Investigation of Stress State and Rock-Mechanical Behaviour of Backfill and Host Rock in Dependence on Temperature (BAMBUS-II Project)
Investigation of Stress State and Rock-Mechanical Behaviour of Backfill and Host Rock in Dependence on Temperature (Contribution to the BAMBUS-II Project)

Final Report

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1 BACKGROUND

The BAMBUS-II project (Backfill and Material Behaviour in Underground Salt Repositories, phase II) was a joint undertaking of nine partners from five countries:

FZK  Germany (coordinator)
BGR  Germany
DBE  Germany
G.3S  France
GRS  Germany
IfG  Germany
NRG  The Netherlands
UPC  Spain
USDOE  USA.

It started on 1 August 2000 and it lasted 33 months till 30 April 2003. The project was funded by the European Commission and the national governments and authorities.

The project aimed at the behaviour of backfill and Excavation Disturbed Zone (EDZ) in a geological repository for heat generating radioactive waste in rock salt. Its objective was to increase the knowledge on important phenomena and processes in and around backfilled drifts and to strengthen the scientific basis required for repository design and performance assessments. The project consisted of four major parts:

- in-situ investigations,
- laboratory studies,
- modelling studies, and
- desk studies.

The in-situ investigations were performed in the Asse salt mine. These investigations were addressed to the consolidation of backfill material in heated emplacement drifts and to the properties of the Excavation Disturbed Zone around drifts and emplacement rooms in the repository.

To support the so-called drift emplacement concept, in the TSDE experiment (Thermal Simulation of Drift Emplacement) in two drifts electrically heated mock-ups of disposal casks were localized. The drifts with a height of about 3.5 m and a width of 4.5 m were backfilled using crushed salt. Weight, dimensions and heat capacity of the containers as well as the drift dimensions matched the preliminary layout of the drift emplacement concept. The heating period started on 25 September 1990 and terminated on 1 February 1999. During this time temperatures, stresses, rock deformation, gas generation, backfill compaction and permeability were measured. The in-situ measurement results were compared with calculation results of different computer codes.

One of the two TSDE experimental drifts was dismantled after more than eight years of heating and subsequent cooling down to temperatures low enough to allow working in the drift. The backfill material was removed and the actual conditions of backfill and surrounding rock were studied. The studies of backfill and rock-salt behaviour were completed by specific studies of the EDZ around the drift.
Whereas in situ mainly the complex performance of backfill, rock salt and EDZ was investigated, specific parameter values like compaction behaviour, permeability, thermal conductivity were measured in laboratory tests. In these tests, material parameters were obtained under simulated in-situ conditions and at progressing compaction. In a Laboratory Benchmark, selected tests were conducted under almost equal laboratory conditions.

Corrosion studies were carried out on material samples placed on the central heater cask at temperatures between 170°C and 200°C and in the backfill at temperatures of about 100°C. The samples were retrieved and examined in laboratory tests. The aim was to evaluate the corrosion behaviour of potential container materials under conditions similar to those expected in the repository.

In order to validate the data obtained during the TSDE experiment performance and to evaluate the reasons for instrument failures, instruments located on the heater casks, in the backfill, and in the surrounding rock salt were retrieved and the recorded data from the TSDE heating phase were compared with the actual conditions in the drift. Based on the instrument performance evaluation, recommendations were made for instrumentation in future repositories.

A key issue of the project was to develop further material models and computer codes by comparing calculated results with the outcome of the studies in the dismantled drift and accompanying laboratory studies. The participants refined their models and codes and reduced uncertainties in describing the relevant processes. The data obtained from the experiments were used to calibrate and refine the constitutive laws needed for predicting the mechanical and hydraulic behaviour of the backfill and the rock in the vicinity of emplacement drifts. By comparing calculation results with experimental data the predictive capability of the numerical models was assessed.

In a desk study the boundary conditions for the retrievability of highly active waste and spent fuel were investigated. The consequences of the accessibility of the waste during the retrievability period was assessed.

The investigation results of all partners are summarized in the final report BAMBUS-II (Bechthold et al., 2003). The contributions of IfG Leipzig are presented in the following report. IfG Leipzig was participated in the working packages:

- Determination of hydraulic parameters and of backfill compaction behaviour
- Laboratory investigations on strength and deformation behaviour of rock salt
- Rock stress measurements in situ using the hydraulic fracturing method
- Modelling studies analysing the thermo-mechanical interaction of the TSDE-experiment

The research activities were financially supported by the European Commission and the German Federal Ministry of Economics (FKZ 02 E 9259).
2 BACKFILL PERFORMANCE

2.1 Introduction

Whereas in-situ tests were carried out to study the complex interaction of backfill compaction, drift convergence and stress redistribution in the surrounding rock salt, laboratory tests on samples from the dismantled test drift were used to measure specific properties, such as porosity, permeability and compaction state. The question is, how the backfill was compacted during the more than eight years of heating and how the hydraulic parameters changed. In laboratory tests material parameters can be obtained under defined boundary conditions for studying the mechanical material response.

Furthermore, important topics are the magnitude of backfill pressure at in-situ conditions and the ratio between axial and lateral stress in the backfill. The in-situ pressure build-up depends on the closure rate (the lower the rate the lower the pressure). In the laboratory, low compaction rates comparable to those observed in situ are difficult to achieve. Therefore, the backfill compaction under in-situ rates must be extrapolated from the higher laboratory rates.

To find the reasons for differences in the material characterization between different laboratories, selected tests were conducted as a laboratory benchmark under conditions as equal as possible among the participating laboratories. The detailed investigation course in each lab was prescribed regarding the agreed procedure, own experiences and in dependence on the available computer codes.

2.2 Laboratory tests on drilled backfill cores

2.2.1 Borehole location and sample preparation

For the post-test analyses in the laboratory, samples of the in situ compacted backfill were taken from different cross sections in the heated and unheated area. Figure 2.1 shows the layout of the tests drifts on 800-meter level and the monitoring cross sections and Figure 2.2 depicts cross section I1 including the boreholes for backfill drilling. For investigation of porosity, permeability and compaction behaviour in dependence on stress state backfill cores from the horizontal boreholes K 7, K 8 and K 9 were provided. They were drilled in the northern drift B. The cores were taken in a borehole depth of about 27 m to 34 m. The boreholes were located adjacent to the first heater cask.

![Figure 2.1: Test drifts with monitoring cross sections (Bechthold et al., 2003)](image-url)
Test drift B - BAMBUS II

Cross section MQ I1

- Boreholes GRS:
  - Core diameter: 101 mm
  - Length: 19.3 m
  - B. started 16.3 m from cross section I1
  - K3
  - Core diameter: 280 mm
  - Length: 3.5 m
  - K4
  - K5
  - K6

- Boreholes BGR / IfG:
  - Core diameter: 100 mm
  - Length: 3.0 m
  - K7
  - K8
  - K9

Figure 2.2: Cross section I1 of drift B and locations of borehole drilling
(Bechthold et al., 2003)

The nominal diameter of the drill cores was approximately 100 mm. Because of the irregular shape of the outer surface of the cores (see Figure 2.3), the actual diameters varied between roughly 97 mm and 101 mm. The test specimens for the laboratory investigations were manufactured by cutting the ends flat and parallel and smoothing the ends and sides using a paste made of halite powder and a clue. The high quality of the samples can be seen in Figure 2.4.

Figure 2.3: Drilled backfill samples before smoothing of the mantle surface
2.2.2 Measurement of porosity and permeability

All specimens produced for the later strength and permeability tests are presented in Table 2.1. The average density of the samples before the tests was \( \rho = 1.673 \pm 0.022 \text{ g/cm}^3 \) with extreme values of 1.639 and 1.713 g/cm\(^3\). The porosity was calculated on basis of the halite grain density of 2.187 g/cm\(^3\). This leads to a mean value of porosity \( \phi = 0.235 \pm 0.010 \) with extreme values of 0.217 and 0.251.

Table 2.1: Density and calculated porosity of samples from boreholes K7, K8 and K9

<table>
<thead>
<tr>
<th>Sample number</th>
<th>( \rho ) (g/cm(^3))</th>
<th>( \phi ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>K 7/5</td>
<td>1.648</td>
<td>24.65</td>
</tr>
<tr>
<td>K 7/1</td>
<td>1.688</td>
<td>22.82</td>
</tr>
<tr>
<td>K 7/3</td>
<td>1.650</td>
<td>24.55</td>
</tr>
<tr>
<td>K 7/4</td>
<td>1.639</td>
<td>25.06</td>
</tr>
<tr>
<td>K 8/10</td>
<td>1.665</td>
<td>23.87</td>
</tr>
<tr>
<td>K 8/9</td>
<td>1.682</td>
<td>23.09</td>
</tr>
<tr>
<td>K 8/7</td>
<td>1.674</td>
<td>23.46</td>
</tr>
<tr>
<td>K 8/6</td>
<td>1.661</td>
<td>24.05</td>
</tr>
<tr>
<td>K 8/8</td>
<td>1.688</td>
<td>22.82</td>
</tr>
<tr>
<td>K 9/11</td>
<td>1.684</td>
<td>23.00</td>
</tr>
<tr>
<td>K 9/14</td>
<td>1.650</td>
<td>24.55</td>
</tr>
<tr>
<td>K 9/12</td>
<td>1.708</td>
<td>21.90</td>
</tr>
<tr>
<td>K 9/13</td>
<td>1.713</td>
<td>21.67</td>
</tr>
</tbody>
</table>

Table 2.2: Density and porosity in dependence on height above drift floor

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Height above floor (m)</th>
<th>Mean value density (g/cm(^3))</th>
<th>Mean value porosity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>K7</td>
<td>2.8</td>
<td>1.656</td>
<td>24.27</td>
</tr>
<tr>
<td>K8</td>
<td>2.3</td>
<td>1.674</td>
<td>23.46</td>
</tr>
<tr>
<td>K9</td>
<td>1.2</td>
<td>1.689</td>
<td>22.78</td>
</tr>
</tbody>
</table>
In Table 2.2 a slight tendency of higher compaction with lower height of the boreholes above drift floor is visible with increasing density and decreasing porosity.

