Technical Report 98-01

Grimsel Test Site

Excavation Disturbed Zone Experiment (EDZ)

July 2012

B. Frieg & P.C. Blaser (eds.)


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July 2012

B. Frieg \(^{1}\) & P.C. Blaser \(^{2}\) (eds.)

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This report was prepared on behalf of Nagra. The viewpoints presented and conclusions reached are those of the author(s) and do not necessarily represent those of Nagra.

ISSN 1015-2636

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Foreword

Concepts which envisage the disposal of radioactive waste in geological formations are crucially dependent on a thorough knowledge of the host rock and neighbouring rock strata. Since 1984, NAGRA has been operating the Grimsel underground rock laboratory (Felslabor Grimsel – FLG), which complements NAGRA's work on repositories. This laboratory, which provides a generic test rock environment, lies 450 m below the eastern flank of the Juchlistock. It is located in the granitic rock of the Aar-Massif, at a height of 1730 m and can be reached via a horizontal access tunnel.

The most important purposes of the Grimsel underground rock laboratory are:

- The accumulation of know-how in planning, execution and interpretation of underground experiments in various scientific and technical fields.
- Acquisition of practical experience in the development and use of those experimental methods, measurement procedures and equipment which could be used in the search for potential sites.
- Experimental investigation of processes crucial to the safety of a radioactive waste repository.

In 1984, as part of a German/Swiss collaboration, various experiments were initiated by NAGRA and its German partner, the Federal Institute for Geosciences and Natural Resources (Bundesanstalt für Geowissenschaften und Rohstoffe – BGR) together with the Research Centre for Environment and Health (Forschungszentrum für Umwelt und Gesundheit – GSF). Work performed by the German partner was sponsored by the Federal Ministry for Education, Science, Research and Technology (Bundesministerium für Bildung, Wissenschaft, Forschung und Technologie – BMBF). Phase IV (1994 – 1996) of the investigation programme has already been completed. A special issue of the Nagra Bulletin 1996 (German version "Nagra informiert" 27/1996) provides an overview of the investigation programme, and the status up to 1996 is described.

International collaboration in the FLG has been strengthened through the years by collaboration agreements with the following partner organisations: ANDRA (France), ENRESA (Spain), EU (European Union), PNC (Japan), SKB (Sweden) and US-DOE (United States).

The **Excavation Disturbed Zone Experiment (EDZ)** was carried out as part of the near-field programme during Phase IV of the Grimsel Test Site investigation programme. EDZ is a multidisciplinary study aimed at investigating the hydraulic regime of the near-field of drilled tunnel sections under fully saturated conditions and contributing to the development of methods for measuring and modelling axial water flow along tunnels and caverns.

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1 Now: Gesellschaft für Anlagen- und Reaktorsicherheit (GRS)
2 Now: Bundesministerium für Wirtschaft (BMWi)
3 Now: Japan Atomic Energy Agency, Japan (JAEA)
Vorwort


Das Felslabor Grimsel dient folgenden übergeordneten Zielsetzungen:

- Aufbau von Know-how in der Planung, Ausführung und Interpretation von Untertageversuchen in verschiedenen wissenschaftlichen und technischen Fachgebieten
- Erwerb praktischer Erfahrung in der Entwicklung und der Anwendung von Untersuchungsmethoden, Messverfahren und Messgeräten, die für die Erkundung von potentiellen Standorten in Frage kommen
- Experimentelle Untersuchungen von Prozessen, die für die Sicherheit eines Endlagers für radioaktive Abfälle entscheidend sind.


Die internationale Zusammenarbeit im FLG wurde im Verlauf der Jahre durch Zusammenarbeitsverträge mit folgenden Partnerorganisationen ANDRA (Frankreich), ENRESA (Spanien), EU (Europäischen Union), PNC (Japan), SKB (Schweden) und US-DOE (Vereinigte Staaten) ausgebaut.

Der Versuch Excavation Disturbed Zone (EDZ) wurde als Teil des Nahfeldprogrammes innerhalb der Phase IV der Untersuchungen im Felslabor Grimsel durchgeführt. EDZ ist eine interdisziplinäre Studie mit der Zielsetzung, die Auflockerungszone im Nahbereich von Stollen unter vollständig gesättigten Bedingungen zu erkunden und zugleich zur Entwicklung einer geeigneten Untersuchungsmethodik für die Messung und Modellierung axialer Wasserflüsse entlang von Stollen beizutragen.

4 heute: Gesellschaft für Anlagen- und Reaktorsicherheit (GRS)
5 heute: Bundesministerium für Wirtschaft (BMWi)
6 heute: Japan Atomic Energy Agency, Japan (JAEA)
Préface

Lors de l’élaboration de concepts concernant le stockage final des déchets radioactifs dans des formations géologiques, il est nécessaire de bien connaître la roche d'accueil et les formations encaissantes. En complément de ses recherches sur les sites de stockages potentiels, la Nagra exploite depuis 1984 le laboratoire souterrain du Grimsel (LSG), qui permet de poursuivre des expériences indépendamment du site qui sera sélectionné ultérieurement. Aménagé à 450 mètres au-dessous du flanc du Juchlistock, dans les roches granitiques du massif de l'Aar, à une altitude de 1730 m, il est accessible par une galerie horizontale.

Le laboratoire souterrain du Grimsel poursuit les objectifs suivants:

- Elaboration d'un savoir-faire en matière de planification, de réalisation et d'interprétation d'essais souterrains réalisés dans différents domaines scientifiques et techniques.
- Acquisition d'une expérience pratique dans le développement et l'utilisation de méthodes d'investigation, de procédures et d'instruments de mesure qui permettront d'étudier des sites potentiels.
- Analyse expérimentale de processus déterminants pour la sûreté d'un dépôt final pour déchets radioactifs.


Au cours des années suivantes, d'autres accords de coopération ont pu être conclus avec les organisations partenaires suivantes: ANDRA (France), ENRESA (Espagne), UE (Union Européenne), PNC (Japon), SKB (Suède) et US-DOE (Etats-Unis).

Le projet "Zone de décompression" (Excavation Disturbed Zone EDZ) réalisé au cours de la phase IV des recherches au LSG représente une partie du programme du "Champ proche". Ce projet (EDZ) est une étude interdisciplinaire ayant pour buts l’investigation de la zone de décompression dans le champ proche de galeries sous des conditions de saturation totale, et par la même occasion le contribution d’une méthodologie de recherche adéquate pour les mesures et la modélisation de circulations d’eau dans la direction axiale le long de galeries.

7 aujourd’hui: Gesellschaft für Anlagen- und Reaktorsicherheit (GRS)
8 aujourd’hui: Bundesministerium für Wirtschaft (BMWi)
9 aujourd’hui: Japan Atomic Energy Agency, Japan (JAEA)
Location of Nagra’s underground test facility at the Grimsel Pass in the Central Alps (Bernese Alps) of Switzerland
Grimsel area (view to the west)

1 Grimsel Test Site  2 Lake Raeterichsboden  3 Lake Grimsel  4 Juchlistock

Grimsel Test Site (GTS)
Grimsel Test Site
GTS

- KWO-Access tunnel
- Laboratory tunnel
- Central Aaregranite (CAGR)
- High biotite content CAGR
- Grimsel-Granodiorite
- Shear zone
- Lamprophyre
- Investigation borehole
- Central facilities
- Fracture system flow
- Rock stresses
- Migration
- Ventilation test
- Heater test

GTS Phase IV 1994–1996

- BOS: Borehole Sealing
- TOM: Further Development of Seismic Tomography
- EDZ: Excavation Disturbed Zone
- TPF: Two-Phase Flow
- RRP: Radionuclide Retardation Project
- ZPK: Two-Phase Flow in Fracture Networks of the Tunnel Near-field
- ZPM: Two-Phase Flow in the Unsaturated Matrix of Crystalline Rocks
- FEBEX: 1:1 EBS – Demonstration (HLW)
SUMMARY

The "Excavation Disturbed Zone Experiment (EDZ)" was conducted at the Grimsel Test Site (GTS) within the framework of the near-field programme in investigation Phase IV (1994 – 1996). It concentrated on investigating the hydraulic regime of the near-field of drilled tunnel sections under fully saturated conditions, with the aim of contributing to the development of methods for measuring and modelling axial water flow along tunnels and caverns. The studies focused on the mechanical and hydraulic properties of the rock mass in the direct vicinity of the tunnel wall. The so-called excavation disturbed zone (EDZ) is defined as the zone around the tunnel where excavation has altered the rock properties.

This report provides an overview of the results obtained during the EDZ experiment. The selected test location was a tunnel section in the heater test drift (WT) where mechanical stressing of the rock and some breakouts had been observed. Detailed geological mapping confirmed the suitability of the chosen test site and also provided background information for locating short test boreholes. During site preparation, in-situ stress measurements using a borehole slotter probe were performed in order to record the actual stress redistribution in the tunnel near-field induced by excavation of the tunnel for rock mechanical design calculations. A small stress increase and microfissures could be identified in the tunnel near-field, which suggested the potential existence of a plastic zone.

The stress measurements and the results of the geological mapping formed the basis for the rock mechanical modelling of the EDZ. The aim of the modelling was to obtain information on the development and geometry of the EDZ (understanding of the primary and secondary stress field). For this purpose, two different models were used:

- The regional 3D stress field modelling indicated that the topography has a significant influence on the primary stress field and not the fault zone systems that were included in the model. A good agreement between the measured and calculated stresses in the GTS was achieved by applying an additional far-field tectonic stress component.

- With the local 2D numerical disturbed zone modelling of the tunnel section itself, stress redistributions, possible plastifications and joint behaviour (closure, opening and shear displacements) in the near-field of the tunnel were investigated. The initial and boundary conditions were derived from the 3D model. All displacements of the rock matrix and the shear displacements of the discontinuities seem to be the result of the tunnel excavation. The displacement field and the geomechanical behaviour are strongly influenced by the discontinuities. Also, temporary plastifications and the subsequent stress redistributions are strongly linked to these discontinuities (material returns to the elastic range). In the different model cases, maximum shear deformations of 2 – 5 mm occur at the tunnel wall. The largest convergence (inward movement) of up to 10 mm occurs on the eastern tunnel wall.

Prior to the hydraulic test phase, the test location was decoupled from the normal GTS tunnel ventilation using partitioning walls, which allow this tunnel section to be isolated. In this way, complete saturation of the rock was achieved and single-phase conditions established. The status of rock saturation was checked by evaporation measurements. Afterwards, a surface sealing with epoxy resin was implemented to establish defined boundary conditions for the hydraulic testing and to avoid short-circuits in the test area with direct outflows into the tunnel during the injection tests. The EDZ was then investigated by 4 cored short radial boreholes (EDZ95.001 to EDZ95.004) arrayed perpendicularly to the tunnel axis. To optimise the hydraulic tests in these boreholes in terms of configuration and performance, hydraulic design calculations were carried out using two different modelling approaches (Equivalent Porous Medium [EPM] and Discrete Fracture Network [DFN]).
For the hydraulic testing, a Modular Minipacker System (MMPS) was developed which allows a wide range of test configurations to be realised in a small borehole with a diameter of 50 mm.

The hydraulic testing campaign included a series of Pulse Tests, Constant Head Tests, Constant Rate Tests and Pressure Recovery Tests in different borehole intervals to provide an overview of the distribution of the hydraulic characteristics/properties. Of the 18 tested intervals, 14 had hydraulic conductivities between $3 \times 10^{-12}$ and $3 \times 10^{-11}$ m/s. Based on the results of these screening tests, the remaining four intervals with relatively higher hydraulic conductivities ($3 \times 10^{-7}$ m/s to $4 \times 10^{-10}$ m/s) were selected for more detailed characterisation. The higher hydraulic conductivity of these 4 intervals appears to be related to a feature independent of EDZ origin (i.e. a pre-existing fracture). Overall, the hydraulic test data show a zone with roughly constant conductivity of $2 \times 10^{-12} - 3 \times 10^{-12}$ m/s beyond 2 m from the tunnel wall, and a zone with conductivities $\geq 8 \times 10^{-12}$ m/s (which is larger than the expected matrix conductivity in all zones) within 1 m of the tunnel wall. This suggests the presence of an EDZ around the tunnel. From rock mechanical modelling, the shape of the EDZ would probably be elliptical, but this could not be confirmed by the results of the hydraulic testing due to the small number of drilled and tested boreholes.

During the single-hole hydraulic tests, acoustic emissions were registered in two separate boreholes (EDZ95.005 and EDZ95.006) to monitor the test site especially during hydrotesting. The measurements indicated that no new fractures were created by the hydrotests performed at the test location.

Following active testing, the monitoring system was left in place to monitor the recovery and development of hydraulic head distribution over a longer period of time.

A methodology for estimating axial flow in the near-field of the tunnel after closure and resaturation of the excavation had been developed. Based on the discrete fracture network models (DFN) used for the design calculations and the characterisation of the EDZ, a (revised) conceptual model of the EDZ was elaborated. The effects of changes in stress and pore pressure occurring between the characterisation of the damaged zone and closure and resaturation of the excavation were considered. The results of modelling post-closure flow through the EDZ suggested bounding values for (post-closure) effective axial conductivity from $3 \times 10^{-11}$ to about $6 \times 10^{-8}$ m/s. The higher value corresponds to the situation where highly transmissive damaged zone features are extensive and well connected and the lower value to the situation where small-scale fracturing is dominant in the damaged zone. However, the effective axial conductivity of the rock around the tunnel at the GTS site will be controlled by the extent and connectivity of the high transmissivity features within the damaged zone.

In general, the experimental aims have been met and the equipment and methodology developed are suitable for determining the hydraulic properties of the EDZ.
ZUSAMMENFASSUNG


Im Rahmen der Standortvorbereitungen wurden In-situ-Spannungsmessungen mit einer sogenannten Bohrloch-Schlitzsonde durchgeführt, um die durch den Stollenausbruch verursachten Spannungsumlagerungen im Stollennahbereich zu ermitteln. Diese Daten gingen in felsmechanische Designberechnungen zur Versuchsauslegung ein. Im Stollennahbereich konnten eine geringfügige Spannungserhöhung und Mikrorisse festgestellt werden, die auf eine plastische Zone schliessen lassen.

Die Spannungsmessungen und die Ergebnisse der geologischen Kartierung bildeten die Grundlage für die felsmechanische Modellierung der EDZ. Das Ziel dieser Modellierung war es, Informationen über die Ausbildung und Geometrie der EDZ zu erhalten (Verständnis des primären und sekundären Spannungsfelds). Hierfür wurden zwei verschiedene Modelle verwendet:

- Die regionale 3D-Spannungsfeldmodellierung ergab, dass lediglich die Topographie einen bedeutenden Einfluss auf das primäre Spannungsfeld ausübt, und nicht die vorhandenen Diskontinuitäten bzw. Störungssysteme, die in das Blockmodell integriert wurden. Durch Anwendung einer zusätzlichen tektonischen Spannungskomponente im Fernfeld des Stollens wurde eine gute Übereinstimmung zwischen den gemessenen und den berechneten Spannungen im FLG erzielt.


Für die Durchführung der hydraulischen Tests wurde ein modulares Minipackersystem (MMPS) entwickelt, mit dem es möglich ist, die unterschiedlichsten Testkonfigurationen in einem Bohrloch mit einem kleinen Durchmesser von nur 50 mm zu realisieren.

Die hydraulischen Tests umfassten ein Reihe von Pulse Tests, Constant Head Tests, Constant Rate Tests und Pressure Recovery Tests in verschiedenen Bohrlochintervallen, um einen Überblick über die Verteilung der hydraulischen Eigenschaften im Stollennahbereich zu ermöglichen. Von den 18 getesteten Intervallen ergaben 14 hydraulische Durchlässigkeiten zwischen $3 \times 10^{-12}$ und $3 \times 10^{-11}$ m/s. Basierend auf diesen Testergebnissen wurden vier Intervalle mit etwas höherer Durchlässigkeit ($3 \times 10^{-7}$ m/s bis $4 \times 10^{-10}$ m/s) zur genaueren Charakterisierung ausgewählt. Die erhöhte hydraulische Durchlässigkeit dieser vier Intervalle scheint auf eine Struktur unabhängig von der EDZ zurückzuführen zu sein (z.B. bestehende Kluftzone). Insgesamt zeigen die hydraulischen Testdaten im Abstand von mehr als 2 m von der Stollenwand eine Zone mit relativ konstanter Durchlässigkeit von $2 \times 10^{-12}$ bis $3 \times 10^{-12}$ m/s und innerhalb 1 m von der Stollenwand eine Zone mit einer Durchlässigkeit von $\geq 8 \times 10^{-12}$ m/s (höher als die erwartete Matrixdurchlässigkeit aller Zonen). Dies weist auf die Gegenwart einer Auflockerungszone um den Stollen hin. Aufgrund der felsmechanischen Modellierung ist die Form der EDZ wahrscheinlich elliptisch, dies konnte allerdings mangels ausreichender Anzahl getesteter Bohrungen nicht durch die Resultate der hydraulischen Tests bestätigt werden.

Während der Hydrotests (Single Hole) wurden akustische Emissionen in zwei separaten Bohrungen (EDZ95.005 und EDZ95.006) aufgezeichnet, um den Versuchsstandort insbesondere während der hydraulischen Tests zu überwachen. Diese Messungen ergaben, dass keine neuen Klüfte durch die am Versuchsstandort durchgeführten hydraulischen Tests erzeugt wurden.

Im Anschluss an die active Testphase blieb das Testsystem eingebaut, um die Erholung und Entwicklung der Druckhöhenverteilung über eine längere Zeitdauer kontrollieren zu können.

Im vorliegenden Bericht wurde eine Methode zur Abschätzung des radialen Flusses im Nahbereich von versiegelten Stollen und Kavernen nach deren Wiederaufsättigung entwickelt. Basierend auf den Kluftnetzwerkmodellen, die für die Designberechnungen und die Charakterisierung der EDZ benutzt wurden, konnte ein (revidiertes) konzeptuelles Modell der EDZ erstellt werden. Die Effekte von Spannungs- und Porendruckänderungen, die nach der Charakterisierung der sogenannten Bruchzone 'damaged zone' bei der Versiegelung und Wiederaufsättigung der Kaverne auftraten, wurden dabei ebenfalls berücksichtigt. Die Ergebnisse der Modellierung des Flusses durch die EDZ nach Verschluss der Kaverne lassen auf Grenzwerte für die effektive axiale Durchlässigkeit (nach Verschluss) zwischen $3 \times 10^{-11}$ und ca. $6 \times 10^{-8}$ m/s schliessen.
Dabei repräsentiert der höhere Wert eine Situation mit hoch durchlässigen Elementen in der Bruchzone, die ausgeprägt und gut untereinander verbunden sind und der niedrigere Wert eine Situation mit dominierender kleinmassstäblicher Klüftung in der Bruchzone. Allerdings wird die effektive axiale Durchlässigkeit des Gebirges im Stollennahbereich des FLG durch das Ausmaß und den Verbindungsgrad der hoch transmissiven Elemente innerhalb der Bruchzone kontrolliert.

Generell konnten die experimentellen Ziele erreicht werden. Die entwickelte Methode und das Equipment sind geeignet, um die hydraulischen Eigenschaften der EDZ zu bestimmen.
RÉSUMÉ

Le projet "Zone de décompression" (Excavation Disturbed Zone EDZ) a été réalisé au Laboratoire souterrain du Grimsel (LSG) dans le cadre du programme du "Champ proche" au cours de la phase IV des recherches (1994 – 1996). Ce projet a été consacré à l'étude du régime hydraulique dans le champ proche de tronçons de galeries sous des conditions de saturation complète, dans le but de contribuer au développement de méthodes de mesure et de modélisation des écoulements axiaux le long de tunnels ou de cavernes. Les recherches se sont concentrées sur les propriétés mécaniques et hydrauliques de la masse rocheuse à proximité immédiate de la paroi de la galerie. La zone de décompression (EDZ) est définie comme la zone autour de la galerie où l’excavation a altéré les propriétés de la roche.

Le présent rapport donne une vue d’ensemble des résultats obtenus durant le projet EDZ. Le site d’expérimentation sélectionné était une portion de la galerie utilisée pour le projet des "Tests thermiques" (WT), où l’on avait observé une altération mécanique de la roche et quelques éclats d’excavation sur les parois. Un lever géologique détaillé a confirmé l’adéquation de ce site et a fourni l’information nécessaire à la localisation de forages courts de test. Au cours de la préparation du site, des mesures de contraintes in-situ ont été effectuées au moyen d’une sonde à fentes, afin d’évaluer la redistribution réelle des contraintes après l’excavation, dans le champ proche de la galerie, pour la planification des calculs de mécanique des roches. On a pu identifier une légère augmentation des contraintes et l’apparition de microfissures dans le champ proche de la galerie, qui suggèrent l’existence possible d’une zone plastique.

Les mesures des contraintes et les résultats du lever géologique constituent les éléments de base pour le modèle de mécanique des roches de la zone de décompression (EDZ). La modélisation avait pour objectif d’obtenir des informations sur le développement et la géométrie de la zone de décompression (compréhension des champs de contraintes primaire et secondaire). A cette fin, deux modèles ont été utilisés:

- La modélisation 3D du champ régional des contraintes a montré que la topographie exerçait une influence significative sur le champ primaire des contraintes, mais pas les systèmes de failles inclus dans le modèle. Une bonne correspondance entre les contraintes mesurées et calculées au LSG a été obtenue après l’introduction d’une composante additionnelle de contrainte tectonique du champ éloigné.

- La modélisation 2D du champ local des contraintes dans la zone de décompression de la galerie a permis l’étude de la redistribution des contraintes, de plastifications possibles et du comportement des fissures (fermeture, ouverture et déplacements de cisaillement). Les conditions initiales et aux limites ont été tirées du modèle 3D. Les déplacements de la matrice rocheuse et les mouvements de cisaillement des discontinuités semblent tous être dus à l’excavation de la galerie uniquement. Le champ des déplacements et le comportement géomécanique sont fortement influencés par les discontinuités. En outre, les plastifications temporaires et la redistribution des contraintes qui s’ensuivent sont fortement liées aux discontinuités (les matériaux retournent à l’état élastique). Dans les différents cas examinés par le modèle, les déformations maximales dues au cisaillement atteignent 2 à 5 mm dans le champ proche de la galerie. La plus grande convergence (mouvement vers l’intérieur) atteint 10 mm et se manifeste sur la paroi orientale de la galerie.

Avant la phase des tests hydrauliques, le site de test a été isolé de la ventilation de la galerie au moyen de parois de séparation pour permettre une complète saturation en eau de la roche et établir ainsi des conditions de phase unique. L’état de saturation a été contrôlé par des mesures d’évaporation. Ensuite, on a colmaté la surface de la roche au moyen d’une résine afin d’établir
pour les tests hydrauliques des conditions aux limites définies et afin d’éviter dans la région testée des court-circuits provoquant des venues d’eau dans la galerie lors des tests d’injection. La zone de décompression a été auscultée au moyen de 4 courts forages radiaux carottés (EDZ95.001 à EDZ95.004), disposés perpendiculairement à l’axe de la galerie. Afin d’optimiser les tests hydrauliques dans ces forages, en termes de configuration et de performance, des calculs hydrauliques de planification ont été réalisés au moyen de deux modèles différents (milieu poreux équivalent EPM et réseau de fractures discrètes DFN).

Pour l’exécution des tests hydrauliques, on a développé un système entièrement modulaire de mini-obturateurs (MMPS). Ce système autorise une large gamme de configurations de test dans un petit forage de 50 mm de diamètre.

La campagne des tests hydrauliques a compris des tests à impulsion (Pulse Tests), à charge constante (Constant Head Tests), à débit constant (Constant Rate Tests) et à restitution de pression (Pressure Recovery Tests) dans différents intervalles de forage. Elle fournit une vue d’ensemble de la distribution des propriétés et caractéristiques hydrauliques de la roche. Sur 18 intervalles testés, 14 ont une conductivité hydraulique située entre $3 \times 10^{-12}$ et $3 \times 10^{-11}$ m/s. Sur la base de ces tests de reconnaissance, 4 intervalles à conductivité relativement élevée ont été sélectionnés pour une caractérisation plus poussée ($3 \times 10^{-7}$ m/s to $4 \times 10^{-10}$ m/s). La conductivité plus élevée de ces intervalles semble être liée à la présence d’une fracture préexistante à la décompression. Dans l’ensemble, les résultats des tests hydrauliques font apparaître une zone à conductivité à peu près constante de $2 \times 10^{-12}$ à $3 \times 10^{-12}$ m/s au-delà des 2 m entourant la paroi de la galerie, et une zone de conductivité supérieure à $8 \times 10^{-12}$ m/s de la paroi de la galerie jusqu’à 1 m de profondeur. Ceci suggère la présence d’une zone de décompression (EDZ) autour de la galerie. D’après le modèle de mécanique des roches utilisé, la forme de cette zone est probablement elliptique, mais ceci n’a pas pu être confirmé par les tests hydrauliques, en raison du trop petit nombre de forages et de tests.

Durant la campagne des tests hydrauliques effectués successivement dans chacun des 4 forages susmentionnés, les émissions acoustiques ont été enregistrées dans deux autres forages (EDZ95.005 et EDZ95.006), pour une surveillance particulière du site durant les essais hydrauliques. Ces mesures indiquent qu’aucune nouvelle fracture n’a été créée sur le site au cours des essais hydrauliques.

A la suite des tests hydrauliques actifs, le dispositif de surveillance a été maintenu en place pour enregistrer la restitution des pressions et la distribution spatiale de la charge hydraulique sur une longue période.

Une méthodologie permettant d’estimer le flux hydraulique axial dans le champ proche de la galerie après sa fermeture et la resaturation de l’excavation a été développée. A partir des modèles de réseaux de fractures discrètes (DFN) élaborés pour les calculs de planification et la caractérisation de la zone de décompression (EDZ), un modèle conceptuel révisé de la zone de décompression a été mis au point. On a tenu compte des effets des changements de contrainte et de pression dans les pores survenant entre la caractérisation de la zone endommagée et la resaturation de l’excavation après sa fermeture. Les résultats de la modélisation de l’écoulement dans la zone de décompression après la fermeture du site suggèrent un domaine de conductivité axiale effective (après fermeture) s’étendant de $3 \times 10^{-11}$ à environ $6 \times 10^{-8}$ m/s. La valeur la plus élevée correspond à une situation où les éléments les plus transmissifs de la zone endommagée sont étendus et bien connectés entre eux, tandis que la valeur inférieure correspond à une situation où domine une fracturation à petite échelle dans la zone endommagée. Dans tous les cas, la conductivité hydraulique axiale effective de la roche autour de la galerie au LSG sera contrôlée par l’extension et la connectivité des éléments les plus transmissifs dans la zone endommagée.
Dans l’ensemble, on considère que les objectifs expérimentaux ont été atteints et que les équipements et méthodologies développés sont adéquats pour la détermination des propriétés hydrauliques de la zone de décompression (EDZ).
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List of Attachments

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disturbed/damaged zone between 75 and 88 m
Attachment 2: Detailed structural map of the excavation disturbed/damaged zone
1 INTRODUCTION AND REPORT OVERVIEW

At a potential repository site, the rock mass in the immediate vicinity of tunnels and caverns, the "near-field", is of particular importance. The near-field represents the transition between the engineered systems of the repository and the geosphere and is characterised by specific hydraulic and rock mechanical properties, which influence water flow and therefore the potential transport of radionuclides out of the repository (Fig. 1.1).

Because of the disturbance of the rock in the tunnel near-field induced by the excavation process, it is possible that a hydraulic connection between water-conducting features can be created which is able to induce axial flow along the tunnel. The hydraulic properties of the Excavation Disturbed Zone (EDZ) in the tunnel near-field therefore directly influence the technical realisation and quality of tunnel sealing.

The "Excavation Disturbed Zone Experiment (EDZ)" which is presented in this report focuses on the determination and modelling of such axial flows, which have to be minimised during the sealing of the tunnel (Fig. 1.2). It is part of the L/ILW (low- and intermediate-level waste) and HLW/ILW (high-level and long-lived intermediate-level waste) programme and contributes to the methodology for the derivation of a near-field geodata set and to the further development of in-situ investigation methods and technologies, which can be implemented in the characterisation of a potential repository site.

Fig. 1.1: Illustration of the near-field with the EDZ for the low- and intermediate-level waste repository (L/ILW) (after FRIEG et al., 1996).
The EDZ belongs to the Phase IV investigation programme at the Grimsel Test Site, which was performed in the period between 1994 and 1996. The Grimsel Test Site (GTS) was established in 1983 as a centrepiece of Nagra's research and development programme. This facility provides access to a relevant deep geological environment for experiments, which are performed to improve the understanding of the processes influencing the long-term evolution of a repository.

Earlier research activities of Phases I to III (1983 – 1993) provided the basis for further projects in Phase IV. The technical programme of Phases I to III focused on the geological and hydrogeological understanding of the GTS and on developing the methodology for performing scientific investigation programmes under realistic field conditions. The experiments included the Heater Test (WT), Rock Stress Measurements (GS), the Fracture System Flow Test (BK), the Ventilation Test (VE) and the Migration Experiment (MI), which were performed as joint exercises with international partner organisations.

For Phase IV of the investigations at the GTS, the emphasis was on selecting projects of a more performance assessment (PA) relevant nature. These projects were: Seismic Tomography (TOM), Borehole Sealing (BOS), Two-Phase Flow (TPF), Radionuclide Retardation Project (RRP) and the Excavation Disturbed Zone (EDZ).

### 1.1 Background and scope

The EDZ is defined as the mechanically altered zone (with potentially altered hydraulic properties) which develops during and after the excavation of tunnels and caverns. The development and geometry of the EDZ depend on both the applied excavation technique and on the stress field and the rock properties. Rock mechanical investigations together with modelling calculations give evidence about the development and spatial extent (i.e. the geometry) of the disturbed zones around tunnels and caverns. What is relevant for the safety analysis, however, is whether there are significant changes in the hydraulic properties of such zones. At the moment there exists no international "standard" investigation procedure for the characterisation of the EDZ.

At the international EDZ workshop held in 1996 (MARTINO & MARTIN, 1996) it was stated that the EDZ should be minimised during repository development and must be prevented from becoming a pathway for radionuclide transport after sealing. To accomplish this, the EDZ must be characterised using methods which do not create a greater problem when the repository is to be sealed. FAIRHURST & DAMJANAC (1996) stated in this context that the terms Excavation...
Disturbed (or Damaged) Zone (EDZ) and Disturbed Rock Zone (DRZ) are used synonymously to describe the region of the rock adjacent to an underground excavation that has been significantly damaged or disturbed due to the redistribution of in-situ rock stresses that occur upon creation of the excavation and any damage to the rock produced by the method of excavation. Various examples of EDZ relevant experiments from underground laboratories e.g. Stripa, AECL, Olkiluoto, Åspö, GTS, Asse were presented as representing the current understanding of the extent and properties of the EDZ (MARTINO & MARTIN, 1996).

Nagra was also a participant in the ZEDEX project (EMSLEY et al., 1997), which was undertaken as a joint project by SKB, ANDRA and UK NIREX with significant contributions from BMBF and Nagra during 1994 to 1996. The objectives of this project performed in several drifts at the Åspö Hard Rock Laboratory were to understand the mechanical behaviour of the excavation disturbed zone with respect to its origin, character, magnitude of property change, extent and its dependence on the applied excavation method. Furthermore, studies to increase the understanding of the hydraulic relevance of the EDZ and to test equipment and methodology were carried out. The results from the ZEDEX project have shown that there is a "damaged zone" close to the drift wall dominated by changes in rock properties which are mainly irreversible, and that there is a "disturbed zone" beyond the damaged zone that is dominated by changes in stress state and hydraulic head and where changes in rock properties are small and mainly reversible (see Fig. 1.3).

