (CTN). Nagra Technical Report 99-07, Nagra, Wettingen.

Martin C D, Kozak E T, Chandler N A, 1992. Hydraulic properties of the excavation-disturbed zone around underground openings. Proc 45th Geotechnical Conf. Toronto, October 26–28, Canadian Geotechnical Society. Paper 89, 9p.

Martin C D, Christiansson R, Söderhäll J, 2001. Rock stability considerations for siting and constructing a KBS-3H repository. Based on experiences from Äsol HRL, AECL's URL tunnelling and mining. SKB TR-01-38, Svensk Kärnbränslehantering AB.

Martin D, 2005. Preliminary assessment of potential underground stability (wedge and spalling) at Forsmark, Simpevarp and Laxemar sites. SKB R-05-71, Svensk Kärnbränslehantering AB.

Martino J B, Thompson P M, Chandler N A, Read R S, 1997. The in situ stress program at AECL's Underground Research Laboratory. Ontario Hydro, Nuclear Waste Management Division Report 06819-REP-01200-0053-R00.

Martino J B, 2000. A review of excavation damage studies at the underground research laboratory and the results of the excavation damage zone study in the tunnel sealing experiment. Ontario Power Generation, Nuclear Waste Management Division Report 06819-REP-01200-10018-R00. Toronto, Ontario.

Martino J B, 2003. The 2002 International EDZ Workshop – The Excavation Damage Zone – causes and effects. Ontario Power Generation, Nuclear Waste Management Division Report 06819-REP-01200-10105-R00. Toronto, Ontario.

Martino J B, Chandler N A, 2004. Excavation-induced damage at the Underground Research Laboratory. International Journal of Rock Mechanics & Mining Sciences; 41: 1413-1426 Elsevier.

Martino J B, Kozak E. Woodcock D, Stroes-Gascoyne S, 2004. The Blast Damage Assessment Project at the URL. Ontario Power Generation, Nuclear Waste Management Division Report 06819-REP-01200-10123-R00. Toronto, Canada.

Martino J B, Dixon D, Stroes-Gascoyne S, Guo R, Kozak E T, Gascoyne M, Fujita T, Vignal B, Sugita Y, Masumoto K, Saskura T, Bourbon X, Gingras-Genois A, Collins D, 2008. The Tunnel Sealing Experiment. 10 Year Summary Report.URL-121550-REPT-001. AECL, Ontario.

Matsui H, Sato T, Sugihara K, Kikuchi T, 1998. Overview of the EDE (Excavation Disturbance Experiment) -II at the Kamaishi mine. Kamaishi Int. Workshop Proc. 24 – 25 Aug. 1998. PNC TN7413 98-023. JNC, Tokyo.

McEwen T, 2005. Review of the conclusions of previous EDZ workshops. In EU, 2005. Impact of excavation disturbed or damaged zone (EDZ) on the performance of radioactive waste geological repositories. Proc European Commission Cluster Conf and Workshop. 137-142. Luxembourg Nov 3–5, 2003. European Commission EUR 21028 En.

NEA, 1989. Excavation response in geological repositories for radioactive waste, Proceedings of an NEA Workshop, Winnipeg, Canada, 26–28 April 1988, OECD/NEA, Paris.

NEA, 2002. Characterisation and representation of the Excavation Disturbed or Damaged Zone (EDZ). Lessons learnt from a SEDE Topical Session, September 1998, Paris. NEA Report NEA/RWM(2002)1.

Neretnieks I, 2006. Flow and solute transport in a zone damaged due to spalling. SKB R-06-91, Svensk Kärnbränslehantering AB.

Nowak T, 2005. LASGIT: Hard Rock Laboratory Äspö. Hydraulic tests with surface packer systems. BGR 10982/2005, Hanover. SKB IPR-07-14, Svensk Kärnbränslehantering AB.

Olsson O, Winberg A, 1996. Current understanding of extent and properties of the excavation disturbed zone and its dependency of excavation method. Martino J B, Martin C D (eds.), 1996:

Designing the Excavation Disturbed Zone for a nuclear repository in hard rock. Proc. Can. Nucl. Soc. Sept 20, 1996, Winnipeg, Canada. Canadian Nuclear Society, Toronto. ISSN 0-919784-44-5.

Olsson M, Ouchterlony F, 2003. New formula for blast induced damage in the remaining rock (In Swedish). SveBeFo Rapport 65, Stockholm ISSN 1104 – 1773.

Olsson M, Niklasson B, Wilson L, Andersson C, 2004. Experiences of blasting of the TASQ tunnel. SKB R-04-73, Svensk Kärnbränslehantering AB.

Olsson M, Markström I, Pettersson A, 2008. Methodology study for documentation and 3D modelling of blast induced fractures. SKB R-08-90, Svensk Kärnbränslehantering AB.

Ouchterlony F, 1997. Prediction of crack lengths in rock after cautious blasting with zero interhole delay. SveBefo Rapport 31. Stockholm

Ouchterlony F, Olsson M, Bergqvist I, 2002. Towards new Swedish recommendations for cautious perimeter blasting'. Int. J. for Blasting and Fragmentation V 6, 235–261.

Pettersson S, Lönnerberg B, 2008. Final Repository for spent nuclear fuel in granite – The KBS-3V concept in Sweden and Finland. Proc. Proc. Underground Disposal Unit Design & Emplacement Processes For a Deep Geological Repository. "Operational & Safety Considerations". Prague 16–18 June 2008 (in print).