The permeability measurements are based on the determination of the volumetric flow of a fluid or gas in a medium with a given porosity per time unit under a certain pressure assuming a simple flow geometry like homogeneous or radial flow. Each of both, fluid or gas, has special advantages or disadvantages. Fluids are incompressible, their disadvantage is characterized by higher viscosity values. Additionally, the most fluids show a behaviour which is not completely inert. In contrast, gas has the advantage of inert behaviour due to the lower viscosity. The disadvantage of gas is given by a lower compressibility.

The permeability tests at IfG Leipzig were performed using nitrogen. While the tests the samples were loaded in a range of the in-situ pressure. A triaxial cell was used applying the axial load \( \sigma_1 \) and the confining pressure \( \sigma_3 \) independently from each other. In Figure 2.5 the equipment including the triaxial cell is presented as a scheme.

![Permeability test configuration under triaxial loading conditions](image)

Figure 2.5: Permeability test configuration under triaxial loading conditions

The gas pressure at the high pressure side is generated by a pneumatic amplifier to the necessary value \( p_1 \), which is constant and ranged in the in situ relevant magnitude. The gas pressuring system consists of an oil pump and a pressure storage. The pressure storage (bladder accumulator) mediates between the oil pump and the pneumatic system.
All samples were sealed at the top and down surface with a 4 mm thick brass filter plate and O-rings. The gas penetration takes place through the lower brass disk in direction to the upper disk. The pressure increase at the low pressure side is observed in relation to time using a gas meter.

The loading conditions axial stress $\sigma_1$ and confining pressure $\sigma_3$ were defined in dependence on the in-situ backfill stress regime. The stress conditions were applied in a confining pressure range 2 MPa, 3 MPa and 4 MPa. The axial load had a magnitude of $1.3 \cdot \sigma_3$. Simultaneously to the pressure built up, a temperature compensation took place. The nitrogen pressure $p_1$ was 1.3 bar in a range between 1.0 and 1.5 bar.

The permeability $K$ is calculated on basis of the two pressure values $p_1$ (nitrogen pressure) and $p_2$ (air pressure) and the gas flow rate $\dot{V}$ assuming Darcy flow:

$$K = \frac{2 \cdot \mu \cdot l}{A} \cdot \dot{V} \cdot \frac{p_2}{p_1^2 - p_2^2}$$

(2.1)

$l$, $A =$ height and cross section of the test specimen
$\mu = 1.74 \cdot 10^{-5}$ Pa · s (dynamic viscosity of the nitrogen at room temperature)

In Table 2.3 it is visible that the permeability is not dependent on the compression state or porosity. The scattering range is to high. In result of the loading conditions while the lab test, the density was raised and porosity decreased.

Table 2.3: Permeability of the cores from borehole K 7, K 8, K 9

<table>
<thead>
<tr>
<th>Specimen Number</th>
<th>Density $\rho$ (g/cm³)</th>
<th>Porosity $\phi$ (%)</th>
<th>$K$ (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>K 7/4</td>
<td>1.661</td>
<td>24.05</td>
<td>$5 \cdot 10^{-13}$</td>
</tr>
<tr>
<td>K 8/6</td>
<td>1.674</td>
<td>23.46</td>
<td>$6 \cdot 10^{-14}$</td>
</tr>
<tr>
<td>K 8/8</td>
<td>1.704</td>
<td>22.09</td>
<td>$7 \cdot 10^{-14}$</td>
</tr>
<tr>
<td>K 9/13</td>
<td>1.727</td>
<td>21.03</td>
<td>$3 \cdot 10^{-13}$</td>
</tr>
</tbody>
</table>

**2.2.3 Determination of the ratio $\sigma_h/\sigma_v$ equivalent to oedometer like conditions**

In the usual laboratory tests conducted on backfill in oedometers or compaction cells, the material is compacted in the axial direction by applying a vertical stress $\sigma_v$ using the active loading capabilities of the testing machine. In contrast, the horizontal stress $\sigma_h$ is induced rather than applied by the stiff chamber wall of the oedometer due to prevented lateral deformation. In relatively flat cells (for instance in oedometer cells with a diameter/height ratio of about 2) both stresses are of the same magnitude. Such a geometrical shape is an exception under in-situ conditions however and $\sigma_h$ is expected to have a lower magnitude than $\sigma_v$. The knowledge of the stress ratio $\lambda = \sigma_h / \sigma_v$ is relevant for evaluation of the backfill resistance to maintain stability and tightness of the host rock.
To understand the effect of the stress ratio, 9 cylindrical specimens (3 from each borehole K7, K8 and K9) were deformed at ambient temperature (≈ 30 °C) with constant stress rates $\dot{\sigma}_h = 0.01$ MPa/s and $\dot{\sigma}_v = \frac{\dot{\sigma}_h}{\lambda}$. In these tests the ratio $\lambda = \frac{\dot{\sigma}_h}{\dot{\sigma}_v}$ was defined to be 0.500, 0.435 and 0.400. The magnitudes $\lambda_{\text{defined}}$ were established on the basis of experiences with earlier backfill tests. The purpose of the procedure of the stress driven compaction tests was to find such a stress ratio $\lambda$, where the cross section of the test specimens remains nearly constant. This condition of constancy of the cross section corresponds with the loading on the backfill material in the oedometer or compaction cell. In contrast to both, in a stress driven compaction test no wall friction occurs.

To calculate the cross section of the samples, the diameters were measured along two azimuths and at intervals of 1 cm along the heights of the test specimen (19 levels). On the basis of mean values before and after loading, the area change $\Delta A$ expressed as a percentage was calculated.

Table 2.4 shows the changes in cross section $\Delta A$ measured for the three stress ratios $\lambda_{\text{defined}}$. The factor $\lambda$ corresponding to $\Delta A = 0$ can be estimated using a linear regression of the three measurement points.

Table 2.4: Determination of the ratio $\lambda = \frac{\sigma_h}{\sigma_v}$ corresponding to no lateral deformation

<table>
<thead>
<tr>
<th>Borehole K 7</th>
<th>Borehole K 8</th>
<th>Borehole K 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\lambda_{\text{defined}}$</td>
<td>$\Delta A / %$</td>
<td>$\lambda_{\text{defined}}$</td>
</tr>
<tr>
<td>0.500</td>
<td>-0.7073</td>
<td>0.500</td>
</tr>
<tr>
<td>0.435</td>
<td>+0.1265</td>
<td>0.435</td>
</tr>
<tr>
<td>0.400</td>
<td>+1.0255</td>
<td>0.400</td>
</tr>
</tbody>
</table>

$\lambda = 0.450$ $\lambda = 0.468$ $\lambda = 0.480$

In conclusion, the stress ratio ranges between 0.45 and 0.48. It is assumed, that such a magnitude is to be expected in a backfill compression cell as well.

### 2.2.4 Extrapolation to the in situ relevant backfill resistance

All stress controlled tests were interrupted when the confining pressure $\sigma_h$ reached 5 MPa and 10 MPa, and relaxation phases having durations $> 24$ hours were initiated. The vertical stress drop with time can be described using the following equation:

\[
\frac{\sigma_v(t)}{\sigma_v(t = 0)} = 1 - a \cdot \ln \left(1 + \frac{t}{b}\right)
\]

(2.2)

The factor $\sigma_v(t = 0)$ is the vertical stress at the start of relaxation at the time $t = 0$. The parameters $a$ and $b$ are constants which are determined on the basis of a relaxation curve approximation. The stress rate $\dot{\sigma}_v(t)$ decreases significantly while relaxation. For the loading system the following conditions are valid:
\[
\varepsilon^\text{total} = \varepsilon^\text{elastic} + \varepsilon^\text{inelastic} \\
\dot{\varepsilon}^\text{total} = 0 \\
\dot{\varepsilon}^\text{elastic} = -\dot{\varepsilon}^\text{inelastic}
\]

(2.3)

The elastic modulus \( E_{\text{System}} \) of the loading system includes the test specimen and backfill cell pressure piston. With this procedure, the equivalent inelastic deformation rate \( \dot{\varepsilon}_v(t) \) can be calculated on basis of \( \dot{\sigma}_v(t) \) and using the relation explained above a stress extrapolation to the in situ relevant deformation rate in a range of \( \dot{\varepsilon}_1 = 10^{-10} \text{ s}^{-1} \) can be performed.

The stress-strain curves with loading rates of \( \dot{\sigma}_h = 0.01 \text{ MPa/s} \) (confining pressure) and \( \dot{\sigma}_v = 0.025 \text{ MPa/s} \) (axial stress) are exemplary shown in Figure 2.6. The included relaxation phases are visible in detail in Figure 2.7.

**Figure 2.6:** Triaxial compression test under constant stress ratio

**Figure 2.7:** Axial stress relaxation up to 28 hours
Figure 2.8 demonstrates how the extrapolation to $\sigma_v$ valid for the in situ relevant compaction rate was done.