Fig. 1.3: Extent of the disturbed zone (EDZ = excavation disturbed zone) around a drill-and-blast tunnel as opposed to a TBM tunnel. Compilation of results from ZEDEX experiments in the Åspö HRL (SKB, 1999).
The changes in rock properties and rock stress with distance from the rock wall of the excavation are gradational, and there is hence no distinct boundary between the two zones. The damaged zone is characterised by excavation induced fracturing whereas the disturbed zone shows elastic displacements and no induced fracturing. The results from ZEDEX indicate that the role of the EDZ at the site as a preferential pathway for radionuclide transport is limited to the damaged zone, which is the hydraulically significant part. The extent of this damaged zone can be limited through application of appropriate excavation methods.

The following definition for the Excavation Disturbed Zone (EdZ) and Excavation Damaged Zone (EDZ) was given by TSANG & BERNIER (2005) as a conclusion from the European Commission cluster conference and workshop related to the impact of the excavation disturbed or damaged zone on the performance of geological repositories for radioactive waste (DAVIES & BERNIER, 1996):

- The *Excavation Disturbed Zone* (EdZ) is a zone with hydromechanical and geochemical modifications, without major changes in flow and transport properties.
- The *Excavation Damaged Zone* (EDZ) is a zone with hydromechanical and geochemical modifications inducing significant changes in flow and transport properties. These changes can, for example, include a one or more orders of magnitude increase in flow permeability.

These broad definitions are more along the lines proposed by the indurated clay working group. For the crystalline rock, the EdZ was described as a region where only reversible (recoverable) elastic deformation has occurred. It was also stated that it is theoretically impossible to define the outer limits of the EdZ. In contrast, the EDZ was defined as region of irreversible deformation with fracture propagation and/or development of new fractures. The EDZ has a strong transient behaviour and depends on construction methods as well as stress redistribution.

Because there are many different issues related to the EDZ that involve several disciplines, the terminology is not always the same for all disciplines. Therefore it is important to clearly define what is meant by the EDZ. The EDZ can also be defined as a zone around openings that may have altered properties relevant to the post-closure performance of the overall repository system due to excavation of these openings (ZUIDEMA, 2005).

BÄCKBLOM (2008) summarised the present knowledge of the excavation damaged and disturbed zones in crystalline rock based on the results and analyses from experiments performed at Stripa (OECD/NEA, Sweden), Grimsel Test Site (GTS, Switzerland), AECL URL (Canada), Åspö HRL (SKB, Sweden), Olkiluoto (Posiva, Finland) and Kamaishi (JAEA, Japan). The following definitions were given (comp. Fig. 1.4):

- **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 a change of state (e.g. stress, hydraulic head). The changes in rock mass properties are insignificant or reversible.
Fig. 1.4: 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 (after BÄCKBLOM, 2008).

In this report the term EDZ is normally used for the excavation disturbed zone. If the excavation damaged zone is meant it is explicitly mentioned.

1.1.1 Objectives of EDZ experiment

The specific objectives of the EDZ experiment at the Grimsel Test Site are described as follows:

- to develop a general site-independent method for identifying a hydraulically relevant excavation disturbed zone in the near-field of tunnels and caverns
- to determine the spatial extent of the EDZ perpendicular to the tunnel axis and its axial variability, using geophysical and rock mechanical techniques
- to determine the radial hydraulic permeability and its axial variability in the tunnel near-field, using standard hydraulic test techniques
- to test and further develop existing hydraulic test equipment and, if needed, to develop new equipment in order to obtain input parameters for the hydraulic model.

The selected test location is a tunnel section within the heater test drift (WT) at the Grimsel Test Site (GTS) where mechanical stressing of the rock and some breakouts of the tunnel have been observed. In this area the heater test experiments were carried out by Nagra's German partner GRS (former GSF) during Phases I and II of the Grimsel Test Site investigation programme (SCHNEEFUSS et al., 1989). Detailed geological mapping of the tunnel confirmed the suitability of the chosen test site and also provided background information for locating the planned short test boreholes. The test programme is described in detail in Chapter 2.
1.2 Complementary investigations in the near-field programme

Since 1984 different methods have been tested at the GTS as part of the near-field programme in cooperation with international institutions. Apart from investigating transport in fractured rock, the evaluation of the influence of the gas phase on the hydraulic properties of the rock has become one of the major issues of the German/Swiss investigations as of 1991. During Phases I to III of the GTS investigations, several projects were conducted in this respect, e.g. the interpretation of both the flow conditions for water and the mobility of the gas phase in the so-called desiccation zone in the near-field of the Ventilation Test Gallery VE (KULL & MIEHE, 1995) or the Fracture System Flow Test BK (PAHL et al., 1992), see Figure 1.5 for location.


<table>
<thead>
<tr>
<th>Test</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>BOS</td>
<td>Borehole sealing</td>
</tr>
<tr>
<td>EDZ</td>
<td>Excavation disturbed zone</td>
</tr>
<tr>
<td>EP</td>
<td>Excavation of the MI shear zone</td>
</tr>
<tr>
<td>FEBEX</td>
<td>Full-scale engineered barriers experiment</td>
</tr>
<tr>
<td>TOM</td>
<td>Further development of seismic tomography</td>
</tr>
<tr>
<td>TPF</td>
<td>Two-phase flow</td>
</tr>
<tr>
<td>CP</td>
<td>Connected porosities</td>
</tr>
<tr>
<td>ZPK</td>
<td>Two-phase flow in fracture networks</td>
</tr>
<tr>
<td>ZPM</td>
<td>Two-phase flow in the rock matrix</td>
</tr>
</tbody>
</table>

Test areas

<table>
<thead>
<tr>
<th>Test</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>BK</td>
<td>Fracture system flow test</td>
</tr>
<tr>
<td>MI</td>
<td>Migration experiment</td>
</tr>
<tr>
<td>US</td>
<td>Underground seismics</td>
</tr>
<tr>
<td>VE</td>
<td>Ventilation test</td>
</tr>
<tr>
<td>WT</td>
<td>Heater test</td>
</tr>
<tr>
<td>KWO</td>
<td>Kraftwerke Oberhasli AG</td>
</tr>
<tr>
<td>ZB</td>
<td>Central facilities</td>
</tr>
</tbody>
</table>

Fig. 1.5: Sketch of the tunnel system of the Grimsel Test Site with test areas and tests of Phase IV.
The Excavation Disturbed Zone Experiment forms part of a complex of investigations of the tunnel near-field (near-field programme) within Phase IV experiments of the Grimsel Test Site, which also includes the following projects (Figure 1.1):

- two-phase flow in fracture and shear zones (TPF) (carried out by Nagra)
- two-phase flow in fracture networks (ZPK) (carried out by BGR\(^1\))
- two-phase flow in the rock matrix (ZPM) (carried out by GRS\(^2\))

These projects had the following objectives:

- development of techniques for rock characterisation
- investigation of two-phase flow properties in the near-field of tunnels and caverns
- modelling two-phase flow parameters of fracture and matrix zones.

Together with the EDZ project, this series of studies of the rock mass in the vicinity of the tunnel wall covers aspects of the characterisation of the excavation disturbed zone and hydrological changes during repository operation and after closure of the repository.

The EDZ experiment was planned to be performed under completely saturated conditions. Therefore the EDZ project concerns only the single-phase condition. Investigations in the tunnel near-field under two-phase flow conditions, which can occur because of the repository gas release, are performed in the GTS experiments "TPF" and "ZPM".

1.3 Report organisation

The form of the report as an edited volume was chosen to give credit to all persons involved in the particular subtasks.

- Chapter 1 (FRIEG, B.): Gives an introduction of the project with aims, background and scope as well as a description of complementary investigations in the near-field programme.
- Chapter 2 (FRIEG, B.): Provides more details on the investigation programme which was conducted in 1994 – 1996 and describes the procedure and milestones.
- Chapter 3 (DOLLINGER, H. & FRIEG, B.): Describes the test site selection and preparation and gives a description of the local geology including the input parameters for hydraulic modelling.
- Chapter 4 (based on HARTKORN & WOHNLICH (1995) and KONIETZKY (1995)): Describes the in-situ stress measurements as well as the rock mechanical modelling.
- Chapter 5 (KUHLMANN, U. & LANYON, G.W.): Documents the hydraulic design calculations based on the equivalent porous medium approach and the fracture network approach.
- Chapter 6 (ADAMS, J. & FRIEG, B.): Provides more details about the hydraulic testing campaigns including a description of the test equipment, the performed hydrotests as well as the analysis of the test data.

\(^1\) BGR (Bundesanstalt für Geowissenschaften und Rohstoffe, Hannover)
\(^2\) GRS (Gesellschaft für Anlagen- und Reaktorsicherheit mbH, Köln)
• Chapter 7 (ALBERT, W): Documents the registered acoustic emissions during hydraulic testing campaigns.
• Chapter 8 (LANYON, G.W.): Describes in detail the hydraulic fracture network modelling which was carried out for the EDZ based on the hydraulic test results and the geological input.
• Chapter 9 (LANYON, G.W.): All results are discussed and a conceptual model of the tunnel near-field is developed.
• Chapter 10 (FRIEG, B. & LANYON, G.W.): Provides the conclusions and recommendations for further projects within the EDZ experiments.

The work of the EDZ programme was carried out by the following persons and/or companies:

Geological mapping: H. Dollinger (Geotechnisches Institut, Solothurn, Switzerland)
Evaporation measurements: E. Meier (E. Meier + Partner AG, Winterthur, Switzerland)
Tunnel sealing: Sika AG (Bern, Switzerland)
In-situ stress measurements: P. Hartkorn & M. Wohnlich (Interfels, Rottenburg, Germany)
Rock mechanical modelling: H. Konietzky (ITASCA Consultants, Bochum, Germany)
Hydraulic design calculations: EPM: U. Kuhlmann (TK-Consult, Zürich, Switzerland)
DFN: G.W. Lanyon (GeoScience, United Kingdom)
Hydrotesting: J. Adams (Solexperts, Schwerzenbach, Switzerland)
Acoustic emissions: C. Cosma (Vibrometric, Oy, Finland), W. Albert & S. Schwere (Nagra)
Hydraulic fracture network modelling: G.W. Lanyon (GeoScience, United Kingdom)
2 EXPERIMENT PLAN AND INVESTIGATION PROGRAMME

In this chapter the experiment plan of the EDZ project is described. The experiment was divided into two main phases with several milestones. These milestones were used to verify the specific objectives of the EDZ experiment and could be adapted, if needed, to possible changes of the investigation programme based on any new information.

2.1 Phase 1

Phase 1 represents the site preparation for the EDZ experiment and rock mechanical modelling with the following specific objectives (comp. Tab. 2.1):

- extension of the database for rock mechanical modelling
- site investigation
- site preparation
- optimisation of the test configuration
- test design for the hydraulic tests in Phase 2.

Within the pre-evaluation framework the heater test drift (WT tunnel section in Figure 1.1) was selected as the test site for the EDZ experiment because mechanical stressing of the rock and some breakouts had been observed there. The test site ends within the lamprophyres, whose possible influence had to be evaluated. The breakouts occur in the homogeneous matrix and are not related to shear zones or other tectonic faults.

Detailed geological mapping of the tunnel section was planned to confirm the suitability of the test site (indication of excavation disturbance and anisotropy) and to provide structural geological data.

Furthermore, in-situ stress measurements were planned to measure the in-situ stress redistribution induced by excavation of the tunnel.

Refraction seismic measurements were originally planned to estimate the geometry of the EDZ and its axial variability as well as to detect tectonic faults (not performed due to budget constraints).

The results derived from these pre-investigations formed the basis for the rock mechanical modelling. This modelling would contribute to the determination of the extent and geometry of the EDZ, which is significant for locating the radial boreholes in Phase 2. Within the rock mechanical modelling, the influence of topography, tunnel geometry, pre-existing fractures, tectonic faults and anisotropies (matrix rock and strength) were to be estimated and a hypothetical transmissivity structure in the tunnel near-field derived.

At the end of Phase 1 and before the actual test phase, it was planned to decouple the test location from the tunnel ventilation using partitioning walls to allow complete saturation of the rock. The saturation would be controlled by evaporation measurements. Afterwards it was planned to implement a surface sealing with resin. Thus, defined boundary conditions would be created for the hydraulic tests carried out in Phase 2.
The end of Phase 1 was defined by a milestone at which the heater test tunnel had been prepared as the site for the EDZ investigations and the geometry of the EDZ was known from rock mechanical modelling and geophysical investigations.

**Tab. 2.1: Overview of Phase 1 with objectives and planned activities with milestone.**

<table>
<thead>
<tr>
<th>Phases</th>
<th>Objectives</th>
</tr>
</thead>
<tbody>
<tr>
<td>I. Site preparation and rock mechanical modelling</td>
<td>1. Extending the database for rock mechanical modelling</td>
</tr>
<tr>
<td></td>
<td>2. Site reconnaissance</td>
</tr>
<tr>
<td></td>
<td>3. Site preparation</td>
</tr>
<tr>
<td></td>
<td>4. Optimising test configuration and test design for the subsequent hydraulic tests (Phases II a+b)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>No.</th>
<th>Activity</th>
<th>Purpose</th>
<th>Object</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Geological tunnel mapping</td>
<td>Classification of fractures (structural geology), fracture statistics, indications of excavation disturbance and anisotropy</td>
<td>1. / 2.</td>
</tr>
<tr>
<td>2</td>
<td>In situ stress measurements</td>
<td>Undisturbed rock stress state, excavation-induced stress changes</td>
<td>1.</td>
</tr>
<tr>
<td>3</td>
<td>Refraction seismics</td>
<td>Thickness of EDZ, variability along tunnel, axial correlation lengths, detection of structural (geological) disturbances</td>
<td>1. / 2. / 4.</td>
</tr>
<tr>
<td>5</td>
<td>Infrastructure at site</td>
<td>Realisation of experimental boundary conditions</td>
<td>3.</td>
</tr>
<tr>
<td>6</td>
<td>Saturation of site</td>
<td>Realisation of experimental boundary conditions</td>
<td>3.</td>
</tr>
<tr>
<td>7</td>
<td>Surface sealing</td>
<td>Realisation of experimental boundary conditions</td>
<td>3.</td>
</tr>
</tbody>
</table>

**Milestone 1:** Heater test tunnel prepared as site for the investigations. Geometry of EDZ determined on the basis of rock mechanical modelling and geophysical investigations.
2.2 Phase 2

Phase 2 represents the evaluation of the hydraulic relevance and hydraulic measurements for the characterisation of the EDZ and is subdivided into Phase 2a and Phase 2b. Within the investigations of Phase 2a, the hydraulic relevance of the EDZ at the GTS was to be estimated. Phase 2b includes the development of an appropriate measuring methodology for the hydraulic characterisation of the EDZ. Thus, during Phase 2 mainly hydraulic investigations were carried out.

2.2.1 Phase 2a

In Phase 2a the EDZ was investigated by 4 cored radial short boreholes (EDZ95.001 to EDZ95.004) arranged perpendicularly to the tunnel axis. The structural geological description of the cores would provide additional data for integration with the geological mapping results along the tunnel. For different core sections rock mechanical laboratory investigations were originally planned which would then have been used to calibrate the final rock mechanical models (not performed due to budget constraints).

Then hydraulic design calculations were planned using two different modelling approaches in order to optimise the hydraulic tests in the radial boreholes in terms of configuration and performance. The array and the number of boreholes would depend on the results obtained from Phase 1. For the hydraulic testing, a new completely Modular Minipacker System (MMPS) was developed, which allows a wide range of test configurations to be realised in a small borehole (diameter: 50 mm). By means of these hydraulic tests it should be possible to determine whether there is a hydraulically relevant EDZ in the GTS.

During performance of the single-hole hydraulic tests, acoustic emissions were planned to be registered in two separate boreholes (EDZ95.005 and EDZ95.006) to estimate whether data models with a hydraulic/rock mechanical coupling have to be taken into account for the interpretation of the obtained test results, and also to eliminate any artefacts in the interpretation.

An additional milestone was set at the end of Phase 2a, at which the radial distribution of the transmissivity with the distance from the tunnel surface was to be determined.
Tab. 2.2: Overview of Phase 2a with objectives and planned activities with milestone.

<table>
<thead>
<tr>
<th>Phases</th>
<th>Objectives</th>
</tr>
</thead>
<tbody>
<tr>
<td>II. a Determining the hydraulic relevance of the EDZ</td>
<td>1. Extending the database for rock mechanical modelling</td>
</tr>
<tr>
<td></td>
<td>2. Site reconnaissance</td>
</tr>
<tr>
<td></td>
<td>3. Site preparation</td>
</tr>
<tr>
<td></td>
<td>5. Preparing equipment for hydrotests</td>
</tr>
<tr>
<td></td>
<td>6. Optimisation of test configuration and test design</td>
</tr>
<tr>
<td></td>
<td>7. Radial transmissivity and head profiles</td>
</tr>
<tr>
<td></td>
<td>8. Identifying coupled hydraulic / rock mechanical processes</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>No.</th>
<th>Activity</th>
<th>Purpose</th>
<th>Object</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>Evaluation and, if necessary, development of minipacker and injection systems</td>
<td>Preparing equipment for hydrotests, build-up of know-how for Wellenberg (WLB)</td>
<td>5.</td>
</tr>
<tr>
<td>9</td>
<td>Drilling radial boreholes</td>
<td>Recovery of drillcores, locations for hydraulic and seismic borehole measurements</td>
<td>1./2./3.</td>
</tr>
<tr>
<td>10</td>
<td>Completion of boreholes</td>
<td>Preventing desaturation, head measurements, registration of acoustic emissions</td>
<td>3./4./7.</td>
</tr>
<tr>
<td>11</td>
<td>Core logging</td>
<td>Structural (geology) classification of fractures, fracture statistics, fracture genesis</td>
<td>1./2.</td>
</tr>
<tr>
<td>12</td>
<td>Rock mechanical laboratory experiments</td>
<td>Rock mechanical parameters</td>
<td>1.</td>
</tr>
<tr>
<td>13</td>
<td>Hydraulic design calculations</td>
<td>Configuration and dimensioning of boreholes, optimisation of test duration and test sequences, influence of test geometry and tunnel drainage</td>
<td>6.</td>
</tr>
<tr>
<td>14</td>
<td>Hydraulic single-hole tests</td>
<td>Hydraulic bounding of EDZ</td>
<td>7.</td>
</tr>
<tr>
<td>15</td>
<td>Measurement of acoustic emissions</td>
<td>Determining transient rock mechanical processes during hydrotests, locating fractures</td>
<td>1./8.</td>
</tr>
<tr>
<td>16</td>
<td>Evaluation / interpretation of hydrotests</td>
<td>Determining transmissivities</td>
<td>7.</td>
</tr>
</tbody>
</table>

**Milestone 2:** Radial transmissivity distribution determined as a function of distance from the tunnel surface.
2.2.2 Phase 2b

If the EDZ in the GTS is hydraulically relevant, the investigations were then to be intensified (e.g. crosshole tests) during Phase 2b in order to obtain radial transmissivity profiles and to determine the axial variability of the transmissivity. These investigations form the base for the development of a simple conceptual model and a methodology for the hydraulic characterisation of an EDZ by means of discrete hydraulic measurements. By the end of Phase 2b a measuring methodology for the hydraulic characterisation of the EDZ was to be developed.

The end of Phase 2b was marked by a milestone, which represents the successful development of a measuring method for the hydraulic characterisation of the EDZ by means of discrete hydraulic measurements. Also the radial and axial transmissivity distribution within the EDZ was to have been determined.

Tab. 2.3: Overview of Phase 2b with objectives and planned activities with milestone.

<table>
<thead>
<tr>
<th>Phases</th>
<th>Objectives</th>
</tr>
</thead>
<tbody>
<tr>
<td>II. b</td>
<td>Continuation of hydraulic measurements for hydraulic characterisation of the EDZ</td>
</tr>
<tr>
<td></td>
<td>Radial transmissivity and head profiles and their axial variability</td>
</tr>
<tr>
<td></td>
<td>Measurement technique for the hydraulic characterisation of the EDZ</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>No.</th>
<th>Activity</th>
<th>Purpose</th>
<th>Object</th>
</tr>
</thead>
<tbody>
<tr>
<td>17</td>
<td>Drilling additional radial boreholes and programme according to Phase 2a (activities 9 – 16)</td>
<td>Increasing the density of the measurement grid for radial transmissitity profiles</td>
<td>9. / 10.</td>
</tr>
<tr>
<td>18</td>
<td>Hydraulic crosshole tests</td>
<td>Determining effective transmissivities</td>
<td>9. / 10.</td>
</tr>
</tbody>
</table>

**Milestone 3**: Development of a technique for hydraulic characterisation of the EDZ using discrete hydraulic measurements completed. Radial and axial transmissivity distribution within the EDZ determined.
3 EVALUATION, DESCRIPTION AND PREPARATION OF THE TEST SITE

3.1 Site selection

During the construction of underground facilities, the primary stress field is altered and a secondary stress field develops around the openings. Depending on the mechanical host rock properties and on the applied excavation technique, this stress redistribution may lead to the development of an Excavation Disturbed Zone around the tunnel. If the rock strength has been exceeded and non-elastic deformations occur, a so-called plastic zone is developed. Such a zone can be detected by visual inspection of the tunnel surface.

Such zones are very rarely encountered in the GTS. Most deformations seem to have occurred in the elastic field. Exceptions are biotite-rich mylonitic shear zones and lamprophyre dykes and also a small number of sites in the granitic matrix.

The site of the EDZ experiment was selected on the basis of the following four criteria:

- **Plastic deformations related to tunnel excavations**: a zone where plastic deformations have occurred. These deformations must be linked either to the redistribution of the stress field or to the applied excavation technique.
- **Excavation technique**: a section excavated with a tunnel boring machine (TBM) should be chosen according to the aims of the EDZ experiment (90% of the GTS was excavated with a TBM). Also a potential repository will probably be excavated by this technique.
- **Geometry of the zone**: the zone should be macroscopically visible with an axial extent of at least 10 m. The zone should affect both tunnel walls (or roof and floor), according to the stress field.
- **Geology of the zone**: the zone should not be limited only to mylonitic shear zones, but should also affect the granitic matrix.

A suitable site where all four requirements defined above are satisfied was found in the "heater test drift" between tunnel metres 75 and 88. The zone has an extent of almost 15 m, lies in the Central Aare Granite and reveals plastic deformations in the form of clearly visible breakouts. These breakouts are found on both tunnel walls and, additionally, brittle fracturing was observed in these zones. This EDZ must have developed just after excavation, in view of the fact that rock bolts were emplaced and iron meshes were fixed immediately after removal of the TBM to stabilise and secure this part of the heater test tunnel. A detailed geological map of the "heater test drift" between tunnel metres 60 and 89 can be found in Attachment 1.

3.2 Geology

3.2.1 Geological mapping of the tunnel section

The test drift of the heater test tunnel section containing the EDZ was mapped in detail (Attachment 1). Besides the detailed mapping of the lithologies, three types of structures were mapped:

- ductile structures
- brittle structures and
- structures closely related to the EDZ.

These lithologies and structures are briefly explained in the following.
Lithologies
The tunnel wall is dominated by the Central Aare Granite (CAGr), one of the main rock types at the GTS. Locally dark and light variations, which are linked to the biotite content, are found. In the Central Aare Granite, many xenoliths were observed and mapped. These xenoliths were deformed during the ductile deformation phase and can be used to trace the cleavage along the tunnel surface (longitudinal axis parallel to the cleavage plane). Aplite dykes striking parallel to the cleavage were found at tunnel metres 65 and 73. They contain uranium oxide precipitations. One lamprophyre dyke almost one metre thick has been mapped at tunnel metre 87. This dyke bounds the EDZ to the west. The lamprophyre dyke behaves somewhat differently from the matrix. In particular, the boundary of the dyke has been reactivated and the centre of the dyke shows extension veins.

Ductile structures
The cleavage is defined by the parallel alignment of sheet silicates, mainly biotite and, to a lesser extent, also feldspars and quartz. The longitudinal axis of the xenoliths was used to trace the cleavage on the tunnel wall. In the whole mapped section, no ductile mylonitic shear zones were observed. Other structures, which may be linked to the ductile structures, are sealed (quartz) veins at tunnel metre 80, which exhibit an en-échelon pattern.

Brittle structures
Due to the fact that no mylonitic shear zones were found and thus no brittle reactivation could occur, practically no fractures were observed between tunnel metres 60 and 80. At 85 m, two subhorizontal fracture traces were mapped on both sides of the tunnel wall; these lie in the zone of the breakouts. Many small fractures splay off this main fracture.

EDZ structures
Four different grades of excavation disturbed zone have been mapped, which are also presented in Attachment 2. These are:
- zone with breakouts
- zone with strongly altered fabric
- zone with moderately altered fabric
- zone with weakly altered fabric.

Unusual sigmoidal shaped cavities, which are arranged vertically along the SE tunnel wall, have been observed. In these cavities, unloading fractures with small trace lengths were formed (see Attachment 2). The displacement along these fractures shows a systematic pattern: the right-hand part (foot wall in the SW) has moved downward relative to the left-hand part. This is indicated on the map with positive (hanging wall) and negative (foot wall) signs.
3.2.2 Core logging and borehole video

Core logging

During September and October 1995, six short boreholes were drilled between tunnel metres 83 and 85 (see Attachment 2). Boreholes EDZ95.001 to EDZ95.004 were drilled in a plane (tunnel metre 84) perpendicular to the tunnel wall in the array shown in Figure 3.1. Depths were between 2.45 m (EDZ95.004) and 2.54 m (EDZ95.002).

Fig. 3.1: Borehole configuration EDZ in the heater test tunnel (section normal to tunnel axis, tunnel metre 84).

Note: The locations of the unloading fractures in the boreholes are indicated – the extent of the fractures is unknown.
The two "acoustic emission" boreholes EDZ95.005 and EDZ95.006 were drilled at the same level, one metre to the right and left of EDZ95.003. They were also perpendicular to the tunnel wall and were 0.60 and 0.63 m deep respectively. All boreholes were drilled with a single tube core barrel and were water-flushed. The core barrel had an outer diameter of 35 mm, resulting in a core diameter of 22 mm.

No oriented drillcores were recovered. Examples of recovered cores (EDZ95.002 and EDZ95.004 to EDZ95.006) are shown in Figure 3.2. During the recording of cores from EDZ95.001 to EDZ95.006, the number of discontinuities per 0.1 m was counted (see Table 3.1). The following discontinuities were distinguished:

- **disking**: discontinuities perpendicular to the drillcore. These are either unloading joints or artificial fractures caused by drilling. It was also reported whether these discontinuities are open or closed
- **pre-existing fractures**: tectonic origin
- **unloading joints**: due to excavation effects
- **schistosity**: discontinuities which have opened along the schistosity.

The lengths of the core sequences in the individual boreholes were also recorded. Because of the small borehole and core diameter, and the fact that the boreholes were drilled with a single core barrel, it proved impossible to avoid core losses (Table 3.1). In the majority of cases, these core losses could not be located within the core sequence and it can only be guessed whether or not they are in fact associated with marked unloading and/or fracturing. For this reason, the boreholes EDZ95.001 to EDZ95.004 were examined again at a later stage using a digital borehole camera.
<table>
<thead>
<tr>
<th>Bore-hole EDZ</th>
<th>Number of discontinuities</th>
<th>Core sequence [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Disking (open/closed)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>f = pre-existing fracture, s = schistosity</td>
<td></td>
</tr>
<tr>
<td>95.001</td>
<td></td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.00</td>
</tr>
<tr>
<td></td>
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<td></td>
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<td>1/0</td>
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<td>0/0</td>
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<td></td>
<td></td>
<td>7/1</td>
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<td>8/2</td>
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<td>2.00</td>
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<td>1/0</td>
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<td>1/0</td>
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<tr>
<td></td>
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<td>0/0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0/0</td>
</tr>
<tr>
<td>95.002</td>
<td>Minimal core recovery, not able to locate</td>
<td>2.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1/0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0/0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1/0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3/0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5/1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7/2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4/0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5/1</td>
</tr>
</tbody>
</table>

95.003 Minimal core recovery, not able to locate

95.004 KV n.lok. = core loss, not able to locate

95.005 KV n.lok. = core loss, not able to locate

95.006 KV n.lok. = core loss, not able to locate

KV n.lok. = core loss, not able to locate
The drillcore logs presented in Table 3.1 can be interpreted as follows:

- In boreholes EDZ95.002 and EDZ95.003, which were drilled in the most extensively disturbed zones with breakouts (Attachment 2), core recoveries were minimal to a depth of at least 1.5 m.
- In boreholes EDZ95.001, EDZ95.004 and EDZ95.006, where cores were recovered from the tunnel wall on, an accumulation of discontinuities can be observed only from a depth of 0.22 to 0.24 m. From this depth, the frequency of discontinuities increases strongly in all boreholes. This first increase in frequency is observed to extend to a depth of 0.39 m in EDZ95.001, to a depth of 1.45 m in EDZ95.004 and, in EDZ95.006, to the end of the core at 0.63 m.
- In EDZ95.001, further increases in frequency of discontinuities were observed between depths of 1.05 and 1.38 m and 1.59 and 1.67 m. In EDZ95.002, these occur from a depth of 2.17 m.
- In borehole EDZ95.005, which was drilled to detect acoustic emissions, a core loss of 19 cm occurred (equal to about one third of the whole borehole) and could not be located along the borehole. Therefore no conclusions could be drawn from this borehole.

### Borehole video

As already mentioned, it was impossible to decide on the basis of the core logs alone whether the numerous discontinuities running perpendicular to the drillcore (Figure 3.2) are partly unloading joints or whether they are almost exclusively due to the drilling process, i.e. so-called drilling breaks. To clarify this, on 24th April 1997 boreholes EDZ95.001 to EDZ95.004 were investigated using a digital borehole camera.

In boreholes EDZ95.001 and EDZ95.004, no discontinuities were observed with the camera. Detecting discontinuities was made difficult by the numerous drilling grooves on the borehole walls (Figure 3.3) and by the fact that the borehole walls cannot be viewed at right angles with the camera used.

Only in boreholes EDZ95.002 and EDZ95.003, which were drilled in the most extensively loosened zones with breakouts, were unloading joints observed. Around 17 of these features were observed in EDZ95.002 (Figure 3.4) from the mouth of the borehole to a depth of 0.15 m; in EDZ95.003 one unloading joint (Figure 3.5) was observed within 0.05 m from the borehole mouth.

In addition, within approximately 1.35 m in EDZ95.002 one open fracture was observed running along the entire length of the borehole (Figure 3.4 and 3.6). This could be also interpreted as a borehole breakout due to high stress concentration along the borehole wall (ORTLEPP, 1997).

### 3.2.3 Fracture mapping

Only in borehole EDZ95.004 could an extension vein filled with quartz and epidote be observed at a depth of 1.50 to 1.60 m. In the first two core lifts from EDZ 95.002, core recovery was minimal to a depth of 1.54 m due to the presence of a joint running along the borehole axis.

As already discussed in section 3.2.2, both ductile and brittle structures can be observed in the heater test drift between tunnel metres 75 and 88. The veins at tunnel metre 80, which are healed mainly with quartz and to a lesser extent with epidote, dip with an angle of around 60° towards the NNW.
Fig. 3.3: Borehole EDZ95.003 (depth of 1.80 m from borehole mouth).
Drilling grooves on the borehole wall

Fig. 3.4: Borehole EDZ95.002 (depth of 0.15 m from borehole mouth).
Unloading joints near the mouth of the borehole combined with an open fracture running along the length of the borehole
Fig. 3.5: Borehole EDZ95.003 (depth of 0.05 m from borehole mouth)
One unloading joint

Fig. 3.6: Borehole EDZ95.002 (depth of 0.30 m from borehole mouth)
Open fracture running along the entire length of the borehole
At tunnel metre 85, two subhorizontal fracture traces were mapped on both sides of the tunnel wall; these lie within the zone of the breakouts. Many small fractures splay off this main fracture. Both fracture traces seem to belong to one fracture, which dips with almost 45° to the W. This steeply dipping fracture seems to have promoted or even initiated the breakouts on the tunnel surface. The fracture orientation and extent suggest that the fracture itself may be older than the formation of the EDZ.