Pusch R, 2005. Lessons learned with respect to EDZ in crystalline rock. In: EU, 2005. Impact of excavation disturbed or damaged zone (EDZ) on the performance of radioactive waste geological repositories. Proc. European Commission Cluster Conference and Workshop. 137-142. Luxembourg Nov 3–5, 2003. European Commission EUR 21028 En.

Pusch R, 2008. Geological storage of radioactive waste. Springer Verlag, 379 p.

Read R S, Martino J B, Dzik E J, Oliver E J, Falls S, Young R P, 1997. Analysis and interpretation of AECL's Heated Failure Test. Ontario Hydro Report 06819-REP-01200-0070-R00. Ontario Power Generation, Toronto.

Read R S, 2004. 20 years of excavation response studies at AECL's Underground Research Laboratory. International Journal of Rock Mechanics & Mining Sciences 41 (2004) 1251–1275. Elsevier Ltd.

Reyes-Montes J M, Pettitt W S, Young R P, 2008. Enhanced spatial resolution of caving-induced microseismicity. Proc. MassMin 2008. Luleå June 9–11. Luleå Univ. of Technology. 961–970.

Sabet B, Shao H, Autio J, Elorza F J, Cañamon I, Perez J C, 2005a. EDZ assessment in the Febex II Project. In: EU, 2005. Impact of excavation disturbed or damaged zone (EDZ) on the performance of radioactive waste geological repositories. Proc European Commission Cluster Conference and Workshop. 137-142. Luxembourg Nov 3–5, 2003. European Commission EUR 21028 En.

Sabet B, Shao H, Autio J, Elorza F J, 2005b. FEBEX II project post-mortem analysis: EDZ-assessment. Publicación técnica 11/2004. Empresa Nacional de Residuos Radiactivos, Madrid. 134 op.

Sabet B, Shao H, Autio J, Elorza J C, Cañamon I, Perez J C, Pusch R, Svemar C, 2005c. EDZ in granite. In: Proc. Alonso, E.E, Ledesma A. (eds.) International symposium on large scale field tests in granite, Sitges, Barcelona, Spain 12–14th November 2003, pp 151–162. Balkema Publishers.

Shin S W, Martin C D, Park E S, Christiansson R, 2007. Methodology for estimation of excavation damaged zone around tunnels in hard rock Proc. 1st Canada-US Rock Mechanics Symposium, Vancouver, Eberhardt, E.; Stead, D. & Morrison, T. (ed.) Taylor & Francis Group, London, 2007, 1, 485–491

Silvast M, Wiljanen B, 2008. Onkalo EDZ-measurements using Ground Penetrating Radar (GPR) method. Working Report 2008-58. Posiva Oy.

Simmons G R, 1992. The Underground Research Laboratory Room 209 Excavation Response Test – A Summary Report. AECL-10564, Manitoba, Canada.

SKB, 1999. SR 97 – Deep repository for spent nuclear fuel. SR 97 – Post-closure safety. Main report – Vol. I, Vol. II and Summary. SKB TR-99-06, Svensk Kärnbränslehantering AB, Stockholm

SKB, 2002. General design requirements for the deep repository in the KBS-3-system. (In Swedish). SKB R-02-44. ISSN 1402-3091. Svensk Kärnbränslehantering AB

SKB, 2006a. Long-term safety for KBS-3 repositories at Forsmark and Laxemar – a first evaluation. Main Report of the SR-Can project. SKB TR-06-09, Svensk Kärnbränslehantering AB.

SKB, 2006b. Data report for the safety assessment SR-Can. SKB TR-06-25, Svensk Kärnbränslehantering AB.

SKB, 2006c. Model summary report for the safety assessment SR-Can SKB TR-06-26. Svensk Kärnbränslehantering AB.

SKB, 2007. RD&D-Programme 2007. Programme for research, development and demonstration of methods for the management and disposal of nuclear waste, including social science research. Svensk Kärnbränslehantering AB.

Tsang C-F, Bernier F, 2005. Definitions of excavation disturbed zone and and excavation damaged zone. In: EU, 2005. Impact of excavation disturbed or damaged zone (EDZ) on the performance of radioactive waste geological repositories. Proc European Commission Cluster Conf and Workshop. 137-142. Luxembourg Nov 3–5, 2003. European Commission EUR 21028 En.

Zuidema P, 2005. The Importance of the Excavation Damaged Zone for Post-Closure Safety of Deep Geological Repositories: Some Introductory Remarks. In EU, 2005. Impact of excavation disturbed or damaged zone (EDZ) on the performance of radioactive waste geological repositories. Proc European Commission Cluster Conf and Workshop. 137-142. Luxembourg Nov 3–5, 2003. European Commission EUR 21028 En.

Winberg A, Chan T, Griffiths P, Nakka B, 1989. Post-excavation analysis of a revised hydraulic model of the Room 209 fracture, URL, Manitoba: A part of the joint AECL/SKB characterization of the 240 m level at the URL, Manitoba, Canada. SKB TR 89-27, Svensk Kärnbränslehantering AB.

Öhberg A (ed), Hellä P, Koskinen L, Mellanen S, 2008 . Final report of EDZ Programme. Working Report 2008-66. Posiva Oy.

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