![Graph showing extrapolation to $\sigma_v$ on basis of the conversion $\dot{\sigma}_v(t)$ to $\dot{\varepsilon}_v(t)$](image)

Figure 2.8: Extrapolation to $\sigma_v$ on basis of the conversion $\dot{\sigma}_v(t)$ to $\dot{\varepsilon}_v(t)$

Assuming, that the cross section remains constant during the deformation, on basis of axial deformation and porosity at the beginning of the tests the porosity can be calculated.

In Figure 2.9 the confining pressures 5 MPa and 10 MPa (circles) and the corresponding vertical stresses (triangles) at the start of relaxation are presented. The backfill pressure $p$ (squares) is the calculated mean value $(\sigma_v + 2\sigma_h)/3$.

![Graph showing backfill stresses in stress controlled triaxial tests](image)

Figure 2.9: Backfill stresses in stress controlled triaxial tests
Using the procedure outlined above the extrapolation of the backfill stresses to relevant in situ convergence rates in the order of $10^{-3}$ per year ($10^{-10}$ per second) leads to Figure 2.10.

![Figure 2.10: Estimated stresses for relevant in situ convergence rates](image)

Both figures represent the dependence of backfill stresses on low porosity, a behaviour that will be achieved in the post operating phase. Figure 2.10 describes a reasonable and well founded extrapolation of the compaction behaviour. Such a relation between backfill stress and porosity could be observed in the TSDE experiment.

### 2.3 Laboratory benchmark tests on crushed salt

#### 2.3.1 Testing material and equipment

Each participant of the benchmark tests was supplied with about 60 kg of reference backfill in the same specification. The crushed salt with a maximum grain size of 2 mm was excavated at the entrance of drift A.

For the investigations, a backfill compaction cell was used. The difference to a usual oedometer cell, used at BGR for instance, is that the diameter/height ratio is about 0.5 and in case of a oedometer cell about 2. The sample dimensions correspond to a filling height of 310 mm and a diameter of 155 mm. The filling volume is about 6 dm³. Such a shape of the backfill body is attributed to be typical for the mining situations which have been investigated and evaluated by IfG in different geomechanical stability assessments.
Figure 2.11 shows the backfill pressure cell, vertical loaded by a servohydraulic testing machine up to a force of 250 kN (equivalent to a vertical stress of 13.2 MPa).

The horizontal stress is generated in dependence on the prevented lateral deformation. The magnitude of the horizontal stresses is determined by strain gauges fixed on the outer steel wall of the cell in the half height of the backfill body. The strain gauges were calibrated in special tests using a grease as pressure transmitting medium. Corresponding to its high viscosity resulting in a ratio of vertical to horizontal stress of one, a hydrostatic stress state can be assumed and on basis of the known vertical stress the gauges can be calibrated.

In the backfill, a friction along the steel wall is produced resulting in a stress profile from the lower to the upper side of the sample. Due to this fact, it is necessary to measure the vertical stress downside the backfill body as well. This is performed using a load cell on the bottom. The friction can be calculated from the difference between the vertical stress upside, applied by the machine, and downside. For that, a numerical function along the backfill height is defined.

The evaluation of the compaction tests is done in such a way, that the vertical and horizontal stresses $\sigma_v$ and $\sigma_h$ in the half backfill height are described versus the vertical displacement of the load platen upside (volumetric compaction). The mean value of the backfill pressure is calculated by:

$$\text{mean value of backfill pressure} = \text{calculated value}.$$
\[ p = \frac{\sigma_v + 2\sigma_h}{3} \] (2.4)

The vertical compaction displacement is measured with an inductive displacement transducer.

### 2.3.2 Experimental procedure

#### 2.3.2.1 Test stages and sample preparation

The compaction tests on crushed salt were performed according to the test procedure in agreement with the other participants. At IfG, for oedometer-like compaction tests on crushed salt were carried out in the backfill pressure cell. Two strain controlled tests were conducted at temperatures of 30°C and 70°C including relaxation phases and, additionally, two stress controlled tests were done at the same temperatures including creep phases. The background of the interruptions was to observe the stress drop while relaxation and the compaction rate decrease while creep for extrapolation of the backfill resistance in a long term sense.

The following test stages were committed:

a) Initial density = 1.4 g/cm³ (initial porosity = 35 %)  
b) Increase of the temperature up to 70°C in case of two tests  
c) Application of a vertical stress of 0.5 MPa in 1 hour (stress controlled)  
d) Vertical loading up to 10 MPa using a rate of \( \varepsilon = 5 \cdot 10^{-6} \text{ s}^{-1} \) (deformation controlled) with embedded relaxation phases (30°C and 70°C)  
e) Vertical loading up to 10 MPa in 6 hours (stress controlled) with embedded creep phases (30°C and 70°C)

Under room temperature, the cell was filled up to a height of 310 mm. This was equivalent to a mass of 8.084 kg backfill material and a initial density \( \rho_0 \) of 1.4 g/cm³.

The initial porosity \( \phi_0 \) can be calculated by

\[ \phi_0 = 1 - \frac{\rho_0}{\rho_s} \] (2.5)

\( \rho_s \) = grain density of halite (2.187 g/cm³)

The installed specimens were heated up to the temperature of 30°C or 70°C. In case of the elevated temperature, the heating phase had a time interval of 2 days.

The stress controlled application of 0.5 MPa vertical stress in 1 hour led to a small consolidation, so the deformation and stress controlled tests began at a density of 1.42 g/cm³.

#### 2.3.2.2 Deformation controlled tests at 30°C and 70°C

The strain controlled tests at a rate of \( \dot{\varepsilon}_1 = 5 \cdot 10^{-6} \text{ s}^{-1} \) are presented in Figures 2.12 to 2.14
(30°C) and Figures 2.15 to 2.17 (70°C). It is depicted for each temperature the measured vertical and horizontal stress and the calculated backfill pressure in the half height of the samples versus time. In a second graph the stress ratio $\sigma_{\text{horizontal}}/\sigma_{\text{vertical}}$ and the porosity versus time are sketched. The third picture of each series shows the dependence of stresses and stress ratio on porosity. The test duration was up to 7 days.

Figure 2.12: Strain controlled test with relaxation phases at $\dot{\varepsilon} = 5 \cdot 10^{-6}$ s$^{-1}$ and 30°C (stresses versus time)

Figure 2.13: Strain controlled test with relaxation phases at $\dot{\varepsilon} = 5 \cdot 10^{-6}$ s$^{-1}$ and 30°C (stress ratio and porosity versus time)
Figure 2.14: Strain controlled test with relaxation phases at $\dot{e} = 5 \cdot 10^{-6} \text{s}^{-1}$ and 30°C (stresses and stress ratio versus porosity)

In all tests after stopping the machine, a significant stress relaxation is visible. The observation time comprised up to 4 days. On basis of the backfill stress rate decrease $\dot{\rho}$ in time the equivalent volumetric compaction rate $\dot{\varepsilon}_v$ in time can be calculated using a constitutive law. This context enables the determination of the backfill resistance as function of time in dependence on $\dot{\varepsilon}_v$ and so an extrapolation to the in situ relevant compaction rates can be performed similar to Chapter 2.2.

Figure 2.15: Strain controlled test with relaxation phases at $\dot{e} = 5 \cdot 10^{-6} \text{s}^{-1}$ and 70°C (stresses versus time)
Figure 2.16: Strain controlled test with relaxation phases at $\dot{\varepsilon} = 5 \cdot 10^{-6} \text{ s}^{-1}$ and 70°C (stress ratio and porosity versus time)

Figure 2.17: Strain controlled test with relaxation phases at $\dot{\varepsilon} = 5 \cdot 10^{-6} \text{ s}^{-1}$ and 70°C (stresses and stress ratio versus porosity)

The stress ratio varies in a relative large scattering range. The mean value 0.55 at 30°C is higher than 0.45 at 70°C. While the relaxation an increase of the ratio can be observed.

The porosity is reduced from 35 % to 26 % (30°C) and to 21 % (70°C).
2.3.2.3 **Stress controlled tests at 30°C and 70°C**

The stress controlled tests at a rate of $\dot{\sigma}_1 = 1.67 \text{ MPa} \cdot \text{h}^{-1}$ are presented in Figures 2.18 to 2.20 (30°C) and Figures 2.21 to 2.23 (70°C). To permit a comparison, the results are plotted in the same dependence on time and porosity. The investigations took up to about 7 days.

![Stress controlled test graph](image)

**Figure 2.18:** Stress controlled test at $\sigma = 1.67 \text{ MPa} \cdot \text{h}^{-1}$ and 30°C with creep phases (stresses versus time)

![Stress ratio and porosity graph](image)

**Figure 2.19:** Stress controlled test at $\sigma = 1.67 \text{ MPa} \cdot \text{h}^{-1}$ and 30°C with creep phases (stress ratio and porosity versus time)
Figure 2.20: Stress controlled test at $\sigma = 1.67$ MPa·h$^{-1}$ and 30°C with creep phases (stresses and stress ratio versus porosity)

The vertical stress was stepwise increased resulting in 3 and 4 creep phases up to 3 days. The experimental test layout with various creep phases aimed to the extrapolation of the backfill resistance at the relevant in situ compaction rates. Both the relaxation and the creep tests permit an estimation of the backfill stress in a long term sense.