### 3.2.4 Geological input parameters for hydraulic modelling

Based on an analysis of the recorded cores, tunnel mapping and images obtained with the borehole camera, the geological parameters presented in Table 3.2 were obtained.

#### Tab. 3.2: Input parameters for hydraulic modelling derived from recorded core analysis, tunnel mapping and video imaging of boreholes.

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Pre-existing fractures</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Orientation [azimuth/angle of dip]</td>
<td>Frequency</td>
</tr>
<tr>
<td>EDZ95.001</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>EDZ95.002</td>
<td>285°/45°</td>
<td>&gt; 5 and &lt; 10</td>
</tr>
<tr>
<td>EDZ95.003</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>EDZ95.004</td>
<td>330°/60°</td>
<td>1 / 1.6 m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Unloading joints</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Orientation [azimuth/angle of dip]</td>
<td>Frequency</td>
</tr>
<tr>
<td>EDZ95.001</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>EDZ95.002</td>
<td>120°/45° (perpendicular to borehole axis)</td>
<td>17 / 0.15 m</td>
</tr>
<tr>
<td>EDZ95.003</td>
<td>120°/45° (perpendicular to borehole axis)</td>
<td>1 / 0.05 m</td>
</tr>
<tr>
<td>EDZ95.004</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

These parameters were used as input for hydraulic modelling. The frequency is given for pre-existing fractures and unloading joints, the orientation is taken from tunnel mapping and the extent is estimated (see also Figure 3.1).

### 3.2.5 Conclusions

As previously mentioned, the breakouts in the EDZ were promoted or even initiated by a fracture steeply dipping at 45° towards the West. Many smaller fractures splay off this fracture. The other EDZ structures, such as the sigmoidal shaped cavities and the unloading joints, are associated with the drilling of the tunnel using the tunnel boring machine (TBM). At the level of the EDZ, the TBM excavated the heater test tunnel in a curve. The breakout phenomena occur only in this curve.
Although the excavation disturbed zones in the Central Aare Granite are expected to be very small, it nevertheless proved possible to characterise one of these zones macroscopically.

### 3.3 Saturation of the rock

For the preparation of the test site to establish single-phase (water) conditions, it was decided to isolate the test drift from the normal tunnel ventilation in the GTS. This was performed by means of two partitioning walls with a double door (used as an airlock), thus allowing complete saturation of the rock. Evaporation measurements were carried out in order to control the status of the saturation.

For this purpose, a so-called "Evapometer" (see FRIEG & VOMVORIS, 1994) was installed in the EDZ drift behind the doors and put into operation on 04.12.94. During the measurement, air from the test drift is sucked through the evaporation cell of the Evapometer, which was fixed on the rock of the drift wall. Temperature and relative humidity of the air were measured before and after the cell and these values were compared. The air stream becomes warmer and thus dryer as it is sucked into the cell. Therefore the temperature and relative humidity data do not represent absolute values for the drift climate, but relate to the operation of the Evapometer itself. By measuring the "suction", the relative humidity conditions in the drift could be estimated. The data are graphically represented in Figure 3.7.

The closure of the doors on 23.12.94 (= Julian day 1995: -7) is clearly visible in the relative humidity record (in Figure 3.7). Before closing the doors, the relative humidity was about 88 %, because the EDZ drift was still connected to the tunnel ventilation at this time. After closure of the door the relative humidity increased above 97 %. In Figure 3.8 the total vapour flux (evaporation rate) is graphically represented. This was calculated from the measured temperatures, relative humidities, relative pressure and volumetric air flow at the inlet and outlet of the Evapometer (see FRIEG & VOMVORIS, 1994). After closure of the door a negative flux can be observed, which indicated that humidity was entering the rock of the drift wall. After nearly three weeks of measurement the curve approached and exceeded the zero line and remained relatively constant. It can be concluded that there was no longer any fluid flow into the rock. Using the Evapometer measurements it was shown that the EDZ drift was completely saturated after a period of approximately 5 weeks.
Fig. 3.7: Changes in relative humidity during saturation of the EDZ drift.

Fig. 3.8: Changes in total flux during saturation of the EDZ drift.
3.4 Surface sealing

After complete saturation of the rock in the test area it was decided to seal the tunnel surface with a resin. The aim was to establish defined experimental boundary conditions during hydraulic testing and avoid short circuits with outflows into the tunnel during the injection tests.

To decide what kind of resin should be used, a small test programme was set up. The following three different resin paint materials were tested to determine their suitability (by SIKA AG, Zürich):

- Sikadur 30 (two component epoxy resin)
- Sikadur 31 (two component epoxy resin)
- Combination of Epocem 75 and Sikagard 62.

The main objective was to demonstrate complete contact and the adhesive tensile strength of the different resins on the rock surface. The test procedure included the following steps:

- 13.04.95: Three areas (1 m² each) were prepared by applying Sikadur 30, Sikadur 31 and a first paint of a combination of Epocem 75 + Sikagard 62.
- 18.04.95: A second paint of the Epocem 75/Sikagard 62 area was put on and test measurements of the adhesive tensile strength of the areas painted with Sikadur 30 and 31 were performed.
- 24.04.95: The adhesive tensile strength of the Epocem 75 / Sikagard 62 area was also tested.

The test results are summarised in Table 3.3. From these tests it was decided that Sikadur 31 provided the best results and was easy to employ.

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity required [kg/m²]</th>
<th>Age of sample [days]</th>
<th>Diameter [cm]</th>
<th>Adhesive tensile strength [N/mm²]</th>
<th>Breakage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epocem 75 + Sikagard 62</td>
<td>4 (Epocem) 0.6 (Sikagard)</td>
<td>11</td>
<td>2.6</td>
<td>0.2 0.4 0.2</td>
<td>Break in the granodiorite</td>
</tr>
<tr>
<td>Sikadur 30</td>
<td>5</td>
<td>7</td>
<td>5.3</td>
<td>0.3 0.3 0.8</td>
<td>Break in the granodiorite</td>
</tr>
<tr>
<td>Sikadur 31</td>
<td>5</td>
<td>7</td>
<td>5.3</td>
<td>0.8 0.3 0.5</td>
<td>Break in the granodiorite</td>
</tr>
</tbody>
</table>

\(^{1)}\) triplicate samples

Therefore the tunnel sealing was performed with Sikadur 31 with adhesive tensile strengths between 0.3 and 0.8 Newtons per square millimetre (Table 3.3) in October 1995. The surface sealing with resin created defined boundary conditions for the hydraulic tests with potential injection pressures up to 0.8 MPa.
4 ROCK MECHANICAL MEASUREMENTS AND MODELLING

The stress field in the near-field of excavations has a decisive influence on the hydraulic near-field behaviour. This is mainly due to the fact that even small variations in the aperture of fractures caused by stress redistributions due to excavation can result in large variations in the flow rate because of the strong non-linearity between aperture and flow rate (the so-called “cubic law”, see for example WITHERSPOON et al., 1980).

The rock mechanical measurements and modelling were carried out as part of the main phase 1 of the EDZ experiment during preparation of the test area (see Chapter 2).

4.1 In-situ stress measurements

The objective of these investigations was to record the stress redistributions in the tunnel near-field due to excavation for rock mechanical design calculations. In-situ stress measurements from the past indicated that a clear anisotropy is present at the Grimsel Test Site, which can be responsible for the development of an EDZ at the GTS (Table 4.1). The stress, which is 4 - 5 times higher than the lithostatic pressure of around 9 – 12 MPa, indicates the presence of significant horizontal forces in the main compression direction NW-SE (PAHL et al., 1986; KEUSEN et al., 1989).

Tab. 4.1: In-situ stress measurements at the GTS.

<table>
<thead>
<tr>
<th></th>
<th>EGGER &amp; DESCOEUDRES (1985)</th>
<th>PAHL et al. (1989)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. horizontal stress ($S_{h}$)</td>
<td>28 MPa NW-SE</td>
<td>28 – 45 MPa NNW-SSE NW-SE</td>
</tr>
<tr>
<td>Medium stress (inclined)</td>
<td>16 MPa 37°- to NE</td>
<td></td>
</tr>
<tr>
<td>Low stress (inclined)</td>
<td>12.6 MPa 53°- to SW</td>
<td>15 – 32 MPa ($S_{h}$) 10 – 14 MPa ($S_{v}$) Min. horizontal stress</td>
</tr>
</tbody>
</table>

During site preparation, in-situ stress measurements were performed using a borehole slotter probe (BOCK, 1993) in the radial borehole EDZ94.001 in the heater test tunnel. The borehole was drilled subhorizontal (upwards at 5°) in a NW direction (azimuth 324°) in the Central Aare Granite.

The measurements were performed during 5th to 6th December 1994.

4.1.1 Borehole slotter

Since 1990 the borehole slotter has been utilised in numerous research and commercial projects in order to obtain reliable in-situ stress measurement data (HARTKORN & WOHNLICH, 1995). It consists of a swivelling diamond saw and a strain sensor as central part of the probe, which are installed in a steel casing. Two hydraulic cylinders at each end of the casing anchor the probe in the borehole.
The borehole slotter was used in the GTS to obtain a vertical profile of 2D horizontal stresses in borehole EDZ94.001 at distances between 0.5 m to 9.1 m from the tunnel wall along the borehole. The condition of the borehole core was considered to be suitable for reliable in-situ stress measurements. In total 70 slotting tests were carried out.

For the interpretation of the stress data, the measured strain versus time curves were critically reviewed and verified for plausibility. Because of this procedure only 21 data curves from 70 were accepted. These values were interpreted by the means of the PC program IFSLOT-D (HARTKORN & WOHNLIICH, 1995) and the results were presented graphically. For this purpose a YOUNG's modulus E of 40 GPa (35 – 47 GPa, EGGER & DESCOEUDRES, 1985; 40 GPa, PAHL et al., 1989) and a POISSON's ratio ν of 0.25 (0.2 – 0.25, EGGER & DESCOEUDRES, 1985; 0.25 PAHL et al., 1989) were used.

Stress profile results are given in Figure 4.1 and the raw data (average values) are included in HARTKORN & WOHNLIICH (1995).

4.1.2 Results

The calculated in-situ stresses are characterised by the following parameters (see Table 4.2):

- Magnitude of the major principal stress normal to borehole $\sigma_1$
- Magnitude of the minor principal stress normal to borehole $\sigma_2$
- Orientation of $\sigma_1$ in relation to the local reference orientation
- Correlation coefficient $R^2$ (only if obtained value is based on more than 3 single slot measurements).

Figure 4.1 includes a graphical representation of the results in the form of a borehole depth plot and also a sum diagram.

The magnitude of the major principal stress $\sigma_1$ varies between 9.0 MPa at 9.0 m depth to 14.7 MPa at 2.0 m depth. The minor principal stress $\sigma_2$ reaches values between 5.2 MPa at 9.0 m depth and 8.0 MPa at 2.0 m depth. The orientation of $\sigma_1$ is in the range of 130 to 147° and corresponds to the results of earlier measurements at the Grimsel Test Site (PAHL et al., 1989).

All measurements were performed in the Central Aare Granite, which is homogeneous in this area and only slightly fractured and represents relatively ideal conditions for the stress measurements. This is confirmed by the sum diagram in Figure 4.1, where the calculated principal stresses are scattered over a relatively narrow area.

A small stress increase and microfissures could be identified in the tunnel near-field, which suggests the potential existence of a plastic zone, but the investigations carried out are not sufficient to reliably detect such a zone. For further investigations, the density of measurements should be enhanced and carried out in several boreholes which are differently oriented so that conclusions about the 3D stress-state can be drawn.
Fig. 4.1: Results of the measurements with the borehole slotter probe in borehole EDZ94.001.

Above: Depth diagram with orientation of $\sigma_1$ and the magnitude of principal stress, below: Sum diagram with values of $\sigma_1$ and $\sigma_2$ and presentation of amount and direction (after: WOHNLIICH & HARTKORN, 1995).
Tab. 4.2: Calculated principal stresses normal to borehole from measurements with the borehole slotter probe at different depths in borehole EDZ94.001 (WOHNLICH & HARTKORN, 1995).

<table>
<thead>
<tr>
<th>Depth [m]</th>
<th>Orientation: Azimuth of $\sigma_1$ [$^\circ$]</th>
<th>$\sigma_1$ [MPa]</th>
<th>$\sigma_2$ [MPa]</th>
<th>Correlation coeff. $R^2$ [%]</th>
<th>Slot No.</th>
<th>YOUNG's modulus E [GPa]</th>
<th>POISSON's ratio $\nu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>130</td>
<td>13.7</td>
<td>9.1</td>
<td>-</td>
<td>3</td>
<td>40</td>
<td>0.25</td>
</tr>
<tr>
<td>1.0</td>
<td>147</td>
<td>10.8</td>
<td>5.5</td>
<td>-</td>
<td>3</td>
<td>40</td>
<td>0.25</td>
</tr>
<tr>
<td>1.1</td>
<td>149</td>
<td>10.8</td>
<td>5.3</td>
<td>98</td>
<td>4</td>
<td>40</td>
<td>0.25</td>
</tr>
<tr>
<td>2.0</td>
<td>133</td>
<td>14.7</td>
<td>8.0</td>
<td>100</td>
<td>4</td>
<td>40</td>
<td>0.25</td>
</tr>
<tr>
<td>2.2</td>
<td>133</td>
<td>14.6</td>
<td>8.0</td>
<td>-</td>
<td>3</td>
<td>40</td>
<td>0.25</td>
</tr>
<tr>
<td>5.0</td>
<td>130</td>
<td>13.3</td>
<td>6.2</td>
<td>100</td>
<td>4</td>
<td>40</td>
<td>0.25</td>
</tr>
<tr>
<td>5.0</td>
<td>129</td>
<td>13.3</td>
<td>6.3</td>
<td>-</td>
<td>3</td>
<td>40</td>
<td>0.25</td>
</tr>
<tr>
<td>8.8</td>
<td>146</td>
<td>9.8</td>
<td>5.0</td>
<td>-</td>
<td>3</td>
<td>40</td>
<td>0.25</td>
</tr>
<tr>
<td>8.9</td>
<td>146</td>
<td>9.4</td>
<td>5.4</td>
<td>98</td>
<td>6</td>
<td>40</td>
<td>0.25</td>
</tr>
<tr>
<td>9.0</td>
<td>143</td>
<td>9.0</td>
<td>5.2</td>
<td>-</td>
<td>3</td>
<td>40</td>
<td>0.25</td>
</tr>
<tr>
<td>9.1</td>
<td>147</td>
<td>9.1</td>
<td>5.8</td>
<td>-</td>
<td>3</td>
<td>40</td>
<td>0.25</td>
</tr>
</tbody>
</table>

4.2 Rock mechanical modelling

The previous stress measurements (section 4.1) and the results of geological mapping (Chapter 3) formed the basis for the rock mechanical modelling of the EDZ. The objective of the modelling was to develop an understanding of the primary in-situ stress field and the secondary stress field around the caverns in the area of the Grimsel Test Site, including the behaviour of the discontinuities. The model results should give an estimate of the extent and the main characteristics of the excavation disturbed zone around the tunnel. The rock mechanical modelling represents the basis for the design of the hydrological investigations within the EDZ project.

A regional calibrated three-dimensional model of the GTS and the surrounding rock mass (near-field) is described in section 4.2.1. There, the influence of the topography as well as the influence of the existing tectonic discontinuities or fracture systems on the primary in-situ stress field is investigated.

The obtained 3D stress state is used for describing the initial and boundary conditions for the local 2D model of the WT drift cross-section in the GTS in section 4.2.2. This two-dimensional model investigates the stress redistributions, possible plastifications and the joint behaviour (joint closure, joint opening and joint shear displacements) in the near-field of the tunnel.
4.2.1 Regional 3D stress field modelling

For the regional 3-dimensional modelling, a block model (10 × 8.5 × 6 km) was set up to determine the primary stress-state and the stress boundary conditions for the later local-scale modelling of a tunnel section (KONIETZKY, 1995). The code 3DEC (three-dimensional Distinct Element Code), a dynamic path-dependent and energy-balanced code, which reaches static equilibrium by damping was used (ITASCA, 1994).

The modelled area was subdivided into right polyhedrons with triangular bases of different sizes based on topographic maps and digitised data. The subdivision was performed in such a way that the main topographic features were adequately represented and the model size in terms of calculation time and computer memory was acceptable.

This model consists – regarding the block structure – of a denser inner part, which contains the Grimsel Test Site, and a more coarsely subdivided outer area. All blocks comprising the model were internally discretised into tetrahedral zones. The model contains about 152,000 3D elements. Both the inner and the outer model area have a rectangular horizontal projection plane.

The inner model area has a horizontal extent of 2.25 × 2.625 km, whereas the whole model has a horizontal extent of 10 × 8.5 km. The model boundaries were oriented parallel and perpendicular to the main tectonic stress orientation of 135° (rotation of 45° to the coordinate system). Figure 4.2 shows the whole model in block structure (without the net), presented together with the aerial photograph of the Grimsel area. In Figure 4.3 the whole model is shown; different regions are plotted in different colours.

The model contains 9 discontinuities. Only discontinuities or fracture systems recognised in the area of the Grimsel Test Site based on KEUSEN et al. (1989) were implemented.

The rock mass was represented by a linear elastic material law, whereas the joints were represented by a COULOMB friction law with limited tensile strength and variable dimension (see KONIETZKY, 1995). The following material parameters for the rock mass were used for the modelling calculations:

- YOUNG’s modulus E: 40 GPa (PAHL et al., 1989)
- POISSON’s ratio \(\nu\): 0.33 (PAHL et al., 1989 ; KEUSEN et al., 1989)
- Density \(\rho\): 2700 kg/m^3 (KEUSEN et al., 1989)

In the regional 3D numerical stress field modelling the results from several in-situ stress field measurements (see KEUSEN et al., 1989, PAHL et al., 1989 and WOHNLICH & HARTKORN, 1995) were incorporated. The results from the modelling suggest that the primary in-situ stress field in the area of the Grimsel Test Site can be described as follows:

The primary in-situ stress field in the GTS is heavily influenced by the topography. The pure gravitational modelling (Figure 4.4; model case C1), where the model was consolidated under gravitational load, has revealed that even when making this simple assumption the vertical direction is not a principal stress direction.

A comparison between the modelling results from the pure gravitational model (model case C1) and the in-situ stress measurements has shown that the primary in-situ stress field cannot be explained by the effect of topography and gravity alone, especially regarding the high quasi-horizontal maximum principal stress component.
Fig. 4.2: Whole 3DEC model in block structure (lakes in black) and the corresponding aerial photograph of the Grimsel area (KONIETZKY, 1995).

Grimsel area  (view to the west)

1  Test Site
2  Juchlistock
3  Lake Raeterichsboden
4  Lake Grimsel
Figure 4.5 shows that a good agreement between the measured and calculated stresses inside the GTS (see also section 4.2.2) was achieved by applying an additional tectonic far-field stress component with a linear increase from 0 MPa at +2000 m to 50 MPa at -3000 m at the model bottom (model case C3). The vertical stress gradient of 0.01 MPa/m is in the direction of 135° (NW-SE).

The results regarding the stress field in the GTS (1730 m asl; inner domain of the model with a refined discretisation) derived from model case C3 are summarised in Table 4.3 below (after KONIETZKY, 1995). The orientation of $\sigma_2$ and $\sigma_3$ are in satisfactory agreement with the orientation of the maximum and minimum normal stresses obtained by the borehole slotter measurements.
Tab. 4.3: Summary of modelling results from model case C3.

Ranges of magnitude, dip and dip direction of the principal stress directions of the primary stress field in the vicinity of the GTS (KONIETZKY & MARSCHALL, 1997)

<table>
<thead>
<tr>
<th>Stress component</th>
<th>Magnitude [MPa]</th>
<th>Dip [°]</th>
<th>Dip direction [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_1$</td>
<td>24.7 – 30.0</td>
<td>12 – 20</td>
<td>121 – 129</td>
</tr>
<tr>
<td>$\sigma_2$</td>
<td>11.6 – 14.1</td>
<td>29 – 35</td>
<td>19 – 30</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>7.0 – 9.7</td>
<td>47 – 57</td>
<td>235 – 250</td>
</tr>
</tbody>
</table>

The fault zone system has (given the model assumptions used for this study) no significant influence on the primary in-situ stress field. Furthermore, the activation of the fault zones has not led to any significant stress changes in the model, but the application of the tectonic stress component does. The influence of the fault zones is very limited. The main reason for this are:

- The fault zones are crossing the whole model and reach the vertical and bottom boundaries, where the normal displacements are zero.
- Due to the orientation of fault zones with respect to the principal stresses and the applied loads, only small shear stresses along the faults are induced.
- The chosen joint normal and joint shear stiffness parameters as well as the frictional angle are relatively high. They correspond to unweathered and relatively stiff faults.
- The chosen modelling sequence has included that the stiffness parameters (elastic fault response) are already active during the consolidation of the model. That means that their influence is already included in the pure gravitational model C1.

Based on the model results, appropriate far-field stresses can be deduced for all subsequent geomechanical and hydrological calculations for the GTS.

4.2.2 Local 2D numerical disturbed zone modelling results

The tunnel section itself was then modelled (KONIETZKY, 1995) with the two-dimensional Distinct Element Code UDEC (ITASCA, 1993). The model considered a vertical plane $24 \times 24$ m assuming plane strain conditions. The model geometry was based on the geological data summarised in Chapter 3. The tunnel radius is 1.85 m and the radius of the disturbed zone is 3.85 m. Figure 4.6 shows the model in block structure including the joint/discontinuity pattern. In Figure 4.7 the whole model after tunnel excavation is shown. Due to the discontinuities, the model is subdivided into 42 blocks.
Fig. 4.4: Principal stress vectors for pure gravitational load, vertical cross-section; colours correspond to the stress magnitude of $\sigma_1$, stresses are given in Pa (KONIETZKY, 1995.)
Fig. 4.5: Principal stress vectors (gravitational load + additional tectonic far-field stress component), vertical cross-section; colours correspond to the stress magnitude of $\sigma_1$, stresses are given in Pa (KONIETZKY, 1995).
Each block consists of 12,011 zones, which corresponds to 6799 grid points (KONIETZKY, 1995).

The following material parameters (for the rock mass) were chosen for the reference case calculation. These have been defined based on the newest results from laboratory investigations, performed in the framework of the planning stage of the Lötschberg base tunnel (GHARAVIZADEH, 1995):

- Bulk modulus: 28.2 GPa
- Shear modulus: 19.5 GPa
- Density: 2659.5 kg/m$^3$
- Cohesion: 4.7 MPa
- Friction angle: 51°
- Tensile strength: 0 MPa
The elasto-plastic material behaviour was represented by the MOHR-COULOMB law with non-associated flow rule (dilation angle = 0) and tension limit. The joints were represented by a COULOMB friction law with limited tensile strength and variable dilation.

The following model calculations were performed for different elasto-plastic calculation cases with parameter variations (see KONIETZKY, 1995):

- Calculations until static equilibrium is reached for the primary stress state.
- Setting all displacements for the rock matrix and the joints to zero except the joint normal displacements.
- Creation of the tunnel (removal of the material inside the tunnel).
- Calculations until static equilibrium is reached for the disturbed state.
All displacements for the rock matrix and the shear displacements of the discontinuities are the result of the tunnel creation alone. Figure 4.8 shows the displacement vectors and the plasticity index for the reference calculation case.

From this figure it can be seen that the elasto-plastic stress redistributions in the immediate near-field of the tunnel are most pronounced along the discontinuities. Directly at the tunnel wall tensile failure can be observed. The displacement field is inhomogeneous and anisotropic around the opening. Maximum displacements of about 10 mm occur, induced by the tunnel opening. The displacement field is strongly influenced by the discontinuities.

In the different model cases, the maximum shear deformations of two to five millimetres occur in the tunnel near-field and the stress distribution is strongly influenced by the present discontinuities. The largest convergence movements of up to 10 mm occur on the eastern tunnel wall (Figure 4.8). For the reference calculation case the maximum tangential stress at the tunnel surface reaches values of up to 60 MPa (see contour image in Figure 4.9).

For comparison, a model run without discontinuities was performed. This led to a totally different stress and displacement field. For this homogeneous continuum model, the stress redistribution depends mainly on the orientation of the principal stress directions to the tunnel axis and to the strength of the rock mass. The maximum displacements at the tunnel wall are about 2 mm and show the expected “regular” pattern.
Fig. 4.8: Reference calculation case.

above: displacement vectors [m] for a window plot; below: plasticity index for a window plot (KONIETZKY, 1995).
4.2.3 Conclusions

From the numerical rock mechanical studies on the regional scale it can be concluded for the vicinity of the GTS:

- The orientations of the principal stress components are highly variable due to the complex topography.
- The primary in-situ stress state is characterised by quasi-horizontal stress components due to the regional tectonic stress field.
- The vertical direction is not a principal stress direction.
- The maximum principal stress direction is NW-SE directed and can be considered as quasi-horizontal (deviation from the horizontal plane lower than 20°).

On the basis of the local 2D numerical rock mechanical disturbed zone modelling results, the mechanical near-field behaviour of the tunnel can be characterised as follows:

- Compared to a continuum approach, the discontinuum modelling has revealed a very inhomogeneous and anisotropic mechanical behaviour, where the discontinuities have a decisive influence on the mechanical behaviour. The discontinuum approach has resulted in more than 5 times higher displacement values compared to the continuum approach. Also temporary plastifications and the ensuing stress redistributions are strongly connected to the discontinuities. Therefore, the joint material parameters and the joint pattern have a more decisive influence on the geomechanical behaviour than the rock matrix parameters.
The displacement field shows strong inward movements. They reach values of about 10 mm at the ESE-side and only about 3 mm at the NWN-side for the discontinuum approach, again strongly influenced by the discontinuity pattern.

All model calculations lead to stable conditions for the tunnel. Due to stress redistributions in the immediate near-field/vicinity of the tunnel, temporary plastifications are overcome and the material returns to the elastic range.

Due to the tunnel opening, induced joint shear displacements of up to about 5 mm and joint normal displacements up to about 1 mm are observed. The induced change in joint closure and joint normal displacements behind the disturbed zone (up to 2 m behind the tunnel wall) is less than 10%. Nearly all observed movements along the discontinuities are elastic. The pronounced joint movements occur immediately at the tunnel wall, with a penetration of not more than 1 m into the rock mass. Joint separation and joint slip is very limited and restricted to some joints at the ESE-side of the tunnel wall and one intersection of two joints with the upper tunnel wall.
5 HYDRAULIC DESIGN CALCULATIONS

In order to optimise the hydraulic tests in terms of configuration and performance, hydraulic design calculations were first performed, using two different modelling approaches. The excavation disturbed zone was modelled on the one hand as an equivalent porous medium and, on the other hand, as a fracture network. In subsequent analysis of the hydraulic tests, the different approaches could be distinguished on the basis of the pressure evolution in the test intervals. The programme of hydraulic tests in the radial boreholes was prepared on the basis of this modelling.

5.1 Scope and objectives

As already discussed in section 1.1, the aim of the EDZ project is to develop a general, site-independent investigation concept, which will allow a reliable hydrogeological characterisation of the EDZ. Point measurements using hydrotests were planned as part of this development process. The analysis and interpretation of these measurements should provide the hydraulic parameters (T, K) and their variability and, whenever possible, the identification of the underlying hydraulic flow model.

Against this background, the first numerical simulations were carried out in the form of hydraulic design calculations, to provide support for planning the set up and procedure for the physical tests. Two different numerical modelling approaches were used. While the so-called EPM model assumes the excavation disturbed zone to be an equivalent porous medium, the alternative approach characterises the disturbed zone using a Discrete Fracture Network (DFN).

The objectives of the design calculations can be summarised as follows:
- Determination of input parameters for the design of the hydraulic tests
- Support of the conceptual flow model identification for the interpretation of hydraulic test data
- Sensitivity analysis using different hydraulic parameters
- Improvement of the understanding of flow processes in the tunnel vicinity (influence of geometry) during the execution of hydraulic tests
- Evaluation of the model concept used

The results of the design calculations provide a basis for planning the hydraulic tests, by allowing concrete suggestions to be formulated for selecting technical design parameters such as:
- the dimensioning of boreholes and test intervals (e.g. diameter, length)
- the configuration for crosshole tests (e.g. number and location of measurements) and
- the duration and sequence of the hydrotests (e.g. type/scenario of the test, pumping rates etc.)

In this context, design is to be understood in the sense of optimisation. By simulating the hydrotests numerically in different configurations, it is possible to determine quantitatively the influence of individual (design) parameters on the flow conditions. Finally, a comparison procedure is used to evaluate the different system reactions in terms of their suitability for later inverse modelling consideration (parameter estimation). If this estimation is to be successful, informative data will be required which reflect both the hydrogeological parameters and their spatial structure.
Furthermore, the numerical EDZ models also offer the possibility to differentiate the general system behaviour by making different assumptions with respect to the underlying parameter structure. Graphic processing of the numerical calculation results using diagnostic plots (type curves) allows the different structures to be visualised. During the 'quick-look analysis' of the experimental data, such diagrams may substantially simplify the flow model identification.

The calculation methods used (transient sensitivity coefficients) also allow the influence of the hydrogeological parameters to be quantified at each time point in the simulations (and thus as an approach also during the later hydrotests). This contributes substantially to the understanding of flow conditions during hydrotests in the tunnel vicinity.

5.1.1 Database

A set of basic assumptions regarding the geometry and structure of an excavation disturbed zone was made at the planning stage of the project (MARSCHALL, 1994). These assumptions were derived from a series of documented investigation results. VOMVORIS et al. (1993) summarise current information and draw the following general conclusions for modelling the crystalline basement of Northern Switzerland in the context of performance assessment:

- The radial extent of an excavation disturbed zone, which is defined as the zone with a hydraulic conductivity at least 100 times higher than that of the undisturbed rock, should be in the order of 1 m (for a tunnel diameter of approximately 3.5 m). This is followed by a further zone, extending up to one tunnel diameter, which has a K value increase by a factor of 10.

- As a first approximation, the shape of the disturbed zone can be assumed to be circular. However, further studies based on stress calculations suggest rather an anisotropic structure in the form of an ellipse (Figure 5.1).

- Independent of the form of the EDZ, it is expected that the axial hydraulic conductivity could increase by up to three orders of magnitude.

- The change in radial conductivity is dependent on local in-situ stress conditions. Based on observations from various underground laboratories (e.g. Stripa), it is expected that a decrease rather than an increase will occur.

In an earlier investigation phase (e.g. “Auflockerungszone / AU experiment, EGGER & DESCOEUDRES, 1985; Gebirgsspannungen / GS experiment, PAHL et al., 1989), rock mechanical stress measurements were performed at the Grimsel Test Site. These investigations revealed a marked anisotropy, which pointed to the existence of a disturbed zone. The seismic measurements performed as part of the AU project showed a decrease of the p-wave velocity of 0.55 km/s (≈ 10 %) and indicated altered conditions to a depth of approximately 0.6 m in the Grimsel Granodiorite (full-face tunnel boring section). In lamprophyres and shear zones, the decrease was about 20 % (eq. 1 km/s) and indicated a depth of the EDZ of about 1.2 m around the tunnel. Further seismic crosshole measurements with high frequency signals performed as part of the FRI project (Fracture zone Investigation, MAJER et al., 1990) indicated the existence of a disturbed zone which is estimated to extend to a depth of approximately 1.5 m in the drilled MI tunnel of the GTS. In the drill and blast main access tunnel, the EDZ is expected to have a thickness of about 3 m. Based on these measurements, the extent of the hydraulically active excavation disturbed zone with an approximately 10 times higher conductivity is assumed to be 1 m.
5.2 Equivalent Porous Medium (EPM) approach

5.2.1 Basic concept and assumptions

Modelling concept

The basic assumption behind this approach is that the excavation disturbed zone in the GTS can be considered to be an equivalent porous medium (EPM). Due to the complex geometry of the EDZ in the tunnel vicinity, a fully three-dimensional representation of the flow fields as induced by the hydrotests is required. For this purpose, a section of a cylindrical tunnel is modelled, from which one or more arrays of radial boreholes equipped with multipacker systems are drilled.