Figure 2.21: Stress controlled test at $\sigma = 1.67$ MPa·h$^{-1}$ and 70°C with creep phases (stresses versus time)
Figure 2.22: Stress controlled test at $\sigma = 1.67 \text{ MPa h}^{-1}$ and 70°C with creep phases (stress ratio and porosity versus time)

Figure 2.23: Stress controlled test at $\dot{\sigma} = 1.67 \text{ MPa h}^{-1}$ and 70°C with creep phases (stresses and stress ratio versus porosity)
The stress ratio results in a relative large scattering range again. The mean value with 0.48 at 30°C is higher than 0.44 at 70°C. The same tendency was found in the deformation controlled tests. While the creep phases no significant changes of the ratio can be observed.

The porosity is reduced from 35 % to 22.5 % (30°C) and to 18 % (70°C), respectively.

Figures 2.20 and 2.23 show the dependence of the vertical and horizontal stress, backfill pressure and stress ratio on porosity. The stress ratio is relative low at the beginning of the tests and reaches a range of 0.4 to 0.5 in the compaction course. That statement is valid for the deformation driven experiments as well.

### 2.3.2.4 Backfill resistance in a long term sense

Figures 2.24 and 2.25 facilitate the comparison of both the strain and stress controlled tests at the applied temperatures. Merely the relation between backfill pressure (mean value of the principal stresses) and porosity is presented aiming to get an extrapolation to the backfill resistance at the compaction rates acting in situ \((1\cdot10^{-10} \text{ s}^{-1})\). Such slow rates, 4 - 5 orders of magnitudes lower than realized in laboratory, cannot be performed in the tests in a acceptable time period. Therefore, a sophisticated procedure is needed to allow such a transformation.

![Graph showing the relationship between backfill pressure and porosity](image)

**Figure 2.24:** In laboratory generated backfill stresses (with embedded creep and relaxation phases, temperature = 70°C) and long term backfill resistance calculated using a constitutive law

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As outlined above, the relaxation phases were used to enable an extrapolation to the in situ relevant backfill stresses (backfill resistance) at a compaction rate of $1 \cdot 10^{-10}$ s$^{-1}$. The procedure is similar to Chapter 2.2. On the other hand, on basis of the creep tests a porosity was found at which the compaction rate is approaching to a deformation magnitude of $1 \cdot 10^{-10}$ s$^{-1}$ as well. In the evaluation of both tests, a curve approximation was made and using a constitutive law an extrapolation to the backfill stress at constant porosity (relaxation) and to the backfill porosity at constant stress (creep) could be achieved. In the Figures 2.24 and 2.25 the extrapolation using relaxation is represented by a vertical arrow and the extrapolation on basis of creep by a horizontal one.

Both investigations lead to the long term resistance, plotted as single symbols (unfilled triangles and squares). Furthermore, in Figure 2.25 the long term backfill stresses determined on the drilled cores are transferred from Figure 2.10. It can be emphasized, that the results sketch a common range for lower and higher porosities (drilled cores and crushed salt). It means, the results on both materials are consistent.

The long term backfill resistance can be described by the following equation as well (The curve is marked by black dots):
\[ h_i(\phi) = \frac{a}{(\phi^e - \phi_0^e + d)^m} \]  (2.6)

\begin{align*}
& a = 16.48 \cdot 10^{-3} \quad \phi = \text{porosity} \quad c = 0.1 \\
& d = 3 \cdot 10^{-4} \quad m = 2.25 \quad \phi_0 = \text{initial porosity}
\end{align*}

\[ p(\phi) = \left[ \frac{\text{rate}}{A} \cdot \exp \left( \frac{Q}{R \cdot T} \right) \cdot \frac{3}{h_i(\phi)^3} \right]^{0.2} \]  (2.7)

rate = compaction rate in s\(^{-1}\)  Q/R = 6520 K  
A = 5 \cdot 10^{-8} \text{ s}\(^{-1}\) \text{ MPa}\(^{-5}\)  T = temperature (K)

This is the constitutive model formulation for crushed salt proposed by Hein and modified by Korthaus (Bechthold et al., 1999). The formulation is useable for the performed tests on drilled backfill cores and crushed salt in a very satisfying manner.

### 2.4 Conclusions

The main objective of the investigations in Chapter 2 “Backfill Performance” was to examine the different mechanical and hydraulical properties of in situ over a more than eight years heating phase compacted crushed salt in comparison with loose backfill from the drift entrance. It can be stated, that, assuming equivalent porosity, no significant different behaviour was found.

Despite the scatter in the results, a good agreement between the compaction behaviour of both backfill materials was observed. The horizontal stress was generated in oedometer-like conditions in a range of 40\% to 50\% of the vertical stress. While the relaxation phases, a significant increase of the ratio \(\sigma_{\text{horizontal}}/\sigma_{\text{vertical}}\) was visible. Regarding this statement, it is assumed that the ratio between horizontal and vertical stress could raise still more in longer observation periods. It means, under in situ conditions, an approximation to one cannot be excluded.

Furthermore, much effort was dedicated to the backfill resistance under in situ convergence rates. Under in situ conditions in a mine, the relevant convergence processes are 4 - 5 orders of magnitudes lower than in laboratory and therefore a significant lower backfill stress is to expect. Using various extrapolation methods on basis of the detected constitutive behaviour an estimation of the in situ relevant backfill resistance was carried out. The agreement with the PTE-INE formulation is very well and therefore the necessary input parameters for the model calculations can be provided.

The initial porosities determined on the drilled backfill cores before the laboratory tests agree well with the values measured in situ. The investigated permeabilities were found in a range of \(3 \cdot 10^{-13} - 7 \cdot 10^{-13}\) m\(^2\) (valid for a porosity range of 21 - 24\%). The summarizing Figure 2.26 (Rothfuchs et al., 2003) shows that these results sketch the post-test data very well.
In general, the knowledge of the compaction behaviour of crushed salt with its consequences to the barrier function has been not only confirmed but also extended.

3 ROCK-SALT PERFORMANCE

3.1 Introduction

In an underground repository in rock salt, the host rock is the most important barrier which has to isolate the radioactive wastes from the biosphere for a long time. Thus, in the repository performance assessment the long-time behaviour of host rock’s strength and tightness must be predicted. The proof must be conducted that no damaging processes will take place which potentially form flow paths for fluids and gases.

Under elevated temperatures in repositories for high level radioactive wastes, an acceleration of convergences in the underground openings is to be expected. The higher creep rates will cause rock stress redistributions between the emplacement drifts and the surrounding cooler rock. In the project BAMBUS-II, there was a unique opportunity to investigate such thermo-mechanical interactions between the TSDE-field and the surrounding rock salt. Taking into account these effects, rock stress measurements were carried out in the pillar between the two heater drifts and in the northern access drift in a distance of about 30 m into drift B for both
the heating and post heating phase.

In this Chapter both mentioned aspects, the consequences of an eight years heating period on strength and deformation behaviour of rock salt and the rock stress development at the test site shall be discussed and evaluated.

In two series of strain controlled strength tests at room temperature and at 70°C on rock salt specimens extracted from the pillar, strength-deformation curves were measured and the differences to Staßfurt rock salt in the absence of any former temperature history were evaluated.

Since the strength of rock salt depends significantly on the strain rate and this rate in usual testing procedures is by four orders of magnitudes higher than the rate in situ, special relaxation tests were carried out to extrapolate to the in-situ strength (bearing capacity). Such a procedure is relevant, because the long-term bearing capacity is significantly lower than the short-term strength as measured in the laboratory. A laboratory strength test under in-situ rates would take years and cannot be performed as a general rule.

The rock stresses were measured continuously by BGR using stress monitoring stations until the post heating phase. The results will be compared with minimum stress measurements using the hydraulic fracturing method in periodically repeated campaigns and they will be evaluated on basis of thermo-mechanical model calculations. The used vertical model in a cross-section perpendicular to the test drifts is helpful to understand and to interpret the measured stress development.

### 3.2 Mechanical properties under laboratory conditions and elevated temperatures

#### 3.2.1. Intention and material characterisation

The aim of the laboratory testing was to determine if the mechanical behaviour of the formerly heated material was altered. In short-term tests, stress-deformation curves of cylindrical specimens were recorded under various confinement pressures at a deformation rate of \( \dot{\varepsilon}_i = 2.5 \cdot 10^{-5} \text{ s}^{-1} \) for the temperatures 23°C (laboratory condition) and 70°C. The results can be compared with data found on specimens of Staßfurt rock salt without former heating. In addition to observing the short-term strength, relaxation tests were conducted to determine the long-term strength (load-bearing capacity) corresponding to the in situ relevant strain rate 1 \( \cdot 10^{-10} \text{ s}^{-1} \).

In the laboratory tests, drill cores with a diameter of 63.8 mm and a length of 33 m were obtained from the TSDE field, borehole K1. This borehole is located in the middle of the pillar between the heater drifts. The stratigraphic layer is Staßfurt rock salt.

The cylindrical test specimens were prepared by planing the ends flat and parallel and grinding the outer surfaces of the drill cores. The diameters and heights of the 32 test specimens range between 63.5 ± 0.1 mm and 126.5 ± 0.2 mm. The density is
\( \rho = 2.20 \pm 0.02 \text{ g/cm}^3 \). The density \( \rho = 2.24 \text{ g/cm}^3 \) was measured on two test specimens with an apparently higher content of sulfate minerals.

Before strength testing, the ultrasound velocities of P- and S-waves and the resulting dynamic elastic constants were determined for all test specimens. The mean values for the 32 tested specimens were

\[
E_{\text{DYN}} = 36.7 \pm 1.7 \text{ GPa} \\
\nu_{\text{DYN}} = 0.26 \pm 0.02
\]

The rock salt consists of approximately 90% halite and up to 10% accessory sulfate minerals. Under transmitted light, inclusions of anhydrite and polyhalite can be observed (see Figure 3.1). The diameter of the salt rock grains is not uniform. The predominant grain sizes in the area of the former layers vary between 1–5 mm, in other areas the predominant sizes are larger and stretch into a cm range. The larger grains are elongated and are situated parallel to the former annual rings. In addition to clear halite grains, some grains were observed with cloudy fine-grained (20–50 mm) constituents. The grain boundaries are often submicroscopic, creating an opaque appearance.

As main impurities polyhalite \((K_2SO_4 \cdot MgSO_4 \cdot 2 CaSO_4 \cdot 2 H_2O)\) and anhydrite \((CaSO_4)\) were identified using optical microscopy. The grain habit of the impurities is elongated. In addition, to the fine-grained, intra-crystalline inclusions with grain sizes of approximately 5–50 \(\mu\)m and intercrystalline grains up to 2 mm were observed along the grain boundaries.

![Figure 3.1: Halite sample with included anhydrite bands before testing (episcopic and transmitted light) and after testing at \(\sigma_3 = 10 \text{ MPa}\) up to a deformation \(\varepsilon_1 = 21\%\)]
3.2.2 Strength tests at room temperature and at 70°C

The stress-deformation curves (in strain control) were recorded for a constant deformation rate $\dot{\varepsilon}_1 = 2.5 \cdot 10^{-5}$ s$^{-1}$ at room temperature (23° C) and at 70° C. The constant confining pressures were applied in stages of $\sigma_3 = 0; 1; 2; 4; 6; 8; 10; 12; 15; 20$ and 25 MPa. Such a range is sufficient for evaluating the strength behaviour of rock salt.

Figure 3.2: Summarizing of triaxial strength tests at temperature 23°C

Figure 3.3: Summarizing of triaxial strength tests at temperature 70°C
The curves measured in the course of the tests at room temperature specifying the stress difference ($\sigma_1 - \sigma_3$) in dependence on the axial deformation (working curves) are summarized in Figure 3.2.

One can see a clear increase in the maximum strength in dependence on the confining pressure $\sigma_3$. After reaching the maximum, a residual strength of bearing capacity can be observed, which decreases with $\varepsilon_1$. The lower $\sigma_3$ the stronger the stress drop due to damage accumulation and strain-softening. Above $\sigma_3 = 12$ MPa, no stress drop is visible in the investigated deformation range.

Figure 3.3 summarizes the test results at 70°C. The elevated temperature reduces the maximum strength in dependence on confining pressure. Furthermore, elevated temperature gave rise to an increase in the axial deformation. With the exception of temperature, all other test conditions remained the same.

### 3.2.3 Extrapolation to the load-bearing capacity in situ

In addition to the strength tests, other triaxial tests incorporating relaxation phases were conducted at temperatures of 23°C and 70°C and confining pressures of $\sigma_3 = 0; 4; 10; 15$ and 20 MPa. To initiate the relaxation phase, the strain-control loading was stopped at deformations of $\varepsilon_1 = 2.5 \%; 5 \%$ and 10 % and the stress drop was monitored for a duration of approximately 18 to 20 hours. The Figures 3.4 and 3.5 show the stress-deformation course and the Figures 3.6 and 3.7 the stress relaxation in time for 23°C and 70°C under a confining pressure of 4 MPa.

![Working curve $\sigma_{\text{diff}} - \varepsilon_1$ at $\sigma_3 = 4$ MPa and 23°C](image)
Figure 3.5: Working curve $\sigma_{\text{diff}} \sim \varepsilon_1$ at $\sigma_3 = 4$ MPa and 70°C

Figure 3.6: In the test embedded relaxation phases at $\sigma_3 = 4$ MPa and 23°C
Figure 3.7: In the test embedded relaxation phases at $\sigma_3 = 4$ MPa and 70°C

Figure 3.8: Long-term stress extrapolation on basis $\sigma_{\text{diff}} \sim \dot{\varepsilon}_1$ at $\sigma_3 = 4$ MPa and 23°C
Figure 3.9: Long-term stress extrapolation on basis $\sigma_{\text{diff}} \sim \dot{\varepsilon}_i$ at $\sigma_3 = 4$ MPa and 70°C

Following each relaxation test, an unloading loop is applied to determine the elastic modulus $E_{\text{System}}$ of the loading system including the elastic response of the test specimen and triaxial cell axial pressure piston. $E_{\text{System}}$ is required to interpret the data. The stress relaxation in time $\sigma_{\text{diff}}(t)$ can be described using the relation

$$\frac{\sigma_{\text{diff}}(t)}{\sigma_{\text{diff}}(t = 0)} = 1 - B \cdot \ln \left(1 + \frac{t}{t_0}\right)$$

(3.1)

The factor $\sigma_{\text{diff}}(t = 0)$ is the differential stress at the start of relaxation at the time $t = 0$. The parameters $B$ and $t_0$ are constants which are determined on the basis of a curve approximation (Figures 3.6 and 3.7). The stress rate $\dot{\sigma}_{\text{diff}}$ decreases significantly during relaxation. For the loading system the following conditions are valid:

$$\varepsilon_{\text{total}} = \varepsilon_{\text{elastic}} + \varepsilon_{\text{inelastic}} \quad \dot{\varepsilon}_{\text{total}} = 0 \quad \dot{\varepsilon}_{\text{elastic}} = -\dot{\varepsilon}_{\text{inelastic}}$$

(3.2)

$$\dot{\varepsilon}_{\text{elastic}} = \frac{\dot{\sigma}_{\text{diff}}}{E_{\text{System}}} \quad \dot{\varepsilon}_{\text{inelastic}} = -\frac{\dot{\sigma}_{\text{diff}}}{E_{\text{System}}}$$

With this procedure, the equivalent inelastic deformation rate can be calculated. It is based on the observed stress rate dependence and using the relation explained above a stress extrapolation to the in situ relevant deformation rate in a range of $\dot{\varepsilon}_i = 10^{-10}$ s$^{-1}$ can be performed. The so found stress states in dependence on $\sigma_3$ are consistent to the in situ deformation rate. The same pairs of values $\sigma_{\text{diff}}$ and $\dot{\varepsilon}_{\text{diff}}$ should be observed eventually in the
stationary phase of creep tests.

The Figures 3.8 and 3.9 depict the extrapolation and the differential stresses corresponding to an in situ deformation rate of $\dot{\varepsilon}_i = 10^{-10} \text{s}^{-1}$. As to expect, regarding the stresses valid in a long-term sense the temperature influence is visible as well indicating significant lower stress levels than at room temperature. While at 23°C the mean value is in a range of 25 MPa, at 70°C a magnitude of about 12 MPa was determined.

The maximum strength measured in the short-term laboratory tests and the extrapolation to a strain rate $\dot{\varepsilon}_i = 10^{-10} \text{s}^{-1}$ are summarized in Figure 3.10. For each curve, both temperatures were investigated. The curves for bearing capacity represent the lower boundary of the scattering range.

The model used to describe the curves shown in Figure 3.10 can be expressed as:

$$\sigma_{\text{diff}} = \sigma_D + \frac{\sigma_{\text{Max}} - \sigma_D}{\sigma_\phi + \sigma_3} \cdot \sigma_3$$  \hspace{1cm} (3.3)

and was proposed by Minkley et al., 2001. The parameters $\sigma_D$, $\sigma_{\text{Max}}$ and $\sigma_\phi$ are given in Figure 3.10.

![Figure 3.10: Short-term strength (above) with an approximation after Menzel and Schreiner, 1976 and long-term bearing capacity (below) in dependence on confining pressure at 23°C and 70°C](image-url)
The same results calculated in terms of octahedral stresses as plotted in Figure 3.11 can be described by:

\[
\tau_o = a \cdot \left( \frac{\sigma_n}{\sigma_{\text{norm}}} \right)^b
\]

\[
\sigma_{\text{norm}} = 1 \text{ MPa}
\]

(3.4)

The octahedral calculation was carried out to enable a comparison with other usual strength hypotheses. Both diagrams clearly indicate that the long-term bearing capacity is significantly lower than the strength determined from the short-term tests.

### 3.2.4 Comparison with unheated rock salt and evaluation of the strength reduction due to raised temperature

Aiming to a comparison of the short-term strength curve with results determined on Staßfurt rock salt without former heating period, the reference Dürr et al., 1983 can be given. In this report strain controlled tests at a rate of \( \dot{e}_1 = 6.7 \cdot 10^{-5} \text{ s}^{-1} \) are described and on basis of a non-linear regression the following relation was found:

\[
\tau_o = \frac{\sigma_o + 0.58}{0.49 + 0.0206 \cdot (\sigma_o + 0.58)}
\]

(3.5)

Figure 3.11: Short-term strength (above) and long-term bearing capacity (below) at 23° C and 70° C in octahedral description
Figure 3.11 shows the curve and as conclusion can be drawn that the given relation 3.5 describes the scattering range of the specimens influenced by heating very well. There is no different short-term behaviour to detect.