Based on symmetry considerations, the model can be reduced in a vertical direction (height) to a semi-circular section. The radial extent of the modelled domain (thickness) – beyond the dimensions of the EDZ mentioned above (approximately 1 m) – must be chosen in such a way that, during the simulated hydrotests, the influence of the model boundary facing towards the rock matrix remains negligible. Depending on the hydrogeological conditions considered and the selected duration of the simulation, it has to be ensured from case to case that the perturbation front induced by the tests remains within the modelled domain. The latter also applies when defining the length of the modelled tunnel section. The part of the model boundary which is towards the tunnel is considered to be impermeable (seal). The injection borehole is
also modelled explicitly (see section 5.2.2). Making use of its symmetric properties, the model can be cut in the plane of the borehole arrays (half-pipe). The number and arrangement of the observation boreholes were varied.

All the numerical simulations in this study are carried out using the program CASA (KUHLMANN, 1994). Calculation of transient groundwater flow using the finite element (FE) method is made possible by the fully three-dimensional model. Quadratic iso-parametric elements are used, which allow a realistic representation of curved model boundaries (e.g. tunnels and boreholes). With respect to the solution of the FE-equations, the iterative Pre-Conditioned Conjugate Gradient (PCCG) method is used which has been proven to be very efficient in various 3D applications.

Conceived as an inverse model for automatic estimation of model parameters, the program also provides the possibility to calculate sensitivity coefficients at any given point of the model (e.g. measurement locations). The integrated calculation and analysis of the resulting covariance matrix containing the involved parameters makes it possible to quantify the influence of individual parameters as well as the information content of the generated data.

A two-dimensional structure is assumed for the distribution of permeability within the excavation disturbed zone, with the permeability in the axial direction remaining constant. The definition of the hydrogeological parameters (hydraulic conductivity, storage capacity) is made separately for each zone according to the procedure documented above (see section 5.1.1). Three different forms of permeability structure are investigated (Figure 5.1):

- P1 - without EDZ (homogeneous distribution of permeability)
- P2 - annular EDZ core zone with transition zone (Figure 5.1)
- P3 - elliptical EDZ core zone with circular transition zone (Figure 5.1)

A conductive anisotropy of the excavation disturbed zone caused by preferred orientation of microfractures cannot be ruled out. This aspect is taken into account as a calculation variant.

Experiment design

To meet the goals formulated at the outset, it is meaningful to evaluate the design calculations using different methods:

a) Analysis of simulated potential heads in the observation intervals in the derivative plot: in this diagnostic method, which is frequently applied in practice, the evolution of pressure or potential head with time $h(t)$, as well as its derivative $\frac{\partial h}{\partial (\ln t)}$, are analysed in a log-log plot (MISHRA et al., 1991). This method emphasises changes in the rate of pressure change and contributes considerably to the identification of the underlying model structure (e.g. dimensionality of flow, flow barriers, etc.).

b) Analysis of potential evolution at characteristic points of the model: comparing the development of potential head at these points (e.g. tunnel wall or transition points between parameter zones) allows characteristics of the diagnostic plot to be assigned to corresponding physical processes.

c) Calculation of the sensitivity matrix and graphic representation of sensitivity curves: sensitivity coefficients describe the change in a performance measure of the model (e.g. potential head in an observation interval $x_i$ at the time $t_k$), induced by an infinitesimal change in a model parameter. Applied to the time sequence of a transient hydrotest, the sensitivities indicate the influence of the model parameters in every test interval. As part of
the design calculations, sensitivities are calculated in order to define the relevance of the involved hydrogeological parameters in time and space. Assuming that the results of the simulation are the data observed in the experiment, sensitivities additionally provide the basis for quantifying the measured information content. They thus allow the suitability of the underlying test design to be evaluated.

d) Calculation of the covariance matrix of the model parameters using sensitivities; and eigen-value analysis of the covariance matrix and evaluation of design criteria (see below).

Design criteria

Reliable identification of parameter values and parameter structure during the subsequent interpretation requires data, which are quantitatively as well as qualitatively sufficient. This relates not only to the accuracy of the measurements (measurement errors), but also to the location and timing of their recording. The experiment should be arranged in such a way as to allow measurements to be made that deliver data with a high information content with respect to the parameters of interest. This implies that measurements should be made at times and locations where measured values react as sensitively as possible to parameter changes. This requirement for informative data for successful treatment of the inverse problem automatically leads to a demand for criteria, which allow a quantitative judgement of the information content of the data and thus serve as a measure for the quality of an experiment.

A logical approach is to choose a criterion that relates to the expected accuracy of the involved model parameters (GOODWIN & PAYNE, 1977). A possible design criterion in this context is to minimise the determinants of the covariance matrix of the involved parameters with respect to the design variables. This procedure reduces the uncertainty of the estimated model parameters to a minimum (CARRERA 1988; SUN & YEN 1990). Conditioning the covariance matrix gives further important indications of the prospects of success of a subsequent parameter estimate. Using an eigenvalue analysis, the size and orientation of the confidence interval can be approximated. The form of the confidence interval (convexity) is a measure of the expected stability of the inverse problem and thus also of the convergence behaviour of a minimisation algorithm. Besides analysis of extreme values, the condition number can be used to quantify this behaviour. This number is derived from the extreme eigenvalues of the covariance matrix. The larger the condition number, the more certain it is that it will be possible to identify the involved model parameters. Since none of the named criteria can be favoured, it is usual in practice to determine the optimum alternatives by considering several design criteria simultaneously (CARRERA & NEUMAN 1986c).
5.2.2 Numerical model

Discretisation

The EDZ is discretised along a tunnel section as a sector of an annulus using three-dimensional elements (see section 5.2.1). To ensure the accuracy of the transient simulations and to provide a sufficient resolution of the hydrogeological properties of the EDZ, a sufficiently fine grid was selected with an edge length of the elements being in the decimetre range. Using quadrant segments, the injection borehole was discretised three dimensionally in the form of a tube. The immediate vicinity of the injection/extraction interval requires a higher degree of discretisation than the other areas. The interval packer arrangement as well as the variable length of the borehole are obtained by corresponding permeability variation of the 'borehole elements'. Three variants of finite element (FE) grids corresponding to the permeability structures defined above were constructed (see section 5.2.1):

- P1 – without a disturbed zone (homogeneous distribution of permeability)
- P2 – annular disturbed zone with transition zone (Figure 5.2)
- P3 – elliptical disturbed zone with transition zone (Figure 5.3)

Besides the spatial discretisation density, the size of the time step also has an influence on the accuracy of the numerical model. Both the structure of the grid in the borehole vicinity (size of elements) and the parameters of the time step scheme (time step size) were determined iteratively by comparison with the analytical solution of a Constant Rate Test (according to PAPADOPOLLOS & COOPER in BEAR 1979; p. 327) until an acceptable agreement (< 1 %) was reached.

Initial and boundary conditions

For the time-dependent hydrotest simulations, both boundary conditions and initial conditions must be given; applying the superposition principle, the latter can be simplified to \(h(x, t = 0) = 0\), where \(h\) is the head or groundwater potential (see section 5.2.1). With the exception of the condition in the test interval in question, where the pumping rates are defined according to the test conditions to be investigated, the FE models investigated are assigned uniform boundary conditions.

Both the tunnel wall (seal) and the symmetry boundaries of the model are assigned 'no flow' conditions. Because the extent of the model is chosen in such a way that the outer boundaries (radial and axial) have no influence on the simulation results, the type of boundary condition can be freely chosen here. For better control of the flow, fixed potential conditions are used (prescribed head \(PH = 0\)).
Fig. 5.2: FE discretisation of the permeability model P2 with a circular EDZ.
Fig. 5.3: FE discretisation of the permeability model P3 with elliptical EDZ.
5.2.3 Case definitions

Base case

First of all a base case is defined. This reflects pre-existing hydrogeological knowledge and the reference values for the technical design parameters. The geometric design parameters were first compiled based on the technical boundary values (Table 5.1). Table 5.2 lists the relevant hydrogeological parameters. Table 5.3 includes the scenario for the base case with three injection intervals.

Tab. 5.1: Geometric design parameters (base case B0).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Borehole diameter</td>
<td>0.05 m</td>
</tr>
<tr>
<td>Borehole length</td>
<td>1.0 m</td>
</tr>
<tr>
<td>Number of intervals</td>
<td>5</td>
</tr>
<tr>
<td>Packer/interval length</td>
<td>0.1/0.1</td>
</tr>
<tr>
<td>Number of boreholes per array</td>
<td>3</td>
</tr>
<tr>
<td>Angle between boreholes</td>
<td>22.5°</td>
</tr>
<tr>
<td>Number of borehole arrays</td>
<td>3</td>
</tr>
<tr>
<td>Axial distance between arrays</td>
<td>1.0 m</td>
</tr>
</tbody>
</table>

Tab. 5.2: Hydrogeological parameters for base case B0.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydraulic conductivity of the EDZ</td>
<td>K_{EDZ} 2 \times 10^{-10} [m/s]</td>
</tr>
<tr>
<td>Storage capacity of the EDZ</td>
<td>S_{EDZ} 3 \times 10^{-6} [m^{-1}]</td>
</tr>
<tr>
<td>Hydraulic conductivity of the transition zone</td>
<td>K_2 1 \times 10^{-11} [m/s]</td>
</tr>
<tr>
<td>Storage capacity of the transition zone</td>
<td>S_2 1 \times 10^{-6} [m^{-1}]</td>
</tr>
<tr>
<td>Hydraulic conductivity of the intact rock matrix</td>
<td>K_M 1 \times 10^{-12} [m/s]</td>
</tr>
<tr>
<td>Storage capacity of the intact rock matrix</td>
<td>S_M 1 \times 10^{-6} [m^{-1}]</td>
</tr>
<tr>
<td>Anisotropy</td>
<td>K_r/K_y 1.0</td>
</tr>
</tbody>
</table>

Tab. 5.3: Scenario for base case B0.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant Rate Test</td>
<td></td>
</tr>
<tr>
<td>Injection interval</td>
<td>3 (central)</td>
</tr>
<tr>
<td>Duration of test</td>
<td>1 \times 10^{-5} s (~ 1 d)</td>
</tr>
<tr>
<td>Injection rate</td>
<td>0.4 ml/min</td>
</tr>
</tbody>
</table>
Calculation variants

In addition to the base case for the FE models P1, P2 and P3, more than 50 calculation runs were performed with varied test conditions (test configurations). The studied variants can be divided into 3 groups according to their objectives (layout, hydrology, scenario). The nature of the parameter variation for these 3 groups is shown in Table 5.4. All variants were simulated for both structure P2 and structure P3.

Tab. 5.4: Calculation variants.

<table>
<thead>
<tr>
<th>Group</th>
<th>Run No.</th>
<th>Parameter variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layout</td>
<td>5 – 15</td>
<td>Distance between borehole arrays</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Angle between radial boreholes</td>
</tr>
<tr>
<td>Hydrogeology</td>
<td>16, 26, 27</td>
<td>$K_{EDZ}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$K_2$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Anisotropy</td>
</tr>
<tr>
<td>Scenario</td>
<td>17 – 27</td>
<td>Length of interval</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Duration of test</td>
</tr>
</tbody>
</table>

5.2.4 Results

Base case

Using the base case, the results of the different models (P1, P2 and P3) are compared with a pure 3D solution in a homogeneous rock matrix (without influence from tunnel).

For this purpose, a Constant Rate Test in the central injection interval (halfway between the tunnel wall and the transition zone = 0.5 m radial distance from the tunnel wall) was simulated for the FE models P1 (homogeneous), P2 (annular) and P3 (elliptic). With the help of the diagnostic plot and the calculated sensitivity coefficients, the characteristic features of the different system reactions are compared and discussed. Figure 5.4 shows the calculated evolution of potential head in the injection interval in a log-log plot (continuous lines) as well as the diagnostic curves according to the derivative method (dashed lines). The corresponding curves are also illustrated for a homogeneous medium of infinite extent without the influence of the tunnel (3D).

Compared with the pure potential evolution, where only small differences can be seen between the model variants, the effects of geometric and hydrogeological structures show up clearly in the diagnostic representation (Figure 5.2). The sole influence of the sealed tunnel can be recognised by comparing the curves for the 3D and homogeneous (model P1) when, after approximately 400 – 1000 seconds (time A), the pressure front has reached the tunnel wall and its resistance slightly reduces the dimensionality of the flow. After flow around the tunnel (~20,000 s), the "space re-opens" and the derivative of the homogeneous model P1 (green curve) once again approaches the pure 3D solution.

Similar behaviour can be observed for the curves of models P2 (annular) and P3 (elliptic) after 1000 s. However, in this case, reaching the lower permeability transition zone (which coincides with time B, Figure 5.4) increases the reduction of the dimensionality. Because of the more concave form of the elliptical model, this effect is significantly greater.
In Figure 5.5 the potential evolution is represented at other key positions in the model. The second curvature of the curves at time C becomes clear; this is most clearly visible for the circular model P2. At this point, the perturbation front has reached the floor of the tunnel and bypassing/flow around the tunnel is thus complete. The excavation disturbed zone has now "filled" in the vertical section and further migration – besides infiltration into the lower permeability transition zone – is largely in an axial direction and flow takes on a more or less linear character.

The evolution with time of the influence of individual parameters on the potential head development in the injection interval is shown in Figures 5.6 and 5.7 for the model variants P2 and P3. The sensitivity coefficients shown in these Figures are all negative, since an increase in the parameter values (e.g. increase in permeability) leads to a decrease in the observed performance measure (here: potential head in the injection interval). The dominant parameter in the system is the permeability of the excavation disturbed zone $K_{EDZ}$; over the entire calculation period its sensitivity far exceeds that of the other model parameters. As expected, the coefficient for well-bore storage $S_{wb}$ has an influence on the head increase in the injection interval only at the beginning of the test, up to around 1000 s. The storage capacity $S_{EDZ}$ of the excavation disturbed zone shows a similar time-limited effect, albeit somewhat delayed.

The influence of the two parameters for the transition zone $K_2$ and $S_2$ on the observed performance measure begins only towards the end of the calculation period when the more highly permeable disturbed zone has already been completely affected by the injection (Figure 5.7). Due to the smaller volume of the EDZ, this effect is more clearly visible in model P3.
Fig. 5.5: Potential evolution at key positions of the model.
Fig. 5.6: Sensitivity coefficient for the base case and model P2.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Observation interval</td>
<td>B1 - 1.3</td>
</tr>
<tr>
<td>Injection interval</td>
<td>B1 - 1.3</td>
</tr>
<tr>
<td>Anisotropy</td>
<td>1.0</td>
</tr>
<tr>
<td>Injection</td>
<td>0.39 ml/min</td>
</tr>
<tr>
<td>Borehole radius</td>
<td>0.05 m</td>
</tr>
<tr>
<td>Borehole angle</td>
<td>90°</td>
</tr>
<tr>
<td>Interval length</td>
<td>0.1 m</td>
</tr>
<tr>
<td>Packer length</td>
<td>0.1 m</td>
</tr>
<tr>
<td>Distance between arrays</td>
<td>1.0 m</td>
</tr>
</tbody>
</table>

Pressure change h and derivative dh/(ln t) [kPa] vs Time [s]
Fig. 5.7: Sensitivity coefficients for the base case and model P3 (injection interval 3).

- Observation interval: B1 - 1.3
- Injection interval: B1 - 1.3
- Anisotropy: 1.0
- Injection: 0.39 ml/min
- Borehole radius: 0.05 m
- Borehole angle: 90°
- Interval length: 0.1 m
- Packer length: 0.1 m
- Distance between arrays: 1.0 m

Pressure change $h$ and derivative $\frac{dh}{d(\ln t)}$ [kPa]

Sensitivity [l]

Time [s]
Single-hole evaluation

The results of the calculation variants can be evaluated in terms of: (a) influence of the injection interval, (b) influence of anisotropy, (c) influence of injection rate, interval length and diffusivity. The most important findings are summarised below:

(a) The calculated derivative curves show similar results, even if injection is carried out in different intervals. The only exception in this respect is injection in the outermost interval. In this case, the immediate vicinity of the transition zone has an influence on the head development from the very beginning, which means that the resulting data contain information on these parameters (K₂, S₂).

(b) The effect of reducing the radial permeability in the excavation disturbed zone and the transition zone by a factor of 10 is illustrated in Figure 5.8 (model P2). Besides a stronger potential reaction, the reduced radial diffusivity has the effect that the influence of the boundaries occurs significantly later. The late spreading of flow is also characteristic and is recognisable from the dropping derivative curve starting after around 10,000 s. At this point, the perturbation front has already advanced over a longer distance along the tunnel axis. Due to the lower radial permeability, however, the bypassing of the tunnel is not yet complete and the disturbed zone is not completely filled. Flow therefore continues to change towards pure 3D behaviour.

(c) The influence of an increased injected volume (factor 2) on the simulation results in the test interval was examined in runs 25 – 27 (see KUHLMANN 1995). To compensate for the unavoidable rise in pressure, the length of the intervals was also varied. Nevertheless, the result is an increase in the rise in potential head by about 30 metres compared to the base case. The influence of some other parameters (S⁢EDZ, S₂) is also enhanced and lasts longer in some cases. As expected, the increase in the permeability of the EDZ and the transition zone increases the flow by around an order of magnitude. The changes in the amplitudes of the calculated head and sensitivity lines are proportional to the ratio of ΔK/ΔQ and compensate one other (run 27).
Fig. 5.8: Single-hole interpretation in case of anisotropy (model P2, Run 16).

- Observation interval: B1 - 1.3
- Injection interval: B1 - 1.3
- Anisotropy: 1.0
- Injection: 0.39 ml/min
- Borehole radius: 0.05 m
- Borehole angle: 90°
- Interval length: 0.1 m
- Packer length: 0.1 m
- Distance between arrays: 1.0 m
Crosshole evaluation

For the evaluation of the simulation results of crosshole tests, the design criteria introduced in section 5.2.1 were analysed. Two borehole arrays, each with 3 boreholes and 5 test intervals, were assumed to be available for the measurements. The potential head development was calculated in each of the total of 30 test intervals and presented as a discrete time series with approximately 60 data points. For each of these $30 \times 60$ performance measures, the partial derivatives were then calculated for the 5 model parameters (sensitivity coefficients) of interest.

Table 5.5 summarises the crosshole evaluation for variant P2. The aim of runs 4 to 15 was to optimise the test geometry (spacing of borehole arrays, angle between two radial boreholes within the array). For these runs, the calculated determinants $D_{\text{COV}}$ of the covariance matrix differ only slightly (half an order of magnitude at the most). Apparently, the information content varies only slightly for the different geometric configurations considered. Although some trends are recognisable (the smaller the angle and the closer the arrays, the higher the information content), no optimal configuration presented itself. Analysis of the condition number $D_{\text{CN}}$ and the eigenvalues, which have more or less the same value for all runs in this group, does not allow any definite conclusions to be drawn either. Inspection of the eigenvalues $E_v$ within one run confirms that the parameters near injection, such as $S_{\text{sw}}, K_{\text{EDZ}}$ and $S_{\text{EDZ}}$, can be estimated with greater certainty (small eigenvalues) than the parameters of the transition zone. In particular, the possibility of identifying the parameter $S_2$ (storage capacity), which, at around -0.7, has the largest eigenvalue must be considered relatively poor.

While for previous runs injection took place in the central test interval between the tunnel and the transition zone, the following runs 17 to 24 investigated the influence of the injection location on the information content of the data. Runs 17 to 20 were simulated with a test duration of 1000 s which is normal for single-hole tests, while runs 21 to 24 were simulated with a test duration of $10^5$ s. As the duration of testing increases, the increase in the information obtained on the model parameters (determinant) becomes clear. This applies in particular to the parameters of the transition zone (decrease in the eigenvalues for $K_2$ and $S_2$). It is also clear that the shorter test duration is sufficient to allow the local permeability of the transition zone to be estimated sufficiently accurately. However, because the increase in information for $K_{\text{EDZ}}$ predominates, the ratio of extreme eigenvalues (condition number $D_{\text{CN}}$) deteriorates for longer test durations. For the given measurement configuration, the location of injection has a relatively small influence on the overall test results ($D_{\text{COV}}, D_{\text{CN}}$). If the parameters of the transition zone $K_2, S_2$ also have to be determined, injection in the outermost interval (B1-1.5) is preferable (eigenvalue $E_{s2}$).

The last two simulations differ from the previous runs in that they have a higher injected volume and a doubled interval length (run 25), as well as changed hydrogeological conditions (runs 26 and 27). As long as the hydrogeological conditions allow a similar (to that investigated previously) development of pressure, a considerable increase in information can be observed ($D_{\text{COV}}$ in 25, 27). All parameters, but especially $K_2$ and $S_2$, can then be determined with greater certainty (eigenvalues). It is interesting that, in this case, the condition number $D_{\text{CN}}$ also becomes more favourable – a development which was previously (runs 4 to 15) observed to show the opposite trend. The test configuration with an interval length of 0.2 m and double injection rate can be evaluated overall in the best way.

The crosshole evaluations of the simulation based on the structure variant P3 do not result in any fundamental new knowledge with respect to the test design. Again it is confirmed that a dense test configuration is best for ensuring the required informative measured data. The only marked difference compared with the P2 results, is a general decrease in the eigenvalues (and the determinants $D_{\text{COV}}$), particularly those of the transition zone parameters ($K_2, S_2$). Due to the
narrowing structure of the permeability model P3 as the angle increases, a series of observation intervals is located within this zone and thus provide an increase in information. As in the permeability model P2, increasing the interval length together with an increase in the injected volume has the best effect on the identifiability of the model parameters.

Tab. 5.5: Calculated design criteria for model P2.

<table>
<thead>
<tr>
<th>Run</th>
<th>(D_{\text{cov}})</th>
<th>(D_{\text{CN}})</th>
<th>(E_{V_{\text{Swh}}})</th>
<th>(E_{V_{\text{Kedz}}})</th>
<th>(E_{V_{\text{Sedz}}})</th>
<th>(E_{V_{K2}})</th>
<th>(E_{V_{S2}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>P2.04</td>
<td>-12.661</td>
<td>-4.185</td>
<td>-2.440</td>
<td>-4.813</td>
<td>-3.037</td>
<td>-1.742</td>
<td>-0.629</td>
</tr>
<tr>
<td>P2.05</td>
<td>-12962</td>
<td>-4.036</td>
<td>-2.459</td>
<td>-4.824</td>
<td>-3.095</td>
<td>-1.797</td>
<td>-0.787</td>
</tr>
<tr>
<td>P2.06</td>
<td>-12.753</td>
<td>-4.021</td>
<td>-2.429</td>
<td>-4.811</td>
<td>-3.017</td>
<td>-1.705</td>
<td>-0.790</td>
</tr>
<tr>
<td>P2.07</td>
<td>-12.687</td>
<td>-4.020</td>
<td>-2.416</td>
<td>-4.811</td>
<td>-2.993</td>
<td>-1.677</td>
<td>-0.791</td>
</tr>
<tr>
<td>P2.08</td>
<td>-12.967</td>
<td>-4.032</td>
<td>-2.462</td>
<td>-4.820</td>
<td>-3.100</td>
<td>-1.798</td>
<td>-0.788</td>
</tr>
<tr>
<td>P2.09</td>
<td>-13.093</td>
<td>-4.046</td>
<td>-2.475</td>
<td>-4.832</td>
<td>-3.147</td>
<td>-1.853</td>
<td>-0.786</td>
</tr>
<tr>
<td>P2.10</td>
<td>-12.879</td>
<td>-4.026</td>
<td>-2.448</td>
<td>-4.817</td>
<td>-3.063</td>
<td>-1.759</td>
<td>-0.791</td>
</tr>
<tr>
<td>P2.11</td>
<td>-12.811</td>
<td>-4.025</td>
<td>-2.437</td>
<td>-4.817</td>
<td>-3.036</td>
<td>-1.730</td>
<td>-0.792</td>
</tr>
<tr>
<td>P2.12</td>
<td>-12.744</td>
<td>-4.023</td>
<td>-2.428</td>
<td>-4.811</td>
<td>-3.015</td>
<td>-1.702</td>
<td>-0.788</td>
</tr>
<tr>
<td>P2.14</td>
<td>-12.662</td>
<td>-4.020</td>
<td>-2.412</td>
<td>-4.809</td>
<td>-2.985</td>
<td>-1.667</td>
<td>-0.790</td>
</tr>
<tr>
<td>P2.15</td>
<td>-12.598</td>
<td>-4.019</td>
<td>-2.397</td>
<td>-4.809</td>
<td>-2.963</td>
<td>-1.639</td>
<td>-0.790</td>
</tr>
<tr>
<td>P2.16</td>
<td>no calculated sensitivities</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P2.17</td>
<td>-9.935</td>
<td>-3.601</td>
<td>-2.622</td>
<td>-4.203</td>
<td>-1.906</td>
<td>-0.602</td>
<td>-0.602</td>
</tr>
<tr>
<td>P2.18</td>
<td>-10.030</td>
<td>-3.579</td>
<td>-2.650</td>
<td>-4.181</td>
<td>-1.993</td>
<td>-0.602</td>
<td>-0.603</td>
</tr>
<tr>
<td>P2.19</td>
<td>-10.108</td>
<td>-3.555</td>
<td>-2.648</td>
<td>-4.163</td>
<td>-1.982</td>
<td>-0.706</td>
<td>-0.608</td>
</tr>
<tr>
<td>P2.20</td>
<td>-10.224</td>
<td>-3.672</td>
<td>-2.728</td>
<td>-4.287</td>
<td>-1.932</td>
<td>-0.663</td>
<td>-0.614</td>
</tr>
<tr>
<td>P2.21</td>
<td>-12.739</td>
<td>-4.256</td>
<td>-2.485</td>
<td>-4.865</td>
<td>-3.050</td>
<td>-1.730</td>
<td>-0.609</td>
</tr>
<tr>
<td>P2.22</td>
<td>-12.666</td>
<td>-4.211</td>
<td>-2.461</td>
<td>-4.824</td>
<td>-3.037</td>
<td>-1.730</td>
<td>-0.614</td>
</tr>
<tr>
<td>P2.23</td>
<td>-12.736</td>
<td>-4.142</td>
<td>-2.421</td>
<td>-4.828</td>
<td>-3.047</td>
<td>-1.755</td>
<td>-0.685</td>
</tr>
<tr>
<td>P2.24</td>
<td>-12.871</td>
<td>-4.275</td>
<td>-2.454</td>
<td>-4.997</td>
<td>-3.125</td>
<td>-1.573</td>
<td>-0.722</td>
</tr>
<tr>
<td>P2.26</td>
<td>-8.482</td>
<td>-2.934</td>
<td>-1.029</td>
<td>-3.640</td>
<td>-1.793</td>
<td>-0.706</td>
<td>-1.313</td>
</tr>
<tr>
<td>P2.27</td>
<td>-14.549</td>
<td>-3.545</td>
<td>-2.234</td>
<td>-5.038</td>
<td>-3.163</td>
<td>-1.493</td>
<td>-2.621</td>
</tr>
</tbody>
</table>
5.2.5 Conclusions

As part of design calculations based on an EPM modelling approach, two structural alternatives of an excavation disturbed zone (models P2 and P3) were investigated. The model variants, which are derived from rock mechanical hypotheses and earlier seismic measurements in the Grimsel tunnels, differ in terms of their structure (one annular, one elliptical). Both models are characterised by a disturbed core zone (maximum thickness 1 m). This is followed by a circular, slightly disturbed zone, which forms the transition to the intact rock matrix.

The complex geometric configuration required a three-dimensional simulation of the transient hydrotests. This was carried out using the finite element method. Making use of its symmetric properties, the FE grids were created individually for each design variant. A procedure specially developed for the EDZ project made the grid construction fully automatic.

The evaluation of the design variants followed a two-sided strategy. On the one hand, the influence of the different model structures was visualised using diagnostic representations (derivative plots). To complement this, calculation of transient sensitivity coefficients allowed the influence of the involved hydrogeological parameters to be quantified at each time point in the simulation. By the means of experiment design methods, the investigated design variants were evaluated in terms of their suitability for later parameter estimates in a second phase. The design criteria used can be calculated based on the covariance matrix of the involved model parameters and a subsequent eigenvalue analysis.

For the actual design calculations (Constant Rate Tests), a base case was first defined. This represents existing hydrogeological knowledge and the reference values for the technical design parameters. The 23 selected calculation variants aimed to optimise the geometric configuration of a crosshole test on the one hand, and to estimate the influence of the hydrogeological conditions (diffusivity, anisotropy) and the hydrotest conditions (volume injected, injection interval) on the other.

Evaluation of the calculated evolution of the hydraulic head in the injection interval in the form of derivative curves (single-hole evaluation) indicated that the results depended to a significant extent on the assumed form of the excavation disturbed zone (circle, ellipsis). A further comparison with homogeneous hydrogeological conditions, with and without the influence of a sealed tunnel, allows the different phases of flow during a Constant Rate Hydrotest to be recognised. This helps to make clear the dimensionality of the flow:

a) Transition from well-bore storage-dominated to ideal (3D) flow in the initial phase of the test (positive curvature, followed by dropping curve)

b) Arrival of the pressure front at the tunnel boundary or less permeable zones (positive curvature = reduction of dimensionality)

c) Flow bypassing the tunnel beyond the roof of the tunnel cross-section (negative curvature = reduction of dimensionality)

d) Junction of flow after bypass of tunnel completed (positive curvature, transition to more linear behaviour)

The form of the excavation disturbed zone (qualitative trend of cross-sectional surface area, constant or decreasing) is visualised using the slope of the derivative curve between phases (b) and (d).
Further conclusions from the different calculation variants can be summarised as follows:

- The form of an adjacent boundary (geometric or less permeable) can be estimated from the curvature of the derivative curve: positive = convex, negative = concave.

- A hydrogeological anisotropy (higher axial permeability) seems to be visible from a relatively late and intensified drop of the derivative curve during the flow (by-passing) round the tunnel. The performed simulations do not allow a complete evaluation of the influence of the anisotropy.

- An increase in diffusivity of the excavation disturbed zone does not change the characteristic features of the curves. It only causes a shift in time.

The calculated, transient sensitivity curves quantify the influence of the model parameters of interest on the development of potential head in the injection interval. They all show the dominant influence of the permeability of the disturbed zone $K_{\text{EDZ}}$ throughout the entire duration of the test. The parameters for storage capacity $S_{\text{wb}}$ and $S_{\text{EDZ}}$ show influences which are limited in time. Their maximum can be found during phase (b). The parameters of the transition zone $K_2$ and $S_2$ are only of secondary importance within the test periods considered here and are barely identifiable in this way.

For the evaluation of crosshole tests, the pressure reactions in the observation intervals (with the corresponding diagnostic plots) are available in addition to the information in the injection interval discussed above (see appendix KUHLMANN 1995). For the analysis, basically the same features have to be observed. Beyond this, the individual design variants were analysed, using the described design criteria, in terms of their information content for the subsequent parameter estimate. Basically, both structure variants (P2 and P3) investigated gave a similar picture. The results obtained, with their implications for the experimental test design, can be summarised as follows:

- A dense crosshole test configuration, with which marked test reactions can be measured, best fulfils the requirement for informative data. For the hydrogeological conditions investigated here, this implies an angle of approximately 15 – 20° between the individual radial boreholes. A second array of boreholes should not be installed more than a metre away from the first.

- Increasing the injected volume basically has a positive effect on the measurable information of a Constant Rate Test. Because a limit on the hydraulic head (about 50 m) has to be observed and the borehole diameter is already fixed at its maximum value, an increase in flow has to be compensated by extending the interval length. Therefore, the suggestion was to double the originally planned length of 0.1 to 0.2 m.
5.3 Fracture network design calculations

This section describes design calculations performed prior to the experiments using a discrete fracture network model of the natural and damaged zone fractures.

5.3.1 Aims of design calculations using discrete fracture network models

The fracture network modelling was used to investigate the consequences of the expected heterogeneity in hydraulic properties around the tunnel. The work used the NAPSAC discrete fracture network code (NAPSAC 1991).

In particular the aims of the design calculations were:

- to assess the impact of a fracture network representation of the EDZ on the testing approach
- to assess the variability of the hydraulic conductivity to be expected for different representations of the natural and induced fracture system
- to estimate the extent of hydraulic testing necessary to adequately characterise the geometry of the EDZ and its hydraulic properties
- to provide recommendations for the hydraulic test programme.

This work was complementary to the continuum modelling described earlier in that the major emphasis is on considering the likely range of hydraulic behaviour across multiple tests rather than optimisation of test procedure for a single test. It also draws on the work described in section 4.2 in that the model of damaged zone fracturing is based on the stress model predictions for the WT drift.

5.3.2 Model of the matrix and natural fracture system

Inflows to boreholes and tunnels at the GTS have typically been associated with the natural fracture system. Some of the most important flowing features have been associated with larger scale features such as shear zones (so-called K and S zones) and lamprophyres. Very detailed studies of flow and transport have been performed within such zones (see BOSSART & MAZUREK 1991, FRICK et al. 1992 etc.).

However, some component of total flow is associated with the background natural fracture system outside such structures. Results from the Ventilation Drift (VOMVORIS & FRIEG 1992b) suggest that the majority of flow into the drift was associated with a major shear zone. Hydraulic tests away from the shear zone showed that the hydraulic conductivity of the matrix and background fractures varied between $4 \times 10^{-12}$ and $1 \times 10^{-10}$ m/s. In this study a bulk conductivity of $1 \times 10^{-11}$ m/s was assumed. It was assumed that the matrix conductivity was relatively constant with a value of $1 \times 10^{-12}$ m/s.

A model of the background fracture system was developed by LANYON (1995) on the basis of experience in the GTS. The model was based on four sets of fractures (called S, S3, K and sub-horizontal). The model assumed typical values of fracture frequency for the GTS, although there was the possibility that the area around the WT drift was in fact somewhat less fractured. The properties of the four sets are given in Table 5.6. It was assumed that fractures were of uniform transmissivity within every set. More detailed "channelled" models were not developed because of lack of characterisation data.
### Tab. 5.6: Parameters of the GTS natural fracture network (LANYON 1995).

<table>
<thead>
<tr>
<th>FRACTURE SETS</th>
<th>S</th>
<th>S3</th>
<th>K</th>
<th>SUB-HORIZONTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percentage of total number of fractures</td>
<td>40%</td>
<td>8%</td>
<td>32%</td>
<td>20%</td>
</tr>
<tr>
<td>Density of fracture centres/m³</td>
<td>0.00437</td>
<td>0.00087</td>
<td>0.00349</td>
<td>0.00121</td>
</tr>
<tr>
<td>Total fractures in 50 × 50 × 50 m cube</td>
<td>546</td>
<td>109</td>
<td>437</td>
<td>152</td>
</tr>
<tr>
<td>Mean azimuths</td>
<td>145</td>
<td>185</td>
<td>230</td>
<td>0</td>
</tr>
<tr>
<td>Spread in azimuths</td>
<td>30</td>
<td>13</td>
<td>25</td>
<td>180</td>
</tr>
<tr>
<td>Mean dip</td>
<td>90</td>
<td>80</td>
<td>90</td>
<td>0</td>
</tr>
<tr>
<td>Spread in dip</td>
<td>25</td>
<td>10</td>
<td>20</td>
<td>25</td>
</tr>
<tr>
<td>TE length</td>
<td>Log normal with mode of 5m and ( \pm 2\sigma ) of about 14 m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean transmissivity m²/s</td>
<td>( 10^{-10} )</td>
<td>( 10^{-10} )</td>
<td>( 5 \times 10^{-10} )</td>
<td>( 10^{-10} )</td>
</tr>
<tr>
<td>Spread of transmissivity</td>
<td>Log normal distribution with ( \sigma = \ln(2) ) S, S3 and sub-horizontal sets, the ( \pm 2\sigma ) range covers a factor of 16 from ( 4 \times 10^{-10} ) to ( 0.25 \times 10^{-10} )</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Two variants of the natural fracture system model were developed. **Variant 1** used the fracture frequency estimated by P. Bossart (see LANYON 1995), while **Variant 2** used a network with a fracture density quadruple that of **Variant 1**. The fracture transmissivities used in both variants were calibrated to give an effective hydraulic conductivity of \( 1 \times 10^{-11} \) m/s.

The storativity associated with each fracture was set at \( 1 \times 10^{-6} \). As storativity was constant for all fractures within the models, this resulted in a range of diffusivities for the natural fractures from about \( 1 \times 10^{-3} \) to \( 1 \times 10^{-5} \) m²/s.

### 5.3.3 Damaged zone fracture network

As the models were developed prior to drilling the boreholes, no characterisation data from the damaged zone fractures was available. Some results from tunnel mapping were available.

In order to develop a plausible network model for the damaged zone fractures, the stress interpolations from KONIETZKY (1995) were used to predict the fracturing using a failure criterion due to STACEY (1981), and an analytic solution to the stress around a circular opening in a general stress field (HAYES 1965).

Figure 5.9 shows the stresses and strains calculated from the LEEMAN-HAYES analytical solution for stress around a circular opening. The STACEY failure criterion (STACEY 1981) is based on the minimum principal strain. STACEY suggests that micro-fracturing will occur at strains less than \(-200 \mu \text{strain} \) (where tensile strains are negative). Macro-fracturing may only occur at less than \(-400 \mu \text{strain} \). However, for this work an envelope for damaged zone fractures was set at the \(-200 \mu \text{strain} \) contour as shown in Figure 5.8.
Fig. 5.9: Principal stresses and strain from the analytical model of HAYES (1965).

- a) Maximum principal stress magnitudes around the tunnel
- a) Minimum principal stress magnitudes around the tunnel
- c) Minimum principal strain magnitudes around the tunnel
The fracture orientation was taken to be normal to the minimum principal stress direction. Fracture transmissivity was set as a function of the strain so that it decreased away from the tunnel wall. The damaged zone fractures were chosen to be 1 m², with a mean spacing of 20 cm.

Two damaged zone fracture model variants were considered, one based on the interpolated stress field and a second version where an isotropic stress field is assumed with the stress magnitude taken to be that of the estimated maximum principal stress from the interpolated stress field. These two model variants correspond to ideas proposed in VOMVORIS et al. (1993). Figure 5.10 shows realisations of the two different damaged zone fracture networks.

5.3.4 Hydraulic test simulations

The geometry of the models used consisted of an annular region made up of twelve segments. The outer radius of the annulus was chosen as 12 m (approximately 6 tunnel radii) and the inner radius was 1.85 m, which corresponded to the tunnel wall. The boundary conditions applied to the model faces were such that the inner surface was assumed to be a no-flow boundary because
of the resin applied to the surface. All other model surfaces were assumed to be constant head boundaries. This is similar to boundary conditions used in the equivalent porous medium models. Note that the symmetries assumed in the continuum models are no longer valid in the stochastic models.

Initially steady-state flow constant pressure injection simulations were run to determine the likely range of flows from an injection at 500 kPa and the range of crosshole responses expected. Twenty realisations of steady-state injections from 12 of the 20 borehole intervals were performed for three different models:

- Natural fracture network only
- Natural fracture network (high density) only
- Natural fracture network plus anisotropic damaged zone network

In addition 20 realisations of injections from 3 borehole intervals were performed for

- Natural fracture system plus isotropic damaged zone network

The smaller number of intervals was chosen because of the lack of dependence on position round the tunnel for the isotropic damaged zone network.

Sample realisations of the four models are shown in Figure 5.11. The borehole array used within the NAPSAC design calculations was similar to that used by KUHLMANN (1995) and is shown in Figure 5.12. Each of the four boreholes was assumed to be configured with five 10 cm packers separating 10 cm intervals of hole. Boreholes were numbered clockwise from the horizontal borehole. Intervals within each borehole were numbered outwards from the innermost zone. This numbering scheme is different from that finally used at the experimental site.

The results from the steady-state simulations are given in Table 5.7. A small number of intervals intersect a fracture but the fracture itself is not connected to the model boundaries and so has zero flow.

Tab. 5.7: Results from steady-state simulations of 500 kPa injections.

<table>
<thead>
<tr>
<th></th>
<th>Fraction of tested intervals intersecting a fracture</th>
<th>Fraction of tested interval with steady-state flow</th>
<th>Range in flow [ml/min]</th>
<th>Average number of monitoring intervals ≥ 10 kPa pressure response when testing a flowing interval</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Natural fractures only</td>
<td>6/240</td>
<td>5/240</td>
<td>0.22 – 1.02</td>
<td>1.5</td>
</tr>
<tr>
<td>(b) Natural fractures (high density) only</td>
<td>13/240</td>
<td>13/240</td>
<td>0.07 – 0.15</td>
<td>0.7</td>
</tr>
<tr>
<td>(c) Natural fractures + anisotropic damaged zone</td>
<td>71/240</td>
<td>61/240</td>
<td>0.03 – 1.96</td>
<td>3.2</td>
</tr>
<tr>
<td>(d) Natural fractures + isotropic damaged zone</td>
<td>15/60</td>
<td>13/60</td>
<td>0.05 – 2.55</td>
<td>3.3</td>
</tr>
</tbody>
</table>
The lower typical transmissivity of the high density model is reflected in the low flow rates. Otherwise the highest flow rates are similar for all the models. The addition of the damaged zone fractures increases the number of flowing intervals, but some very low transmissivity damaged zone fractures do not allow appreciable flows.

![Diagram showing transmissivity in different models](image)

**Fig. 5.11:** Sample realisation from different models for which steady-state and transient well tests were simulated.

Also the number of pressure responses observed (usually in the same hole) is larger in the models with damaged zone fractures, as might be expected.

The results from the steady-state tests were used to choose a single flow rate for Constant Rate Test simulations. For all models with the standard natural fracture system, this was chosen to be 0.4 ml/min and for models with the high density of natural fractures this was set at 0.04 ml/min. The value of 0.4 ml/min was chosen for compatibility with the Constant Rate Tests simulated by KUHLMANN (1995).
Constant Rate Tests were performed on the subset of flowing realisations from the same 20 used within the steady-state models. From the results of these models log-log diagnostic plots were created. For a small subset of these simulations the pressure time histories were analysed using a sophisticated well test analysis package PanOil™ (Pan System Version 2.2; Edinburgh Petroleum Services Ltd.) The results from the transient solutions show a wide range of behaviours and are tabulated below (Table 5.8).

In general the behaviour observed in the log-log plots is that which might be expected. Intervals in the most transmissive parts of the damaged zones typically show increasing gradients with time, while those on the edge or in the middle of the damaged zone show a variety of behaviour presumably dependent on the local connectivity. Borehole intervals outside the damaged zone show responses that seem to relate to the natural fracture network. It is possible that the early time behaviour observed for simulations where a natural fracture is intersected is related to the relatively coarse discretisation of the larger natural fractures. Several simulated tests showed extended periods of storage-type behaviour with unit gradient indicating flow into poorly connected fractures or clusters of fractures.

Crosshole responses are observed in some tests, but typically only after 1000 seconds or more (in some case 10,000 seconds). Amplitudes of crosshole effects are less than 100 kPa after 100,000 seconds.

For three simulations the results were analysed using the PanOil well test analysis package. In general the transmissivities derived from the analyses were consistent with the natural and damaged zone fracture transmissivities. Dimensions inferred from the analysis were also consistent with the sizes of fractures within the models, but the value of storativity used in the model had been provided to the analyst and so uncertainties in the storage behaviour of the fracture system have not been considered.
### Tab. 5.8: Summary of transient model behaviour.

<table>
<thead>
<tr>
<th>Model</th>
<th>Borehole intervals</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Natural fractures only</td>
<td>All intervals</td>
<td>Most tests show extended radial flow with some suggestion of early time linear flow or storage (may be related to discretisation of larger features)</td>
</tr>
<tr>
<td>(b) Natural fractures (high density) only</td>
<td>All intervals</td>
<td>Early time storage with late (10,000 s) transition to radial flow, delay in transition probably related to lower diffusivities of the higher density network</td>
</tr>
<tr>
<td>(c) Natural fractures + anisotropic damaged zone</td>
<td>BH 1.1, 1.3, 1.5</td>
<td>Does not intersect damaged zone fractures, so results typical of natural fracture system (see (a))</td>
</tr>
<tr>
<td></td>
<td>BH 2.1</td>
<td>Early time radial followed by increasing gradient</td>
</tr>
<tr>
<td></td>
<td>BH 2.3</td>
<td>Early time radial followed by variability post 100 s with both increasing and decreasing gradient</td>
</tr>
<tr>
<td></td>
<td>BH 2.5</td>
<td>Early time radial followed by variability post 100 s with both increasing and decreasing gradient</td>
</tr>
<tr>
<td></td>
<td>BH 3.1</td>
<td>Early time radial usually followed by increasing gradient</td>
</tr>
<tr>
<td></td>
<td>BH 3.3</td>
<td>Early time radial usually followed by increasing gradient post 1000 s both increasing and decreasing gradient</td>
</tr>
<tr>
<td></td>
<td>BH 4.1, 4.3, 4.5</td>
<td>Early time radial usually followed by variable gradient</td>
</tr>
<tr>
<td>(d) Natural fractures + isotropic damaged zone</td>
<td>BH 1.1</td>
<td>Initial radial flow, followed by increasing gradient, greater variability in late time</td>
</tr>
<tr>
<td></td>
<td>BH 1.3</td>
<td>Initial radial flow followed by either increasing or decreasing gradient</td>
</tr>
</tbody>
</table>

Figure 5.13 shows a detail of the fractures intersecting the open borehole intervals in one realisation of model c. Both the fractures that intersect an interval and those connected to such fractures are shown. A fracture with transmissivity of $3 \times 10^{-11}$ m²/s intersects the borehole interval (shown in blue), and then intersects a larger feature with transmissivity of $1 \times 10^{-9}$ m²/s very close to the borehole (shown in green).

Figure 5.14 shows the PanOil log-log plot and radial flow plot. The analysis indicated a composite system with an inner transmissivity of $2 \times 10^{11}$ and an outer transmissivity of $8 \times 10^{-10}$ m²/s, which is in very good agreement with the fractures in the model. The PanOil analysis suggested that the distance to the change in transmissivity was 0.16 m from the borehole; again this appears to be in good agreement with model geometry.

However, it should be emphasised that such analyses are only possible when a good understanding of the storage properties is available and when well-bore storage is minimal. The version of NAPSAC used in these simulations did not include any well-bore storage.
Fig. 5.13: Fracture intersections with open intervals in one realisation.
Fig. 5.14: Sample log-log diagnostic plots from a simulation with anisotropic damaged zone fracture network and natural fracture network.

Model results:
Radial composite
Infinitely acting
\( K = 1.888 \times 10^{-17} \text{ m}^2 \)
\( k_h = 1.888 \times 10^{-18} \text{ m} \cdot \text{m}^2 \)

Wellbore storage not defined due to eliminated early data.
Bilinear flow diagnosed in early time.
Linear flow not apparent.
Composite radial flow model diagnosed for middle and late time behaviour.
5.3.5 Conclusions

The modelling performed was part of the design process for the EDZ testing and fed into the development of the test procedures. The major conclusions of the work were:

- Typical inflow rates that might be expected for the testing are in the range 0.01 to 2.0 ml/min. In some matrix-like intervals even lower flows of about 0.002 ml/min might be expected.
- Interval transmissivity will be variable and some form of Constant Head or Pulse Pre-test will be needed to determine interval transmissivity prior to the planning of any constant rate testing.
- Constant rate testing can determine the local transmissivity geometry but the connectivity of the fractures will produce variability of responses in late time.
- Crosshole results will be limited and only occur post 1000 seconds.
- Sophisticated well test analysis can determine fracture geometry if wellbore storage is small and fracture storage properties are understood.

Integration of geological data from the boreholes was believed to be an important part of any test design and the following programme of tests (see Table 5.9) was suggested depending on the model that was thought to best fit the borehole observations.

Tab. 5.9: Recommended test procedures for each geological model.

<table>
<thead>
<tr>
<th>Model from Observation</th>
<th>Test procedure</th>
<th>Number of tests</th>
<th>Benefit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sparse natural fractures with no damaged zone fractures</td>
<td>Pre-test with 0.5 m interval lengths using Constant Head; perform Constant Rate testing on most transmissive intervals</td>
<td>6 × Constant Head Injection Tests (HI; 0.5 m intervals) per borehole; 1 – 5 × Constant Rate Injection Test (RI) in total</td>
<td>Demonstration of no significant hydraulic EDZ and characterisation of natural fracture system</td>
</tr>
<tr>
<td>Dense natural fractures with no damaged zone fractures</td>
<td>Pre-test with 0.25 m interval lengths using Constant Head; perform Constant Rate testing on most transmissive intervals</td>
<td>12 × HI (0.25 m intervals) per borehole; 5 – 10 × RI in total</td>
<td>Demonstration of no significant hydraulic EDZ and characterisation of natural fracture system</td>
</tr>
<tr>
<td>Natural fracture system + damaged zone fractures</td>
<td>Test all 0.1 m intervals with Constant Head; Constant Rate testing on most transmissive intervals</td>
<td>20 × HI (0.1 m intervals); 1 – 5 × RI in total</td>
<td>Small-scale transmissivity distribution local to borehole intervals; larger scale EDZ geometry from RI test analysis and crosshole data</td>
</tr>
</tbody>
</table>
6 HYDRAULIC TESTING

Field characterisation of the hydraulic properties of the EDZ was conducted using the Modular Mini-Packer System (MMPS). The MMPS tool was developed as part of the EDZ experiment (ADAMS, 1996a). Development of the tool and the methods and results of the field testing are described in the following sections.

6.1 Development of test equipment

6.1.1 Design criteria

The MMPS was developed by Solexerts AG based on the following design specifications defined by Nagra:

- Borehole diameter: 50 mm
- No. of packers: 6
- No. of test intervals within 1 m: 5
- Test interval length min: 10 cm
- Standard packer length: 10 cm
- Guard packer length: 100 cm
- Maximum working pressure: 15 bar
- Interval instrumentation: 1 pressure measurement line
  1 injection/withdrawal line
- Maximum flow rates: 50 ml/min

6.1.2 System description

To fulfil the given requirements and to maximise system flexibility, a modular packer system was developed that allows up to 6 individual packer modules to be coupled in a variety of configurations (Figure 6.1). Each packer module is a stand-alone unit with a packer inflation line, an interval flow line and a pressure measurement line.

Technical specifications of the components are provided in Table 6.1. Three complete systems were produced, allowing instrumentation of three separate boreholes with 6 packers each.

Each packer module consists of three components: a mandrel, a packer element and control lines (Figure 6.1). The packer element slides directly onto the mandrel. O-rings between the packer element and mandrel provide a seal for packer inflation. Packer elements can be replaced in case of packer damage or failure.

The mandrels are coupled together by sliding the O-ring-protected male end into the larger female end and tightening down three set screws (Figure 6.1). The minimum spacing between packers is 10 cm. The interval length can be increased by inserting a spacer between two packer modules.
Fig. 6.1: Structure of the Modular Mini-Packer System (MMPS) for the EDZ project (FRIEG et al. 1996).

Tab. 6.1: Technical specifications for components of the Modular Mini-Packer System.

<table>
<thead>
<tr>
<th>Component</th>
<th>Material</th>
<th>Max. O.D. [mm]</th>
<th>Min. I.D. [mm]</th>
<th>Max. pressure [bar]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mandrel</td>
<td>304 Stainless Steel</td>
<td>45</td>
<td>22</td>
<td>-</td>
</tr>
<tr>
<td>Packer Inflation Line</td>
<td>304 Stainless Steel</td>
<td>4</td>
<td>2</td>
<td>400</td>
</tr>
<tr>
<td>Pressure and Injection / Withdrawal Lines</td>
<td>304 Stainless Steel</td>
<td>4</td>
<td>3</td>
<td>200</td>
</tr>
<tr>
<td>Packer Element</td>
<td>Steel Reinforced</td>
<td>46</td>
<td>35</td>
<td>60</td>
</tr>
<tr>
<td>Expansion Material</td>
<td>Natural Rubber</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Packer Element Ends</td>
<td>304 Stainless Steel</td>
<td>48</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

O.D. = Outer Diameter, I.D. = Inner Diameter
Three 4 mm (outer diameter) stainless-steel lines are soldered directly onto each mandrel for:

- packer inflation/pressure measurement
- interval pressure measurement and
- fluid injection/withdrawal.

An end cap with 2 control lines is fitted on the inner-most packer for isolating and monitoring the zone between the end of the borehole and the packer system. Stainless steel "Quick Couplers" that slide on or off the 4 mm control lines are used to attach to the surface control units, thus maintaining the integrity of the lines for re-use.

After the use of the MMPS equipment in the framework of the EDZ experiment at the GTS the tool was also successfully used for the hydraulic characterisation of the EDZ at the Mont Terri rock laboratory (Switzerland) in a clay environment (ADAMS & GEMPERLE, 1997). For flexible use of the MMPS in different environments, the rubber of the packers was changed to a more flexible material to be able to use the MMPS also in boreholes with a non-perfect caliper (e.g. enlarged diameter) and/or with little breakouts. The hazard of packer bypass during hydraulic testing was thus minimised and perfect sealing could also be achieved with lower packer pressure in lower strength and softer rock.

Nowadays the MMPS can be flexibly used in boreholes from 50 mm up to a diameter of 65 mm (original 55 mm) with a maximum working pressure of 40 bars. The equipment could also be used at various locations worldwide for the characterisation of the EDZ, e.g. Rokkasho test site (JNFL / Japan; 2005), Chungju test site (KIGAM / Korea; 2007), etc.

6.1.3 System performance

The 10 cm packer elements were subject to a series of laboratory tests in order to characterise performance properties such as packer diameter as a function of packer pressure, packer contact length when inflated in a 50 mm diameter test pipe and system stiffness (compressibility).

For practical purposes, the maximum inflation diameter is 55 mm. The 10 cm packer elements exhibit about 5 mm shortening when inflated within a 50 mm test pipe. Contact length of the element at this diameter is practically 100 % (i.e. 9.5 cm).

System stiffness was investigated by conducting a series of 1 bar compressibility Pulse Tests in the interval between two packers at various packer inflation pressures. For packer pressures greater than 10 bar, the system compressibility values range from $6 \times 10^{-8}$ to $7.5 \times 10^{-8}$ Pa$^{-1}$. Below 10 bar packer pressure, the system compressibility increases significantly.
6.2 Field testing

The EDZ hydraulic testing, which was conducted between November 1995 and July 1996, consisted of three phases (ADAMS 1996b):

- Phase I: Pre-test Monitoring / November 15, 1995 – January 9, 1996
- Phase II: Active Testing / January 9, 1996 – February 16, 1996

The Pre-test Monitoring was conducted with the objective of reaching stable pressures in the test intervals prior to initiating active hydraulic testing.

The Active Testing Phase comprised a series of "screening tests" (Constant Head and/or Pulse Tests) in all of the intervals to provide an overview of the distribution of hydraulic characteristics within boreholes EDZ95.001, EDZ95.002 and EDZ95.003. Based on the results of the screening tests, three intervals were chosen in which to perform Constant Rate Tests for more detailed characterisation.

Following the Active Testing Phase, the monitoring system was left in place to document the long-term pressure reactions within the test site.

6.2.1 Borehole and interval configuration

Three of the four boreholes (EDZ95.001, EDZ95.002 and EDZ95.003) were instrumented with 6-packer MMPS tools (Figure 6.2).

Fig. 6.2: Schematic borehole and packer configuration for the EDZ project.
Borehole EDZ95.004 was instrumented with a single MMPS module. 6-packer MMPS tools were configured to isolate five 10 cm intervals within the first metre away from the tunnel wall. The 1 metre guard packer was placed from 1 to 2 m depth to minimise the effects of any non-EDZ features that may be present near the tunnel. The boreholes and all pressure and flow lines were saturated with water prior to final expansion of the packers.

6.2.2 Surface equipment and data acquisition

Surface equipment used for hydraulic testing includes a pressure vessel and a variety of flow-control and flow-measurement devices. The surface equipment set-up for various test types is depicted in Figure 6.3.

Data acquisition was conducted using the Solexperts GeoMonitor System in combination with a Fluke Helios A/D converter.
Fig. 6.3: Surface equipment set-up for hydraulic testing.
6.2.3 Testing and interpretation procedures

The hydraulic testing methods applied include Pulse Tests, Constant Head Tests, Constant Rate Tests and Pressure Recovery Tests. A summary of the analysis methods applied in the EDZ project is presented in Table 6.2. Descriptions of the various test types and standard analysis techniques are also readily available in existing Nagra documents, such as ADAMS & WYSS (1994).

<table>
<thead>
<tr>
<th>Test type</th>
<th>Technical reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pulse Test</td>
<td>BREDEHOEFT &amp; PAPADOPULOS (1980)</td>
</tr>
<tr>
<td>Constant Head Test</td>
<td>JACOB &amp; LOHMAN (1952)</td>
</tr>
<tr>
<td>Transient</td>
<td>Steady-State Approximation</td>
</tr>
<tr>
<td>Constant Rate Test</td>
<td>COOPER &amp; JACOB (1946)</td>
</tr>
<tr>
<td>Transient</td>
<td>Steady-State Approximation</td>
</tr>
<tr>
<td>Pressure Recovery after Constant Head / Constant Rate Tests</td>
<td>AGARWAL (1980), HORNER (1951)</td>
</tr>
</tbody>
</table>

6.2.4 Pressure responses during the entire experiment

The pressure responses over the entire test period are presented in Figures 6.4a-d. With respect to pressures at the end of the Post-Testing Phase, pressures generally increase with distance from the tunnel wall.

Pressures in intervals nearest the tunnel range from atmospheric (81 kPa) to 91 kPa. Interval EDZ95.002-2 exhibits a pressure lower than the pressures observed in intervals EDZ95.002-3 and EDZ95.002-4. The pressure distribution in EDZ95.002 indicates that interval 2 is in hydraulic connection with the tunnel wall via a highly conductive feature.

The pressures in intervals farthest from the tunnel wall are 208 kPa in EDZ95.001-1, 262 kPa in EDZ95.003-1 and 478 kPa in EDZ95.002-1. The relatively lower pressures observed in EDZ95.001-1 and EDZ95.003-1 are logical because of the close proximity of these intervals to the main laboratory tunnel (Figure 6.2) and the fact that they are situated above the bottom of the tunnel.

The pressure fluctuations observed in EDZ95.002-1 are related to the testing activities in the Fracture System Flow Test (BK), which was conducted during Phase I to III of the GTS investigation programme (PAHL et al. 1992). These pressure responses are primarily related to testing activities at the BK site.

A cursory comparison of the data from the two projects indicates a negative reaction in interval EDZ95.002-1 to the activities at the BK site (i.e. increase in pressure at BK results in a decrease in pressure at EDZ). The pressure reactions are most likely due to an elastic hydro-mechanical coupling.
Fig. 6.4: Interval pressures during the entire testing period.
6.2.5 Screening tests

Constant Head Injection and/or Pulse Tests were conducted in all intervals to provide an overview of the distribution of hydraulic properties. The tests were designed to provide results for all 18 intervals within a relatively short period of time.

Short-term Constant Head Tests of 1 to 2 hours duration were conducted in 15 test intervals. Injection pressures for the Constant Head Tests were approximately 5 bar above static pressure. Flow rates as low as 0.005 ml/min were measured using the water-column flow meter. Both straight-line and steady-state approximation analyses were conducted, where possible.

Pulse Tests were performed in 5 test intervals: EDZ95.001-1, EDZ95.001-2 and EDZ95.001-5, EDZ95.002-1 and EDZ95.003-2. The screening test results indicate that all but four of the tested intervals have hydraulic conductivity values between $3 \times 10^{-12}$ m/s and $3 \times 10^{-11}$ m/s (Table 6.3).

The four intervals exhibiting higher values are discussed below:

- Intervals EDZ95.002-5 and EDZ95.002-6 reacted simultaneously and exhibited a relatively high hydraulic conductivity ($1 \times 10^{-8}$ m/s), indicating that the intervals were hydraulically interconnected.
- Interval EDZ95.002-2 exhibits a high hydraulic conductivity; this is consistent with the pressure build-up response, which indicates connection with the tunnel through a high-permeability feature (possibly via a high-conductivity feature along the adjacent lamprophyre). The constant head test performed in this interval provides only qualitative results because it was not possible to maintain a constant pressure in the high-conductivity interval.
- The short-term test indicates a hydraulic conductivity value of $5 \times 10^{-10}$ m/s for interval EDZ95.003-5. This value is approximately 1 order of magnitude greater than those of the neighbouring intervals.

6.2.6 Constant Rate Tests

Constant Rate Tests were planned for intervals EDZ95.002-2, EDZ95.002-5 and EDZ95.002-6 and EDZ95.003-5. The objective of the constant rate testing was to provide field data for evaluating flow model characteristics of the EDZ at the GTS.

Borehole EDZ95.003, Interval 5

A Constant Rate Test was performed in interval EDZ95.003-5 using an injection rate of 2 grams/hour. The pressure response during the first 90 hours of the test was quite irregular, with sudden pressure decreases and increases (Figure 6.5). During this period, the flow rate remained steady. After approximately 90 hours, the pressure suddenly increased to nearly 11 bar (the maximum pressure set for the pressure vessel). At this point, the test essentially transformed into a Constant Head Injection Test. The flow rate decreased from 2 g/h to a final rate of 1.4 g/h. Cross-interval reactions were clearly observed in the neighbouring intervals. An explanation for the observed pressure reaction is not inherently obvious. The unsteady pressure build-up during the constant rate portion of the test may be related to the opening of microfractures during injection, to complex multi-phase conditions near the interval, or to equipment effects. Equipment problems, such as leakage to an adjacent interval or through a coupling in the injection line, are not likely based on the fact that the pressure eventually increased to over 10 bar and no increase in flow rate was observed.
Tab. 6.3: Summary of the EDZ hydraulic testing results.