However, the question arises whether a change in the mechanical behaviour, caused by damaging or weakening due to the eight years heating period, is to be detected at elevated temperature. With respect to the effect of raised temperatures on the short-term strength, the reference Menzel and Schreiner, 1976 shall be used. In this report, the authors outlined a strength reduction formulation that incorporates strength dependence on the temperature. The strength boundary is described by:

\[ \sigma_{\text{eff}} = \sigma_D \cdot (1 + \kappa \cdot \sigma_Z) - \sigma_1 \]  

A parameter approximation to the results of the short-term tests at laboratory temperature in Figure 3.10 results in parameter estimates \( \sigma_D = 24.0 \text{ MPa} \), \( \kappa = 2.9 \) and \( \sigma_Z = 1.3 \text{ MPa} \). The Figure depicts the almost equivalent course of both Equation 3.3 and Equation 3.6. On basis of the reference, the changed parameters \( \sigma_D = 20.4 \text{ MPa} \), \( \kappa = 3.1 \) and \( \sigma_Z = 1.3 \text{ MPa} \) for the temperature of 70°C can be calculated, which lead to the lower curve in Figure 3.10.

In result, the conclusion stated above can be confirmed for the short-term strength under raised temperature as well. It can be concluded, that the strength reduction calculated on basis of an empirical formulation for rock salt without a long-term temperature influence is the same as found in the own laboratory tests on samples after heating. There was no effect in the mechanical behaviour, caused by damaging or weakening due to the eight years heating period, to be detected.

3.3 Rock stress measurements in situ during and after heating by using hydraulic fracturing

3.3.1 Experimental set-up

The magnitude of the minimum principal rock stress and its orientation were measured using a special hydraulic fracturing borehole tool. This borehole tool which was developed by IfG Leipzig and GMuG Ober-Mörlen (Manthei et al., 1998) utilizes acoustic emission measurements (AE) aimed at monitoring and localizing hydraulically induced fracture planes. Figure 3.12 shows the borehole tool, electrical pump and the data acquisition. The borehole tool consists of two parts – the hydraulic pressurization unit in the middle and two AE sensor arrays. The overall length of the borehole tool is about 2 m. Each sensor array includes four AE transducers in a cross section perpendicular to the borehole axis. The distance between the AE arrays is approximately 1.5 m. The transducers with integrated preamplifiers are placed in a common housing, which is screwed into the pressurization unit. The transducers are pressed pneumatically against the borehole wall. The features and performance of the borehole tool are:

- Determination of the fracture orientation and extension using only one borehole.
- Because of the same distance between injection interval and sensor arrays the sensitivity of AE registration is always the same independent of the borehole depths.
- Determination of the real fracture propagation in distances up to 15 to 20 times of the borehole diameter.
- Estimation of the fracture opening due to the measured injected oil volume.
- Expensive visual inspection of the fracture is not needed.

Figure 3.12: Hydraulic fracturing borehole tool, electrical pump and data acquisition

The preamplified signals are supplied to an 8-channel transient recorder card which is controlled by a portable personal computer. The transient recorder card (sampling rate 1.25 MHz, resolution 12 bit) is read each time a signal passes the trigger threshold. The borehole pressure and the pressure which is applied to the packer are measured using pressure cells. The signals of the pressure cells are digitized in general each second and stored at the hard disk of a notebook.

3.3.2 Experimental procedure

Before the hydraulic fracturing test, the tightness of the lines, the pressuring interval and of the packers used to seal up the injection interval was checked by applying a constant hydraulic pressure below the expected shut-in pressure for 15 minutes. Then, a high pressurization rate was applied to fracture the rock salt. In the shut-in phase the pressure drop was observed between 30 and 60 minutes. After each hydraulic fracturing test, a refracturing test was performed to confirm the observed shut-in pressure. The refracturing tests normally took 30 minutes each. The shut-in pressure is the nearly constant pressure level reached at the end of the test to keep the induced fracture open. In this equilibrium state, the shut-in pressure equals the minimum principal stress. Under the assumption that the fracture opens in the direction of least resistance, the direction of the minimum principal stress coincides with the normal direction of the fracture plane (e.g. Kehle, 1964).
To determine the stress state change in the rock mass around the TSDE test field, in total about 49 hydraulic fracturing tests were carried out in several injection wells at the end of the heating period and during cooling down of the rock mass. Two injection wells were located in the heated area in the center of the pillar parallel to the two test drifts. In these boreholes six measuring campaigns were performed from January 1999 to January 2003. From March 1997 to January 2003, in the unheated area five fracturing campaigns were carried out in two horizontal injection wells which were drilled from the northern access drift towards the TSDE drifts. In Figure 3.13 the pillar between the two test drifts and the northern access drift is depicted.

**Table 3.1: Date of measurement, injection well, and number of hydraulic fracturing tests**

<table>
<thead>
<tr>
<th>Heated area</th>
<th>Date</th>
<th>Borehole</th>
<th>Number of tests</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>January 1999</td>
<td>K1</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>January 2001</td>
<td>K1</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>August 2001</td>
<td>K1.1</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>January 2002</td>
<td>K1.1</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>September 2002</td>
<td>K1.1</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>January 2003</td>
<td>K1.1</td>
<td>4</td>
</tr>
</tbody>
</table>

Figure 3.13: Investigation site with TSDE test field and northern access drift (right-hand side)
Due to the fast borehole closure of borehole K1 in the heated area it was not possible to perform any further fracturing test in greater borehole depth after the first two fracturing series. Therefore an additional well K1.1 was drilled parallel to the existing one in a distance of 1 m.

### 3.3.3 Results of stress measurements

Figures 3.14 and 3.15 show the magnitude of the minimum principle stress measured in the heated and unheated area, respectively, versus borehole depth.

![Figure 3.14: Minimum principal stresses measured in the heated area in boreholes K1 and K1.1](image)

**Figure 3.14:** Minimum principal stresses measured in the heated area in boreholes K1 and K1.1
Figure 3.14 shows that at the end of the TSDE heating phase, when the rock temperature in the center of the pillar between the test drifts was 85° C, the minimum principle stress ranges between 6.3 MPa and 8.3 MPa over the 33 m borehole depth (see label “campaign 1/1999”). The later performed measurements indicate an increase of the minimum principal stress up to 10 MPa and 11 MPa over a borehole depth of 14 m. In greater borehole depths, the tests could not be carried out due to the continuing borehole closure. A comparison of the five consecutively performed fracturing campaigns (campaign in 1/2001 up to 1/2003) shows a slight increase of the minimum principal stress with decreasing temperature in the pillar.

Figure 3.15: Minimum principal stresses measured in the unheated area

In the unheated area, the minimum principal stress depicts the inverse trend of a general decrease with time and temperature (Figure 3.15). In 1997, the stress level at a borehole depth greater than 6 m reached a nearly constant value between 16 and 17 MPa. Five years later in the cooling phase, the minimum principal stress decreases to values between 13 and 15 MPa.

Figure 3.16: Left-hand side: Located events in projection to two coordinate planes for fracturing campaign 1/2001 in borehole K1 in the heated area. Right-hand side: Orientation of the minimum principal stress in the Schmidt-net diagram.
Figure 3.16 shows at the left-hand side the location of the acoustic emission events in projection to two coordinate planes (x-y-plane: top view; y-z-plane: lateral view). The events were detected during hydraulic fracturing tests in 2 m, 3 m, and 3.8 m borehole depths in borehole K1 in the heated area. This fracturing campaign was performed in January 2001. The location of the transducer arrays and the injection interval are indicated by circles and rectangles, respectively. The y-axis is parallel to the injection well. In total, more than 6,882 acoustic emission events could be located during the three tests. Because of the short distances between the injection locations the fracture planes overlap each other.

The right-hand side of Figure 3.16 displays the orientation of the minimum principal stress (filled dots) in a so-called Schmidt-net diagram. The orientation of the injection well is marked by a thick line. Due to the fast borehole closure it was not possible to do further fracturing tests in greater depths.

Figure 3.17: Located events in projection to the rotated coordinate planes for fracturing test in 2 m in borehole K1.

Figure 3.17 shows separately the events of the fracturing and re-fracturing tests in 2 m borehole depth in a rotated coordinate system where the fracture plane lies in x'-z'-coordinate plane. It can be seen that the fracture plane appears to be nearly circular with a diameter of about 2 m corresponding to a penny-shape oriented event front. The frac initiates at the center of the injection interval and propagates mainly in positive x'-direction. The minimum principal stress (see right hand side of Figure 3.16) is nearly horizontal (maximal inclination is 24 degrees) and rotated against the borehole axis by about 60 degrees.
Figure 3.18 gives an overview of the located events (6,125 in total) which were recorded one year later during fracturing tests in 3.5 m, 7.0 m, 10 m, and 13 m in the borehole K1.1 in the heated area. In the x-y-plane it can be seen that the fracture extends between the transducer arrays parallel to the borehole axis. All fracture planes have nearly the same orientation independent of borehole depth. The minimum principle stress lies horizontal and nearly perpendicular to the injection well (see right-hand side of Figure 3.18).

Figure 3.19 shows 12,281 events which were located during the fracturing tests in 3 m, 7 m, and 11 m borehole depths in the unheated area. A large fracture occurred in the vicinity of the opening in zone where the stress is influenced by the opening itself. More than 8,800 events were localized. On the contrary, in larger borehole depths much less events could be located in spite of the fact that the same oil volume was injected. This observation may be explained by larger deviatoric stresses close to the contour of the openings. The fractures initiated at the center of the injection interval and propagate in radial direction mainly in negative x-direction. Except for the fracture plane at 7 m which is more inclined, the minimum principal stress is nearly horizontal. The inclination of the fracture plane in 7 m is reversed compared with the fracture planes in 3 m and 11 m. Therefore the intersection point of the minimum principal stress appears opposite in the right-hand side of Figure 3.18. The inclination is about 27 degrees.
Hydraulic fracturing tests were performed at the end of the TSDE heating period and during cooling down of the rock mass after switching off the heater casks. The fracturing tests were carried out in two horizontal injection wells in the heated and in the unheated area each.