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Interval No.</th>
<th>Best-Estimate K [m/s]</th>
<th>Hydraulic Test Phases and Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>EDZ95.001</td>
<td>1</td>
<td>$3 \times 10^{-12}$</td>
<td>PI</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>$8 \times 10^{-12}$</td>
<td>PI</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>$1 \times 10^{-11}$</td>
<td>HI</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>$1 \times 10^{-11}$</td>
<td>HI</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>$2 \times 10^{-11}$</td>
<td>HI, PI</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>$2 \times 10^{-11}$</td>
<td>HI</td>
</tr>
<tr>
<td>EDZ95.002</td>
<td>1</td>
<td>$2 \times 10^{-12}$</td>
<td>HI, PW</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>$3 \times 10^{-7}$</td>
<td>(HI), RI could not maintain a constant pressure in the test interval during the HI because of friction loss in the injection line</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>$8 \times 10^{-12}$</td>
<td>HI</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>$8 \times 10^{-12}$</td>
<td>HI</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>$1 \times 10^{-8}$</td>
<td>HI, RI Interval 6 reacted instantaneously (interval length of 0.3 m used for calculating K)</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>$(1 \times 10^{-9})$</td>
<td>(HI, RI) not tested separately from interval 5</td>
</tr>
<tr>
<td>EDZ95.003</td>
<td>1</td>
<td>$2 \times 10^{-12}$</td>
<td>HI</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>$3 \times 10^{-11}$</td>
<td>PI, HI</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>$1 \times 10^{-11}$</td>
<td>HI</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>$1 \times 10^{-11}$</td>
<td>HI</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>$4 \times 10^{-10}$ (inner zone) $3 \times 10^{-11}$ (outer zone)</td>
<td>HI, long-term RI/HI</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>$3 \times 10^{-11}$</td>
<td>HI</td>
</tr>
</tbody>
</table>

PI: Pulse Injection
PW: Pulse Withdrawal
HI: Constant Head Injection
RI: Constant Rate Injection
Fig. 6.5: Plot showing pressure responses (intervals EDZ95.003-4, -5 and -6) and flow rate during Constant Rate Injection Test in interval EDZ95.003-5.
The increase in pressure after 90 hours may be related to some sort of boundary conditions, such as the saturation of a previously air-filled vug or isolated fracture. The slow and steady increase in pressures in the neighbouring intervals during the constant-head portion of the test indicates a hydraulic connection probably through the matrix material.

Two estimates of hydraulic conductivity were made from the injection (RI and HI) period. A steady-state approximation from the RI-Phase yields \( K = 5 \times 10^{-10} \text{ m/s} \), which is consistent with the result of the Constant Head Injection Test result. A steady-state approximation from the subsequent HI-Phase yields \( K = 3 \times 10^{-11} \text{ m/s} \), which is consistent with that of the neighbouring intervals.

Log-log diagnostic and semi-log plots of the recovery period following injection are shown in Figures 6.6 and 6.7, respectively. Because steady-state conditions were reached in the prior injection period, the plots are prepared using normal time. The derivative of the pressure build-up stabilises between approximately 40 and 100 minutes, indicating possible infinite-acting radial flow conditions. After 100 minutes, the rate of recovery suddenly increases, causing a hump in the derivative, and then decreases, resulting in a downward-trending derivative. After 1000 minutes, the derivative appears to stabilise again.

A straight-line match was made for the time period from 40 to 100 minutes. Using \( q = 1.4 \text{ g/h (3.9 } \times 10^{-10} \text{ m}^3/\text{s)} \), the match yields a hydraulic conductivity of \( 2 \times 10^{-11} \text{ m/s} \), which is consistent with the late-time steady-state approximation from the injection phase.

![Log-log diagnostic plot for pressure recovery following Constant Rate Injection Test in interval EDZ95.003-5.](image)
The cause of the perturbation, or hump, in the derivative after 100 minutes is not immediately obvious. Possible explanations are either changing phase conditions in the interval or equipment effects.

**Borehole EDZ95.002, Intervals 5 and 6 (RI)**

A Constant Rate Injection Test was designed for intervals EDZ95.002-5 and EDZ95.002-6 with an injection rate of 10 ml/min using an HPLC pump and D6 flow meter (Figure 6.3). Injection was conducted simultaneously in both intervals by installing a bridged injection line to the intervals. The observed pressure response is again quite unsteady, characterised by sudden drops of pressure (Figure 6.8). During the course of the test, approximate steady-state conditions were achieved and water was observed flowing from the borehole, i.e. around packer No. 6. A slight cross-interval reaction was also observed in EDZ95.003-4.

A steady-state approximation using late-time data yields $K = 9 \times 10^{-9}$ m/s based on an interval length of 30 cm. A log-log diagnostic plot of the subsequent pressure recovery period (plotted using normal time because steady-state conditions were reached) indicates a constant head boundary, as would be expected based on the RI reaction and the nearness to the tunnel wall (Figure 6.9). No further analysis of the shut-in and recovery (RIS) period was performed.
Fig. 6.8: Plot showing pressure responses (intervals EDZ95.003-4, EDZ95.002-5 and -6) and flow rate during Constant Rate Injection Test in intervals EDZ95.002-5 and EDZ95.002-6.
Borehole EDZ95.002, Interval 2 (RI)

A Constant Rate Injection Test was designed for interval EDZ95.002-2 with an injection rate of 100 ml/min using an HPLC pump and D6 flow meter (Figure 6.3). The duration of the test was limited by the availability of filtered water required for the HPLC pump. The observed pressure response is again quite unsteady (Figure 6.10). The slight cross-interval reactions observed in the adjacent intervals are the result of packer pressure changes in response to the pressure change in the test interval.

The pressure response is too irregular to provide a transient analysis. A steady-state approximation using late-time data yields $K = 3 \times 10^{-7}$ m/s. A log-log diagnostic plot of the subsequent pressure recovery period (Figure 6.11) indicates a constant head boundary, which is consistent with the anomalous static formation pressure (nearly atmospheric).
Fig. 6.10: Plot showing pressure responses (intervals EDZ95.002-1, -2 and -3) and flow rate during Constant Rate Injection Test in interval EDZ95.002-2.
6.3 Results and conclusions

6.3.1 Pressure distribution

The pressures observed at the end of the Post-test Monitoring Phase are presented in Figure 6.12. The pressure distribution around the tunnel is non-uniform. The non-uniformity reflects the effects of the tunnel acting as a groundwater sink and, in certain cases, the influence of conductive features that intersect the tunnel.

6.3.2 Hydraulic conductivity distribution

The "best-estimate" hydraulic conductivity values are presented in Table 6.3 and are depicted in Figure 6.13. The distribution of hydraulic conductivity is discussed here in the following context:

• general distribution of values
• variations of values for individual intervals with respect to distance from the tunnel wall
• comparison results in each borehole with respect to borehole orientation.

Of the 18 tested intervals, 14 have hydraulic conductivities between $3 \times 10^{-12}$ and $3 \times 10^{-11}$ m/s. EDZ95.003-5 appears to have a composite K-distribution with an inner K-value of $5 \times 10^{-10}$ m/s and an outer K-value of $3 \times 10^{-11}$ m/s. Higher permeability values (equivalent to $> 1 \times 10^{-8}$ m/s) were clearly observed for EDZ95.002 in intervals 2, 5 and 6.
Fig. 6.12: Distribution of formation pressure in the EDZ boreholes at the end of the pressure recovery period.

* Pressures at the end of the PSR period
Fig. 6.13: Distribution of hydraulic conductivity in the EDZ boreholes.

The relatively high hydraulic conductivity values observed in EDZ95.002, intervals 5 and 6, indicate a pre-existing geological feature, the hydraulic conductivity of which may have been enhanced by tunnel construction. The high hydraulic conductivity of EDZ95.002, interval 2, is also related to a feature independent of EDZ origin (i.e. a pre-existing fracture). Both of these features exhibit similar characteristics to features observed in the Radionuclide Migration Experiment MI (FRICK et al., 1992) and the Fracture System Flow Test BK (VOMVORIS & FRIEG, 1992a).

The composite K-distribution in EDZ95.003-5 may indicate the presence of a poorly-developed fracture that does not interconnect with other fractures. It is not possible to determine whether this feature is related to stress redistribution in connection with the development of an EDZ, or if it is a pre-existing (i.e. prior to tunnel construction) feature.

Hydraulic conductivity values for the intervals 2 to 2.5 metres from the tunnel wall (interval 1 of each borehole) are quite consistent, ranging from $2 \times 10^{-12}$ m/s to $3 \times 10^{-12}$ m/s. These values are representative for the matrix at the test site and are consistent with results of tests recently
conducted in the matrix in the nearby Borehole Sealing Experiment. It is noteworthy that previous studies (VOMVORIS & FRIEG 1992) have indicated typical matrix hydraulic conductivities to be approximately $1 \times 10^{-11}$ m/s.

Excluding the higher permeability intervals mentioned above (EDZ95.002, intervals 2, 5 and 6 and EDZ95.003-5), the remainder of the intervals within 1 metre of the tunnel have hydraulic conductivity values ranging from $8 \times 10^{-12}$ to $3 \times 10^{-11}$ m/s, which is approximately one order of magnitude greater than that for the intervals 2 metres from the tunnel. This suggests the presence of an EDZ around the tunnel, albeit with hydraulic conductivity one order of magnitude less than was expected based on the typical matrix values reported in VOMVORIS & FRIEG (1992b).

Mathematical modelling indicates that the development of an EDZ would likely be elliptical in form, with greater fracturing along the vertical axis of the tunnel than along the horizontal axis (LANYON 1994). If this is in fact the case at the test site, the EDZ should be more developed in boreholes EDZ95.002 and EDZ95.003 than in EDZ95.001 (Figure 6.2). The results of the hydraulic testing do not indicate a difference in EDZ development between the three boreholes. However, because of the small number of boreholes tested, this cannot be considered conclusive regarding the nature of the EDZ at the GTS.
7 ACOUSTIC EMISSIONS

Triggered by hydrotesting, new fractures in the rock can form along hydraulically active fracture zones or existing fractures can be sheared or extended. This gives rise to acoustic emissions. During hydrotesting acoustic emissions were monitored to control the test site with the following main aims:

- detection of artificial fracturing during hydrotesting (e.g. mainly injection tests)
- to avoid the creation of new fractures or the shear/extension of existing fractures
- to investigate whether signals from fracture formation occur which are not triggered by the hydrotesting.

The injection pressure was limited to a maximum of 5 bar in advance. If, however, acoustic emissions do occur due to the performance of the hydrotesting, it was planned to lower the injection pressure respectively.

7.1 Data acquisition

The acoustic emission data were registered in two boreholes (EDZ95.005 and EDZ95.006) close to the radial borehole array and each was equipped with an accelerometer, at a depth of 50 cm using a PC-controlled system (SCHWERE & ALBERT 1996).

The configuration of the boreholes is shown in the sketch in Figure 7.1. With this configuration, it is possible to register acoustic emissions, but not to locate the point of origin of the emissions.

Acoustic emissions are not registered continuously, but rather only individual events whose amplitudes exceed a fixed threshold value. Before registering the first data, the noise level of the two probes was analysed and the threshold value set.

The data were registered in two different datasets. The first began on 27.11.95 and ended on 10.01.96 after two hydrotests and the data were then stored. Dataset 1 consists of 2658 events.

Registration of the second dataset began on 11.01.96 and ended on 27.02.96. Dataset 2 consists of 5613 events.
7.2 Data analysis

After a first check through the data, the events were divided into 12 different categories, which are briefly described in Table 7.1. The difference between the events registered from probe 1 and probe 2 is immediately noticeable. It can generally be said that probe 1 shows stronger signals than probe 2 but, for some categories, the relationship is exactly the opposite.

Some typical events could already be identified in the test phase before registration of the first dataset. Closing the air-tight door, for example, gave a very characteristic low frequency event, the same being true for footsteps in the tunnel.

Mainly the events of categories $s$, $ps$ and $disp$ were considered as potential seismic signals as their form corresponds to that of seismic events. The only surprise was that the events of categories $s$ and $ps$ generally showed a phase difference of $180^\circ$ without any traveltime difference between the first motions. For acoustic emissions, this is possible only if the event was located in the exact centre between the two probes, where the boreholes for the hydrotests are also located.
Tab. 7.1: Characterisation of categories for registered acoustic emissions.

<table>
<thead>
<tr>
<th>Category</th>
<th>Characterisation</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>s</td>
<td>standard event</td>
<td>2506.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3375.2 low frequency</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3381.2 high frequency</td>
</tr>
<tr>
<td>ps</td>
<td>similar to s, but with clearly differing primary and secondary events</td>
<td>2230.2</td>
</tr>
<tr>
<td>ver</td>
<td>similar to s, but generally oscillates longer and has a clearly overlaid lower frequency</td>
<td>4584.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4592.2</td>
</tr>
<tr>
<td>Disp</td>
<td>Probe 2: low frequency, amplitude initially increasing then decreasing; Probe 1: higher frequency than Probe 2, amplitude decreasing</td>
<td>1742.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1624.2 large amplitude</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1566.2 time difference</td>
</tr>
<tr>
<td>gr</td>
<td>Probe 1: very high frequency initially then decreasing Probe 2: only high frequency events can be recognised</td>
<td>1732.2</td>
</tr>
<tr>
<td>spike</td>
<td>spikes</td>
<td>3351.2</td>
</tr>
<tr>
<td>hf</td>
<td>events with very high frequency</td>
<td>1873.2 multiple event</td>
</tr>
<tr>
<td>doors</td>
<td>very low frequency event, regular oscillation, frequently only seen on Probe 2, cause: doors closing, footsteps etc.</td>
<td>1482.2</td>
</tr>
<tr>
<td>10</td>
<td>similar to doors, but with larger amplitudes and irregular oscillations</td>
<td>1610.2</td>
</tr>
<tr>
<td>12</td>
<td>similar to doors (Probe 2), but Probe 1 shows higher frequency oscillations</td>
<td>1611.2</td>
</tr>
<tr>
<td>14</td>
<td>Probe 1: similar to disp Probe 2: no clear event recognisable, similar to doors</td>
<td>1628.2</td>
</tr>
<tr>
<td>freq</td>
<td>similar to doors but with a high frequency overlay</td>
<td>1577.2</td>
</tr>
</tbody>
</table>

7.3 Statistical interpretation

For the statistical interpretation, the events were entered in Databank 1. Even at the stage of entering the data in the databank, it was noticed that very high numbers of events were registered on certain days. Some of these could be explained by the work in the heater test drift and the set-up test of the registration equipment (SCHWERE & ALBERT 1996). Databank 2 was formed by removing these events (filtered data).

7.3.1 Evaluation according to date

The graph in Figure 7.2 (raw data) and 7.3 (filtered data) shows that the days up to and including 01.01.96 were all relatively quiet, with an average of around 20 events per day. From 02.01.96, the number of events then rose. The days 09.01.96 (mainly category disp), 26.01.96 and 20.02.96 show significantly more events than other days.

All further descriptions concern the graphs in SCHWERE & ALBERT (1996, Appendix 5.1 & 5.2).
The categories *doors*, *spike* and *hf* occur more or less regularly, with a weekly rhythm sometimes being recognisable. With the exception of the category *hf*, this is also confirmed by analysis of the week-day databank.

Events of the *disp* category occur properly for the first time on 02.01.96 and reach a clear maximum on 09.01.96 (333 events). This can still be recognised even if all categories are considered together (see Figure 7.2 and 7.3). After 10.01.96, this category occurred only on isolated occasions.

Category *gr* shows similar behaviour, although the total number of events (maximum of 27) is much lower than for the *disp* category. The period of increased events also begins on 02.01.96 and ends on 08.02.96, after which this category occurs only sporadically (see SCHWERE & ALBERT 1996).

The categories *s* and *ps* occur more frequently only from 06.01.96. Five periods with an increased number of events (100 – 200 events per day), each lasting a few days, can be recognised. In between these there are days with practically no events and no pattern can thus be derived. The same is true for category *ver*. The first two-day event occurs on 26./27.12.95 and the second on 25./26.01.96 (64 events), followed by further 3 – 4 day time spans with increased numbers of events (maximum 104).
Fig. 7.2: Histogram with distribution of all events according to date (raw data).
Fig. 7.3: Histogram with distribution of all events according to date (filtered data).
7.3.2 Evaluation according to week-day

The days with clear artefacts are almost all Tuesdays (28.11., 19.12., 09.01., 16.01.) and it is therefore not surprising that most events occur on Tuesdays (Figure 7.4). What is surprising is that, even after eliminating these days, the maximum can still be recognised (filtered data in Figure 7.5).

![Histogram with distribution of all events according to week-day (all data).](attachment:Figure_7.4.png)

![Histogram with distribution of filtered events according to week-day.](attachment:Figure_7.5.png)

A more detailed analysis shows that the categories disp (09.01.96), s (09.01.96, 06.02.96, 20.02.96) and ver (20.02.96) also show a higher number of events on Tuesdays than on other days and are therefore responsible for this maximum (see SCHWERE & ALBERT 1996).
Operation of the system started on Monday 27.11.95 and stopped on Tuesday 27.02.96, which meant that one Monday and Tuesday more than other days were included. The values for these two days must therefore be adjusted downwards to allow a meaningful comparison. Data registration for Wednesday 10.01.96 (up to 15:00) and Thursday 11.01.96 (from 8:50) is incomplete since the change from dataset 1 to dataset 2 was made at this point. For this reason, the number of events for Wednesday and Thursday could be slightly higher than is shown. If these two effects are taken into consideration, it can be seen that, from Monday to Friday, around the same number of events are registered, with the exception of Tuesday as mentioned above.

Fewest events were registered on Saturdays, with Sunday lying just below other days. A trend towards fewer events at the weekend is thus not recognisable because only Saturday has clearly fewer events.

The category doors occurs practically only from Monday to Friday, which again indicates that it can be explained by activity at the GTS. The same is true for the categories 10, 12 and 14 (maximum of category 14 on Tuesday corresponds to 16.01.1996). A weekly rhythm can also be seen for the category spike, although 5 events were always registered on Sundays (compared to an average of 25 for other days).

For the remainder of the categories (disp, freq, gr, ps, s, ver), no weekly rhythm can be recognised, but some characteristics can be identified. With 430 events, compared to the average of 100, the category disp shows a clear maximum on Tuesday. Events of category ps are very rare on Wednesdays. Category s is more frequent on Tuesday and Sunday (609 and 452 events respectively compared with an average of 290). Category ver also shows a maximum on Tuesday (244 events) and a minimum on Sunday and particularly on Saturday (72 and 27 events respectively). Events of category gr are more or less evenly distributed over the days. It is difficult to say anything about category freq since a maximum of only 9 events per week-day occurs.

7.3.3 Evaluation according to time of day

In the two hourly intervals 10 (10:00 – 10:59) and 11 hours (11:00 – 11:59), there is a clear increase in the number of events (Figure 7.6). This is an effect of 16.01.1996. If these events are ignored and the others which correspond to artefacts are eliminated, the following picture results (Figure 7.7): an increase in the number of events from 01 hours to a maximum at 06 hours, followed by a decrease, then increased values again from 10 – 15 hours; the number of events at 12 and 14 hours was somewhat lower.

From 16 hours to 02 hours, the values are consistently low, with the exception of the value at 20 hours. This is mainly caused by the category disp. The period from 10 hours to 15 hours is partly due to categories 10, 12, 14, doors and spike, which can probably be traced back to working activity at the GTS. The increase in the number of events between 01 and 06 hours is somewhat unusual.

The category hf has events at all times, but at 06, 07 and 08 hours the events are almost double those at other times. The trend towards an increase in the morning hours is clearly recognisable.

The distributions of the categories s and ps show similar features. They occur over the entire 24 hour period, but a trend can be recognised. From 01 hours, the number of events begins to increase, with a maximum being reached at 06 and 07 hours. Here again the morning hours are the most active, which is comparable with category hf.
Fig. 7.6: Histogram with distribution of events according to time of day (all data).

Category ver has some very interesting features. It occurs almost only during the night (from 17 to 07 hours) and is only sporadic during the day.

It is difficult to say anything about category freq because there are too few events (maximum of 7 per hourly interval). They do, however, give the impression of occurring more or less randomly.
7.3.4 Evaluation according to travel-time difference

If one looks at the date graph in the travel-time difference databank (Figure 7.8), it can be seen that it is dominated completely by the distribution of the category \textit{disp} (see SCHWERE & ALBERT 1996). The reason for this is that many of the events in this category show a travel-time difference, while those of the other categories do not.

On Tuesdays and Thursdays, the events with travel-time difference are higher than on other days. There is no clear decrease at weekends. The Tuesday peak (caused by 09.01.96) was to be expected, but the peak on Thursday was a surprise.

The daytime graph (Figure 7.9) clearly shows an increase in events between the time intervals 10 and 16 hours. It was during this time period that most of the hydrotests were carried out. It is therefore possible to postulate a connection between the hydrotests and the events with travel-time differences.

If events are grouped according to the size of travel-time difference then the following picture emerges: most of the events show travel-time differences between 0 and 0.2 ms.

The event can generally be seen first at probe 1 and more rarely at probe 2. The few events with very large travel-time differences are from the categories \textit{12}, \textit{14} and \textit{doors}. They are probably mostly two different events on both channels.
Fig. 7.9: Histogram with distribution of events according to travel-time difference (daytime).
7.3.5 Correlation with hydrotests

Table 6.3 shows the tests carried out (hydraulic testing log). What is important for the purpose of this report is the information on the three types of hydraulic test:

- PI: Pulse Injection Test
- HI: Constant Head Injection Test
- RI: Constant Rate Injection Test

The system was installed on 08./09.11 and 14./15.11.95. The pressure build-up phase lasted from 15.11.95 to 09.01.96. The first hydraulic tests were carried out on 09.01.96.

There does not seem to be any correlation between the hydrotests and the acoustic emissions (see SCHWERE & ALBERT 1996, Appendix 5.8). Only during one test did the number of events increase. This was the HI test of 09.01.96, which was performed from 13:37 – 15:33. The disp category shows a clear increase in the number of events during this period. However, it should be noted that 09.01.96 was a fairly eventful day in any case and the number of disp events actually began to increase from 12:00 hours. This indicates that there is no correlation between the hydraulic test HI EDZ95.002-3 and the acoustic emissions.

If the graphs in the hydrotest databank are examined, it can be seen that the PI tests have no simultaneous events, but that the HI test periods do contain such events. This is not surprising since the HI tests have a duration of several hours. The same is true for the RI tests.

09.01.1996 is the only test day for which excessive numbers of category disp events occur. It is also the only day on which PI tests were performed. Although no acoustic emissions were registered at the exact times of the PI tests, it cannot be ruled out, given the accumulation of events, that acoustic emissions were generated following the PI tests.

The days when RI tests were carried out are dominated by events of the categories s and ps. It should however be noted that some of these days have only two events per hour.

Taken together, all this implies that there is no correlation between the hydrotests and the acoustic emissions.

7.4 Conclusions

A large number of acoustic emissions was registered, many of which could be linked directly to the operation of the test site – for example work in the heater test tunnel, doors closing, etc. – and then filtered out.

The majority of the events show no travel-time difference, but only a phase difference of approximately 180° between the probes. A statistical analysis shows that, for most events, there is only a slight correlation with normal working days (a lot on Tuesdays, fewest on Saturdays and around the same for the other days). Most events do not correlate with the normal working hours at the GTS, with many of them occurring in the early hours of the morning (05 – 07 hours). If it is attempted to explain these events by formation or expansion of fissures in the rock, then the events would require exactly the same travel-time to both probes. However, since these events do not correlate in time with the hydrotests, this would be a major coincidence. Another possibility for explaining events with a travel-time difference of 0 ms and a phase difference of approximately 180° would be electrical interference reaching the system from the main supply. This could be checked by trying to correlate voltage fluctuations with registrations, but it is not clear whether the Oberhasli power plant has suitable numerical information to allow this to be determined.
Acoustic emissions caused by fissure formation should, as a rule, show a travel-time difference between the two probes of up to 0.5 ms for p waves \((V_p > 4000 \text{ m/s}, \Delta x = 2 \text{ m})\) and up to 1.4 ms for Stoneley waves. Only 10% of the events (787 of 8473) have a recognisable travel-time difference (recognition threshold: 0.01 ms). The events entered in the databank under the category disp show the most travel-time differences. The form of the wavetrain also indicates that the events of this category could be caused by fissure formation in the rock. Events with travel-time difference occur mainly in the time period from 01.01.96 to 16.01.96, with peaks on 9.1 and 4.1; before 01.01.96 and after 16.01.96 they occur only rarely. It should be checked whether, during this period, there were increased rock mechanical stresses due to e.g. weather effects, filling of the reservoir, etc.

The hydrotest boreholes EDZ95.001 – EDZ95.004 are equidistant from both probes. Assuming that the hydrotests cause fissures mainly in the vicinity of the boreholes, then the travel-time difference for hydrotest-induced acoustic emissions should frequently be 0 ms. It is therefore likely that, for many events of this type, no travel-time difference will be detected. The acoustic emissions accumulated on only one day (09.01.96) when hydrotests were being performed. Apart from this, no temporal correlation could be identified between hydrotesting and events of the acoustic emissions. On 09.01.96, four PI tests and one HI test were carried out. There were no events exactly at the times of the PI tests. During the HI test there were many events, but the increase actually started before the test began. It could be that the accumulation of events before the HI test is a result of the PI tests, which had been carried out previously, but there is no concrete evidence for this.

To summarise, it can be said that the measurements of acoustic emissions have shown:

- that acoustic emissions can be measured at the GTS
- that the fracturing behaviour of hydrotests can be monitored
- that acoustic emissions were registered which confirm that small/micro-scale fractures are either extending, shearing or forming in the rock mass
- that the acoustic emissions cannot be explained by the hydrotests.

The type of registration can be improved in future measurement programmes. Interference from the main supply could be minimised by a battery-buffered system. To determine which events can be explained by footsteps, doors, drilling work etc. in the test zone, it would be necessary for the local staff to log all activities in the test zone.

For financial reasons, only two probes were used in these measurements. This meant that the events could not be located with certainty, as this is impossible with only two probes. Only by using four probes are three travel-time differences obtained and, with this, the possibility to locate the events three-dimensionally. The number of probes should therefore be increased to at least four in the future.
8 FRACTURE NETWORK MODELLING OF THE EDZ

This chapter describes modelling performed following characterisation of the EDZ described in the previous chapters. It is based on the fracture network models developed for the design calculations discussed in section 5.3. The results of the hydraulic and geological characterisation together with the requirement to predict post-closure effective hydraulic properties required modification of these models as described below.

8.1 Conceptual model of Excavation Disturbed Zone

The EDZ was defined at the start of this report as the mechanically altered zone (with potentially altered hydraulic properties) which develops during and after the excavation of tunnels and caverns. This zone includes the damaged zone: a region where fractures have been induced, plastic deformations have occurred and the rock properties permanently altered. In the EDZ but outside this damaged zone, property changes are largely reversible and deformation elastic.

The conceptual model used in the design calculations was one that included two fracture networks, a natural fracture network and a damaged zone (i.e. that part of the disturbed zone where new fractures were induced by excavation) fracture network. The description of the natural fracture network was generic for the GTS and drew on geological judgement and observations in other parts of the GTS. Only tunnel mapping data were available to characterise the damaged zone fracture network and hence a "plausible" description was developed so that the design calculations could be performed. This description of the damaged zone was based on induced fracturing within an envelope defined by a theoretical strain threshold (STACEY 1981). The model of damaged zone fracture transmissivity included a dependence on the calculated strains and an estimate of the likely spacing of the fractures. The strains were calculated from an analytical solution of stress around a tunnel in a homogeneous linear elastic medium. More sophisticated models of the stress distribution could have been used (for example see section 4.2 and VOMVORIS et al. 1997), however given the level of characterisation data available it was felt that use of the analytical models was reasonable.

The models as implemented for the design calculations did not consider any stress-dependence of either fracture system, this was on the assumption that at the scale of testing envisaged the dominant mechanism for changes in hydraulic properties would be the creation of damaged zone fractures. The work discussed here has as its major aim the development of a methodology for estimating the axial flow properties around the tunnel after closure and resaturation of the excavations. It is this case that is most relevant to the assessment of the safety of a radioactive waste repository. Therefore it is necessary to consider the effects of changes in stress and pore pressure that will occur between the characterisation of the damaged zone and closure and resaturation of the excavations.

The revised conceptual model is based on the data from the GTS characterisation and on requirements for prediction of post-closure properties. The models assume that the natural fracture system has been characterised in 'undisturbed' conditions while the damaged zone has been characterised in a disturbed state due to the presence of the excavation. An illustration of this revised concept is shown in Figure 8.1. The next sections summarise the inputs from characterisation and their effects on the conceptual model and its parameters.
### CHARACTERISATION OF UNDISTURBED SYSTEM

<table>
<thead>
<tr>
<th>Parameter</th>
<th>State Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>PORE PRESSURE</td>
<td>Minor disturbance from well test pressures</td>
</tr>
<tr>
<td>STRESS</td>
<td>Undisturbed regional stress</td>
</tr>
<tr>
<td>SATURATION</td>
<td>Fully saturated</td>
</tr>
<tr>
<td>ROCK</td>
<td>Minor damage close to borehole</td>
</tr>
</tbody>
</table>

### CHARACTERISATION OF EXCAVATION DISTURBED SYSTEM (OPERATIONAL PHASE)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>State Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>PORE PRESSURE</td>
<td>Drawdown by presence of excavations</td>
</tr>
<tr>
<td>STRESS</td>
<td>Stress redistribution caused by excavation</td>
</tr>
<tr>
<td>SATURATION</td>
<td>Close to excavation partially unsaturated</td>
</tr>
<tr>
<td>ROCK</td>
<td>Properties altered within the damaged zone</td>
</tr>
</tbody>
</table>

### TIME RELEVANT TO PREDICTION OF PROPERTIES (POST-CLOSURE PHASE)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>State Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>PORE PRESSURE</td>
<td>Assumed to return close to undisturbed conditions</td>
</tr>
<tr>
<td>STRESS</td>
<td>Stress redistribution caused by excavation and backfill</td>
</tr>
<tr>
<td>SATURATION</td>
<td>Fully saturated (some gas generation from waste)</td>
</tr>
<tr>
<td>ROCK</td>
<td>Properties altered within the damaged zone</td>
</tr>
</tbody>
</table>

Fig. 8.1: Conceptual model of characterisation and prediction.
8.1.1 Geological input from GTS experiments

Geological input was limited because of poor core recovery and the lack of high-quality images of the borehole wall. However the following information was provided by MÖRI & BOSSART (1997):

- Positions of identified natural fractures
- Core recovery and density of open and closed core disking
- Identification of breakout zone extent
- Identification of some breakout fractures (Auflockerungsklüfte)

The data are summarised in Figure 8.2. It is important to note the fracture that runs along the length of borehole EDZ95.002.

Fig. 8.2: Results from core and borehole characterisation (after MÖRI & BOSSART 1997).
The density of breakout fractures (Auflockerungsklüfte) observed close to the tunnel wall is highly variable. In EDZ95.003 only a single such feature is visible in the first 5 cm, while in EDZ95.002 17 features are recorded in the first 15 cm. Data from boreholes ZPK94.001 to ZPK94.008 of the ZPK experiment within the BK area (MARSCHALL et al. 1999; see also section 1.2) suggest open fractures (offene Trennflächen) in the first 30 cm with 60 % of such features in the first 10 cm and 94 % in the first 20 cm from the tunnel wall. The results from the two locations therefore appear to be in agreement with regard to the position of breakout fractures, but the data from EDZ95.002 suggest a higher density given that the only 32 such features were identified in all 8 ZPK boreholes. This high density may be related to the natural fracture intersected in EDZ95.002.

The estimate of breakout zone (Auflockerungszone) extent provided from borehole inspection is significantly deeper than the observations of breakout fractures and is probably influenced by the core disking and core recovery data. The depth of the zone is large compared to estimates of damaged zone extent based on sonic velocities from the ZPK boreholes where sonic velocities appear to be roughly constant beyond 50 cm from the tunnel wall (MARSCHALL et al. 1999). This may however reflect differences between the two locations. Ideally quantitative comparisons should be made by measuring the sonic velocity in the EDZ boreholes.

### 8.1.2 Hydraulic test results

The hydraulic data from the testing described in Chapter 6 are shown in Figure 8.3. The hydraulic conductivity data show the following:

- a zone of rock with roughly constant conductivity of $2 \times 10^{-12}$ m/s beyond 2 m from the tunnel wall
- hydraulic conductivities of $8 \times 10^{-12}$ m/s or greater, which is larger than the expected matrix conductivity in all zones within 1 m of the tunnel wall
- high hydraulic conductivity intervals associated with borehole EDZ95.002 in the first metre with the most conductive short interval zone being the farthest from the tunnel wall.

Figure 8.4 shows a histogram of log$_{10}$ hydraulic conductivity for the intervals tested. The distribution appears bi-modal, with the lowest conductivities all relating to the outer 0.5 m intervals. The hydraulic conductivity of the outer zones is comparable to that estimated for the matrix in the design calculations ($10^{-12}$ m/s).

However the zone of elevated matrix conductivity appears to extend beyond the 1 m radius of the small-scale testing with the MMPS system and does not correspond to the breakout zone estimated from the geology. It may be that the zone of change of hydraulic properties extends to nearly 2 m from the tunnel wall. This may relate to excavation induced disturbance or alternatively it may relate to some local rock heterogeneity or disturbance from drilling of the boreholes.

MARSCHALL et al. (1999) suggest the presence of a similar zone of higher matrix conductivity around the ZPK boreholes and present data from core samples with gas permeabilities of $2 \times 10^{-18}$ m$^2$ (equivalent to approximately $2 \times 10^{-11}$ m/s hydraulic conductivity).
The pressure/head data measured with the MMPS shows the following:

- Near atmospheric pressures in the first 60 cm of the boreholes with pressures starting to climb beyond this.
- A low pressure/head associated with the most permeable interval in borehole EDZ95.002 suggesting connection back to the tunnel.
- Increased head/pressure in the outer 0.5 m intervals with the highest pressure in EDZ95.002, which is furthest from the main excavations.

Integration of the hydraulic data with the geology raises a key question: Are the higher transmissivities observed in EDZ95.002 related to the natural fracture or to damaged zone features?
If the high transmissivity intervals relate to damaged zone features then these may occur elsewhere within the damaged zone and it is necessary to include a class of high transmissivity features within any models of the damaged zone. This suggests that the effective conductivity of the damaged zone will depend on the length along the tunnel and frequency of such high transmissivity features. This case has been modelled as *Variant 1* within the calculations described later in section 8.3.

Alternatively if the high conductivities relate to "channels" within the pre-existing natural fracture then the results suggest a conceptual model with a zone of elevated matrix conductivity of approximately $1 \times 10^{-11}$ m/s extending between 1 and 2 m from the tunnel, and a stress-perurbed channelled fracture of higher conductivity intersecting EDZ95.002. This second model would imply that damaged zone effective conductivity might be only slightly greater than that of the matrix away from the fractures. This means that unless natural fractures run sub-parallel to the tunnel over the length of any repository seal zone, damaged zone flows will be controlled by the elevated matrix conductivity. A channelled fracture model has not been fully developed during this study. Instead it has been assumed that the channelling is relatively small scale and that the uniform transmissivity models used in the design calculations represent a simplified version of such a network. This case has been modelled as *Variant 2* within the calculations described later in section 8.4.

These two conceptual models are illustrated in Figure 8.5. While the second model seems to provide a simpler explanation of the observations, the quality of observations together with the potential for high flow in the first model make it difficult to discount the first model.

### 8.2 Model development and description

The results of the characterisation described previously and the requirement to couple the models to the effects of stress and pore pressure changes that will occur with closure and resaturation mean that a range of modifications and developments to the existing models were needed.
Fig. 8.5: Model variants used in axial flow calculations.
8.2.1 Modifications to models of the fracture networks

Natural fracture network

Minor revisions could have been made to the natural fracture network data but this would have been based on only the two fractures intersected in the four boreholes. Furthermore, the extension vein features probably only occur in the margins of lamprophyre dykes. Evidence of channelling from the fracture intersected in borehole EDZ95.002 could also have been used to modify the network. The observations suggest channel widths of 10 – 30 cm, with the transmissive channels being separated by zones of slightly higher conductivity than the surrounding matrix.

Channelling at these scales was not considered in the model used in the design calculations and the transmissivities used assume some averaging scale larger than the channel size as fractures were represented by planes of uniform transmissivity.

It is difficult to integrate the results from EDZ95.002 into a revised conceptual model because the likelihood of randomly drilling along a fracture is very low. It is possible that the borehole was specifically drilled along the fracture, in which case it is not possible to simply integrate the results into a stochastic model of fracture occurrence.

Given the limited nature of the observations and the difficulty of fitting them into the framework of a stochastic model, the original fracture network from the design calculations was left unchanged for the calculations described here.

Damaged zone extent

The high conductivity intervals within the damaged zone fall within the damaged zone extent predicted from design calculations and the breakout zone extent estimated by MÔRI & BOSSART (1997). The estimates of extent are in reasonable agreement with the strain threshold envelope for damaged zone fractures used in the design calculations so this has been used as the envelope for high transmissivity damaged zone features within the fracture network modelling described in this chapter.

The region of enhanced matrix conductivity extends around the tunnel for 1 – 2 m with no clear boundary observable from the MMPS system. MARSCHALL et al. (1999) suggest a zone of enhanced conductivity matrix (~ $1 \times 10^{-18}$ m$^2$ or $1 \times 10^{-11}$ m/s) to a distance of about 1 m in the ZPK boreholes around the BK tunnel.

Damaged zone fracture frequency

All intervals within 2 m of the tunnel wall had hydraulic conductivities above the estimate for the undisturbed matrix. However, only 3 intervals had conductivities above $1 \times 10^{-10}$ m/s (one other interval showed composite behaviour with locally higher conductivity close to the borehole).

If the high conductivity intervals relate to a natural fracture, then the other intervals correspond to either a high density network of small-scale fractures/micro-fractures or to an enhanced matrix conductivity. In reality it is likely that there is some component of small-scale fracturing close to the tunnel wall and an enhanced matrix conductivity. The observed relatively uniform hydraulic conductivity suggests that the zone is best treated as porous medium with enhanced
conductivity rather than a zone with discrete fracture flow and as such has not been modelled within the discrete fracture network (DFN) models that have been developed. However the results of the DFN models should be interpreted in the light of the existence of such a zone.

If the high conductivity intervals relate to damaged zone features and it is assumed that they are oriented normal to the borehole (as suggested by the geological evidence), the frequency is calculated as the number of features divided by the length of sample. It has been assumed that the high conductivity damaged zone features occur only within the damaged zone envelope used in the design calculations, and that the frequency is therefore 2 features per 1 m of hole (as only the intervals in EDZ95.002 and EDZ95.003 were tested within this envelope). In fact three zones showed higher conductivity, but two (EDZ95.002-5/6) appear to be hydraulically linked as if they intersected a single feature. An alternative would be to assume that such features can occur anywhere within 2 m of the tunnel wall, in which case the frequency would be 2 per 1.5 m of borehole as intervals in any of the three boreholes could have intersected such features.

**Damaged zone fracture transmissivity**

The previous damaged zone model incorporated a relationship between strain and transmissivity, but no such relationship is observable in the data and there appears to be no trend in transmissivity with depth into the tunnel wall or floor. It was therefore assumed that damaged zone fracture transmissivity was random within the damaged zone.

If the effective interval conductivities shown in Figure 7.4 are converted to transmissivities, it is possible to calculate statistics for the two groups of 0.1 m intervals, assuming a log-normal distribution for each group. The statistics are given in Table 7.1.

As stated in the previous chapter, the lower conductivity intervals ($1 \times 10^{-11}$ to $1 \times 10^{-10}$ m/s) are assumed to correspond to a zone of equivalent porous medium-like material rather than one of discrete fracture flow and so these features have not been included within the models.

**Damaged zone fracture shape and extent**

There is relatively little information on fracture extent or shape. The geological input from MÖRI & BOSSART (1997) suggested fracture lengths from 5 to 50 cm. However, some of the fractures visible in the zones of strongest breakout have trace-lengths of centimetres to approximately 5 m. In this study fractures have been modelled as squares or rectangles with a width of 1 m and length parallel to the intermediate stress direction (approximately along the tunnel) of 1 or 10 m.

**Damaged zone fracture orientation**

In the design calculations, damaged zone fractures were oriented normal to the calculated minimum principal stress. Close to the tunnel this results in the fractures being approximately tangential to the excavation surface and hence at right angles to the boreholes as observed. At greater distances the orientation is affected by the regional stresses. In order to avoid the creation of many sub-parallel fractures a small random component is added to increase connectivity of the damaged zone fractures. The orientations appear to be approximately in agreement with observation and are therefore unchanged from the design calculations.
Tab. 8.1: Transmissivity statistics for 10 cm intervals.

<table>
<thead>
<tr>
<th></th>
<th>All intervals</th>
<th>Low conductivity intervals</th>
<th>High conductivity intervals</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of intervals</td>
<td>15</td>
<td>12</td>
<td>3</td>
</tr>
<tr>
<td>Mean transmissivity (m²/s)</td>
<td>$2.2 \times 10^{-9}$</td>
<td>$1.6 \times 10^{-12}$</td>
<td>$1.1 \times 10^{-8}$</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>$7.7 \times 10^{-9}$</td>
<td>$9.3 \times 10^{-13}$</td>
<td>$1.6 \times 10^{-8}$</td>
</tr>
<tr>
<td>transmissivity (m²/s)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Standard error transmissivity (m²/s)</td>
<td>$2.0 \times 10^{-9}$</td>
<td>$2.7 \times 10^{-13}$</td>
<td>$9.5 \times 10^{-9}$</td>
</tr>
<tr>
<td>Log₁₀ mean transmissivity (m²/s)</td>
<td>-11.16</td>
<td>-11.85</td>
<td>-8.39</td>
</tr>
<tr>
<td>Log₁₀ standard deviation transmissivity (m²/s)</td>
<td>1.48</td>
<td>0.24</td>
<td>0.75</td>
</tr>
<tr>
<td>Log₁₀ standard error transmissivity (m²/s)</td>
<td>0.38</td>
<td>0.07</td>
<td>0.43</td>
</tr>
</tbody>
</table>

8.2.2 Development of effective stress dependence model

The second part of the work was to include facilities within NAPSAC for accounting for changes in environment between the time of characterisation and the time relevant to prediction. The models assumed that the major influence would be due to effective stress normal to fractures. Shear deformations were assumed to be of minor importance but would require investigation as part of any more comprehensive analysis.

Other factors such as temperature, fluid chemistry and fracture mineralisation might also be relevant but have not been considered here. The change in effective stress is due to both changes in total stress from excavation and backfilling of the tunnel and to changes in pore pressure.

If we assume that the natural fracture characterisation occurs from boreholes at moderate distances from excavations and that post-closure conditions revert to similar conditions, then changes in natural fracture system properties will be most important during the operational phase close to tunnels where significant drawdown will have occurred (potentially causing closure of the fracture), in addition to any opening or closure caused by the opening of the excavation.

For the damaged zone fractures, it is clear that there are major changes between characterisation and post-closure environment. In particular stresses change due to the backfilling of the excavation and pore pressures recover from drawdown. In addition, if the rock in the damaged zone becomes partially saturated during characterisation this may have important effects on properties determined for the damaged zone. While it has been possible to account for the pore pressure and stress changes using analytical or numerical models, changes in saturation are more difficult to deal with. There is evidence of partially saturated behaviour in the well test data from the GTS experiments, but it has been assumed that, as properties of the more conductive intervals have been derived largely from recovery after injection, the effects should be minimised.
The scheme used for calculation of stresses post-backfilling and resaturation is illustrated in Figure 8.6. The values used for pore pressures in the characterisation and post-closure phases are also given there. Figure 8.7 shows the total and effective stresses calculated for the operational and post-closure phases. It can be seen that the dominant effects are in the effective stresses where pore pressure change is important. In particular, a large zone of tensile effective stresses is predicted in the post-closure case in zones away from the damaged zone fractures.

It is necessary to consider whether backfilling and resaturation might induce further damage and changes to material properties. Inspection of the predicted total stresses suggest this is unlikely unless the backfill were to induce significant swelling pressures, in which case tensile failure might occur, creating new fractures away from the existing damaged zone.

The coupling between changes in effective stress and changes in feature transmissivity has been based on a simple power law of the form:

1) \[ \frac{T}{T_0} = \left( \frac{\sigma'}{\sigma'_0} \right)^b \]

Where \( T \) is transmissivity in \( m^2/s \) and \( \sigma' \) is effective stress in MPa. The 0 subscript refers to the conditions of an original measurement. Thus the law implies that the ratio of transmissivities at two different effective stresses is equal to the ratio of the effective stress to the power \(-b\).

DERSHOWITZ et al. (1991) reviewed a range of core measurements from fractures in granite and suggested values for \( b \) of between 0.2 and 2.

Alternative models could have been used such as those of BARTON and BANDIS (see for example BARTON & BAKHTAR 1987), however very little characterisation data concerning fracture properties are available from the GTS site and this simple approach was thought sufficient for the purposes of the modelling. Ideally information on fracture stiffness and shear behaviour should be acquired from core and in-situ testing. This would reduce uncertainty in stress-transmissivity coupling and in fracture storage properties.

For parts of some fractures tensile effective stresses are predicted, but in this case it is not possible to use the model given above. Instead a constant tensile hydraulic aperture has been used. Two values have been used in the study: 100 and 333 microns; these correspond to approximately \( 1 \times 10^{-6} \) and \( 3 \times 10^{-5} \) \( m^2/s \) transmissivity. These values are 30 and 1000 times greater than the most transmissive interval measured in the MMPS array. In general it was found that axial conductivities were not sensitive to the choice of tensile aperture, indicating that flow is limited by other parts of the fracture network.
8.3 Modelling of hydraulic tests

The hydraulic tests performed during the EDZ characterisation work in the WT drift area are described in Chapter 4. Three types of tests were performed: Pulse Tests, Constant Head Tests and Constant Rate Tests. The Pulse Tests were all performed in low conductivity intervals with hydraulic conductivity ranging between $4 \times 10^{-12}$ m/s and $1 \times 10^{-11}$ m/s. Therefore they correspond to zones of matrix-like conductivity.
Fifteen Constant Head Tests were performed with a duration of 1 to 2 hours. Of these only four intervals showed hydraulic conductivities above $3 \times 10^{-11}$ m/s. Each of the more conductive intervals was re-tested using a constant rate. The majority of the tests showed a reasonable fit to a semi-log model (JACOB & LOHMAN 1952), although some intervals showed more complex behaviour especially at very low flows. The relatively short duration and the low conductivity of the majority of the tests suggest that the radius of investigation of the tests is small. Of the remaining four intervals, in EDZ95.002 intervals 5 and 6 appear to be hydraulically connected and react simultaneously. Therefore only three Constant Rate Tests were attempted. The results from each test are briefly discussed below.
8.3.1 Constant Rate/Head Test in EDZ95.003-5

The Constant Head Test performed in EDZ95.003-5 indicated a conductivity 10 times greater than the surrounding intervals. However, this was based on a steady-state approximation, as the flow rate data did not fit the JACOB-LOHMANN model well. In fact the flow rate after the first hour was slightly larger than the flow into the surrounding intervals during Constant Head Tests. This suggests that the higher conductivity may relate only to a small region around the interval.

The Constant Rate Test was performed at a rate of 2 g/h; for the first 90 hours the pressure responses were very irregular, but after 90 hours the pressure climbed steeply to the maximum value attainable with the equipment and the test effectively became a Constant Head Test. The volume pumped during the constant rate part of the test was approximately 180 ml, which is several times the volume of the interval. This suggests that the interval itself and the surrounding rock may not have been fully saturated. Analysis of the pressure recovery suggested a conductivity of $2 \times 10^{-11}$ m/s, which was in agreement with the late time data from the Constant Head Test. This supports the suggestion that any region of higher conductivity must be local to the borehole. Given that the pressure recovery data appear to be dominated by the matrix response, modelling of this test was not felt to be useful.

8.3.2 Constant Rate Test in EDZ95.002-5 and EDZ95.002-6

A Constant Rate Test at an injection rate of 10 ml/min was performed for approximately 50 hours. During this time the pressure response was again very unsteady but appeared to see some form of constant head boundary. In fact flow was observed to occur from the packer at the tunnel wall. Analysis of the pressure recovery showed the influence of the constant head boundary immediately after well-bore storage and no period of radial flow was observed. The position of the fracture running along EDZ95.002 and the observed constant head boundary makes modelling of the tests of limited use in understanding the axial connectivity of the flow paths. In addition the poor injection phase data contribute additional uncertainty to any model.

8.3.3 Constant Rate Test in EDZ95.002-2

A Constant Rate Test was performed in EDZ95.002-2 at a rate of 100 ml/min. The duration of the test was limited by the availability of filtered water and the injection only lasted about 40 minutes. The pressure data from the injection period test were too irregular for analysis; however, analysis of the recovery indicated a constant head boundary which is consistent with the low pressure observed in the interval. No period of radial flow was observed.

8.3.4 Summary of Constant Rate Tests

The data from the Constant Rate Tests show irregular pressure responses during all the injection periods. The recovery data can be analysed and for interval EDZ95.003-5 indicate a conductivity similar to the matrix, while late time data for the intervals in EDZ95.002 indicate a constant head boundary which may well be associated with flow back into the tunnel, perhaps along the fracture.

The nature of the injection test results make it difficult to develop models for the testing as the injection phase responses appear to be related to unsaturated effects (other causes are possible but it is felt that these are less likely). Without accounting for such effects it would be difficult to simulate the responses observed in the Constant Rate Tests.
If the recovery periods are considered, for one interval a matrix-like conductivity is observed, while the results in EDZ95.002 are dominated by a constant head boundary, which probably relates to the tunnel wall. To simulate these results in a stochastic model and to develop a match would again be difficult because of the influence of the fracture that runs along the borehole. The likelihood of such a feature occurring in a model is low and would not easily be treated by the NAPSAC model since boreholes are treated as one-dimensional objects.

The methodology developed from the design calculations was that initial Constant Head and Pulse Tests should be used to identify the most transmissive features, and that subsequent Constant Rate Tests would then be used to identify the geometry and connectivity of the individual features within the EDZ.

The results from the tests have partially fulfilled this but the most transmissive features appear to be associated with structures that connect back to the tunnel, which acts as a constant head boundary for the injections. The tests cannot therefore provide information about connectivity along the tunnel unless such short-circuits can be sealed. Borehole interval EDZ95.002-2 was drilled along a fracture that is visible in the tunnel wall so it is not surprising that such a connection should exist.

Any models of the tests can therefore only be used to confirm the consistency of our models in terms of rock conductivity and connectivity back to the tunnel wall. They cannot be used to identify connectivity within the EDZ. This is a severe limitation of the test method and future tests should aim to limit flow into the tunnel, which in addition might help the creation of saturated conditions at higher pressure around the tunnel.

### Consistency tests

8.3.5 Consistency tests

In view of the limitations discussed only a small amount of modelling of the tests has been performed and this has concentrated on evaluating the consistency of the models with observation. The majority of intervals tested had low conductivities, which have been associated with the matrix in our models.

Thus the observations that the models should be consistent with are the three high conductivity intervals EDZ95.002-2 and EDZ95.002-5/6. Interval EDZ95.003-5 appears to be more transmissive only locally to the borehole, and in fact this higher transmissivity is derived from a steady-state approximation of the constant head period which showed an erratic response that might have been the result of saturation effects.

As a test of the model the transmissivity of features at each borehole location was computed for 40 realisations. The fracture network immediately around the borehole array was simulated and the stress disturbed transmissivities calculated for features intersecting the 15 10 cm intervals. The results are summarised below in Table 8.2.

The occurrence of high transmissivity features in the models is shown in Table 8.3. Comparison of the results with the statistics of the high transmissivity intervals in Table 8.1 shows that the model is consistent with the input data. As can be seen from Table 8.3, borehole EDZ95.001 rarely intersects features because of the damaged zone envelope used, while EDZ95.002 and EDZ95.003 are equally likely to contain such features. In approximately half of the realisations, 2 or more features are intersected in one borehole, suggesting that a pattern of intervals similar to that observed in EDZ95.002 can be simulated using the model.
### Tab. 8.2: Transmissivity statistics for features intersecting MMPS array (model Variant 1).

<table>
<thead>
<tr>
<th>Stress/transmissivity exponent</th>
<th>Tensile aperture [mm]</th>
<th>Excavation pressure [MPa]</th>
<th>Pore pressure [MPa]</th>
<th>Mean $\log_{10}$ transmissivity [$m^2/s$]</th>
<th>Standard deviation $\log_{10}$ transmissivity [$m^2/s$]</th>
<th>Mean count in 15 0.1 m intervals</th>
<th>Mean $\log_{10}$ transmissivity [$m^2/s$]</th>
<th>Standard deviation $\log_{10}$ transmissivity [$m^2/s$]</th>
<th>Mean count in 15 0.1 m intervals</th>
<th>Mean $\log_{10}$ transmissivity [$m^2/s$]</th>
<th>Standard deviation $\log_{10}$ transmissivity [$m^2/s$]</th>
<th>Mean count in 15 0.1 m intervals</th>
</tr>
</thead>
<tbody>
<tr>
<td>-1</td>
<td>100</td>
<td>0.1</td>
<td>0.2</td>
<td>-8.6</td>
<td>0.8</td>
<td>2.4</td>
<td>-9.7</td>
<td>0.5</td>
<td>0.4</td>
<td>-8.4</td>
<td>0.7</td>
<td>2.1</td>
</tr>
<tr>
<td>-2</td>
<td>333</td>
<td>0.1</td>
<td>0.2</td>
<td>-8.6</td>
<td>0.8</td>
<td>2.4</td>
<td>-9.6</td>
<td>0.7</td>
<td>0.4</td>
<td>-8.4</td>
<td>0.7</td>
<td>2.1</td>
</tr>
<tr>
<td>-1</td>
<td>100</td>
<td>4</td>
<td>4.0</td>
<td>-8.5</td>
<td>0.8</td>
<td>2.4</td>
<td>-9.4</td>
<td>0.7</td>
<td>0.4</td>
<td>-8.3</td>
<td>0.7</td>
<td>2.1</td>
</tr>
<tr>
<td>-2</td>
<td>333</td>
<td>4</td>
<td>4.0</td>
<td>-8.3</td>
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<td>2.4</td>
<td>-9.0</td>
<td>1.3</td>
<td>0.4</td>
<td>-8.2</td>
<td>0.8</td>
<td>2.1</td>
</tr>
</tbody>
</table>
In addition we have simulated some transient flow tests with model \textit{Variant 1} (see section 8.3.2) to examine the response expected. The models can only be indicative because of the peculiar geometry of the fracture intersecting borehole EDZ95.002 and the evidence of unsaturated zone effects. A point to note is that there is no good quality data on fracture storage; this property is typically dominated by fracture stiffnesses that are uncertain as discussed earlier.

Figure 8.8 shows a diagnostic plot, a realisation where borehole EDZ95.002 intersected a transmissive feature. The models were not conditioned on the observed transmissivities and flow rates from the interval were set at 1 ml/min, which is lower than those used in the testing. However the influence of the constant head boundary at the tunnel wall is evident, as is the influence of well-bore storage.

8.4 Modelling of post-closure axial flow

This section describes modelling performed to estimate the likely effective axial hydraulic conductivity of the rock around a tunnel at the GTS. The damaged zone properties are derived from those measured at the GTS, while the natural fracture system model is generic to the GTS and the stress-transmissivity coupling used is from published literature.

Work on the ZEDEX experiment (OLSSON et al. 1998) has considered the tunnel near-field as extending to about 2 m from the tunnel wall and this spatial division has been used in the modelling described below. The effective conductivities are given as averages over annuli of rock around the tunnel. Locally conductivities will be higher, and there may be zones of consistently higher conductivities that relate to the anisotropic stress field. The decision to calculate effective conductivities for the various annuli is based on the likely implementation of such estimates in porous medium models of flow around a repository (see for example NAGRA 1997).

8.4.1 Analytical estimate of upper bound on hydraulic conductivity of damaged zone

If it is assumed that the measurements of hydraulic conductivity made in the MMPS system are typical of the damaged zone and that this may extend a maximum of 2 m around the tunnel, it is possible to calculate an upper bound to the likely axial hydraulic conductivity.

Assuming that the high conductivity features are infinite in length and arranged parallel to the tunnel, the effective conductivity can be calculated as the arithmetic mean conductivity. The arithmetic mean conductivity of the 15 intervals is $2.2 \times 10^{-8}$ m/s, however the standard deviation is large and the standard error on the mean is $2 \times 10^{-8}$ m/s, suggesting that a 95 % confidence limit for the mean might be as high as $6.5 \times 10^{-8}$ m/s. This assumes that backfilling of the tunnel and resaturation have no effect on the conductivity of the damaged zone. The mean estimate of $2.2 \times 10^{-8}$ m/s corresponds to a factor of 2000 increase over the estimate of bulk conductivity for the GTS of approximately $1 \times 10^{-11}$ m/s. Such a zone might extend between 1 and 2 m from the tunnel wall. This model is highly conservative and is not supported by some of the evidence from the EDZ experiment; the next section considers more realistic models using the NAPSAC code.
Tab. 8.3: Simulated borehole intersection statistics for MMPS array, transmissivity [m²/s] for each interval in each realisation is shown together with realisation and interval totals.

<table>
<thead>
<tr>
<th>Realisation</th>
<th>Borehole EDZ95.001 intervals</th>
<th>Borehole EDZ95.002 intervals</th>
<th>Borehole EDZ95.003 intervals</th>
<th>Total Count</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.4E-08</td>
<td>8.0E-09</td>
<td>3.7E-09</td>
<td>22</td>
</tr>
<tr>
<td>2</td>
<td>1.4E-08</td>
<td>1.5E-07</td>
<td>1.0E-07</td>
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<td>3</td>
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<td>1.0E-08</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>6.9E-09</td>
<td>5.5E-09</td>
<td>2.2E-09</td>
<td>8</td>
</tr>
<tr>
<td>5</td>
<td>2.9E-08</td>
<td>1.0E-07</td>
<td>1.8E-07</td>
<td>1</td>
</tr>
<tr>
<td>6</td>
<td>8.3E-09</td>
<td>1.0E-08</td>
<td>1.5E-07</td>
<td>3</td>
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<tr>
<td>10</td>
<td>4.7E-09</td>
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<td>2.6E-08</td>
<td>8</td>
</tr>
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Note: The transmissivity values are given in [m²/s] for each interval in each realisation, along with the total count for each realisation and interval.
8.4.2 **NAPSAC models of post-closure flow**

The general set-up of all the models to estimate post-closure effective conductivity in the disturbed zone is as shown in Figure 8.9.

The NAPSAC volume is approximately modelled as a hollow cylinder with inner radius 1.85 m and outer radius either 3.85, 5.55 or 12 m. The small external radius model relates to flow only in the zone of induced fractures close to the tunnel, while the larger models are used to estimate an effective conductivity for the EDZ and the natural fracture system as a whole. The 12 m radius corresponds to over 6 excavation radii, beyond which point it is assumed that disturbed zone effects are insignificant. In addition some models calculated the effective conductivity for an annulus from 5.55 m to 12 m (i.e. excluding the zone with induced fractures) to demonstrate the lack of changes in hydraulic properties outside this zone.

The model is aligned with the orientation of the tunnel at the EDZ experimental area. The length of the cylinder was varied between 10 and 20 m. The cylinder is approximated as 12 trapezoidal prisms.

For the calculation of effective axial conductivity a pressure gradient was applied across the two end faces of the hollow cylinder while all other faces are set as no-flow boundaries. The effective hydraulic conductivity is estimated using DARCY’s Law. The imposition of no-flow boundaries on the cylindrical faces of the models reduces connectivity close to the boundaries and so effective properties of the thin annuli may be underestimated in sparse or poorly connected fracture networks.
a) Model geometry and boundary conditions

b) Different annuli considered and approximate correspondence to damaged, disturbed and undisturbed rock zones

Fig. 8.9: Axial effective conductivity model geometry.
The effective conductivity of each model was calculated over 10 realisations. The results from each of the models are given in a series of Table 8.4a-f.

Table 8.4a presents the effective axial conductivities ($K_{\text{eff-axial}}$) for the case where there are no high conductivity damaged zone fractures (i.e. Model Variant 2 according to section 8.3.2). The model radius is 12 m and there is no significant difference between the case where stress coupling is included to account for the change in transmissivity due to stress redistribution and pore pressure changes.

The effective conductivity of both models is in good agreement with the calibrated network effective conductivity of $1 \times 10^{-11}$ m/s (see section 5.3).

The effective conductivities calculated for the 3.85 m radius models are given in Table 8.4b. As can be seen, the high transmissivity damaged zone fracture network is poorly connected at fracture sizes of 1 m and the effective conductivity of the damaged zone network is comparable to that of the undisturbed matrix. However, it should be remembered that the model does not include the zone of enhanced matrix conductivity which has a hydraulic conductivity of approximately $3 \times 10^{-11}$ m/s. Also at this scale the imposed boundary conditions reduce the conductivity of the natural fracture network below the typical effective conductivity of $1 \times 10^{-11}$ m/s. Therefore the damaged zone conductivity for the 1 m size fracture models is probably dominated by these two components.

The conductivities for the 10 m damaged zone fracture network are much higher and between $1 \times 10^{-8}$ and $1 \times 10^{-9}$ m/s. In general the conductivities are relatively insensitive to the stress coupling and reduced by about a factor of two when the scale of measurement is increased from 10 m to 20 m. Table 8.4c gives similar results for cylinders of 5.55 m radius with the effective conductivity of the 10 m damaged zone fracture network reducing as it is averaged over a greater radius.
### Tab. 8.4 a-b: Effective axial conductivity.

**8.4a:** for 12 m radius cylinder – no damaged zone

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* indicates model variant with larger random orientation component.
### Effective Axial Conductivity

#### 8.4c: for 5.5 m radius cylinders

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* indicates model variant with larger random orientation component

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<td>333.3</td>
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<td>9.1E-11</td>
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<td>1.6E-10</td>
<td>-9.7</td>
<td>0.2</td>
<td>9.3E-11</td>
<td>1.1E-09</td>
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* indicates model variant with larger random orientation component

<table>
<thead>
<tr>
<th>f) EDZ fracture size [m]</th>
<th>Cylinder length scale [m]</th>
<th>Stress coupling</th>
<th>Tensile aperture [mm]</th>
<th>Stress–transmissivity exponent</th>
<th>Mean Keff-axial [m/s]</th>
<th>Standard deviation Keff-axial [m/s]</th>
<th>Mean log&lt;sub&gt;10&lt;/sub&gt; Keff-axial [m/s]</th>
<th>Standard deviation log&lt;sub&gt;10&lt;/sub&gt; Keff-axial [m/s]</th>
<th>Minimum Keff-axial [m/s]</th>
<th>Maximum Keff-axial [m/s]</th>
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<td>10</td>
<td>No</td>
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<td></td>
<td>1.2E-11</td>
<td>7.9E-12</td>
<td>-11.0</td>
<td>0.4</td>
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<td>2.6E-11</td>
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<td>8.7E-12</td>
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<td></td>
<td>1.0E-10</td>
<td>3.5E-11</td>
<td>-10.0</td>
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<td>5.0E-11</td>
<td>1.7E-10</td>
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<tr>
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<td>No</td>
<td></td>
<td></td>
<td>2.4E-10</td>
<td>1.2E-10</td>
<td>-9.7</td>
<td>0.2</td>
<td>1.2E-10</td>
<td>4.9E-10</td>
</tr>
</tbody>
</table>
Table 8.4d gives effective conductivities for the portion of the network from 5.55 m to 12 m. In all cases this is close to $1 \times 10^{-11}$ m/s, indicating that no significant conductivity enhancement occurs beyond 5.55 m (2 radii from the tunnel wall).

The effective conductivities averaged over the 12 m radius cylinder are given in Table 8.4e. As can be seen the effects of changes in pore pressure and stress due to backfilling of the tunnel pressure recovery are relatively small. The major effects relate to the scale of the high transmissivity damaged zone fractures. The networks with 1 m damaged zone fractures are relatively poorly connected at scales of 10 m and the effective conductivity is on average comparable to the enhanced matrix conductivity of about $3 \times 10^{-11}$ m/s, which has not been included in the fracture network models.