The results show that the minimum principal stress measured in the unheated rock decreased from 16 MPa, measured during the heating period, to 14 MPa. At the same time the minimum principal stress in the heated rock increased from approximately 7 MPa up to 10 MPa. Accompanying acoustic emission measurements pointed out that the minimum principal stress is nearly horizontal. This orientation was found in the heated zone as well as in the unheated area. In the heated area the minimum stress lies perpendicular to the axis of the test drifts. In borehole depths close to the contour of the openings, the orientation of the fracture planes are influenced by the surface of the opening. This fact can be explained by larger deviatoric stresses close to the contour. In this zones the fractures occurred to be larger and more locatable acoustic emission events were emitted as in greater borehole depths.
Figure 3.20: Comparison of stresses measured by stress monitoring and hydraulic fracturing and calculated by using a strain plane model in cross section A-A

The increase of the minimum principal stress in the pillar in the post-heating phase determined by hydraulic fracturing corresponds fairly well to the results found in model calculations (see Chapter 4). In Figure 3.20 after the re-excavation of drift B, a slight raise is to recognise as well. The deviation of about 2 MPa between measurements (red dots) and calculation is considered acceptable.

The observations can be explained as follows: In the test field and the surrounding rock, including the northern access drift, a stress interchange takes place. Due to the heating, a stronger creep and in consequence a stress relaxation is caused in the TSDE-field which affects a stress increase in the surrounding cooler rock. This stress redistribution decreases after heater switching off and leads to the slight stress built up observed in the formerly heated pillar.

For a comparison of both calculated and permanently measured stresses, in Figure 3.20 the results of stress monitoring station Bo36-MS1 are depicted. Borehole 36 is drilled parallel to the pillar axis in the middle of the pillar. Station MS1 is located in a distance of 7.25 m to cross section A-A and of about 30 m to the drift entrance (see Figure 2.1). The station is the nearest to the vertical model plane in cross section A-A. To enable a comparison to the calculation results, the measured stress values are used as stress changes with respect to the beginning of the heating (end of September 1990) only. This means that the stress values have been set to the calculation value at this point of time and afterwards just the changes have been added. All stress values measured before the beginning of heating are not considered. The agreement between measurement and calculation confirms the model used with its constitutive laws and parameters. A more comprehensive discussion of the model calculations will be given in Chapter 4 (modelling).
4 MODELLING OF THE TSDE-EXPERIMENT

4.1 Introduction

For the model calculations described below, the software FLAC (Itasca, 1998) was used. It represents a two-dimensional explicit finite difference programme for engineering mechanics computations. The programme offers a wide range of capabilities to solve geotechnical and mining problems. In BAMBUS-II, FLAC was used to perform coupled thermo-mechanical analyses as a function of time. As the basis for the constitutive law, the time dependent backfill compaction behaviour was determined in own laboratory tests. The thermo-mechanical interactions, particularly the influence of backfill porosity on thermal conductivity, were investigated in many case studies. The aim was to find a sufficient agreement with the in-situ measurements.

An important conclusion from the BAMBUS project (Bechthold et al., 1999) was the statement that the temperature field around the heaters (each with a length of 5.5 m) in the two test drifts with a length of 75 m each cannot be calculated realistically in a two-dimensional model. Due to the basic assumption of the plane strain model that all elements have an infinite length perpendicular to the model plane, the temperature development is over-estimated. Furthermore, no heat transfer can take place in the suppressed third dimension of the model. Therefore, the calculated temperature around the heaters was too high. Recognizing these findings in BAMBUS-II, three-dimensional thermo-mechanical model calculations were elaborated.

On the other hand, one topic of IfG’s investigations besides modelling of the thermo-mechanical interactions in the test drifts itself, was to study the stress field alteration in the surrounding rock salt during cool down. For that purpose, stress measurements were carried out in the pillar between the test drifts and in the northern access drift in dependence on the cooling process. In order to explain the observed stress changes, a two-dimensional thermo-mechanical model for the investigation of the far-range stress redistribution was developed. The following remarks will show that such a procedure is permissible if the temperature development in time corresponds to the three-dimensional situation. As a reference, a three-dimensional temperature calculation or the temperature run observed in situ can be used. Both can be transferred to the two-dimensional model as boundary condition at the heater surface.

In BAMBUS-I it was concluded that the stress state around the TSDE-field is influenced over large distances by the chambers excavated during the rock salt mining at the southern flank of the Asse mine. The effect of the surrounding stress field on the geomechanical behaviour and reactions of the test site was identified. The temperature influences the stress field as well, therefore the investigation of these interactions is an important consideration.

4.2 Model description

4.2.1 Geometry, boundary and initial conditions

The plane strain model represents a vertical cross-cut perpendicular to the test drifts and the
heaters, analogous to the monitoring cross-section A-A (Figure 2.1). The model is depicted in Figure 4.1. The width is 160 m and the height 100 m. The model basis is fixed in vertical direction and the sides are fixed horizontally. The upper boundary is loaded by 10.9 MPa, which results from a rock salt density of 2187 kg/m³ and the gravity loading, relating to the distance of 50 m between boundary and investigation level. The model consists of 14640 elements.

Figure 4.1: Plane strain model located in cross section A-A perpendicular to the test drifts

Test drifts A and B have a maximal width of 4.5 m and a maximal height of 3.5 m with a
10 m wide pillar between the drifts. The diameter of the heaters is approximately 1.5 m. This detailed situation is presented in Figure 4.2. The northern access drift and a parallel branch are shown in Figure 4.1 with two rectangular cross-cuts in a distance to drift B (wall to wall) of 26 m and 44 m, respectively, both 4 m wide and 3 m high. The initial stress state in the level of TSDE-field is $\sigma_x = \sigma_y = \sigma_z = 12$ MPa. The vertical stress gradient amounts 21.45 kPa/m. The initial rock temperature was assumed as $36^\circ$ C (309 K). The temperature development on the heater surface is applied by the relationship as presented in Figure 4.3. It was approximated by curve-fitting on basis of the in-situ measurements.

### 4.2.2 Time table

In the calculation turn the following time table was kept:

Table 4.1: Time interval and calculation steps

<table>
<thead>
<tr>
<th>Date</th>
<th>Time interval</th>
<th>Calculation steps</th>
</tr>
</thead>
<tbody>
<tr>
<td>April 1989 to September 1990</td>
<td>18 months</td>
<td>Free drift closure</td>
</tr>
<tr>
<td>End of September 1990</td>
<td></td>
<td>Backfilling of the test drifts without gap, start of heating</td>
</tr>
<tr>
<td>End of September 1990 to begin of February 1999</td>
<td>100 months</td>
<td>Heating period</td>
</tr>
<tr>
<td>Begin of February 1999 to begin of September 2000</td>
<td>19 months</td>
<td>Cooling down phase</td>
</tr>
<tr>
<td>September 2000</td>
<td></td>
<td>Backfill excavation drift B</td>
</tr>
<tr>
<td>September 2000 to October 2002</td>
<td>25 months</td>
<td>Observation time until end of calculation</td>
</tr>
</tbody>
</table>

### 4.2.3 Constitutive laws

**Rock salt**

Based on Olivella, 2000 the elastic parameters are

Table 4.2: Elastic parameters of rock salt

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Magnitude</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>2187</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Young’s modulus</td>
<td>25000</td>
<td>MPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.27</td>
<td></td>
</tr>
</tbody>
</table>
The creep behaviour is described by the relationship

\[ \dot{\varepsilon}_{cr} = A_{cr} \cdot \exp\left(-\frac{Q}{R \cdot T}\right) \cdot \left(\frac{q}{q_{ref}}\right)^n \]  

(4.1)

\[ \dot{\varepsilon}_{cr} = \text{creep strain rate} \]
\[ A_{cr} = \text{creep coefficient} = 0.18 \text{ d}^{-1} \]
\[ Q = \text{activation energy} = 54000 \text{ J/mol} \]
\[ R = \text{gas constant} = 8.314 \text{ J/(mol·K)} \]
\[ T = \text{temperature [Kelvin]} \]
\[ q = \text{deviatoric (equivalent) stress [MPa]} \]
\[ q_{ref} = \text{reference stress} = 1 \text{ MPa} \]
\[ n = \text{stress exponent} = 5 \]

The linear thermal expansion, thermal conductivity and volumetric heat capacity are considered in dependence on temperature (in degrees centigrade) in reference to Olivella, 2000 as follows:

Table 4.3: Thermal parameters of rock salt

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Magnitude</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear thermal expansion</td>
<td>4.2·10^{-6}</td>
<td>1/°C</td>
</tr>
<tr>
<td>Thermal conductivity</td>
<td>5.734 – 1.838·10^{-2} T + 2.86·10^{-5} T^2 – 1.51·10^{-8} T^3</td>
<td>W/(°C · m)</td>
</tr>
<tr>
<td>Volumetric heat capacity</td>
<td>1.8705·10^0 + 387.72 T</td>
<td>J/(°C · m²)</td>
</tr>
</tbody>
</table>

Backfill

The programme FLAC provides a crushed-salt constitutive model to simulate volumetric and deviatoric creep compaction behaviour. In the model implementation it is assumed that the creep compaction is irreversible (the density can only increase and cannot decrease) and limited by reaching the rock salt density. The volumetric creep compaction rate is formulated by the expression

\[ \dot{\varepsilon}_v = \frac{1}{\rho} \cdot B_0 \cdot \left[1 - \exp(-B_1 \cdot p)\right] \cdot \exp(B_2 \cdot \rho) \]  