The networks with 10 m damaged zone fractures show a much higher effective conductivity that declines slightly from the 10 m scale to the 20 m scale as might be expected. The effective conductivity of the network is still however significantly below the analytical upper bound suggested in section 7.4.1. The importance of the scale of EDZ features in determining axial conductivities is illustrated by the difference in effective conductivities between the two networks.

Table 8.4f presents results for the 12 m radius cylinders where no stress coupling has been used. The effective conductivities are typically a factor of 2 – 3 lower than in Table 7.4e for the 10 m damaged zone fractures and unchanged for the 1 m networks.

If we consider the results from the effective conductivity calculations in terms of conductivity enhancement factors for several annuli of rock around the tunnel, as has typically been the case in effective porous medium models of repositories (see Figure 5.1 and the equivalent porous medium calculations in Chapter 5), we can calculate suitable enhancement factors that are in agreement with the values estimated for the different annuli from the models. These are given in Table 7.5. As can be seen, the conductivity of the zone nearest to the tunnel wall including the damaged zone fracture network (as described by the two feature length scales) has a critical importance for the values of effective hydraulic conductivity.

The increased conductivity in the zone from 3.85 to 5.55 m (transition zone in Figure 5.1) may be partly due to the model boundary effects described earlier and to some interaction between the damaged zone fractures and the stress disturbed natural fractures.

### 8.5 Conclusions

The results suggest that the effective axial conductivity of the rock around the tunnel at the GTS site will be controlled by the connectivity of the high transmissivity features within the damaged zone. Where such features are extensive and well connected (compared to the scale of flow – e.g. seal zone length), the results suggest that conductivities of between $1 \times 10^{-8}$ and $1 \times 10^{-9}$ m/s are likely over the first two metres of rock. This is 100 – 1000 times greater than the undisturbed conductivity of the rock mass. If the features are considerably smaller than the flow scale the models predict a lower effective conductivity which will be dominated by the small-scale fracturing in the damaged zone. From the MMPS results the damaged zone would locally have a typical conductivity of about $3 \times 10^{-11}$ m/s (a factor of three greater than the estimated undisturbed rock conductivity).
### Post-closure axial effective hydraulic conductivity for network models and factor model

#### I. Damage zone fractures 10 m in length, tensile aperture = 100 mm and transmissivity exponent = 1

<table>
<thead>
<tr>
<th>Inner radius [m]</th>
<th>Outer radius [m]</th>
<th>Length of model [m]</th>
<th>Geometric mean conductivity [m/s]</th>
<th>Arithmetic mean conductivity [m/s]</th>
<th>Estimated conductivity [m/s]</th>
<th>Zone</th>
<th>Factor increase in conductivity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.85</td>
<td>3.85</td>
<td>10</td>
<td>3.8E-09</td>
<td>5.3E-09</td>
<td>5.0E-09</td>
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<td>1.85</td>
<td>5.55</td>
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<td>2.7E-09</td>
<td>Disturbed Zone</td>
<td>100</td>
</tr>
<tr>
<td>1.85</td>
<td>12</td>
<td>10</td>
<td>4.2E-10</td>
<td>4.9E-10</td>
<td>5.3E-10</td>
<td>Undisturbed Rock</td>
<td>1</td>
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<td>5.55</td>
<td>12</td>
<td>10</td>
<td>1.2E-11</td>
<td>1.4E-11</td>
<td>1E-11</td>
<td>Fit to 10 m flow length</td>
<td></td>
</tr>
<tr>
<td>1.85</td>
<td>3.85</td>
<td>20</td>
<td>1.3E-09</td>
<td>1.4E-09</td>
<td>1.5E-09</td>
<td>Damage Zone</td>
<td>150</td>
</tr>
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<td>1.85</td>
<td>5.55</td>
<td>20</td>
<td>7.1E-10</td>
<td>8.1E-10</td>
<td>7.4E-10</td>
<td>Disturbed Zone</td>
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</tr>
<tr>
<td>1.85</td>
<td>12</td>
<td>20</td>
<td>1.5E-10</td>
<td>1.7E-10</td>
<td>1.5E-10</td>
<td>Undisturbed Rock</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Fit to 20 m flow length</td>
<td></td>
</tr>
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#### II. Damage zone fractures 10 m in length, tensile aperture = 100 mm and transmissivity exponent = 2

<table>
<thead>
<tr>
<th>Inner radius [m]</th>
<th>Outer radius [m]</th>
<th>Length of model [m]</th>
<th>Geometric mean conductivity [m/s]</th>
<th>Arithmetic mean conductivity [m/s]</th>
<th>Estimated conductivity [m/s]</th>
<th>Zone</th>
<th>Factor increase in conductivity</th>
</tr>
</thead>
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<tr>
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<td>3.85</td>
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<td>800</td>
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<td>1.85</td>
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<td>10</td>
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<td>Disturbed Zone</td>
<td>150</td>
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<td>1.85</td>
<td>12</td>
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<td>7.0E-10</td>
<td>8.2E-10</td>
<td>8.3E-10</td>
<td>Undisturbed Rock</td>
<td>1</td>
</tr>
<tr>
<td>5.55</td>
<td>12</td>
<td>10</td>
<td>1.2E-11</td>
<td>1.4E-11</td>
<td>1.0E-11</td>
<td>Fit to 10 m flow length</td>
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</tr>
<tr>
<td>1.85</td>
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<td>2.0E-09</td>
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<td></td>
<td></td>
<td></td>
<td>Fit to 20 m flow length</td>
<td></td>
</tr>
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</table>
The results given above assume that high transmissivity intervals detected using the MMPS system correspond to macro-scale damaged zone features running along the tunnel. If however these features are locally channelled and possibly stress-relieved parts of pre-existing fractures cutting across the tunnel rather than running along its length, then effective axial conductivities will again be dominated by small-scale fracturing in the damaged zone and be closer to the $3 \times 10^{11}$ m/s.

A point to note is that the stress models predict zones of rock in a tensile state. These zones have not been hydraulically characterised because of the limited number of boreholes. The tensile stresses are predicted as a result of the high stress anisotropy predicted for the area (see section 4.2). These zones are typically at right angles to the breakout zones and so may not appear as important at the tunnel wall. Tensile stresses will lead to opening of fractures and potentially highly transmissive features (see MARTIN et al. 1990). Without detection of features within the hypothesised tensile regions it is not possible to be certain whether such features exist.
9 INTERPRETATION AND DISCUSSION

The interpretations given here relate directly to the characterisation work performed in the WT drift area at the GTS. Additional results from the investigations performed by BGR (MARSCHALL et al. 1999) have also been integrated where possible. It should however be remembered that the area characterised by the EDZ boreholes in the WT drift showed the most extensive tunnel wall damage observed within the GTS and so might be considered as relating to worst-case rather than typical properties of the EDZ in the crystalline rocks at the GTS. Stress measurements in the BGR area suggest a lower stress anisotropy than those predicted for the WT drift (see Table 9.1).

Tab. 9.1: Interpretation of stresses in the BGR investigation area.

<table>
<thead>
<tr>
<th>Stress</th>
<th>Predicted stresses at WT drift</th>
<th>Measured stresses for BGR investigations</th>
</tr>
</thead>
<tbody>
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<td></td>
<td>Magnitude [MPa]</td>
<td>Orientation dip/dip direction</td>
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<tr>
<td>σ1</td>
<td>25.7</td>
<td>18/125</td>
</tr>
<tr>
<td>σ2</td>
<td>12.9</td>
<td>32/23</td>
</tr>
<tr>
<td>σ3</td>
<td>7.7</td>
<td>52/239</td>
</tr>
</tbody>
</table>

It should also be remembered that the characterisation work in the EDZ boreholes may have been performed in two-phase conditions close to the tunnel wall and that there may therefore be some error in the derived hydraulic properties. Ideally such characterisation work should be performed in conditions close to that envisaged for the post-closure phase and considerable efforts were made to resaturate the tunnel. However the well test data suggest that some zone of partial saturation existed within the rock close to the tunnel.

Results from work in other underground laboratories in granitic rock have also been used to support the interpretations. The large amount of world-wide experience in granitic rocks in underground laboratories in Sweden, Finland and Canada is of particular relevance to this.

The ZEDEX Project was undertaken as a joint project by ANDRA, UK Nirex, SKB, BMBF and Nagra to understand the mechanical behaviour of the Excavation Disturbed Zone and its hydraulic significance. In this project the excavation with tunnel boring and different blasting methods was studied in several drifts of the Åspö Hard Rock Laboratory. The results have shown that there is an excavation damaged zone close to the drift wall dominated by changes in rock properties. Such changes are mainly irreversible and there is a excavation disturbed zone beyond the damaged zone that is dominated by changes in stress state and hydraulic head with only small changes in rock properties and mainly reversible (EMSLEY et al. 1997). There is no distinct boundary between the two zones. The role of the EDZ as a preferential pathway to radionuclide transport is limited to the damaged zone; its extent can be limited by appropriate excavation methods.
9.1 Damaged zone

9.1.1 Geological and geophysical evidence

There is evidence from the geological characterisation of the EDZ boreholes for an excavation damaged zone extent of between 0.4 to 1.5 m depth. However damaged zone fractures are visible in only the first 15 – 20 cm of the boreholes. Core data from the ZPK boreholes in the BGR experiments suggested that excavation induced fractures and microfractures were limited to the first 30 cm from the tunnel wall. Interval velocities for P and S waves measured in the ZPK boreholes also showed little change beyond 50 cm from the tunnel wall.

Overall these results appear to be in reasonable agreement with measurements in other rock laboratories at similar depths in granitic rock. Results from sonic measurements around the ZEDEX Drill and Blast and tunnel boring (TBM) drifts at Åspö (EMSLEY et al. 1997) show a low velocity zone extending about 10 – 100 cm from the drifts. The greatest reduction in velocity occurs within the first 10 cm for the TBM drift and within 30 cm for the drill and blast drift.

Results from mapping in the AECL Underground Research Laboratory (URL) shaft in Canada suggest a zone of microfracturing up to 30 cm around shaft. The stress conditions at the URL are however more extreme than those encountered at the GTS. Also the shaft was excavated using drill and blast methods.

There appears to be a consistent body of evidence suggesting that excavation induced fractures occur only in the first 30 cm to 1 m around most excavations (both TBM and careful drill and blast). The results from the heater test drift (WT) suggest a slightly deeper zone at some points around the excavation.

9.1.2 Hydraulic properties

There appears to be a reduction in matrix conductivity between the inner test intervals (within 1 – 2 m of the tunnel wall) and the outer test intervals in both the EDZ and ZPK boreholes. However there are no measurements between 1 and 2 m from the tunnel wall in the EDZ boreholes. In addition to this there is no clear trend in the hydraulic properties with depth in the first 1 m of the EDZ boreholes. This may partly be due to the influence of natural fractures. In addition the use of low disturbance excavation methods in a strong rock may result in hydraulic changes in the damaged zone that are small and hence difficult to identify where no data from before excavation are available.

A similar conclusion was reached at Åspö where the ZEDEX Experiment (EMSLEY et al. 1997) found no well defined increase in permeability in the damaged zone and no significant difference between TBM and drill and blast drifts. Data from the excavation of full scale deposition holes at Olkiluoto (AUTIO 1996) suggest a region of increased matrix conductivity by about a factor of 50 in the first 20 cm of the core.

Positive evidence for a considerably more conductive damaged zone is reported in PUSCH and STANFORS (1992) and CHANDLER et al. (1996). Both cases relate to drill and blast tunnels. PUSCH & STANFORS (1992) describe experiments at the Stripa BMT site where considerable thermal disturbance had occurred as a result of heater experiments. They estimated that there was a shallow 0.75 m zone around the drift with an effective conductivity of $1.2 \times 10^{-8}$ m/s, with a somewhat higher conductivity in the floor of the drift. This was approximately 2 – 3 orders of magnitude over the estimated background rock conductivity.
Two "connected permeability" experiments have been performed at the URL in Canada. In both experiments fluid is held behind a dam within the tunnel and allowed to flow through the floor and damaged zone into a slot cut in the floor. CHANDLER et al. (1996) demonstrated that the connected permeability in the Room 209 experiment was limited between blast rounds so that permeabilities measured over 4 m length indicated permeabilities of one fiftieth of that measured over 2 m. They estimate an effective conductivity of $1 \times 10^{-8}$ m/s for the 4 m flow path length. This is an increase of five orders of magnitude over the estimated intact rock conductivity of $1 \times 10^{-13}$ m/s. The second connected permeability experiment was performed at greater depth in the Mine-By tunnel and flow was concentrated in a small notch of failed material. The notch is caused by a breakout process due to the very high stress anisotropy (6:1). The conductivity of this notch zone was estimated as $1 \times 10^{-6}$ m/s. The large factor increase in conductivity of the damaged zones at the URL is due to the very low fracture density and permeability of the undisturbed rock in the URL at this depth.

The various measurements are summarised in Table 9.2 for experiments where axial conductivity has been measured and where multiple point measurements only are available.

Tab. 9.2: Damaged zone effective conductivities.

<table>
<thead>
<tr>
<th>Damaged zone extent</th>
<th>Effective conductivity</th>
<th>Factor increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stripa BMT Drift</td>
<td>$1.2 \times 10^{-8}$ m/s in walls and roof&lt;br&gt;$2 \times 10^{-8}$ m/s in floor</td>
<td>200 – 1000</td>
</tr>
<tr>
<td>URL Room 209 (4 m flow path)</td>
<td>$10^{-9}$ m/s probably less in roof and walls</td>
<td>100,000</td>
</tr>
<tr>
<td>URL Mine-By 0.05 m² notch zone</td>
<td>$10^{-6}$ m/s</td>
<td>10,000,000</td>
</tr>
</tbody>
</table>

Multiple Point Measurements of conductivity in the damaged zone

| ZEDEX | No detectable change in matrix permeability, no clear pattern in fracture permeability<br>SEPI testing ranged from $1 \times 10^{-16}$ to $1 \times 10^{-15}$ m²<br>($1 \times 10^{11}$ to $1 \times 10^{9}$ m/s) with highest values within 20 cm of tunnel wall |
| GTS WT drift | 1 – 2 m thick zone | locally $1 \times 10^{-11}$ to $3 \times 10^{-7}$ m/s<br>Arithmetic mean $2 \times 10^{-8}$ m/s<br>Geometric mean $7 \times 10^{-11}$ m/s | 7 – 2000 |
| Olkiluoto core from deposition holes | 20 cm thick | factor 50 increase in matrix conductivity |

9.1.3 Suggested model for the GTS

There is evidence of an increase in matrix conductivity within 1 – 2 m of the tunnel wall by a factor of 10 from both the EDZ and ZPK boreholes. This seems consistent with other observations given that matrix conductivity will rarely be the major component of rock mass hydraulic conductivity.
It would appear that damaged zone fractures are confined to a relatively thin shell around the tunnel with a thickness of approximately 30 cm. The zone may be deeper in borehole EDZ95.002 and EDZ95.003 where core recovery is low.

The most probable explanation for the higher conductivity intervals in borehole EDZ95.002 is that they correspond to channels within a pre-existing and possibly stress disturbed fracture. However problems with core recovery make it impossible to be sure that they do not relate to damaged zone features. Given such uncertainties, the suggested model would be that the high conductivity intervals relate to the natural fractures but an alternative less probable model would be that they relate to a set of damaged zone features that cannot be identified because of the poor core recovery. The suggested model is illustrated in Figure 9.1. It should be noted that we expect the EDZ to be heterogeneous and that zones of largely undisturbed rock may exist within the EDZ or the smaller damaged zone within it. It is therefore necessary to identify an "envelope" for these zones rather than some volume that only contains disturbed or damaged rock (Figure 9.2).

9.2 Excavation Disturbed Zone

Very little data are available for the properties of the disturbed zone at the GTS. The results from the three 0.5 m intervals tested show a relatively consistent hydraulic conductivity of about $3 \times 10^{-12}$ m/s. This is in reasonable agreement with estimates of matrix hydraulic conductivity, indicating that disturbed zone effects on the matrix are small.

With regard to the effect of tunnel excavation on natural fractures away from the damaged zone no data are available. Even if fractures had been found in the 0.5 m zone, the lack of before and after comparisons would have made any interpretation speculative. Evidence from the GTS suggests that fractures may be sensitive to pore pressure disturbance (MAJER et al. 1990) and presumably to changes in total normal stress, but that such fractures are relatively stiff (~200 GPa/m and higher).

In the modelling performed for this study a fracture transmissivity/effective normal stress power law relationship has been used. This has resulted in fracture hydraulic stiffness (i.e. the ratio of stress change to hydraulic aperture change) that is dependent on the transmissivity of the feature (and hence its hydraulic aperture), its orientation to the stress field and on the change in stresses. At low stresses, fractures are considerably less stiff (more compliant). Figure 9.3 shows stiffnesses for three fractures oriented normal to the three different principal stresses as calculated from this model. The results suggest ranges of stiffness from below 1 to above 200 GPa/m. It is possible that the models result in fractures that are too compliant when compared with the results from MAJER et al. (1990), but in the absence of more data from the GTS site it is not possible to be more definitive. Having said this, the results from the modelling show very little dependence of axial conductivity on stress relief of fractures, indicating that disturbed zone stress effects are small. This is in agreement with results from other modelling studies and the comments of OLSSON & WINBERG (1996).
Fig. 9.1: Model variants and preferred model.

(a) Model variant 1

(b) Model variant 2

Preferred model high transmissivity intervals are channels in natural fractures and some EDZ fracturing close to tunnel wall.

Diagram of BGR investigations (ZPK area see Fig. 1.5; MARSCHALL et al. 1999)
Fig. 9.2: Conceptual model of the EDZ.

Fig. 9.3: Fracture hydraulic aperture stiffness for $1 \times 10^{-9}$ m$^2$/s, fractures oriented normal to each of the three principal stresses for two values of power law exponent.
9.3 Conceptual model of the tunnel near-field

9.3.1 Damaged zone

All mechanical methods of excavation are likely to create a zone of damage local to the excavation, where material properties of the rock are significantly affected and where new fractures are created. The size of this damaged zone will be a function of the rock properties, the in-situ stresses and the excavation method. In moderate stress environments in hard rocks the dominant factor is likely to be the excavation method and excavation methods that minimise damage can be selected (e.g. TBM or cautious drill and blast). When such methods are employed it has been shown at a variety of sites that the damaged zone will typically extend not more than a metre into the rock, with a slightly deeper penetration into the excavation floor where higher charge weights are used in drill and blast.

In highly stressed environments such as the URL in Canada, the stress redistribution effects can be dominant (see for example the failure zone associated with the Mine-By tunnel). In such environments excavation methods may still have some influence, but techniques that accommodate the stresses by selecting the most favourable orientations and excavation aspect ratios must be used (see for example READ 1996).

Delineation of the damaged zone requires both geological and geophysical characterisation. An integrated programme of core characterisation, borehole imaging and geophysical logging (e.g. interval sonic measurements) should deliver a robust description of the damaged zone envelope. Unfortunately the poor core recovery and limited imaging and logging capability in the EDZ boreholes make the results from the current phase of characterisation uncertain.

While the damaged zone may be dominated by changes in material properties and the creation of new fractures, it is also the area where environmental changes in stress, pore pressure, temperature and saturation are likely to be greatest. The changes in rock properties and creation of fractures require that the damaged zone is explicitly characterised as it is likely to have different properties from the undisturbed rock. However the potentially large change in environment between characterisation and post-closure mean that it is important to perform the damaged zone characterisation in ways that minimise such changes. Thus it is important to ensure fully saturated conditions and pore pressures close to those expected at resaturation so that uncertainties associated with these changes are minimised.

The scale of the damaged zone requires relatively small-scale characterisation compared to the undisturbed rock. It is therefore likely that details of the small-scale properties of the rock will be important in understanding the results of the damaged zone characterisation. Thus it is important that, in order to reduce uncertainties, both small-scale radial investigations and larger scale axial measurements are made. In the modelling described above it has been shown that the axial extent of the high conductivity features within the damaged zone controls the estimate of axial conductivity.

The most conservative models suggest axial conductivities almost 3 orders of magnitude higher than models where the high conductivity elements are relatively small compared to the flow path length. The best way of reducing such uncertainties would be to measure the axial conductivity, which is the key property for understanding the likely flow around a repository seal zone. Figure 9.4 shows a suggested experimental geometry for such a test. An alternative geometry might use an experimental set-up similar to the Connected Permeability experiments at the URL. Pre-modelling of such an experiment suggests that it would be an excellent diagnostic of the axial conductivity.
9.3.2 Disturbed zone

The definition of the disturbed zone suggested by READ 1996 means that the hydraulic properties of the EDZ are dominated by the response of pre-existing fractures to the excavation and the disturbance that it causes. The most important disturbances are likely to be due to stress redistribution, pore pressure change, temperature change and possible saturation changes. However other changes, for example in fluid chemistry, may also be important at some sites or for particular applications. The major effect of small temperature changes will be changes in stress due to local warming/cooling and associated expansion or contraction of the rock. Thus a key question is how fractures respond to changes in stress and pore pressure. Usually these two changes can be considered as a single property: effective stress.

Data on fracture response to effective stress change at Grimsel are limited to the work by MAJER et al. (1990) and a limited amount of core testing. For this reason generic fracture transmissivity/stress laws have been used in this study. However investigations for any repository site will require both core and in-situ testing to derive a suitable model of fracture behaviour. In most applications it will be the normal dilation behaviour that is most important and fracture normal stiffness will be the key property that determines the magnitude of hydraulic changes in the disturbed zone. This property is also important in determining fracture storativity and hence the overall diffusivity of any low porosity fractured rock.

Results from the numerical experiments and from other work (VOMVORIS et al. 1997 and OLSSON & WINBERG 1996) suggest that unless the dominant natural fractures are oriented sub-parallel to the excavation, disturbed zone effects will be small when measured over distances longer than an individual feature. This is because flow occurs along the fracture plane.
and hence is likely to pass through zones of both stress increase and decrease as long as it is not oriented in a pathological tunnel parallel geometry. It should be pointed out that such pathological geometries are forced by the usage of two-dimensional models of tunnel excavations which can over-emphasise the importance of the disturbed zone (see for example PUSCH & STANFORS (1992), who suggest that such 2D models may over-estimate axial conductivity by one or more orders of magnitude).

9.4 Radial flow properties

While we have restricted ourselves to considering axial flow around a seal zone, it should be remembered that there is evidence for considerable anisotropy in the damaged and disturbed zone that may influence behaviour during the operational and post-closure phases.

Axial flow will be most important around seal zones or other situations where the tunnel has been filled with low permeability material. Emplacement caverns for low- and intermediate-level wastes may contain a relatively permeable backfill or even an open void space, as was planned by UK Nirex for the deep repository at Sellafield (NIREX 1995). In such circumstances the EDZ flow properties will be important if they either promote or restrict flow through the waste. A high radial conductivity might focus flow through the permeable backfill, while axial conductivity might be relatively unimportant.

Evidence for reductions in radial conductivity due to the presence of an excavation is relatively well documented and summarised below.

9.4.1 Reduced inflow to tunnels

There has been a range of experiments that have all suggested that inflows to drifts are smaller than might be expected. The most well known experiment is probably the Stripa SCV drift where inflows to a 50 m drift were a factor of 8 below that of the equivalent borehole array prior to excavation.

Similar results have been reported for the BMT drift at Stripa (PUSCH & STANFORS 1992), where the authors suggest that the radial conductivity might be one or two orders of magnitude below the axial conductivity. Similar results have also been reported for drifts at Olkiluoto (AUTIO 1996). This has been related to the presence of a hydraulic skin around excavations corresponding to a reduction in radial permeability.

A reduction in transmissivity was also reported for a conductive fracture in the Room 209 Excavation Response Experiment at the URL in Canada (LANG 1988). There, a single conductive feature was instrumented (the Room 209 Fracture) prior to excavation and its response was monitored while a tunnel was excavated through the feature. Although most measurements of hydraulic aperture showed some change, this was usually followed by recovery to pre-excavation conditions. However a permanent reduction in hydraulic aperture was measured in the part of the fracture in the roof of the tunnel.

Radial fractures might be expected to experience fracture closure due to increased hoop stresses (see for example KELSALL et al. 1984). This might lead to situations where axial conductivity was enhanced but radial conductivity reduced. However for the SCV drift at Stripa, LONG et al. (1992) have suggested that the reduction in inflow relates to a zone where two-phase flow occurs due to groundwater degassing. If this is the case the effect relates only to the operational
phase and is not relevant to post-closure and any measurements of the EDZ should be performed in a way that minimises such effects.

To summarise, while a reduction in radial conductivity due to excavation is relatively well documented, its association with transient saturation conditions probably makes it difficult to take credit for such a situation in any assessment of post-closure flow.
10 CONCLUSIONS AND RECOMMENDATIONS

The work on the GTS stress field documented in Chapter 4 provides a basis for understanding the in situ stress throughout the GTS site. It is a significant development in the use of calibrated three-dimensional models for understanding stresses at locations where surface relief cannot be ignored. The work should prove useful in this and future phases of the GTS.

The design calculations were an important input to the measurement programme and allowed the development of a successful hydraulic characterisation of the EDZ. The ability of the continuum models to demonstrate the sensitivity and usefulness of specific choices of test procedure together with the ability of the fracture network models to consider the expected variability provided a firm basis for the operational decisions.

The results from the characterisation work at the GTS have been useful in providing a detailed small-scale hydraulic characterisation of the damaged zone around the tunnel. The poor core recovery and difficult imaging conditions have limited the amount of geological data that were acquired and hence the ability to synthesise the hydraulic data with the geological characterisation. In addition the lack of characterisation prior to excavation increases the uncertainty in the effects of excavation.

The following statements can be made from the hydraulic characterisation:

- All intervals within 1 m of the tunnel wall showed higher conductivity than that expected for the undisturbed matrix.
- The conductivity appears to be bi-modally distributed with a small number of more conductive intervals that are associated with a pre-existing fracture.
- The zone closest to the tunnel appears to be at near atmospheric pressure and partially saturated conditions may exist close to the tunnel.
- Intervals at 2 m from the tunnel wall have conductivities close to that estimated for the undisturbed matrix.

The results from the work have been taken forward into modelling of post-closure flow through the EDZ. However without better geological understanding and integration with the hydraulic data it is only possible to suggest bounds for the post-closure effective axial conductivity. The results of modelling suggest values of axial conductivity from $3 \times 10^{-11}$ to about $6 \times 10^{-8}$ m/s, the higher value corresponding to a very conservative interpretation of the data. The major control on axial conductivity is the extent and connectivity of high transmissivity features within the damaged zone.

It is possible that better geological characterisation could reduce the uncertainty. For example, if it could be shown that the high conductivity intervals correspond to pre-existing features that are not oriented parallel to the tunnel, then effective axial conductivity is likely to be significantly smaller than the upper bound given above.

Ideally in-situ hydraulic measurements of axial conductivity should be performed in conditions similar to those that will exist post-closure (see e.g. Fig. 9.4). If such measurements are made, this removes the necessity to extrapolate from data that may have been significantly affected by the presence of the open tunnel.
The results of the EDZ experiment and also other experiments in the tunnel near-field at GTS indicate that no measurable axial conductivity due to a connected higher transmissivity fracture network in general exists. Therefore the setup of an axial flow experiment (e.g. Fig. 9.4) for the determination of the effective axial conductivity must be carefully evaluated and should be done, where the geological, hydrogeological and rock-mechanical boundary conditions for such a test are better suited. However, the developed methodology and techniques of the EDZ experiment at GTS with the feasibility to detect very small changes of the EDZ properties could be easily transferred to other locations and/or host rocks.

With regard to the aims of the EDZ experiment set out in Chapter 1, the following assessments are given.

**Development of a site-independent method for identification of the EDZ**

The use of detailed scale hydraulic testing in cored radial boreholes has allowed the identification of unloading fractures close to the tunnel and a region of higher matrix conductivity around the tunnel. It is however difficult to assess the extent to which the hydraulic properties of the natural fracture that run along EDZ 95.002 were altered by the excavation. This is due to the inherent variability of fracture flow and the small scale of the EDZ hydraulic measurements. In fractured rock where axial flow structures might exist a measurement of axial flow is required.

**Determination of EDZ extent**

The MMPS has allowed the determination of the outer limits of the EDZ, however the small number of boreholes did not provide sufficient coverage around the tunnel. With testing in additional radial boreholes the modular system can achieve this objective.

**Radial hydraulic permeability**

The test results from the MMPS have provided profiles of permeability. However, the small number of boreholes and the selected geometry with the 1 m guard zone between 1 and 2 m from the tunnel wall showed some uncertainties as to the distribution of radial permeability. A second issue relates to the distribution of permeability very close to the tunnel within 10 – 30 cm of the tunnel wall where "unloading fractures" had been identified. However the modular nature of the MMPS together with developments such as the BGR surface packer and/or a combined system of short interval packers with the surface packer system as already applied and tested at the ZPK site at GTS (MARSCHALL et al., 1999) means that the aim is achievable.

**Testing and development of equipment**

The testing and development of the MMPS was achieved successfully during the EDZ experiment in crystalline rock. With later established modifications it was possible to use the equipment successfully in different environments (clay, limestone, sandstone, tuff) for the hydraulic characterisation of the EDZ. The acoustic emissions systems were successfully deployed and the recording and analysis identified candidate rock fracture events. However further development is required to understand the signals from the system and to provide location estimates.
Summary and conclusions

In general the experimental aims have been met, although budgetary restrictions have limited the work in terms of the number of boreholes that it was possible to instrument and test. The work suggests that the equipment and methodology developed will be suitable for determining the hydraulic properties of the EDZ in similar rocks if additional work is performed to determine axial conductivity via experiments such as those suggested in the report (see Fig. 9.4). The following points are, according to the experience gained, very important for the set-up of any new EDZ experiment:

- Site-specific conceptualisation of the EDZ (e.g. hydraulic, rock mechanical) and basic understanding of the tunnel near-field processes (e.g. rock alteration).
- Performance of design calculations prior to the finalisation of the design of the experiment, taking into account the applied excavation technique, which will influence the size of the EDZ.
- Creation of controlled boundary conditions for the EDZ experiment (e.g. fully saturated, constant head boundary) to avoid for example any influence by the open tunnel to ensure adequate investigations and solely testing of the EDZ.
- Execution of axial hydraulic tests with reasonable gradients.
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Structural and lithological map of the heater test tunnel, excavation disturbed/damaged zone between 75 and 88 m

Legend:
- Excavation disturbed/damaged zone
- Brittle structures
  - Fractures
    - + hanging wall
    - - foot wall
- Ductile structures
  - Cleavage trace
  - Mineral veins
- Lithologies
  - Central Aare Granite (CAGr)
  - Xenoliths and mafic inclusions
  - bright / dark CAGr
  - Aplite
  - Lamprophyre
  - Fluorescent exudations (Schröckingerit)
- Drillholes
  - deformation measurements (carried out)
  - EDZ94.001
  - EDZ94.002

Scale = 1 : 100

DAT.: Feb. 2000
Central Aare Granite (CAGr)
Fluorescent exudations (Schröckingerit)
EDZ95.994
EDZ95.003
EDZ95.002
EDZ95.001
Drillholes
Lamprophyre
Aplite
Xenoliths and mafic inclusions
bright / dark CAGr
Lithologies
Brittle structures
Cleavage trace
Mineral veins
Ductile structures
Cleavage trace
Mineral veins
Brittle structures
Fractures
Legend:
Excavation disturbed/damaged zone
Breakouts on tunnel surface
Strongly altered fabric
Moderately altered fabric
Weakly altered fabric
Sigmoidal cavities on tunnel surface
Sigmoidal cavities on tunnel surface
Fractures
* hanging wall
* foot wall
DAT.: Feb. 2000

EDZ95.001
EDZ95.002
EDZ95.003
EDZ95.994
EDZ95.005 (acoustic emission)
EDZ95.006 (acoustic emission)