(4.2)

\[ \rho = \text{density [kg/m}^3]\]
\[ p = \text{mean stress [MPa]} \]
\[ B_0, B_1 \text{ and } B_2 = \text{parameters to be determined in experiments} \]

The conversion to the mean stress \( p \) leads to

\[ p = \frac{1}{B_1} \cdot \frac{\ln\left[1 - \frac{\dot{\varepsilon}_v \cdot \rho}{B_0 \cdot \exp(B_2 \cdot \rho)}\right]}{\dot{\varepsilon}_v} \]  

(4.3)
and using

\[ \rho = \frac{\rho_0(1 - \phi)}{1 - \phi_0} \]  

(4.4)

\( \phi \) = porosity
\( \rho_0 \) = initial density = 1400 kg/m³
\( \phi_0 \) = initial porosity = 0.35

the following dependence on \( \phi \) can be found

\[
 p = -\frac{1}{B_1} \ln \left( 1 - \frac{\dot{\varepsilon}_d \cdot \rho_0 \cdot (1 - \phi)}{(1 - \phi_0) \cdot B_0 \cdot \exp \left( \frac{B_2 \cdot \rho_0 \cdot (1 - \phi)}{1 - \phi_0} \right) } \right) 
\]  

(4.5)

In the laboratory course (see Chapter 2), several laboratory tests on backfill from the TSDE-field were performed under different temperatures (30 °C and 70 °C). The backfill resistance \( p \) valid for in-situ relevant compaction rates in a range of \( 10^{-10} \text{s}^{-1} \) could be found by extrapolation using assumptions based on relaxation and creep phases.

The dependence of backfill pressure on the porosity (equation 4.5) can be approximated by the parameters

\[
 B_0 = -1 \cdot 10^{8} \text{kg/(m}^3 \cdot \text{s}) \\
 B_1 = -0.5 \text{ MPa}^{-1} \\
 B_2 = -22 \cdot 10^{-3} \text{m}^3/\text{kg} 
\]

The relation between backfill pressure and porosity can be described with other formulations as well. For instance the constitutive model formulation proposed by Hein and modified by Korthaus is appropriated with similar parameters found in the BAMBUS project (Bechthold et al., 1999).

The elastic bulk modulus was 15 MPa and the elastic shear modulus 5 MPa.

The deviatoric creep rate is defined by

\[
 \dot{\varepsilon}_d = D \cdot \left( \frac{q \cdot \rho_{ns}}{q_{ref} \cdot \rho} \right)^n \cdot \exp \left( -\frac{Q}{R \cdot T} \right) 
\]  

(4.6)

\( \dot{\varepsilon}_d \) = creep strain rate
\( D \) = creep coefficient = 0.18 d¹
\( \rho_{ns} \) = density of rock salt = 2187 kg/m³
\( \rho \) = density [kg/m³]
\( Q \) = activation energy = 54000 J/mol
\( R \) = gas constant = 8.314 J/(mol·K)
\( T \) = temperature [Kelvin]
\( q \) = deviatoric stress [MPa]
\( q_{ref} \) = reference stress = 1 MPa
\( n \) = stress exponent = 5
The deviatoric creep rate corresponds to the rock salt creep with a deviation only in the stress magnitude.

The linear thermal expansion and the volumetric heat capacity of the backfill are defined in an equivalent way as for rock salt. The thermal conductivity strongly depends on the backfill porosity \( \phi \). Regarding the proposal of G.3S the following expression was taken into account:

\[
\lambda_{cs} = -270\phi^4 + 370\phi^3 - 136\phi^2 + 1.5\phi + 5 \quad [W/(m\cdot K)]
\]  

\[ (4.7) \]

Heaters

The behaviour of the two heaters is simulated by a linear elastic constitutive law with

Table 4.4: Elastic parameters of the heaters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Magnitude</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>7700</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Young’s modulus</td>
<td>210000</td>
<td>MPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.28</td>
<td></td>
</tr>
</tbody>
</table>

4.3 Results and comparison to in-situ measurements

The results of the model calculations are summarized in the Figures 4.3 to 4.8. The selection was done for nodal points with available in-situ measurements.

The temperature course in backfill is visible in Figure 4.3. The heater surface temperature as a function of time is applied by a boundary condition. Case studies at the beginning of the investigations with an interior heat source or a surface flux led to unrealistic high backfill temperatures.

![Figure 4.3: Temperature development in backfill](image)

Institut für Gebirgsmechanik GmbH, Friederikenstraße 60, 04279 Leipzig, Tel.: 0341/336000 Fax: 0341/33600308
Figure 4.4: Temperature development in rock salt

Heating started at the end of September 1990 and ended at the beginning of February 1999. The best agreement with the measurements was reached at floor level (deviation at end of heating 7%) and 1.1 m above floor (deviation at end of heating 6%). The over-estimation at the roof is relatively high with 18%. One can summarize that despite the correct boundary condition at heater surface the disadvantages of the two-dimensional model are still visible. On the other hand they are acceptable regarding the investigation aim outlined above.

These remark is valid for Figure 4.4 as well. The temperature over-estimation at end of heating in the surrounding rock salt is between 15% in a depth of 0.3 m below floor and 22% in a distance of 2 m above roof.

Figure 4.5: Temperature in Kelvin after heater switch off in February 1999
Figure 4.6: Temperature in Kelvin in September 2002

The Figures 4.5 and 4.6 give a large-sized overview of the calculated temperatures in Kelvin after heater switch off in February 1999 and for comparison in September 2002.

Figure 4.7: Drift closure in cross section A-A

Figure 4.7 depicts the comparison between calculation and measurements for vertical and horizontal drift closure in drift B. The calculated closure at the end of the heating was 13% higher than the measured one, which was to be expected due to the over-estimation of the surrounding rock temperatures. The backfill excavation was done in September 2000 and led to a significant increase of the drift closure rate.
Figure 4.8: Backfill pressure and porosity

The comparison of backfill pressure (mean value of stresses) and backfill porosity at the end of heating can be seen in Figure 4.8. The backfill pressure at roof is over-estimated again by about 25%. The under-estimation of porosity is negligible. The porosity was calculated as a mean value regarding the whole drift cross-cut. The start value at the beginning of heating was 35%.

Figure 4.9: Stresses calculated and measured by hydraulic fracturing in the pillar
Figure 4.10: Maximal and minimal stresses calculated and measured by hydraulic fracturing between drift B and northern access drift

In Figures 4.9 and 4.10 the calculated principal stresses for two discrete locations from the begin of heating until October 2002 are presented. The locations are measurement points, at which in repeated campaigns hydrofrac stress measurements took place. The determined minimal stresses are, considering the general known scattering range of this method, in a good agreement to the calculated magnitudes. Generally, a consistent trend with time was found. It can be concluded that in the TSDE-field and the surrounding cooler rock salt a stress interchange takes place. Figure 4.9 shows that on basis of the pillar cooling at first the stresses decrease and later, after backfill excavation in drift B, a built up is visible. The stress increase is continued until the end of observation time due to a stress redistribution from the surrounding rock salt. The stress decrease around the northern access drift, as can be seen in Figure 4.10, confirms this effect.

Figure 4.11: Minimal principal stress in Pa after heater switch of in February 1999
4.4 Conclusions

With the used constitutive laws and parameters the measured thermal and mechanical values could be confirmed with more or less accuracy. The known disadvantages of the two-dimensional model were discussed. As main result, the rock stress interactions between the formerly heated pillar and the cooler surrounding rock found by stress measurements could be explained.

The Figures 4.11 and 4.12 show the significant distinction in minimal principal stress between February 1999 after heater switch off and September 2002. The discussed large-sized stress redistribution can be seen clearly.

5 SUMMARY

The dismantling of the TSDE experiment made it possible to study the complex behaviour of backfill and surrounding rock in a repository for heat-generating waste in rock salt. Studies were done both in situ in the dismantled drift and in laboratory. These studies helped to substantially improve the understanding of significant processes in a repository in rock salt and to extend the basis for optimising the repository design and construction and for predicting the long-term performance of the most important barriers in the repository.

One of the main objectives included further clarification of backfill properties. Re-entry into the test drift provided the opportunity to obtain samples that had been exposed to simulated repository conditions over nearly one decade. Further examination of these samples illustrated the mechanisms of compaction at higher temperatures. It could be shown that the models...
applied for mechanical compaction behaviour of crushed salt were reasonable, within the range of application.

The mechanisms of deformation and the constitutive models applied thereto have been well established for rock salt, in general. Elevated temperatures expected in repositories for high-level radioactive wastes will give rise to accelerated creep processes. In BAMBUS-II measurements between the TSDE test field and the surrounding rock salt quantified these phenomena at a repository scale. At the beginning of the project, it was postulated that the long-term heating cycle may influence strength-deformation response of the rock salt. Suitable samples were obtained and tested at room temperature and at 70°C. The strength investigations show similar results as formerly tested rock salt specimens which had not experienced a heating period.

The modelling efforts led to development of 2D-models of the TSDE in situ test. The overall comparisons included temperature profiles, closure magnitudes, backfill pressure and porosity. With the used constitutive laws and parameters the measured thermal and mechanical values could be confirmed with more or less accuracy. The known disadvantages of the two-dimensional model were discussed. As main result, the rock stress interactions between the formerly heated pillar and the cooler surrounding rock found by stress measurements could be explained.
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